INFORMATION HANDOUT

For Contract No. 10-0S1104
At Road: 10-SJ-4,5-Var
IN SAN JOAQUIN COUNTY IN STOCKTON AT VARIOUS LOCATIONS

Identified by
Project ID 1000000229

RAILROAD RELATIONS

Railroad Relations and Insurance Requirements

MATERIALS INFORMATION

Foundation Report, Dated December 20, 2012

Geotechnical Design Report, Dated May 4, 2012

Installation Details For Battery Backup System (BBS Cabinet Mounting Details and Wiring Details)

Crash Cushion Absorb 350 (Test Level 2)

RAILROAD RELATIONS AND INSURANCE REQUIREMENTS

EXHIBIT "C"

CONTRACTOR REQUIREMENTS

1.01 General:

- **1.00.00 The Contract must not begin work on railroad property until June 1, 2014.**

- **1.01.01** The Contractor must cooperate with **BNSF RAILWAY COMPANY**, hereinafter referred to as "**Railway**" and **CENTRAL CALIFORNIA TRACTION COMPANY**, hereinafter referred to as "**CCT**," where work is over or under on or adjacent to Railway property and/or right-of-way, hereafter referred to as "Railway Property", during the construction of CA SR 4 Overpass, Stockton, CA, DOT # 929071S, CPUC # 002-1122.95-A, LS 7200, MP 1122.95, Stockton Subdivision**.**

  CCT is a tenant of Railway and operates separately from Railway and owns their tracks within Railway property.

  The term "railways" shall mean Railway and CCT collectively.

- **1.01.02** The Contractor must execute and deliver to the Railway duplicate copies of the Exhibit "C-1" Agreement, in the form attached hereto, obligating the Contractor to provide and maintain in full force and effect the insurance called for under Section 3 of said Exhibit "C-1". Questions regarding procurement of the Railroad Protective Liability Insurance should be directed to Rosa Martinez at Marsh, USA, 214-303-8519.

  The Contractor must also make separate arrangements with CCT for rights of entry and railroad protective liability insurance.

- **1.01.03** The Contractor must plan, schedule and conduct all work activities so as not to interfere with the movement of <u>any</u> trains on Railway Property.

- **1.01.04** The Contractor's right to enter Railway's Property is subject to the absolute right of Railway **and CCT** to cause the Contractor's work on Railway's Property to cease if, in the opinion of Railway, Contractor's activities create a hazard to Railway's Property, employees, and/or operations. Railway will have the right to stop construction work on the Project if any of the following events take place: (i) Contractor (or any of its subcontractors) performs the Project work in a manner contrary to the plans and specifications approved by Railway; (ii) Contractor (or any of its subcontractors), in Railway's opinion, prosecutes the Project work in a manner which is hazardous to Railway property, facilities or the safe and expeditious movement of railroad traffic; (iii) the insurance described in the attached <u>Exhibit C-1</u> is canceled during the course of the Project; or (iv) Contractor fails to pay Railway for the Temporary Construction License or the Easement. The work stoppage will continue until all

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necessary actions are taken by Contractor or its subcontractor to rectify the situation to the satisfaction of Railway's Division Engineer or until additional insurance has been delivered to and accepted by Railway. In the event of a breach of (i) this Agreement, (ii) the Temporary Construction License, or (iii) the Easement, Railway may immediately terminate the Temporary Construction License or the Easement. Any such work stoppage under this provision will not give rise to any liability on the part of Railway. Railway's right to stop the work is in addition to any other rights Railway may have including, but not limited to, actions or suits for damages or lost profits. In the event that Railway desires to stop construction work on the Project, Railway agrees to immediately notify the following individual in writing:

Mohammed Waziri
District Resident Engineer
1110 West Kettleman Ln, Suite 35
Lodi, CA 95240
Phone: (209) 470-8790
Fax: (209) 333-6944

- **1.01.05** The Contractor is responsible for determining and complying with all Federal, State and Local Governmental laws and regulations, including, but not limited to environmental laws and regulations (including but not limited to the Resource Conservation and Recovery Act, as amended; the Clean Water Act, the Oil Pollution Act, the Hazardous Materials Transportation Act, CERCLA), and health and safety laws and regulations. The Contractor hereby indemnifies, defends and holds harmless Railway for, from and against all fines or penalties imposed or assessed by Federal, State and Local Governmental Agencies against the Railway which arise out of Contractor's work under this Agreement.

- **1.01.06** The Contractor must notify **State at 209-470-8790** and Railway's Manager Public Projects, **John R. Stilley,** telephone number 909-386-4474 at least thirty (30) calendar days before commencing any work on Railway Property. Contractor's notification to Railway must refer to Railway's file ***BF10004393.***

- **1.01.07** For any bridge demolition and/or falsework above any tracks or any excavations located with any part of the excavations located within, whichever is greater, twenty-five (25) feet of the nearest track or intersecting a slope from the plane of the top of rail on a 2 horizontal to 1 vertical slope beginning at eleven (11) feet from centerline of the nearest track, both measured perpendicular to center line of track, the Contractor must furnish the Railway five sets of working drawings showing details of construction affecting Railway Property and tracks. The working drawing must include the proposed method of installation and removal of falsework, shoring or cribbing, not included in the contract plans and two sets of structural calculations of any falsework, shoring or cribbing. For all excavation and shoring submittal plans, the current "BNSF-UPRR Guidelines for Temporary Shoring" must be used for determining the design loading conditions to be used in shoring design, and all calculations and submittals must be in accordance with the current "BNSF-UPRR Guidelines for Temporary Shoring". All submittal drawings and calculations must be stamped by a

registered professional engineer licensed to practice in the state the project is located. All calculations must take into consideration railway surcharge loading and must be designed to meet American Railway Engineering and Maintenance-of-Way Association (previously known as American Railway Engineering Association) Coopers E-80 live loading standard. All drawings and calculations must be stamped by a registered professional engineer licensed to practice in the state the project is located. The Contractor must not begin work until notified by the Railway that plans have been approved. The Contractor will be required to use lifting devices such as, cranes and/or winches to place or to remove any falsework over Railway's tracks. In no case will the Contractor be relieved of responsibility for results obtained by the implementation of said approved plans.

- **1.01.08** Subject to the movement of **railways** trains, railways will cooperate with the Contractor such that the work may be handled and performed in an efficient manner. **In the 4th quarter of each calendar year, work will not be permitted in Railway's right of way.** The Contractor will have no claim whatsoever for any type of damages or for extra or additional compensation in the event his work is delayed by the Railway.

1.02 Contractor Safety Orientation

- **1.02.01 No employee of the Contractor, its subcontractors, agents or invitees may enter Railway Property without first having completed Railway's Engineering Contractor Safety Orientation, found on the web site www.contractororientation.com. The Contractor must ensure that each of its employees, subcontractors, agents or invitees completes Railway's Engineering Contractor Safety Orientation through internet sessions before any work is performed on the Project. Additionally, the Contractor must ensure that each and every one of its employees, subcontractors, agents or invitees possesses a card certifying completion of the Railway Contractor Safety Orientation before entering Railway Property. The Contractor is responsible for the cost of the Railway Contractor Safety Orientation. The Contractor must renew the Railway Contractor Safety Orientation annually. Further clarification can be found on the web site or from the Railway's Representative.**

1.03 Railway Requirements

- **1.03.01** The Contractor must take protective measures as are necessary to keep railway facilities, including track ballast, free of sand, debris, and other foreign objects and materials resulting from his operations. Any damage to railway facilities resulting from Contractor's operations will be repaired or replaced by Railway **and / or CCT** and the cost of such repairs or replacement must be paid for by the **Contractor**.

- **1.03.02** The Contractor must notify the Railway's Division Engineer at (909)386-4504 and CCT's General Manager, Dave Buccolo at (209) 471-6251 and provide blasting plans to **for review and approval** seven (7) calendar days prior to conducting any blasting operations adjacent to or on Railway's Property.

- **1.03.03** The Contractor must abide by the following temporary clearances during construction:

    - 15’-0” Horizontally from centerline of nearest track
    - 21’-6” Vertically above top of rail
    - 27'-0" Vertically above top of rail for electric wires carrying less than 750 volts
    - 28'-0" Vertically above top of rail for electric wires carrying 750 volts to 15,000 volts
    - 30'-0" Vertically above top of rail for electric wires carrying 15,000 volts to 20,000 volts
    - 34'-0" Vertically above top of rail for electric wires carrying more than 20,000 volts

- **1.03.04** Upon completion of construction, the following clearances shall be maintained:

    - 25’ Horizontally from centerline of nearest track
    - 23’ 6” Vertically above top of rail

- **1.03.05** Any infringement within State statutory clearances due to the Contractor's operations must be submitted to the **railways** and to the **(STATE)** and must not be undertaken until approved in writing by the **railways**, and until the **(STATE)** has obtained any necessary authorization from the State Regulatory Authority for the infringement. No extra compensation will be allowed in the event the Contractor's work is delayed pending railway approval, and/or the State Regulatory Authority's approval.

- **1.03.06** In the case of impaired vertical clearance above top of rail, Railway will have the option of installing tell-tales or other protective devices Railway deems necessary for protection of Railway operations. The cost of tell-tales or protective devices will be borne by the Agency.

- **1.03.07** The details of construction affecting the Railway's Property and tracks not included in the contract plans must be submitted to the Railway by **(STATE)** for approval before work is undertaken and this work must not be undertaken until approved by the Railway.

- **1.03.08** At other than public road crossings, the Contractor must not move any equipment or materials across Railway's tracks until permission has been obtained from the Railway. The Contractor must obtain a "Temporary Construction Crossing Agreement" from the Railway prior to moving his equipment or materials across the Railways tracks. The temporary crossing must be gated and locked at all times when not required for use by the Contractor. The temporary crossing for use of the Contractor will be constructed and, at the completion of the project, removed at the expense of the Contractor.

- **1.03.09** Discharge, release or spill on the Railway Property of any hazardous substances, oil, petroleum, constituents, pollutants, contaminants, or any hazardous waste is prohibited

and Contractor must immediately notify the Railway's Resource Operations Center at 1(800) 832-5452, of any discharge, release or spills in excess of a reportable quantity. Contractor must not allow Railway Property to become a treatment, storage or transfer facility as those terms are defined in the Resource Conservation and Recovery Act or any state analogue.

- **1.03.10** The Contractor upon completion of the work covered by this contract, must promptly remove from the Railway's Property all of Contractor's tools, equipment, implements and other materials, whether brought upon said property by said Contractor or any Subcontractor, employee or agent of Contractor or of any Subcontractor, and must cause Railway's Property to be left in a condition acceptable to the Railway's representative.

1.04 Contractor Roadway Worker on Track Safety Program and Safety Action Plan:

- **1.04.01** Each Contractor that will perform work within 25 feet of the centerline of a track must develop and implement a Roadway Worker Protection/On Track Safety Program and work with Railway Project Representative to develop an on track safety strategy as described in the guidelines listed in the on track safety portion of the Safety Orientation. This Program must provide Roadway Worker protection/on track training for all employees of the Contractor, its subcontractors, agents or invitees. This training is reinforced at the job site through job safety briefings. Additionally, each Contractor must develop and implement the Safety Action Plan, as provided for on the web site **www.contractororientation.com**, which will be made available to Railway prior to commencement of any work on Railway Property. During the performance of work, the Contractor must audit its work activities. The Contractor must designate an on-site Project Supervisor who will serve as the contact person for the Railway **and CCT** who will maintain a copy of the Safety Action Plan, safety audits, and Material Safety Datasheets (MSDS), at the job site.

1.05 Railway Flagger Services:

- **1.05.01** The Contractor must give Railway's **Roadmaster Wayne Morris (209) 460-6340** a minimum of thirty (30) calendar days advance notice when flagging services will be required so that the Roadmaster can make appropriate arrangements (i.e., bulletin the flagger's position). If flagging services are scheduled in advance by the Contractor and it is subsequently determined by the parties hereto that such services are no longer necessary, the Contractor must give the Roadmaster five (5) working days advance notice so that appropriate arrangements can be made to abolish the position pursuant to union requirements. **The Contractor must also do the same for CCT with the same time requirements to arrange for flagger services by contacting its General Manager, Dave Buccolo, at (209) 466-6927.**

- **1.05.02** Unless determined otherwise by **CCT and** Railway's Project Representative, Railway flagger will be required and furnished when Contractor's work activities are located over, under and/or within twenty-five (25) feet measured horizontally from centerline of the nearest track and when cranes or similar equipment positioned beyond 25-feet from the

track centerline could foul the track in the event of tip over or other catastrophic occurrence, but not limited thereto for the following conditions:

- **1.05.02a** When, upon inspection by Railway's and **CCT Representatives**, other conditions warrant.

- **1.05.02b** When any excavation is performed below the bottom of tie elevation, if, in the opinion of Railway's representative, track or other Railway facilities may be subject to movement or settlement.

- **1.05.02c** When work in any way interferes with the safe operation of trains at timetable speeds.

- **1.05.02d** When any hazard is presented to Railway track, communications, signal, electrical, or other facilities either due to persons, material, equipment or blasting in the vicinity.

- **1.05.02e** Special permission must be obtained from the Railway before moving heavy or cumbersome objects or equipment which might result in making the track impassable.

- **1.05.03** Flagging services will be performed by qualified Railway flaggers.

- **1.05.03a** Flagging crew generally consists of one employee. However, additional personnel may be required to protect Railway Property and operations, if deemed necessary by the Railways Representative.

- **1.05.03b** Each time a flagger is called, the minimum period for billing will be the eight (8) hour basic day.

- **1.05.03c** The cost of flagger services provided by the Railway will be borne by **(Agency)**. The estimated cost for one (1) flagger is approximately between $800.00-$1,600.00 for an eight (8) hour basic day with time and one-half or double time for overtime, rest days and holidays. The estimated cost for each flagger includes vacation allowance, paid holidays, Railway and unemployment insurance, public liability and property damage insurance, health and welfare benefits, vehicle, transportation, meals, lodging, radio, equipment, supervision and other costs incidental to performing flagging services. Negotiations for Railway labor or collective bargaining agreements and rate changes authorized by appropriate Federal authorities may increase actual or estimated flagging rates. **THE FLAGGING RATE IN EFFECT AT THE TIME OF PERFORMANCE BY THE CONTRACTOR HEREUNDER WILL BE USED TO CALCULATE THE ACTUAL COSTS OF FLAGGING PURSUANT TO THIS PARAGRAPH.**

- **1.05.03d** The average train traffic on this route is 11 freight trains per 24-hour period at a timetable speed 55 MPH and 8 passenger trains at a timetable speed of 79 MPH.

1.06 Contractor General Safety Requirements

- **1.06.01** Work in the proximity of railway track(s) is potentially hazardous where movement of trains and equipment can occur at any time and in any direction. All work performed by contractors within 25 feet of any track must be in compliance with FRA Roadway Worker Protection Regulations.

- **1.06.02** Before beginning any task on Railway Property, a thorough job safety briefing must be conducted with all personnel involved with the task and repeated when the personnel or task changes. If the task is within 25 feet of any track, the job briefing must include the Railway's flagger, as applicable, and include the procedures the Contractor will use to protect its employees, subcontractors, agents or invitees from moving any equipment adjacent to or across any Railway track(s).

- **1.06.03** Workers must not work within 25 feet of the centerline of any track without an on track safety strategy approved by the Railway's Project Representative. When authority is provided, every contractor employee must know: (1) who the railways flaggers are, and how to contact the flagger, (2) limits of the authority, (3) the method of communication to stop and resume work, and (4) location of the designated places of safety. Persons or equipment entering flag/work limits that were not previously job briefed, must notify the flagger immediately, and be given a job briefing when working within 25 feet of the center line of track.

- **1.06.04** When Contractor employees are required to work on the Railway Property after normal working hours or on weekends, the Railway's representative in charge of the project must be notified. A minimum of two employees must be present at all times.

- **1.06.05** Any employees, agents or invitees of Contractor or its subcontractors under suspicion of being under the influence of drugs or alcohol, or in the possession of same, will be removed from the Railway's Property and subsequently released to the custody of a representative of Contractor management. Future access to the Railway's Property by that employee will be denied.

- **1.06.06** Any damage to Railway Property, or any hazard noticed on passing trains must be reported immediately to the Railway's representative in charge of the project. Any vehicle or machine which may come in contact with track, signal equipment, or structure (bridge) and could result in a train derailment must be reported immediately to the Railway representative in charge of the project and to the Railway's Resource Operations Center at 1(800) 832-5452. Local emergency numbers are to be obtained from the Railway representative in charge of the project prior to the start of any work and must be posted at the job site.

- **1.06.07** For safety reasons, all persons are prohibited from having pocket knives, firearms or other deadly weapons in their possession while working on Railway's Property.

- **1.06.08** All personnel protective equipment (PPE) used on Railway Property must meet applicable OSHA and ANSI specifications. Current Railway personnel protective equipment

requirements are listed on the web site, **www.contractororientation.com,** however, a partial list of the requirements include: a) safety glasses with permanently affixed side shields (no yellow lenses); b) hard hats; c) safety shoe with: hardened toes, above-the-ankle lace-up and a defined heel; and d) high visibility retro-reflective work wear. The Railway's representative in charge of the project is to be contacted regarding local specifications for meeting requirements relating to hi-visibility work wear. Hearing protection, fall protection, gloves, and respirators must be worn as required by State and Federal regulations. **(NOTE – Should there be a discrepancy between the information contained on the web site and the information in this paragraph, the web site will govern.)**

- **1.06.09 THE CONTRACTOR MUST NOT PILE OR STORE ANY MATERIALS, MACHINERY OR EQUIPMENT CLOSER THAN 25'-0" TO THE CENTER LINE OF THE NEAREST RAILWAY TRACK. MATERIALS, MACHINERY OR EQUIPMENT MUST NOT BE STORED OR LEFT WITHIN 250 FEET OF ANY HIGHWAY/RAIL AT-GRADE CROSSINGS OR TEMPORARY CONSTRUCTION CROSSING, WHERE STORAGE OF THE SAME WILL OBSTRUCT THE VIEW OF A TRAIN APPROACHING THE CROSSING. PRIOR TO BEGINNING WORK, THE CONTRACTOR MUST ESTABLISH A STORAGE AREA WITH CONCURRENCE OF THE RAILWAY'S REPRESENTATIVE.**

- **1.06.10** Machines or vehicles must not be left unattended with the engine running. Parked machines or equipment must be in gear with brakes set and if equipped with blade, pan or bucket, they must be lowered to the ground. All machinery and equipment left unattended on Railway's Property must be left inoperable and secured against movement. (See internet Engineering Contractor Safety Orientation program for more detailed specifications)

- **1.06.11** Workers must not create and leave any conditions at the work site that would interfere with water drainage. Any work performed over water must meet all Federal, State and Local regulations.

- **1.06.12** All power line wires must be considered dangerous and of high voltage unless informed to the contrary by proper authority. For all power lines the minimum clearance between the lines and any part of the equipment or load must be; 200 KV or below - 15 feet; 200 to 350 KV - 20 feet; 350 to 500 KV - 25 feet; 500 to 750 KV - 35 feet; and 750 to 1000 KV - 45 feet. If capacity of the line is not known, a minimum clearance of 45 feet must be maintained. A person must be designated to observe clearance of the equipment and give a timely warning for all operations where it is difficult for an operator to maintain the desired clearance by visual means.

1.07 Excavation:

- **1.07.01** Before excavating, the Contractor must determine whether any underground pipe lines, electric wires, or cables, including fiber optic cable systems are present and located within the Project work area. The Contractor must determine whether excavation on Railway's Property could cause damage to buried cables resulting in delay to Railway traffic

and disruption of service to users. Delays and disruptions to service may cause business interruptions involving loss of revenue and profits. Before commencing excavation, the Contractor must contact **BNSF's Field Engineering Representative (Jason Sanchez, 909-386-4075).** All underground and overhead wires will be considered HIGH VOLTAGE and dangerous until verified with the company having ownership of the line. **It is the Contractor's responsibility to notify any other companies that have underground utilities in the area and arrange for the location of all underground utilities before excavating.**

- **1.07.02** The Contractor must cease all work and notify the Railway immediately before continuing excavation in the area if obstructions are encountered which do not appear on drawings. If the obstruction is a utility and the owner of the utility can be identified, then the Contractor must also notify the owner immediately. If there is any doubt about the location of underground cables or lines of any kind, no work must be performed until the exact location has been determined. There will be no exceptions to these instructions.

- **1.07.03** All excavations must be conducted in compliance with applicable OSHA regulations and, regardless of depth, must be shored where there is any danger to tracks, structures or personnel.

- **1.07.04** Any excavations, holes or trenches on the Railway's Property must be covered, guarded and/or protected when not being worked on. When leaving work site areas at night and over weekends, the areas must be secured and left in a condition that will ensure that Railway employees and other personnel who may be working or passing through the area are protected from all hazards. All excavations must be back filled as soon as possible.

1.08 Hazardous Waste, Substances and Material Reporting:

- **1.08.01** If Contractor discovers any hazardous waste, hazardous substance, petroleum or other deleterious material, including but not limited to any non-containerized commodity or material, on or adjacent to Railway's Property, in or near any surface water, swamp, wetlands or waterways, while performing any work under this Agreement, Contractor must immediately: (a) notify the Railway's Resource Operations Center at 1(800) 832-5452, of such discovery: (b) take safeguards necessary to protect its employees, subcontractors, agents and/or third parties: and (c) exercise due care with respect to the release, including the taking of any appropriate measure to minimize the impact of such release.

1.09 Personal Injury Reporting

- **1.09.01** The Railway is required to report certain injuries as a part of compliance with Federal Railroad Administration (FRA) reporting requirements. Any personal injury sustained by an employee of the Contractor, subcontractor or Contractor's invitees while on the Railway's Property must be reported immediately (by phone mail if unable to contact in person) to the Railway's representative in charge of the project. The Non-Employee Personal

Injury Data Collection Form contained herein is to be completed and sent by Fax to the Railway at 1(817) 352-7595 and to the Railway's Project Representative no later than the close of shift on the date of the injury.

NON-EMPLOYEE PERSONAL INJURY DATA COLLECTION

(If injuries are in connection with rail equipment accident/incident, highway rail grade crossing accident or automobile accident, ensure that appropriate information is obtained, forms completed and that data entry personnel are aware that injuries relate to that specific event.)

Injured Person Type:

- ☐ Passenger on train (C)
- ☐ Non-employee (N)
  (i.e., emp of another railroad, or, non-BNSF emp involved in vehicle accident, including company vehicles)
- ☐ Contractor/safety sensitive (F)
- ☐ Contractor/non-safety sensitive (G)
- ☐ Volunteer/safety sensitive (H)
- ☐ Volunteer/other non-safety sensitive (I)
- ☐ Non-trespasser (D) - to include highway users involved in highway rail grade crossing accidents who did not go around or through gates
- ☐ Trespasser (E) - to include highway users involved in highway rail grade crossing accidents who went around or through gates
- ☐ Non-trespasser (J) - Off railroad property

If train involved, Train ID: _____

Transmit attached information to Accident/Incident Reporting Center by:
Fax 1-817-352-7595 or by Phone 1-800-697-6736 **or email to:** Accident-Reporting.Center@BNSF.com

Officer Providing Information:

_____ _____ _____
(Name) (Employee No.) (Phone #)

REPORT PREPARED TO COMPLY WITH FEDERAL ACCIDENT REPORTING REQUIREMENTS AND PROTECTED FROM DISCLOSURE PURSUANT TO 49 U.S.C. 20903 AND 83 U.S.C. 490

NON-EMPLOYEE PERSONAL INJURY DATA COLLECTION

INFORMATION REQUIRED TO BE COLLECTED PURSUANT TO FEDERAL REGULATION. IT SHOULD BE USED FOR COMPLIANCE WITH FEDERAL REGULATIONS ONLY AND IT IS NOT INTENDED TO PRESUME ACCEPTANCE OF RESPONSIBILITY OR LIABILITY.

1. Accident City/St: _____ 2. Date: _____ Time: _____

County: _____ 3. Temperature: _____ 4. Weather: _____
(if non BNSF location)

Mile Post / Line Segment: _____

5. Driver's License No (and state) or other ID: _____ **SSN (required)**: _____

6. Name (last, first, mi): _____

7. Address: _____ City: _____ St: _____ Zip: _____

8. Date of Birth: _____ and/or Age: _____ Gender: _____
(if available)

Phone Number: _____ Employer: _____

9. Injury: _____ 10. Body Part: _____
(i.e., Laceration, etc.) (i.e., Hand, etc.)

11. Description of Accident (To include location, action, result, etc.): _____

_____

_____

_____

12. Treatment:

- ☐ First Aid Only _____
- ☐ Required Medical Treatment _____
- ☐ Other Medical Treatment _____

13. Dr. Name: _____ Date: _____

14. Dr. Address:

Street: _____ City: _____ St: _____ Zip: _____

15. Hospital Name: _____

16. Hospital Address:

Street: _____ City: _____ St: _____ Zip: _____

17. Diagnosis: _____

REPORT PREPARED TO COMPLY WITH FEDERAL ACCIDENT REPORTING REQUIREMENTS AND PROTECTED FROM DISCLOSURE PURSUANT TO 49 U.S.C. 20903 AND 83 U.S.C. 490

EXHIBIT "C-1"

Agreement Between
BNSF RAILWAY COMPANY
and the
CONTRACTOR

BNSF RAILWAY COMPANY
Attention: Manager Public Projects

Railway File: BF10004495
Agency Project: _____

Gentlemen:

The undersigned (hereinafter called, the "Contractor"), has entered into a contract (the "Contract") dated _____, 2013, with **State of California, Department of Transportation (STATE)** for the performance of certain work in connection with the following project: Route 4 Extension Project. Performance of such work will necessarily require Contractor to enter **BNSF RAILWAY COMPANY** ("Railway") right of way and property ("Railway Property"). The Contract provides that no work will be commenced within Railway Property until the Contractor employed in connection with said work for STATE (i) executes and delivers to Railway an Agreement in the form hereof, and (ii) provides insurance of the coverage and limits specified in such Agreement and Section 3 herein. If this Agreement is executed by a party who is not the Owner, General Partner, President or Vice President of Contractor, Contractor must furnish evidence to Railway certifying that the signatory is empowered to execute this Agreement on behalf of Contractor.

Accordingly, in consideration of Railway granting permission to Contractor to enter upon Railway Property and as an inducement for such entry, Contractor, effective on the date of the Contract, has agreed and does hereby agree with Railway as follows:

Section 1. RELEASE OF LIABILITY AND INDEMNITY

Contractor hereby waives, releases, indemnifies, defends and holds harmless Railway for all judgments, awards, claims, demands, and expenses (including attorneys' fees), for injury or death to all persons, including Railway's and Contractor's officers and employees, and for loss and damage to property belonging to any person, arising in any manner from Contractor's or any of Contractor's subcontractors' acts or omissions or any work performed on or about Railway's property or right-of-way. **THE LIABILITY ASSUMED BY CONTRACTOR WILL NOT BE AFFECTED BY THE FACT, IF IT IS A FACT, THAT THE DESTRUCTION, DAMAGE, DEATH, OR INJURY WAS OCCASIONED BY OR CONTRIBUTED TO BY THE NEGLIGENCE OF RAILWAY, ITS AGENTS, SERVANTS, EMPLOYEES OR OTHERWISE, EXCEPT TO THE EXTENT THAT SUCH CLAIMS ARE PROXIMATELY CAUSED BY THE WILLFUL MISCONDUCT OR SOLE NEGLIGENCE OF RAILWAY.**

THE INDEMNIFICATION OBLIGATION ASSUMED BY CONTRACTOR INCLUDES ANY CLAIMS, SUITS OR JUDGMENTS BROUGHT AGAINST RAILWAY UNDER THE FEDERAL EMPLOYEE'S LIABILITY ACT, INCLUDING CLAIMS FOR STRICT LIABILITY UNDER THE SAFETY APPLIANCE ACT OR THE LOCOMOTIVE INSPECTION ACT, WHENEVER SO CLAIMED.

Contractor further agrees, at its expense, in the name and on behalf of Railway, that it will adjust and settle all claims made against Railway, and will, at Railway's discretion, appear and defend any suits or actions of law or in equity brought against Railway on any claim or cause of action arising or growing out of or in any manner connected with any liability assumed by Contractor under this Agreement for which Railway is liable or is alleged to be liable. Railway will give notice to Contractor, in writing, of the receipt or dependency of such claims and thereupon Contractor must proceed to adjust and handle to a conclusion such claims, and in the event of a suit being brought against Railway, Railway may forward summons and complaint or other process in connection therewith to Contractor, and Contractor, at Railway's discretion, must defend, adjust, or settle such suits and protect, indemnify, and save harmless Railway from and against all damages, judgments, decrees, attorney's fees, costs, and expenses growing out of or resulting from or incident to any such claims or suits.

In addition to any other provision of this Agreement, in the event that all or any portion of this Article shall be deemed to be inapplicable for any reason, including without limitation as a result of a decision of an applicable court, legislative enactment or regulatory order, the parties agree that this Article shall be interpreted as requiring Contractor to indemnify Railway to the fullest extent permitted by applicable law. **THROUGH THIS AGREEMENT THE PARTIES EXPRESSLY INTEND FOR CONTRACTOR TO INDEMNIFY RAILWAY FOR RAILWAY'S ACTS OF NEGLIGENCE.**

It is mutually understood and agreed that the assumption of liabilities and indemnification provided for in this Agreement survive any termination of this Agreement.

Section 2. TERM

This Agreement is effective from the date of the Contract until (i) the completion of the project set forth herein, and (ii) full and complete payment to Railway of any and all sums or other amounts owing and due hereunder.

Section 3. INSURANCE

Contractor shall, at its sole cost and expense, procure and maintain during the life of this Agreement the following insurance coverage:

A. Commercial General Liability insurance. This insurance shall contain broad form contractual liability with a combined single limit of a minimum of $2,000,000 each occurrence and an aggregate limit of at least $6,000,000 but in no event less than the amount otherwise carried by the Contractor. Coverage must be purchased on a post 2004 ISO occurrence form or equivalent and include coverage for, but not limit to the following:

- Bodily Injury and Property Damage
- Personal Injury and Advertising Injury
- Fire legal liability
- Products and completed operations

This policy shall also contain the following endorsements, which shall be indicated on the certificate of insurance:

- The definition of insured contract shall be amended to remove any exclusion or other limitation for any work being done within 50 feet of railroad property.
- Waver of subrogation in favor of and acceptable to Railway.

- Additional insured endorsement in favor of and acceptable to Railway.
- Separation of insureds.
- The policy shall be primary and non-contributing with respect to any insurance carried by Railway.

It is agreed that the workers' compensation and employers' liability related exclusions in the Commercial General Liability insurance policy(s) required herein are intended to apply to employees of the policy holder and shall not apply to ***Railway*** employees.

No other endorsements limiting coverage as respects obligations under this_Agreement may be included on the policy with regard to the work being performed under this agreement.

B. Business Automobile Insurance. This insurance shall contain a combined single limit of at least $1,000,000 per occurrence, and include coverage for, but not limited to the following:

- Bodily injury and property damage
- Any and all vehicles owned, used or hired

The policy shall also contain the following endorsements or language, which shall be indicated on the certificate of insurance:

- Waiver of subrogation in favor of and acceptable to Railway.
- Additional insured endorsement in favor of and acceptable to Railway.
- Separation of insureds.
- The policy shall be primary and non-contributing with respect to any insurance carried by Railway.

C. Workers Compensation and Employers Liability insurance including coverage for, but not limited to:

- Contractor's statutory liability under the worker's compensation laws of the state(s) in which the work is to be performed. If optional under State law, the insurance must cover all employees anyway.
- Employers' Liability (Part B) with limits of at least $500,000 each accident, $500,000 by disease policy limit, $500,000 by disease each employee.

This policy shall also contain the following endorsements or language, which shall be indicated on the certificate of insurance:

- Waiver of subrogation in favor of and acceptable to Railway.

D. Railroad Protective Liability insurance naming only the ***Railway*** as the Insured with coverage of at least $2,000,000 per occurrence and $6,000,000 in the aggregate. The policy Must be issued on a standard ISO form CG 00 35 12 04 and include the following:

- Endorsed to include the Pollution Exclusion Amendment
- Endorsed to include the Limited Seepage and Pollution Endorsement.
- Endorsed to remove any exclusion for punitive damages.
- No other endorsements restricting coverage may be added.
- The original policy must be provided to the ***Railway*** prior to performing any work or services under this Agreement

- Definition of "Physical Damage to Property" shall be endorsed to read: "means direct and accidental loss of or damage to all property owned by any named insured and all property in any named insured' care, custody, and control arising out of the acts or omissions of the contractor named on the Declarations.

In lieu of providing a Railroad Protective Liability Policy, Licensee may participate (if available) in Railway's Blanket Railroad Protective Liability Insurance Policy.

Other Requirements:

Where allowable by law, all policies (applying to coverage listed above) shall contain no exclusion for punitive damages.

Contractor agrees to waive its right of recovery against ***Railway*** for all claims and suits against ***Railway***. In addition, its insurers, through the terms of the policy or policy endorsement, waive their right of subrogation against ***Railway*** for all claims and suits. Contractor further waives its right of recovery, and its insurers also waive their right of subrogation against ***Railway*** for loss of its owned or leased property or property under Contractor's care, custody, or control.

Allocated Loss Expense shall be in addition to all policy limits for coverages referenced above.

Contractor is not allowed to self-insure without the prior written consent of ***Railway.*** If granted by ***Railway***, self-insured retention or other financial responsibility for claims shall be covered directly by Contractor in lieu of insurance. Any and all ***Railway*** liabilities that would otherwise, in accordance with the provisions of this ***Agreement***, be covered by Contractor's insurance will be covered as if Contractor elected not to include a deductible, self-insured retention or other financial responsibility for claims.

Prior to commencing the Work, Contractor shall furnish to ***Railway*** an acceptable certificate(s) of insurance from an authorized representative evidencing the required coverage(s), endorsements, and amendments. The certificate should be directed to the following addresses:

BNSF Railway Company
c/o CertFocus
P.O. Box 140528
Kansas City, MO 64114
Toll Free: 877-576-2378
Fax number: 817-840-7487
Email: BNSF@certfocus.com
www.certfocus.com

Contractor shall notify ***Railway*** in writing at least 30 days prior to any cancellation, non-renewal, substitution, or material alteration.

Any insurance policy must be written by a reputable insurance company acceptable to ***Railway*** or with a current Best's Guide Rating of A- and Class VII or better, and authorized to do business in the state(s) in which the service is to be provide.

If coverage is purchased on a "claims made" basis, Contractor hereby agrees to maintain coverage in force for a minimum of three years after expiration, cancellation or termination of this contract. Annually, Contractor agrees to provide evidence of such coverage as required hereunder.

Contractor represents that this ***Agreement*** has been thoroughly reviewed by Contractor's insurance agent(s)/broker(s), who have been instructed by Contractor to procure the insurance coverage required by this ***Agreement***.

Not more frequently than once every five years, ***Railway*** may reasonably modify the required insurance coverage to reflect then-current risk management practices in the railroad industry and underwriting practices in the insurance industry.

If any portion of the operation is to be subcontracted by Contractor, Contractor shall require that the subcontractor shall provide and maintain the insurance coverage(s) set forth herein, naming ***Railway*** as an additional insured, and shall require that the subcontractor shall release, defend, and indemnify ***Railway*** to the same extent and under the same terms and conditions as Contractor is required to release, defend, and indemnify ***Railway*** herein.

Failure to provide evidence as required by this section shall entitle, but not require, ***Railway*** to terminate this ***Agreement*** immediately. Acceptance of a certificate that does not comply with this section shall not operate as a waiver of Contractor's obligations hereunder.

The fact that insurance (including, without limitation, self-insurance) is obtained by Contractor shall not be deemed to release or diminish the liability of Contractor including, without limitation, liability under the indemnity provisions of this ***Agreement***. Damages recoverable by ***Railway*** shall not be limited by the amount of the required insurance coverage.

In the event of a claim or lawsuit involving ***Railway*** arising out of this agreement, Contractor will make available any required policy covering such claim or lawsuit.

These insurance provisions are intended to be a separate and distinct obligation on the part of the Contractor. Therefore, these provisions shall be enforceable and Contractor shall be bound thereby regardless of whether or not indemnity provisions are determined to be enforceable in the jurisdiction in which the work covered hereunder is performed.

For purposes of this section, ***Railway*** means "Burlington Northern Santa Fe LLC", "BNSF RAILWAY COMPANY" and the subsidiaries, successors, assigns and affiliates of each.

Section 4. EXHIBIT "C" CONTRACTOR REQUIREMENTS

The Contractor must observe and comply with all provisions, obligations, requirements and limitations contained in the Contract, and the Contractor Requirements set forth on Exhibit "C" attached to the Contract and this Agreement, , including, but not be limited to, payment of all costs incurred for any damages to Railway roadbed, tracks, and/or appurtenances thereto, resulting from use, occupancy, or presence of its employees, representatives, or agents or subcontractors on or about the construction site.

Section 5. TRAIN DELAY

Contractor is responsible for and hereby indemnifies and holds harmless Railway (including its affiliated railway companies, and its tenants) for, from and against all damages arising from any unscheduled delay to a freight or passenger train which affects Railway's ability to fully utilize its equipment and to meet customer service and contract obligations. Contractor will be billed, as further provided below, for the economic losses arising from loss of use of equipment, contractual loss of incentive pay and bonuses and contractual penalties resulting from train delays, whether caused by Contractor, or subcontractors, or by the Railway performing work under this Agreement. Railway agrees that it will not perform any act to unnecessarily cause train delay.

For loss of use of equipment, Contractor will be billed the current freight train hour rate per train as determined from Railway's records. Any disruption to train traffic may cause delays to multiple trains at the same time for the same period.

Additionally, the parties acknowledge that passenger, U.S. mail trains and certain other grain, intermodal, coal and freight trains operate under incentive/penalty contracts between Railway and its customer(s). Under these arrangements, if Railway does not meet its contract service commitments, Railway may suffer loss of performance or incentive pay and/or be subject to penalty payments. Contractor is responsible for any train performance and incentive penalties or other contractual economic losses actually incurred by Railway which are attributable to a train delay caused by Contractor or its subcontractors.

The contractual relationship between Railway and its customers is proprietary and confidential. In the event of a train delay covered by this Agreement, Railway will share information relevant to any train delay to the extent consistent with Railway confidentiality obligations. Damages for train delay are currently $382.20 per hour per incident. **THE RATE THEN IN EFFECT AT THE TIME OF PERFORMANCE BY THE CONTRACTOR HEREUNDER WILL BE USED TO CALCULATE THE ACTUAL COSTS OF TRAIN DELAY PURSUANT TO THIS AGREEMENT.**

Contractor and its subcontractors must give Railway's representative (Jason Sanchez 909-386-4075) four (4) weeks advance notice of the times and dates for proposed work windows. Railway and Contractor will establish mutually agreeable work windows for the project. Railway has the right at any time to revise or change the work windows due to train operations or service obligations. Railway will not be responsible for any additional costs or expenses resulting from a change in work windows. Additional costs or expenses resulting from a change in work windows shall be accounted for in Contractor's expenses for the project.

Contractor and subcontractors must plan, schedule, coordinate and conduct all Contractor's work so as to not cause any delays to any trains.

Kindly acknowledge receipt of this letter by signing and returning to the Railway two original copies of this letter, which, upon execution by Railway, will constitute an Agreement between us.

_____

Contractor

By: _____

Printed Name: _____

Title: _____

Contact Person: _____

Address: _____

City: _____

State: _____ Zip: _____

Fax: _____

Phone: _____

E-mail: _____

BNSF Railway Company

By: _____

Name: _____
Manager Public Projects

Accepted and effective this _____day of 2013.

Exhibit F

Bridge Requirements

BRIDGE DESIGN, PLANS & SPECIFICATIONS:

Bridge design calculations will be submitted to BNSF for review and approval. Design calculations will be summarized on a cover sheet to STATE the design dead load, design live load and amount of impact and any secondary loads considered. Bridge design by other than a BNSF pre-approved consultant will be subject to a third party design check at the STATE's expense.

For railroad bridges and culverts over waterways, BNSF will be provided with the Hydrology and Hydraulic design criteria, calculations and site specific data, including electronic data utilized in computer simulation modeling. Design considerations should include future land use changes that would increase the volume, velocity or sediment transport characteristics of the streambed flows. Design shall include sediment transport calculations and erosion control mitigation measures if necessary.

Bridge plans will be submitted to BNSF for review and approval at the 30%, 60% and 90% stage of design. The final set of plans must be approved by BNSF prior to letting of bids for the Project.

Prior to issuing any invitation to bid on construction of the Structure, the STATE should conduct a pre-bid meeting where prospective Providers have the opportunity to communicate with BNSF personnel as to the site specific train speeds, train density, and general safety requirements for men and equipment working near live tracks. Any invitation to bid and specifications for the Structure and the Project must be submitted to BNSF for review and approval prior to letting of bids for the Project.

BRIDGE CONSTRUCTION:

After awarding the bid, but prior to the Provider entering BNSF right of way, the STATE should conduct a pre-construction meeting with BNSF personnel in attendance to reiterate the safety requirements of construction activity adjacent to live tracks.

During construction, BNSF will require an experienced Quality Control Inspector to be present during certain critical times of the Project, including but not limited to: driving foundation piles, erecting falsework, construction of shoring and retaining walls, placing concrete, placing soil backfill and compaction processes. Certain components, normally produced off site, will require a QC inspector to be present, including the production of precast concrete components and steel fabrication. The QC inspector will provide reports to BNSF including pile driving records, concrete compressive test records, and other such reports requested by BNSF. QC Inspection services can be provided by the STATE subject to acceptance by BNSF. If the QC services provided are not acceptable, BNSF will provide an independent QC Inspection service to supplement the deficiencies. The STATE shall reimburse BNSF for all costs of supplemental QC Inspection services.

Within 90 days of the conclusion of the Project and final acceptance by BNSF, the STATE will provide BNSF with a complete electronic set of the bridge plans labeled "As Built". Those plans will reflect any and all deviations from the original plans that occurred during construction. The "As Built" plans will be submitted in Micro Station *.dgn electronic format (preferred) or AutoCAD *.dwg format. Electronic plans are to be submitted in the original format used for CAD plan preparation and not converted to another format prior to submission. The plans must show dimensioned locations of existing and relocated utilities.

BRIDGE MAINTENANCE:

BNSF will be responsible for maintenance of the track and ballast over the bridge and the earth embankment approaches. The STATE will be responsible for maintenance of the structural components of the bridge, including deck waterproofing and paint.

For bridges over roadways, the STATE will maintain the roadways under the bridge including signs, pedestrian walkways, fences, drains, landscaping, earth retention components, embankment slopes, erosion control, surface drainage, paint, walkways, handrails, lighting and other improvements associated with the Project. The STATE will be responsible to post highway vertical clearance signs and maintain such signs to reflect potential future pavement profile elevation changes. Clearance signs, traffic control signals or other signs should not be attached to a load-carrying member of the railroad bridge.

For railroad bridges over waterways, the STATE will be responsible for all aspects of channel maintenance across BNSF's right-of-way, including toe of slope erosion control where the railroad embankment meets the streambed. The STATE will be responsible for debris removal that may accumulate at bridge piers as well as silt removal, local and general scour and remediation and streambed lateral migration and streambed degradation remediation.

Fencing and other trespass controls within BNSF's right-of-way and incorporated into the Project shall be designed and maintained by the STATE. Graffiti removal will be the responsibility of the STATE.

BRIDGE INSPECTION:

BNSF will conduct annual routine structural inspections of railroad bridges over public roadways and waterways at no cost to the STATE. The STATE shall provide traffic control under those bridges spanning roadways to enable BNSF personnel or representatives to safely conduct the annual inspections. For routine annual inspections, BNSF will give the public STATE 7 days advance notice of traffic control requirements. In the event of an earthquake, fire, flood, damage from vehicular impacts or other emergent situations, the STATE will provide traffic control in roadways to allow immediate inspection by BNSF personnel or representatives. Traffic control will include lane closures or other such measures to allow BNSF personnel, providers and equipment to be safely positioned under the superstructure.

Upon the conclusion of the BNSF inspection, the STATE will be notified of items in need of repair. If the STATE does not complete the repairs requested by BNSF within a reasonable time period, BNSF personnel and/or providers will perform the repairs and invoice the STATE for the entire cost of such repairs. Regardless of the nature or cause of such maintenance and repairs, traffic control will be provided by the STATE at no cost to BNSF to allow safe access to the bridge.

FOUNDATION REPORT
ROUTE 4 CROSSTOWN FREEWAY EXTENSION
SAN JOAQUIN COUNTY, CALIFORNIA

December 20, 2012

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ONLY THE CLIENT OR ITS DESIGNATED REPRESENTATIVES MAY USE THIS DOCUMENT AND ONLY FOR THE SPECIFIC PROJECT FOR WHICH THIS REPORT WAS PREPARED.

KLEINFELDER
Bright People. Right Solutions.

A Report Prepared for:

San Joaquin Council of Governments
555 East Weber Avenue
Stockton, CA 95202

Designer:

Rajappan & Meyer Consulting Engineers, Inc.
1038 Leigh Avenue, Suite 100
San Jose, CA 95126

FOUNDATION REPORT
ROUTE 4 CROSSTOWN FREEWAY EXTENSION
SAN JOAQUIN COUNTY, CALIFORNIA

Kleinfelder Job No. 111904.G01
December 20, 2012

Prepared by:

Hamid "Saeed" Parsa, C.E., No. 74120
Project Engineer

REGISTERED PROFESSIONAL ENGINEER
HAMID REZA GHARIB PARSA
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CIVIL
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Reviewed by:

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File No.111904.G01
December 20, 2012

Mr. Keith Meyer
Rajappan & Meyer Consulting Engineers, Inc.
1038 Leigh Avenue, Suite 100
San Jose, CA 95126

Subject: **Foundation Report**
Route 4 Crosstown Freeway Extension
San Joaquin County, California

Dear Mr. Meyer:

Kleinfelder is pleased to present the results of our geotechnical services as part of the 100% Design submittal for the proposed Route 4 Crosstown Freeway Extension Project in San Joaquin County, California. The accompanying report includes background information regarding the anticipated construction, the purpose and the scope of our services provided, and regional and local geological conditions. In addition, descriptions regarding our investigative procedures and the site conditions encountered during our field exploration are presented. Finally, geotechnical conclusions and recommendations are provided for project design and construction. The appendices of the report include logs of exploratory borings and cone penetration tests, results of laboratory tests, and geotechnical design analysis performed for the project. We have also included an information sheet published by the American Society of Foundation Engineers (ASFE). Our firm is a member of ASFE, and we feel this sheet will help you better understand geotechnical engineering reports.

We appreciate the opportunity to provide our services for this project. If you have questions regarding this report or if you need further assistance, please contact us.

Sincerely,

KLEINFELDER WEST, INC.

Hamid "Saeed" Parsa, G.E. No. 74120
Project Engineer

SGP:lr
4c: Client

REGISTERED PROFESSIONAL ENGINEER
HAMID REZA GHARIB PARSA
No. 74120
CIVIL
STATE OF CALIFORNIA

Reviewed by:

Carl Henderson, Ph.D., G.E., No. 2886
Project Manager

REGISTERED PROFESSIONAL ENGINEER
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FOUNDATION REPORT
ROUTE 4 CROSSTOWN FREEWAY EXTENSION
SAN JOAQUIN COUNTY, CALIFORNIA

1.0 INTRODUCTION

1.1 GENERAL

The San Joaquin Council of Governments, in cooperation with the California Department of Transportation (Caltrans), proposes to extend State Route 4 (SR-4) West, also known as the "Crosstown Freeway," southwesterly from its current terminus at Fresno Avenue to Navy Drive in the City of Stockton (see Plate 1) in order to improve traffic access to the Port of Stockton and alleviate truck traffic through the Boggs Tract neighborhood. The improvements proposed for the SR-4 Crosstown Freeway Extension Project also include the widening of Navy Drive just south of the Burlington Northern Santa Fe (BNSF) Underpass to Fresno Avenue, stripping improvements on West Charter Way (SR-4) from Army Court to Tillie Lewis Drive, and installation of traffic signals at the Navy Drive/Tillie Lewis and West Charter Way/Tillie Lewis Drive intersections.

The purpose of our investigation was to evaluate the subsurface soil conditions at the project site in order to develop recommendations related to the geotechnical aspects of the project design and construction. Conclusions and recommendations presented in this report are based on review of previous geotechnical studies completed at the project site, publishes geologic data referenced in this report, and subsurface soil conditions encountered in test borings and cone penetration tests (CPTs) performed for this investigation. Recommendations presented herein should not be extrapolated to other areas or used for other projects without our prior review.

1.2 PROJECT DESCRIPTION

According to the "Draft Project Report - State Route 4 between Garfield Street Overhead (PM 15.7) and Navy Drive (PM 14.6) and West Charter Way between Army Court (PM 14.4) and Tillie Lewis Drive (PM 14.8), the proposed project extends SR-4 West southwesterly from its

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current terminus at Fresno Avenue to Navy Drive. The total length of the project is approximately 1.1 miles. The approximate location of the project site is shown on Plate 1.

1.2.1 Elevated Structure

The proposed extension will include approximately ½ mile of elevated structure spanning from just east of Fresno Avenue to about 1,100 feet west of South Ventura Avenue. The proposed 21-span, elevated structure will be twin viaducts supported by embankments at both ends and concrete columns varying in height from approximately 30 to 55 feet above existing ground surface.

The proposed elevated structure will have 12-foot wide lanes (2 lanes in each direction). A third lane providing through movement for eastbound Crosstown Freeway traffic will be reconstructed to begin east of Fresno Avenue and conform to the existing lane at the Garfield Street Overhead. Two viaducts were proposed from just west of South Ventura Avenue to just east of Fresno Avenue. The twin viaducts are each 42 feet wide and are separated by about 33 feet. The proposed eastbound and westbound mainline will each have 5-foot wide inside shoulders and 10-foot wide outside shoulders. Concrete barriers and railings will be constructed and placed on both the inside and outside shoulders at the edge of pavement for the entire length of the elevated structure.

1.2.2 Embankment

Embankments are proposed at both ends of the elevated structure. The western embankment, containing the eastbound on-ramp and westbound off-ramp, will extend from Navy Drive to the viaduct structure just south of the BNSF Railway. This embankment will vary in width from about 75 to 110 feet and be retained by three mechanically stabilized embankment (MSE) walls. The MSE walls are approximately 555 feet (Retaining Wall 1), 790 feet (Retaining Wall 2) and 83 feet (Retaining Wall 3) in length and range in height from about 0 feet at Navy Drive (west) to 45 feet at the viaduct interface (east). A Standard Type 1 wall (Retaining Wall 4) with a maximum height of 8 feet and approximately 750 feet long will be located on the south side of Navy Drive.

The eastern embankment for the eastbound and westbound mainline from just east of Fresno Avenue to the Garfield Street Overhead will include sloped embankments. It will measure

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about 170 feet wide and vary in height from about 0 to 30 feet above the existing ground level, with its highest point just east of Fresno Avenue. A Standard Type 1 Wall (Retaining Wall 5) with a maximum height of 14 feet and approximately 300 feet long will adjoin the east bound viaduct, eastern abutment.

1.2.3 Basins

Retention basins (two total) are proposed to be constructed adjacent the eastbound on-ramp and westbound off-ramp of the western approach embankment. Detention Basins (two total) are proposed to be constructed at the east end of the alignment, located adjacent (southwest and southeast) of Fresno Avenue. The existing "LKQ" property basin located within the approximate footprint of proposed Bents 4 and 5 of the eastbound viaduct structure will be partially filled to be used as a temporary staging area for bridge construction. An 11 feet high (exposed height) gabion basket gravity wall is proposed to be placed to retain the crushed drain rock which will be used to partially fill the drainage basin.

1.3 PURPOSE AND SCOPE OF WORK

The purpose of this report is to evaluate the general soil conditions along the project alignment, to evaluate their engineering properties, and to provide foundation design recommendations for the proposed overcrossing structures. The scope of work included the following:

- A visual site reconnaissance to investigate the surface conditions at the project site
- A review of geologic data pertaining to the project site
- A field investigation that consisted of drilling thirteen test borings, performing eleven CPTs, and advancing five geoprobes within the area of the proposed development to explore the subsurface conditions
- A review of previous studies conducted in the vicinity of the project site to evaluate the general soil conditions and their engineering properties relevant to the project design and construction
- Laboratory testing of representative samples obtained during the field investigation to evaluate relevant physical and engineering parameters of the subsurface soils

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- Evaluation of the data obtained and an engineering analysis to develop our geotechnical conclusions and preliminary recommendations for foundations
- Preparation of this report which includes:
  - A description of the proposed project
  - A description of the field and laboratory investigations
  - A description of the surface and subsurface conditions encountered during our field investigation
  - Conclusions and recommendations related to the geotechnical aspects of the project design and construction
  - Plates that include site vicinity and exploration location maps, geologic map, fault map and recommended ARS curves, and
  - Appendices that include logs of borings and CPTs for current and previous investigations, results of laboratory tests for current and previous investigations, ARS curves development, results of liquefaction analysis, soil profile models used for pile analysis, axial capacity analysis results for CIDH piles, MSE Wall external and global stability analysis, and embankment settlement analysis.

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2.0 PREVIOUS STUDY

Parikh performed a preliminary geotechnical investigation at the project site. Their findings and preliminary foundation recommendations for the project were summarized in a report titled "Preliminary Foundation Report, Route 4 Crosstown Connector, San Joaquin County, California," dated March 2010. As part of their investigation, three exploratory borings and two CPTs were performed along the project alignment to a maximum depth of about 121 feet below existing ground surface. Logs of exploratory borings and CPTs are presented in Appendix B.

Parikh also performed laboratory tests in accordance with ASTM Standards on selected soil samples collected during field exploration to evaluate their physical characteristics and engineering properties. The laboratory testing program was formulated with emphasis on the evaluation of natural moisture content, in-place density, plasticity, consolidation, and grain size distribution of the soil samples. Results of laboratory tests are presented in Appendix D.

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3.0 FIELD EXPLORATIONS

3.1 FIELD INVESTIGATION

For Kleinfelder's bridge foundation investigation, thirteen test borings and eleven CPTs were advanced to depths ranging from about 31½ to 151½ feet below existing grade. Subsurface information obtained from five geoprobe explorations advanced to depths ranging from about 16 to 20 feet, from a Detailed Site Investigation completed by Kleinfelder for the Debco Autowrecking, Inc. facility (Kleinfelder File No. 113961.4/STO11R302, dated December 16, 2011) was also used for our roadway design investigation (Specifically for Retaining Wall 4). Detailed descriptions of site surface conditions encountered in our test borings, CPTs, and geoprobe explorations are presented in Section 6.2 of this report. The approximate boring, CPT, and geoprobe locations are presented on Plate 2.

The borings were drilled using a CME-75 or 85 truck-mounted drill rig equipped with 8- inch O.D. hollow-stem auger and using rotary wash method below groundwater. During the drilling operations, penetration tests were performed in accordance with ASTMD-1586 at regular intervals using a modified California sampler to evaluate the relative density of coarse-grained (cohesionless) soil and to retain soil samples for laboratory testing. A pocket penetrometer was used to evaluate the consistency of fine-grained (cohesive) soil samples retained. The penetration tests were performed by initially driving the sampler 6 inches into the bottom of the bore hole using a cathead system and a 140-pound trip-hammer, falling 30 inches to penetrate loose soil cuttings and "seat" the sampler. Thereafter, the sampler was progressively driven an additional 12 inches, with the results recorded as the corresponding number of blows required to advance the sampler 12 inches, or any part thereof. A representative with our firm maintained a log of the borings and visually classified the soils encountered according to the unified soil classification system. Soil samples obtained from the borings were packaged and sealed in the field to reduce moisture loss and disturbance and brought to our Stockton laboratory for future testing. Additionally, 3 inch diameter Shelby Tubes were used to obtain relatively undisturbed soil samples at several locations.

The CPTs were performed by hydraulically pushing a 1.4-inch diameter Dutch Cone Penetrometer into the subsurface soils using an enclosed truck-mounted 20-ton ram system. During penetration, the cone or tip resistance and sleeve friction resistance is recorded on a

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nearly continuous basis to the depth of exploration. Based on published correlations, the data obtained is used to estimate stratigraphy, soil type, groundwater depth, and in situ soil parameters such as undrained shear strength and standard penetration (N) values. The soils classifications and blow counts encountered in the test borings are consistent with the data correlations obtained from the CPTs.

Geoprobes/soil probes (probes) were continuously advanced in 4- to 5-foot intervals. During advancement of the probes, Kleinfelder collected soil samples for logging purposes, field screening, and potential chemical analysis. The probes were pneumatically and/or hydraulically advanced using a core barrel. Soil samples were continuously collected by inserting a new acetate liner into the core barrel prior to each sampling interval. While probing, an experienced environmental geologist/engineer classifies the soil, logs the stratigraphy of the borings, and collected soil samples.

Descriptions of soil materials encountered in the explorations are presented on the Logs of Test Borings in Appendix A. The borings, CPTs, and geoprobes were located in the field by visual sighting and/or pacing from existing site features; therefore, the locations shown on Plate 2 should be considered approximate. The penetration resistance (blows/foot) shown on the logs of borings represents field penetration that has not been corrected for overburden pressure, sampler size, hammer type, borehole diameter, rod length, sampling method or any other correction factor. The preparation of boring logs was according to the Caltrans "Soil & Rock Logging, Classifications, and Presentation Manual."

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4.0 GEOTECHNICAL TESTING

4.1 IN SITU TESTING

In situ testing on the project was limited to penetration testing during drilling operations with the drill rig and pocket penetrometer tests on the soil samples retrieved. Blow counts were recorded on the drilling logs to document the apparent relative density of the coarse-grained (cohesionless) soils encountered. The testing was performed in general accordance with ASTM standard test method D-1586 using a trip hammer. The pocket penetrometer test results were used to evaluate the consistency of fine-grained (cohesive) soils.

4.2 LABORATORY TESTING

Laboratory tests were performed in accordance with current ASTM standards on selected soil samples to evaluate their physical characteristics and engineering properties. The laboratory testing program was formulated with emphasis on the evaluation of natural moisture content, in-place density, grain-size distribution, percent soil passing the #200 sieve, plasticity, consolidation, and undrained shear strength of the materials encountered. In addition, pH, minimum resistivity, sulfate concentration and chloride concentration tests were performed on near-surface soil samples to evaluate the general corrosivity of the soils to buried concrete and ferrous metals. For design of the proposed pavement sections within the project site, five resistance value (R-Value) tests were performed on selected near-surface soil samples.

The results of Kleinfelder laboratory tests are summarized in Appendix C. This information, along with the field observations, was used to prepare the final test boring logs.

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5.0 SITE CONDITIONS

5.1 SURFACE CONDITIONS

The project site is bounded by the Stockton Deep Water Channel on the North, San Joaquin River and Rough and Ready Island on the west, West Charter Way on the South, and I-5 on the East. The project alignment is generally located in a semi-developed area consisting of residential, industrial, and commercial buildings. The terrain along the alignment west of Fresno Avenue is generally flat, with an elevation of about 6 feet, increasing to an elevation of about 30 feet at the eastern end of the proposed project.

5.2 SUBSURFACE CONDITIONS

The subsurface conditions encountered during our field investigations were consistent with the previous findings throughout the project site. The following table presents depths to groundwater and the subsurface soils conditions encountered in our field explorations. Subsurface soils can be generally characterized as fine grained and cohesive (CL, ML, SC) with some interbedded layers of silty and "clean" sand. Soils generally become stiffer with depth and can be characterized as an intermediate geo material at depths of approximately 90 feet and deeper below existing grade.

Table 5.1
Summary of Subsurface Conditions

Exploration ID	Depth (Feet)	Depth to GW (Feet)	Subsurface Soil Conditions
R-11-001	91½	8	Firm to hard fat clay to sandy lean clay to about 30 to 34 feet, followed by medium dense to dense silty to relatively "clean" sand and very hard fat clay
R-11-002	91½	8	
R-11-003	151½	8	Firm to hard fat clay to sandy lean clay to about 19 feet, followed by medium dense to dense silty to relatively "clean" sand to about 39 feet, hard lean clay to about 54 feet underlain by interbedded layers of silt, lean clay, and poorly graded sand
R-11-004	91½	18	Firm to hard sandy lean clay and clayey sand to about 24 feet, followed by interbedded layers of "clean" sand and sandy clay to about 50 feet, underlain by sandy lean clay
R-11-005	91½	18	Firm to hard Sandy lean clay and clayey sand to about 70 feet, underlain by dense to very dense silty to relatively "clean" sand
R-11-006	91½	18	Firm to hard lean clay to sandy lean clay to the maximum depth with interbedded layers of silty sand between 26 to 49 feet below grade

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Exploration ID	Depth (Feet)	Depth to GW (Feet)	Subsurface Soil Conditions
R-11-007	91½	11	Firm to hard lean clay with interbedded layers of silty to relatively "clean" sand to 45 feet underlain by hard to very hard sandy lean clay
R-11-008	91½	15	Firm to hard lean clay with interbedded layers of silty to relatively "clean" sand to about 78 feet underlain by hard to very hard fat clay
R-11-009	121½	14	Firm to hard lean clay to fat clay to about 34 feet underlain by interbedded layers of sandy silt and silty to relatively "clean" sand to about 80 feet followed be very hard sandy lean clay to 118 feet underlain by "clean" sand
R-11-010	51½	36	Medium dense silty sand to about 19 feet, underlain by firm to hard sandy lean clay to 44 feet followed by medium dense silty sand
R-11-011	151½	14	Firm to hard fat clay to 10 feet followed by medium dense silty to relatively "clean" sand to 20 feet, sandy lean clay to 60 feet, medium dense to dense silty to relatively "clean" sand to 70 feet, and hard to very hard sandy lean clay to fat clay.
R-11-012	51½	14	Firm to hard sandy lean clay to 20 feet, followed by medium dense "clean" sand to 29 feet, and firm to hard sandy lean clay, clayey sand, and fat clay.
R-11-013	31½	14	Firm to hard sandy lean clay
CPT-11-001	90	11	Lean Clay to sandy lean clay and silt to 60 feet underlain by silty to relatively "clean" sand to 65 feet, lean clay 86 feet and sand
CPT-11-002	90	10	Lean Clay to sandy lean clay and silt with occasional interbedded layers of silty to relatively "clean" sand
CPT-11-003	90	10	Lean Clay to 25 feet followed by relatively "clean" sand to 30 feet, sandy silt and clay to 42 feet, clayey to silty sand and "clean" sand to 47 feet , underlain by sandy silt and sandy lean clay to 84 feet, and silty to relatively "clean" sand
CPT-11-004	90	11	Lean Clay with interbedded seams of sandy silt to about 22 feet followed by silty to relatively "clean" sand to about 25 feet, lean clay with occasional interbedded layers of sandy silt to 67 feet, silty sand to "clean" sand to 76 feet, sandy lean clay to 82 feet, and silty to "clean" sand to 87 feet followed by sandy silt and clay
CPT-11-005	90	14	Sandy lean clay and silt with seams of silty sand
CPT-11-006	90	13	Sandy lean clay and silt with a 2-foot thick interbedded layer of silty to relatively "clean" sand at 84 feet below site grade
CPT-11-007	90	10¾	Sandy lean clay and silt to 20 feet followed by silty sand and "clean" sand with interbedded seams of clay and silt to 42 feet followed by sandy lean clay and silt
CPT-11-008	90	11	Sandy lean clay and silt to 36 feet followed by silty sand and "clean" sand with interbedded seams of clay and silt to 49 feet followed by sandy lean clay and silt to 78 feet, underlain by sandy lean clay
CPT-11-009	50	24¾	Silty sand to 5 feet followed by sandy lean clay and silt to 38 feet, underlain by silty to relatively "clean" sand
CPT-11-010	105 (refusal)	9	Sandy lean clay and silt to 37 feet followed by silty sand and "clean" sand to 42 feet, sandy lean clay and silt to 48 feet followed by sandy lean clay and silt to 62 feet, underlain by sandy lean clay

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Exploration ID	Depth (Feet)	Depth to GW (Feet)	Subsurface Soil Conditions
			to 76 feet, silty sand to "clean" sand to 84 feet, and sandy lean clay to the end
CPT-11-011	50	7½	Sandy lean clay and silt to 27 feet followed by silty to "clean" sand to 43 feet and sandy silt and clay
O-11-1 (Geoprobe)	16	13	Sandy gravel to 2 feet followed by clay with sand to 16 feet. An interbedded layer of sand with clay was encountered between 13 and 14 feet.
O-11-2 (Geoprobe)	20	16	Clay with sand to a depth of 20 feet, interbedded with a layer of sand with clay between 6.5 and 11 feet.
O-11-3 (Geoprobe)	16	14	Clay with little sand to 11 feet, underlain by sand with clay to depth of 16 feet.
O-11-4 (Geoprobe)	16	11	Sandy gravel to 2 ½ feet followed by clay with sand to 16 feet. An interbedded layer of sand with clay was encountered between 10 ½ and 13 ½ feet.
O-11-5 (Geoprobe)	16	12	Gravelly sand to 2 ½ feet followed by clay with sand to 11 ½ feet. A layer of sand with clay was the next layer to 14 ½ feet followed by gravel with trace sand to 16 feet.

5.3 GROUNDWATER DEPTHS

The test borings and CPTs were checked for the presence of groundwater during and immediately following drilling operations. Groundwater was encountered at depths ranging from of about 7½ (elev. ± -1½) to 36 (elev. ± -18) feet, from west to east, below existing grade. Records maintained by the California Department of Water Resources (DWR)[1] indicate that historical high groundwater level in the vicinity of the project site is about 10 to 20 feet below the ground surface. Because of its close proximity to the Port of Stockton, we anticipated that the groundwater at the project site should be located at a relatively shallow depth on the order of about 10 to 15 feet. Groundwater elevations and soil moisture conditions within the project area will vary depending on seasonal rainfall, irrigation practices, land use, and/or runoff conditions not apparent at the time of our field investigation. The evaluation of such factors is beyond the scope of this investigation.

[1] *http://wdl.water.ca.gov/gw/*

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6.0 GEOLOGIC CONDITIONS

6.1 REGIONAL GEOLOGY

The project site lies within the central portion of the Great Valley geomorphic province of California. The province is bordered to the north by the Cascade and Klamath ranges; to the west by the structurally complex sedimentary and volcanic rock units of the Coast Ranges, to the east by the granitic and metamorphic basement rocks which form the gently sloping western foothills of the Sierra Nevada range, and to the south by the east-west trending Transverse ranges. About 400 miles (645 km) long and 50 miles (80 km) wide, the Great Valley is an asymmetrical, synclinal trough formed by tilting of the Sierran block during the late Tertiary and Quaternary periods with the western side dropping to form the valley and the eastern side uplifting to form the Sierra Nevada mountains. Within the project area, erosion of the adjacent Sierra Nevada and Coast Ranges has in-filled the valley with a thick sequence of unconsolidated to semi-consolidated Quaternary (Pleistocene and Holocene) age alluvial, basin, and delta plain sediments deposited by the Sacramento and San Joaquin rivers and their tributaries. The thickness of the valley sediments varies from a thin veneer at the edges of the valley to thousands of meters in the western portion. The bedrock complex is composed of metamorphosed marine sediments similar to those found in the foothills of the western Sierra Nevada and the core of the Coast Ranges.

6.2 LOCAL GEOLOGY

Various authors have mapped the local geology of the project area (e.g., Atwater, 1982). These maps differ in scale and detail but agree that the project site is underlain at the surface by Late Pleistocene age (1.7 million years) clay, silt, sand, and gravel of the Modesto Formation (Qm). These soils were probably eroded from the nearby Sierra Nevada Mountain Ranges and deposited as fan, mudflow, terrace, and floodplain deposits from the nearby streams and rivers. A geological map of the general project area is shown on Plate 3.

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7.0 SEISMIC RECOMMENDATIONS

7.1 FAULTING AND SEISMICITY

Based on the information provided in Hart and Bryant (2007), the site is not located within a State-designated Alquist-Priolo Earthquake Fault Zone where site-specific studies addressing the potential for surface fault rupture are required, and no known active faults traverse the site. Stockton is in a region where there are very few active faults.

An active fault is a fault that has experienced seismic activity during historic time (since roughly 1800) or exhibits evidence of surface displacement during Holocene time (Hart and Bryant, 2007). The definition of "potentially active" varies. A generally accepted definition of "potentially active" is a fault showing evidence of displacement that is older than 11,000 years (Holocene age) and younger than 1.7 million years (Pleistocene age). However, "potentially active" is no longer used as criteria for zoning by the California Geologic Survey (CGS), formerly known as the State of California Division of Mines and Geology. The terms "sufficiently active" and "well-defined" are now used by the CGS as criteria for zoning faults under the Alquist-Priolo Earthquake Fault Act. A "sufficiently active fault" is a fault that shows evidence of Holocene surface displacement along one or more of its segments and branches, while a "well-defined fault" is a fault whose trace is clearly detectable by a trained geologist as a physical feature at or just below the ground surface. The definition "inactive" generally implies that a fault has not been active since the beginning of the Pleistocene Epoch (older than 1.7 million years old).

The San Andreas fault (SAF), located about 70 miles (113 km) to the west, is considered to dominate the structure and seismicity of the region. This right-lateral strike-slip fault, which extends from the Gulf of California in Mexico to Cape Mendocino off the Coast of Humboldt County in Northern California, forms a portion of the boundary between two independent tectonic plates on the surface of the earth. To the west of the SAF is the Pacific Plate, which moves north relative to the North American Plate, located east of the fault. In the San Francisco Bay area, movement across this plate boundary is concentrated on the SAF; however, it is also distributed, to a lesser extent, across a number of faults which include the Calaveras, Concord-Green Valley, Clayton-Marsh Creek-Greenville, and Hayward-Rodgers Creek faults among others. Together, these faults are referred to as the SAF system.

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Several secondary or potentially active fault traces are located in the general vicinity of the project site. The closest of these are the Great Valley fault 7, the Marsh Creek-Greenville section of the Greenville fault zone, and the Las Positas fault, located to the west of the site.

The Great Valley fault is characterized by a zone of concealed or "blind" thrust faulting, reverse faults, and folds that extend for several hundred kilometers from the southern San Joaquin Valley in Kern County northward to the northern San Joaquin Valley in Tehama County. Although not exposed at the surface, regional studies have suggested the Great Valley fault system may be comprised of between 18 and 25 segments ranging from about 8 to 35 miles (12 to 57 km) in length, with most segments between 12 and 18 miles (20 and 30 km), on the basis of the seismo-tectonic environment of the area. The characteristic earthquake for the average-length segment is believed capable of yielding earthquakes of magnitude 6.3 to 6.4, or slightly smaller than the 1983 (M6.5) Coalinga earthquake, and the longest segments are believed capable of yielding earthquakes as large as magnitude 6.8 to 7.0, or equal to the Vacaville-Winters earthquake.

Located in the Diablo Range of the Coast Ranges Geomorphic Province, the Greenville fault zone is identified as a right lateral strike slip fault with a dip of 90 degrees. The Marsh Creek-Greenville section of the fault zone experienced minor surface fault rupture in January of 1980.

The table below presents a list of significant active or potentially active faults within approximately 50 kilometers of the project site, their assigned moment magnitudes and the distance from the fault trace at the surface to the site (Merriam and Shantz, 2007). A fault map for the general project area is presented on Plate 4.

Table 7.1
Regional Faults and Characteristics

Fault	Closest Distance to Site (km)	Maximum Magnitude (M_{Max})
Great Valley fault 7	28 (W)	6.7
Greenville fault zone (Marsh Creek-Greenville section)	42 (W)	6.6
Las Positas Fault	43 (W)	6.5

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7.2 SEISMIC DESIGN PARAMETERS

The governing deterministic seismic event for the project site is a magnitude 6.7 earthquake on the Great Valley 7 fault (Mw=6.7, style of faulting:reverse). This fault is located about 17.5 miles from the project site.

Based on the available boring and CPT information, the subsurface soil conditions at the project site consist of alternative layers of stiff to hard lean clay and silt, interbedded with loose to medium-dense silty sand and poorly-graded sand with silt to the maximum depth explored. These types of soils generally match the criteria for Soil Type D, as per Caltrans Seismic Design Criteria (version 1.6, 2010). The average small strain shear wave velocity (V_{S30}) for the upper 100 feet (30 meters) was estimated to be 820 feet per second (ft/s)/249 meter per second (m/s), using the SPT blow count correlations developed by Sykora (1987) and Ohta and Goto (1978).

The acceleration response spectrum (ARS) curve was developed with the procedures newly established by Caltrans (2009a, 2009b). Both the deterministic and probabilistic procedures were adopted to calculate the acceleration response spectrum, using the Deterministic Response Spectrum Spreadsheet and Probabilistic Response Spectrum Spreadsheet, respectively. The probabilistic ARS data developed using ARS Online was compared with the data from the USGS Interactive Deaggregation Tool. Since the maximum difference between the ARS Online data and the USGS data was greater than 10 percent, the USGS data was selected for our probabilistic design spectrum. Furthermore, a comparison made between the probabilistic and deterministic response spectra indicated that the probabilistic response spectrum governs the site. The design ARS Curve for the proposed project is presented on Plate 5. The deterministic ARS data are also presented on Plate 5 for comparison purposes. Details of the ARS curve development are presented in Appendix E.

Recommended seismic design parameters are presented in the table below.

Table 7.2
Seismic Design Parameters

Seismic Parameters	Design Recommendations
Controlling Fault	Great Valley 7
Type of Fault	Reverse

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Seismic Parameters	Design Recommendations
Earthquake Magnitude	6.7 Mw
Peak Ground Acceleration	0.32g
Small Strain Shear Wave Velocity	249 m/s
Design ARS Curve	See Plate 5

7.3 LIQUEFACTION

A common secondary hazard of strong ground shaking is the potential for soil liquefaction. Soil liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary but essentially total loss of shear strength due to build-up of excess pore water pressure under the reversing cyclic shear strength associated with earthquake shaking. Dissipation of the excess pore pressures will produce volume changes within the liquefied soil layer, which can manifest at the ground surface as settlement of structures, floating of buried structures, failure of retaining walls, lateral migration (lateral spreading), and extensional ground cracking of liquefied material. Factors known to influence liquefaction include soil type, structure, grain size, relative density, confining pressure, depth to groundwater, and the intensity and duration of ground shaking. Soils most susceptible to liquefaction are saturated, loose, sandy soils. Clays are generally not susceptible to liquefaction.

Liquefaction analysis was performed based on CPT data assuming a peak ground acceleration of 0.32 g, an earthquake magnitude of 6.7, and empirical methods developed by Robertson and Wride (1998) as summarized by the NCEER Liquefaction Workshop expert panel (Youd, et al., 2001) with modifications proposed by Seed (2003). Parikh also evaluated the liquefaction potential evaluation at the project site in accordance with the methods proposed by Youd, et al. (2001) using CPT procedures. The Parikh analysis identified potentially liquefiable alluvial soil layers (i.e., silty sand and poorly-graded sand with silt) ranging between 2.5 to 4 feet thick, varying at depths of 11.5 to 43 feet below ground surface, with a post-liquefaction settlement on the order of 3 to 4 inches. However the Parikh analysis was performed based on a peak ground acceleration of 0.40g. Kleinfelder has reevaluated liquefaction assessments from CPT-09-001 and CPT-09-002 data , using a peak ground acceleration of 0.32g.

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The results of our liquefaction analysis are summarized in the following table. The data is presented in order of the most "western" CPT location (CPT-11-011) to the most "eastern" CPT location.

Table 7.3
Summary of Liquefaction Analysis

CPT No.	Total Depth (Feet)	Depths of Continuous Liquefied Layers (Feet)	Total settlement (Inches)	Approximate Depth of ½-inch Relative Soil Movement (Feet)
CPT-11-011	50	27-28, 34-38	0.92	35
CPT-11-001	90	40 – 41, 88 - 89	0.58	24
CPT-11-002	90	48 – 49, 73 - 75	0.73	60
CPT-11-010	105	33-34	0.26	NA
CPT-09-001	120	11-13, 18-20, 31-32, 43	1.90	21
CPT-09-002	120	19-23	1.34	22
CPT-11-003	90	17 - 19	0.25	NA
CPT-11-004	90	10 - 11, 20 – 26, 69 - 75	1.89	70
CPT-11-005	90	45 – 46	0.28	NA
CPT-11-006	90	54 - 55	0.13	NA
CPT-11-007	90	20 - 40	3.90	37
CPT-11-008	90	34–47, 64-65, 67-68, 77-78	1.85	55
CPT-11-009	50	--	0	NA

The results of our liquefaction analysis are also presented on graphs in Appendix F. Results of our liquefaction analysis indicate the presence of discontinuous layers of potentially liquefiable soils throughout the site. However a clearly defined continuous liquefiable soil layer does not appear to present. Therefore, it is our opinion that the potential for liquefaction at the project site is low.

When piles are installed through a soil deposit which undergoes settlement, the resulting relative downward movement of the soil around the piles induces "downdrag"/negative shaft resistance forces on the piles. The amount of relative settlement between the soils and pile that is necessary to mobilize negative shaft resistance is about 0.4 and 0.5 inches. For this project site at five of the thirteen CPT exploration locations, liquefaction settlement was estimated to be less than ½ inch. Also, as mentioned above, potentially liquefiable soil layers are discontinuous throughout the site. Therefore it is our opinion that the recommended pile

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lengths presented in Section 9.2.1 of this report are sufficiently conservative to accommodate the potential for downdrag loading without further increase of pile lengths. Further discussion on negative skin friction is provided in Section 9.2.3.

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8.0 CORROSION POTENTIAL

Caltrans Corrosion Guidelines state that the Department considers a site to be corrosive to foundation elements if one or more of the following conditions exist for the soil and/or water samples taken at the site (Caltrans, 2003):

- Chloride concentration is greater than or equal to 500 ppm
- Sulfate concentration is greater than or equal to 2000 ppm
- pH is 5.5 or less

The corrosivity of soils towards buried ferrous metals is commonly evaluated based on the correlations provided in Table 8.1:

Table 8.1
Corrosivity vs. Soil Resistivity

Soil Resistivity	Corrosivity
0 to 1,000 ohm-cm	Severely corrosive
1,000 to 2,000 ohm-cm	Corrosive
2,000 to 10,000 ohm-cm	Moderately corrosive
Over 10,000 ohm-cm	Mildly corrosive

Corrosion tests were performed on composite soil samples in accordance with Caltrans and EPA procedures to evaluate the corrosivity of the soils to buried concrete and ferrous metals. The corrosion test results of the soil samples are summarized in Table 8.2. The subsurface soils at the project site are potentially severely to moderately corrosive to buried ferrous metals.

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Table 8.2
Results of Soil Corrosion Tests

Sample Locations	Sample Depth (feet)	USCS Soil Type	pH	Sulfate Content (ppm)	Chloride Content (ppm)	Minimum Resistivity (ohm-cm)
Near R-11-003	1-4	Sandy Lean Clay (CL)	8.1	460	230	570
Near R-11-004	1–4	Sandy Lean Clay (CL)	7.5	240	340	470
Near R-11-006	1-4	Sandy Lean Clay (CL)	7.8	400	140	530
Near R-11-008	1-4	Sandy Lean Clay (CL)	7.4	89	17	2,100
Near R-11-009	1-4	Sandy Lean Clay (CL)	7.5	59	18	2,000

The above corrosion test results indicate that the soils near the western portion of the project site are not corrosive to buried concrete. As we indicated above, the soils may be corrosive to buried metals. For steel bars used in concrete, a minimum 3-inch concrete over all steel bars will provide adequate protection against corrosion for the bars.

Kleinfelder has performed these soil corrosivity tests as requested by the client. These tests are only an indicator of soil corrosivity. You may wish to retain a competent corrosion engineer to design corrosion protection systems appropriate for the project.

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9.0 CONCLUSIONS AND RECOMMENDATIONS

9.1 GENERAL

Given the site subsurface soil conditions encountered during our field exploration and anticipated foundation design loads, it is our professional opinion that from a geotechnical standpoint, cast-in-drilled-hole (CIDH) piles should be feasible foundation systems for the abutments and bents of the proposed structure provided the recommendations contained herein are incorporated into the project design and construction. Because of the anticipated shallow groundwater depth, the presence of interbedded sandy soils, and BNSF requirements, permanent steel casing for a depth of 20 feet should be used to provide support for the 9-foot diameter holes excavated for construction of the CIDH pile foundations (Bents 6, 7, 8, and 9) near the BNSF tracks. Additionally, temporary steel casing and/or drilling fluids should be considered for support during excavation of holes for all CIDH piles foundations. It is our professional opinion that one-stage construction technique should be appropriate for the construction of MSE walls. To reduce post-construction consolidation settlements, we recommend that preloading/staged construction be considered for the western approach embankments. Construction of approach embankments should be appropriately staged for foundation soils to achieve sufficient strength to support the overlying embankment loadings. Specific conclusions and recommendations regarding the geotechnical aspects of foundation design and construction are presented in the following sections.

9.2 DEEP FOUNDATIONS

During the PSR phase of the project, 7- and 8-foot diameter CIDH piles were considered for foundation support at Bents 6, 7, 8, 9 and 10, and precast prestressed (PC/PS) concrete piles (Class 200) were considered for all other bents and the abutments. However, because of Boggs Tract resident concerns regarding construction noise in the area from pile driving operations, CIDH piles are now being considered for foundation support at all abutment and bent locations. The decision to consider CIDH piles at all substructure locations was made after our drilling operations were completed.

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9.2.1 Axial Capacity

Per Caltrans design guidelines, deep foundations supporting the bents and Abutments 1W and 1E (which behave like bents) will be designed based on Load Resistance Factor Design (LRFD). Deep foundations supporting Abutments 22W and 22E will be designed based on Working Stress Design. A total of thirteen soil models/profiles were established in our assessment of geotechnical engineering parameters used in pile design. Summaries of the information used in developing our models are presented in Appendix G. Ultimate (nominal) axial compressive capacity was estimated with the procedures outlined in FHWA Publication No. FHWA-NHI-10-016, "Drilled Shafts: Construction Procedures and LRFD Design Methods," dated May 2010. End bearing associated with strain compatibility is insufficient to warrant cleaning the pile bottom. Therefore, axial capacity is based on skin friction only. In accordance with Table 10.5.5.2.4-1 of Caltrans publication "California Amendments to AASHTO LRFD Bridge Design Specifications- Fourth Edition" a resistance factor of 0.7 was used in estimation of factored axial compression design capacity. Estimated axial compression capacity capacities based on the above mentioned soil models are presented on spreadsheets and graphs in Appendix H. Since the upper 20 feet of shaft for 9-foot diameter CIDH piles will be permanently cased, the skin friction contribution within the upper 20 feet of 9-foot diameter CIDH piles was ignored in our axial capacity analysis. Preliminary pile data tables for the foundations are provided below. Since pinned connections are being used for designed, tension loads for the foundations are not considered.

Table 9.1
Pile Data Table – West Viaduct

Support Location	Pile Type	Design Load	Nominal Resistance		Cut-off Elevation	Steel Casing Specified Pile Tip Elevations	Design Tip Elevations	Specified Tip Elevations
			Compression	Tension				
Abut 1W	6' CIDH	N/A	1,390 kips	0 kips	4 ft	N/A	-46 ft (1) -86 ft (3)	-86 ft (3)
Bent 2W	6' CIDH	N/A	2,780 kips	0 kips	4 ft	N/A	-76 ft(1), -86 ft(3)	-86 ft(3)
Bent 3W	6' CIDH	N/A	2,790 kips	0 kips	4 ft	N/A	-76 ft (1), -86 ft(3)	-86 ft(3)
Bent 4W	8' CIDH	N/A	2,570 kips	0 kips	-3 ft	N/A	-67 ft(1), -86 ft(3)	-86 ft(3)
Bent 5W	8' CIDH	N/A	3,360 kips	0 kips	-3 ft	N/A	-78 ft(1), -86 ft(3)	-86 ft (3)
Bent 6	9' CIDH with permanent steel casing	N/A	6,850 kips	0 kips	-2 ft	-22 ft	-92 ft (1), -117 ft(3)	-117 ft (3)

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Support Location	Pile Type	Design Load	Nominal Resistance		Cut-off Elevation	Steel Casing Specified Pile Tip Elevations	Design Tip Elevations	Specified Tip Elevations
			Compression	Tension				
Bent 7	9' CIDH with permanent steel casing	N/A	12,400 kips	0 kips	0 ft	-20 ft	-150 ft(1), -115 ft(3)	-150 ft(1)
Bent 8	9' CIDH with permanent steel casing	N/A	12,150 kips	0 kips	4 ft	-16 ft	-146 ft (1), -111 ft(3)	-146 ft(1)
Bent 9	9' CIDH with permanent steel casing	N/A	8,760 kips	0 kips	4 ft	-16 ft	-126 ft(1), -111 ft(3)	-126 ft(1)
Bent 10W	8' CIDH	N/A	3,490 kips	0 kips	4 ft	N/A	-69 ft (1), -86 ft(3)	-86 ft (3)
Bent 11W	8' CIDH	N/A	3,290 kips	0 kips	4 ft	N/A	-68 ft(1), -86 ft(3)	-86 ft (3)
Bent 12W	8' CIDH	N/A	2,780 kips	0 kips	4 ft	N/A	-62 ft (1), -86 ft (3)	-86 ft (3)
Bent 13W	8' CIDH	N/A	3,310 kips	0 kips	4 ft	N/A	-68 ft (1), -86 ft (3)	-86 ft(3)
Bent 14W	8' CIDH	N/A	3,310 kips	0 kips	4 ft	N/A	-68 ft(1), -86 ft(3)	-86 ft(3)
Bent 15W	8' CIDH	N/A	2,820 kips	0 kips	4 ft	N/A	-63 ft (1), -86 ft(3)	-86 ft(3)
Bent 16W	6' CIDH	N/A	3,280 kips	0 kips	4 ft	N/A	-84 ft (1), -86 ft(3)	-86 ft (3)
Bent 17W	6' CIDH	N/A	3,290 kips	0 kips	4 ft	N/A	-84 ft (1), -86 ft(3)	-86 ft(3)
Bent 18W	6' CIDH	N/A	2,670 kips	0 kips	4 ft	N/A	-74 ft(1), -86 ft(3)	-86 ft(3)
Bent 19W	6' CIDH	N/A	3,330 kips	0 kips	4 ft	N/A	-84 ft(1) -86 ft (3)	-86 ft (3)
Bent 20W	6' CIDH	N/A	2,870 kips	0 kips	4 ft	N/A	-77 ft (1), -86 ft (3)	-86 ft(3)
Bent 21W	6' CIDH	N/A	3,030 kips	0 kips	2 ft	N/A	-81 ft(1), -86 ft(3)	-86 ft(3)
Abut 22W	6' CIDH	800kips	1,600 kips	0 kips	15 ft	N/A	-40 ft(1) -45 ft (3)	-45 ft(3)

Notes:
1. Design tip elevation is controlled by the following demands: (1) Compression, (2) Tension, (3) Lateral, and (4) Liquefaction
2. Design tip elevations for Bents 6, 7, 8, and 9 were estimated based on permanent casing for the upper 20 feet.
3. Specified tip elevation shall not be raised unless allowed by the Geotechnical Engineer.
4. Per direction of Structural Engineer, pile lengths based on lateral demand is considered by determining the depth of zero deflection multiplied by a factor of safety of 1.5.

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Table 9.2
Pile Data Table – East Viaduct

Support Location	Pile Type	Design Load	Nominal Resistance		Cut-off Elevation	Steel Casing Specified Pile Tip Elevations	Design Tip Elevations	Specified Tip Elevations
			Compression	Tension				
Abut 1E	6' CIDH	N/A	1,400 kips	0 kips	4 ft	N/A	-46 ft(1) -86 ft (3)	-86 (3)
Bent 2E	6' CIDH	N/A	2,960 kips	0 kips	4 ft	N/A	-84ft(1), -86 ft(3)	-86 (3)
Bent 3E	6' CIDH	N/A	3,040 kips	0 kips	4 ft	N/A	-84 ft(1), -86 ft(3)	-86 (3)
Bent 4E	8' CIDH	N/A	2,930 kips	0 kips	-8 ft	N/A	-78 ft(1), -98 ft(3)	-98 (3)
Bent 5E	8' CIDH	N/A	3,500 kips	0 kips	-5 ft	N/A	-83 ft(1), -98 ft(3)	-98 (3)
Bent 6	9' CIDH	N/A	6,850 kips	0 kips	-2 ft	-22 ft	-92 ft(1), -117 ft(3)	-117 (2)
Bent 7	9' CIDH	N/A	12,400 kips	0 kips	0 ft	-20 ft	-150 ft(1), -115 ft(3)	-150 (1)
Bent 8	9' CIDH	N/A	12,150 kips	0 kips	4 ft	-16 ft	-146 ft(1), -111 ft(3)	-146 (1)
Bent 9	9' CIDH	N/A	8,760 kips	0 kips	4 ft	-16 ft	-126 ft(1), -111 ft(3)	-126 (1)
Bent 10E	8' CIDH	N/A	3,460 kips	0 kips	4 ft	N/A	-72 ft(1), -86 ft(3)	-86 ft(3)
Bent 11E	8' CIDH	N/A	3,260 kips	0 kips	4 ft	N/A	-68 ft(1), -86 ft(3)	-86 ft(3)
Bent 12E	8' CIDH	N/A	2,750 kips	0 kips	4 ft	N/A	-62 ft(1), -86 ft(3)	-86 ft(3)
Bent 13E	8' CIDH	N/A	3,280 kips	0 kips	4 ft	N/A	-68 ft(1), -86 ft(3)	-86 ft(3)
Bent 14E	8' CIDH	N/A	3,280 kips	0 kips	4 ft	N/A	-68 ft(1), -86 ft(3)	-86 ft(3)
Bent 15E	8' CIDH	N/A	2,790 kips	0 kips	4 ft	N/A	-62 ft(1), -86 ft(3)	-86 ft(3)
Bent 16E	6' CIDH	N/A	3,280 kips	0 kips	4 ft	N/A	-84 ft(1), -86 ft(3)	-86 ft(3)
Bent 17E	6' CIDH	N/A	3,290 kips	0 kips	4 ft	N/A	-84 ft(1), -86 ft(3)	-86 ft(3)
Bent 18E	6' CIDH	N/A	2,550 kips	0 kips	4 ft	N/A	-71 ft(1), -86 ft(3)	-86 ft(3)
Bent 19E	6' CIDH	N/A	3,010 kips	0 kips	4 ft	N/A	-80 ft(1), -86 ft(3)	-86 ft(3)
Bent 20E	6' CIDH	N/A	2,720 kips	0 kips	4 ft	N/A	-76 ft(1), -86 ft(3)	-86 ft(3)
Bent 21E	6' CIDH	N/A	3,020 kips	0 kips	2 ft	N/A	-82 ft(1), -86 ft(3)	-86 ft(3)
Abut 22E	6' CIDH	782 kips	1,570 kips	0 kips	16 ft	N/A	-40 ft(1) -45 (ft) (3)	-45 ft(3)

Notes:
1. Design tip elevation is controlled by the following demands: (1) Compression, (2) Tension, (3) Lateral, and (4) Liquefaction
2. Design tip elevations for Bents 6, 7, 8, and 9 were estimated based on permanent casing for the upper 20 feet.
3. Specified tip elevation shall not be raised unless allowed by the Geotechnical Engineer.
4. Per direction of Structural Engineer, pile lengths based on lateral demand is considered by determining the depth of zero deflection multiplied by a factor of safety of 1.5.

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Kleinfelder has extensive experience with both driven and drilled pile foundation design in similar soil conditions within the City of Stockton and Port of Stockton area. Kleinfelder has typically based pile design in similar soil conditions assuming allowable side friction capacity of 1,100 to 1,500 pounds per square foot (psf) for soils within the top 70 feet or so of soil strata. This assumption has been backed up with data from at least 12 pile load tests within the Stockton area. Attached in Appendix H is a description of the design information obtained from one of these tests. Results from axial pile capacity analysis for this investigation indicates a design "allowable" side friction capacity for axial compression design of approximately 1,100 to 1,200 psf within the top 70 feet of pile embedment, which is consistent with what we have historically found. With piles being extended deeper at most bent locations to accommodate lateral capacity demands, the specified pile tip elevations are sufficient to accommodate axial compression as well as potential downdrag loads.

9.2.2 Lateral Capacity

Resistance to lateral loads will be provided by the resistance of the soil against the piles and by the bending stiffness of the piles themselves. The computer software program LPILE (version 5), developed by ENSOFT, Inc., can be used to estimate the ultimate (nominal) lateral resistance capacity of piles. Generalized soil profiles (13 total soil profiles/models) and soil parameters for our L-Pile analysis are presented in Appendix F. Input files, deflection graphs and moment graphs for our L-pile analysis; utilizing shear and moment demands provided by the Designer and Structural Engineer, are presented in Appendix I. Results from these analyses are also summarized in Appendix I. Approximate depths to zero lateral deflection and moments as estimated from L-Pile analysis are summarized in the following table.

Table 9.3
Approximate Depths to Zero Deflection and Moment

Applicable Substructures	Pile Type	Approximate Depth to Zero Deflection (feet)	Approximate Depth to Zero Moment (feet)
Abutment 1, Bents 2 and 3	6' CIDH	40 to 55	55 to 70
Bents 4 and 5	8' CIDH	60 to 65	70 to 80
Bents 6 through 9	9' CIDH	40 to 75	80 to 95
Bents 10 though 15	8' CIDH	45 to 65	65 to 80
Bents 16 though 21	6' CIDH	40 to 60	55 to 80
Abutment 22	6' CIDH	40	80

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Per direction of Structural Engineer, pile lengths based on lateral demand were considered by determining the depth of zero deflection multiplied by a factor of safety of 1.5. The pile lengths presented in Tables 9.1 and 9.2 reflect this factor of safety.

The lateral resistance of a single pile within a pile group should be reduced by a P-Multiplier in Table 9.4 to account for potential group action effects (in direction of load only). Loading direction and pile spacing are as defined in Table 10.7.2.4, page 10-84A of California Amendments to AASHTO LRFD Bridge Design Specifications – Fourth Edition.

Table 9.4
P- Multipliers for a Single Pile within a Pile Group
(In Direction of Loading Only)

Pile Center to Center Spacing (in the direction of loading)	Row 1 (P-multipliers, Pm)	Row 2 (P-multipliers, Pm)	Row 3 and higher (P-multipliers, Pm)
2.0B	0.60	0.35	0.25
3.0B	0.75	0.55	0.40
4.0B	0.88	0.70	0.55
4.5B	0.94	0.78	0.63
5.0B	1.0	0.85	0.70
6.0B	1.0	0.93	0.80
7.0B	1.0	1.0	0.90
8.0B	1.0	1.0	1.0

Where B = width/diameter of the pile

If the loading direction for a single row of piles is perpendicular to the row (bottom detail in Figure 1 of page 10-84, AASHTO LRFD Bridge Design Specifications), the P-multipliers shown in Table 9.5 may be used:

Table 9.5
P- Multipliers for a Single Pile within a Single- Row Pile Group

Pile Center to Center Spacing	P-multipliers, Pm
2.0B	0.8
3.0B	0.9
4.0B	1.0

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9.2.3 Negative Skin Friction (Downdrag) on Piles

When piles are installed through a soil deposit which undergoes settlement, the resulting relative downward movement of the soil around the piles induces "downdrag"/negative shaft resistance forces on the piles. The amount of relative settlement between the soils and pile that is necessary to mobilize negative shaft resistance is about 0.4 and 0.5 inches. For this project site, at five of the thirteen CPT exploration locations liquefaction settlement was estimated to be less than ½ inch. Also as mentioned in Section 7.3, potentially liquefiable soil layers are discontinuous throughout the site. Because of this information and because at most bent locations piles are deepened to accommodate lateral load demands, it is our opinion that the recommended pile lengths presented in Section 9.2.1 of this report are sufficiently conservative to accommodate the potential for downdrag loading without further increase of pile lengths.

9.2.4 Construction Considerations

Installation of CIDH piles should meet the general requirements presented in Section 49 of Caltrans Standard Specifications (2006) and/or procedures outlined in the publication Drilled Shafts: Construction Procedures and LRFD Design Methods, dated May 2010 (Publication No. FHWA-NHI-10-016). Although most of the subsurface soils are characterized aS fine grained, cohesive, and stiff with "good" stand up potential, some layers or potentially caving sandy soils were encountered during our field explorations. Groundwater depth at the project site is anticipated to be relatively shallow at a depth of about 10 to 15 feet below existing grade.

BNSF has expressed their concern about the effects of construction of CIDH piles on the performance of its tracks in the close proximity of Bents 6, 7, 8 and 9. To address this concern it is recommended that the top 60 feet of holes advanced for CIDH foundations in this area use temporary casing, with the top 20 feet being permanent casing. This should provide adequate support for the stability of the drilled holes with a minimum distance of 25 feet from the railroad tracks. The outside diameter of the casing should not be less than the diameter of the piles, with the temporary casing vibrated into place. The use of impact hammers to install (drive) the temporary casing into place should be prohibited in order to prevent potential BNSF rail track

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displacement. During removal, the casing should be slowly pulled from the excavation as concrete is being placed, and the bottom of the casing should be kept at least 5 feet below the top of the concrete where possible.

An approved drilling fluid will be required to maintain the sidewall integrity of the CIDH shafts below the cased portions. If slurry assisted drilling is used, steel reinforcement and concrete should be placed immediately upon drilling completion of each CIDH excavation to reduce the quantity of suspended soil particles that may settle to the bottom of the hole. The depth of all CIDH excavations should be checked several times before concrete placement to assure excessive sedimentation has not occurred.

Per correspondence with Mr. Robert Small of Malcolm Drilling, the use of dewatering wells is not needed for construction of the proposed drilled shafts. During concrete placement for CIDH installation, the volume of water/drilling fluid inside individual shafts will be displaced and pumped to onsite storage tanks. This water/drilling fluid can be reused from CIDH pile to CIDH pile, however it will need to be disposed of. This may constitute pumping the water (if approved) into city storm water discharge drains or into water trucks and used as dust control. By the end of the project, potentially six to eight, 21,000 gallon storage tanks filled with water or drilling fluid may need disposed of.

The CIDH pile construction should be observed by the geotechnical consultant during drilling to evaluate if the piles have been extended to the recommended depth. Drilled holes should be checked for alignment and plumbness during installation. The amount of acceptable misalignment is approximately 3 inches from the planned location. It is usually acceptable for a pile to be out-of-plumb one percent of the pile depth.

The integrity of CIDH piles should be evaluated after installation. We recommend that access tubes be provided as part of the reinforcing cage prior to the placement of concrete. The access tube should consist of 2-inch internal diameter or larger, plastic (Schedule 40 PVC) pipes placed longitudinally in the drilled shaft by attachment to the reinforcing cage. The lower end of the tubes should be plugged to keep out concrete. The tubes should be filled with water to stabilize the temperature of the tube, keeping it from debonding from the concrete. Acoustic tests, if deemed appropriate, should be performed within a few days of casting the concrete.

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The proper concrete mix design for CIDH piles varies with the design stress intensity, the anticipated concrete placement procedures, and the spacing of the reinforcement. The project specifications should provide provisions as to concrete slump, temperature, air content, homogeneity, job-site conditions, and method of placement. In order to properly develop the design skin friction, current practice calls for concrete to have a 4- to 6-inch slump during placement "in the dry" and no pulling of casing, a 5- to 7-inch slump if casing is to be pulled or the pier shaft is heavily reinforced, and a 7- to 9-inch slump with 3/4 inch maximum size aggregate if concrete is to be placed by tremie or pumping methods.

9.3 APPROACH EMBANKMENTS

Embankments are proposed at both ends of the elevated structure. The western embankment, containing the eastbound on-ramp and westbound off-ramp, will extend from Navy Drive to the viaduct structure just south of the BNSF railway. This embankment will vary in width from 75 to 110 feet and be retained by three MSE walls. The MSE walls are approximately 555 feet (retaining wall 1), 790 feet (retaining wall 2) and 83 feet (retaining wall 3) in length and range in height from about 0 feet at navy drive (west) to 42 feet at the viaduct interface (east). A standard type 1 wall (retaining wall 4) ranging in height from 4 to 6 feet and approximately 560 feet long will be located on the south side of Navy drive.

The eastern embankment for the eastbound and westbound mainline from just east of Fresno Avenue to the Garfield Street Overhead will include sloped embankments. It will measure about 170 feet wide and vary in height from about 0 to 30 feet above the existing ground level, with its highest point just east of Fresno Avenue.

9.3.1 Settlement Waiting Period

Review of boring logs indicates that the project site is generally underlain by soft to medium-stiff lean clay/silt, interbedded with loose to medium-dense silty sand to a depth of about 40 to 60 feet below existing grades. Below this layer are layers of very stiff to hard silt/lean clay and/or dense to very-dense sandy soils to the maximum depths explored. Groundwater was encountered at a depth of about 5 to 10 feet at the western embankment location below existing ground surface. A settlement analysis for the western approach embankment is presented in Appendix J. The required time for 90 percent consolidation varies was estimated in this analysis to be between 3 to 4 months. However, based on experience in the project area, we anticipate the actual time for 90 percent consolidation to be between 1 and 2 months.

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Settlement magnitudes and rates for the eastern approach embankment are anticipated to be significantly less than the western approach embankment as the water table in this area is greater.

Bridge abutment areas are settlement-sensitive. Caltrans practices are such that the remaining total settlement of the abutment embankment should not exceed 0.5 inches prior to the installation of abutments, wingwalls, and pile foundations. We recommend that the actual rate and magnitude of settlement within bridge abutment areas be monitored in the field. Prior to construction, vibrating wire settlement plates and piezometers should be considered for installation. Surface monuments, constructed in accordance with Caltrans Standard Plan A74 or equivalent, should be installed in a timely manner upon completion of the fill placement. Surface monuments should be placed at the south abutment. The actual locations of surface monuments should be determined during grading under the direction of the Geotechnical Engineer.

9.3.2 Ground Improvement Alternatives

Various ground improvement techniques have been used to address long-term consolidation settlement issues for roadway embankment over soft clay. For the proposed project, we considered three alternatives as below:

- Preloading/One or Two-Stage construction
- Prefabricated Vertical (PV) drains
- Geopier-supported embankments

For the PV drains alternative, we considered 4-inch wide PV drains installed to depths of about 50 to 60 feet below existing grade at center-to-center spacing of 5 to 6 feet. For the geopier-supported embankment alternative, we considered 30-inch diameter geopiers installed to a depth of about 30 feet below the ground surface at a center-to-center spacing of 5 to 7 feet. The three ground improvement alternatives are compared in terms of the magnitude and rate of consolidation settlements and approximate costs, as summarized in the table below. Calculations for these alternatives are presented in Appendix J.

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Table 9.6
Ground Improvement Alternatives

Alternative No.	Ground Improvement Technique	Consolidation Settlement [1] (inches)	Time Required for 90 Percent Degree of Consolidation [2] (months)	Approximate Ground Improvement Costs [3] ($)
1	Preloading/One- or Two-stage construction	10-16	3-4	N/A
2	PV drains	10-16	<1	305,000
3	Geopier-supported embankment	4-7	1-2	2,160,000

Notes:
1. Estimated for a 45-foot-high approach embankment (MSE walls)
2. After the end of construction
3. Current cost estimation as of this writing

Depending on the amount of differential settlements expected, two typical construction techniques are usually used in the construction of MSE walls – one-stage construction and two-stage construction. One-stage construction is usually used in areas where the anticipated differential settlements are within the tolerable limit for the precast panel system (i.e., less than 1 percent or 1 foot of differential settlement along a 100-foot wall length). The two-stage MSE wall system involves initial construction of a reinforced earth wire-faced wall with steel soil reinforcements to the desired height of the proposed embankment. The flexible wire facings are able to tolerate significant amounts of settlement without any adverse impact on the structural integrity and performance of the final wall. Following the completion of the consolidation of the foundation soils, precast concrete facing panels are connected to the wire facings/soil reinforcements with an adjustable connection system. After complétion of the connection for each course of precast concrete facing panels, the space between the wire facings and concrete panels is then backfilled with free-flowing gravel to complete the structure.

As indicated in the table above, we anticipate that consolidation settlement at the project site will be on the order of 10 to 16 inches, corresponding to a maximum differential settlement of less than 1 percent. Therefore, it is our professional opinion that the one-stage construction technique should be appropriate for the proposed approach embankments (MSE walls). If the two-stage construction technique is considered, on a preliminary basis we anticipate that its cost will be approximately 40 to 50 percent higher than for one-stage construction, and in this case, ground improvement with geopiers might become more cost-effective because it allows one-stage construction of MSE walls.

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Construction of western and eastern approach embankments should be appropriately staged for foundation soils to achieve sufficient strength to support the overlying embankment loadings. Time required for 90 percent degree of consolidation for Alternative No. 1 – Preloading is estimated to be about 3 to 4 months. Parikh estimated in their preliminary foundation report dated March 8, 2010 that about 18 months will be required for the total consolidation settlement to occur. Our experience within the project area, however, indicates that the consolidation rate of the underlying soils may be much faster than anticipated by Parikh.

In July 2009, we worked for US Gypsum Company on a project in which a test pad was constructed at the Port of Stockton which is about 2 miles west of the project site. For the US Gypsum project, the test pad was constructed in two stages with a total height of about 21 feet above existing site grade. Foundations soils at the site included soft organic clay and silt. Groundwater was encountered less than 5 feet below the ground surface. Settlements and pore water pressures were monitored for about 3 months after the completion of fill construction. Results of our field monitoring indicate that the total settlement was about 6 to 8 inches, and settlement leveled off within about 1 month.

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A more recent project in which settlement magnitudes and rates were measured was during construction of north and south approach embankments for the Port of Stockton Expressway/BNSF Grade Separation project at the Port of Stockton, located approximately 2 miles southwest of the subject project site. Construction of approach embankments for the BNSF grade separation project was performed from August to November 2011 to a maximum embankment height of 42 feet. Because near surface native soils were identified as being organic, soft and compressible, the settlement time for primary consolidation to occur was a concern along with the magnitude of total settlement at each embankment location. Also, because of the soft nature of the native soils and shallow depth to groundwater (approximately 5 feet), the rate at which fill was placed and its effect on subsurface pore pressures were a concern. However, instrumentation of the embankments and native foundation soil using vibrating wire piezometers, vibrating wire settlement cells, and settlement cells indicated the predicted soil behavior by others (magnitude and settlement rate) was very conservative. Piezometer data indicated excess pore pressures dissipated quickly after fill placement, in most cases over night, allowing fill to be placed nearly continuously without causing significant decreases in foundation soil strength. Additionally, maximum total settlement of approach embankments during and after construction ranged between approximately 5 to 10 inches.

Settlement of embankments sometimes causes damage to nearby properties and utilities. A conservative assessment of embankment and MSE wall loading, modeled as a finite width (110 feet wide) and infinite length strip load, indicates that assuming a maximum total potential settlement of 10 to 16 inches inches, the potential for settlement induced by the embankment reduces the further away from the crest of the embankment and/or toe of MSE Wall as follows:

Table 9.7
Estimated Induced Settlement Reduction

Distance from Crest of Embankment/Toe of MSE Walls	Induced Settlement (Based on 10 inches maximum settlement)	Induced Settlement (Based on 16 inches maximum settlement)
10 feet	5 inches	8 inches
25 feet	2 ½ inches	4 inches
50 feet	1 inch	1 ½ inches
75 feet	½ inch	¾ inch
100 feet	--	¼ inch

We recommend that mitigation measures for embankment/MSE wall load induced settlement of nearby structures and utilities be based on a maximum settlement of 10 inches. A review of

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project layout plans indicate that existing structures, left after embankment and MSE Wall construction along the proposed project alignment, will be at least 100 feet away from the crest of embankments/toe of MSE walls. Therefore embankment/MSE wall loading induced settlement for these structures are not a significant concern. Where utilities are located within the distance from the crest of proposed embankments and or toe of MSE walls indicated in Table 9.7, consideration should be given to providing flexible connections that compensate for the anticipated induced settlement. Once settlement is complete, these flexible connections may be replaced with fixed/rigid connections.

It can be concluded that the preloading/one-stage construction alternative is the most cost-effective technique for the proposed project. We understand that the construction schedule may allow for significant time for the construction of approach embankments and application of ground improvement techniques. Therefore, we recommend that preloading/one-stage construction be considered for the proposed project to mitigate the potential consolidation settlements under approach embankment loadings.

9.3.3 Approach Embankment Construction Considerations

Construction of the approach embankment should conform to the general guidelines of Section 19 of Caltrans Standard Specifications and be constructed in layers of uniform thickness not exceeding 8 inches per lift. Each layer should be compacted to at least 90 percent relative compaction. Relative compaction of not less than 95 percent should be obtained for a minimum depth of ½ foot below the grading plane for the width between embankment edges. Relative compaction should also be obtained for a minimum depth of 2½ feet below finished grade for the traveled way plus 3 feet on each side. Except for the outer 5 feet measured horizontally from the embankment side slopes, the full width of embankment within 150 feet of bridge abutments should be compacted to a relative compaction of not less than 95 percent.

9.4 MSE WALLS

As discussed previously, MSE walls are planned for the western approach embankment. The proposed MSE walls were analyzed for external and global stability in accordance with Caltrans guidelines (Caltrans Bridge Design Specifications, Section 5, Retaining Walls 2004). The results of our conservative analysis, presented in Appendix K, indicates the walls will be

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satisfactory with regard to external and global stability provided the following general design parameters summarized below in Table 9.8 are used. Traffic loads and temporary construction loads should be considered in the design of MSE reinforced walls.

Table 9.8
Design Parameters for MSE Walls

Zones	Moist Unit Weight (pcf)	Friction Angle (degrees)	Cohesion (psf)
Reinforced Soil	125	35	0
Retained Soil	125	34	0
Foundation Soil	120	30	0

Based on a minimum wall embedment depth of 2 feet, and a reinforcement length equal to 70 percent of the Design Wall Height, the following allowable bearing capacities are provided in Table 9.9 for MSE wall design. Factors of safety for bearing capacity all were greater than 3.5 (factor of safety of 2.0 required) indicating that wall reinforcement lengths equal to the minimum requirement of 70 percent of Design Wall Height will be satisfactory.

Table 9.9
Allowable Bearing Capacity for MSE Walls

MSE Design Wall Height	Allowable Bearing Capacity
16 feet	2.5 tsf
22 feet	3.5 tsf
28 feet	4.6 tsf
30 feet	5.0 tsf
34 feet	5.6 tsf
36 feet	6.0 tsf
38 feet	6.3 tsf
42 feet	7.0 tsf

Backfill for the reinforced soil mass should be free from organic material or other unsuitable material, and should conform to the gradation and property requirements in Table 9.10.

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Table 9.10
Requirements for Reinforced Soils

Gradation Requirements		Property Requirements
Sieve Size	Percentage Passing	
2 inches	100	pH of 4.5 to 9, sand equivalent of 30 or more, and plasticity index of 10 or less
#4	50 - 80	
#40	0 - 30	
#200	0 - 15	

9.5 GABION BASKET GRAVITY WALL

As mentioned previously, the existing "LKQ" property basin located within the approximate footprint of proposed Bents 4 and 5 of the eastbound viaduct structure will be partially filled to be used as a temporary staging area for bridge construction. An 11 feet high (exposed height) gabion basket gravity wall is proposed to be placed to retain the crushed drain rock which will be used to partially fill the drainage basin. A stability analysis was performed for the proposed gabion basket gravity wall using a spreadsheet developed by a leading gabion basket manufacturer (Terra Aqua Inc.), and is provided in Appendix L. The following recommendations are provided for gabion basket gravity wall design.

1.) Standard Details for Gabion Baskets should be prepared in accordance with Caltrans 2006 Standard Plan Sheets D100A and D100B

2.) At a depth of 11 feet, foundation soil will likely be in a wet, pumping condition. To allow for a firm foundation for wall construction, place a 12 inch leveling layer of granular stone fill wrapped in a Mirafi 600x geotextile fabric or equivalent at the base of the gabion wall.

3.) To provide additional factor of safety against sliding, place the gabion gravity retaining wall on a 6 degree batter toward the retained slope.

4.) The depth of "scour" for the basin is unknown. For initial design, assume a minimum 2 feet foundation embedment depth to accommodate "scour."

5.) Retaining wall backfill should be free draining (drain rock), and filter fabric should be placed between the gabion wall and the backfill soil interface. This will result in the drain rock being thoroughly encapsulated within the filter fabric. Drainrock should consist of clean, durable stone having 100 percent passing the 1-inch sieve and zero

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percent passing the No. 4 sieve. Synthetic filter fabric should conform to the requirement in Section 88 "Engineering Fabrics" of the Caltrans Standard Specifications. The upper 12 inches of engineered backfill above the drainrock backfill should consist of native soil to reduce surface drainage into the drainrock backfill.

6.) Recommended Stone Fill Gradation for Gabion Baskets is as follows:

Table 9.11
Stone fill Gradation for Gabion Basket Wall

Galvanized/Galfan Gabion 8 x 10 cm	4 inches to 8 inches

Specific gravity for stone fill should be no less than 2.5.

7.) To provide the maximum resistance to soil forces, the gabion baskets should be placed with the length dimension of the gabion basket unit running from the back of the gabion wall to the front face of the gabion wall.

8.) The wall configuration may be designed as shown on the gabion basket gravity wall calculation spreadsheet presented in Appendix L. A conceptual wall layout is also provided on the spreadsheet where gabion width and heights are indicated. In summary, the following wall dimensions are recommended:

Table 9.12
Recommended Gabion Basket Wall Dimensions

Wall Row	Width (ft)	Height (ft)	Offset from Toe (ft)
1	9.0	3.0	0.0
2	7.5	3.0	1.5
3	6.0	3.0	3.0
4	4.5	3.0	4.5
5	3.0	3.0	6.0

9.6 TYPE 1 RETAINING WALLS

Type 1 retaining walls (RW 4 and RW 5) may be designed per criteria presented on sheets B3-1 and B3-8 of the Caltrans standard plans published in 2006. Allowable bearing pressure

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for the proposed retaining walls will be at least equivalent to the design toe pressures presented on Caltrans Standard Plan Sheet B3-1.

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10.0 LIMITATIONS

Please be advised that we are performing a professional service and that our conclusions and recommendations are professional opinions only. All work done and all recommendations made are in accordance with generally accepted Geotechnical engineering principles and practices. No warranty, expressed or implied, of merchantability or fitness, is made or intended in connection with our work.

Foundation recommendations contained in this report were developed based on limited information regarding the subsurface soil conditions obtained from our field exploratory borings, CPTs and geoprobes, previous investigations at the project site, and our experience in the vicinity of the project site. It is possible that soil or groundwater conditions could vary between or beyond the points explored. If soil or groundwater conditions are encountered during construction that differ from those described herein, the client is responsible for ensuring that Kleinfelder is notified immediately so that we may reevaluate the recommendations of this report. If the scope of the proposed construction, including the estimated structure loads and the design depths or locations of the foundations, changes from that described in this report, the conclusions and recommendations contained in this report are not considered valid unless the changes are reviewed and the conclusions of this report are modified or approved in writing by Kleinfelder.

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11.0 REFERENCES

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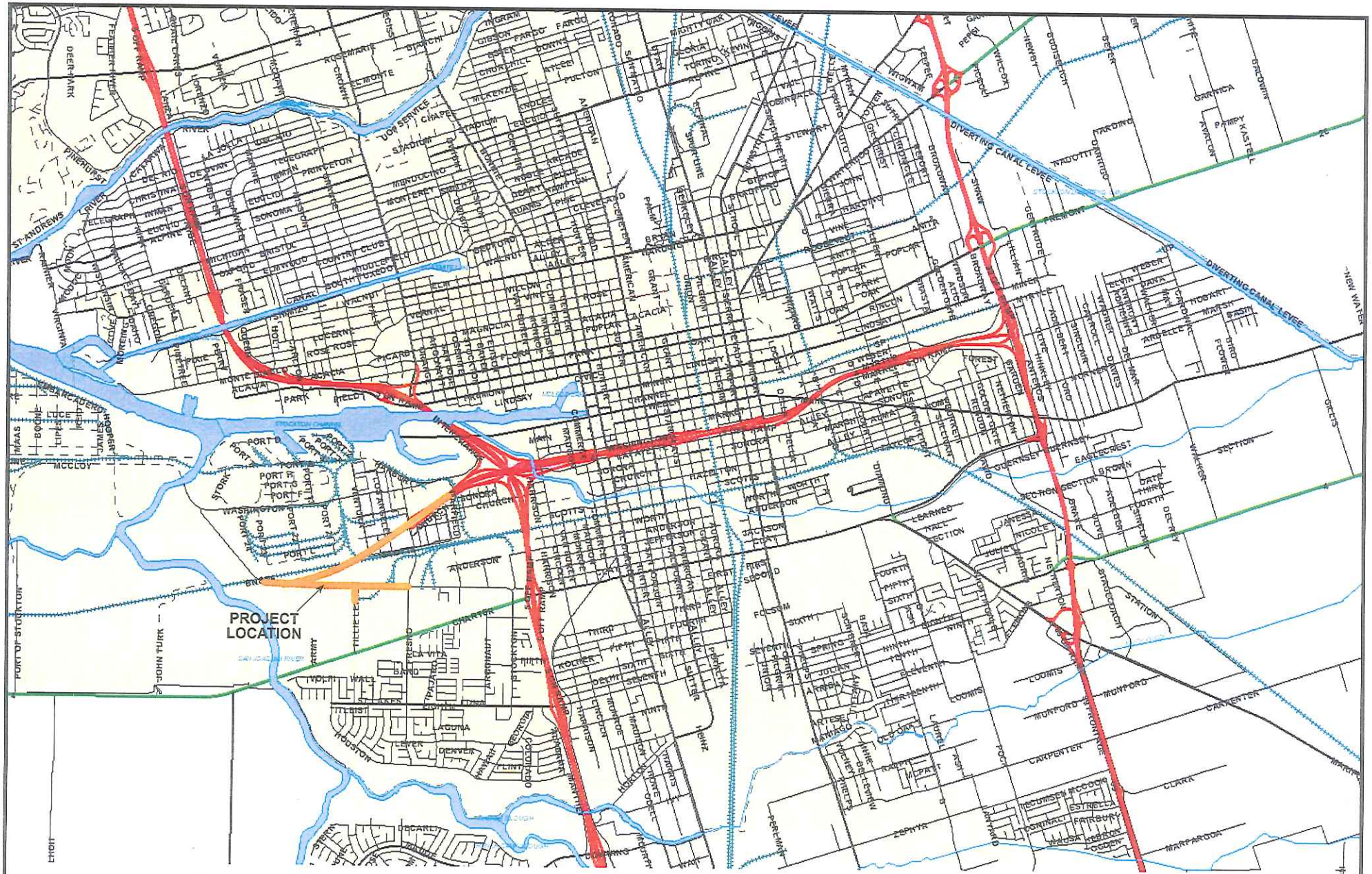

NOT TO SCALE

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Project Number: 111904.G01	
Graphic Date: 7/28/10	
Graphic By: G. GOMEZ	
Checked By:Y.L.	
File Name: VMAP.fh11	

SITE VICINITY MAP
ROUTE 4 CROSSTOWN FREEWAY EXTENSION
SAN JOAQUIN COUNTY, CALIFORNIA

Plate

1

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Port Rd K
Port Rd 22
Port Rd L
Port Rd M
Port Rd 21
W Senora St
S Fresno Ave
W Hazelton Ave
S Ventura Ave
W Scotts Ave
S Los Angeles Ave
Church St
S Merced Ave
Navy Dr
Miller Lewis Dr

R-11-013
R-11-012
CPT-11-011
R-11-001
CPT-11-001
R-11-002
CPT-09-001
CPT-09-002
A-09-001
CPT-11-002
R-11-003
A-09-002
CPT-11-010
R-11-004
CPT-11-003
R-11-005
CPT-11-004
R-11-006
CPT-11-006
CPT-11-005
R-11-007
CPT-11-007
R-11-008
CPT-11-008
R-11-009
CPT-11-009
R-11-010

O-11-01
O-11-02
O-11-03
O-11-04
O-11-05

R-11-013 DENOTES NUMBER AND APPROXIMATE LOCATIONS OF BORINGS PERFORMED FOR THIS INVESTIGATION

O-11-01 DENOTES NUMBER AND APPROXIMATE LOCATIONS OF GEOPROPES ADVANCED FOR DEBCO DSI INVESTIGATION

A-09-001 DENOTES NUMBER AND APPROXIMATE LOCATION OF FIELD EXPLORATIONS PERFORMED BY PARIKH

CPT-11-010 DENOTES NUMBERS AND APPROXIMATE LOCATIONS OF CPT'S ADVANCED FOR THIS INVESTIGATION

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DATE PRODUCED:	5/4/12
PROJ. No.:	111904.G01
DRAWN BY:	G. GOMEZ
CHECKED BY:	C. HENDERSON
FILE NAME:	STO10D135.CAD

EXPLORATION LOCATION MAP
ROUTE 4 CROSSTOWN FREEWAY EXTENSION
SAN JOAQUIN COUNTY, CALIFORNIA

PLATE
2

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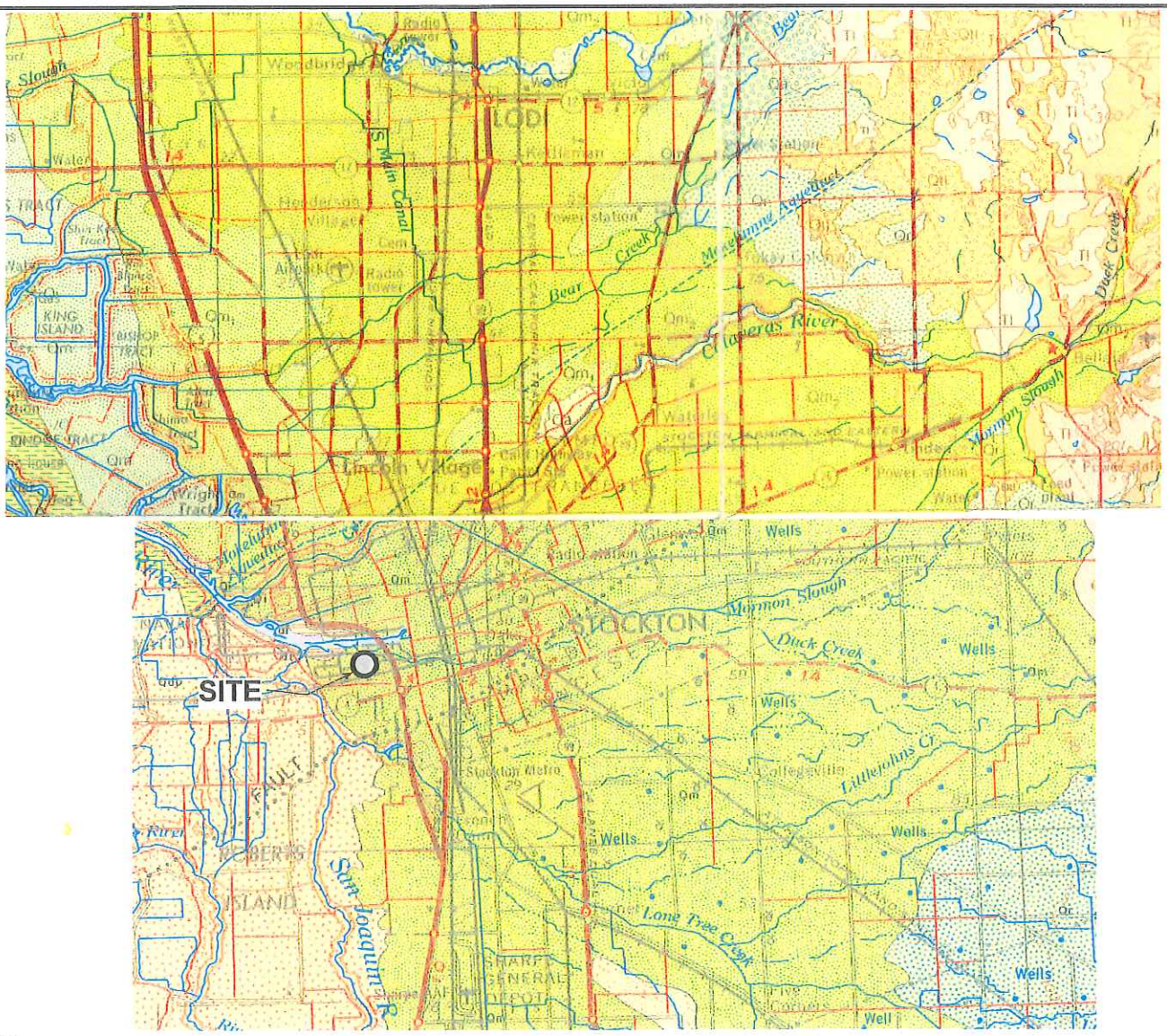

KEY

- O — Alluvium
- t — Mine and dredge tailings
- Qa — Levee and channel deposits
- Qt — Intertidal deposits *(Peaty mud)*
- Qm — Modesto Formation *(Alluvium)*
- Qm_1 / Qm_2 — Upper and lower members
- Qr — Riverbank Formation *(Alluvium)*
- Qmr — Modesto-Riverbank Formations *(Arkosic alluvium)*
- Qtl — Turlock Lake Formation *(Sand, silt, and gravel)*
- Qmg — North Merced Gravel *(Thin pediment veneer)*
- Tl — Laguna Formation *(Consolidated alluvial deposits)*
- Tm — Mehrten Formation *(Andesitic conglomerate, sandstone, and breccia)*
- Qdp — Dos Palos Alluvium

SCALE: 1:125,000

NOTES: THE FAULT SHOWN IS IDENTIFIED AS THE STOCKTON FAULT. PUBLISHED REFERENCES DO NOT IDENTIFY THIS FAULT AS ACTIVE OR POTENTIALLY ACTIVE.

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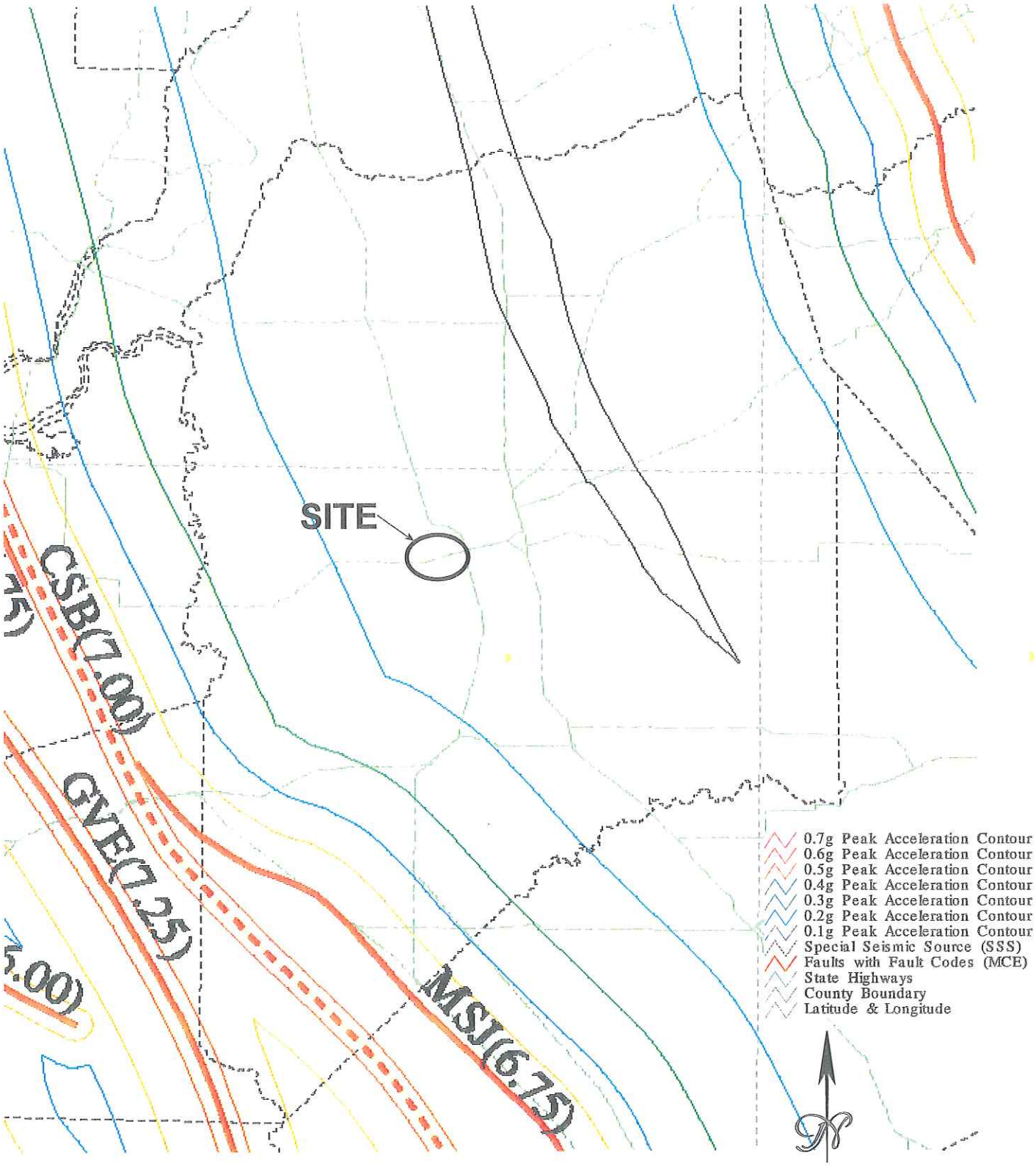
Project Number: 111904.G01	
Graphic Date: 7/29/10	
Graphic By: G. GOMEZ	
Checked By: Y.L.	
File Name: GEO.fh11	

GEOLOGIC MAP
ROUTE 4 CROSSTOWN FREEWAY EXTENSION
SAN JOAQUIN COUNTY, CALIFORNIA

Plate
3



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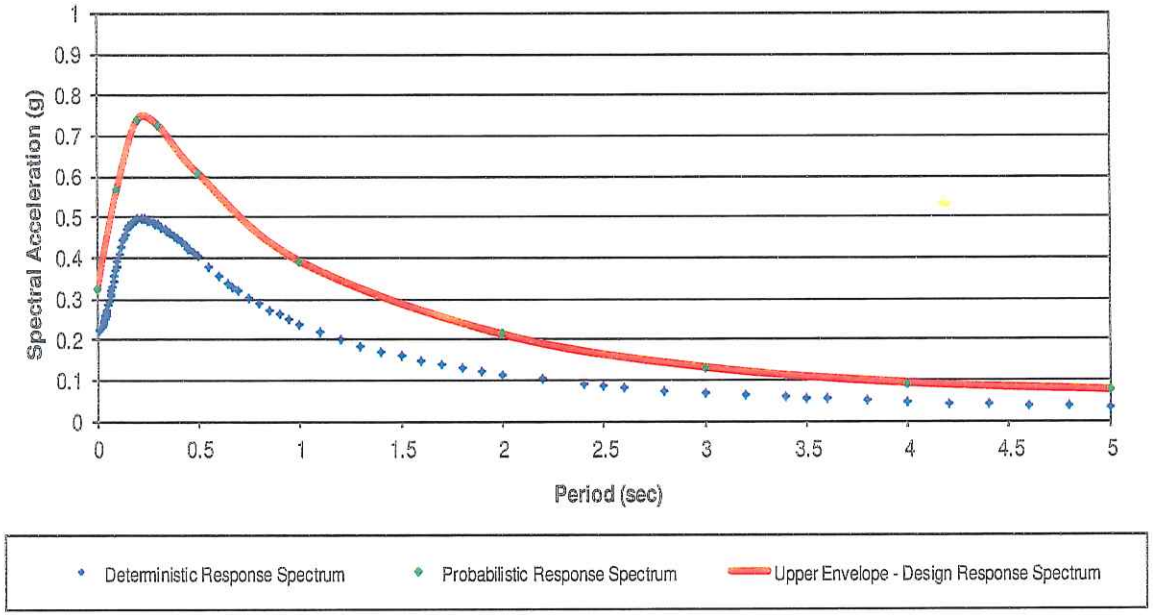
Project Number: 111904	
Graphic Date: 7/11/11	
Graphic By: G. GOMEZ	
Checked By: J.W.	
File Name: SEISMIC.fh11	

FAULT MAP
ROUTE 4 CROSSTOWN FREEWAY EXTENSION
SAN JOAQUIN COUNTY, CALIFORNIA

Plate

4



Period (sec)	Spectral Acceleration (g)	
	Probabilistic	Deterministic
0	0.3244	0.225
0.1	0.5696	0.391
0.2	0.7382	0.497
0.3	0.7281	0.483
0.5	0.6083	0.404
1	0.3919	0.238
2	0.2124	0.114
3	0.13101	0.068
4	0.09244	0.046
5	0.07534	0.034

- Caltrans Geotechnical Services Design Manual (Version 1.0, August 2009)
- ARS Online (V1.0.4, April 2011)
- Governing Deterministic Fault: Great Valley fault 7—Mw 6.7
- V_{S30} : 249 m/s

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PROJECT NO.	111904.G01
DATE:	April 29, 2011
DRAWN BY:	J. Wetenkamp
CHECKED BY:	C. Henderson

Preliminary Acceleration Response Spectrum (ARS) Spectra Comparison

Route 4 Crosstown Freeway Extension
San Joaquin, California

PLATE
5

APPENDIX A
LOGS OF BORINGS AND CONE PENETRATION TESTS PROFILES
(KLEINFELDER INVESTIGATION)

LIST OF ATTACHMENTS

The following are attached and complete this appendix.

Log of Test Borings Sheets
Cone Penetrometer Test Profiles

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Log of Test Boring Sheets

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (2010)

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	4, 5	VAR		

GEOTECHNICAL PROFESSIONAL — DATE 8/24/12

REGISTERED PROFESSIONAL ENGINEER — Carl Henderson — No. 2886 — Exp. 6/30/2013 GEOTECHNICAL — STATE OF CALIFORNIA

PLANS APPROVAL DATE

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of electronic copies of this plan sheet.

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2001 Arch-Airport Rd., Suite 100
Stockton, CA. 95206

SJCOG
555 E. Weber Avenue
Stockton, CA. 95202-2804

CEMENTATION

Description	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

BOREHOLE IDENTIFICATION

Symbol	Hole Type	Description
Size	A	Auger Boring
Size	R P	Rotary drilled boring Rotary percussion boring (air)
Size	R	Rotary drilled diamond core
Size	HD HA	Hand driven (1–inch soil tube) Hand Auger
	D	Dynamic Cone Penetration Boring
	CPT	Cone Penetration Test (ASTM D 5778–95)
	O	Other (Geoprobe)

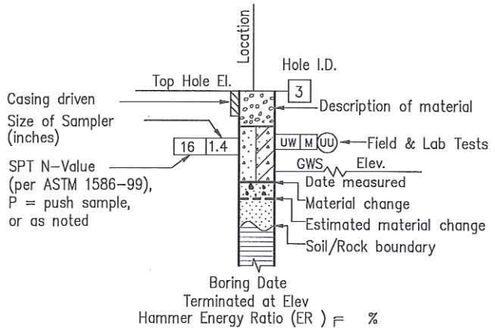
Note: Size in inches.

CONSISTENCY OF COHESIVE SOILS

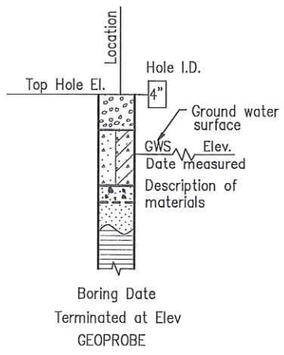
Description	Unconfined Compressive Strength (tsf)	Pocket Penetrometer Measurement (tsf)	Torvane Measurement (tsf)	Field Approximation
Very Soft	< 0.25	< 0.25	< 0.12	Easily penetrated several inches by fist
Soft	0.25 to 0.50	0.25 to 0.50	0.12 to 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 to 1.0	0.50 to 1.0	0.25 to 0.50	Penetrated several inches by thumb with moderate effort
Stiff	1 to 2	1 to 2	0.50 to 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2 to 4	2 to 4	1.0 to 2.0	Readily indented by thumbnail
Hard	> 4.0	> 4.0	> 2.0	Indented by thumbnail with difficulty

PLASTICITY OF FINE-GRAINED SOILS

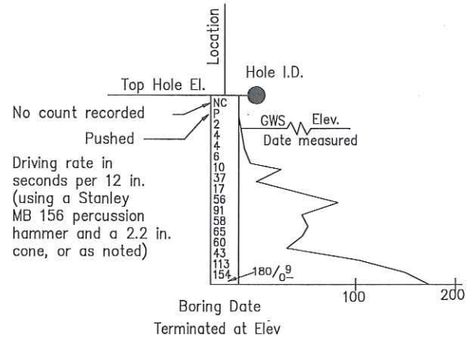
Description	Criteria
Nonplastic	A 1/8–inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.



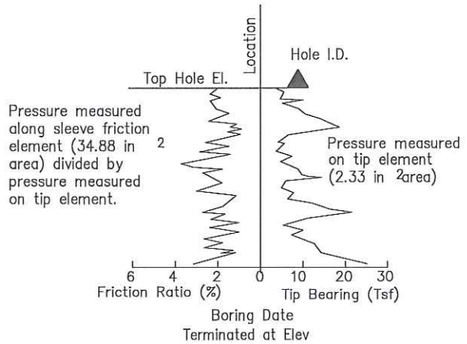
ROTARY BORING



GEOPROBE



DYNAMIC CONE PENETRATION BORING



CONE PENETRATION TEST (CPT) BORING

X DESIGN OVERSIGHT X SIGN OFF DATE	DRAWN BY CHECKED BY	G. GOMEZ Y. LI	H. PARSA FIELD INVESTIGATION BY: DATE: FEBUARY 2011		PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	H. PARSA PROJECT ENGINEER	BRIDGE NO. POST MILES

SR 4 CROSSTOWN VIADUCT

SOIL LEGEND No. 1

GS LOTB SOIL LEGEND SHEET 1 (ENGLISH) (REV. 07/16/10)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: 1454
PROJECT NUMBER & PHASE: 1000020414 CONTRACT NO.: X

FILE => $REQUEST

DISREGARD PRINTS BEARING EARLIER REVISION DATES →

REVISION DATES				SHEET	OF
5/23/12	8/16/12	8/23/12	8/24/12		

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (2010)

GROUP SYMBOLS AND NAMES

Graphic/Symbol	Group Names	Graphic/Symbol	Group Names
GW	Well-graded GRAVEL Well-graded GRAVEL with SAND	CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
GP	Poorly-graded GRAVEL Poorly-graded GRAVEL with SAND		
GW-GM	Well-graded GRAVEL with SILT Well-graded GRAVEL with SILT and SAND	CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
GW-GC	Well-graded GRAVEL with CLAY (or SILTY CLAY) Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)		
GP-GM	Poorly-graded GRAVEL with SILT Poorly-graded GRAVEL with SILT and SAND	ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
GP-GC	Poorly-graded GRAVEL with CLAY (or SILTY CLAY) Poorly-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)		
GM	SILTY GRAVEL SILTY GRAVEL with SAND	OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND		
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND	OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
SW	Well-graded SAND Well-graded SAND with GRAVEL		
SP	Poorly-graded SAND Poorly-graded SAND with GRAVEL	CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
SW-SM	Well-graded SAND with SILT Well-graded SAND with SILT and GRAVEL		
SW-SC	Well-graded SAND with CLAY (or SILTY CLAY) Well-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)	MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
SP-SM	Poorly-graded SAND with SILT Poorly-graded SAND with SILT and GRAVEL		
SP-SC	Poorly-graded SAND with CLAY (or SILTY CLAY) Poorly-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)	OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
SM	SILTY SAND SILTY SAND with GRAVEL		
SC	CLAYEY SAND CLAYEY SAND with GRAVEL	OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY ORGANIC elastic SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL		
PT	PEAT	OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND
	COBBLES COBBLES and BOULDERS BOULDERS		

FIELD AND LABORATORY TESTING

- (C) Consolidation (ASTM D 2435)
- (CL) Collapse Potential (ASTM D 5333)
- (CP) Compaction Curve (CTM 216)
- (CR) Corrosivity Testing (CTM 643, CTM 422, CTM 417)
- (CU) Consolidated Undrained Triaxial (ASTM D 4767)
- (DS) Direct Shear (ASTM D 3080)
- (EI) Expansion Index (ASTM D 4829)
- (M) Moisture Content (ASTM D 2216)
- (OC) Organic Content–% (ASTM D 2974)
- (P) Permeability (CTM 220)
- (PA) Particle Size Analysis (ASTM D 422)
- (PI) Plasticity Index (AASHTO T 90) Liquid Limit (AASHTO T 89)
- (PL) Point Load Index (ASTM D 5731)
- (PP) Pocket Penetrometer
- (R) R–Value (CTM 301)
- (SE) Sand Equivalent (CTM 217)
- (SG) Specific Gravity (AASHTO T 100)
- (SL) Shrinkage Limit (ASTM D 427)
- (SW) Swell Potential (ASTM D 4546)
- (UC) Unconfined Compression–Soil (ASTM D 2166)
Unconfined Compression–Rock (ASTM D 2938)
- (UU) Unconsolidated Undrained Triaxial (ASTM D 2850)
- (UW) Unit Weight (ASTM D 4767)

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	4, 5	VAR		

Carl Henderson 8/24/12
GEOTECHNICAL PROFESSIONAL DATE

REGISTERED PROFESSIONAL ENGINEER
Carl Henderson
No. 2886
Exp. 6/30/2013
GEOTECHNICAL
STATE OF CALIFORNIA

PLANS APPROVAL DATE

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Stockton, CA. 95202-2804

APPARENT DENSITY OF COHESIONLESS SOILS

Description	SPT N_{60} (Blows / 12 in.)
Very Loose	0 – 5
Loose	5 – 10
Medium Dense	10 – 30
Dense	30 – 50
Very Dense	Greater than 50

MOISTURE

Description	Criteria
Dry	No discernable moisture
Moist	Moisture present, but no free water
Wet	Visible free water

PERCENT OR PROPORTION OF SOILS

Description	Criteria
Trace	Particles are present but estimated to be less than 5%
Few	5% – 10%
Little	15% – 25%
Some	30% – 45%
Mostly	50% – 100%

PARTICLE SIZE

Description		Size (in.)
Boulder		Greater than 12
Cobble		3 – 12
Gravel	Coarse	3/4 – 3
	Fine	1/5 – 3/4
Sand	Coarse	1/16 – 1/5
	Medium	1/64 – 1/16
	Fine	1/300 – 1/64
Silt and Clay		Less than 1/300

X
DESIGN OVERSIGHT

X
SIGN OFF DATE

DRAWN BY: G. GOMEZ

CHECKED BY: Y. LI

H. PARSA
FIELD INVESTIGATION BY:

DATE: FEBUARY 2011

PREPARED FOR THE
STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

H. PARSA
PROJECT ENGINEER

BRIDGE NO.

POST MILES

SR 4 CROSSTOWN VIADUCT

SOIL LEGEND No. 2

GS LOTB SOIL LEGEND SHEET 2 (ENGLISH) (REV. 07/16/10)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: 1454
PROJECT NUMBER & PHASE: 1000020414
CONTRACT NO.: X

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES: 5/23/12 8/16/12 8/24/12 8/24/12

SHEET OF

FILE => $REQUEST

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	4, 5	VAR		

BENCHMARK:
The horizontal coordinates values are in the California Coordinate System (CCS 83) Zone 3, Epoch Date 2007.00, in U.S. survey feet.
The vertical control values are based on the North American Vertical Datum of 1988 (NAVD88) in U.S. survey feet, height modernization survey station, (2007.00), and GEOID03 was used for the adjustment.

Carl Henderson 8/24/12
GEOTECHNICAL PROFESSIONAL DATE

REGISTERED PROFESSIONAL ENGINEER — Carl Henderson — No. 2886 — Exp. 6/30/2013 — GEOTECHNICAL — STATE OF CALIFORNIA

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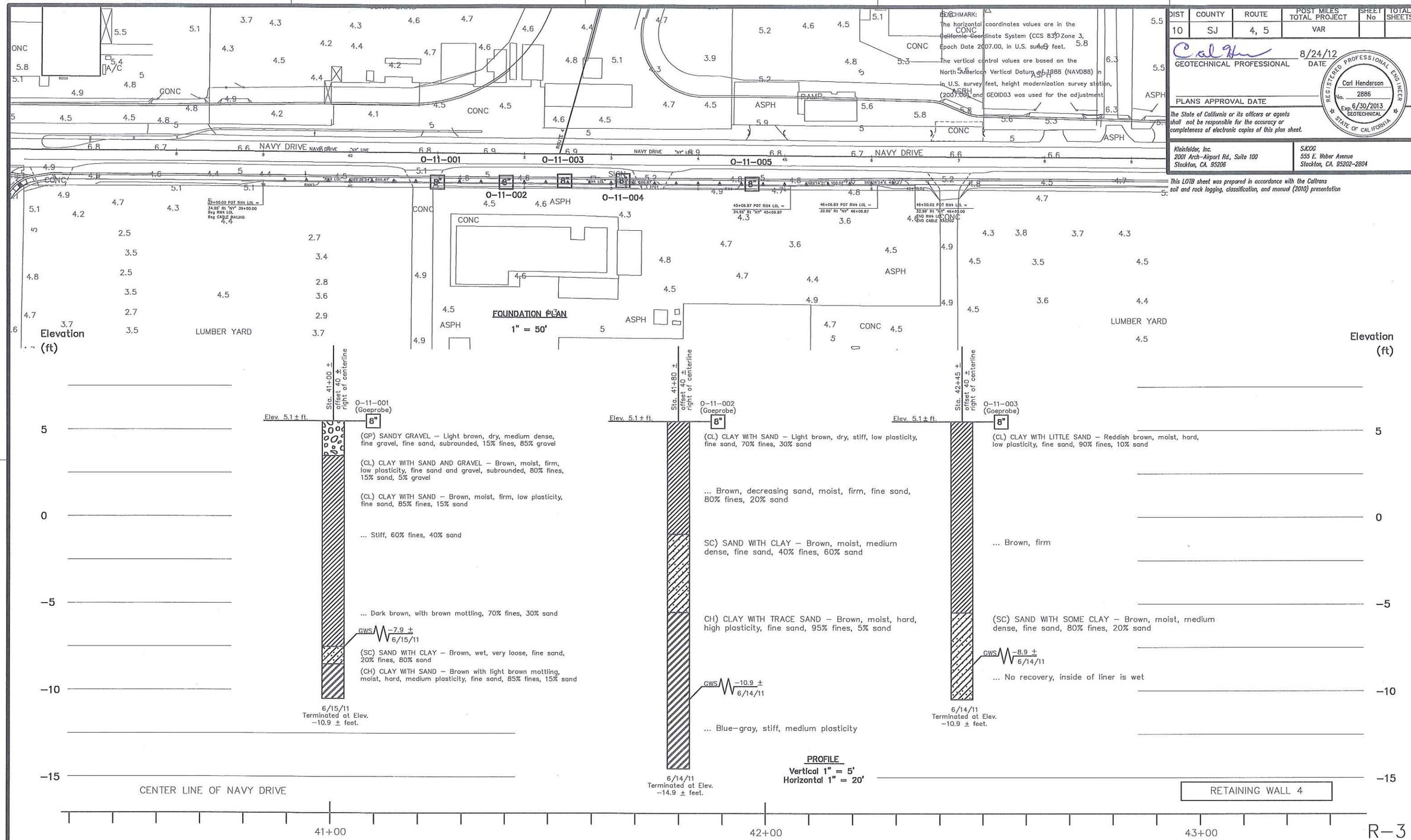

R–3

X DESIGN OVERSIGHT	DRAWN BY	G. GOMEZ	N. BERNER FIELD INVESTIGATION BY:		PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	H. PARSA PROJECT ENGINEER	BRIDGE NO.
X SIGN OFF DATE	CHECKED BY	C. HENDERSON	DATE: JUNE 2011				POST MILES

SR 4 CROSSTOWN VIADUCT

LOG OF TEST BORINGS 1 of 10

GS GEOLOGIST LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 7/16/10)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: 1454
PROJECT NUMBER & PHASE: 1000020414 CONTRACT NO.: X

DISREGARD PRINTS BEARING EARLIER REVISION DATES →

REVISION DATES				SHEET	OF
5/28/12	8/16/12	8/23/12	8/24/12		

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	4, 5	VAR		

BENCHMARK:
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Carl Henderson 8/24/12
GEOTECHNICAL PROFESSIONAL DATE

REGISTERED PROFESSIONAL ENGINEER — Carl Henderson — No. 2886 — Exp. 6/30/2013 GEOTECHNICAL — STATE OF CALIFORNIA

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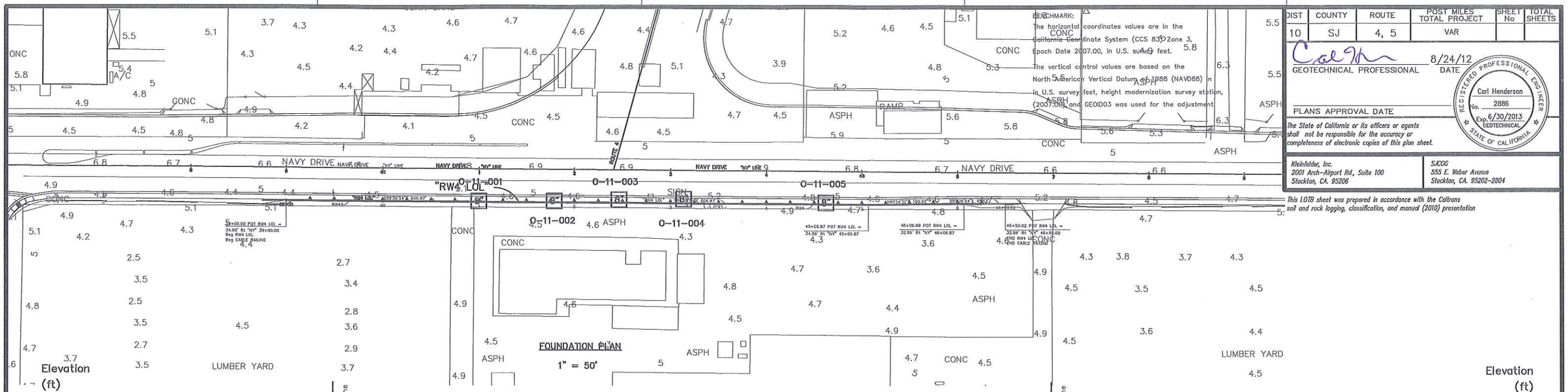

R–4

X DESIGN OVERSIGHT	DRAWN BY	G. GOMEZ	N. BERNER FIELD INVESTIGATION BY:		PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	H. PARSA PROJECT ENGINEER	BRIDGE NO.	SR 4 CROSSTOWN VIADUCT
X SIGN OFF DATE	CHECKED BY	C. HENDERSON	DATE: JUNE 2011				POST MILES	LOG OF TEST BORINGS 2 of 10

GS GEOLOGIST LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 7/16/10)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: 1454
PROJECT NUMBER & PHASE: 1000020414 CONTRACT NO.: X

DISREGARD PRINTS BEARING EARLIER REVISION DATES →

REVISION DATES				SHEET	OF
5/23/12	8/16/12	8/24/12	8/24/12		

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	4, 5	VAR		

8/24/12
GEOTECHNICAL PROFESSIONAL DATE

REGISTERED PROFESSIONAL ENGINEER
Carl Henderson
No. 2886
Exp. 6/30/2013
GEOTECHNICAL
STATE OF CALIFORNIA

PLANS APPROVAL DATE

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BENCHMARK
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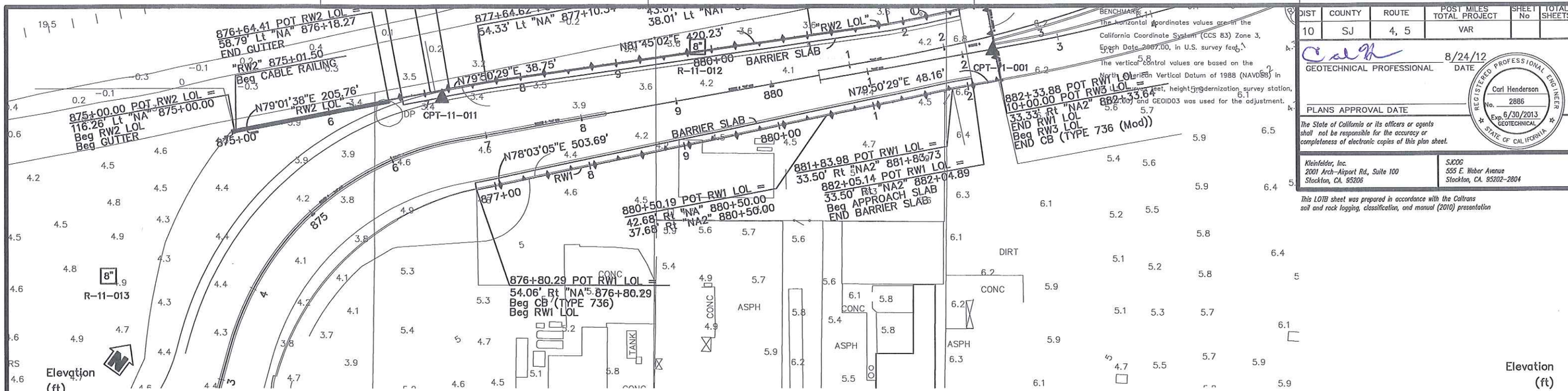
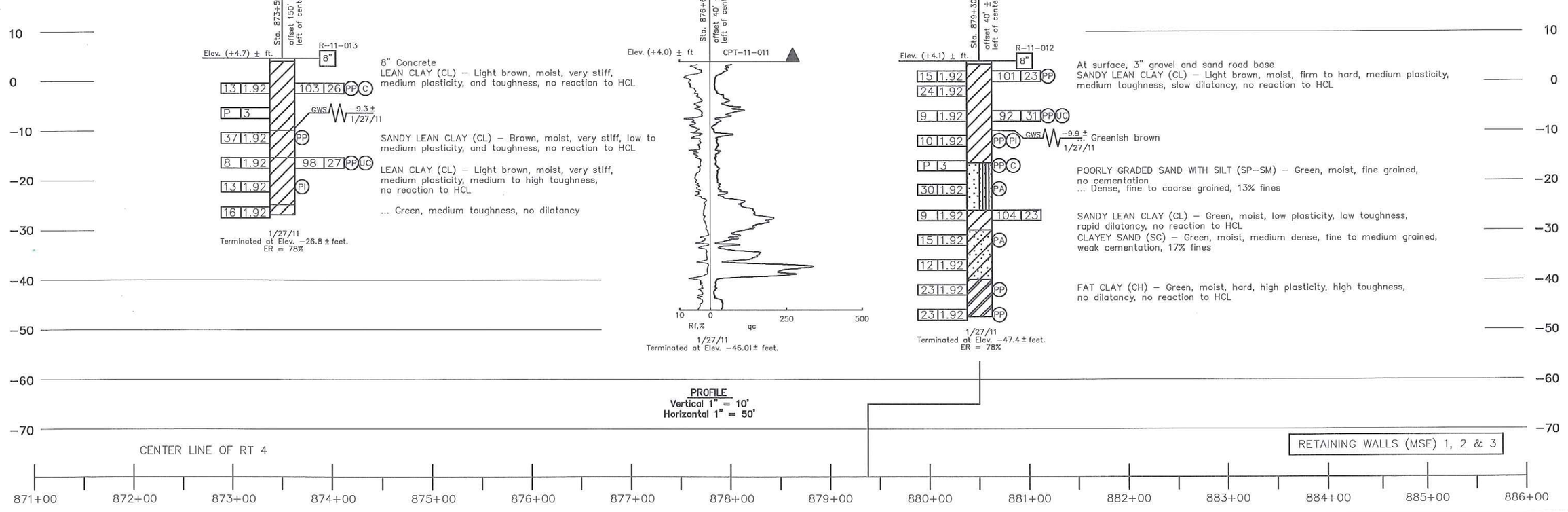


FOUNDATION PLAN
1" = 50'



PROFILE
Vertical 1" = 10'
Horizontal 1" = 50'

CENTER LINE OF RT 4

871+00 872+00 873+00 874+00 875+00 876+00 877+00 878+00 879+00 880+00 881+00 882+00 883+00 884+00 885+00 886+00

RETAINING WALLS (MSE) 1, 2 & 3

X DESIGN OVERSIGHT	DRAWN BY	G. GOMEZ	M, Shubert FIELD INVESTIGATION BY:		PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	H. PARSA PROJECT ENGINEER
X SIGN OFF DATE	CHECKED BY	C. HENDERSON	DATE: January, 2011			

BRIDGE NO.
POST MILES

SR 4 CROSSTOWN VIADUCT
LOG OF TEST BORINGS 3 of 10

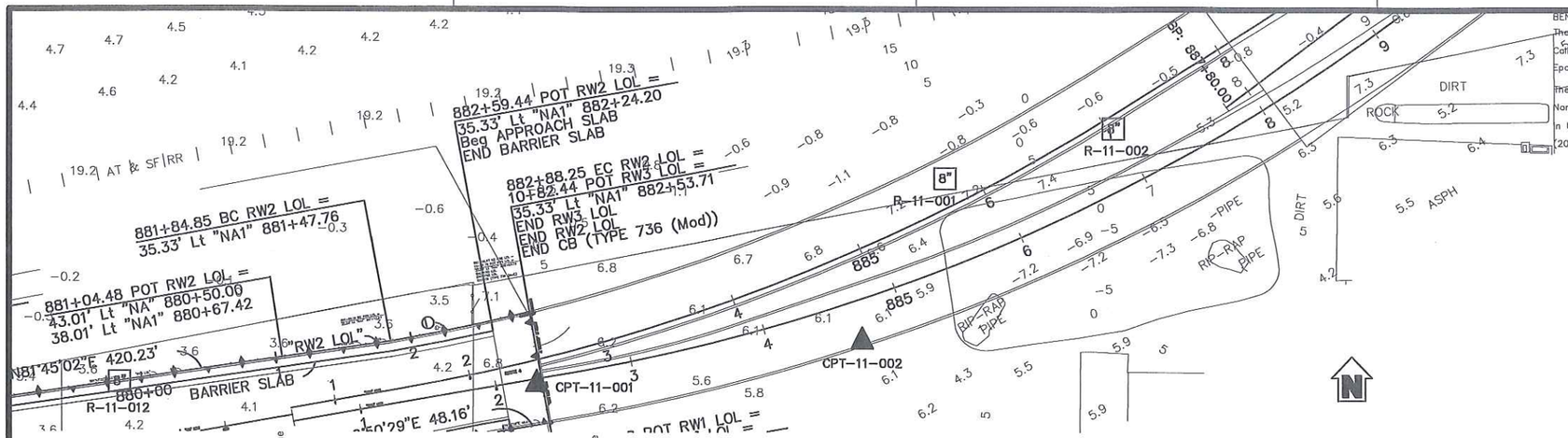
GS GEOLOGIST LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 7/16/10)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: 1454
PROJECT NUMBER & PHASE: 1000020414 CONTRACT NO.: X

DISREGARD PRINTS BEARING EARLIER REVISION DATES →

REVISION DATES				SHEET	OF
5/23/12	8/8/12	8/23/12	8/24/12		



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DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	4, 5	VAR		

Carl Henderson 8/24/12
GEOTECHNICAL PROFESSIONAL DATE

REGISTERED PROFESSIONAL ENGINEER — Carl Henderson No. 2886 Exp. 6/30/2013 GEOTECHNICAL — STATE OF CALIFORNIA

PLANS APPROVAL DATE

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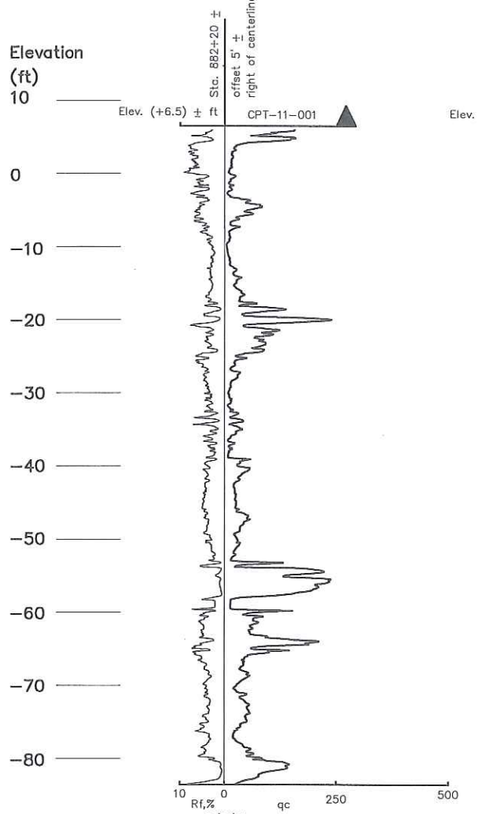

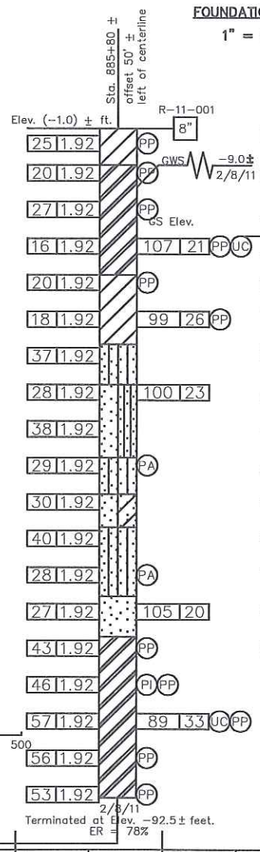

FOUNDATION PLAN
1" = 50'

LEAN CLAY (CL) – Light brown, moist, very stiff, medium to high plasticity, high toughness, no dilatancy

FAT CLAY (CH) – Brown, moist, hard, high plasticity, high toughness, no dilatancy

... With fine sand

... No sand

LEAN CLAY (CL) – Brown, moist, hard, medium to high plasticity, medium toughness, no dilatancy

... Green, high plasticity

SILTY SAND (SM) – Green, moist, medium dense, fine to medium grained, low cementation, 20% fines

POORLY GRADED SAND WITH SILT (SP-SM) – Green, moist, medium dense, fine to medium grained, weak cementation, 10% fines

... About 15% Fines, dense

SILTY SAND (SM) – Greenish gray, moist, medium dense, fine to medium grained, 18% fines

POORLY GRADED SAND WITH CLAY (SP-SC) – Green, moist, dense, fine to medium grained, weak cementation, 10% fines

SILTY SAND (SM) – Green, moist, dense, fine to medium grained, weak to medium cementation, 30% fines, no plasticity

... 45% Fines

POORLY GRADED SAND (SM) – Green, moist, medium dense, fine to medium grained, no cementation, 5% fines

FAT CLAY (CH) – Green, moist, hard, high plasticity, medium toughness, no dilatancy

PROFILE
Vertical 1" = 10'
Horizontal 1" = 50'

CENTER LINE OF RT 4

881+00 882+00 883+00 884+00 885+00 886+00 887+00 888+00 889+00 890+00 891+00 892+00 893+00 894+00 895+00 896+00

RETAINING WALLS (MSE) 1,2 & 3

X DESIGN OVERSIGHT	DRAWN BY	G. GOMEZ	M. Shubert FIELD INVESTIGATION BY:		PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	H. PARSA PROJECT ENGINEER	BRIDGE NO.
X SIGN OFF DATE	CHECKED BY	C. HENDERSON	DATE: February, 2011				POST MILES

SR 4 CROSSTOWN VIADUCT
LOG OF TEST BORINGS 4 of 10

GS GEOLOGIST LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 7/16/10)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: 1454
PROJECT NUMBER & PHASE: 1000020414 CONTRACT NO.: X

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES				SHEET	OF
5/23/12	8/14/12	8/23/12	8/24/12		

COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
SJ	4, 5	VAR		

Carl Henderson 8/24/12
GEOTECHNICAL PROFESSIONAL DATE

REGISTERED PROFESSIONAL ENGINEER — Carl Henderson — No. 2886 — Exp. 6/30/2013 GEOTECHNICAL — STATE OF CALIFORNIA

PLANS APPROVAL DATE

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BENCHMARK:
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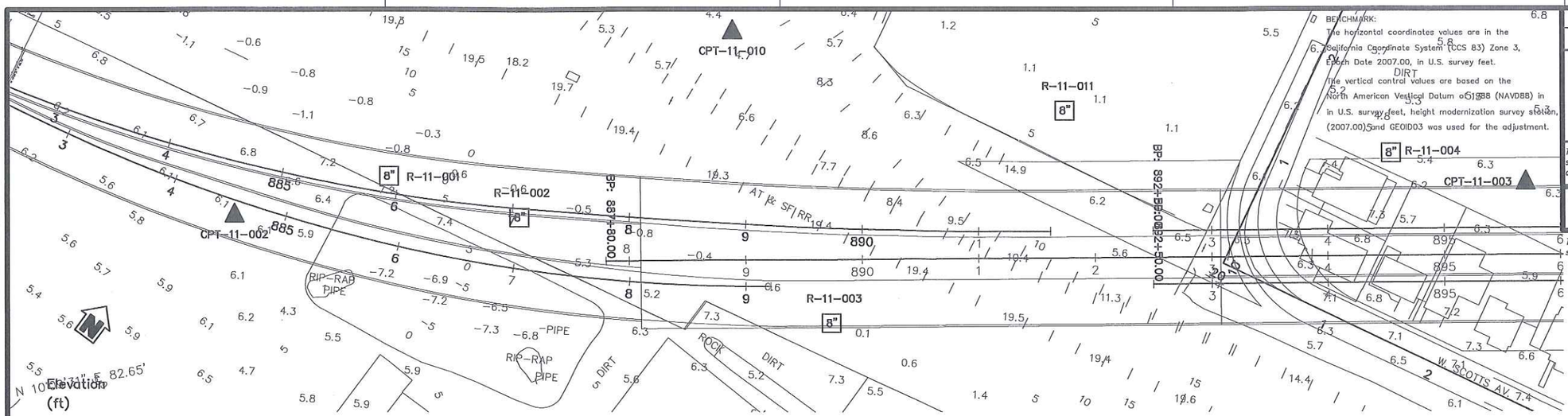
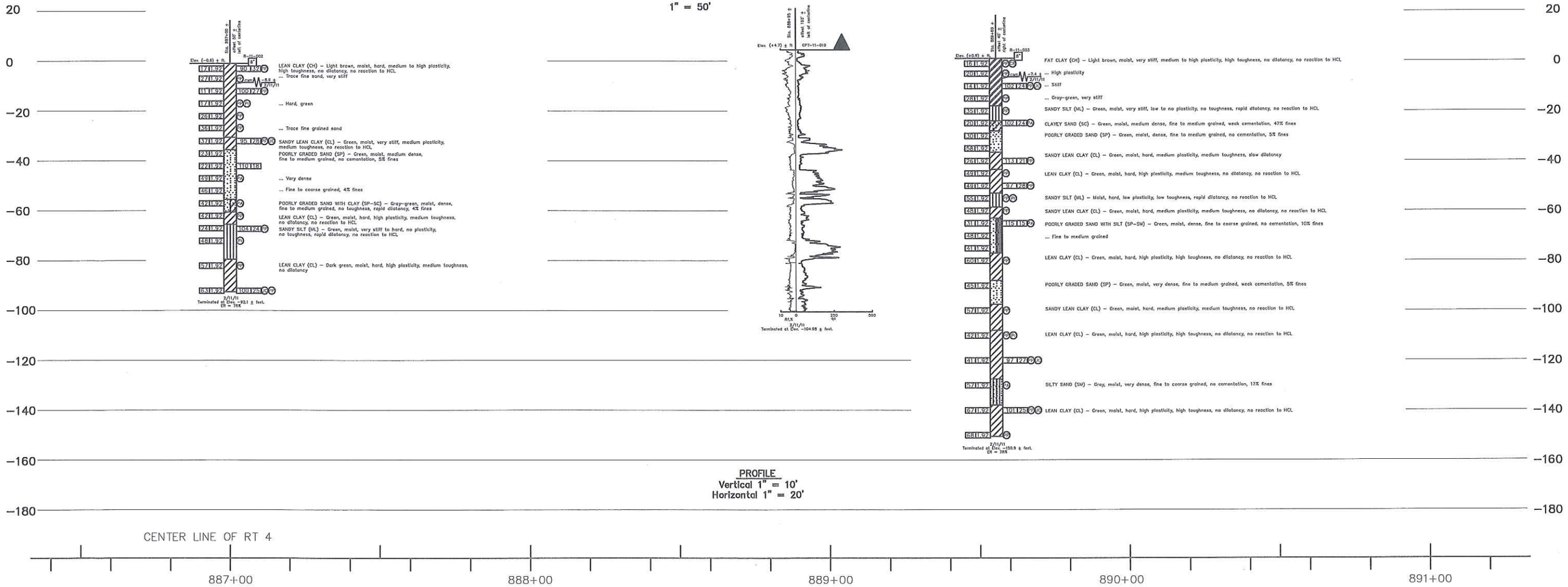


FOUNDATION PLAN
1" = 50'



PROFILE
Vertical 1" = 10'
Horizontal 1" = 20'

X DESIGN OVERSIGHT	DRAWN BY	G. GOMEZ	M. Shubert FIELD INVESTIGATION BY:		PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	H. PARSA PROJECT ENGINEER	BRIDGE NO.
X SIGN OFF DATE	CHECKED BY	C. HENDERSON	DATE: February 2011				POST MILES

SR 4 CROSSTOWN VIADUCT
LOG OF TEST BORINGS 5 of 10

GS GEOLOGIST LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 7/16/10)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: 1454
PROJECT NUMBER & PHASE: 1000020414 CONTRACT NO.: X

DISREGARD PRINTS BEARING EARLIER REVISION DATES →

REVISION DATES				SHEET	OF
5/23/12	8/15/12	8/23/12	8/24/12		

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	4, 5	VAR		

GEOTECHNICAL PROFESSIONAL — 8/24/12 DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER — Carl Henderson — No. 2886 — Exp. 6/30/2013 GEOTECHNICAL — STATE OF CALIFORNIA

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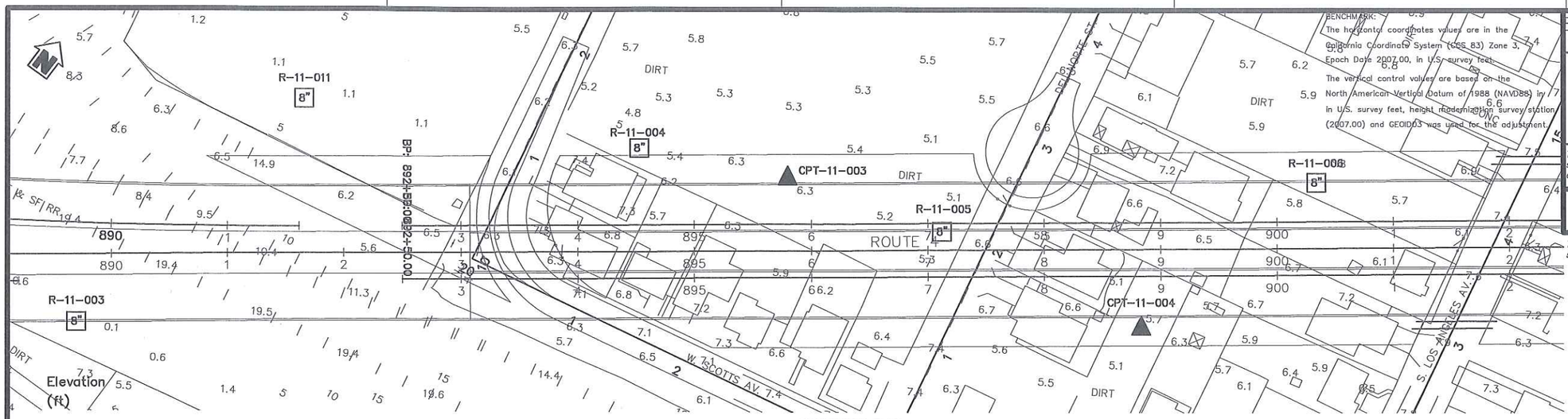
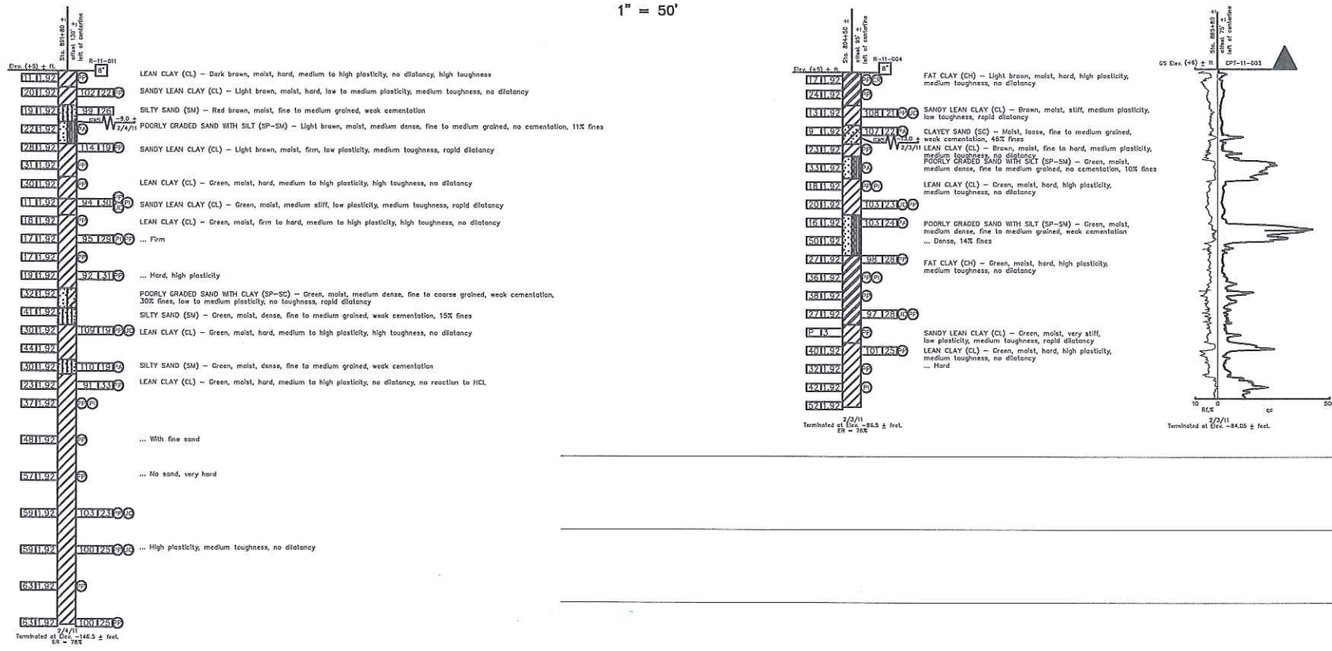
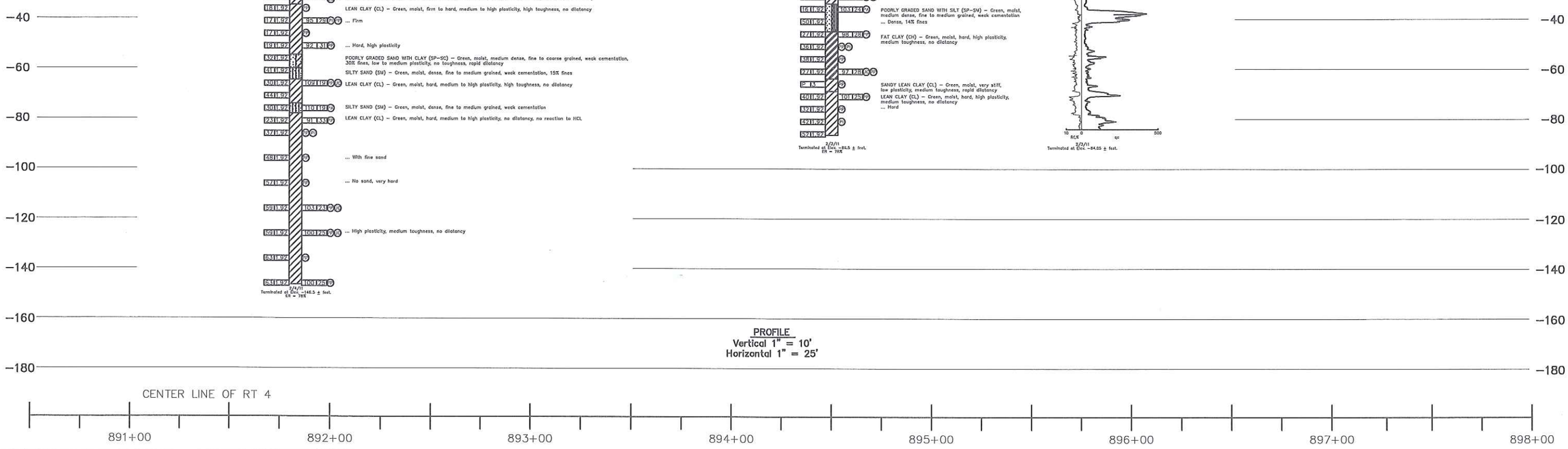


FOUNDATION PLAN
1" = 50'



PROFILE
Vertical 1" = 10'
Horizontal 1" = 25'



X DESIGN OVERSIGHT	DRAWN BY	G. GOMEZ	M. Shubert FIELD INVESTIGATION BY:		PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	H. PARSA PROJECT ENGINEER	BRIDGE NO.
X SIGN OFF DATE	CHECKED BY	C. HENDERSON	DATE: February 2011				POST MILES

SR 4 CROSSTOWN VIADUCT
LOG OF TEST BORINGS 6 of 10

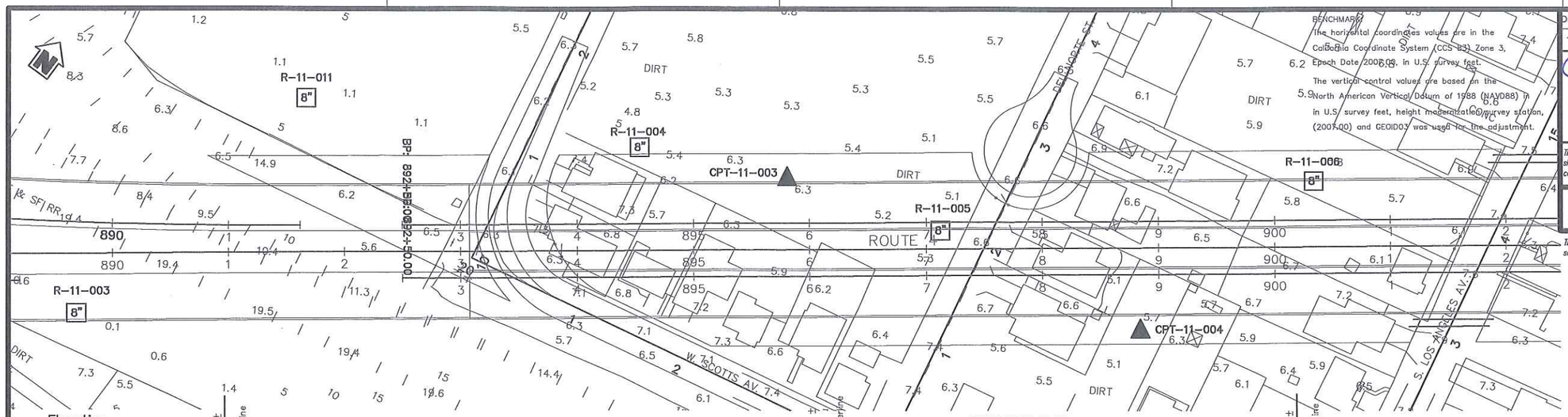
GS GEOLOGIST LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 7/16/10)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: 1454
PROJECT NUMBER & PHASE: 1000020414 CONTRACT NO.: X

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES				SHEET	OF
5/23/12	8/06/12	8/23/12	8/24/12		



BENCHMARK
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DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	4, 5	VAR		

GEOTECHNICAL PROFESSIONAL — 8/24/12 DATE

REGISTERED PROFESSIONAL ENGINEER — Carl Henderson — No. 2886 — Exp. 6/30/2013 GEOTECHNICAL — STATE OF CALIFORNIA

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FOUNDATION PLAN
1" = 50'

PROFILE
Vertical 1" = 10'
Horizontal 1" = 20'

CENTER LINE OF RT 4

Elevation (ft): 10, 0, −10, −20, −30, −40, −50, −60, −70, −80, −90

897+00 | 898+00 | 899+00 | 900+00 | 901+00

R-11-005

Sta. 897+10 ± offset 5' ± left of centerline
Elev. (+5.2) ± ft.

10	1.92	PP			
16	1.92	PP	90	29	PP
14	1.92	PP			
26	1.92	PP GWS	−12.8 ± 2/2/11		
14	1.92	PP	108	21	PP UC
42	1.92	PA			
44	1.92	PP			
35	1.92	PP			
22	1.92	PP PI			
24	1.92	PP	110	22	PP
24	1.92	PP	89	30	PP
25	1.92	PA			
19	1.92	PP	110	20	PP UC
30	1.92				
35	1.92	PA			
60	1.92				
P	3		109	17	
59	1.92				
64	1.92				

2/2/11
Terminated at Elev. −86.3 ± feet.
ER = 78%

LEAN CLAY (CL) – Light brown, moist, hard, medium plasticity, medium toughness, no dilatancy, no reaction to HCL
... Dark brown, high plasticity
... Light brown, firm to hard
... Hard
... Firm to hard
SILTY SAND (SM) – Gray brown, moist, dense, fine to course grained, no cementation, 16% fines
LEAN CLAY (CL) – Green, moist, hard, high plasticity, medium toughness, no dilatancy, trace fine sand
... No sand
CLAYEY SAND (SC) – Green, moist, medium dense, weak cementation, 48% fines
SANDY LEAN CLAY (SC) – Green, moist, firm, low plasticity, medium toughness, rapid dilatancy
POORLY GRADED SAND WITH SILT (SP-SM) – Green, moist, very dense to dense, fine to coarse grained, no cementation, 14% fines
... Weak cementation

CPT-11-004

Sta. 898+80 ± offset 55' ± right of centerline
GS Elev. (+5.1) ± ft

Rf,%: 10, 0
qc: 250, 500

2/2/11
Terminated at Elev. −84.97_+ feet.

R-11-006

Sta. 900+30 ± offset 55' ± left of centerline
Elev. (+5.8) ± ft.

11	1.92	PP CR			
20	1.92	PP			
11	1.92	PP			
18	1.92		104	22	PP UC
13	1.92		99	25	PI GWS −12.2 ± 2/2/11
P	3				
18	1.92		104	25	PP
25	1.92	PA			
22	1.92	PP			
18	1.92		112	24	PA
16	1.92		95	28	PP UC
25	1.92				
24	1.92		101	24	
35	1.92	PA			
25	1.92		100	27	PP PI
34	1.92	PP			
22	1.92		89	33	PP UC
28	1.92	PP			
33	1.92				

2/2/11
Terminated at Elev. −85.7 ± feet.
ER = 78%

FAT CLAY (CH) – Brown, moist, hard, high plasticity, high toughness, no dilatancy, no reaction to HCL
... Very stiff, slight FeOX staining
LEAN CLAY (CL) – Moist, medium to high plasticity, hard, no FeOX staining
SILTY SAND (SM) – Green, moist, fine to medium grained, weak to moderate cementation
LEAN CLAY (CL) – Green, moist, hard, medium to high plasticity, medium toughness, no dilatancy, no reaction to HCL
SILTY SAND (SM) – Green, moist, medium dense, fine to medium grained, weak cementation, 35% fines
LEAN CLAY (CL) – Green, moist, hard, medium plasticity, high toughness, no dilatancy, no reaction to HCL
SILTY SAND (SM) – Green, moist, medium dense, fine to medium grained, weak cementation, 41% fines
LEAN CLAY (CL) – Green, moist, hard, high plasticity, medium toughness, no dilatancy, no reaction to HCL
SANDY LEAN CLAY (CL) – Green, moist, medium dense, fine to medium grained, slight plasticity, moderate cementation
... 65% fines
LEAN CLAY (CL) – Green, moist, hard, medium to high plasticity, high toughness, no dilatancy, no reaction to HCL

X DESIGN OVERSIGHT	DRAWN BY	G. GOMEZ	M. Shubert FIELD INVESTIGATION BY:		PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	H. PARSA PROJECT ENGINEER	BRIDGE NO.
X SIGN OFF DATE	CHECKED BY	C. HENDERSON	DATE: February 2011				POST MILES

SR 4 CROSSTOWN VIADUCT
LOG OF TEST BORINGS 7 of 10

GS GEOLOGIST LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 7/16/10)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: 1454
PROJECT NUMBER & PHASE: 1000020414 CONTRACT NO.: X

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES				SHEET	OF
5/29/12	8/16/12	8/23/12	8/24/12		

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	4, 5	VAR		

Carl Henderson 8/24/12
GEOTECHNICAL PROFESSIONAL DATE

REGISTERED PROFESSIONAL ENGINEER — Carl Henderson No. 2886 Exp. 6/30/2013 GEOTECHNICAL — STATE OF CALIFORNIA

PLANS APPROVAL DATE

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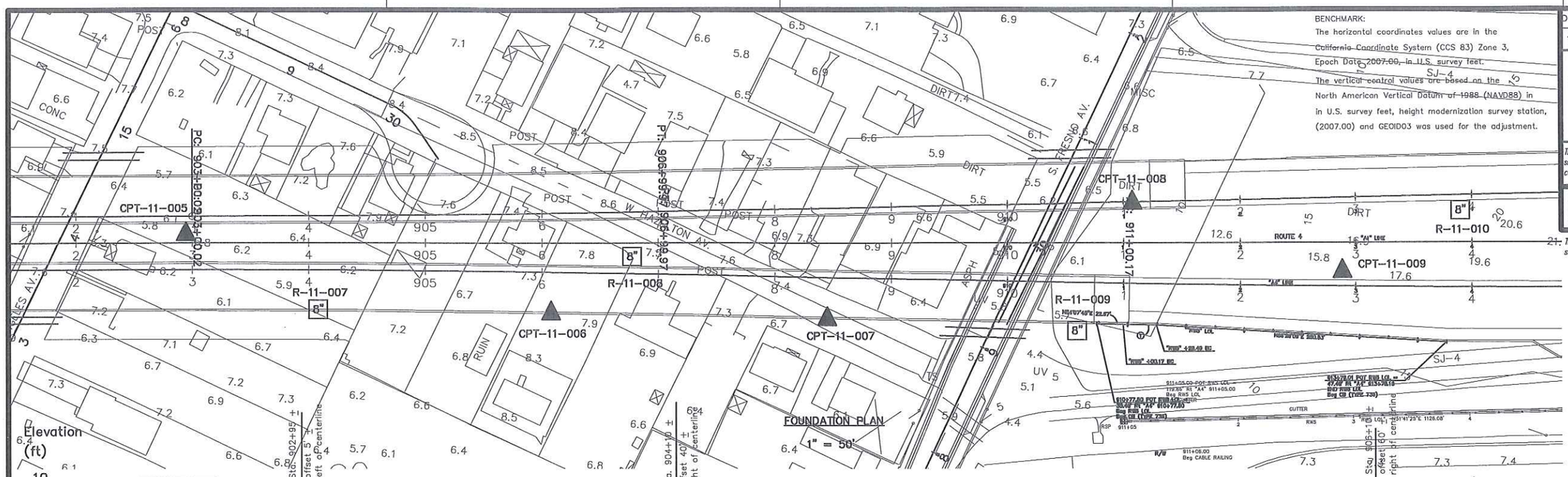
BENCHMARK:
The horizontal coordinates values are in the ~~California Coordinate~~ System (CCS 83) Zone 3, Epoch Date ~~2007.00~~, in U.S. survey feet.
The vertical control values are based on the North American Vertical Datum of 1988 (NAVD88) in in U.S. survey feet, height modernization survey station, (2007.00) and GEOID03 was used for the adjustment.

Kleinfelder, Inc.
2001 Arch-Airport Rd., Suite 100
Stockton, CA. 95206

SJCOG
555 E. Weber Avenue
Stockton, CA. 95202-2804

This LOTB sheet was prepared in accordance with the Caltrans soil and rock logging, classification, and manual (2010) presentation



11	1.92		PP
36	1.92		PP
35	1.92	101	18
P	3	94	29 PP UC
54	1.92		PP
19	1.92	109	21 PP PI
87	1.92		PA
38	1.92		PP
34	1.92		PA
37	1.92	106	22 UC
29	1.92	108	22 PP
34	1.92		PP PI
30	1.92	98	28 PP
35	1.92		PP
54	1.92	102	25 PP UC
36	1.92	91	32 UC
53	1.92		PP
44	1.92		PP
55	1.92		PP

LEAN CLAY (CL) – Brown, moist, hard, medium plasticity, medium toughness, No dilatancy, no reaction to HCL

SANDY SILT (ML) – Light brown, moist, hard, low plasticity

POORLY GRADED SAND (SP) – Light brown, moist, mediun dense, fine to medium grained, no cementation

LEAN CLAY (CL) – Brown, moist, hard, medium to high plasticity, high toughness, no dilatancy

... Slight increase in fine sand

SILTY SAND (SM) – Green, moist, very dense, fine to medium grained, strong cementation, 30% fines

SANDY LEAN CLAY (CL) – Green, moist, hard, medium to high plasticity, medium toughness, no to slow dilatancy, no reaction to HCL

POORLY GRADED SAND WITH SILT (SP–SM) – Green, moist, medium dense, fine to medium grained, no reaction to HCL, 14% fines

LEAN CLAY (CL) – Green, moist, hard, high plasticity and toughness, no dilatancy, no reaction to HCL

FAT CLAY (CH) – Green, moist, hard

PROFILE
Vertical 1" = 10'
Horizontal 1" = 20'

X DESIGN OVERSIGHT	DRAWN BY	G. GOMEZ	M. Shubert FIELD INVESTIGATION BY:		PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	H. PARSA PROJECT ENGINEER	BRIDGE NO.	SR 4 CROSSTOWN VIADUCT
X SIGN OFF DATE	CHECKED BY	C. HENDERSON	DATE: January 2011				POST MILES	LOG OF TEST BORINGS 8 of 10

GS GEOLOGIST LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 7/16/10)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: 1454
PROJECT NUMBER & PHASE: 1000020414 CONTRACT NO.: X

DISREGARD PRINTS BEARING EARLIER REVISION DATES →

REVISION DATES				SHEET	OF
5/23/12	8/16/12	8/23/12	8/24/12		

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	4, 5	VAR		

BENCHMARK:
The horizontal coordinates values are in the California Coordinate System (CCS 83) Zone 3, Epoch Date 2007.00, in U.S. survey feet.
The vertical control values are based on the North American Vertical Datum of 1988 (NAVD88) in in U.S. survey feet, height modernization survey station, (2007.00) and GEOID03 was used for the adjustment.

Carl Henderson — 8/24/12
GEOTECHNICAL PROFESSIONAL — DATE

REGISTERED PROFESSIONAL ENGINEER — Carl Henderson — No. 2886 — Exp. 6/30/2013 GEOTECHNICAL — STATE OF CALIFORNIA

PLANS APPROVAL DATE

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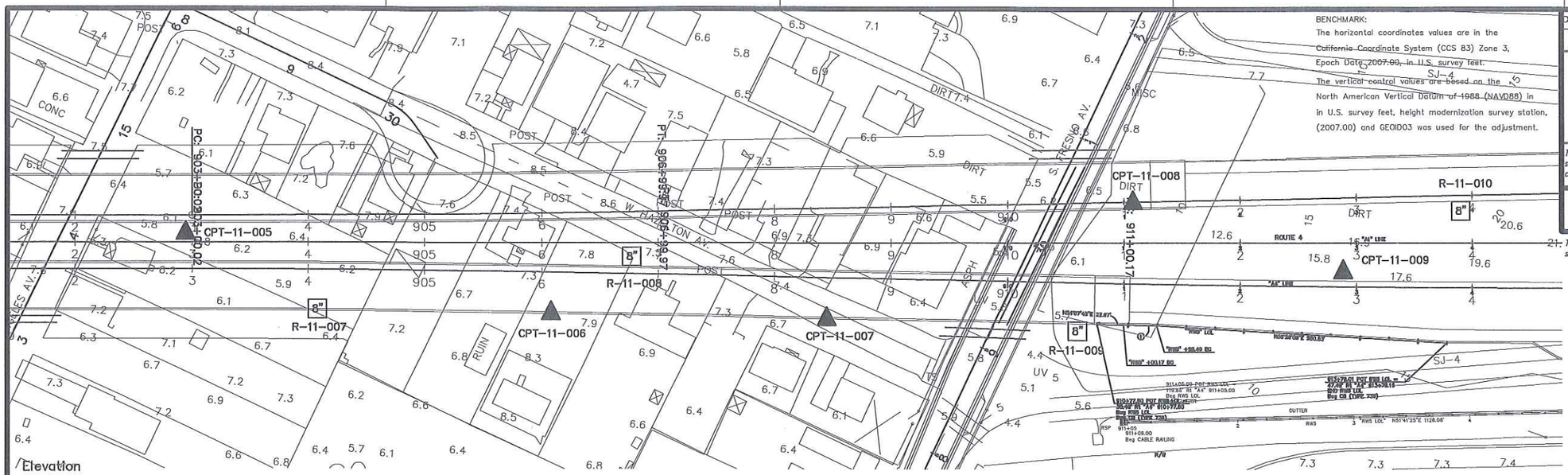
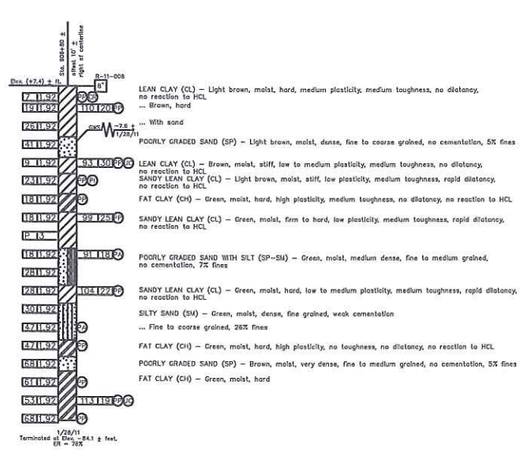
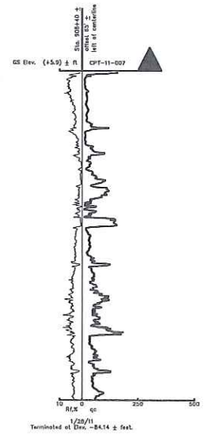
555 E. Weber Avenue
Stockton, CA. 95202-2804

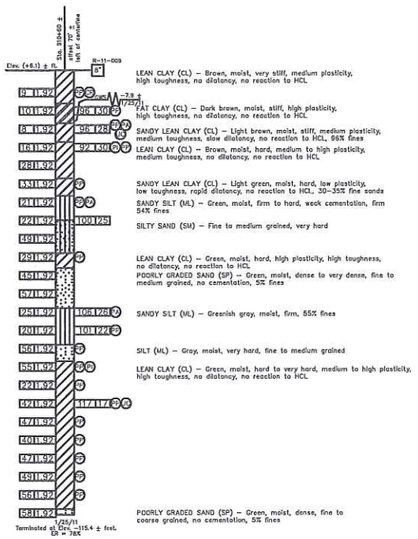
21. *This LOTB sheet was prepared in accordance with the Caltrans soil and rock logging, classification, and manual (2010) presentation*



FOUNDATION PLAN
1" = 50'







PROFILE
Vertical 1" = 10'
Horizontal 1" = 25'

RETAINING WALL 5

CENTER LINE OF RT 4

906+00 907+00 908+00 909+00 910+00 911+00 912+00 913+00

R-6

X DESIGN OVERSIGHT	DRAWN BY	G. GOMEZ	M. Shubert FIELD INVESTIGATION BY:		PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	H. PARSA PROJECT ENGINEER	BRIDGE NO.
X SIGN OFF DATE	CHECKED BY	C. HENDERSON	DATE: January 2011				POST MILES

SR 4 CROSSTOWN VIADUCT
LOG OF TEST BORINGS 9 of 10

GS GEOLOGIST LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 7/16/10)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: 1454
PROJECT NUMBER & PHASE: 1000020414 CONTRACT NO.: X

DISREGARD PRINTS BEARING EARLIER REVISION DATES →

REVISION DATES				SHEET	OF
5/23/12	8/15/12	8/23/12	8/24/12		

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	4, 5	VAR		

GEOTECHNICAL PROFESSIONAL — 8/24/12 DATE

REGISTERED PROFESSIONAL ENGINEER — Carl Henderson — No. 2886 — Exp. 6/30/2013 GEOTECHNICAL — STATE OF CALIFORNIA

PLANS APPROVAL DATE

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BENCHMARK:
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The vertical control values are based on the North American Vertical Datum of 1988 (NAVD88) in in U.S. survey feet, height modernization survey station, (2007.00) and GEOID03 was used for the adjustment.

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Stockton, CA. 95206

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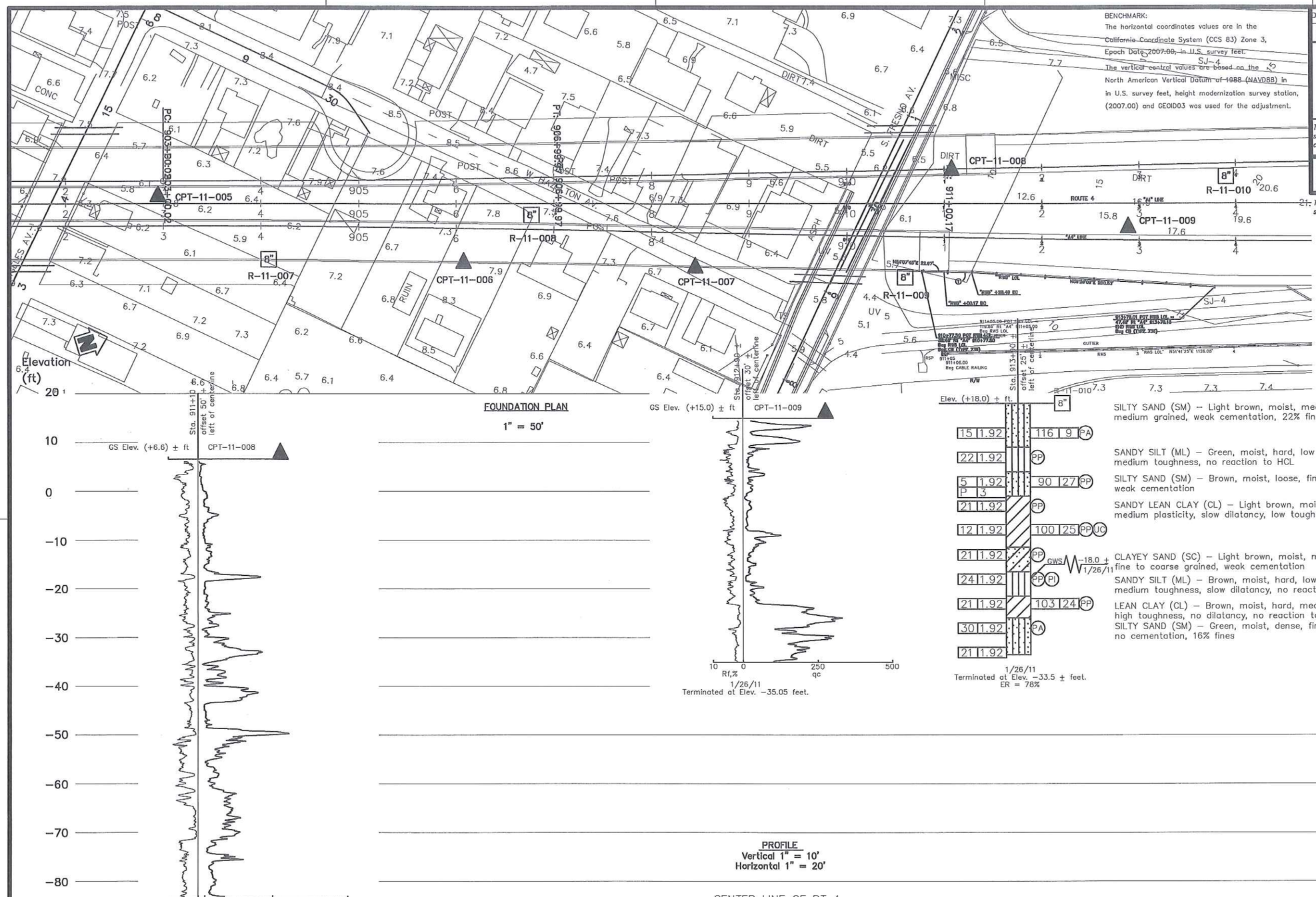
21. This LOTB sheet was prepared in accordance with the Caltrans soil and rock logging, classification, and manual (2010) presentation



FOUNDATION PLAN
1" = 50'

PROFILE
Vertical 1" = 10'
Horizontal 1" = 20'

Elevation (ft): 20, 10, 0, -10, -20, -30, -40, -50, -60, -70, -80

SILTY SAND (SM) – Light brown, moist, medium dense, fine to medium grained, weak cementation, 22% fines

SANDY SILT (ML) – Green, moist, hard, low plasticity, medium toughness, no reaction to HCL

SILTY SAND (SM) – Brown, moist, loose, fine grained, weak cementation

SANDY LEAN CLAY (CL) – Light brown, moist, very stiff, medium plasticity, slow dilatancy, low toughness, no reaction to HCL

CLAYEY SAND (SC) – Light brown, moist, medium dense, fine to coarse grained, weak cementation

SANDY SILT (ML) – Brown, moist, hard, low plasticity, medium toughness, slow dilatancy, no reaction to HCL

LEAN CLAY (CL) – Brown, moist, hard, medium to high plasticity, high toughness, no dilatancy, no reaction to HCL

SILTY SAND (SM) – Green, moist, dense, fine grained, no cementation, 16% fines

CENTER LINE OF RT 4

911+00 912+00 913+00 914+00 915+00

RETAINING WALL 5

R-7

X DESIGN OVERSIGHT	DRAWN BY	G. GOMEZ	M. Shubert FIELD INVESTIGATION BY:	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	H. PARSA PROJECT ENGINEER	BRIDGE NO.	SR 4 CROSSTOWN VIADUCT
X SIGN OFF DATE	CHECKED BY	C. HENDERSON	DATE: JUNE 2011			POST MILES	LOG OF TEST BORINGS 10 of 10

GS GEOLOGIST LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 7/16/10)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: 1454
PROJECT NUMBER & PHASE: 1000020414 CONTRACT NO.: X

DISREGARD PRINTS BEARING EARLIER REVISION DATES → REVISION DATES: 5/29/12, 8/15/12, 8/23/12, 8/24/12

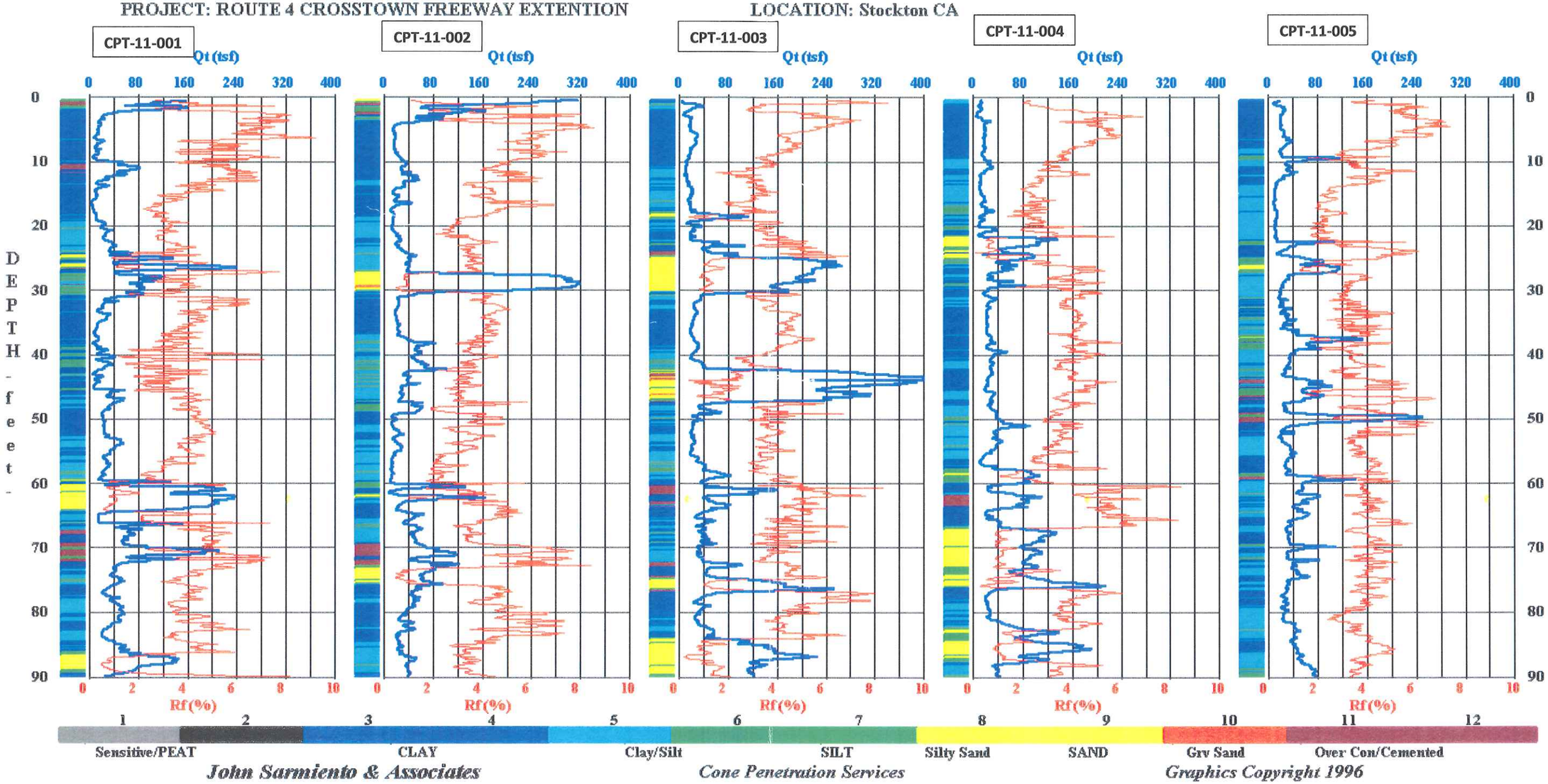
SHEET OF

Cone Penetrometer Test Profiles

KLEINFELDER
Bright People. Right Solutions.

PROJECT: ROUTE 4 CROSSTOWN FREEWAY EXTENTION

LOCATION: Stockton CA

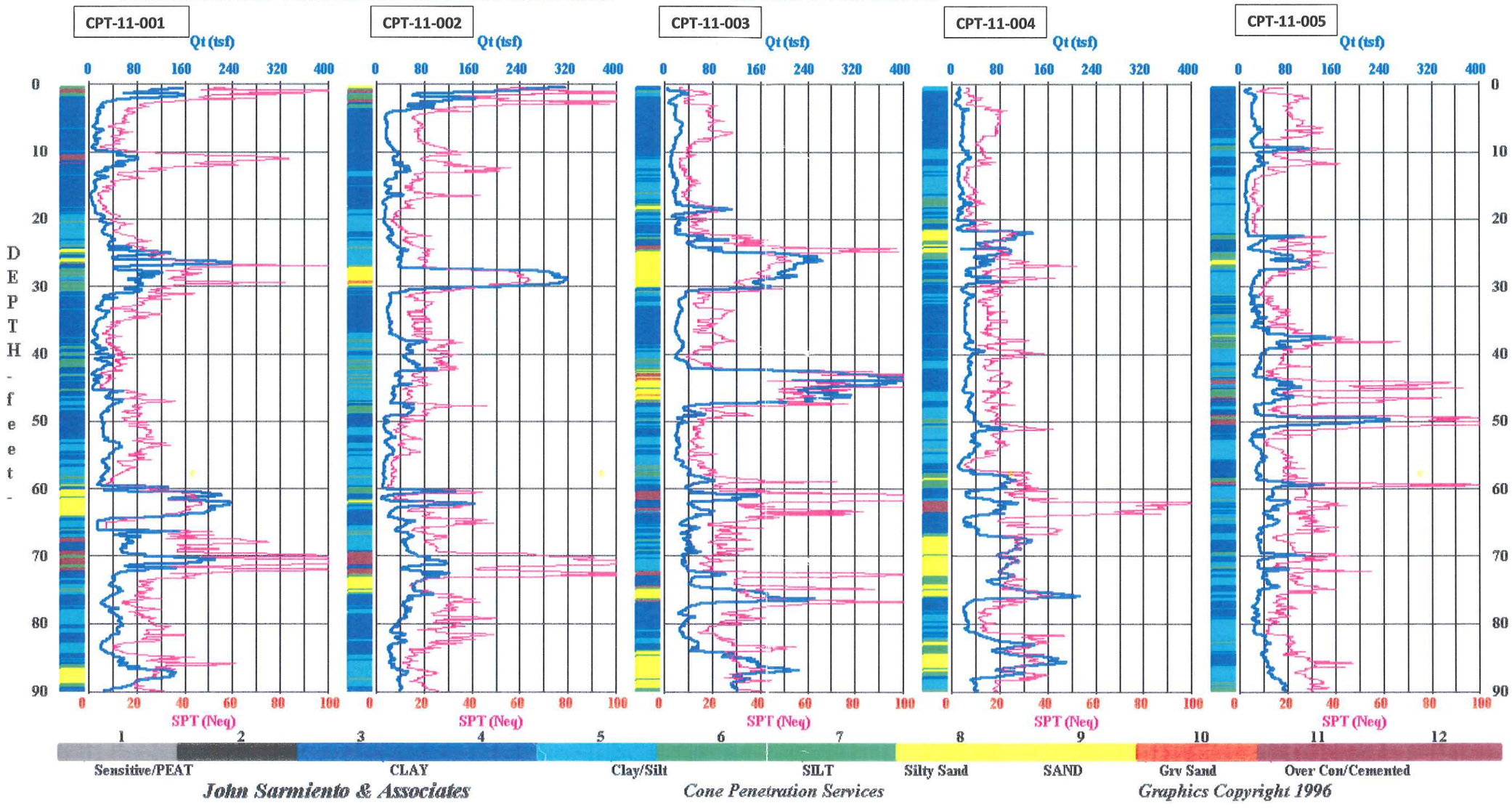


John Sarmiento & Associates *Cone Penetration Services*

Graphics Copyright 1996

PROJECT: ROUTE 4 CROSSTOWN FREEWAY EXTENTION

LOCATION: Stockton CA

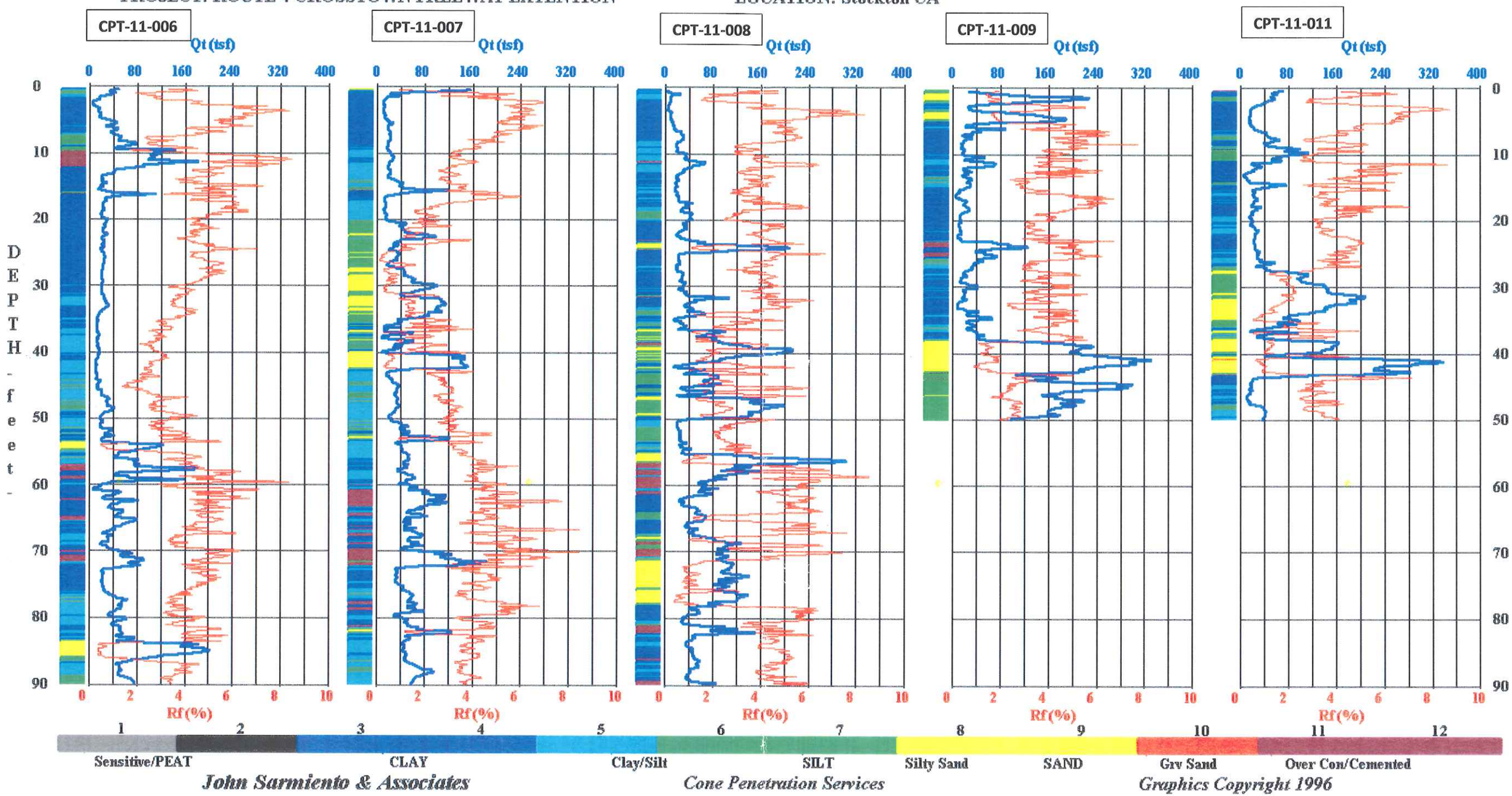


John Sarmiento & Associates *Cone Penetration Services*

Graphics Copyright 1996

PROJECT: ROUTE 4 CROSSTOWN FREEWAY EXTENTION

LOCATION: Stockton CA

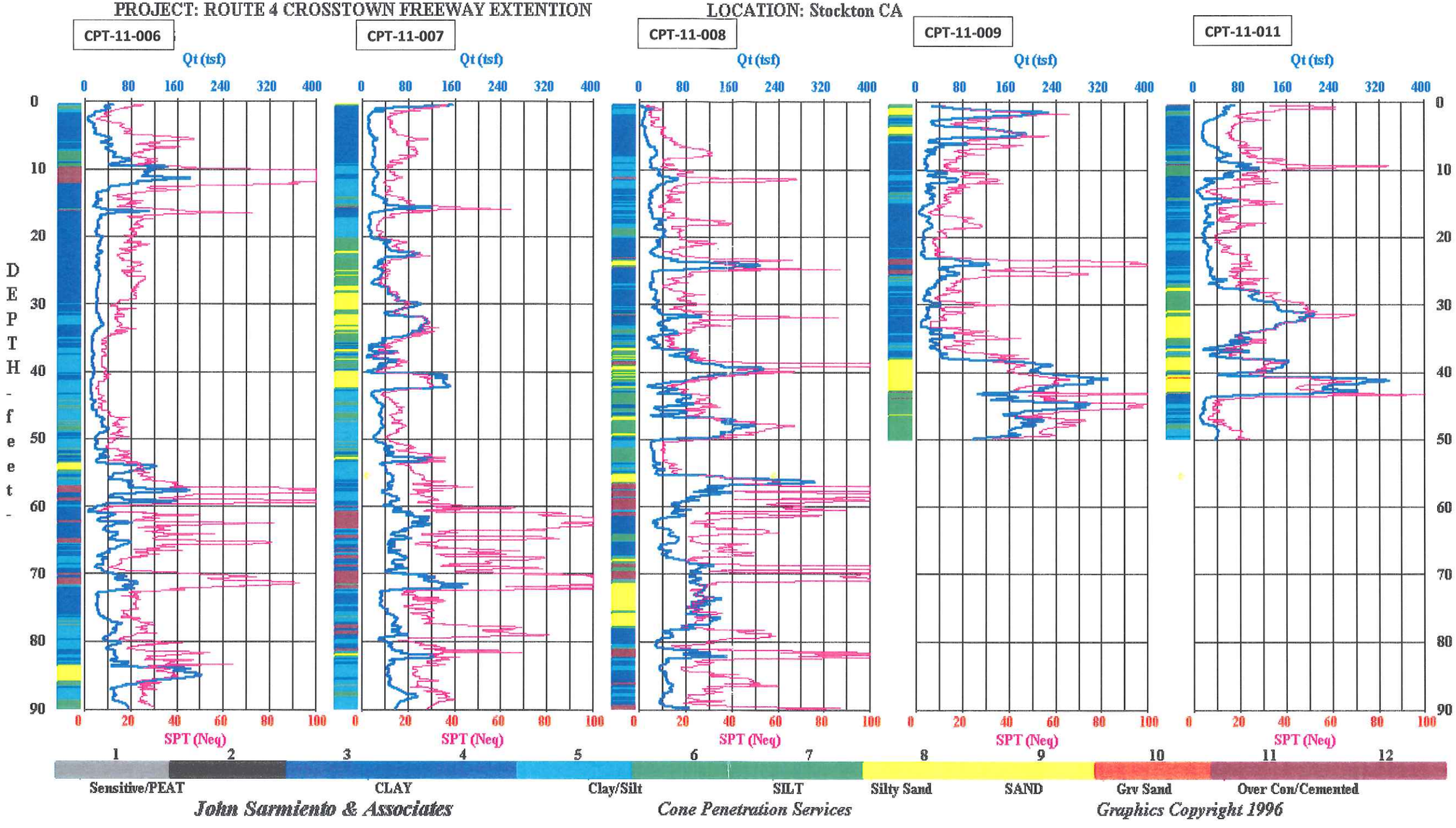


John Sarmiento & Associates *Cone Penetration Services*

Graphics Copyright 1996

PROJECT: ROUTE 4 CROSSTOWN FREEWAY EXTENTION

LOCATION: Stockton CA



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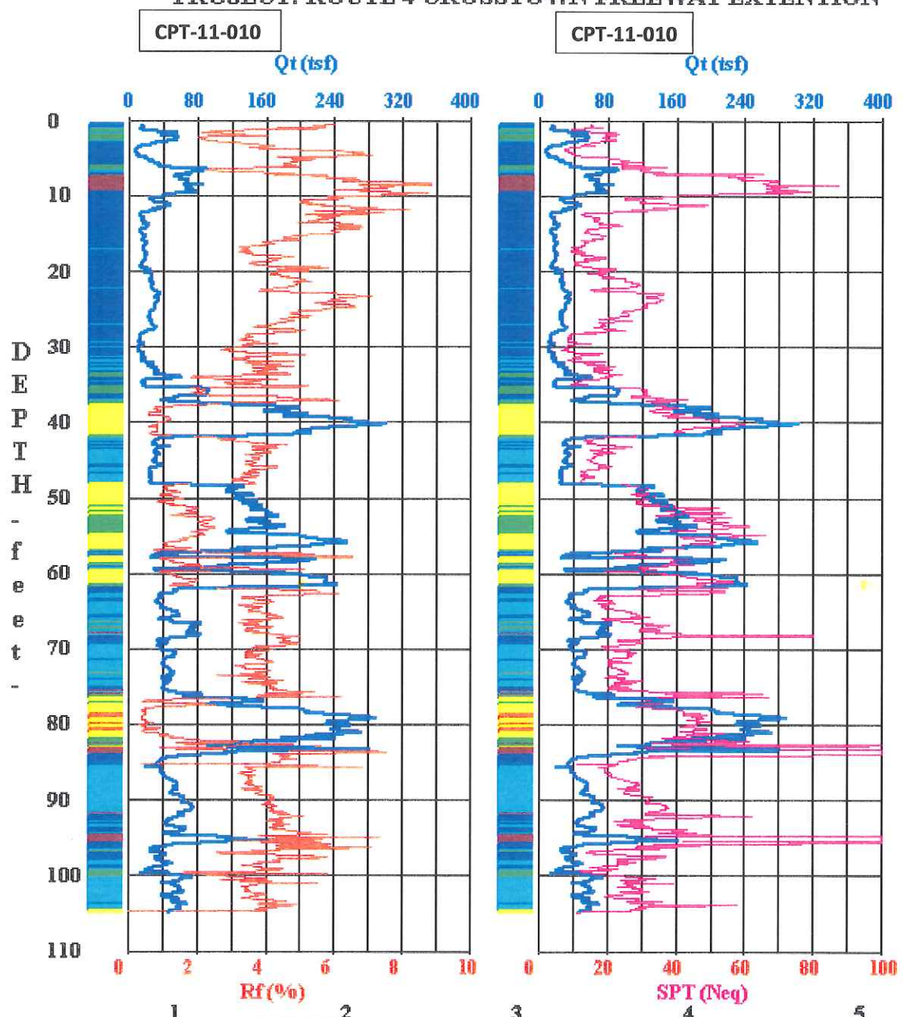
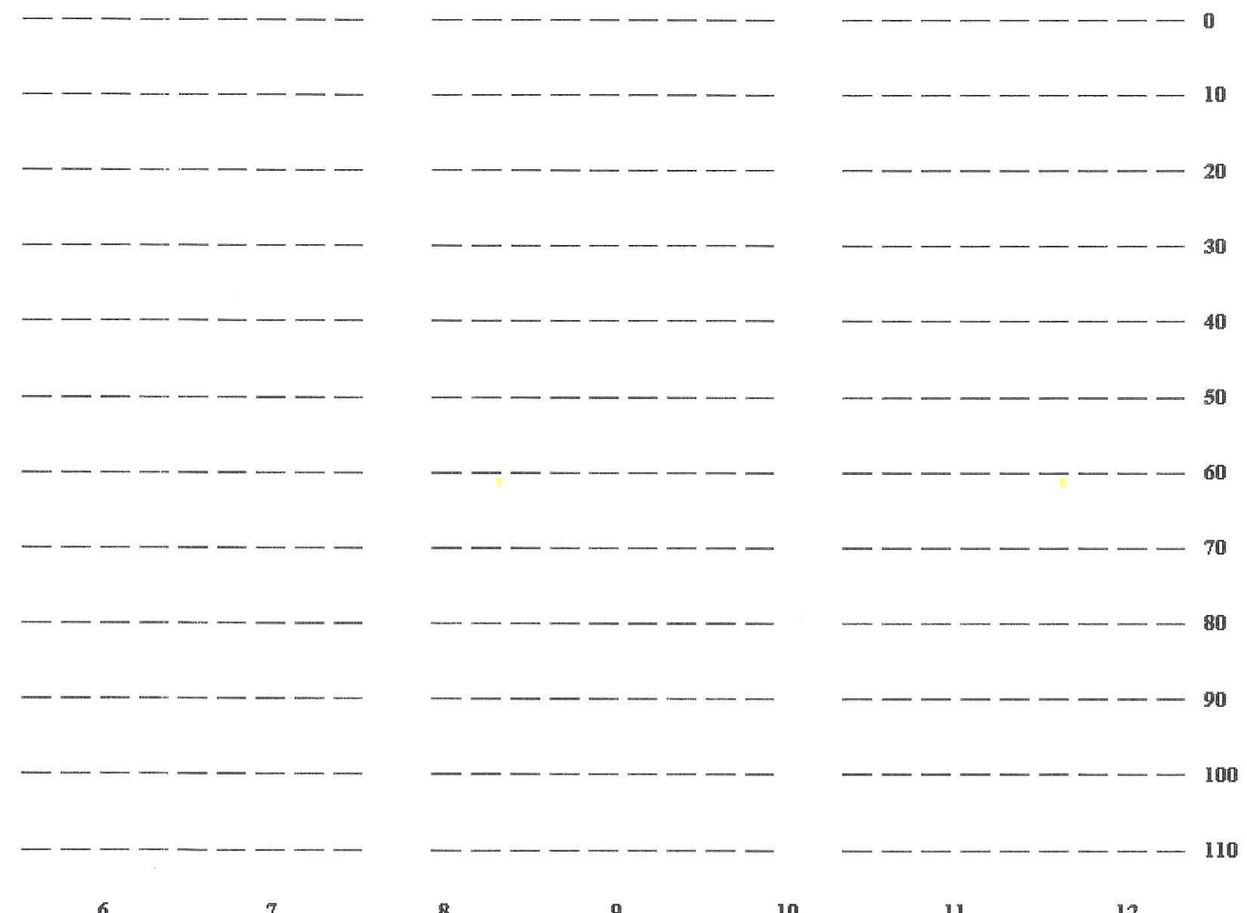
Graphics Copyright 1996

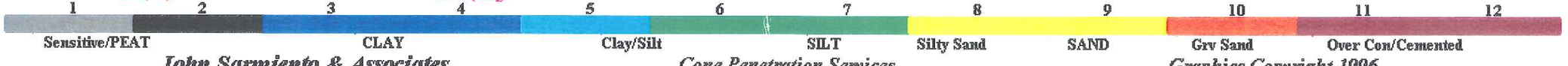
KLEINFELDER
Bright People. Right Solutions.

PROJECT: ROUTE 4 CROSSTOWN FREEWAY EXTENTION

LOCATION: Stockton CA







John Sarmiento & Associates *Cone Penetration Services*

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APPENDIX B
LOGS OF BORINGS AND CONE PENETRATION TESTS PROFILES
(PARIKH CONSULTANT, INC. INVESTIGATION, MARCH 2010)

LIST OF ATTACHMENTS

The following are attached and complete this appendix.

Log of Test Borings Sheets
Cone Penetrometer Test Profiles

Copyright 2012 Kleinfelder

Log of Test Borings Sheets

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (JUNE 2007)

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	4	R12.6/R15.7		

GEOTECHNICAL PROFESSIONAL DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER — GARY PARIKH — No. G.E. 666 — Exp. 12/31/11 — GEOTECHNICAL — STATE OF CALIFORNIA

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131

SAN JOAQUIN COUNCIL OF GOVERNMENTS
555 EAST WEBER AVENUE
STOCKTON, CALIFORNIA 95202

BKF ENGINEERS
4670 WILLOW ROAD, SUITE 250
PLEASANTON CALIFORNIA 94588

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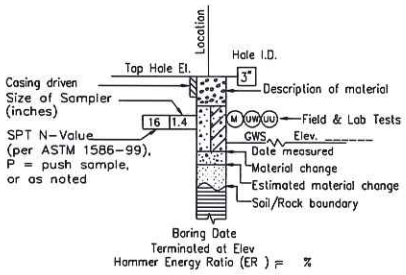
To get to the Caltrans web site, go to: http://www.dot.ca.gov

CEMENTATION	
Description	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

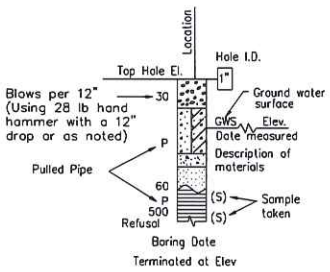
BOREHOLE IDENTIFICATION		
Symbol	Hole Type	Description
Size	A	Auger Boring
Size	R P	Rotary drilled boring Rotary percussion boring (air)
Size	R	Rotary drilled diamond core
Size	HD HA	Hand driven (1-inch soil tube) Hand Auger
●	D	Dynamic Cone Penetration Boring
▲	CPT	Cone Penetration Test (ASTM D 5778-95)
□	O	Other
Note: Size in inches.		

CONSISTENCY OF COHESIVE SOILS				
Description	Unconfined Compressive Strength (tsf)	Pocket Penetrometer Measurement (tsf)	Torvane Measurement (tsf)	Field Approximation
Very Soft	< 0.25	< 0.25	< 0.12	Easily penetrated several inches by fist
Soft	0.25 to 0.50	0.25 to 0.50	0.12 to 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 to 1.0	0.50 to 1.0	0.25 to 0.50	Penetrated several inches by thumb with moderate effort
Stiff	1 to 2	1 to 2	0.50 to 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2 to 4	2 to 4	1.0 to 2.0	Readily indented by thumbnail
Hard	> 4.0	> 4.0	> 2.0	Indented by thumbnail with difficulty

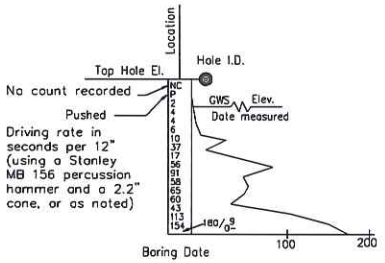
PLASTICITY OF FINE-GRAINED SOILS	
Description	Criteria
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.



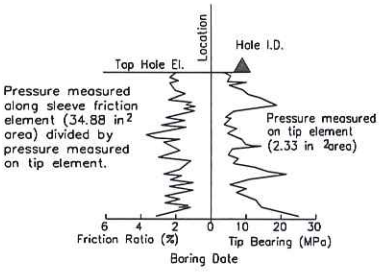
ROTARY BORING



HAND BORING



DYNAMIC CONE PENETRATION BORING



CONE PENETRATION TEST (CPT) SOUNDING

DESIGN OVERSIGHT	DRAWN BY	Amit Bharadwaj	L. S. Bhangoo	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	Gary Parikh	BRIDGE NO. –	ROUTE 4 CROSSTOWN CONNECTOR
SIGN OFF DATE	CHECKED BY	David Wang	FIELD INVESTIGATION BY: DATE: November 2009		PROJECT ENGINEER	POST MILES –	LOG OF TEST BORINGS

005 CIVIL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 08-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU –
EA –

DISREGARD PRINTS BEARING EARLIER REVISION DATES →

REVISION DATES | SHEET | OF

FILE => $REQUEST

USERNAME => $USER DATE PLOTTED => $DATE TIME PLOTTED => $TIME

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (JUNE 2007)

GROUP SYMBOLS AND NAMES

Graphic/Symbol	Group Names	Graphic/Symbol	Group Names
GW	Well-graded GRAVEL Well-graded GRAVEL with SAND	CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
GP	Poorly graded GRAVEL Poorly graded GRAVEL with SAND		
GW-GM	Well-graded GRAVEL with SILT Well-graded GRAVEL with SILT and SAND	CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
GW-GC	Well-graded GRAVEL with CLAY (or SILTY CLAY) Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)		
GP-GM	Poorly graded GRAVEL with SILT Poorly graded GRAVEL with SILT and SAND	ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
GP-GC	Poorly graded GRAVEL with CLAY (or SILTY CLAY) Poorly graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)		
GM	SILTY GRAVEL SILTY GRAVEL with SAND	OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND		
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND	OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
SW	Well-graded SAND Well-graded SAND with GRAVEL		
SP	Poorly graded SAND Poorly graded SAND with GRAVEL	CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
SW-SM	Well-graded SAND with SILT Well-graded SAND with SILT and GRAVEL		
SW-SC	Well-graded SAND with CLAY (or SILTY CLAY) Well-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)	MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
SP-SM	Poorly graded SAND with SILT Poorly graded SAND with SILT and GRAVEL		
SP-SC	Poorly graded SAND with CLAY (or SILTY CLAY) Poorly graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)	OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
SM	SILTY SAND SILTY SAND with GRAVEL		
SC	CLAYEY SAND CLAYEY SAND with GRAVEL	OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY ORGANIC elastic SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL		
PT	PEAT	OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND
	COBBLES COBBLES and BOULDERS BOULDERS		

FIELD AND LABORATORY TESTING

- (C) Consolidation (ASTM D 2435)
- (CL) Collapse Potential (ASTM D 5333)
- (CP) Compaction Curve (CTM 216)
- (CR) Corrosivity Testing (CTM 643, CTM 422, CTM 417)
- (CU) Consolidated Undrained Triaxial (ASTM D 4767)
- (DS) Direct Shear (ASTM D 3080)
- (EI) Expansion Index (ASTM D 4829)
- (M) Moisture Content (ASTM D 2216)
- (OC) Organic Content-% (ASTM D 2974)
- (P) Permeability (CTM 220)
- (PA) Particle Size Analysis (ASTM D 422)
- (PI) Plasticity Index (AASHTO T 90) Liquid Limit (AASHTO T 89)
- (PL) Point Load Index (ASTM D 5731)
- (PM) Pressure Meter
- (PP) Pocket Penetrometer
- (R) R-Value (CTM 301)
- (SE) Sand Equivalent (CTM 217)
- (SG) Specific Gravity (AASHTO T 100)
- (SL) Shrinkage Limit (ASTM D 427)
- (SW) Swell Potential (ASTM D 4546)
- (TV) Pocket Torvane
- (UC) Unconfined Compression-Soil (ASTM D 2166) Unconfined Compression-Rock (ASTM D 2938)
- (UU) Unconsolidated Undrained Triaxial (ASTM D 2850)
- (UW) Unit Weight (ASTM D 4767)
- (VS) Vane Shear (AASHTO T 223)

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	4	R12.6/R15.7		

GEOTECHNICAL PROFESSIONAL DATE

REGISTERED PROFESSIONAL ENGINEER GARY PARIKH No. G.E. 666 Exp. 12/31/11 GEOTECHNICAL STATE OF CALIFORNIA

PLANS APPROVAL DATE

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131

SAN JOAQUIN COUNCIL OF GOVERNMENTS
555 EAST WEBER AVENUE
STOCKTON, CALIFORNIA 95202

BKF ENGINEERS
4670 WILLOW ROAD, SUITE 250
PLEASANTON CALIFORNIA 94588

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APPARENT DENSITY OF COHESIONLESS SOILS

Description	SPT N_{60} (Blows / 12 inches)
Very loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

MOISTURE

Description	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

PERCENT OR PROPORTION OF SOILS

Description	Criteria
Trace	Particles are present but estimated to be less than 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

PARTICLE SIZE

Description		Size
Boulder		> 12"
Cobble		3" to 12"
Gravel	Coarse	3/4" to 3"
	Fine	No. 4 to 3/4"
Sand	Coarse	No. 10 to No. 4
	Medium	No. 40 to No. 10
	Fine	No. 200 to No. 40

DESIGN OVERSIGHT
SIGN OFF DATE

DRAWN BY: Amit Bharadwaj
CHECKED BY: David Wang

L. S. Bhangoo
FIELD INVESTIGATION BY:
DATE: November 2009

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

Gary Parikh
PROJECT ENGINEER

BRIDGE NO. -
POST MILES -

CU -
EA -

ROUTE 4 CROSSTOWN CONNECTOR
LOG OF TEST BORINGS

005 CIVIL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 08-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

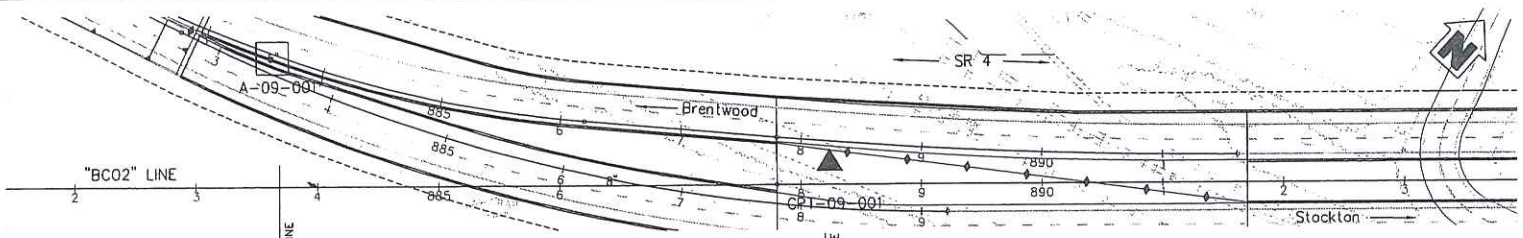
DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES | SHEET | OF

FILE => $REQUEST

USERNAME => $USER DATE PLOTTED => $DATE TIME PLOTTED => $TIME

PLATE A-3



DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	4	R12.6/R15.7		

GEOTECHNICAL PROFESSIONAL DATE

REGISTERED PROFESSIONAL ENGINEER — GARY PARIKH — No. G.E. 666 — Exp. 12/31/11 — GEOTECHNICAL — STATE OF CALIFORNIA

PLANS APPROVAL DATE

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131

SAN JOAQUIN COUNCIL OF GOVERNMENTS
555 EAST WEBER AVENUE
STOCKTON, CALIFORNIA 95202

BKF ENGINEERS
4670 WILLOW ROAD, SUITE 250
PLEASANTON CALIFORNIA 94588

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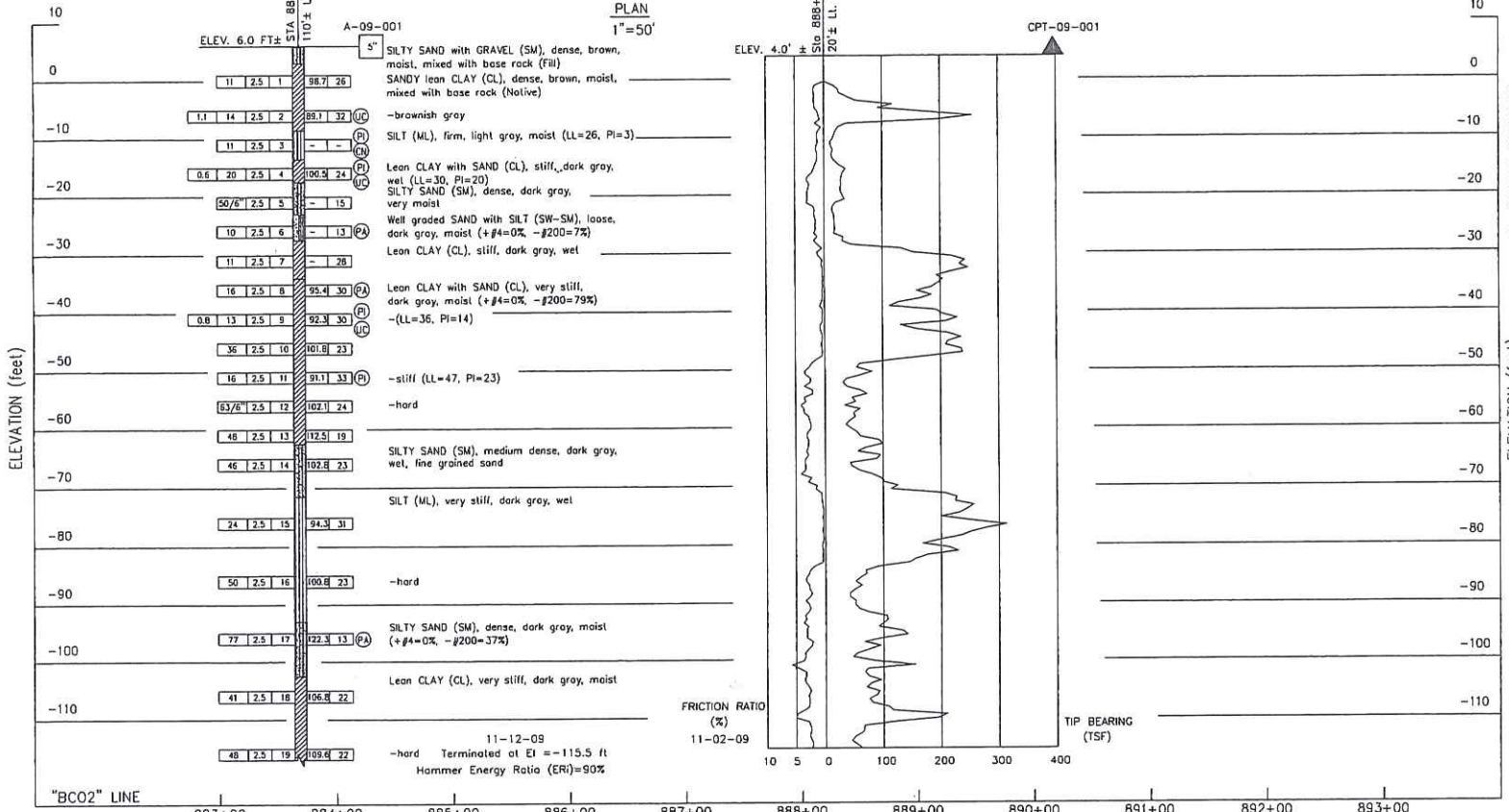
PLAN
1"=50'



DRAFT

Notes:
Modified California Sampler: I.D. = 2.5"; O.D. = 3", Hammer Assembly: A 140 lb hammer with a 30" drop, (Automatic Hammer)

This LOTB sheet was prepared in accordance with the Caltrans Soil & Rock, Logging, Classification, and Presentation Manual (June 2007)

All dimensions are in feet unless otherwise shown

PROFILE
Vert. : 1" = 10'
Hor. : 1" = 50'

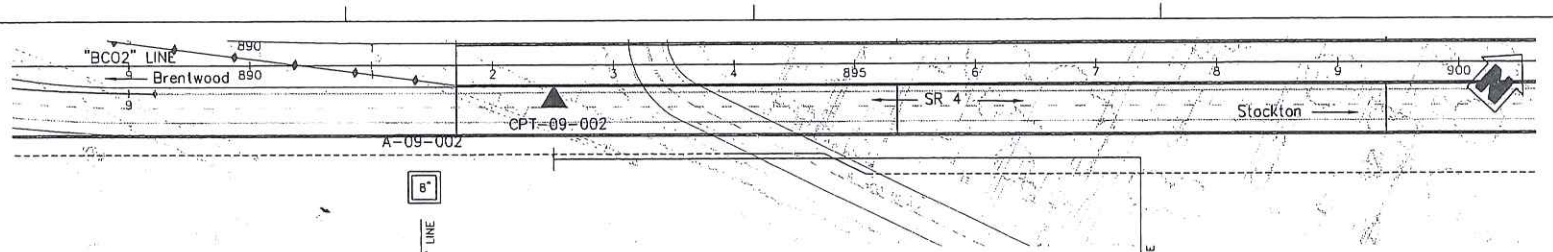
DESIGN OVERSIGHT	DRAWN BY	Amit Bharadwaj	L. S. Bhangoo	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	Gary Parikh	BRIDGE NO. –	ROUTE 4 CROSSTOWN CONNECTOR
	CHECKED BY	David Wang	FIELD INVESTIGATION BY: DATE: November 2009		PROJECT ENGINEER	POST MILES –	LOG OF TEST BORINGS
SIGN OFF DATE					CU – EA –	DISREGARD PRINTS BEARING EARLIER REVISION DATES	REVISION DATES / SHEET / OF

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

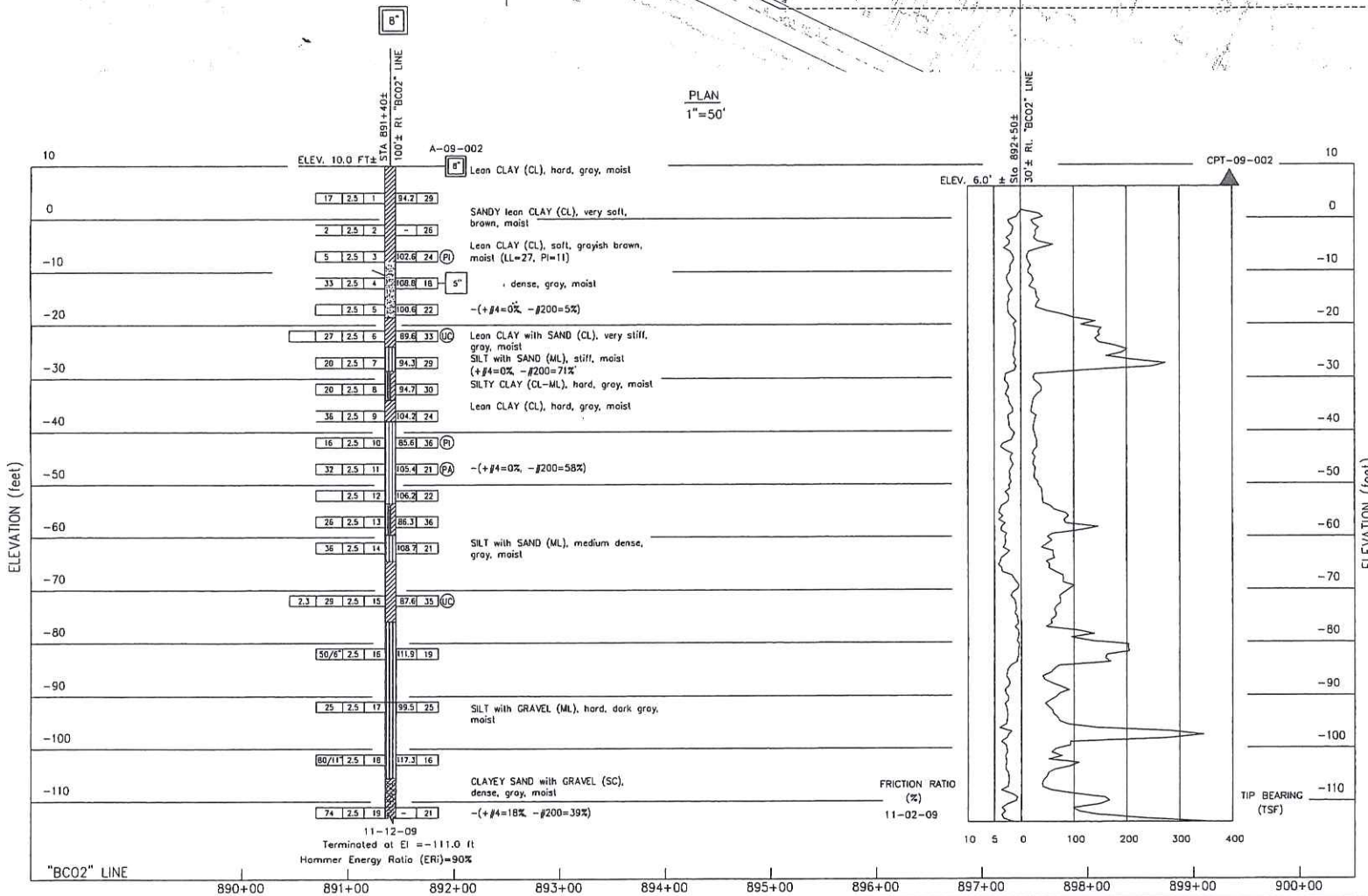
FILE => $REQUEST

OGS GEOTECHNICAL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 08-01-09)

PLATE A-4



PLAN
1"=50'



DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	4	R12.6/R15.7		

GEOTECHNICAL PROFESSIONAL DATE

GARY PARIKH
No. G.E. 666
Exp. 12/31/11
GEOTECHNICAL
REGISTERED PROFESSIONAL ENGINEER
STATE OF CALIFORNIA

PLANS APPROVAL DATE

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131

SAN JOAQUIN COUNCIL OF GOVERNMENTS
555 EAST WEBER AVENUE
STOCKTON, CALIFORNIA 95202

BKF ENGINEERS
4670 WILLOW ROAD, SUITE 250
PLEASANTON CALIFORNIA 94588

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DRAFT

Notes:
Modified California Sampler: I.D. = 2.5"; O.D. = 3", Hammer Assembly: A 140 lb hammer with a 30" drop, (Automatic Hammer)

This LOTB sheet was prepared in accordance with the Caltrans Soil & Rock, Logging, Classification, and Presentation Manual (June 2007)

All dimensions are in feet unless otherwise shown

PROFILE
Vert. : 1" = 10'
Hor. : 1" = 50'

DESIGN OVERSIGHT	DRAWN BY	Amit Bharadwaj	L. S. Bhangoo	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	Gary Parikh
SIGN OFF DATE	CHECKED BY	David Wang	FIELD INVESTIGATION BY: DATE: November 2009		PROJECT ENGINEER
					CU - EA -

BRIDGE NO. -
POST MILES -

ROUTE 4 CROSSTOWN CONNECTOR
LOG OF TEST BORINGS

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES | SHEET | OF

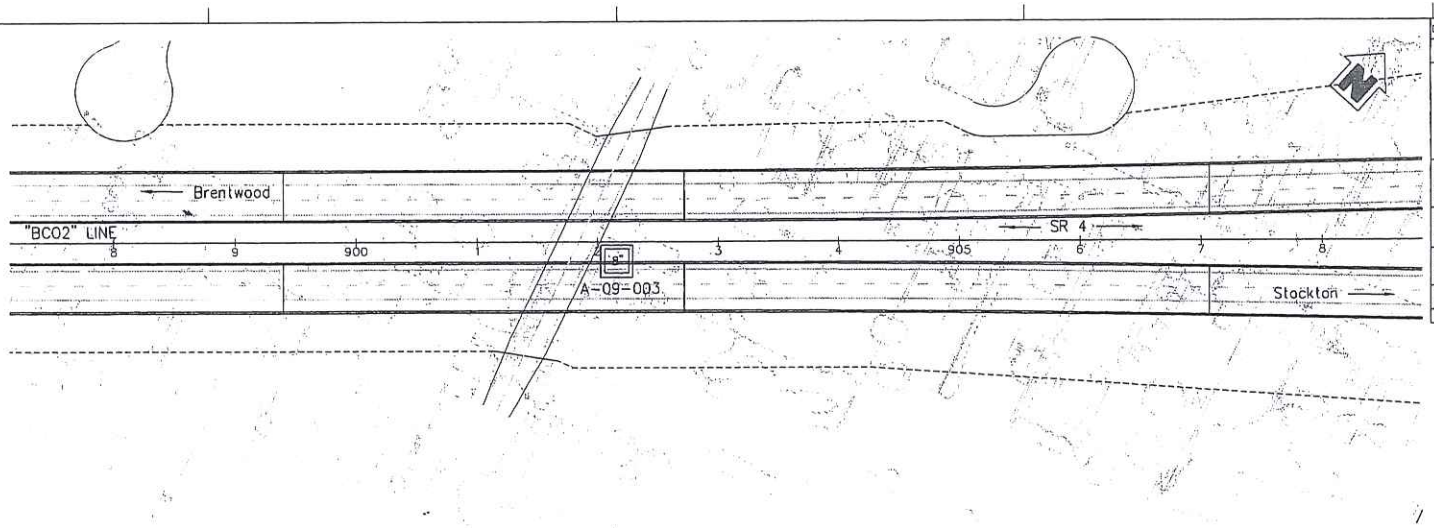
DGS GEOTECHNICAL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 08-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

FILE => $REQUEST

USERNAME => $USER DATE PLOTTED => $DATE TIME PLOTTED => $TIME

PLATE A-5



DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	4	R12.6/R15.7		

GEOTECHNICAL PROFESSIONAL DATE

REGISTERED PROFESSIONAL ENGINEER — GARY PARIKH — No. G.E. 666 — Exp 12/31/11 — GEOTECHNICAL — STATE OF CALIFORNIA

PLANS APPROVAL DATE

PARIKH CONSULTANTS, INC.
2360 OAKE DRIVE, SUITE A
SAN JOSE, CA 95131

SAN JOAQUIN COUNCIL OF GOVERNMENTS
555 EAST WEBER AVENUE
STOCKTON, CALIFORNIA 95202

BKF ENGINEERS
4670 WILLOW ROAD, SUITE 250
PLESANTON CALIFORNIA 94588

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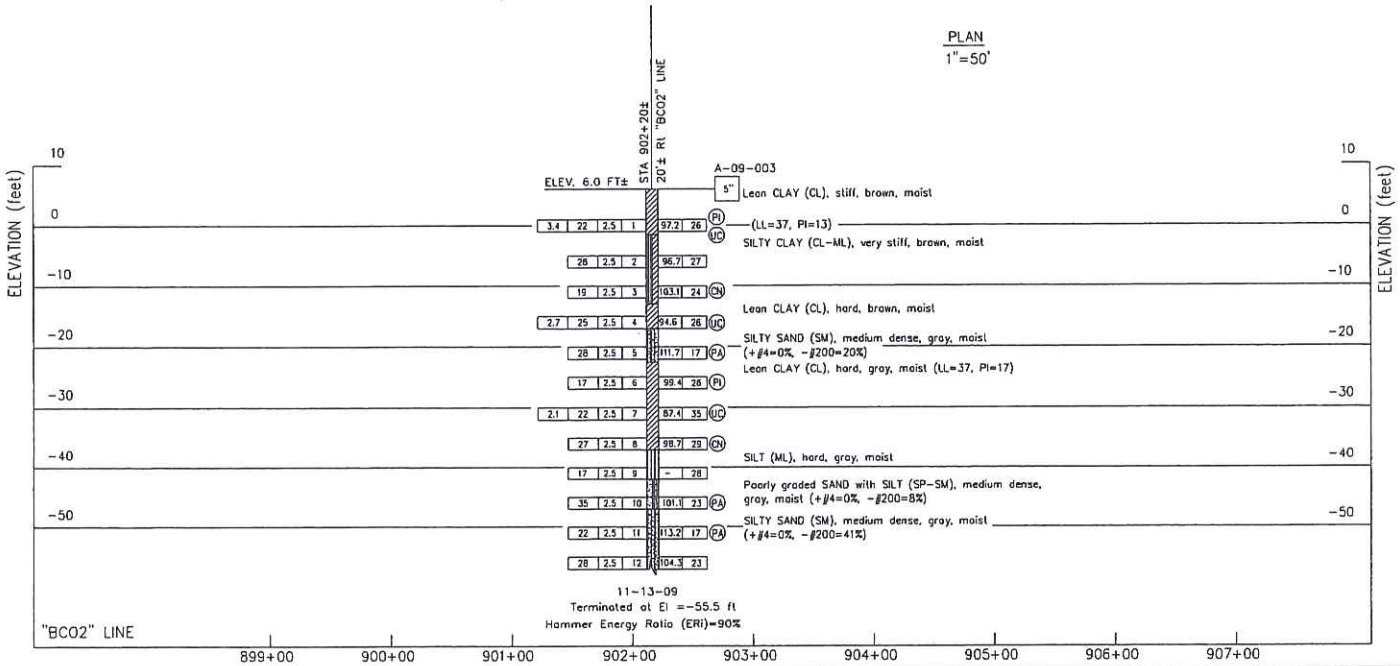
To get to the Caltrans web site, go to: http://www.dot.ca.gov

DRAFT

PLAN
1"=50'



Notes:
Modified California Sampler: I.D. = 2.5"; O.D. = 3", Hammer Assembly: A 140 lb hammer with a 30" drop, (Automatic Hammer)

This LOTB sheet was prepared in accordance with the Caltrans Soil & Rock, Logging, Classification, and Presentation Manual (June 2007)

All dimensions are in feet unless otherwise shown

PROFILE
Vert. : 1" = 10'
Hor. : 1" = 50'

DESIGN OVERSIGHT	DRAWN BY	Amit Bharadwaj	L. S. Bhangoo	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	Gary Parikh	BRIDGE NO. -
SIGN OFF DATE	CHECKED BY	David Wang	FIELD INVESTIGATION BY: DATE: November 2009		PROJECT ENGINEER	POST MILES -

ROUTE 4 CROSSTOWN CONNECTOR
LOG OF TEST BORINGS

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES | SHEET | OF

CU -
EA -

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

DGS GEOTECHNICAL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 06-01-09)

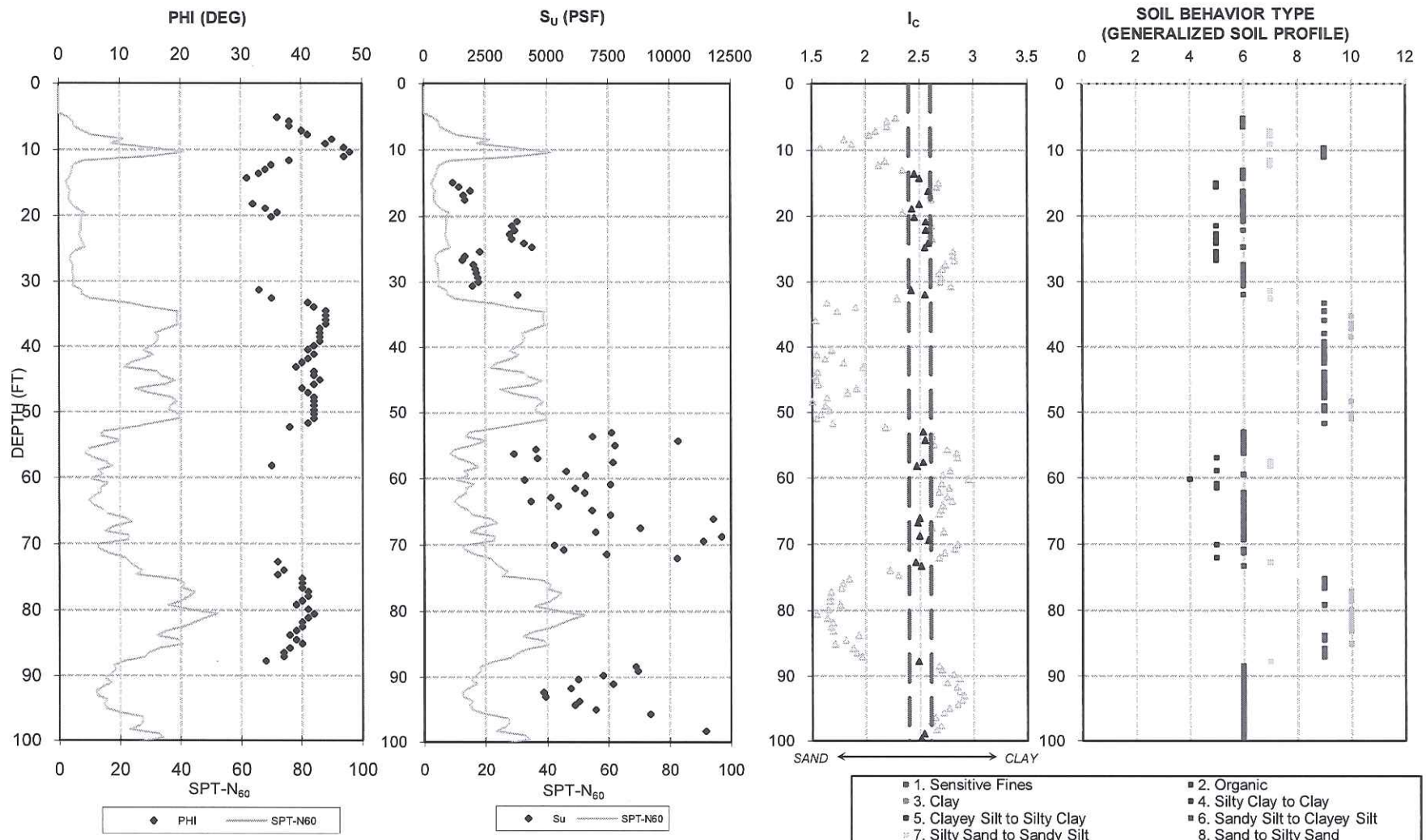
FILE => $REQUEST

USERNAME => $USER DATE PLOTTED => $DATE TIME PLOTTED => $TIME

Cone Penetrometer Test Profiles

CPT DATA CORRELATION

CPT-09-001



Reference: CPT data and correlation provided by Gregg Drilling and Testing, Inc.

PARIKH

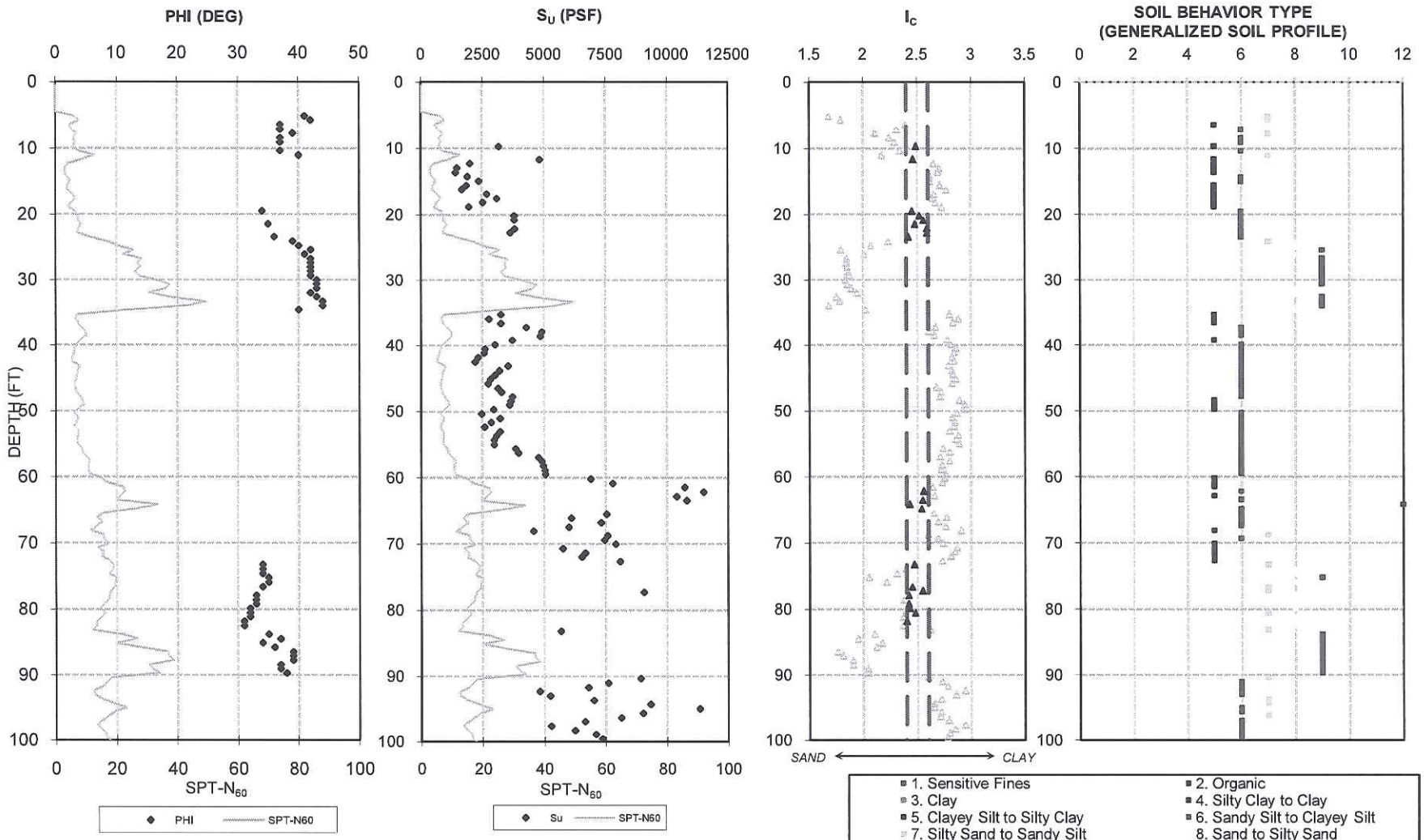
Practicing in the Geosciences

ROUTE 4 CROSSTOWN CONNECTOR
SAN JOAQUIN COUNTY, CALIFORNIA

JOB NO.:	PLATE NO.:

CPT DATA CORRELATION

CPT-09-002



Reference: CPT data and correlation provided by Gregg Drilling and Testing, Inc.

PARIKH

Practicing in the Geosciences

ROUTE 4 CROSSTOWN CONNECTOR
SAN JOAQUIN COUNTY, CALIFORNIA

JOB NO.:	PLATE NO.:

APPENDIX C
RESULTS OF LABORATORY TESTS
(KLEINFELDER INVESTIGATION)

LIST OF ATTACHMENTS

The following are attached and complete this appendix.

Unconfined Compression Tests
Plasticity Charts (Atterberg Limits)
Consolidation Test Results
Sulfate and Chloride Contents by California Laboratory Service

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Unconfined Compression Tests

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

KLEINFELDER

Tested By: M. Baishiki — Date: 3/14/2011

Computed By: M. Baishiki — Date: 3/15/2011

Checked By: Christine Erickson — Date: 3/15/2011

Before Test	Specimen A
Water Content (%)	21.43
Dry Density (pcf)	106.80
Saturation (%)	98.92
Void Ratio	0.59
Diameter (in)	1.91
Height (in)	3.99
Test Data	**A**
Unconfined Strength (ksf)	5.08
Undrained Shear Strength (tsf)	1.27
Undrained Shear Strength (ksf)	2.54
Rate of Strain (in/min)	0.08
Strain at Failure (%)	12.07
Sample Location	B1-15-1

Specimen

Project Information		Specimen Description	
Project Num	111904.G01	Specimen A	Light Brown Lean Clay
Project	Crosstown Freeway SR-4		
Sampling Date	2/8/2011		
Sample #	1103017		
Client		**Test Variables**	
		Specific Gravity	2.72

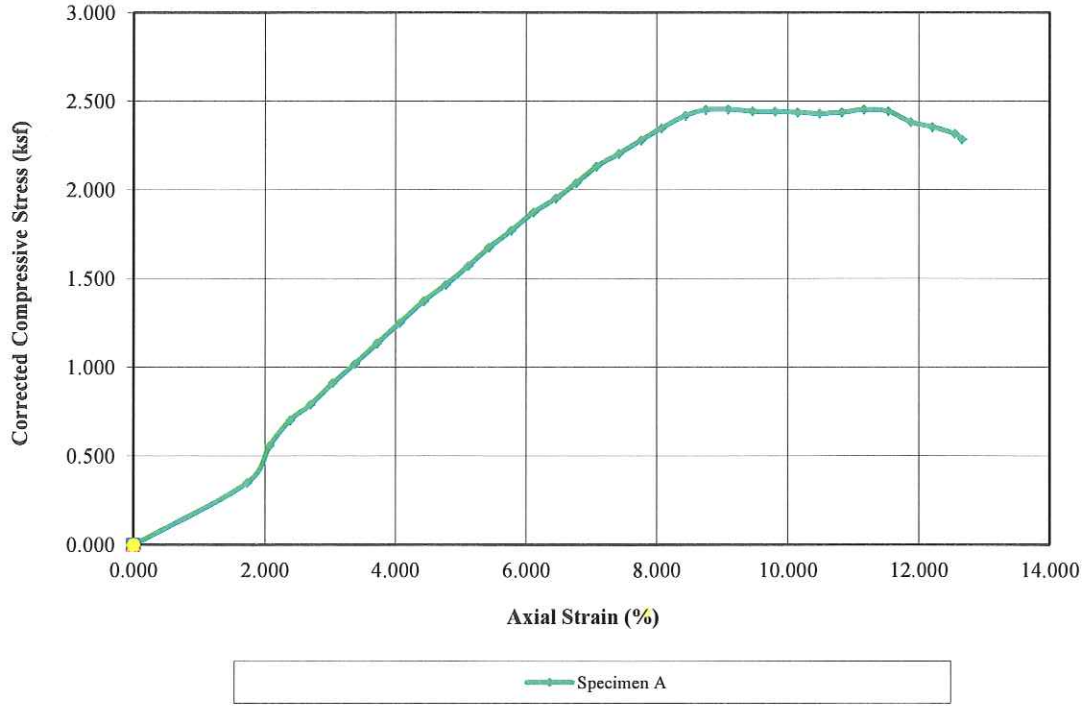
Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

Tested By: M. Baishiki — Date: 3/14/2011
Computed By: M. Baishiki — Date: 3/15/2011
Checked By — Date

Compressive Stress Axial Strain Curve



Before Test	Specimen A
Water Content (%)	33.25
Dry Density (pcf)	88.90
Saturation (%)	99.44
Void Ratio	0.91
Diameter (in)	1.91
Height (in)	3.99
Test Data	A
Unconfined Strength (ksf)	2.45
Undrained Shear Strength (tsf)	0.61
Undrained Shear Strength (ksf)	1.23
Rate of Strain (in/min)	0.08
Strain at Failure (%)	9.09
Sample Location	B1-80-1

Specimen

Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	2/8/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Gray Lean Clay

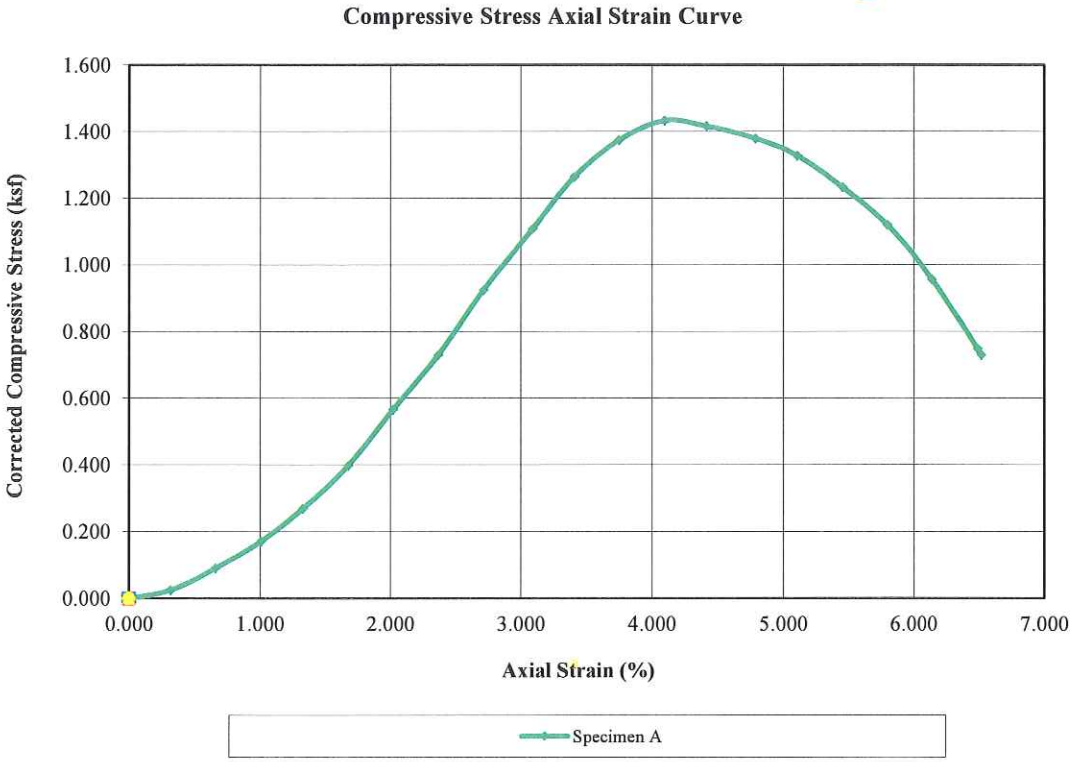
Test Variables	
Specific Gravity	2.72

Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

Tested By: M. Baishiki Date: 3/14/2011
Computed By: M. Baishiki Date: 3/15/2011
Checked By Date



Before Test	Specimen A
Water Content (%)	27.69
Dry Density (pcf)	95.30
Saturation (%)	98.27
Void Ratio	0.76
Diameter (in)	1.91
Height (in)	3.93
Test Data	**A**
Unconfined Strength (ksf)	1.43
Undrained Shear Strength (tsf)	0.36
Undrained Shear Strength (ksf)	0.72
Rate of Strain (in/min)	0.08
Strain at Failure (%)	4.10
Sample Location	B2-30-1

Specimen

Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	2/9/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Greenish Gray Clayey Sand

Test Variables	
Specific Gravity	2.68

Remarks

Kleinfelder

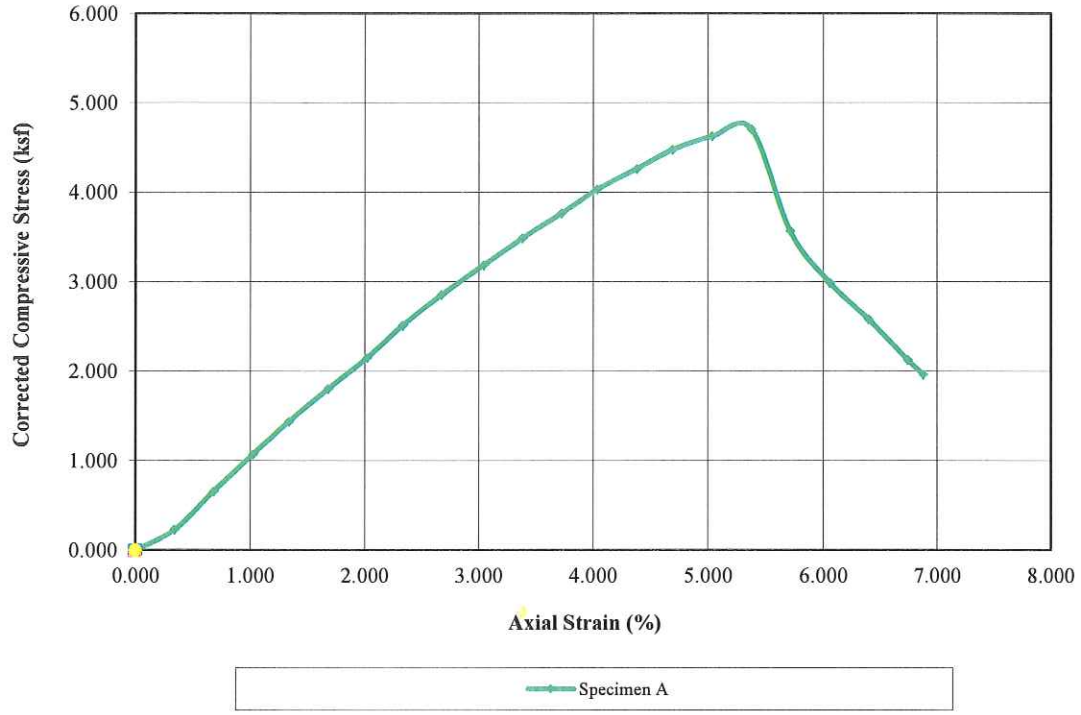
Unconfined Compression Test Report (ASTM D2166)

Tested By: M. Baishiki — Date: 3/14/2011

Computed By: M. Baishiki — Date: 3/15/2011

Checked By — Date

Compressive Stress Axial Strain Curve



Before Test	Specimen A
Water Content (%)	25.01
Dry Density (pcf)	100.40
Saturation (%)	98.46
Void Ratio	0.69
Diameter (in)	1.90
Height (in)	3.99
Test Data	**A**
Unconfined Strength (ksf)	4.71
Undrained Shear Strength (tsf)	1.18
Undrained Shear Strength (ksf)	2.35
Rate of Strain (in/min)	0.08
Strain at Failure (%)	5.37
Sample Location	B2-90-1

Specimen

Project Information		Specimen Description	
Project Num	111904.G01	Specimen A	Brownish Gray Lean Clay
Project	Crosstown Freeway SR-4		
Sampling Date	2/9/2011		
Sample #	1103017		
Client		**Test Variables**	
		Specific Gravity	2.72

Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

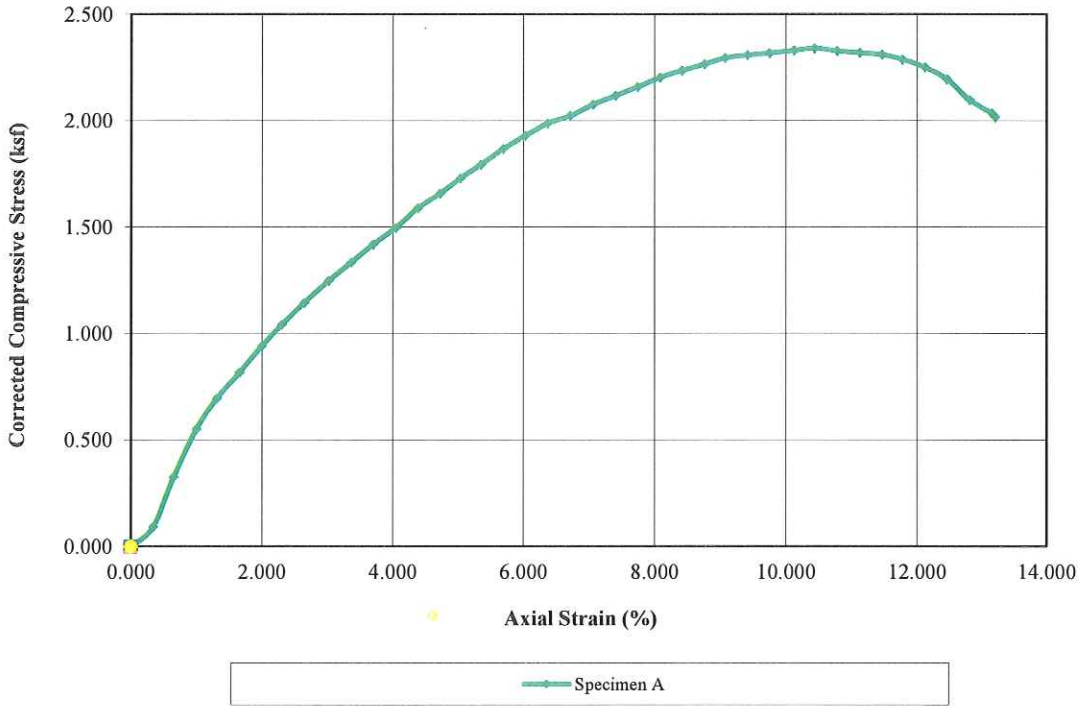
KLEINFELDER

Tested By: M. Baishiki — Date: 3/17/2011

Computed By: M. Baishiki — Date: 3/18/2011

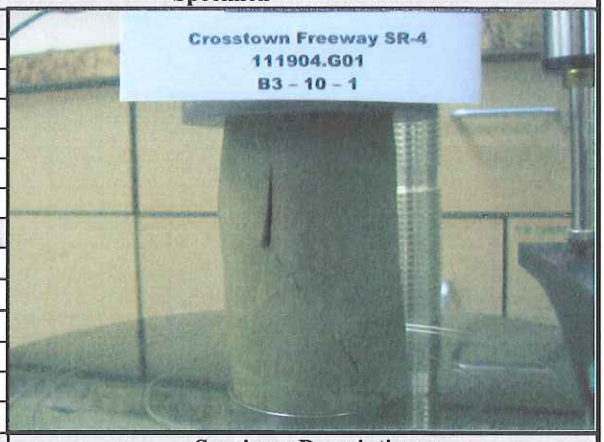
Checked By — Date

Compressive Stress Axial Strain Curve



Before Test	Specimen A
Water Content (%)	24.16
Dry Density (pcf)	102.22
Saturation (%)	99.38
Void Ratio	0.66
Diameter (in)	1.92
Height (in)	3.99
Test Data	**A**
Unconfined Strength (ksf)	2.34
Undrained Shear Strength (tsf)	0.59
Undrained Shear Strength (ksf)	1.17
Rate of Strain (in/min)	0.08
Strain at Failure (%)	10.44
Sample Location	B3-10-1

Specimen



Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	2/10/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Light Brown Lean Clay
Test Variables	
Specific Gravity	2.72

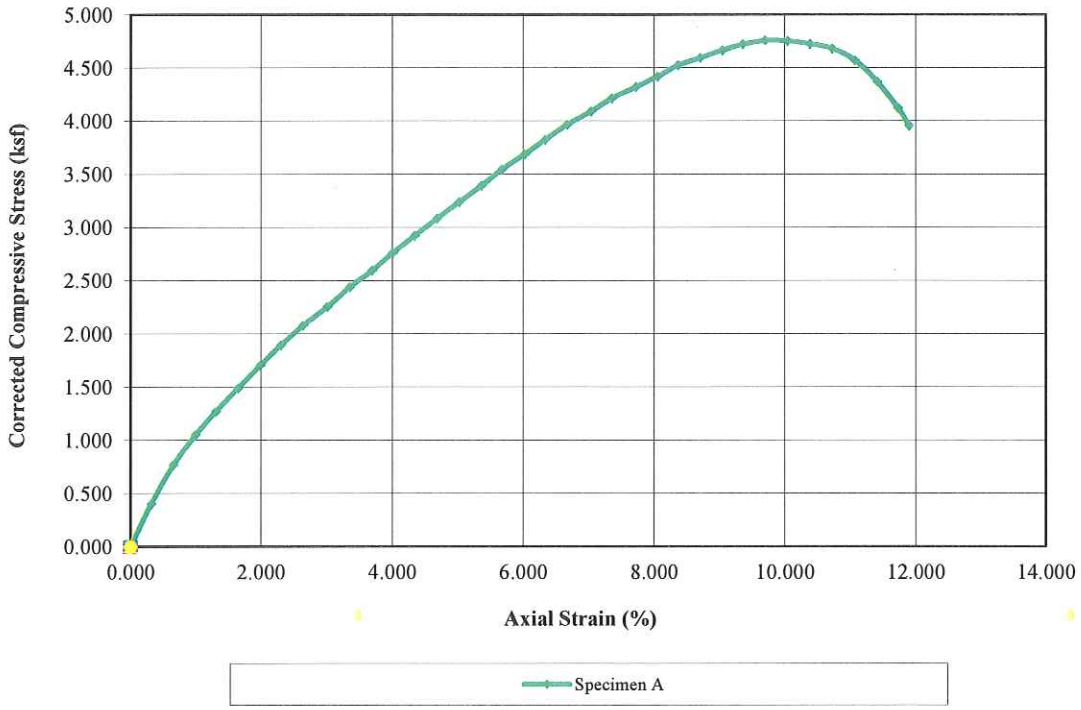
Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

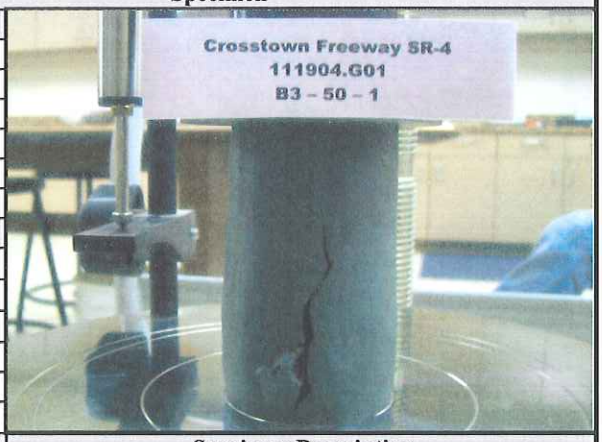
Date
Checked By
3/18/2011
Date
Computed By M. Baishiki
3/17/2011
Date
Tested By M. Baishiki

Compressive Stress Axial Strain Curve



Before Test	Specimen A
Water Content (%)	27.60
Dry Density (pcf)	96.60
Saturation (%)	99.16
Void Ratio	0.76
Diameter (in)	1.90
Height (in)	4.00
Test Data	**A**
Unconfined Strength (ksf)	4.76
Undrained Shear Strength (tsf)	1.19
Undrained Shear Strength (ksf)	2.38
Rate of Strain (in/min)	0.08
Strain at Failure (%)	9.71
Sample Location	B3-50-1

Specimen



Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	2/10/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Greenish Gray Lean Clay
Test Variables	
Specific Gravity	2.72

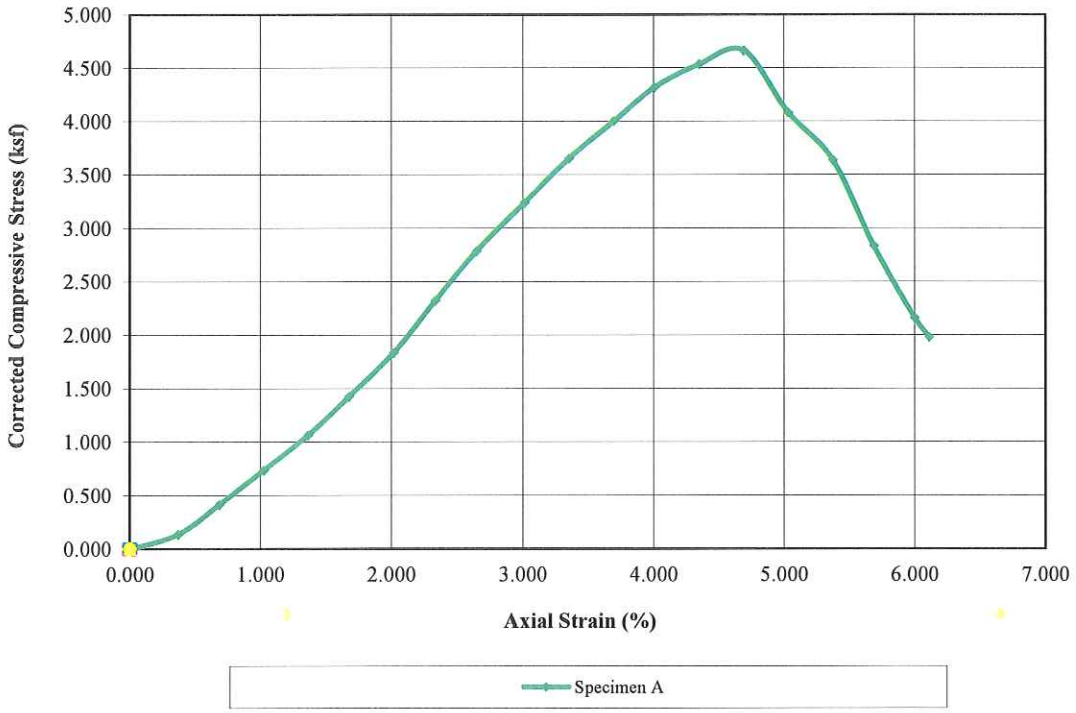
Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

Tested By: M. Baishiki Date: 3/17/2010
Computed By: M. Baishiki Date: 3/18/2011
Checked By: Date:

Compressive Stress Axial Strain Curve



Before Test	Specimen A
Water Content (%)	27.04
Dry Density (pcf)	97.48
Saturation (%)	99.12
Void Ratio	0.74
Diameter (in)	1.91
Height (in)	3.99
Test Data	**A**
Unconfined Strength (ksf)	4.67
Undrained Shear Strength (tsf)	1.17
Undrained Shear Strength (ksf)	2.33
Rate of Strain (in/min)	0.08
Strain at Failure (%)	4.69
Sample Location	B3-120-1

Specimen

Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	2/11/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Greenish Gray Lean Clay

Test Variables	
Specific Gravity	2.72

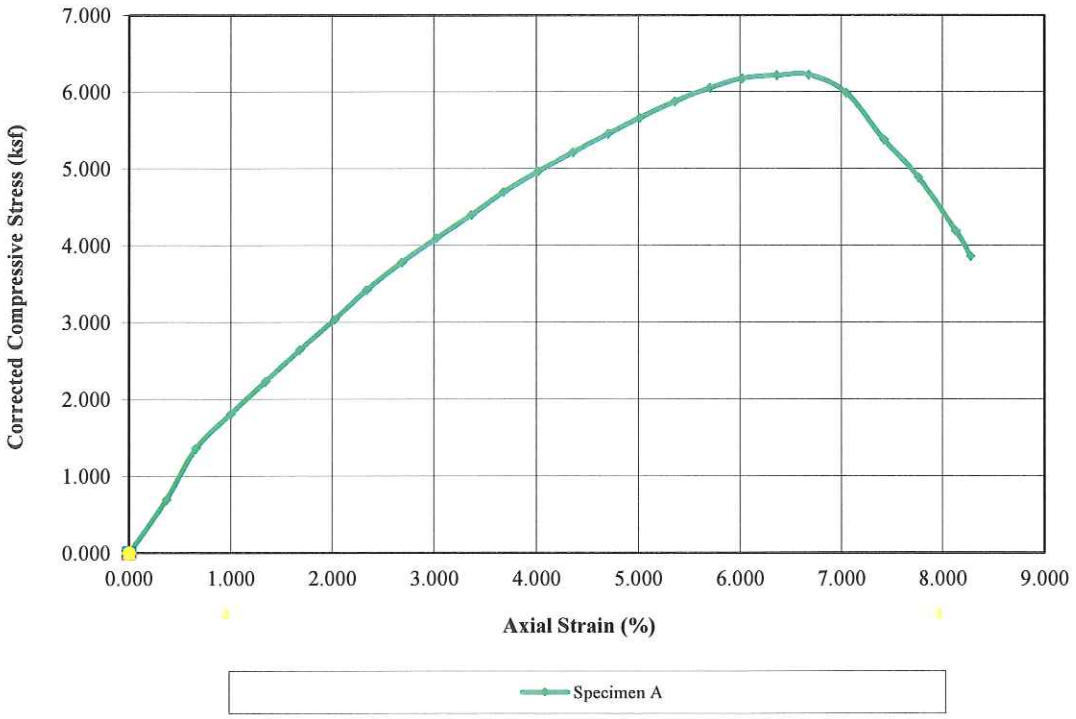
Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

Tested By: M. Baishiki — Date: 3/28/2011
Computed By: M. Baishiki — Date: 3/29/2011
Checked By — Date

Compressive Stress Axial Strain Curve



Before Test	Specimen A
Water Content (%)	24.88
Dry Density (pcf)	101.30
Saturation (%)	100.01
Void Ratio	0.68
Diameter (in)	1.92
Height (in)	3.98
Test Data	**A**
Unconfined Strength (ksf)	6.22
Undrained Shear Strength (tsf)	1.56
Undrained Shear Strength (ksf)	3.11
Rate of Strain (in/min)	0.08
Strain at Failure (%)	6.68
Sample Location	B3-140-1

Specimen

Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	2/11/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Gray Lean Clay

Test Variables	
Specific Gravity	2.72

Remarks

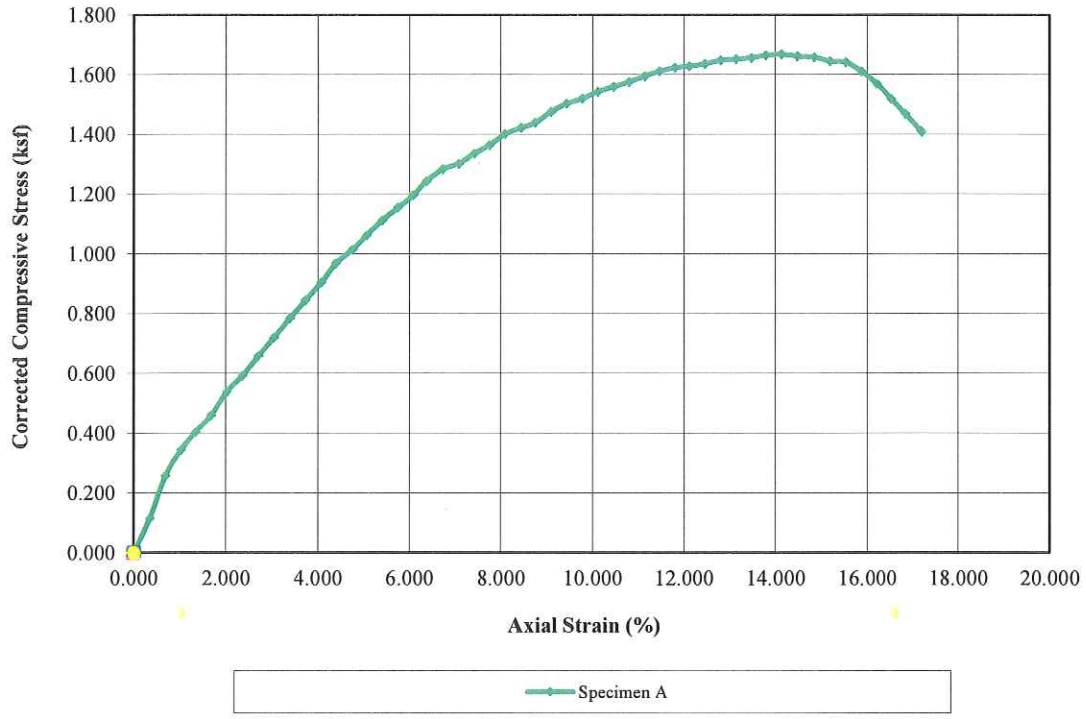
Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

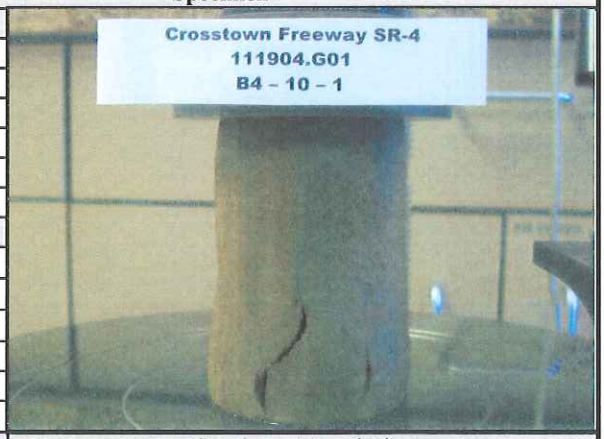
KLEINFELDER

Tested By: M. Baishiki — Date: 3/30/2011
Computed By: M. Baishiki — Date: 3/31/2011
Checked By — Date

Compressive Stress Axial Strain Curve



Before Test	Specimen A
Water Content (%)	20.87
Dry Density (pcf)	107.50
Saturation (%)	99.22
Void Ratio	0.57
Diameter (in)	1.92
Height (in)	3.99
Test Data	**A**
Unconfined Strength (ksf)	1.67
Undrained Shear Strength (tsf)	0.42
Undrained Shear Strength (ksf)	0.83
Rate of Strain (in/min)	0.08
Strain at Failure (%)	14.13
Sample Location	B4-10-1



Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	2/3/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Brown Sandy Lean Clay

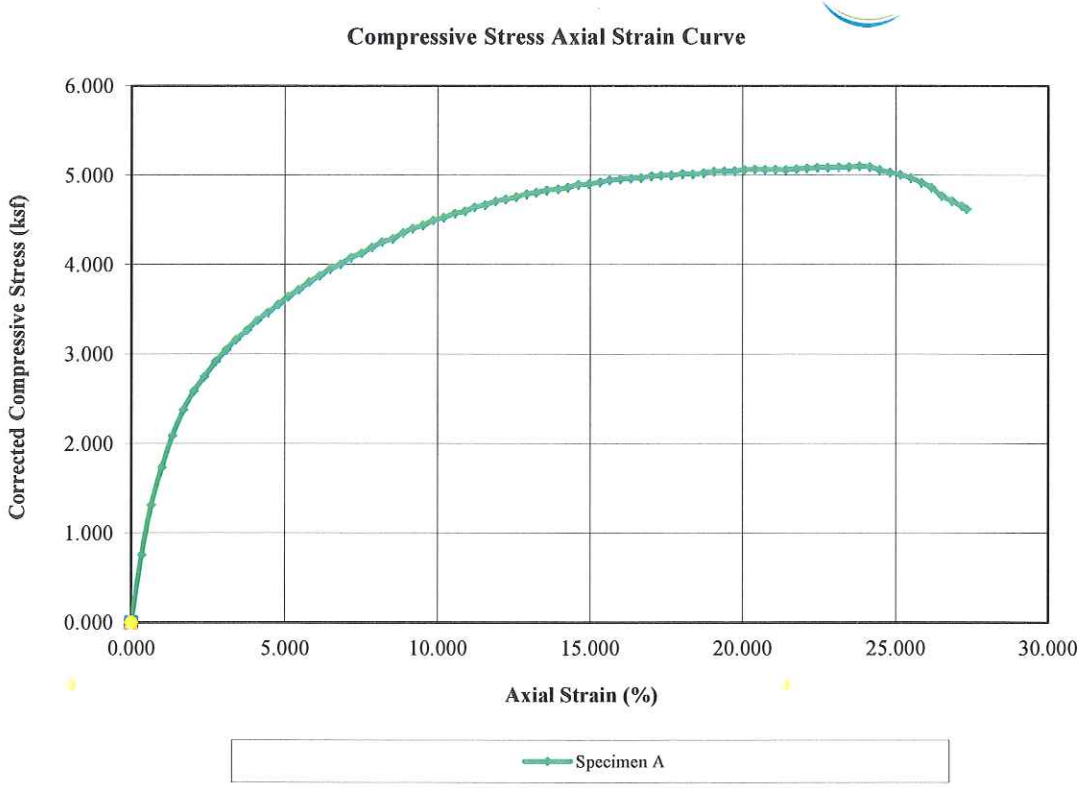
Test Variables	
Specific Gravity	2.70

Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

Date: 3/30/2011
Tested By: M. Baishiki
Computed By: M. Baishiki
Date: 3/31/2011
Checked By:



Before Test	Specimen A
Water Content (%)	23.11
Dry Density (pcf)	103.05
Saturation (%)	98.16
Void Ratio	0.64
Diameter (in)	1.92
Height (in)	3.98
Test Data	**A**
Unconfined Strength (ksf)	5.10
Undrained Shear Strength (tsf)	1.28
Undrained Shear Strength (ksf)	2.55
Rate of Strain (in/min)	0.08
Strain at Failure (%)	23.81
Sample Location	B4-35-1

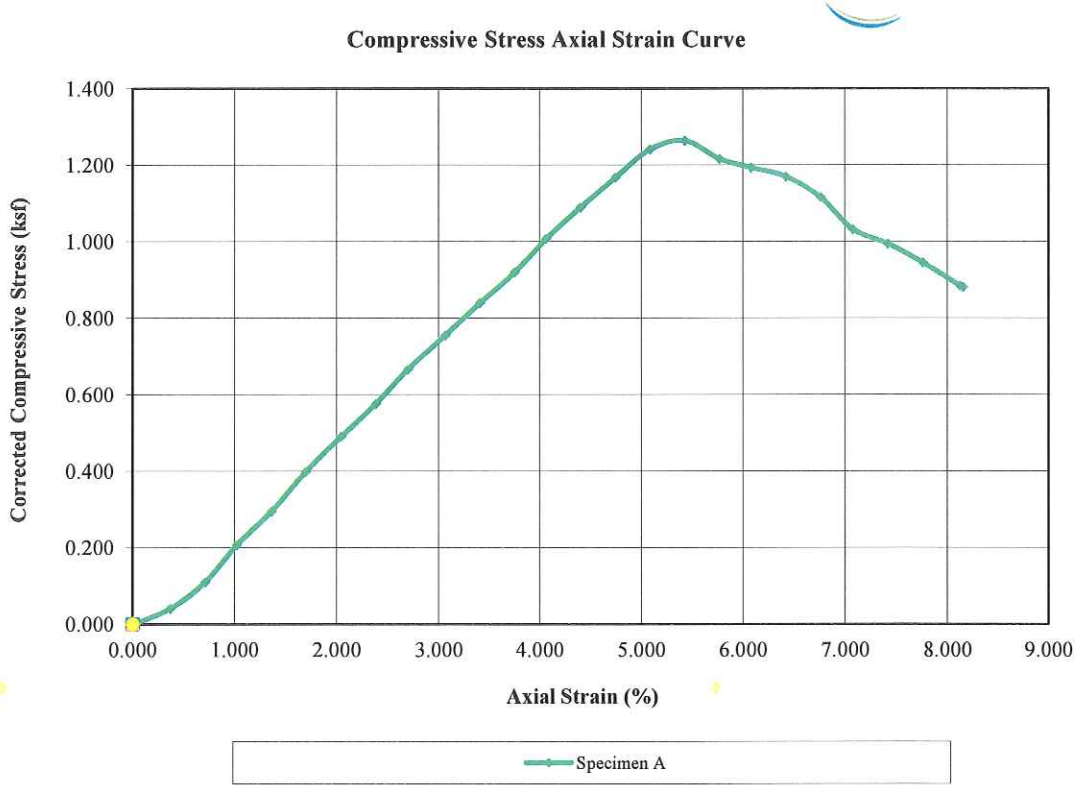
Project Information		Specimen Description	
Project Num	111904.G01	Specimen A	Brownish Gray Lean Clay w/ Sand
Project	Crosstown Freeway SR-4		
Sampling Date	2/3/2011		
Sample #	1103017		
Client		**Test Variables**	
		Specific Gravity	2.70

Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

Tested By: M. Baishiki — Date: 3/30/2011
Computed By: M. Baishiki — Date: 3/31/2011
Checked By — Date



Before Test	Specimen A
Water Content (%)	27.78
Dry Density (pcf)	96.70
Saturation (%)	99.90
Void Ratio	0.76
Diameter (in)	1.92
Height (in)	3.99
Test Data	**A**
Unconfined Strength (ksf)	1.26
Undrained Shear Strength (tsf)	0.32
Undrained Shear Strength (ksf)	0.63
Rate of Strain (in/min)	0.08
Strain at Failure (%)	5.43
Sample Location	B4-65-1

Project Information		Specimen Description	
Project Num	111904.G01	Specimen A	Brownish Gray Silt w/ Sand
Project	Crosstown Freeway SR-4		
Sampling Date	2/3/2011		
Sample #	1103017		
Client		**Test Variables**	
		Specific Gravity	2.72

Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

KLEINFELDER

Date

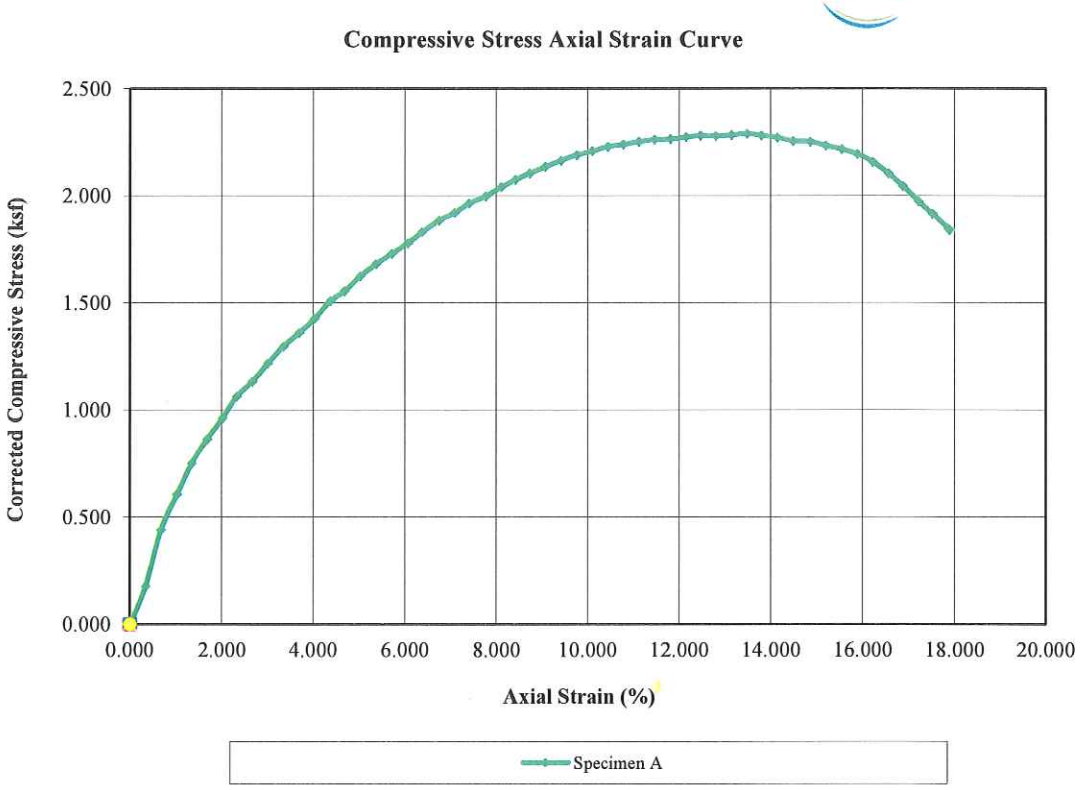
Checked By

3/18/2011 Date

M. Baishiki Computed By

3/17/2011 Date

M. Baishiki Tested By



Before Test	Specimen A
Water Content (%)	20.94
Dry Density (pcf)	108.10
Saturation (%)	99.70
Void Ratio	0.57
Diameter (in)	1.91
Height (in)	3.99
Test Data	**A**
Unconfined Strength (ksf)	2.29
Undrained Shear Strength (tsf)	0.57
Undrained Shear Strength (ksf)	1.14
Rate of Strain (in/min)	0.08
Strain at Failure (%)	13.49
Sample Location	B5-20-1

Project Information		Specimen Description	
Project Num	111904.G01	Specimen A	Brown Lean Clay
Project	Crosstown Freeway SR-4		
Sampling Date	2/2/2011		
Sample #	1103017		
Client		**Test Variables**	
		Specific Gravity	2.72

Remarks

Kleinfelder

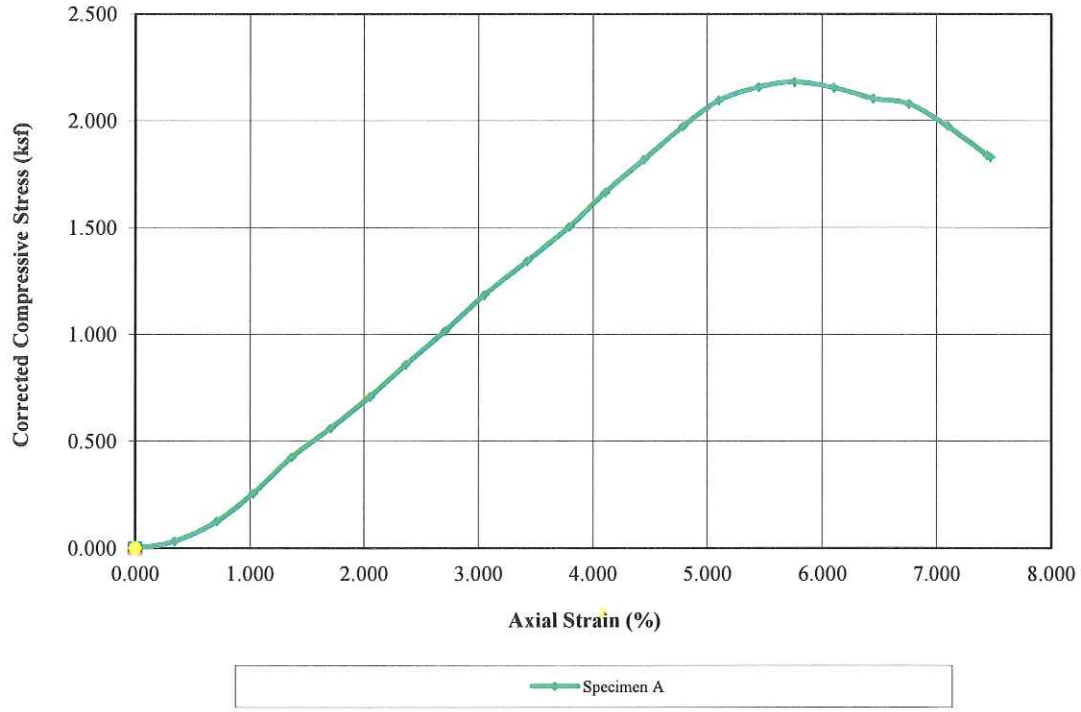
Unconfined Compression Test Report (ASTM D2166)

Tested By: M. Baishiki Date: 3/17/2011

Computed By: M. Baishiki Date: 3/18/2011

Checked By: Date:

Compressive Stress Axial Strain Curve



Before Test	Specimen A
Water Content (%)	20.02
Dry Density (pcf)	110.10
Saturation (%)	99.84
Void Ratio	0.55
Diameter (in)	1.92
Height (in)	3.98
Test Data	**A**
Unconfined Strength (ksf)	2.18
Undrained Shear Strength (tsf)	0.55
Undrained Shear Strength (ksf)	1.09
Rate of Strain (in/min)	0.08
Strain at Failure (%)	5.76
Sample Location	B5-60-1

Specimen

Project Information		Specimen Description	
Project Num	111904.G01	Specimen A	Gray Sandy Lean Clay
Project	Crosstown Freeway SR-4		
Sampling Date	2/2/2011		
Sample #	1103017		
Client		**Test Variables**	
		Specific Gravity	2.73

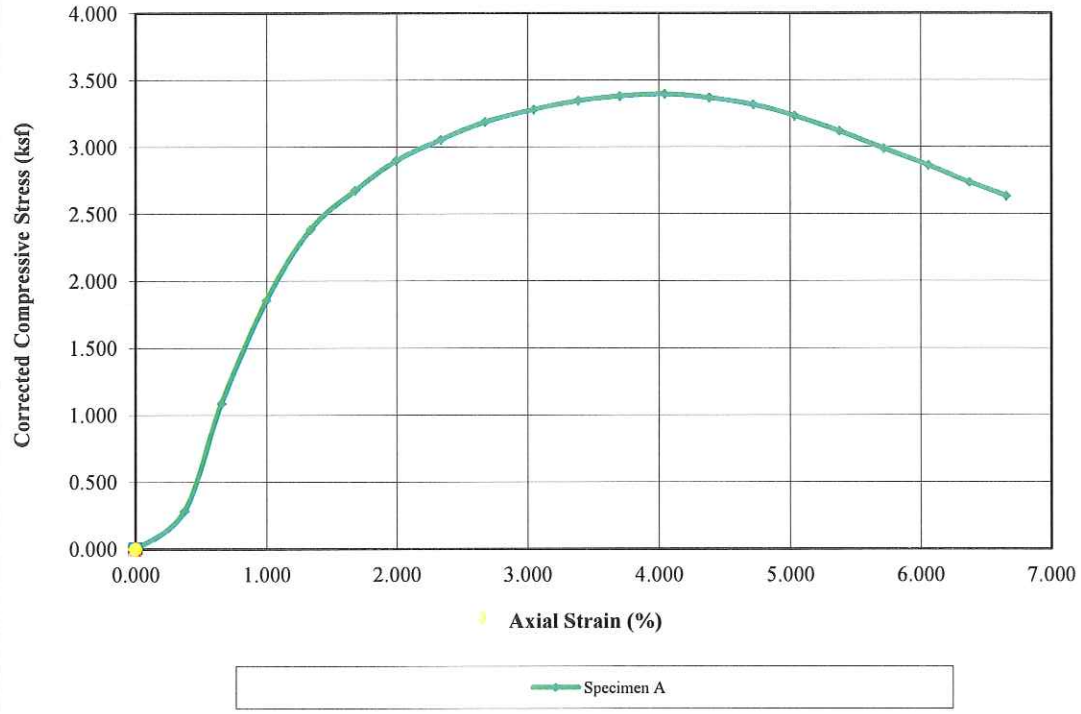
Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

Tested By: M. Baishiki — Date: 3/17/2011
Computed By: M. Baishiki — Date: 3/18/2011
Checked By — Date

Compressive Stress Axial Strain Curve



Before Test	A
Water Content (%)	22.44
Dry Density (pcf)	103.70
Saturation (%)	95.70
Void Ratio	0.64
Diameter (in)	1.90
Height (in)	3.99
Test Data	**A**
Unconfined Strength (ksf)	3.40
Undrained Shear Strength (tsf)	0.85
Undrained Shear Strength (ksf)	1.70
Rate of Strain (in/min)	0.08
Strain at Failure (%)	4.04
Sample Location	B6-15-1

Specimen

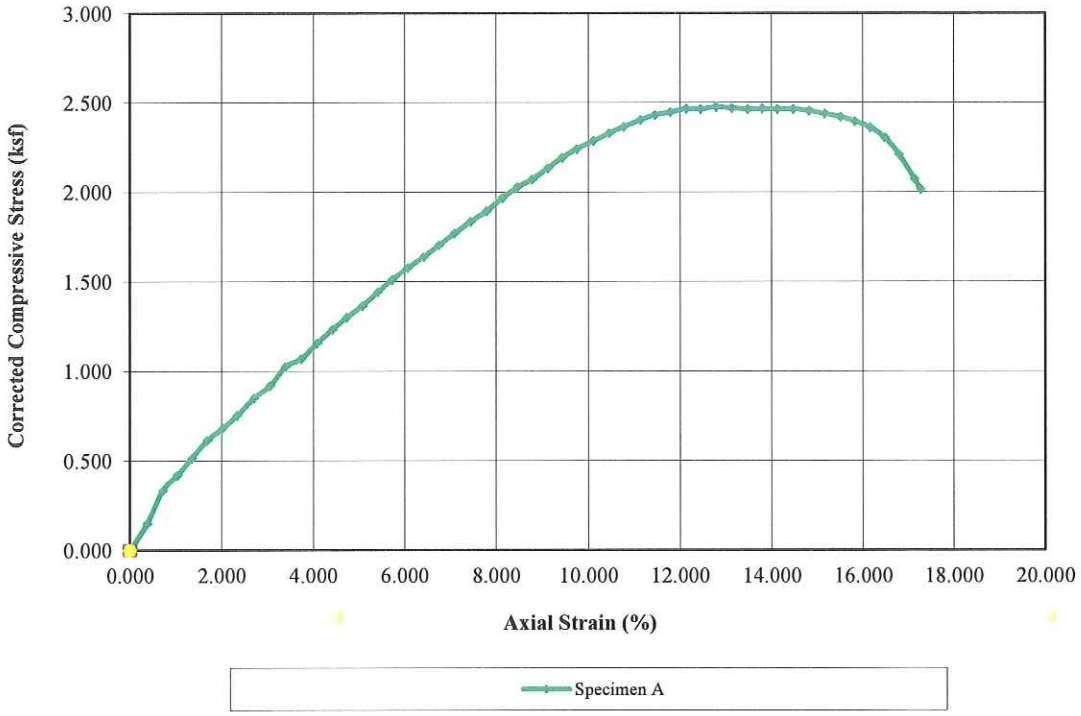
Project Information		Specimen Description	
Project Num	111904.G01	Specimen A	Brown Lean Clay w/ Sand
Project	Crosstown Freeway SR-4		
Sampling Date	2/10/2011		
Sample #	1103017		
Client		**Test Variables**	
		Specific Gravity	2.72

Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

Tested By: M. Baishiki — Date: 3/30/2011
Computed By: M. Baishiki — Date: 3/31/2011
Checked By — Date



Before Test	Specimen A
Water Content (%)	27.66
Dry Density (pcf)	94.60
Saturation (%)	95.63
Void Ratio	0.78
Diameter (in)	1.92
Height (in)	3.98
Test Data	**A**
Unconfined Strength (ksf)	2.48
Undrained Shear Strength (tsf)	0.62
Undrained Shear Strength (ksf)	1.24
Rate of Strain (in/min)	0.08
Strain at Failure (%)	12.80
Sample Location	B6-50-1

Specimen

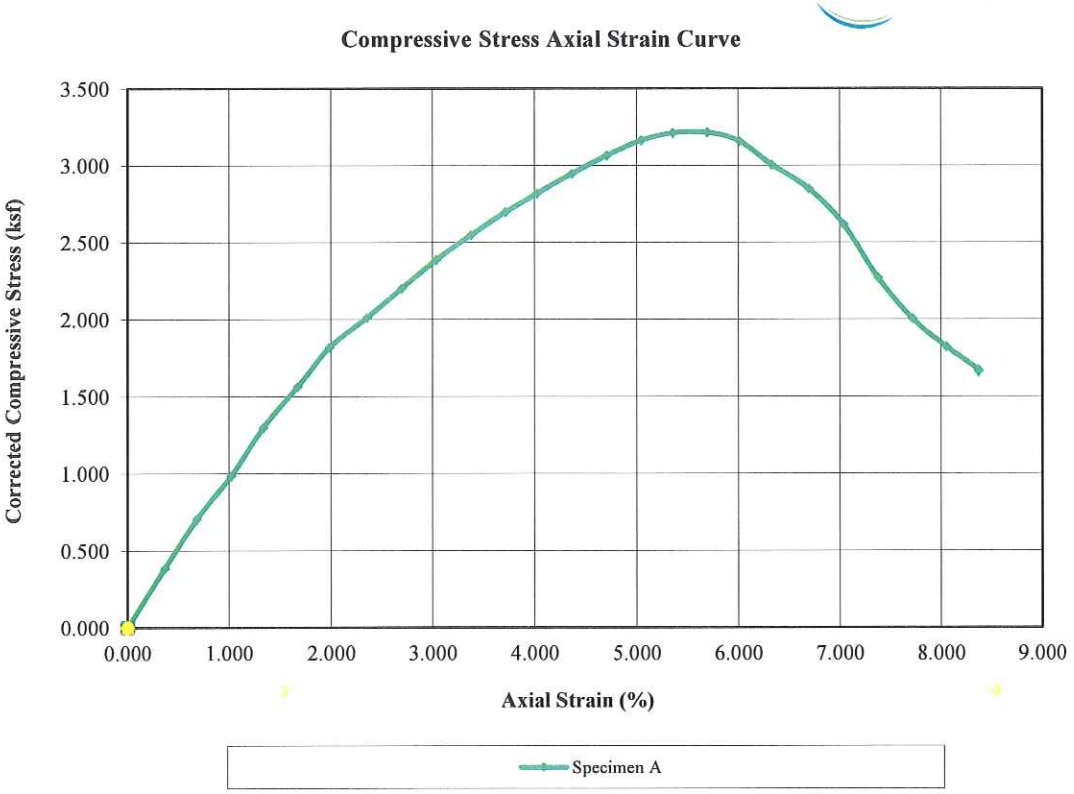
Project Information		Specimen Description	
Project Num	111904.G01	Specimen A	Brownish Gray Lean Clay w/ Sand
Project	Crosstown Freeway SR-4		
Sampling Date	2/1/2011		
Sample #	1103017		
Client		**Test Variables**	
		Specific Gravity	2.70

Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

Date
Checked By
3/31/2011
Date
M. Baishiki
Computed By
3/30/2011
Date
M. Baishiki
Tested By



Before Test	Specimen A
Water Content (%)	33.01
Dry Density (pcf)	89.49
Saturation (%)	99.64
Void Ratio	0.90
Diameter (in)	1.92
Height (in)	4.00
Test Data	**A**
Unconfined Strength (ksf)	3.22
Undrained Shear Strength (tsf)	0.80
Undrained Shear Strength (ksf)	1.61
Rate of Strain (in/min)	0.08
Strain at Failure (%)	5.70
Sample Location	B6-80-1

Project Information		Specimen Description	
Project Num	111904.G01	Specimen A	Gray Lean Clay
Project	Crosstown Freeway SR-4		
Sampling Date	2/1/2011		
Sample #	1103017		
Client		**Test Variables**	
		Specific Gravity	2.73

Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

Date

Checked By

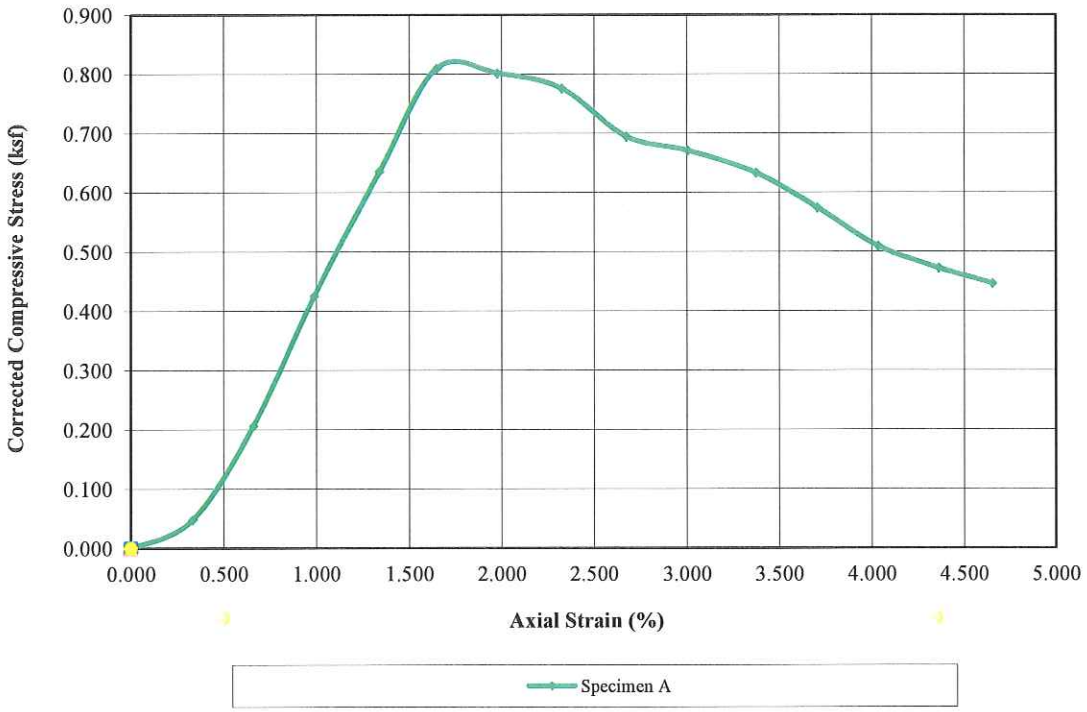
4/1/2011 Date

M. Baishiki Computed By

3/31/2011 Date

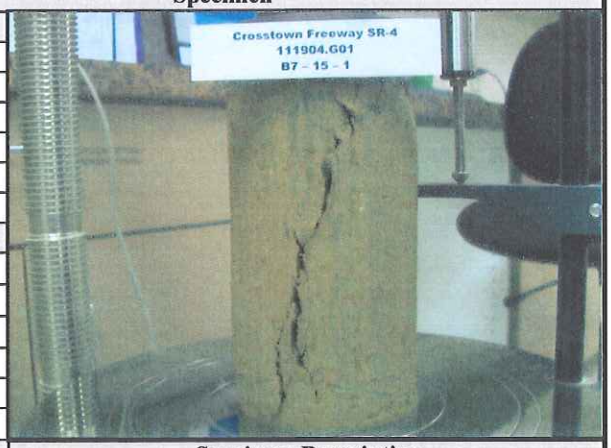
M. Baishiki Tested By

Compressive Stress Axial Strain Curve



Before Test	A
Water Content (%)	28.59
Dry Density (pcf)	94.30
Saturation (%)	98.08
Void Ratio	0.79
Diameter (in)	2.87
Height (in)	5.85
Test Data	**A**
Unconfined Strength (ksf)	0.81
Undrained Shear Strength (tsf)	0.20
Undrained Shear Strength (ksf)	0.41
Rate of Strain (in/min)	0.12
Strain at Failure (%)	1.65
Sample Location	B7-15-1

Specimen



Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	1/31/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Light Brown Lean Clay

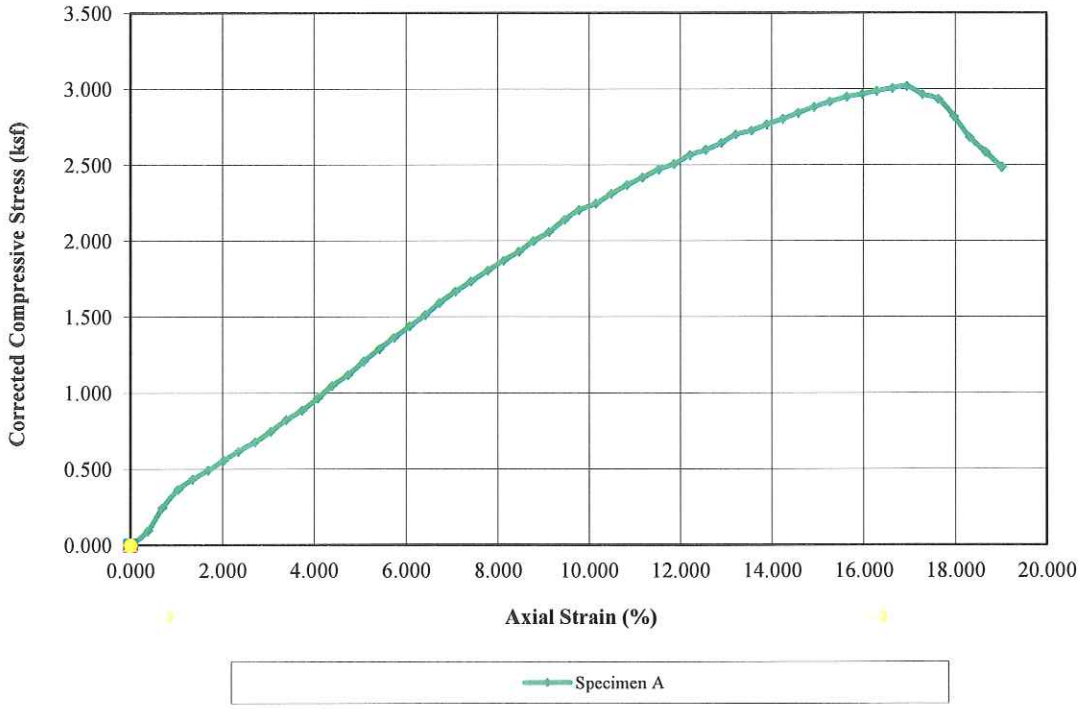
Test Variables	
Specific Gravity	2.70

Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

Date
Checked By
3/31/2011
Date
M. Baishiki
Computed By
3/30/2011
Date
M. Baishiki
Tested By



Before Test	Specimen A
Water Content (%)	21.90
Dry Density (pcf)	105.70
Saturation (%)	99.52
Void Ratio	0.60
Diameter (in)	1.92
Height (in)	3.98
Test Data	**A**
Unconfined Strength (ksf)	3.02
Undrained Shear Strength (tsf)	0.75
Undrained Shear Strength (ksf)	1.51
Rate of Strain (in/min)	0.08
Strain at Failure (%)	16.94
Sample Location	B7-45-1

Specimen

Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	1/31/2011
Sample #	1103017
Client	

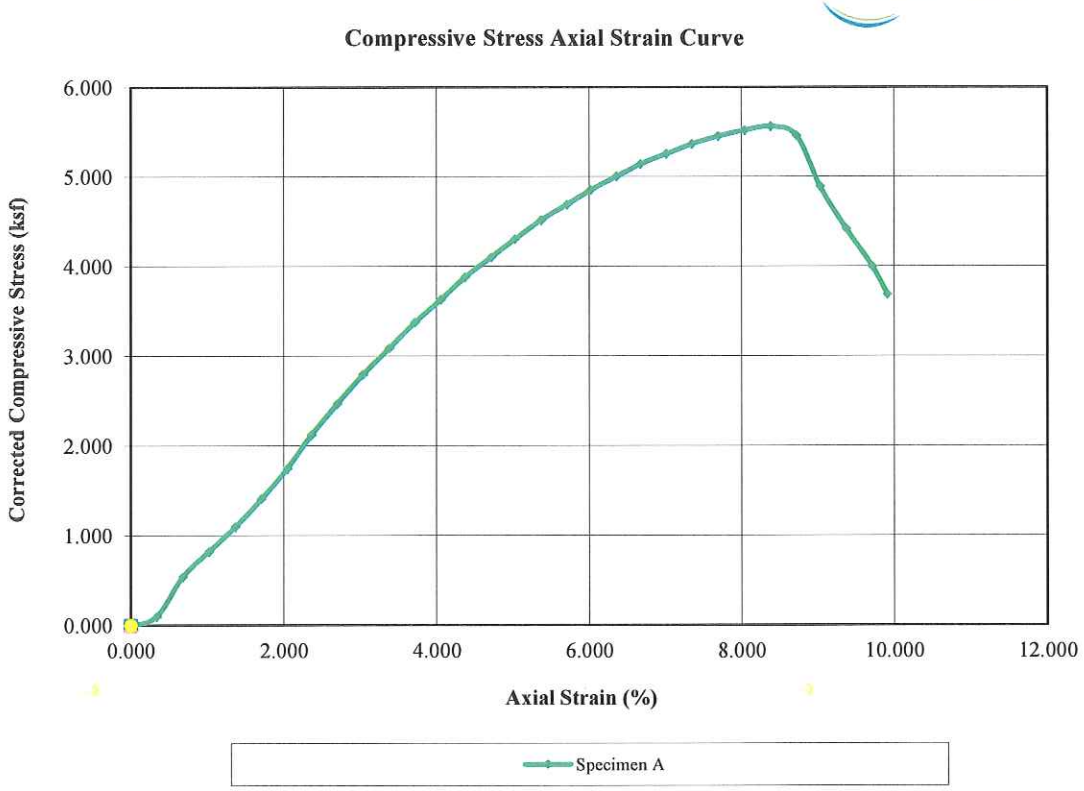
Specimen Description	
Specimen A	Greenish Gray Lean Clay
Test Variables	
Specific Gravity	2.70

Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

Tested By: M. Baishiki — Date: 3/28/2011
Computed By: M. Baishiki — Date: 3/29/2011
Checked By — Date



Before Test	Specimen A
Water Content (%)	24.57
Dry Density (pcf)	101.70
Saturation (%)	99.78
Void Ratio	0.67
Diameter (in)	1.92
Height (in)	3.99
Test Data	**A**
Unconfined Strength (ksf)	5.56
Undrained Shear Strength (tsf)	1.39
Undrained Shear Strength (ksf)	2.78
Rate of Strain (in/min)	0.08
Strain at Failure (%)	8.38
Sample Location	B7-70-1

Specimen

Project Information		Specimen Description	
Project Num	111904.G01	Specimen A	Brownish Gray Lean Clay
Project	Crosstown Freeway SR-4		
Sampling Date	1/31/2011		
Sample #	1103017		
Client		**Test Variables**	
		Specific Gravity	2.72

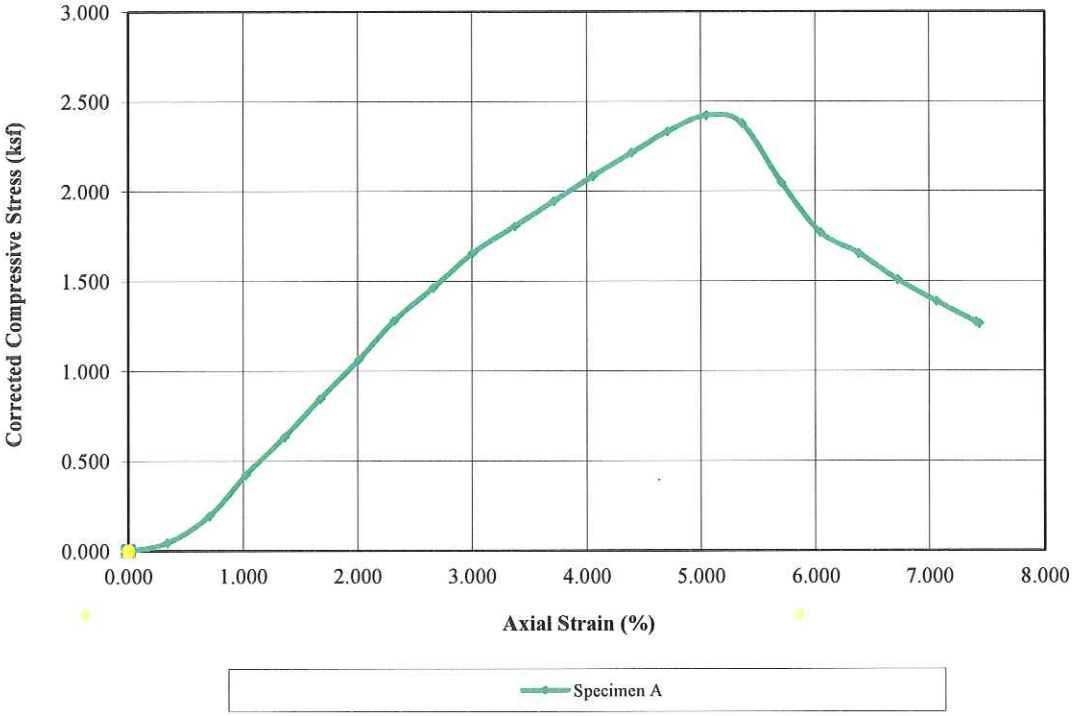
Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

Tested By: M. Baishiki — Date: 3/30/2011
Computed By: M. Baishiki — Date: 3/31/2011
Checked By: — Date:

Compressive Stress Axial Strain Curve



Before Test	Specimen A
Water Content (%)	31.92
Dry Density (pcf)	91.00
Saturation (%)	99.86
Void Ratio	0.87
Diameter (in)	1.92
Height (in)	4.00
Test Data	**A**
Unconfined Strength (ksf)	2.42
Undrained Shear Strength (tsf)	0.61
Undrained Shear Strength (ksf)	1.21
Rate of Strain (in/min)	0.08
Strain at Failure (%)	5.05
Sample Location	B7-75-1

Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	1/31/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Brownish Gray Lean Clay

Test Variables	
Specific Gravity	2.73

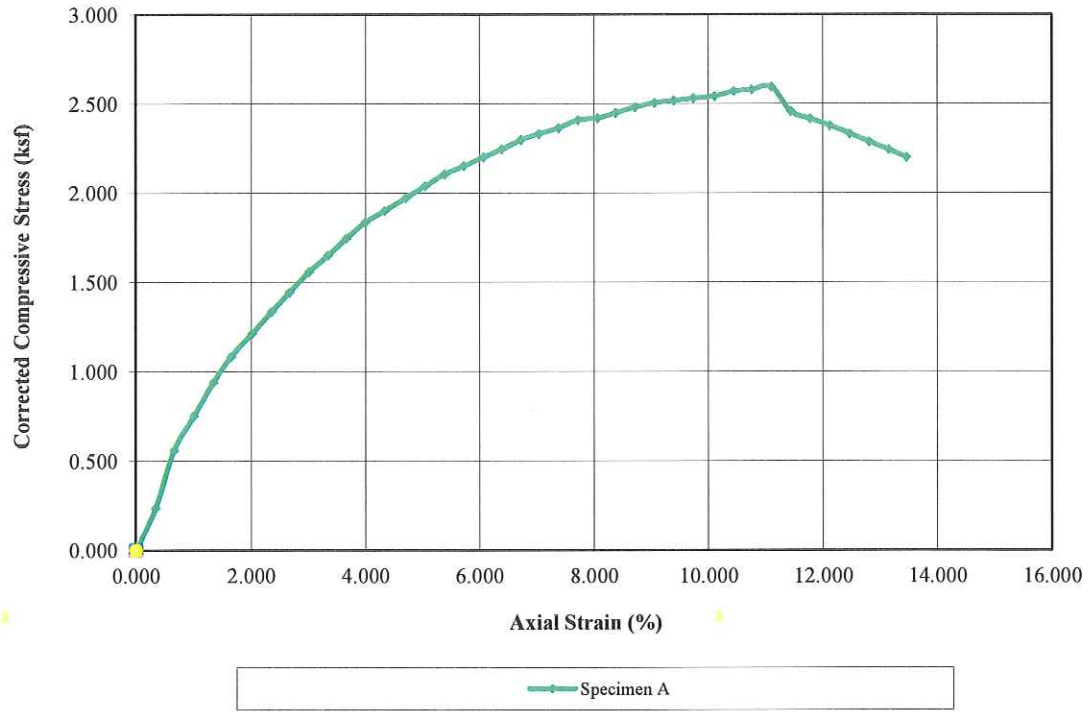
Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

Date
Checked By
4/4/20111
Date
M. Baishiki
Computed By
4/1/2011
Date
M. Baishiki
Tested By

Compressive Stress Axial Strain Curve



Before Test	Specimen A
Water Content (%)	30.09
Dry Density (pcf)	93.40
Saturation (%)	99.94
Void Ratio	0.82
Diameter (in)	1.92
Height (in)	4.00
Test Data	**A**
Unconfined Strength (ksf)	2.59
Undrained Shear Strength (tsf)	0.65
Undrained Shear Strength (ksf)	1.30
Rate of Strain (in/min)	0.08
Strain at Failure (%)	11.10
Sample Location	B8-20-1

Project Information		Specimen Description	
Project Num	111904.G01	Specimen A	Light Brown Lean Clay
Project	Crosstown Freeway SR-4		
Sampling Date	1/28/2011		
Sample #	1103017		
Client		**Test Variables**	
		Specific Gravity	2.72

Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

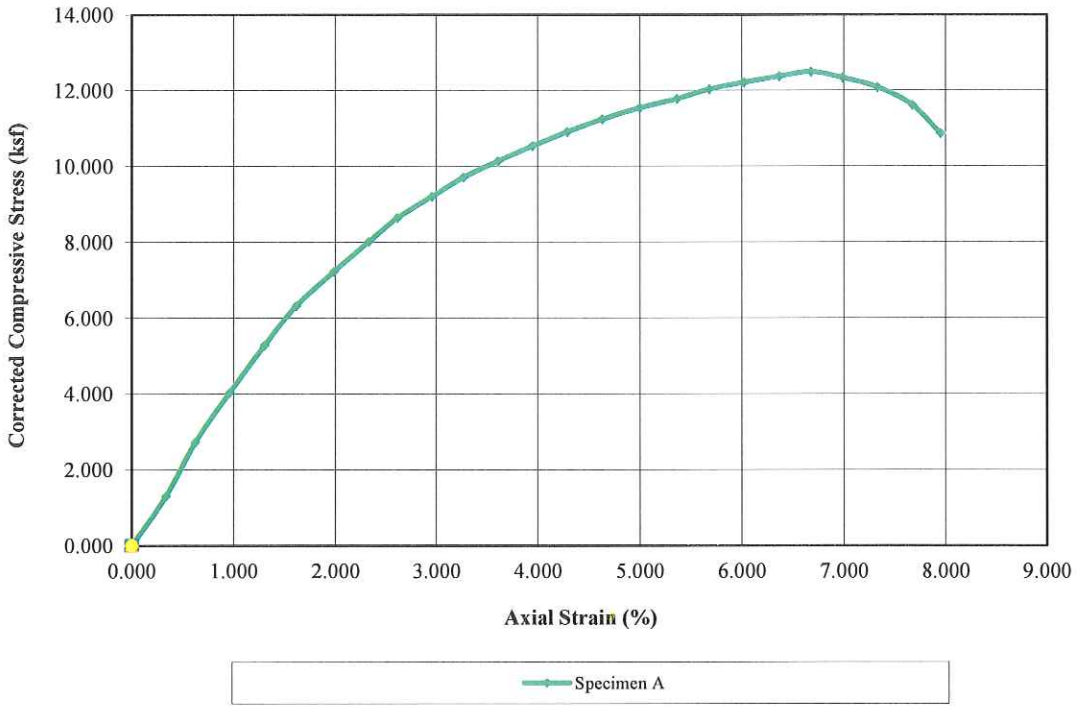
KLEINFELDER

Tested By: M. Baishiki — Date: 4/1/2011

Computed By: M. Baishiki — Date: 4/4/2011

Checked By: — Date:

Compressive Stress Axial Strain Curve



Before Test	Specimen A
Water Content (%)	18.68
Dry Density (pcf)	112.79
Saturation (%)	99.80
Void Ratio	0.51
Diameter (in)	1.92
Height (in)	3.99
Test Data	**A**
Unconfined Strength (ksf)	12.49
Undrained Shear Strength (tsf)	3.12
Undrained Shear Strength (ksf)	6.25
Rate of Strain (in/min)	0.08
Strain at Failure (%)	6.68
Sample Location	B8-85-1

Specimen

Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	1/28/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Greenish Gray Lean Clay
Test Variables	
Specific Gravity	2.73

Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

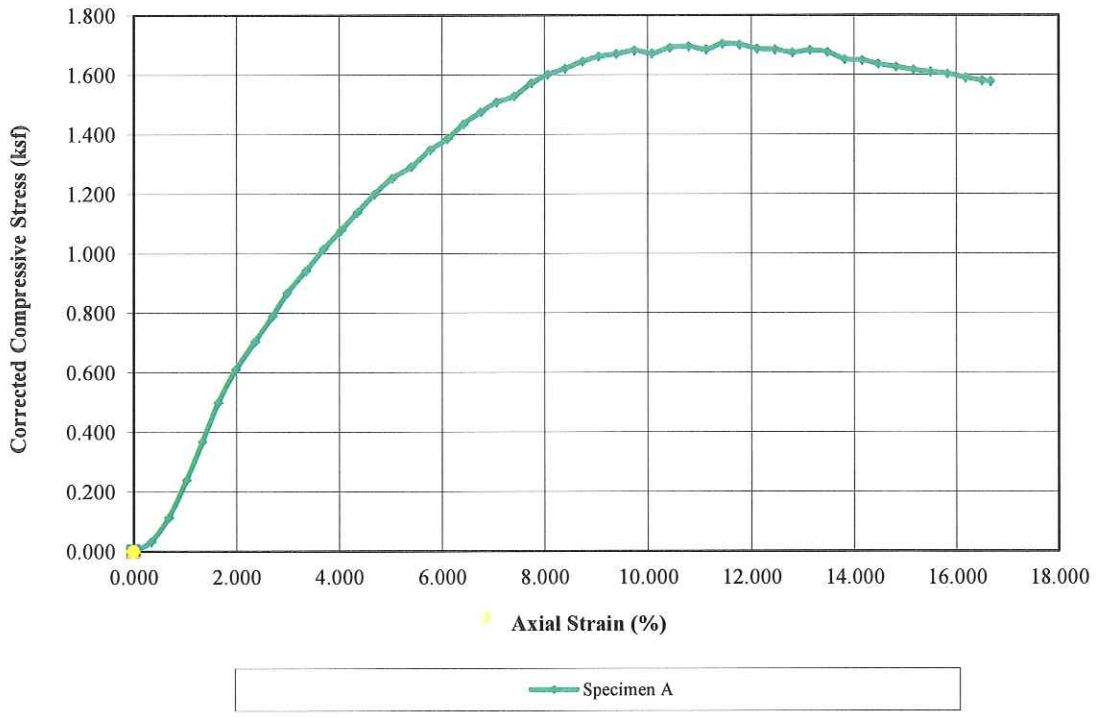
KLEINFELDER

Tested By: M. Baishiki Date: 4/1/2011

Computed By: M. Baishiki Date: 4/4/2011

Checked By: Date:

Compressive Stress Axial Strain Curve



Before Test	Specimen A
Water Content (%)	28.34
Dry Density (pcf)	96.00
Saturation (%)	99.91
Void Ratio	0.77
Diameter (in)	1.92
Height (in)	4.00
Test Data	**A**
Unconfined Strength (ksf)	1.71
Undrained Shear Strength (tsf)	0.43
Undrained Shear Strength (ksf)	0.85
Rate of Strain (in/min)	0.08
Strain at Failure (%)	11.44
Sample Location	B9-15-1

Specimen

Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	1/25/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Light Brown Lean Clay

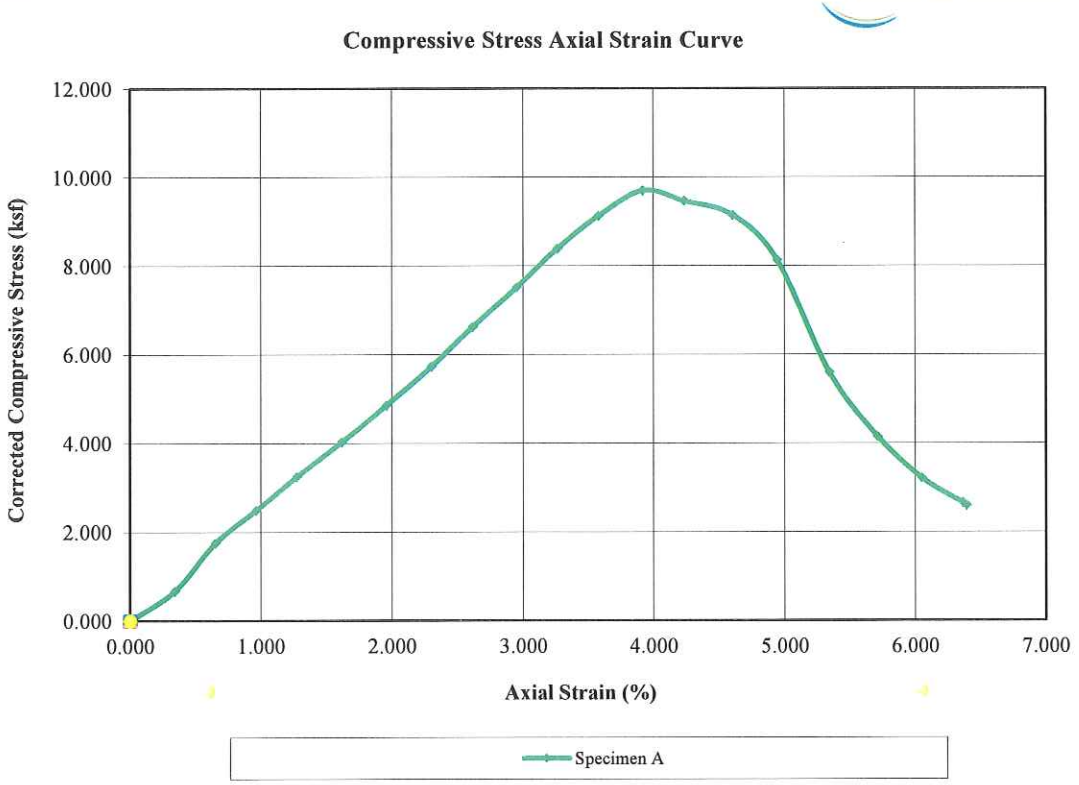
Test Variables	
Specific Gravity	2.73

Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

Date
Checked By
4/4/2011
Date
Computed By M. Baishiki
4/1/2011
Date
Tested By M. Baishiki



Before Test	Specimen A
Water Content (%)	16.46
Dry Density (pcf)	117.47
Saturation (%)	99.67
Void Ratio	0.45
Diameter (in)	1.92
Height (in)	3.99
Test Data	**A**
Unconfined Strength (ksf)	9.70
Undrained Shear Strength (tsf)	2.42
Undrained Shear Strength (ksf)	4.85
Rate of Strain (in/min)	0.08
Strain at Failure (%)	3.92
Sample Location	B9-90-1

Specimen

Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	1/25/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Greenish Gray Lean Clay w/ Sand

Test Variables	
Specific Gravity	2.73

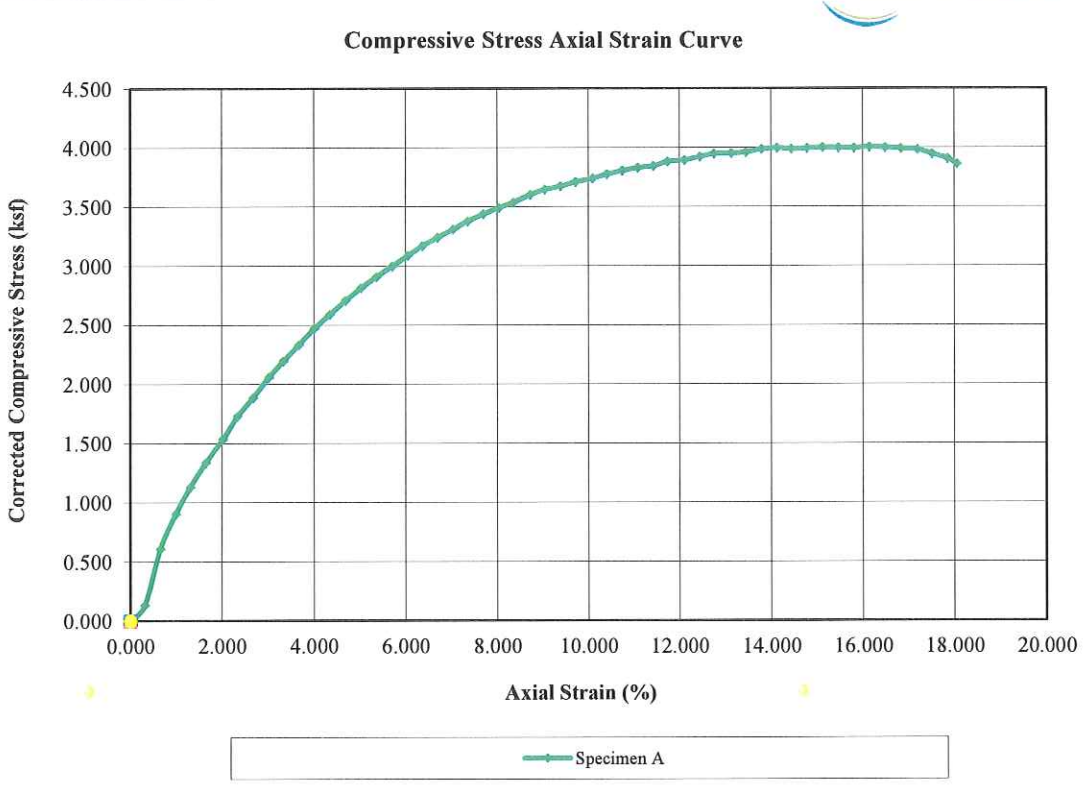
Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

KLEINFELDER

Tested By M. Baishiki Date 4/6/2011 Computed By M. Baishiki Date 4/7/2011 Checked By L.C.E ######



Before Test	Specimen A
Water Content (%)	24.87
Dry Density (pcf)	99.60
Saturation (%)	96.03
Void Ratio	0.70
Diameter (in)	1.92
Height (in)	3.99
Test Data	**A**
Unconfined Strength (ksf)	4.00
Undrained Shear Strength (tsf)	1.00
Undrained Shear Strength (ksf)	2.00
Rate of Strain (in/min)	0.08
Strain at Failure (%)	16.15
Sample Location	B10-25-1

Specimen

Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	1/26/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Brownish Gray Lean Clay

Test Variables	
Specific Gravity	2.72

Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

Date #####

Checked By L.C.E.

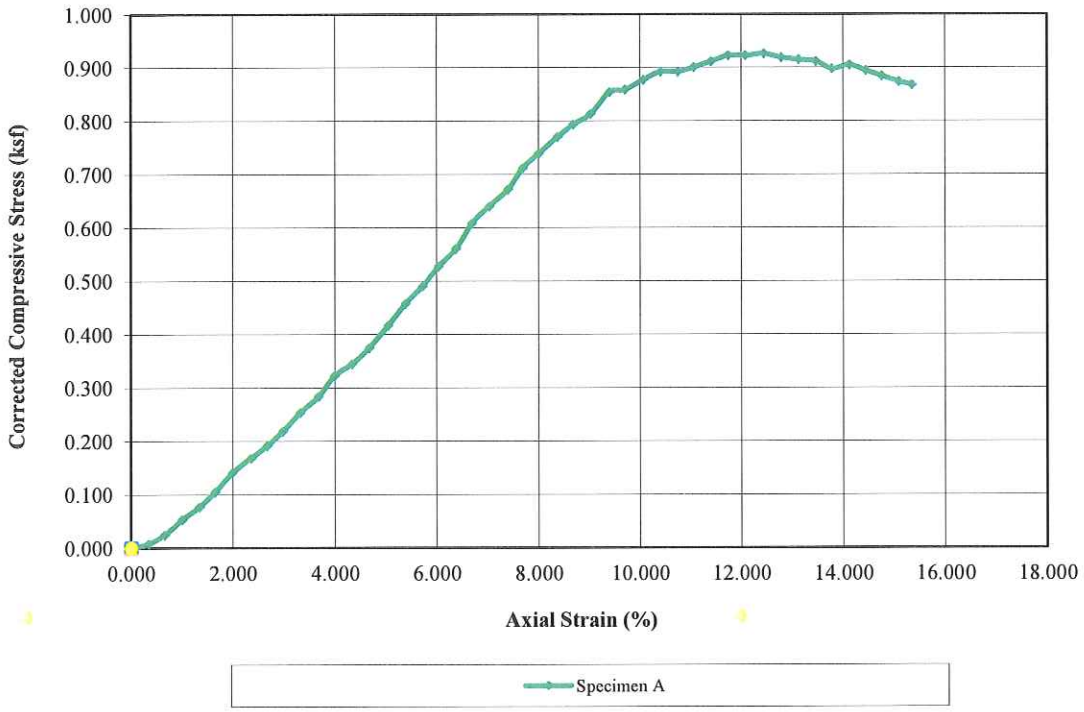
Date 4/7/2011

Computed By M. Baishiki

Date 4/6/2011

Tested By M. Baishiki

Compressive Stress Axial Strain Curve



Before Test	A
Water Content (%)	29.89
Dry Density (pcf)	93.50
Saturation (%)	99.71
Void Ratio	0.82
Diameter (in)	1.92
Height (in)	4.00
Test Data	**A**
Unconfined Strength (ksf)	0.93
Undrained Shear Strength (tsf)	0.23
Undrained Shear Strength (ksf)	0.46
Rate of Strain (in/min)	0.08
Strain at Failure (%)	12.45
Sample Location	B11-35-1

Specimen

Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	2/4/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Brownish Gray Sandy Lean Clay

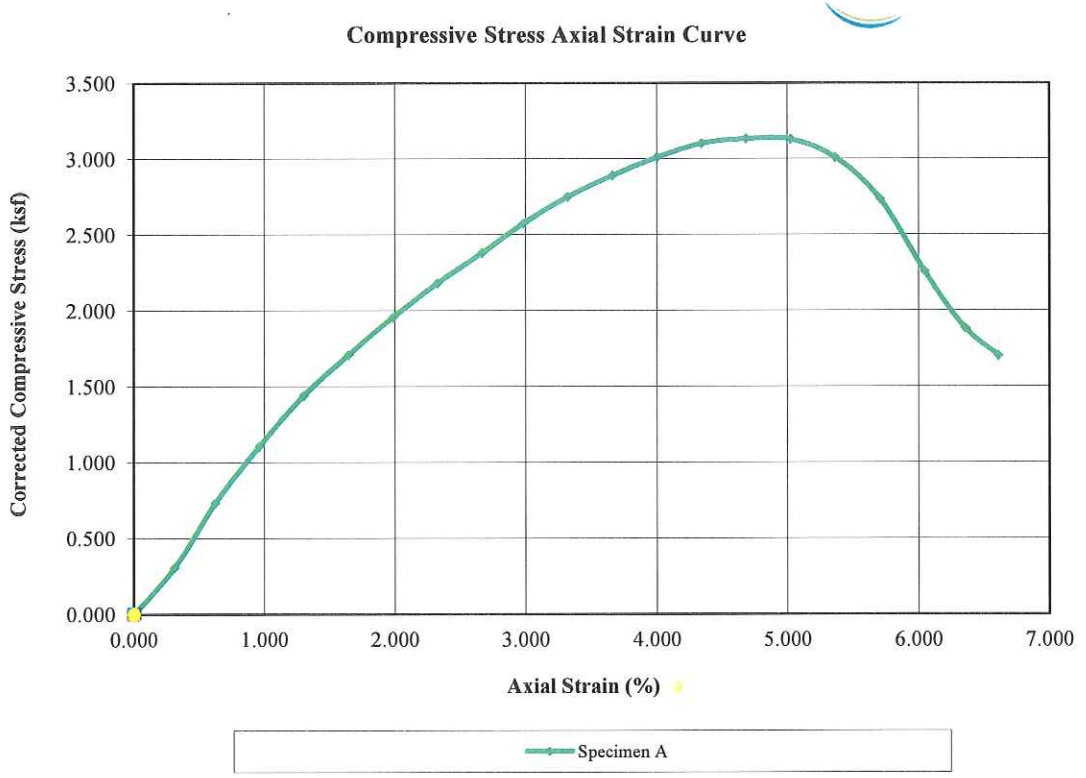
Test Variables	
Specific Gravity	2.72

Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

Tested By: M. Baishiki — Date: 4/6/2011
Computed By: M. Baishiki — Date: 4/7/2011
Checked By: L.C.E — Date: #####



Before Test	Specimen A
Water Content (%)	19.49
Dry Density (pcf)	108.70
Saturation (%)	94.38
Void Ratio	0.56
Diameter (in)	1.92
Height (in)	4.00
Test Data	**A**
Unconfined Strength (ksf)	3.13
Undrained Shear Strength (tsf)	0.78
Undrained Shear Strength (ksf)	1.57
Rate of Strain (in/min)	0.08
Strain at Failure (%)	4.68
Sample Location	B11-70-1

Specimen

Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	2/4/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Brownish Gray Lean Clay w/ Sand

Test Variables	
Specific Gravity	2.72

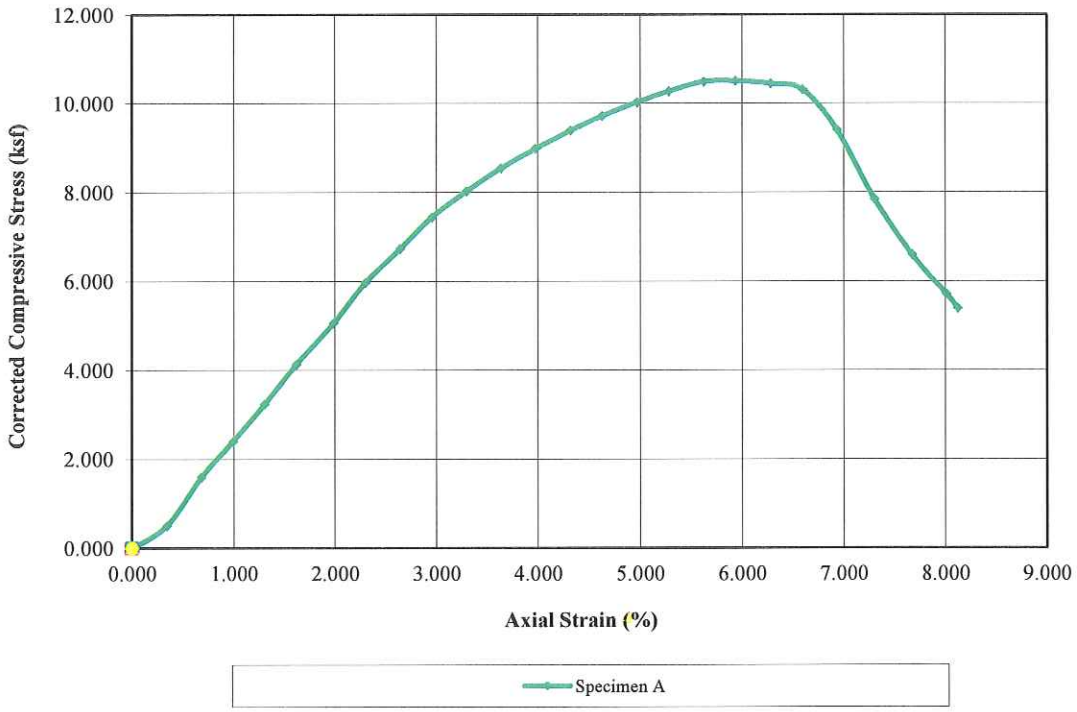
Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

Tested By: M. Baishiki — Date: 3/28/2011
Computed By: M. Baishiki — Date: 3/29/2011
Checked By — Date

Compressive Stress Axial Strain Curve



Before Test	Specimen A
Water Content (%)	23.27
Dry Density (pcf)	103.40
Saturation (%)	98.52
Void Ratio	0.64
Diameter (in)	1.92
Height (in)	3.99
Test Data	**A**
Unconfined Strength (ksf)	10.51
Undrained Shear Strength (tsf)	2.63
Undrained Shear Strength (ksf)	5.25
Rate of Strain (in/min)	0.08
Strain at Failure (%)	5.94
Sample Location	B11-120-1

Specimen

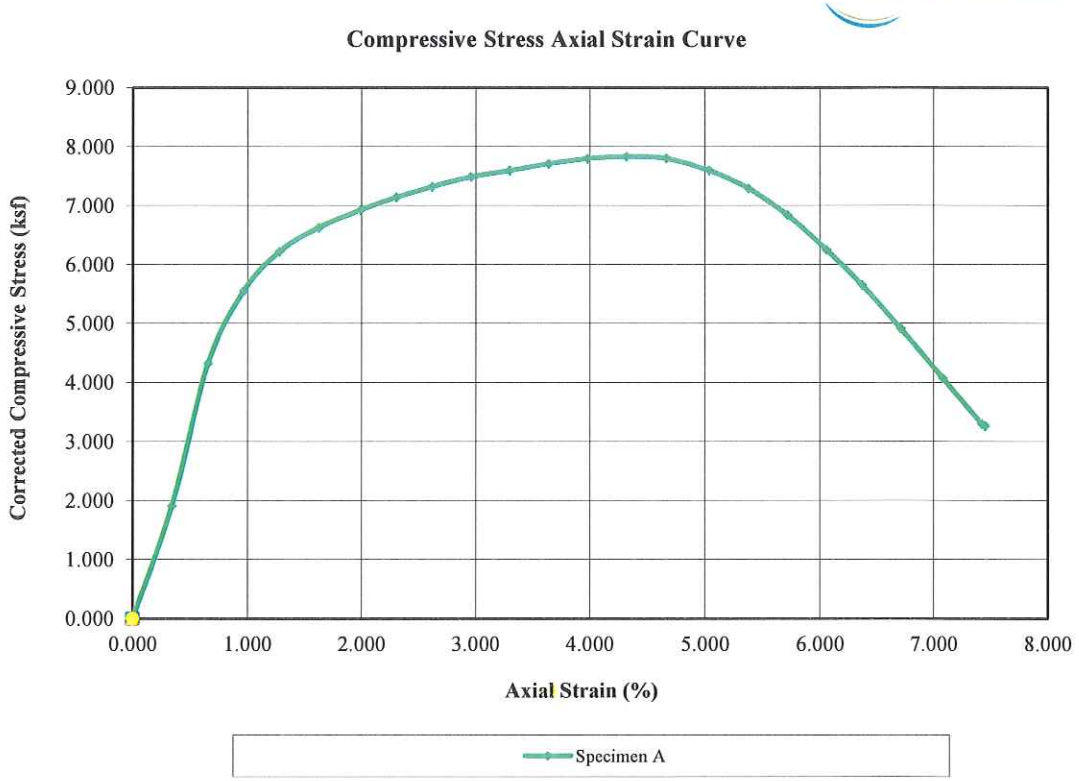
Project Information		Specimen Description	
Project Num	111904.G01	Specimen A	Bluish Gray Lean Clay
Project	Crosstown Freeway SR-4		
Sampling Date	2/7/2011		
Sample #	1103017		
Client		**Test Variables**	
		Specific Gravity	2.72

Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

Tested By: M. Baishiki Date: 4/7/2011
Computed By: M. Baishiki Date: 4/8/2011
Checked By: L.C.E Date: #####



Before Test	Specimen A
Water Content (%)	24.72
Dry Density (pcf)	100.00
Saturation (%)	96.41
Void Ratio	0.70
Diameter (in)	1.92
Height (in)	3.99
Test Data	**A**
Unconfined Strength (ksf)	7.83
Undrained Shear Strength (tsf)	1.96
Undrained Shear Strength (ksf)	3.92
Rate of Strain (in/min)	0.08
Strain at Failure (%)	4.32
Sample Location	B11-130-1

Specimen

Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	2/7/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Brownish Gray Lean Clay
Test Variables	
Specific Gravity	2.72

Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

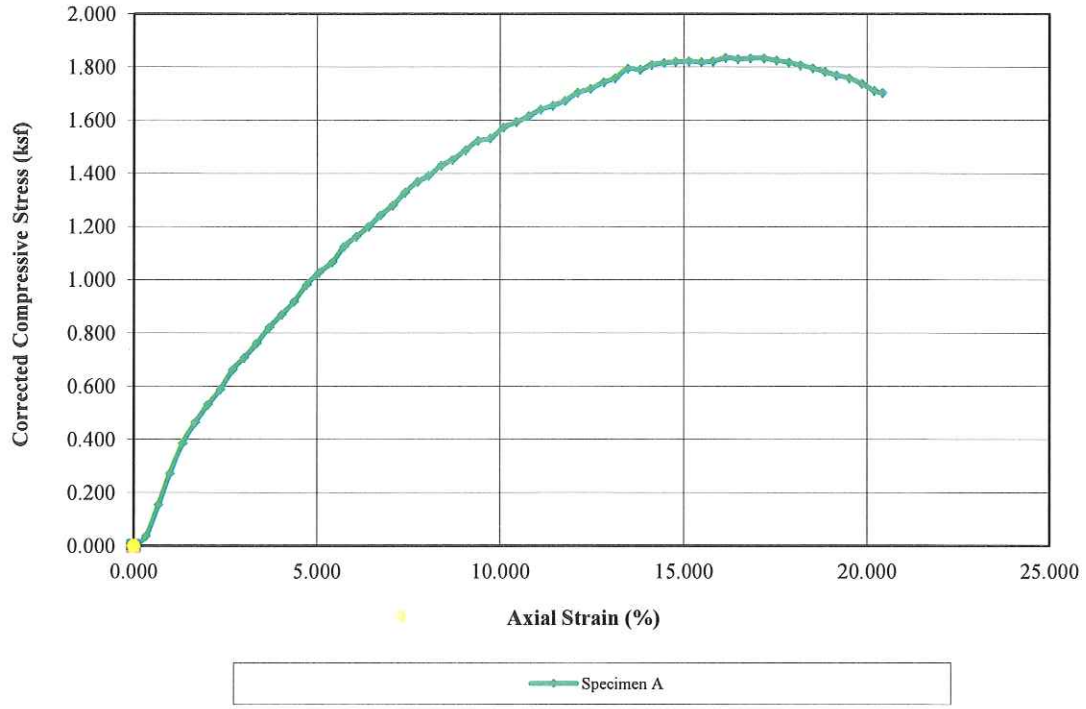
KLEINFELDER

Tested By: M. Baishiki — Date: 4/7/2011

Computed By: M. Baishiki — Date: 4/8/2011

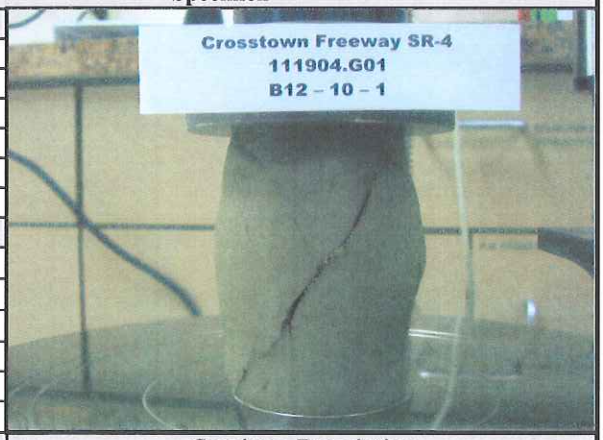
Checked By: L.C.E. — Date: #####

Compressive Stress Axial Strain Curve



Before Test	Specimen A
Water Content (%)	30.75
Dry Density (pcf)	92.40
Saturation (%)	99.95
Void Ratio	0.84
Diameter (in)	1.92
Height (in)	4.00
Test Data	**A**
Unconfined Strength (ksf)	1.84
Undrained Shear Strength (tsf)	0.46
Undrained Shear Strength (ksf)	0.92
Rate of Strain (in/min)	0.08
Strain at Failure (%)	16.14
Sample Location	B12-10-1

Specimen



Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	1/27/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Light Brown Lean Clay

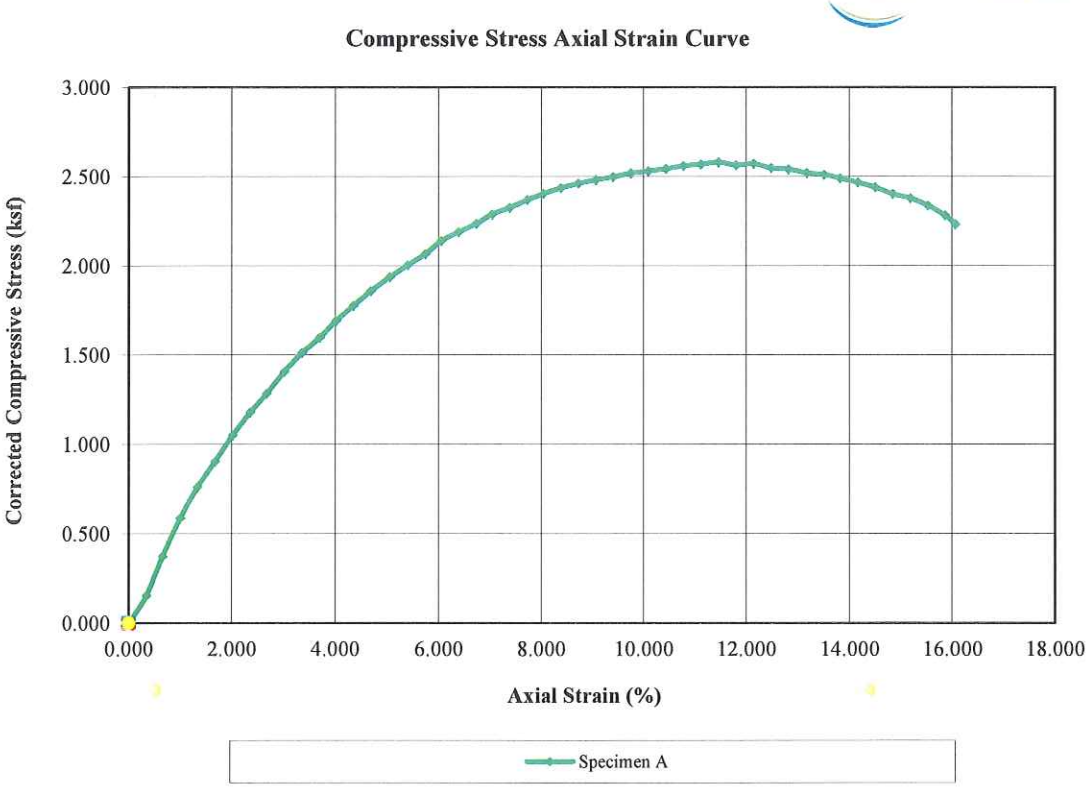
Test Variables	
Specific Gravity	2.72

Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

Tested By: M. Baishiki — Date: 4/1/2011
Computed By: M. Baishiki — Date: 4/4/2011
Checked By — Date



Before Test	Specimen A
Water Content (%)	26.63
Dry Density (pcf)	98.45
Saturation (%)	99.95
Void Ratio	0.72
Diameter (in)	1.92
Height (in)	3.99
Test Data	**A**
Unconfined Strength (ksf)	2.58
Undrained Shear Strength (tsf)	0.65
Undrained Shear Strength (ksf)	1.29
Rate of Strain (in/min)	0.08
Strain at Failure (%)	11.46
Sample Location	B13-20-1

Specimen

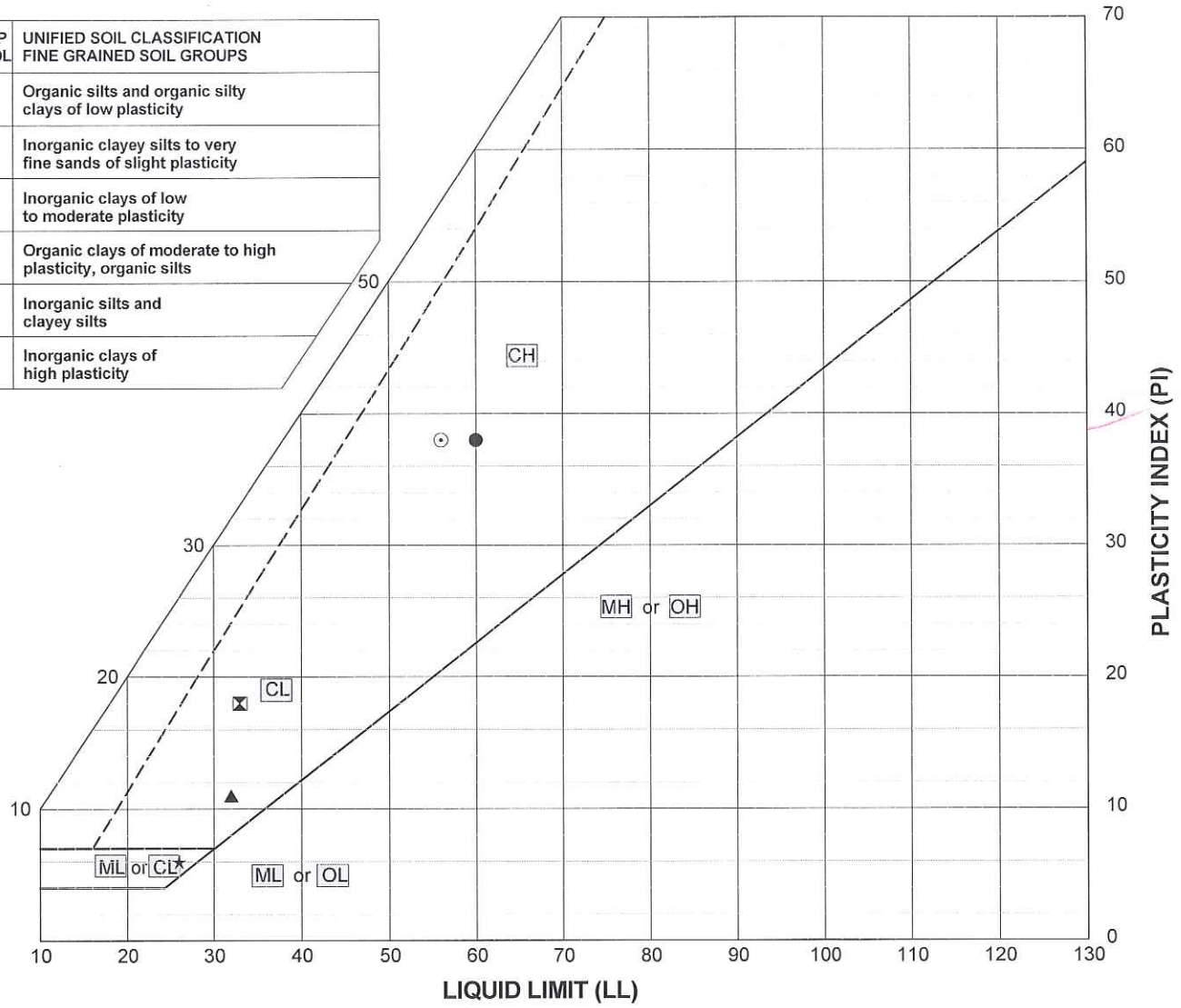
Project Information		Specimen Description	
Project Num	111904.G01	Specimen A	Brown Lean Clay
Project	Crosstown Freeway SR-4		
Sampling Date	1/27/2011		
Sample #	1103017		
Client		**Test Variables**	
		Specific Gravity	2.72

Remarks

Plasticity Charts (Atterberg Limits)

GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE GRAINED SOIL GROUPS
OL	Organic silts and organic silty clays of low plasticity
ML	Inorganic clayey silts to very fine sands of slight plasticity
CL	Inorganic clays of low to moderate plasticity
OH	Organic clays of moderate to high plasticity, organic silts
MH	Inorganic silts and clayey silts
CH	Inorganic clays of high plasticity

EXPLANATION



LEGEND:	SOURCE	DEPTH	LL	PL	PI	DESCRIPTION
●	R-11-001	75.0	60	22	38	
⊠	R-11-002	15.0	33	15	18	
▲	R-11-002	70.0	32	21	11	
★	R-11-003	55.0	26	20	6	
⊙	R-11-003	110.0	56	18	38	

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Drafted By: G. GOMEZ
Date: 7/14/2011

Project No.: 111904
File Number: STO11G015

PLASTICITY CHART
ROUTE 4 CROSSTOWN FREEWAY EXTENSION
SAN JOAQUIN COUNTY, CALIFORNIA

PLATE
1 of 5

ALT ATTERBERG STO11G015.GPJ 7/14/11

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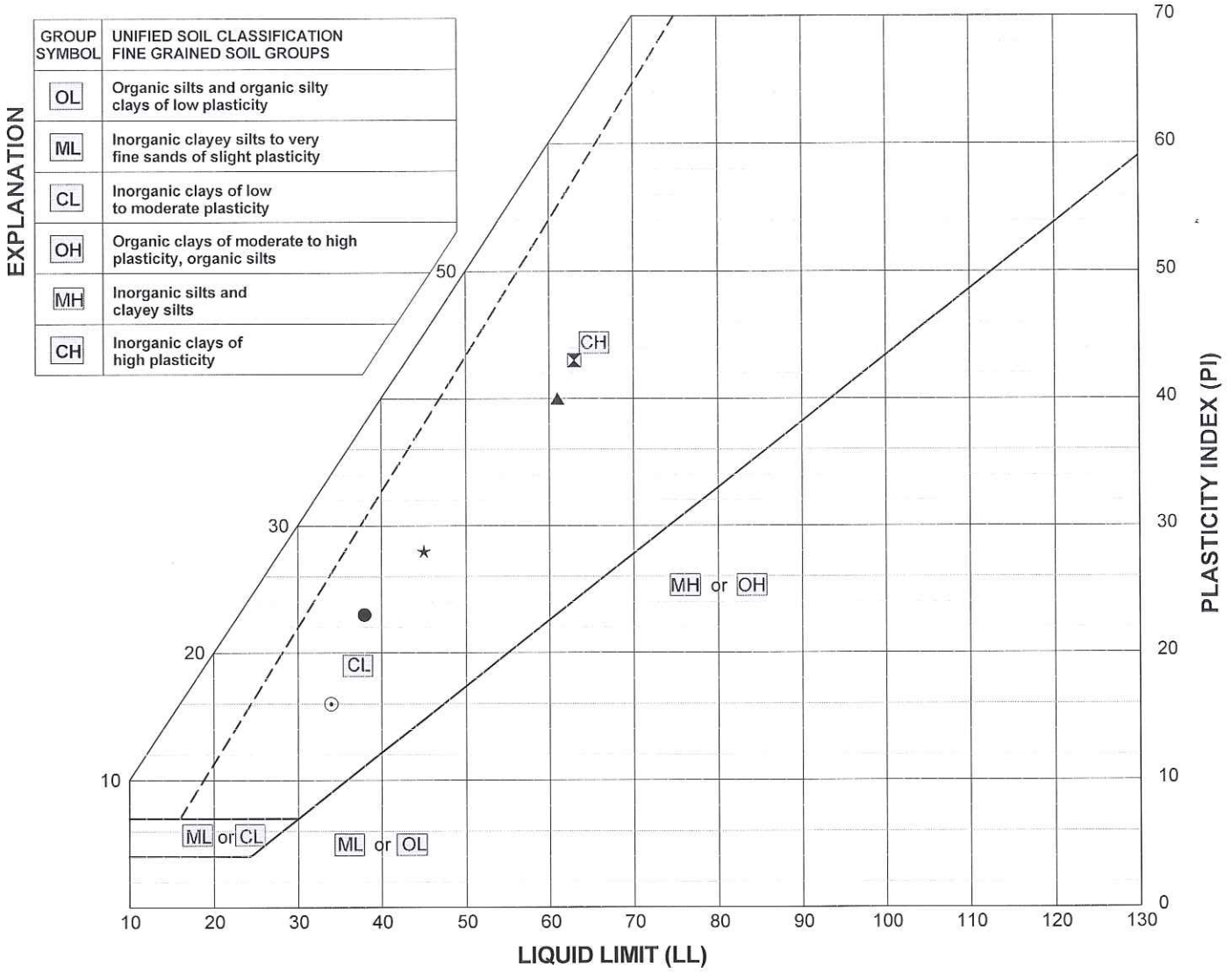

LEGEND:	SOURCE	DEPTH	LL	PL	PI	DESCRIPTION
●	R-11-004	30.0	38	15	23	
⊠	R-11-004	55.0	63	20	43	
▲	R-11-004	85.0	61	21	40	
★	R-11-005	40.0	45	17	28	
⊙	R-11-006	20.0	34	18	16	

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Drafted By: G. GOMEZ
Date: 7/14/2011

Project No.: 111904
File Number: STO11G015

PLASTICITY CHART
ROUTE 4 CROSSTOWN FREEWAY EXTENSION
SAN JOAQUIN COUNTY, CALIFORNIA

PLATE
2 of 5

ALT ATTERBERG STO11G015.GPJ 7/14/11

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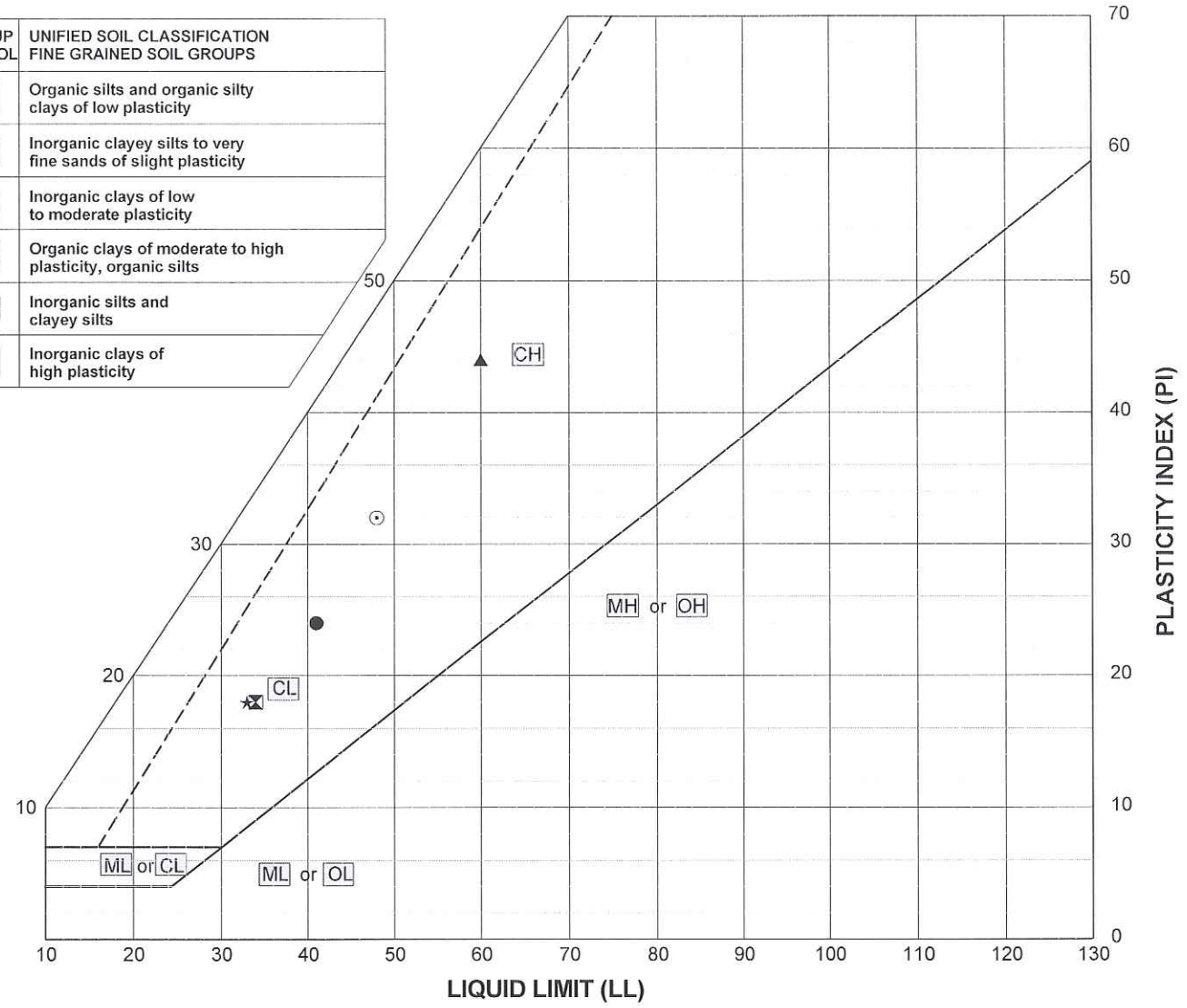
GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE GRAINED SOIL GROUPS
OL	Organic silts and organic silty clays of low plasticity
ML	Inorganic clayey silts to very fine sands of slight plasticity
CL	Inorganic clays of low to moderate plasticity
OH	Organic clays of moderate to high plasticity, organic silts
MH	Inorganic silts and clayey silts
CH	Inorganic clays of high plasticity



LEGEND:	SOURCE	DEPTH	LL	PL	PI	DESCRIPTION
●	R-11-006	70.0	41	17	24	
⊠	R-11-007	25.0	34	16	18	
▲	R-11-007	55.0	60	16	44	
★	R-11-008	25.0	33	15	18	
⊙	R-11-009	20.0	48	16	32	

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PLASTICITY CHART
ROUTE 4 CROSSTOWN FREEWAY EXTENSION
SAN JOAQUIN COUNTY, CALIFORNIA

PLATE
3 of 5

Drafted By: G. GOMEZ
Date: 7/14/2011
Project No.: 111904
File Number: STO11G015

ALT ATTERBERG STO11G015.GPJ 7/14/11

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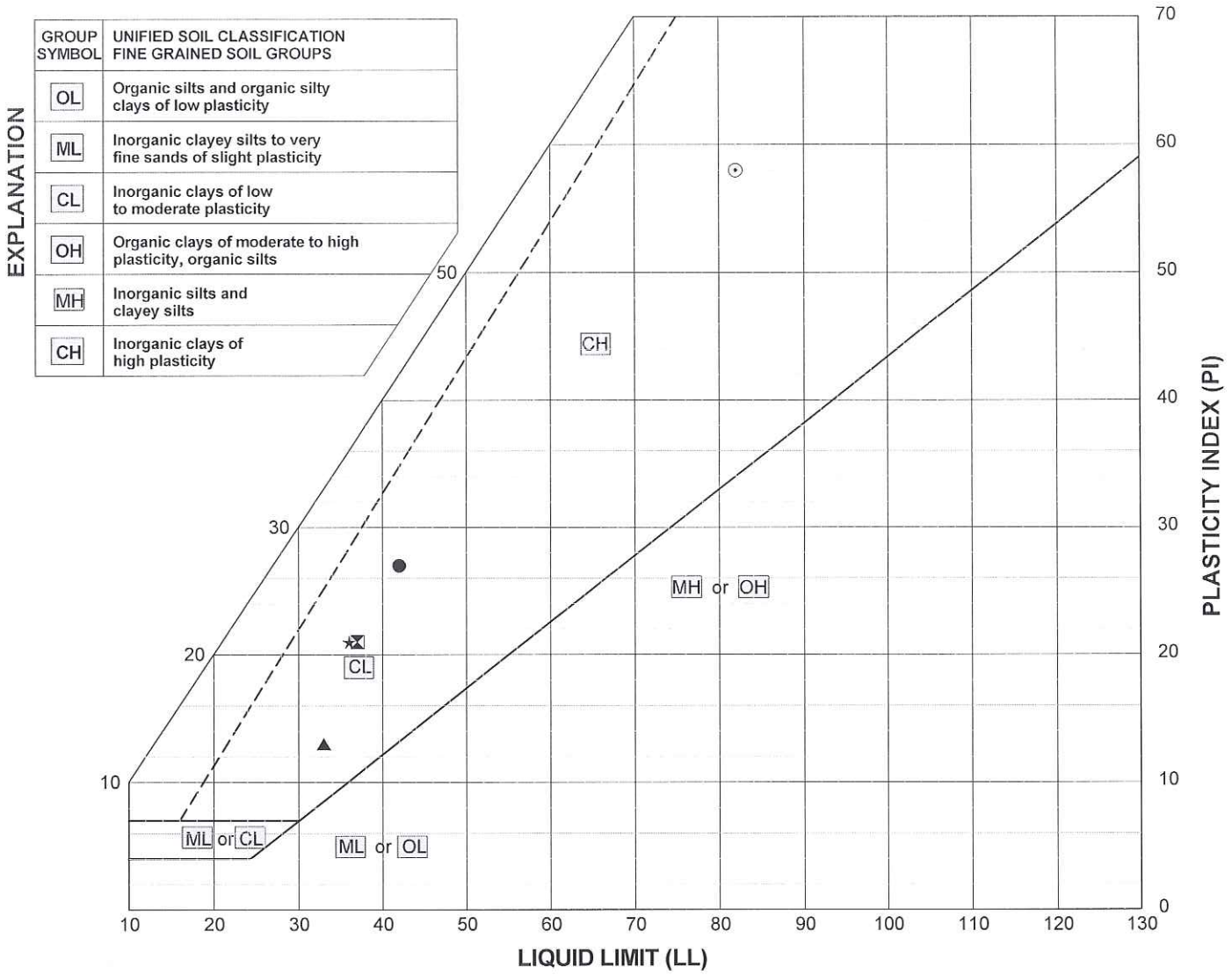

LEGEND:	SOURCE	DEPTH	LL	PL	PI	DESCRIPTION
●	R-11-009	80.0	42	15	27	
⊠	R-11-010	35.0	37	16	21	
▲	R-11-011	35.0	33	20	13	
★	R-11-011	45.0	36	15	21	
⊙	R-11-011	90.0	82	24	58	

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PLASTICITY CHART
ROUTE 4 CROSSTOWN FREEWAY EXTENSION
SAN JOAQUIN COUNTY, CALIFORNIA

PLATE
4 of 5

Drafted By: G. GOMEZ
Date: 7/14/2011
Project No.: 111904
File Number: STO11G015

ALT ATTERBERG STO11G015.GPJ 7/14/11

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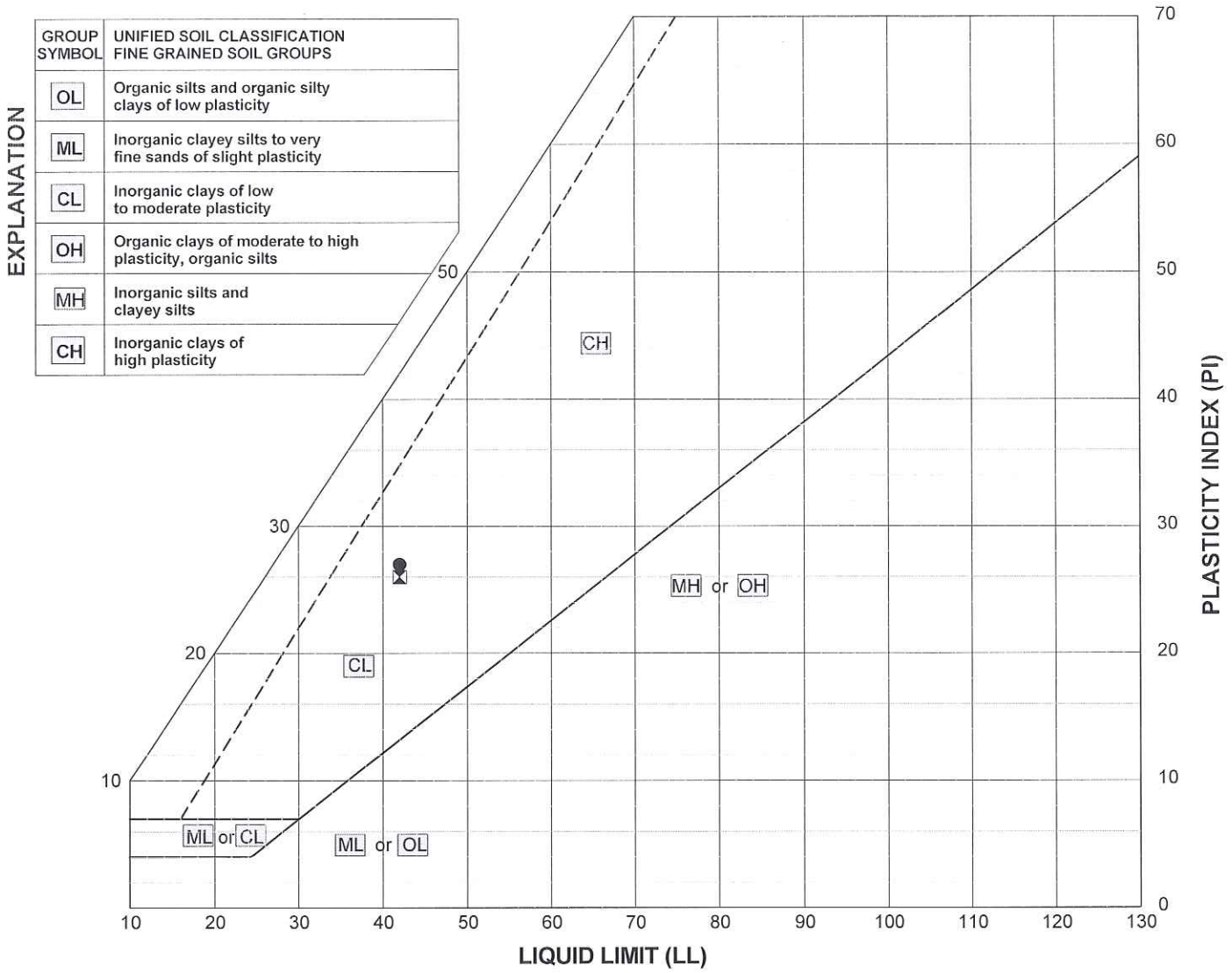

LEGEND:	SOURCE	DEPTH	LL	PL	PI	DESCRIPTION
●	R-11-012	15.0	42	15	27	
⧓	R-11-013	25.0	42	16	26	

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PLASTICITY CHART
ROUTE 4 CROSSTOWN FREEWAY EXTENSION
SAN JOAQUIN COUNTY, CALIFORNIA

PLATE
5 of 5

Drafted By: G. GOMEZ
Date: 7/14/2011
Project No.: 111904
File Number: STO11G015

ALT ATTERBERG STO11G015.GPJ 7/14/11

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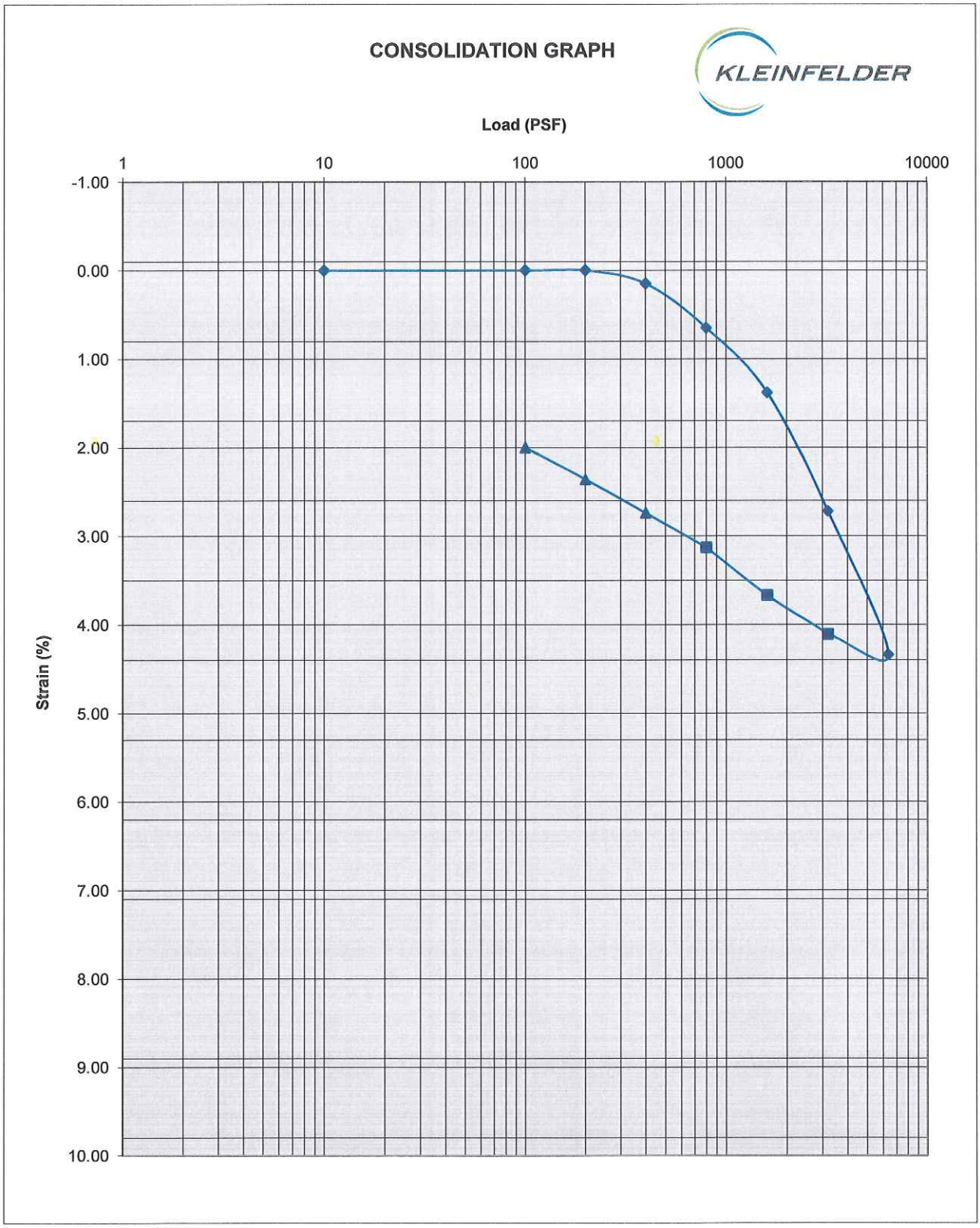
Consolidation Test Results

Laboratory Test Data

Project Name: ***Crosstown Freeway SR-4***
Project No: **111904.G01**
Report Date: **4/15/2011**
Boring No: **B12-20-1**
Sample Description: **Greenish Gray Lean Clay**

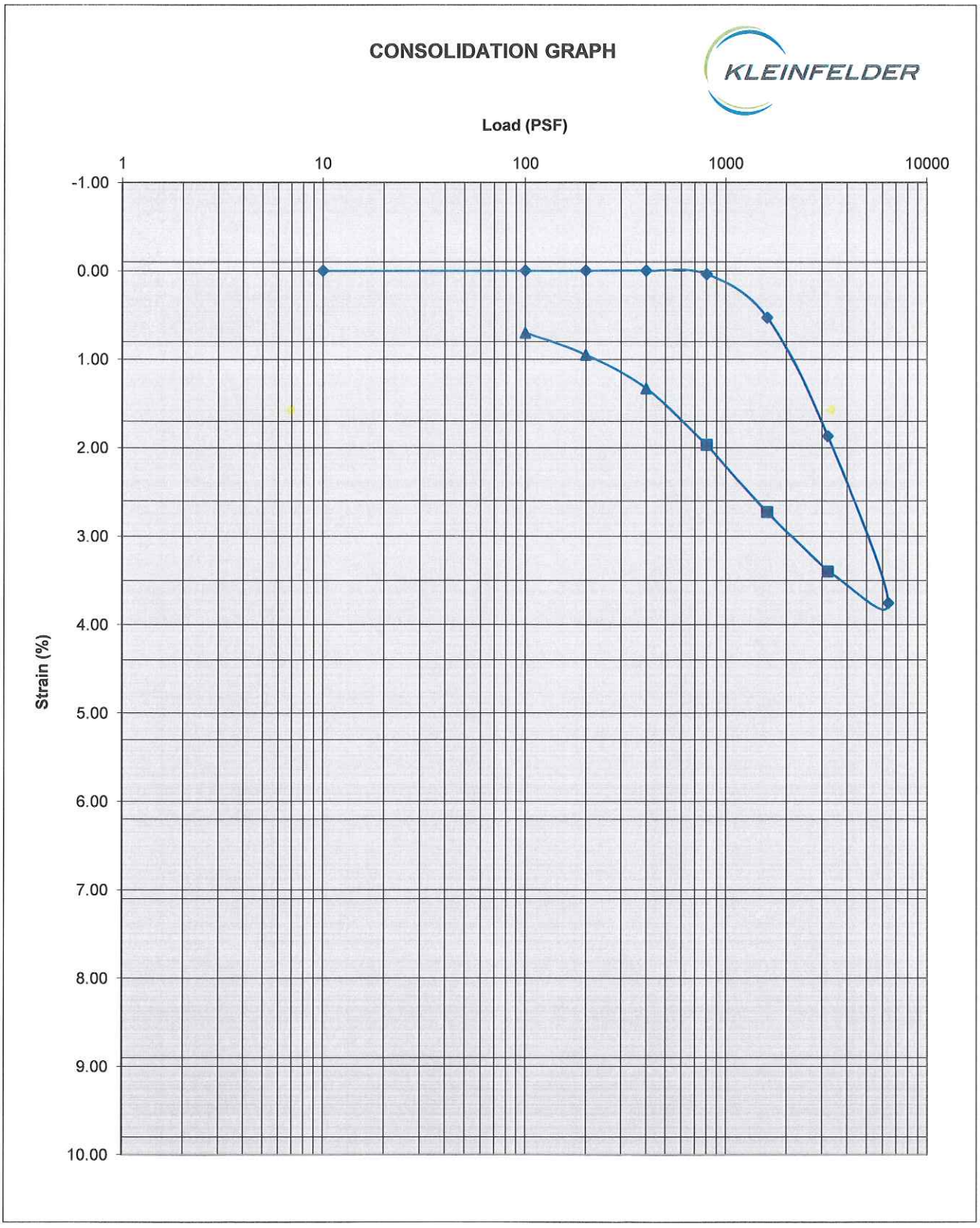
One-Dimensional Consolidation of Soil (ASTM D 2435)



Laboratory Test Data

Project Name: *Crosstown Freeway SR-4*
Project No: 111904.G01
Report Date: 4/19/2011
Boring No: B13-5-1
Sample Description: Greenish Gray Lean Clay

One-Dimensional Consolidation of Soil (ASTM D 2435)



R-Value Test Results

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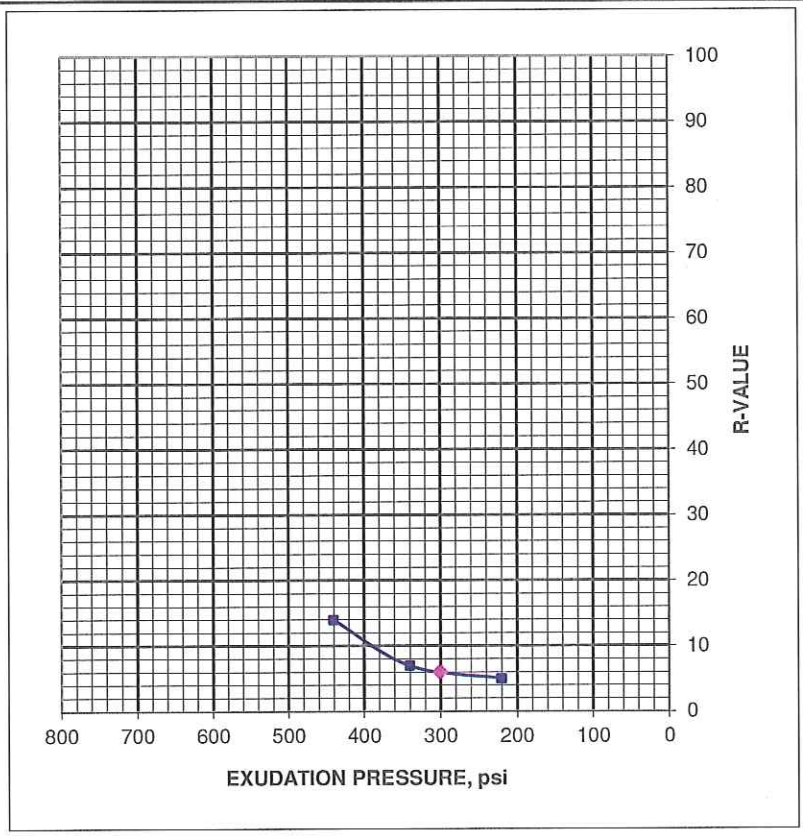
Project Name: Hwy 4 Extension
Project Number: 111904.G01
Date Sampled: February 4, 2011
Laboratory Test Number: 1102008
Sample Description: Brown Clay w/ Sand
Sample Location: 2345 Navy Dr., South Side

CTM 301, Resistance "R" Value of Treated and Untreated Bases



LABORATORY DATA

Specimen	A	B	C
Exudation Pressure, psi	220	340	440
Expansion Pressure, psf	35	143	320
Resistance Value, R	5	7	14
Moisture at Test, %	46.6	45.0	42.5
Dry Density at Test, pcf	82.7	85.4	88.4
R-Value at 300 psi Exudation Pressure			**6**

Reviewed By: _____
L. Christine Erickson
Lab Manager

KLEINFELDER
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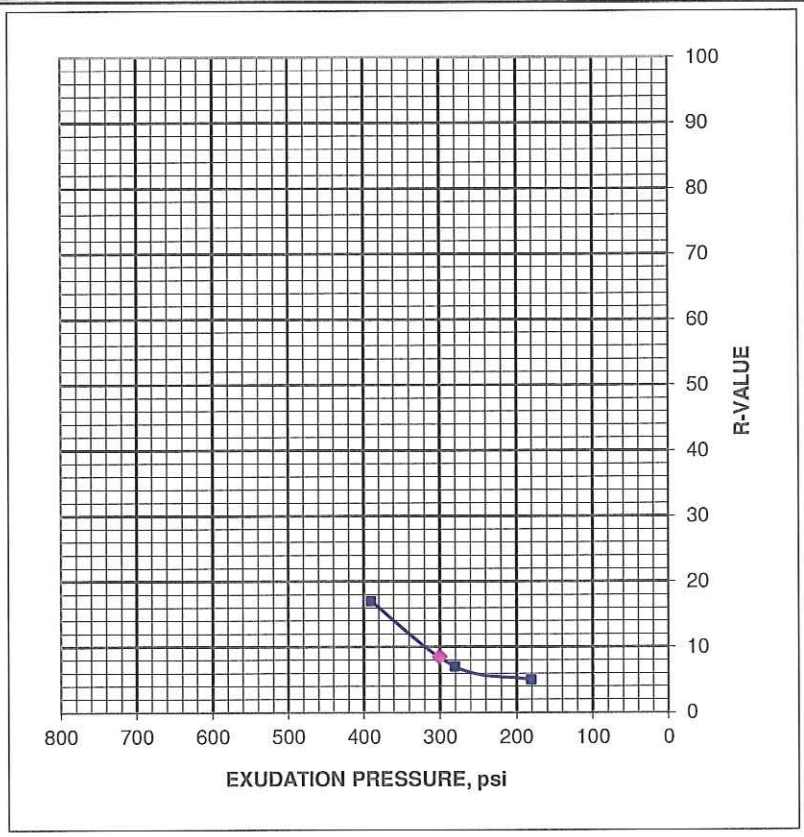
Project Name: Hwy 4 Extension
Project Number: 111904.G01
Date Sampled: February 4, 2011
Laboratory Test Number: 1102009
Sample Description: Brown Clay w/ Sand
Sample Location: Navy Drive & Tellie Lewis

CTM 301, Resistance "R" Value of Treated and Untreated Bases



LABORATORY DATA

Specimen	A	B	C
Exudation Pressure, psi	180	280	390
Expansion Pressure, psf	0	61	320
Resistance Value, R	5	7	17
Moisture at Test, %	41.2	38.7	36.2
Dry Density at Test, pcf	88.0	91.7	95.2
R-Value at 300 psi Exudation Pressure			**9**

Reviewed By: _____
L. Christine Erickson
Lab Manager

KLEINFELDER
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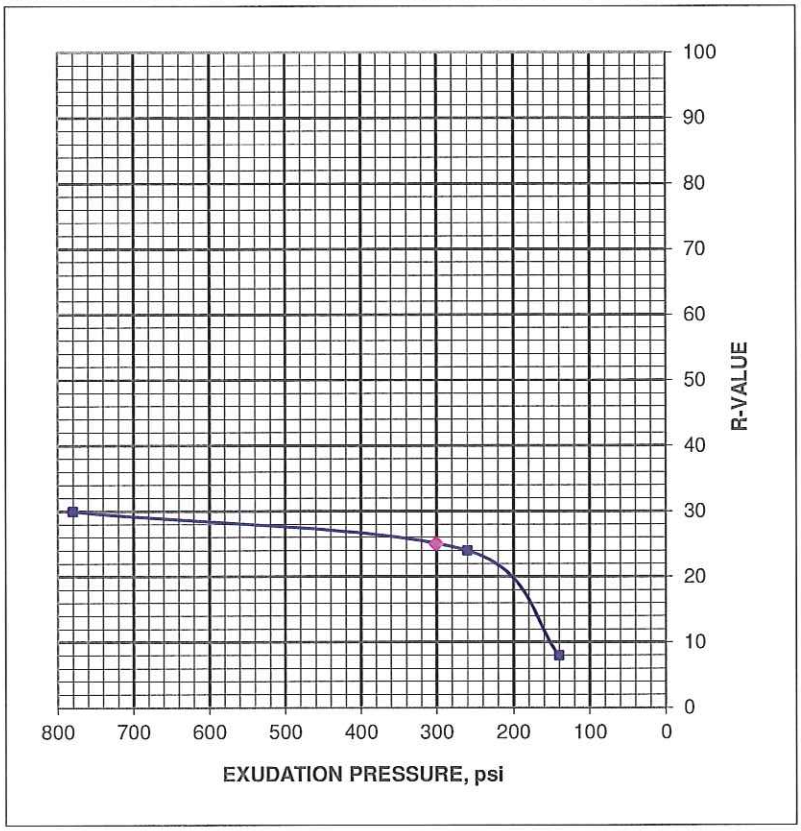
Project Name: Hwy 4 Extension
Project Number: 111904.G01
Date Sampled: February 4, 2011
Laboratory Test Number: 1102007
Sample Description: Brown Clayey Sand
Sample Location: Scotts Ave.; NW Side

CTM 301, Resistance "R" Value of Treated and Untreated Bases



LABORATORY DATA

Specimen	A	B	C
Exudation Pressure, psi	140	260	780
Expansion Pressure, psf	4	69	108
Resistance Value, R	8	24	30
Moisture at Test, %	65.9	58.8	54.1
Dry Density at Test, pcf	73.8	77.6	82.4
R-Value at 300 psi Exudation Pressure			**25**

Reviewed By: _____
L. Christine Erickson
Lab Manager

KLEINFELDER
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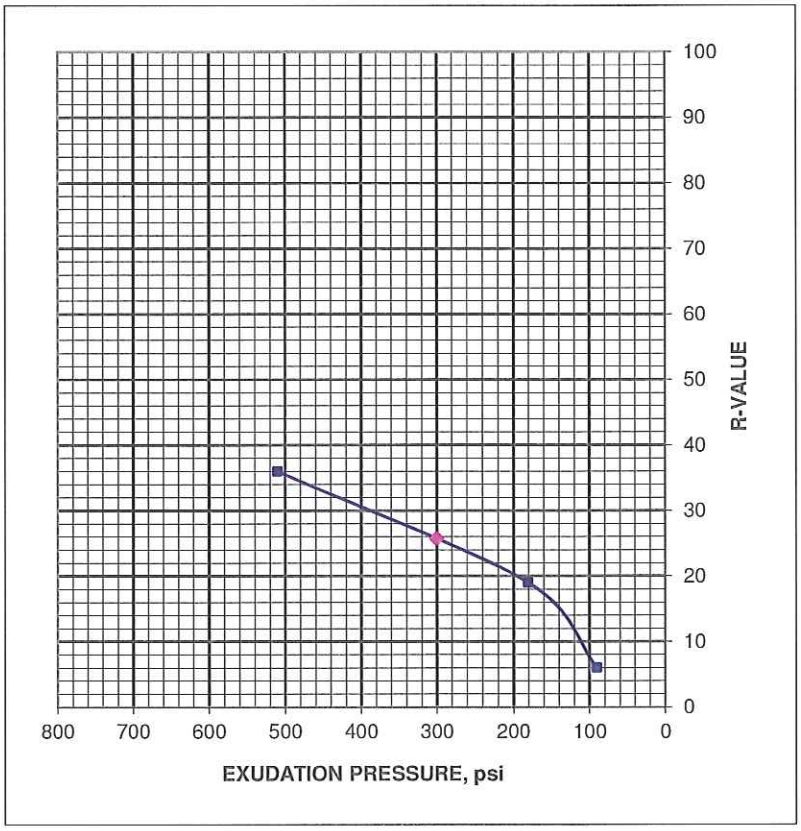
Project Name: Hwy 4 Extension
Project Number: 111904.G01
Date Sampled: February 4, 2011
Laboratory Test Number: 1102005
Sample Description: Brown Clayey Sand
Sample Location: Hazelton Avenue

CTM 301, Resistance "R" Value of Treated and Untreated Bases



LABORATORY DATA

Specimen	A	B	C
Exudation Pressure, psi	90	180	510
Expansion Pressure, psf	0	39	217
Resistance Value, R	6	19	36
Moisture at Test, %	70.4	62.6	57.0
Dry Density at Test, pcf	71.6	75.5	80.0
R-Value at 300 psi Exudation Pressure			**26**

Reviewed By: _____
L. Christine Erickson
Lab Manager

KLEINFELDER
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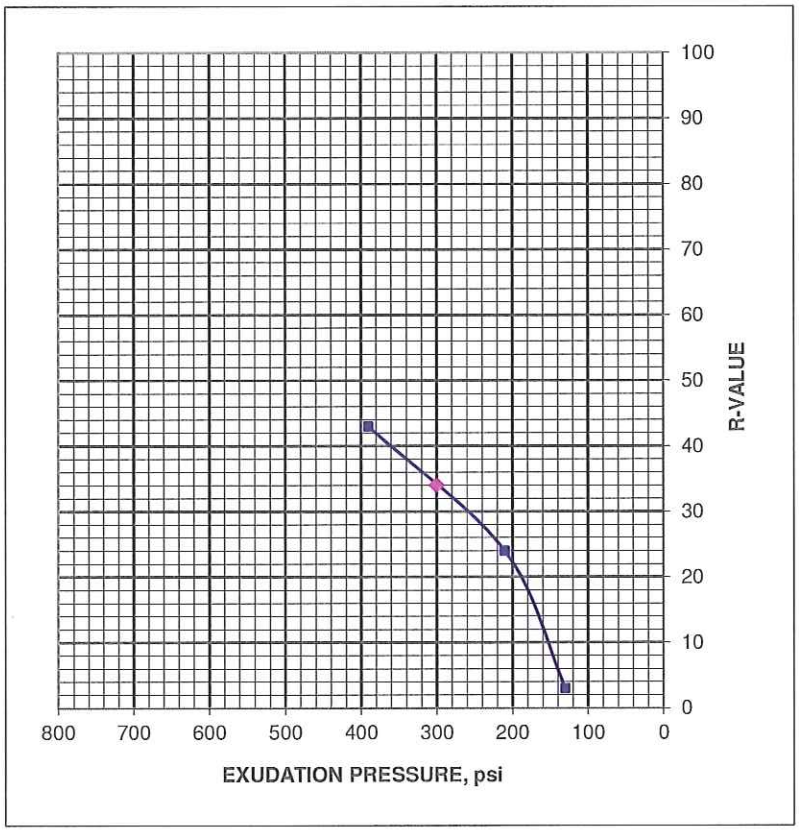
Project Name: Hwy 4 Extension
Project Number: 111904.G01
Date Sampled: February 4, 2011
Laboratory Test Number: 1102006
Sample Description: Brown Silty Sand
Sample Location: Del Norte

CTM 301, Resistance "R" Value of Treated and Untreated Bases



LABORATORY DATA

Specimen	A	B	C
Exudation Pressure, psi	130	210	390
Expansion Pressure, psf	0	0	39
Resistance Value, R	3	24	43
Moisture at Test, %	79.8	73.3	67.5
Dry Density at Test, pcf	67.7	70.0	74.2
R-Value at 300 psi Exudation Pressure			**34**

Reviewed By: _____
L. Christine Erickson
Lab Manager

Sulfate and Chloride Contents by California Laboratory Service

CALIFORNIA LABORATORY SERVICES

3249 Fitzgerald Road Rancho Cordova, CA 95742

March 25, 2011

CLS Work Order #: CUC0760
COC #: 19319

Mackenzie Baishiki
Kleinfelder (Stockton)
2001 Arch-Airport Road, suite 100
Stockton, CA 95206

Project Name: Crosstown Freeway SR-4

Enclosed are the results of analyses for samples received by the laboratory on 03/18/11 14:10. Samples were analyzed pursuant to client request utilizing EPA or other ELAP approved methodologies. I certify that the results are in compliance both technically and for completeness.

Analytical results are attached to this letter. Please call if we can provide additional assistance.

Sincerely,

James Liang, Ph.D.
Laboratory Director

CA DOHS ELAP Accreditation/Registration number 1233

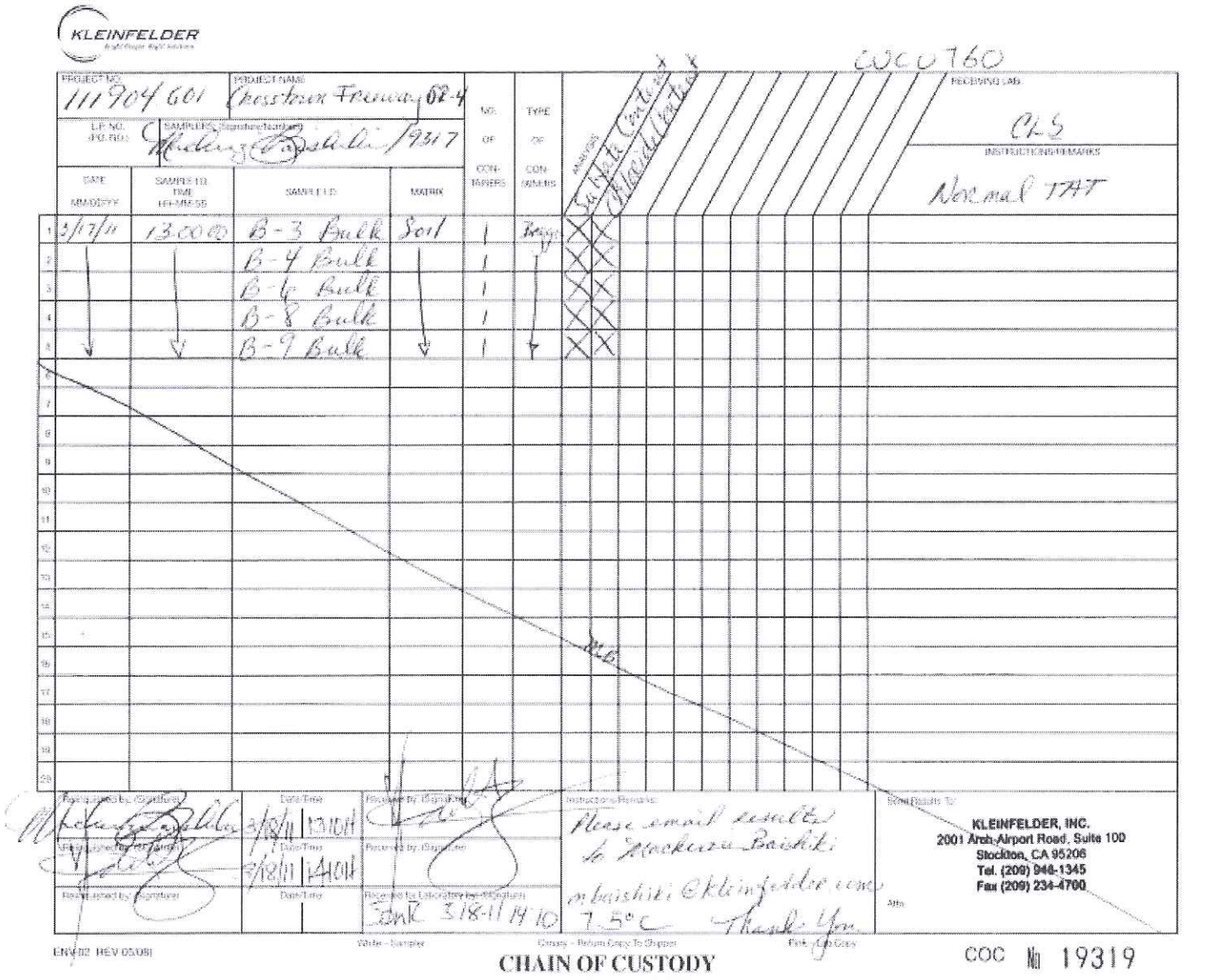
CALIFORNIA LABORATORY SERVICES

03/25/11 09:36

Kleinfelder (Stockton)
2001 Arch-Airport Road, suite 100
Stockton, CA 95206

Project: Crosstown Freeway SR-4
Project Number: 111904.G01
Project Manager: Mackenzie Baishiki

CLS Work Order #: CUC0760
COC #: 19319



KLEINFELDER

CUC0760

PROJECT NO.: 111904.G01
PROJECT NAME: Crosstown Freeway SR-4
P.O. NO.:
SAMPLERS (Signature): Mackenzie Baishiki 9317
RECEIVING LAB: CLS
INSTRUCTIONS/REMARKS: Normal TAT

	DATE (MM/DD/YY)	SAMPLE ID TIME (HH:MM:SS)	SAMPLE I.D.	MATRIX	NO. OF CONTAINERS	TYPE OF CONTAINERS	ANALYSES
1	3/17/11	13:00:00	B-3 Bulk	Soil	1	Bags	X X
2			B-4 Bulk		1		X X
3			B-6 Bulk		1		X X
4			B-8 Bulk		1		X X
5			B-9 Bulk		1		X X

Relinquished by (Signature): Mackenzie Baishiki
Date/Time: 3/18/11 13:10
Received by (Signature):
Relinquished by (Signature):
Date/Time: 3/18/11 14:10
Received by (Laboratory) (Signature): Sonia 3-18-11 14:10

Instructions/Remarks: Please email results to Mackenzie Baishiki mbaishiki@kleinfelder.com 7.5°C Thank You

Send Results To:
KLEINFELDER, INC.
2001 Arch-Airport Road, Suite 100
Stockton, CA 95206
Tel. (209) 946-1345
Fax (209) 234-4760
Attn:

EN002 REV 05/08

White - Sample | Canary - Return Copy To Shipper | Pink - Lab Copy

CHAIN OF CUSTODY

COC № 19319

CALIFORNIA LABORATORY SERVICES

03/25/11 09:36

Kleinfelder (Stockton)
2001 Arch-Airport Road, suite 100
Stockton, CA 95206

Project: Crosstown Freeway SR-4
Project Number: 111904.G01
Project Manager: Mackenzie Baishiki

CLS Work Order #: CUC0760
COC #: 19319

Conventional Chemistry Parameters by APHA/EPA Methods

Analyte	Result	Reporting Limit	Units	Dilution	Batch	Prepared	Analyzed	Method	Notes
B-3 Bulk (CUC0760-01) Soil	**Sampled: 03/17/11 13:00**	**Received: 03/18/11 14:10**							
Chloride	**230**	25	mg/kg	5	CU01902	03/21/11	03/23/11	EPA 300.0	
Sulfate as SO4	**460**	25	"	"	"	"	"	"	
B-4 Bulk (CUC0760-02) Soil	**Sampled: 03/17/11 13:00**	**Received: 03/18/11 14:10**							
Chloride	**340**	5.0	mg/kg	1	CU01902	03/21/11	03/21/11	EPA 300.0	
Sulfate as SO4	**240**	5.0	"	"	"	"	"	"	
B-6 Bulk (CUC0760-03) Soil	**Sampled: 03/17/11 13:00**	**Received: 03/18/11 14:10**							
Chloride	**140**	5.0	mg/kg	1	CU01902	03/21/11	03/21/11	EPA 300.0	
Sulfate as SO4	**400**	5.0	"	"	"	"	"	"	
B-8 Bulk (CUC0760-04) Soil	**Sampled: 03/17/11 13:00**	**Received: 03/18/11 14:10**							
Chloride	**17**	5.0	mg/kg	1	CU01902	03/21/11	03/21/11	EPA 300.0	
Sulfate as SO4	**89**	5.0	"	"	"	"	"	"	
B-9 Bulk (CUC0760-05) Soil	**Sampled: 03/17/11 13:00**	**Received: 03/18/11 14:10**							
Chloride	**18**	5.0	mg/kg	1	CU01902	03/21/11	03/21/11	EPA 300.0	
Sulfate as SO4	**59**	5.0	"	"	"	"	"	"	

Kleinfelder (Stockton)
2001 Arch-Airport Road, suite 100
Stockton, CA 95206

Project: Crosstown Freeway SR-4
Project Number: 111904.G01
Project Manager: Mackenzie Baishiki

CLS Work Order #: CUC0760
COC #: 19319

Conventional Chemistry Parameters by APHA/EPA Methods - Quality Control

Analyte	Result	Reporting Limit	Units	Spike Level	Source Result	%REC	%REC Limits	RPD	RPD Limit	Notes
Batch CU01902 - General Prep										
Blank (CU01902-BLK1)				Prepared & Analyzed: 03/21/11						
Sulfate as SO4	ND	5.0	mg/kg							
Chloride	ND	5.0	"							
LCS (CU01902-BS1)				Prepared & Analyzed: 03/21/11						
Sulfate as SO4	51.9	5.0	mg/kg	50.0		104	75-125			
Chloride	20.3	5.0	"	20.0		101	75-125			
LCS Dup (CU01902-BSD1)				Prepared & Analyzed: 03/21/11						
Sulfate as SO4	52.6	5.0	mg/kg	50.0		105	75-125	1	25	
Chloride	20.7	5.0	"	20.0		104	75-125	2	25	
Matrix Spike (CU01902-MS1)		**Source: CUC0760-04**		Prepared & Analyzed: 03/21/11						
Sulfate as SO4	133	5.0	mg/kg	50.0	88.7	89	75-125			
Chloride	32.8	5.0	"	20.0	17.0	79	75-125			
Matrix Spike Dup (CU01902-MSD1)		**Source: CUC0760-04**		Prepared & Analyzed: 03/21/11						
Sulfate as SO4	137	5.0	mg/kg	50.0	88.7	96	75-125	3	30	
Chloride	35.7	5.0	"	20.0	17.0	94	75-125	9	30	

Kleinfelder (Stockton)	Project: Crosstown Freeway SR-4	
2001 Arch-Airport Road, suite 100	Project Number: 111904.G01	**CLS Work Order #: CUC0760**
Stockton, CA 95206	Project Manager: Mackenzie Baishiki	COC #: 19319

Notes and Definitions

DET	Analyte DETECTED
ND	Analyte NOT DETECTED at or above the reporting limit
NR	Not Reported
dry	Sample results reported on a dry weight basis
RPD	Relative Percent Difference

APPENDIX D
RESULTS OF LABORATORY TESTS
(PARIKH CONSULTANT, INC. INVESTIGATION, MARCH 2010)

LIST OF ATTACHMENTS

The following are attached and complete this appendix.

General Notes
Plasticity Charts (Atterberg Limits)
Grain Size Distribution Curves
Consolidation Test Results

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General Notes

APPENDIX B

LABORATORY TESTS

Classification Tests
The field classification of the samples was visually verified in the laboratory according to the Unified Soil Classification System and is presented in Appendix A.

Moisture-Density
The natural moisture contents and in-situ dry density were determined for selected samples. The results are presented in Appendix A together with material descriptions.

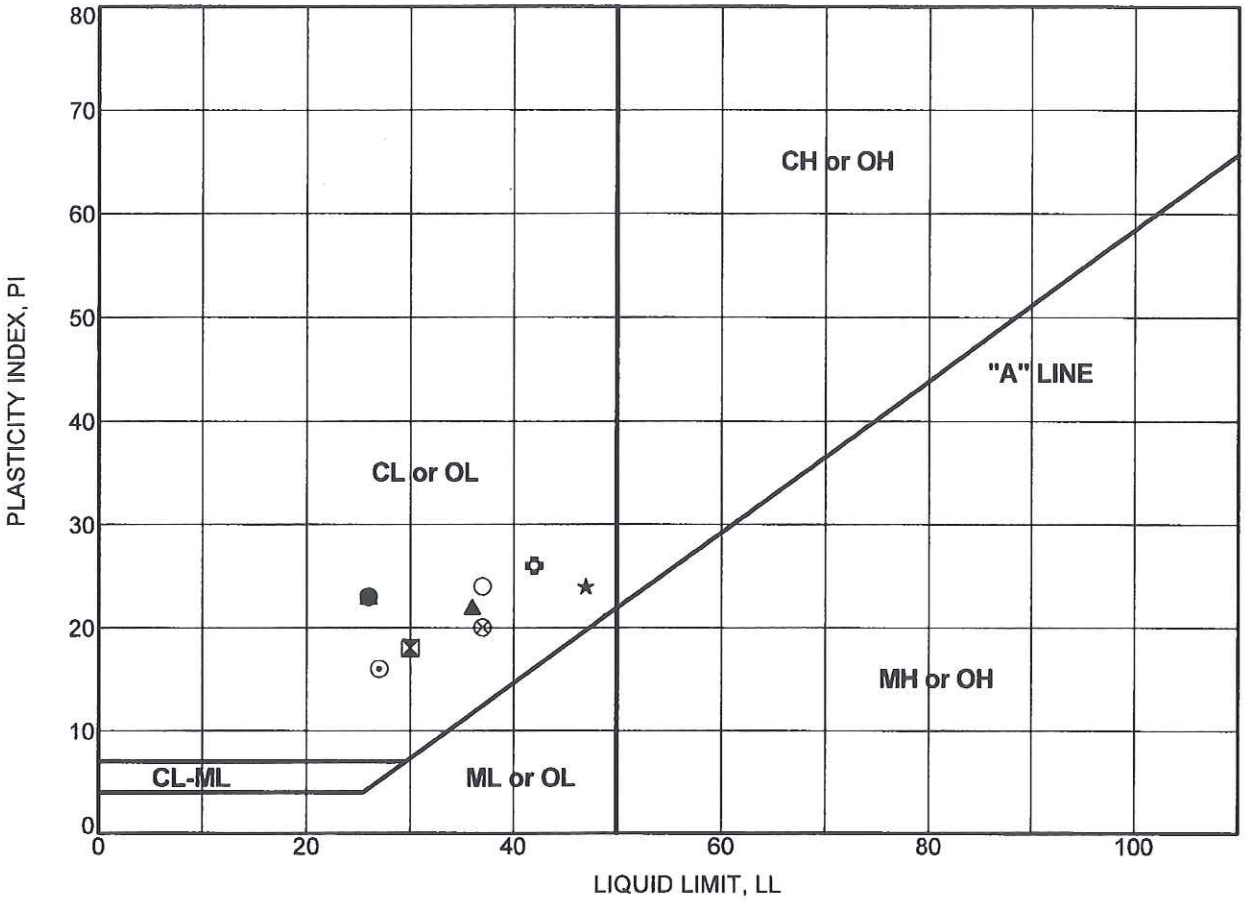
Unconfined Compression Tests
Strength tests were performed on selected undisturbed sample using unconfined compression machine. Unconfined compression test was performed in general accordance with ASTM Test Method D2166. The results are presented on the "Log of Test Borings."

Atterberg Limits
The Atterberg Limits were determined for selected samples of the fine-grained materials. These results were used to classify the soils, as well as to obtain an indication of the expansion potential with variations in moisture content. The Atterberg Limits were determined in general accordance with ASTM Test Method D 4318-93. The results of these tests are presented on Plate B-2, "Plasticity Chart".

Grain Size Classification
Grain size classification tests (ASTM Test Method D 422-63) were performed on selected samples of granular soil to aid in the classification. The results are presented on Plate B-3A to B-3C, "Grain Size Distribution Curves".

Consolidation Tests
Consolidation tests were performed on selected samples in accordance with ASTM Test Method D 2435. The test resulrs are presented on the Plates B-4A to B-4I.

Plasticity Charts (Atterberg Limits)



PLASTICITY CHART

Boring Number	Sample Number	Depth (feet)	Test Symbol	Moisture Content (%)	LL	PL	PI	Description
A-09-001	MC-3	16.0	●	25	26	3	23	SILT (ML)
A-09-001	MC-4	21.0	⊠	24	30	12	18	LEAN CLAY (CL)
A-09-001	MC-9	46.0	▲	30	36	14	22	LEAN CLAY (CL)
A-09-001	MC-11	56.0	★	33	47	23	24	LEAN CLAY (CL)
A-09-002	MC-3	16.0	⊙	24	27	11	16	LEAN CLAY (CL)
A-09-002	MC-7	36.0	✿	29	42	16	26	SILT WITH SAND (SM)
A-09-003	MC-1	5.0	○	26	37	13	24	LEAN CLAY (CL)
A-09-003	MC-3	16.0	△	24	26	3	23	SILT (ML)
A-09-003	MC-6	31.0	⊗	26	37	17	20	LEAN CLAY (CL)

Grain Size Distribution Curves

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GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

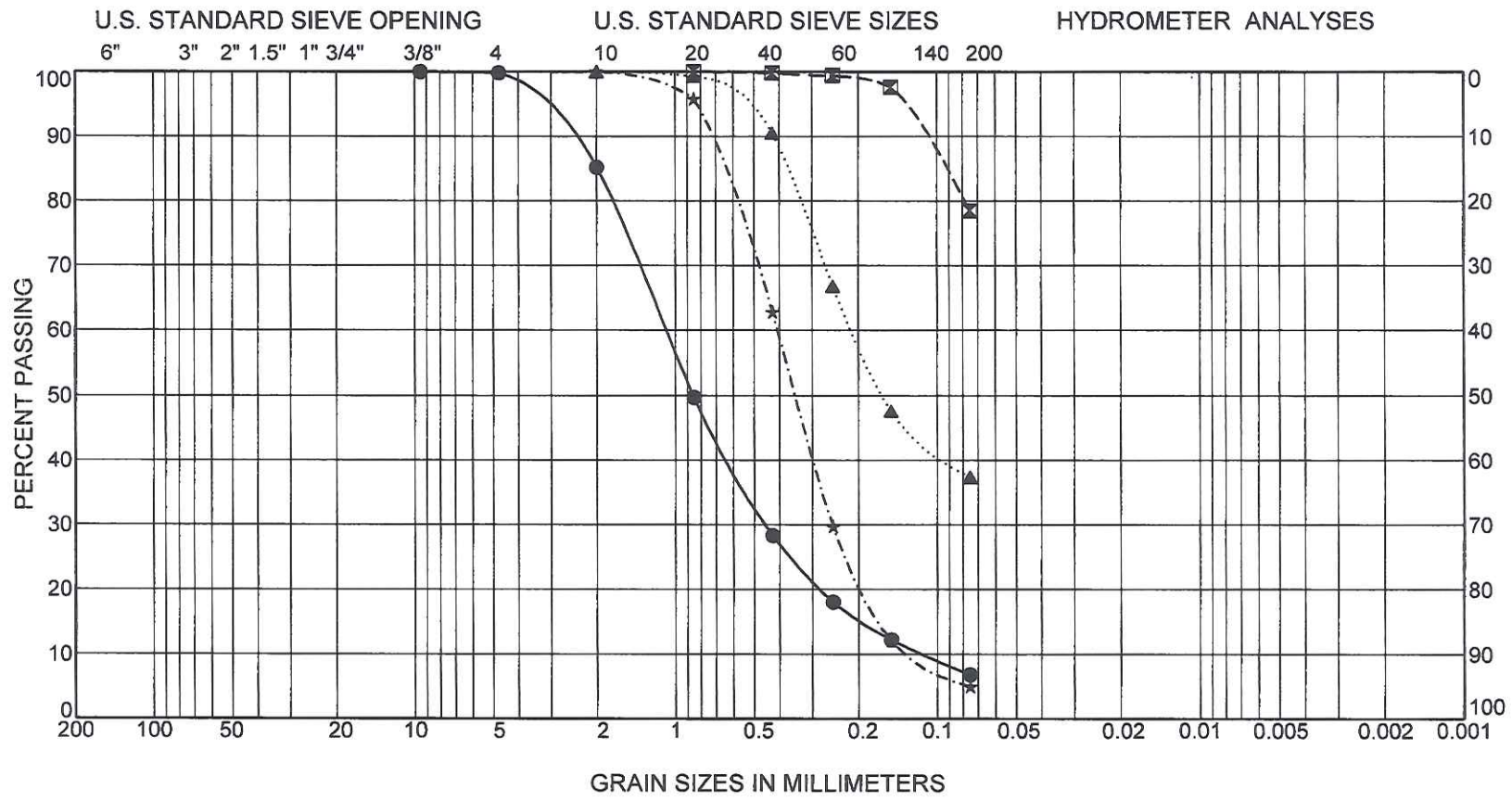
JOB NO: 207145.10

RTE 4 CROSSTOWN CONNECTOR
STOCKTON, CALIFORNIA

PLATE NO: B-3A

GRAIN SIZE DISTRIBUTION CURVES

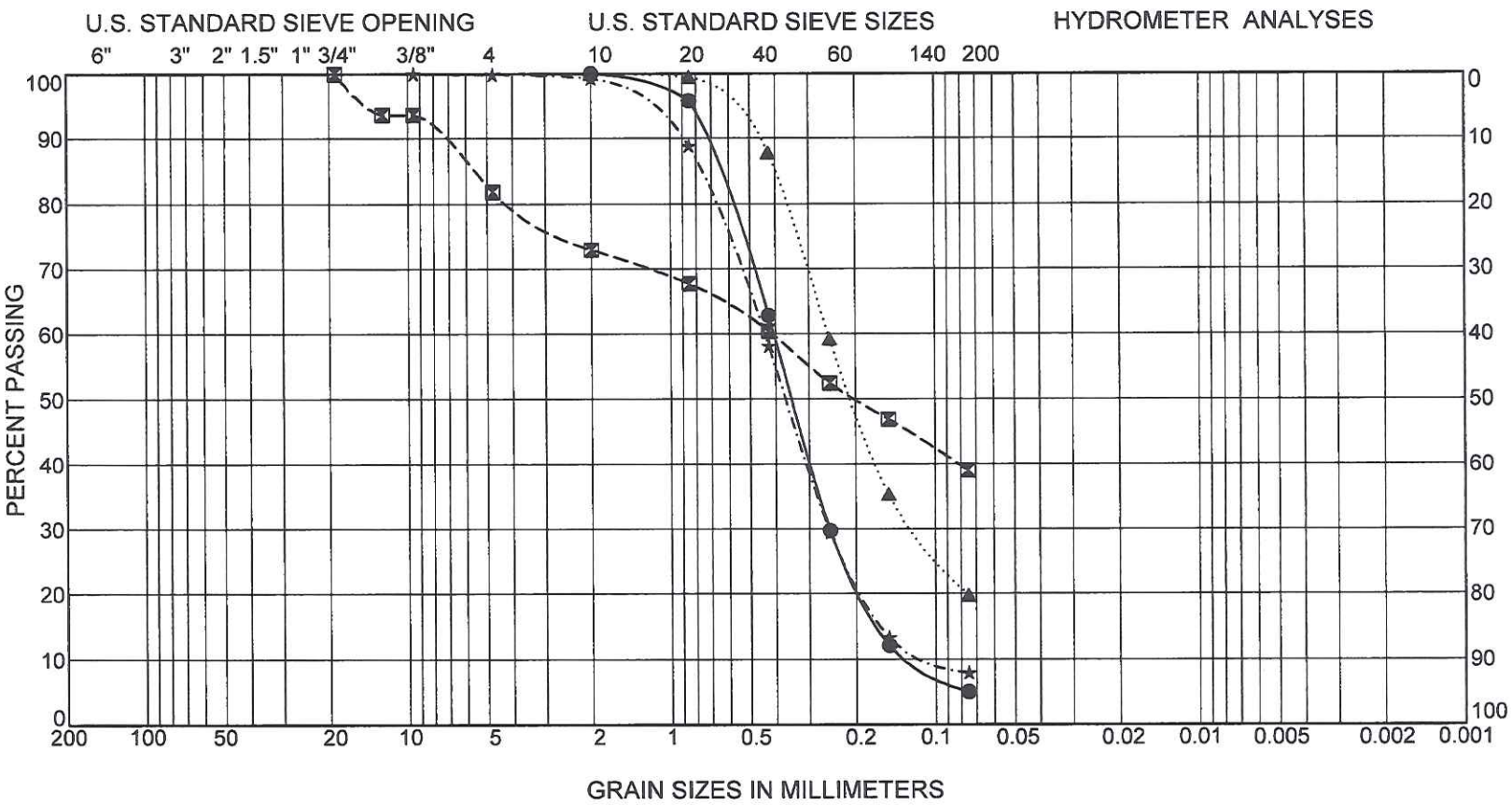
COBBLES	GRAVEL		SAND			SILT AND CLAY
	coarse	fine	coarse	medium	fine	



Boring Number	Sample Number	Depth (feet)	Symbol	LL	PI	Description
A-09-001	SPT-6	31.0	●			WELL GRADED SAND WITH SILT (SW-SM)
A-09-001	MC-8	41.0	⊠			LEAN CLAY WITH SAND (CL)
A-09-001	MC-17	101.0	▲			SILTY SAND (SM)
A-09-002	MC-5	26.0	★			POORLY GRADED SAND WITH SILT (SP-SM)

GRAIN SIZE DISTRIBUTION CURVES

COBBLES	GRAVEL		SAND			SILT AND CLAY
	coarse	fine	coarse	medium	fine	



Boring Number	Sample Number	Depth (feet)	Symbol	LL	PI	Description
A-09-002	MC-7	36.0	●	42	26	SILT WITH SAND (SM)
A-09-002	MC-11	56.0	⊠			SANDY SILT (ML)
A-09-003	MC-5	26.0	▲			SILTY SAND (SM)
A-09-003	MC-10	51.0	★			POORLY GRADED SAND WITH SILT (SP-SM)

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MATERIALS ENGINEERING

RTE 4 CROSSTOWN CONNECTOR
STOCKTON, CALIFORNIA

JOB NO: 207145.10

PLATE NO: B-3B

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

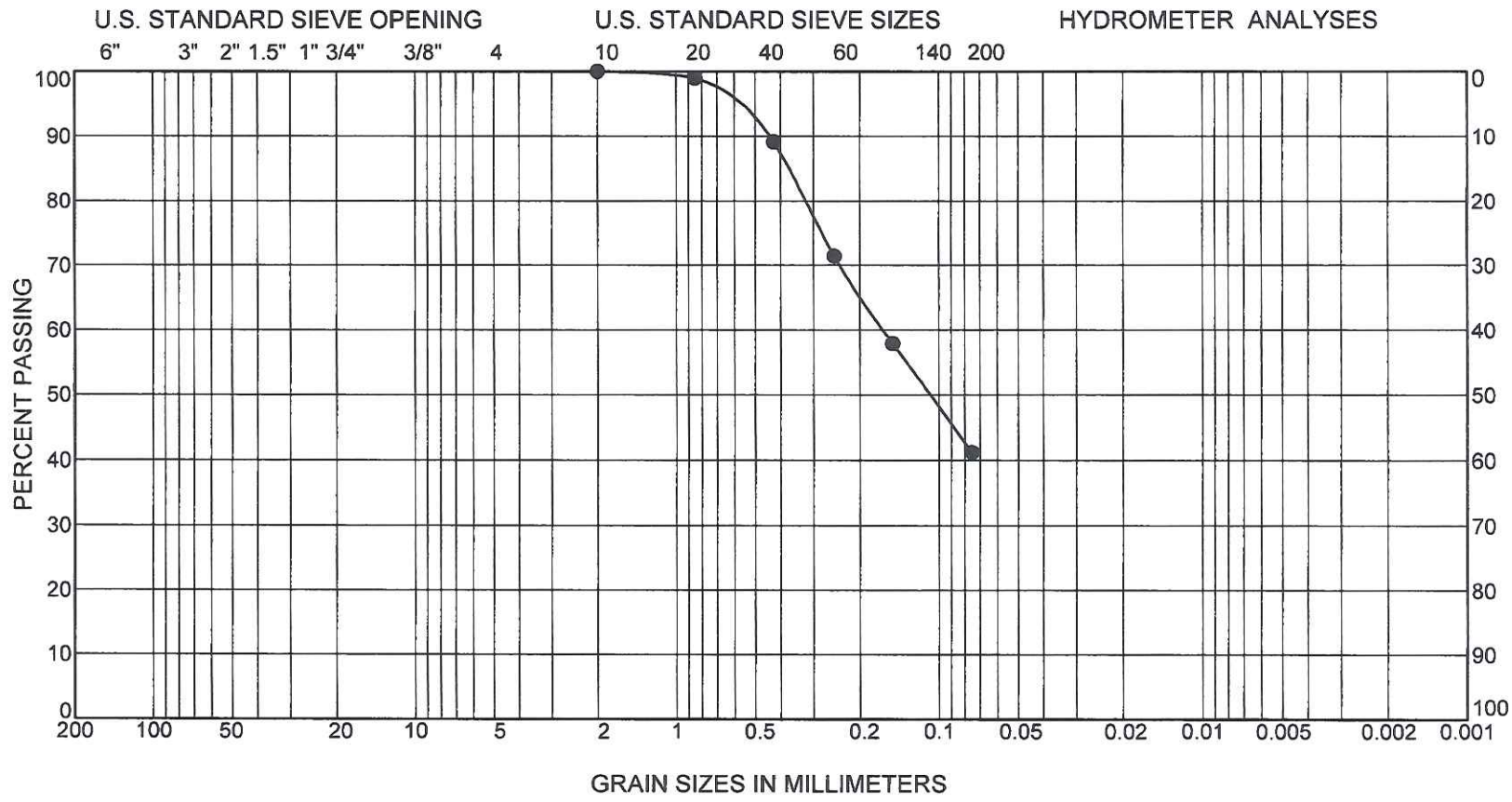
JOB NO: 207145.10

RTE 4 CROSSTOWN CONNECTOR
STOCKTON, CALIFORNIA

PLATE NO: B-3C

GRAIN SIZE DISTRIBUTION CURVES

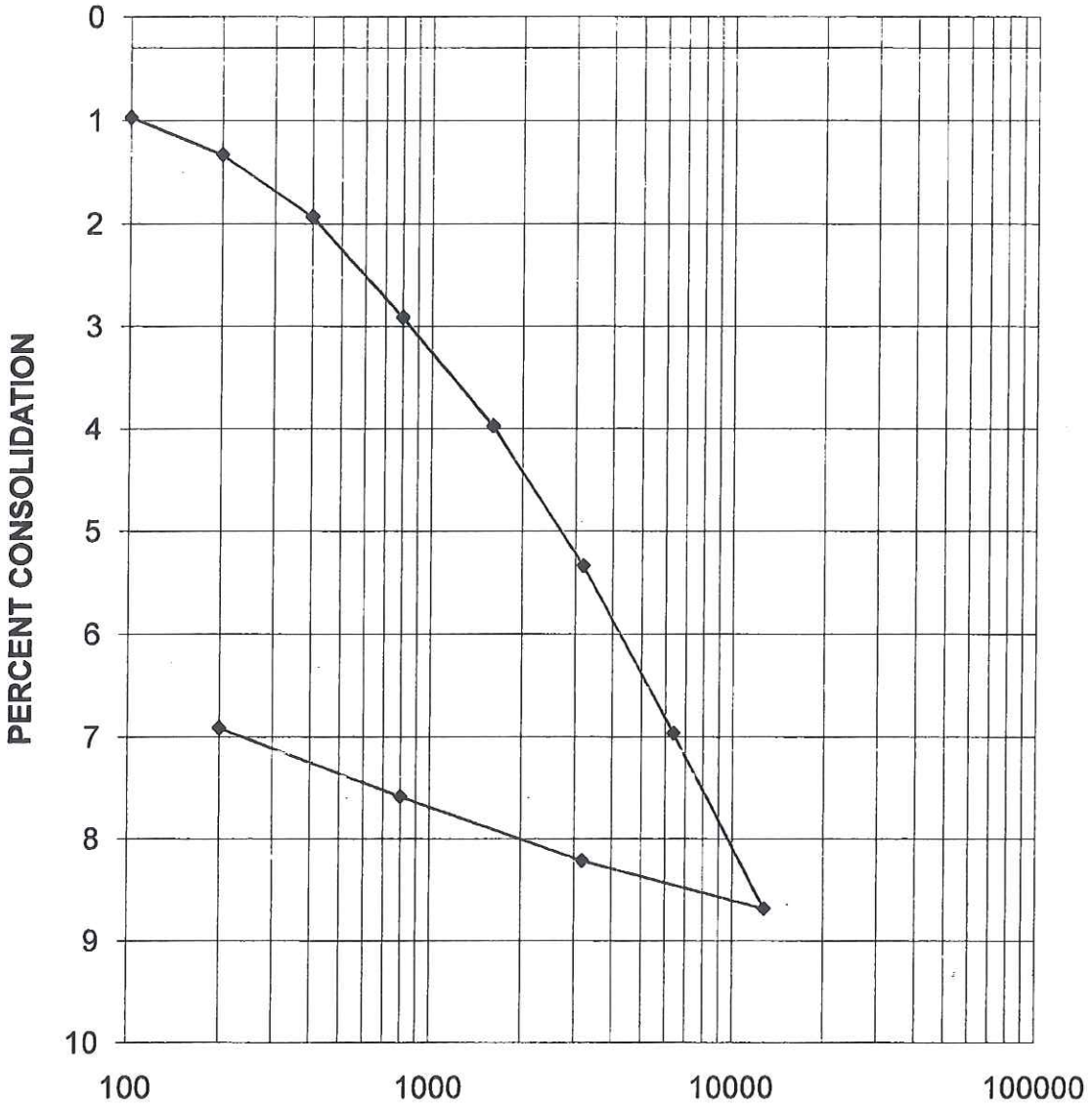
COBBLES	GRAVEL		SAND			SILT AND CLAY
	coarse	fine	coarse	medium	fine	



Boring Number	Sample Number	Depth (feet)	Symbol	LL	PI	Description
A-09-003	MC-11	56.0	●			SILTY SAND (SM)

Consolidation Test Results

CONSOLIDATION TEST RESULTS



	MOISTURE CONTENT %	DRY DENSITY PCF	HEIGHT (INCHES)	DIAMETER (INCHES)
INITIAL	25.0	101.9	1.0000	2.416
FINAL	21.8	109.4	0.9309	2.416

BORING NO.	B1	SAMPLE NO.	3	ELEV. OR DEPTH	16'
DESCRIPTION	Silt, light gray				

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

Rte 4 Crosstown Connector
BKF ENGINEERING

DATE	JOB NO:
12/07/09	207145.PFR

Reported by: Prav Dayah

PLATE NO.: B-4A

CONSOLIDATION TEST - TIME CURVE

JOB #: 207145.PFR

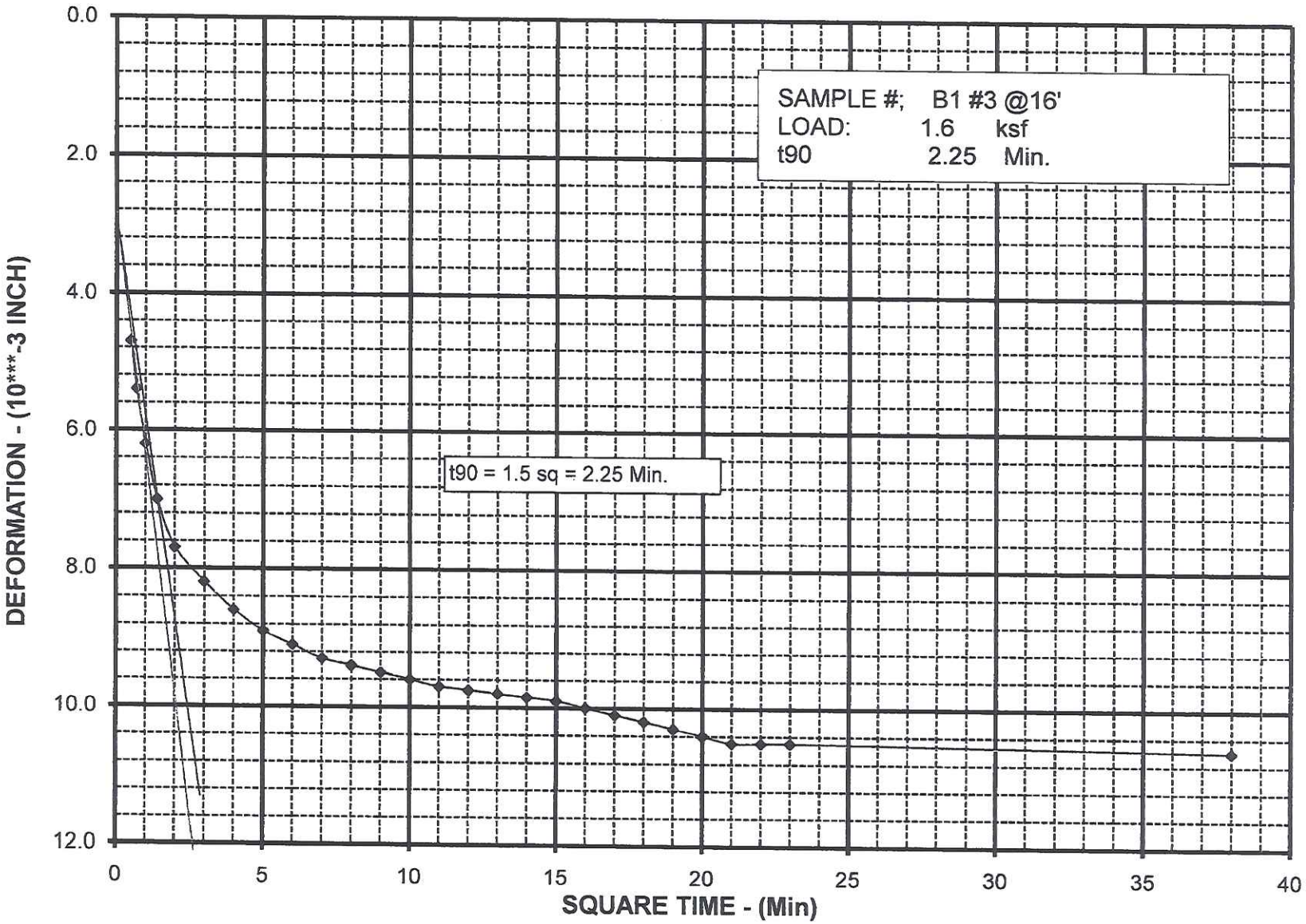


PLATE NO.: B-4B

CONSOLIDATION TEST - TIME CURVE

JOB #: 207145.PFR

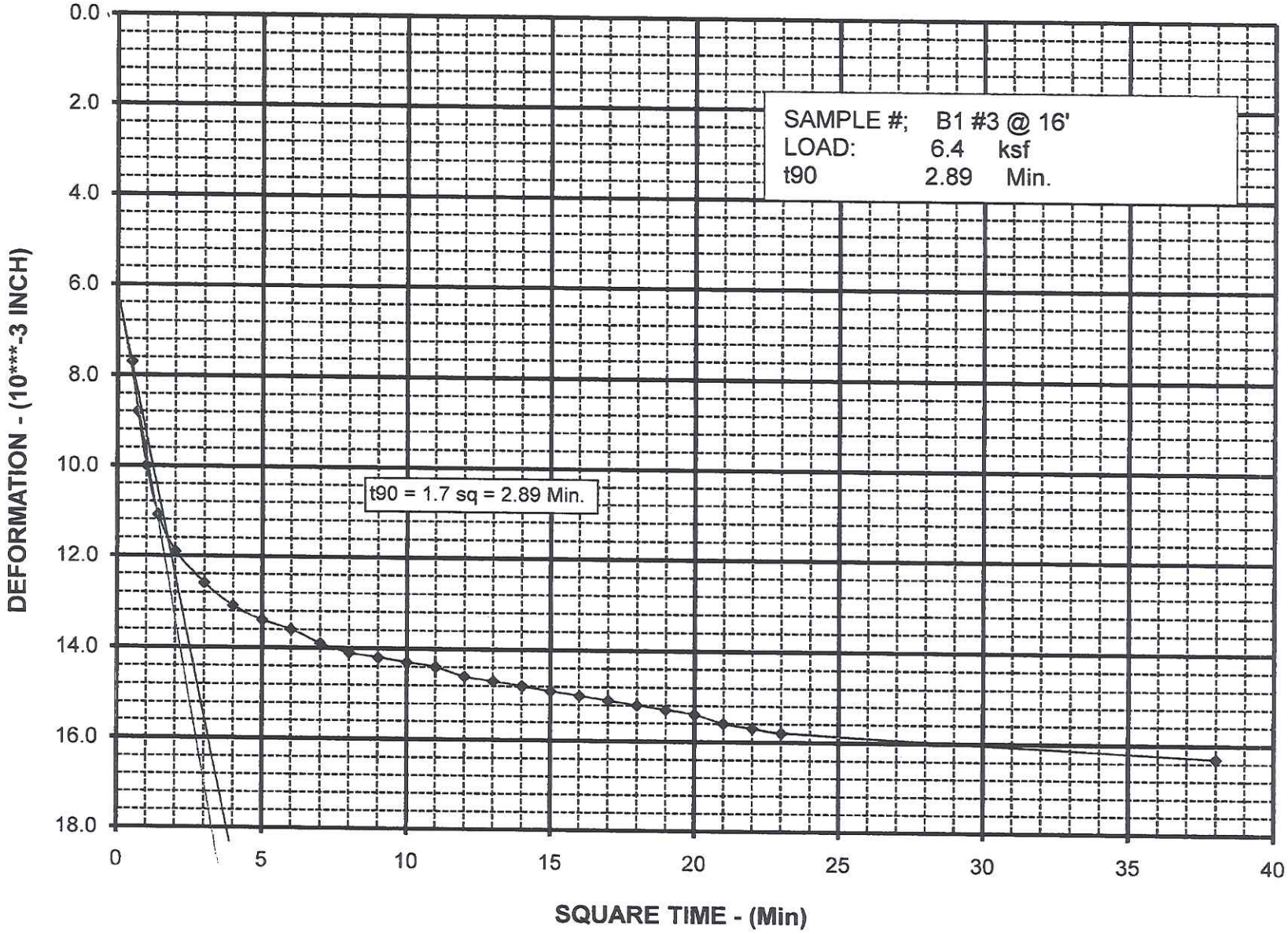
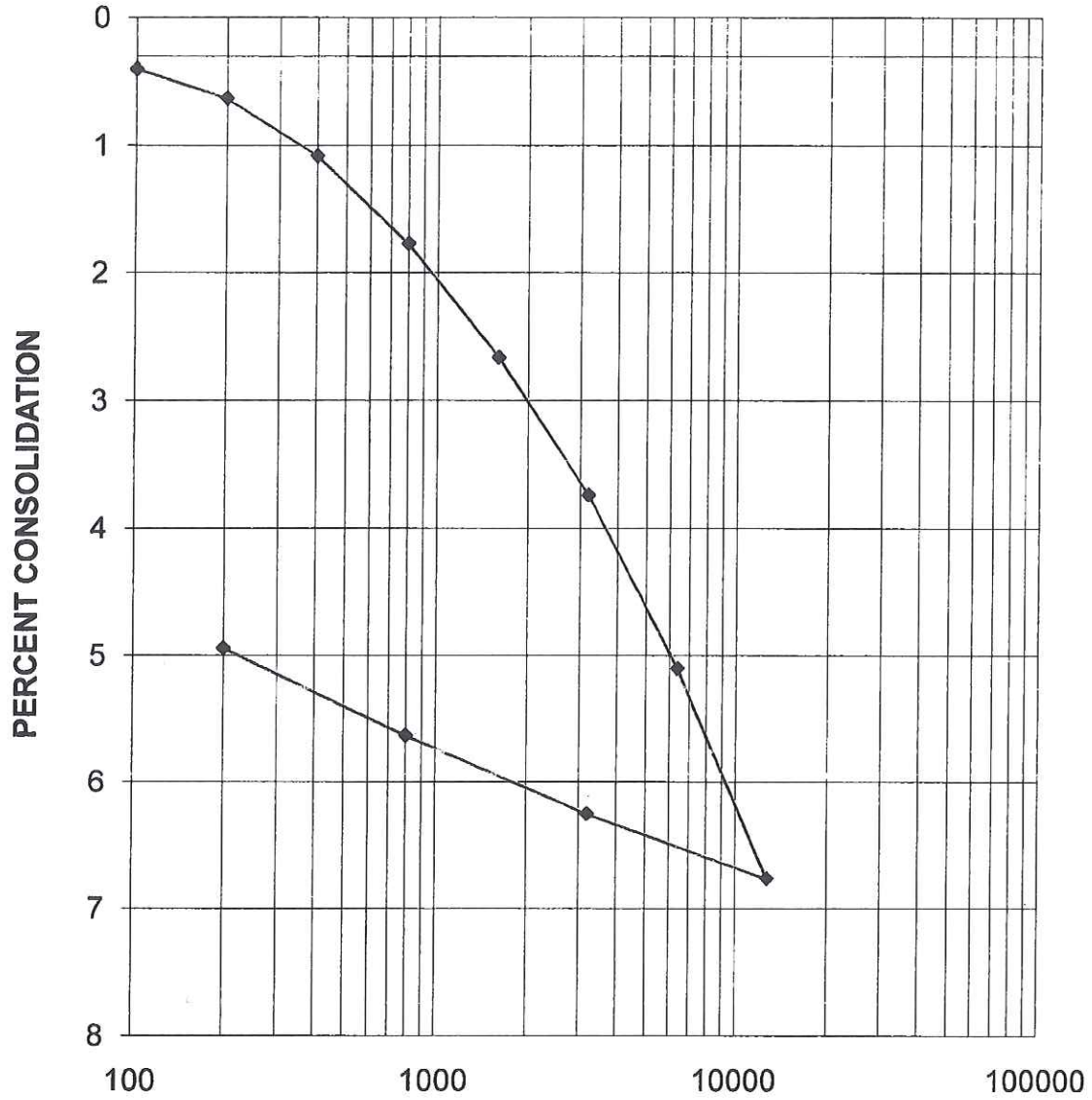


PLATE NO.: B-4C

CONSOLIDATION TEST RESULTS



	MOISTURE CONTENT %	DRY DENSITY PCF	HEIGHT (INCHES)	DIAMETER (INCHES)
INITIAL	24.2	103.1	1.0000	2.416
FINAL	21.6	108.5	0.9506	2.416

BORING NO.	B3	SAMPLE NO.	3	ELEV. OR DEPTH	16'
DESCRIPTION	Lean clay, brown				

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GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

Rte 4 Crosstown Connector
BKF ENGINEERING

DATE	JOB NO:
12/07/09	207145.PFR

Reported by: Prav Dayah

PLATE NO.: B-4D

CONSOLIDATION TEST - TIME CURVE

JOB #: 207145.PFR

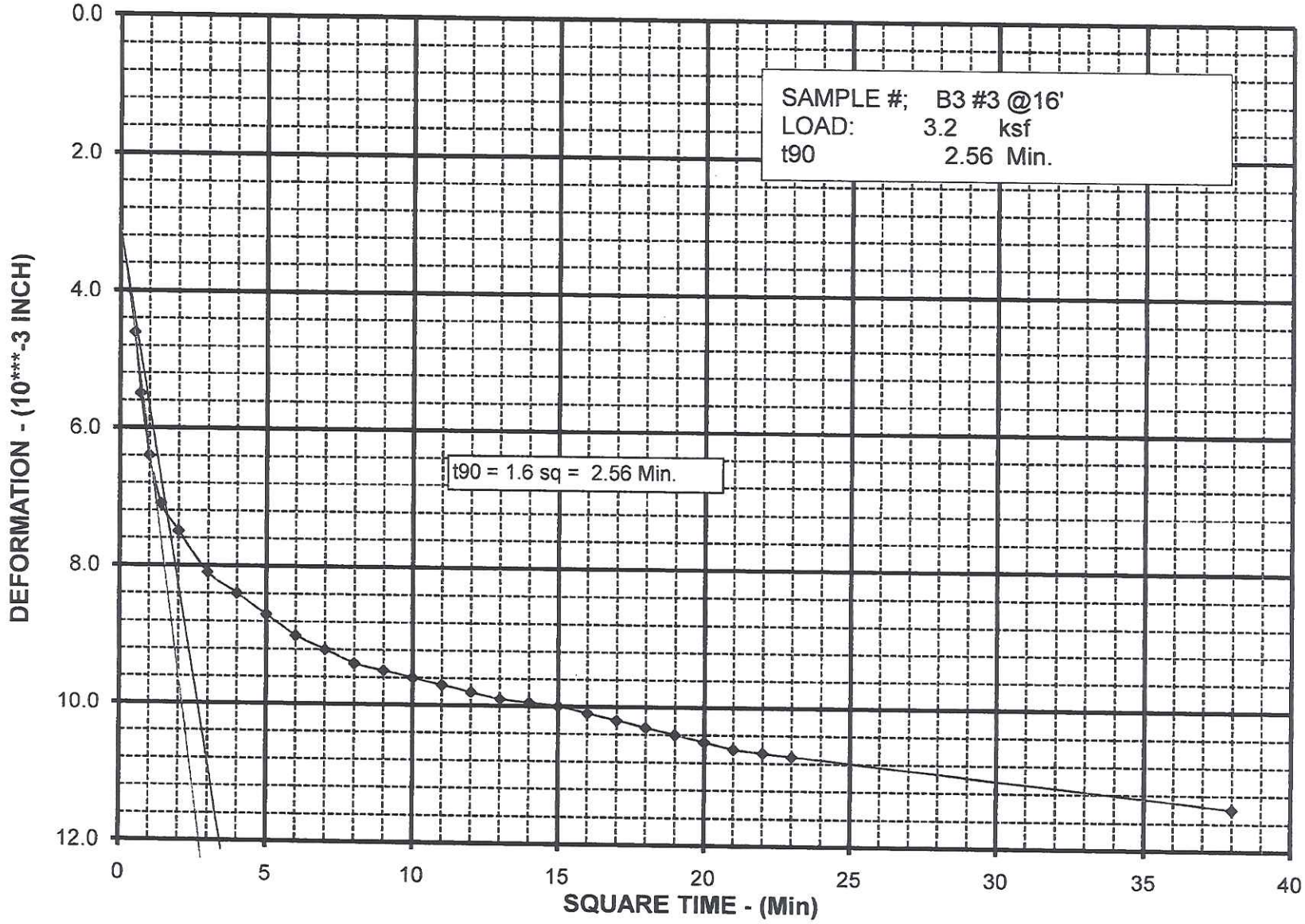


PLATE NO.: B-4E

CONSOLIDATION TEST - TIME CURVE

JOB #: 207145.PFR

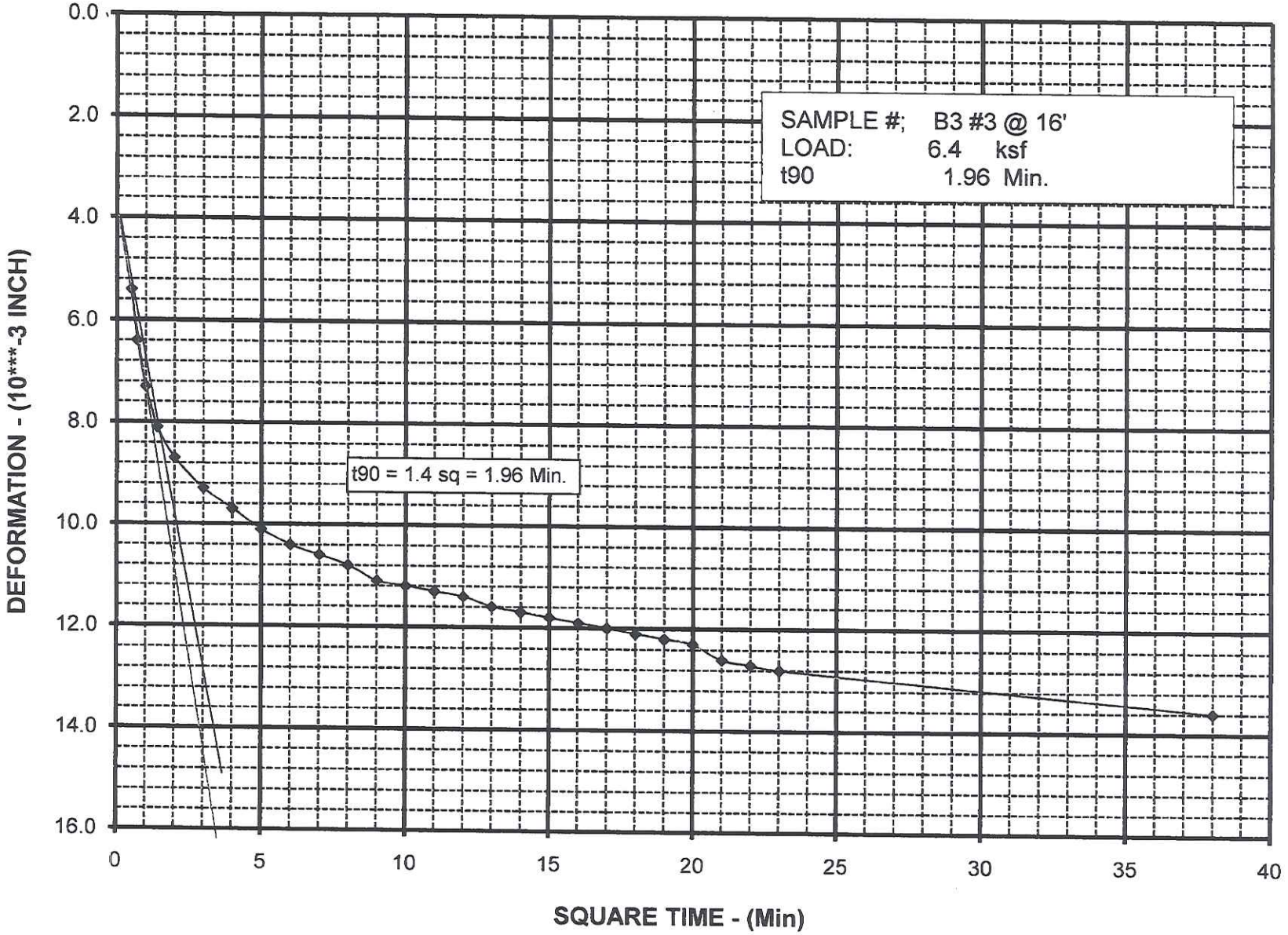
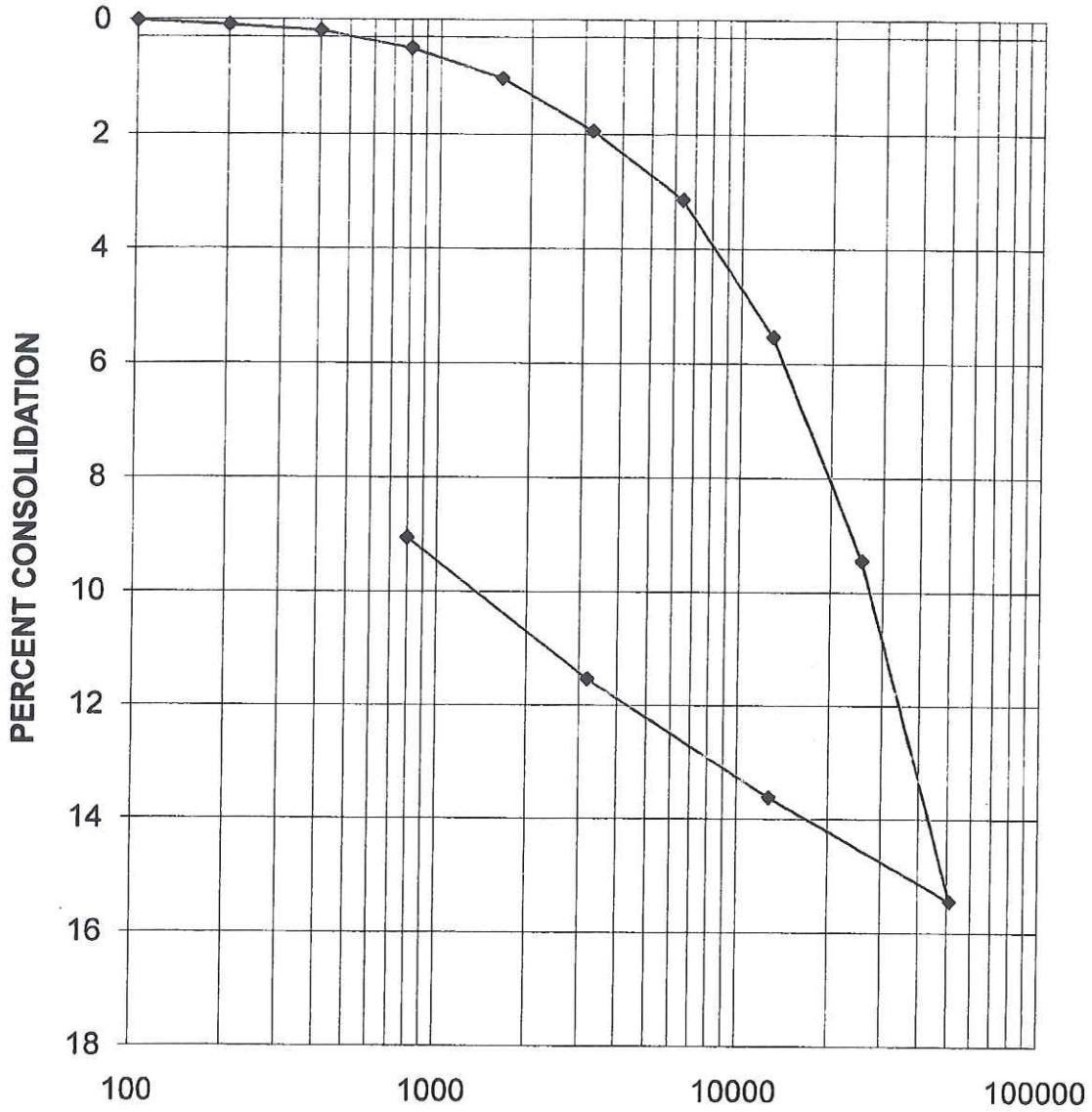


PLATE NO.: B-4F

CONSOLIDATION TEST RESULTS



	MOISTURE CONTENT %	DRY DENSITY PCF	HEIGHT (INCHES)	DIAMETER (INCHES)
INITIAL	29.2	98.7	1.0000	2.416
FINAL	23.8	108.5	0.9095	2.416

BORING NO.	B3	SAMPLE NO.	8	ELEV. OR DEPTH	41'
DESCRIPTION	Lean clay, bluish gray				

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

Rte 4 Crosstown Connector
BKF ENGINEERING

DATE	JOB NO:
12/7/2009	207143.PFR

Reported by: Prav Dayah

PLATE NO.: B-4G

CONSOLIDATION TEST - TIME CURVE

JOB #: 207143.PFR

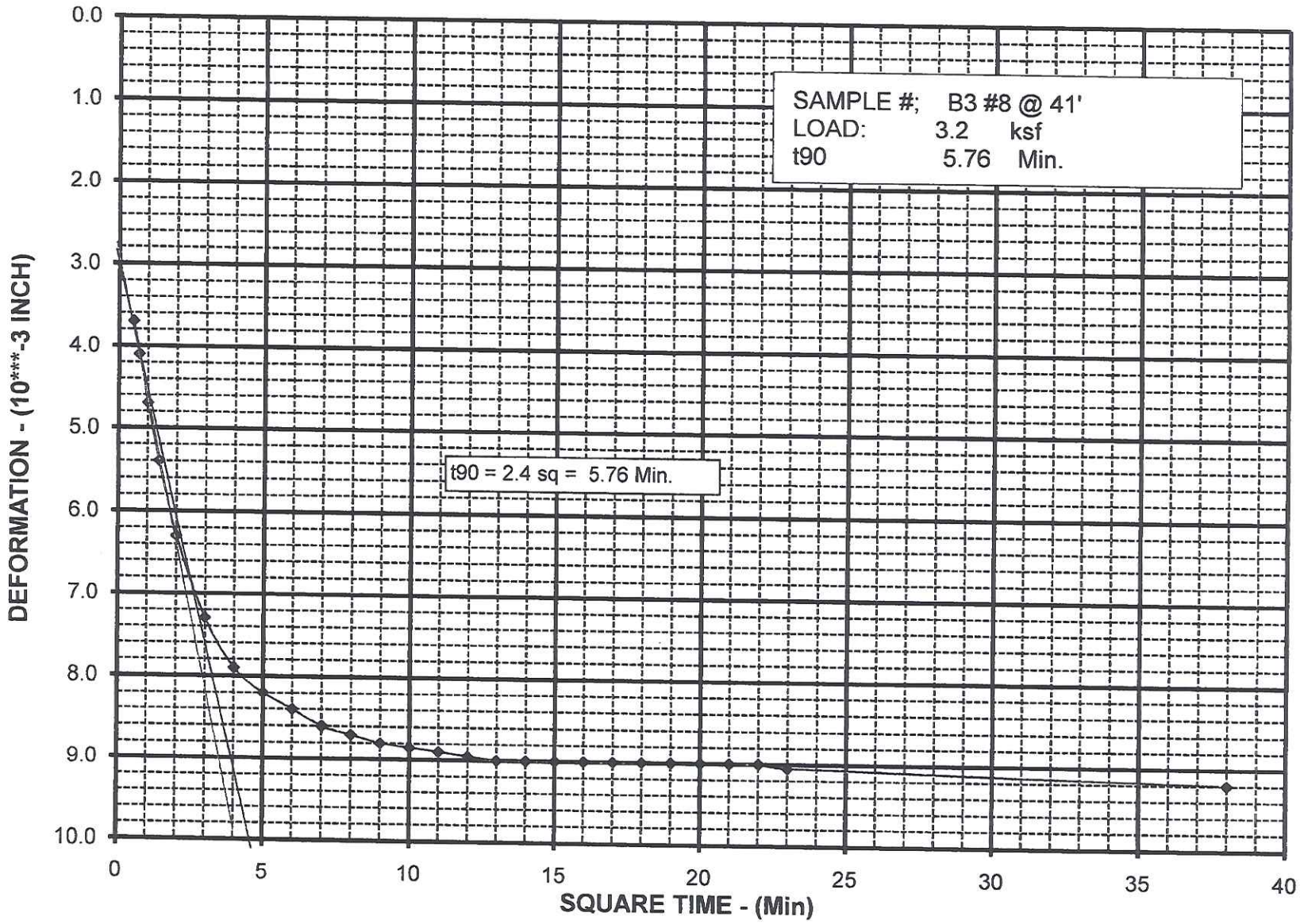


PLATE NO.: B-4H

CONSOLIDATION TEST - TIME CURVE

JOB #: 207143.PFR

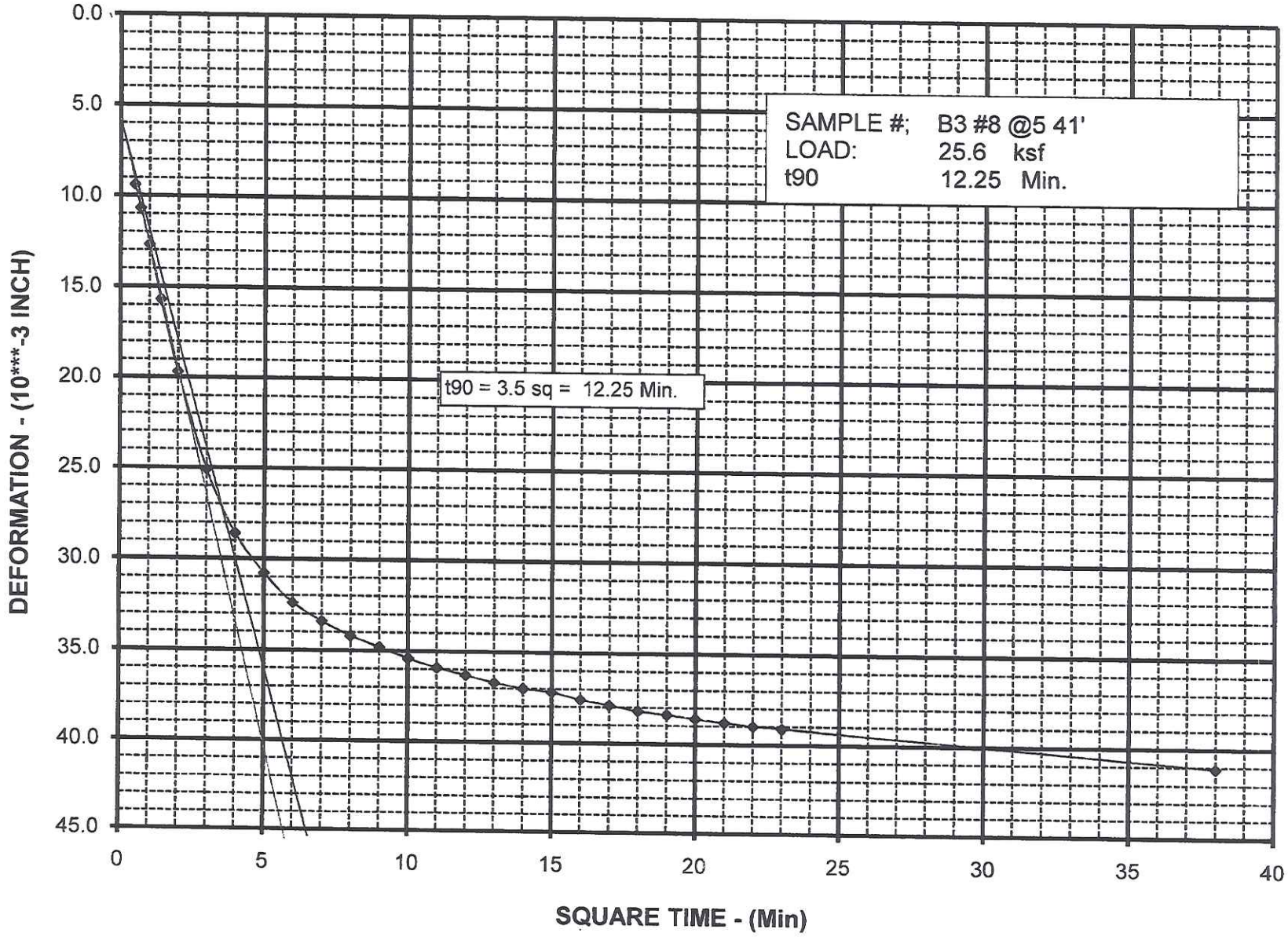


PLATE NO.: B-4I

APPENDIX E
ARS CURVE DEVELOPMENT

LIST OF ATTACHMENTS

The following are attached and complete this appendix.

Development of Shear Wave Velocity Parameter V_{s30}
Development of Probabilistic and Deterministic Response Spectrums

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Development of Shear Wave Velocity Parameter V_{s30}

Project Name	SR-4 Expansion	Calc by:	Jim Wetenkamp
Project Number	111904.G01	Checked by:	Carl Henderson
Date	April 28, 2011		
CPT Number	CPT-11-001	Average small strain sheer wave velocity in meters per second	*Vs30= 221*
CPT Max Depth (ft)	90.05		
Total d_i/V_i	0.42249		

1. For cohesive soils, Mayne and Rix (1995) is used to calculate V_i. $V_S = 1.75(q_t)^{0.627}$
For cohesionless soils, Mayne (2007) is used to calculate V_i. $V_S=277(q_t)^{0.13}(s'_{vo})^{0.27}$

2. If CPT depth is less than 100 feet, Boore (2004) is used to extrapolate to 100 feet.
$V_{S30} = [1.45-(0.015d)]V_{s(d)}$, where d = CPT depth (m), $V_{s(d)}$ = time averaged velocity (m/sec)

Depth (ft)	Layer Thickness (ft)	qt (tsf)	Soil Type	s'_{vo} (ksf)	Shear Wave Velocity (V_i) (m/sec)	d_i/V_i
0.57	0.57	157.15	7	0.07	84.60	0.00674
0.64	0.07	153.53	6	0.08	717.77	0.00010
0.71	0.07	153.86	7	0.09	90.29	0.00078
0.79	0.08	133.07	6	0.1	656.20	0.00012
0.87	0.08	117.89	6	0.11	608.21	0.00013
0.94	0.07	103.47	11	0.12	92.68	0.00076
1.02	0.08	114.62	11	0.13	95.97	0.00083
1.1	0.08	98.92	11	0.14	96.05	0.00083
1.18	0.08	86.12	11	0.15	96.11	0.00083
1.27	0.09	74.78	11	0.16	96.02	0.00094
1.36	0.09	58.66	11	0.17	94.58	0.00095
1.43	0.07	90.12	11	0.18	101.56	0.00069
1.52	0.09	149.55	12	0.2	111.60	0.00081
1.59	0.07	155.55	12	0.21	113.66	0.00062
1.68	0.09	158.18	7	0.22	115.35	0.00078
1.75	0.07	147.01	7	0.23	115.64	0.00061
1.84	0.09	147.6	7	0.24	117.03	0.00077
1.91	0.07	145	7	0.25	118.06	0.00059
1.97	0.06	137.16	7	0.26	118.46	0.00051
2.05	0.08	117.48	6	0.27	606.89	0.00013
2.13	0.08	96.67	5	0.28	537.05	0.00015
2.21	0.08	69.74	11	0.29	111.73	0.00072
2.3	0.09	51.86	3	0.3	363.45	0.00025
2.38	0.08	45.39	3	0.31	334.31	0.00024
2.46	0.08	38.35	3	0.32	300.79	0.00027
2.54	0.08	34.07	3	0.33	279.28	0.00029
2.62	0.08	29.12	3	0.34	253.10	0.00032
2.7	0.08	25.23	3	0.36	231.34	0.00035
2.78	0.08	25.38	3	0.37	232.20	0.00034
2.87	0.09	24.24	3	0.38	225.60	0.00040
2.95	0.08	21.75	3	0.39	210.78	0.00038
3.03	0.08	20.94	3	0.4	205.82	0.00039
3.11	0.08	21.11	3	0.41	206.87	0.00039
3.2	0.09	21.89	3	0.42	211.63	0.00043
3.28	0.08	24.56	3	0.43	227.46	0.00035

Project Name	SR-4 Expansion	Calc by:	Jim Wetenkamp
Project Number	111904.G01	Checked by:	Carl Henderson
Date	April 28, 2011		
CPT Number	CPT-11-002	Average small strain sheer wave velocity in meters per second	*Vs30= 243*
CPT Max Depth (ft)	90.07		
Total d_i/V_i	0.38454		

1. For cohesive soils, Mayne and Rix (1995) is used to calculate V_i. $V_S = 1.75(q_t)^{0.627}$
For cohesionless soils, Mayne (2007) is used to calculate V_i. $V_S=277(q_t)^{0.13}(s'_{vo})^{0.27}$

2. If CPT depth is less than 100 feet, Boore (2004) is used to extrapolate to 100 feet.
$V_{S30} = [1.45-(0.015d)]V_{s(d)}$, where d = CPT depth (m), $V_{s(d)}$ = time averaged velocity (m/sec)

Depth (ft)	Layer Thickness (ft)	qt (tsf)	Soil Type	s'_{vo} (ksf)	Shear Wave Velocity (V_i) (m/sec)	d_i/V_i
0.52	0.52	315.08	9	0.06	88.83	0.00585
0.56	0.04	309.3	9	0.07	92.38	0.00043
0.62	0.06	286.76	9	0.07	91.48	0.00066
0.68	0.06	273.29	9	0.08	94.25	0.00064
0.71	0.03	267.52	9	0.09	97.02	0.00031
0.77	0.06	251.12	9	0.09	96.23	0.00062
0.81	0.04	250.68	9	0.1	98.98	0.00040
0.86	0.05	251.39	9	0.11	101.60	0.00049
0.91	0.05	238.56	7	0.11	100.91	0.00050
0.96	0.05	240.02	7	0.12	103.39	0.00048
1.01	0.05	228.5	7	0.13	104.98	0.00048
1.06	0.05	230.17	7	0.13	105.08	0.00048
1.11	0.05	197.34	12	0.14	105.08	0.00048
1.17	0.06	169.69	12	0.15	104.97	0.00057
1.22	0.05	157.26	12	0.16	105.77	0.00047
1.29	0.07	128.53	11	0.17	104.73	0.00067
1.35	0.06	101.49	11	0.17	101.56	0.00059
1.41	0.06	72.82	11	0.18	98.79	0.00061
1.48	0.07	67.94	11	0.19	99.34	0.00070
1.54	0.06	60.98	11	0.2	99.32	0.00060
1.6	0.06	61.65	11	0.21	100.78	0.00060
1.66	0.06	65.98	11	0.21	101.67	0.00059
1.72	0.06	65.92	5	0.22	422.44	0.00014
1.77	0.05	59.51	5	0.23	396.19	0.00013
1.83	0.06	72.4	6	0.24	448.02	0.00013
1.89	0.06	78.76	6	0.25	472.31	0.00013
1.96	0.07	109.82	7	0.26	115.08	0.00061
2.03	0.07	140.72	7	0.26	118.85	0.00059
2.07	0.04	144.9	7	0.27	120.53	0.00033
2.12	0.05	151.53	8	0.28	122.43	0.00041
2.19	0.07	163.95	7	0.29	124.86	0.00056
2.25	0.06	150.08	7	0.3	124.57	0.00048
2.31	0.06	133.72	6	0.3	658.21	0.00009
2.37	0.06	125.93	6	0.31	633.90	0.00009
2.43	0.06	127.49	6	0.32	638.81	0.00009

Project Name	SR-4 Expansion	Calc by:	Jim Wetenkamp
Project Number	111904.G01	Checked by:	Carl Henderson
Date	April 28, 2011		
CPT Number	CPT-11-003	Average small strain sheer wave velocity in meters per second	*Vs30= 250*
CPT Max Depth (ft)	90.07		
Total d_i/V_i	0.37470		

1. For cohesive soils, Mayne and Rix (1995) is used to calculate V_i. $V_S = 1.75(q_t)^{0.627}$
For cohesionless soils, Mayne (2007) is used to calculate V_i. $V_S=277(q_t)^{0.13}(s'_{vo})^{0.27}$

2. If CPT depth is less than 100 feet, Boore (2004) is used to extrapolate to 100 feet.
$V_{S30} = [1.45-(0.015d)]V_{s(d)}$, where d = CPT depth (m), $V_{s(d)}$ = time averaged velocity (m/sec)

Depth (ft)	Layer Thickness (ft)	qt (tsf)	Soil Type	s'_{vo} (ksf)	Shear Wave Velocity (V_i) (m/sec)	d_i/V_i
0.52	0.52	6.76	3	0.06	101.30	0.00513
0.61	0.09	6.15	3	0.07	95.47	0.00094
0.7	0.09	5.83	3	0.08	92.32	0.00097
0.78	0.08	5.93	3	0.09	93.31	0.00086
0.86	0.08	5.94	3	0.1	93.41	0.00086
0.94	0.08	6.9	3	0.11	102.61	0.00078
1.02	0.08	15.83	3	0.12	172.71	0.00046
1.1	0.08	26.49	5	0.13	238.51	0.00034
1.19	0.09	31.37	5	0.14	265.19	0.00034
1.27	0.08	34.1	5	0.15	279.43	0.00029
1.36	0.09	40.87	6	0.16	313.03	0.00029
1.44	0.08	38.82	5	0.17	303.09	0.00026
1.53	0.09	33.97	5	0.19	278.76	0.00032
1.61	0.08	30.43	5	0.2	260.18	0.00031
1.7	0.09	27.22	5	0.21	242.61	0.00037
1.79	0.09	24.51	5	0.22	227.17	0.00040
1.87	0.08	20.55	5	0.23	203.41	0.00039
1.95	0.08	20.08	5	0.24	200.48	0.00040
2.03	0.08	18.99	5	0.25	193.59	0.00041
2.12	0.09	17.88	5	0.26	186.41	0.00048
2.21	0.09	16.27	5	0.27	175.70	0.00051
2.3	0.09	14.56	4	0.28	163.89	0.00055
2.39	0.09	12.39	3	0.3	148.11	0.00061
2.46	0.07	7.89	3	0.3	111.61	0.00063
2.5	0.04	8.86	3	0.31	120.03	0.00033
2.58	0.08	7.98	3	0.32	112.41	0.00071
2.67	0.09	7.92	3	0.33	111.88	0.00080
2.75	0.08	7.83	3	0.34	111.08	0.00072
2.84	0.09	10.25	3	0.35	131.51	0.00068
2.92	0.08	10.9	3	0.36	136.68	0.00059
3.01	0.09	14.97	3	0.37	166.76	0.00054
3.09	0.08	21.74	3	0.38	210.72	0.00038
3.18	0.09	21.9	3	0.39	211.69	0.00043
3.27	0.09	19.88	3	0.4	199.23	0.00045
3.36	0.09	19.87	3	0.42	199.16	0.00045

Project Name	SR-4 Expansion	Calc by:	Jim Wetenkamp
Project Number	111904.G01	Checked by:	Carl Henderson
Date	April 28, 2011		
CPT Number	CPT-11-004	Average small strain sheer wave velocity in meters per second	*Vs30= 237*
CPT Max Depth (ft)	90.07		
Total d_i/V_i	0.39526		

1. For cohesive soils, Mayne and Rix (1995) is used to calculate V_i. $V_S = 1.75(q_t)^{0.627}$
For cohesionless soils, Mayne (2007) is used to calculate V_i. $V_S=277(q_t)^{0.13}(s'_{vo})^{0.27}$

2. If CPT depth is less than 100 feet, Boore (2004) is used to extrapolate to 100 feet.
$V_{S30} = [1.45-(0.015d)]V_{s(d)}$, where d = CPT depth (m), $V_{s(d)}$ = time averaged velocity (m/sec)

Depth (ft)	Layer Thickness (ft)	qt (tsf)	Soil Type	s'_{vo} (ksf)	Shear Wave Velocity (V_i) (m/sec)	d_i/V_i
0.52	0.52	11.77	5	0.06	143.42	0.00363
0.62	0.10	13.3	5	0.07	154.84	0.00065
0.72	0.10	13.94	5	0.08	159.48	0.00063
0.81	0.09	14.83	5	0.09	165.79	0.00054
0.91	0.10	14.33	5	0.1	162.26	0.00062
1	0.09	13.46	5	0.11	156.01	0.00058
1.08	0.08	12.57	5	0.12	149.46	0.00054
1.17	0.09	11.45	5	0.13	140.97	0.00064
1.27	0.10	10.08	5	0.15	130.14	0.00077
1.36	0.09	9.76	4	0.16	127.53	0.00071
1.45	0.09	10.33	4	0.17	132.15	0.00068
1.53	0.08	11.2	4	0.18	139.03	0.00058
1.63	0.10	12.97	4	0.19	152.42	0.00066
1.72	0.09	14.19	4	0.2	161.26	0.00056
1.81	0.09	14.82	5	0.21	165.72	0.00054
1.9	0.09	14.42	4	0.22	162.90	0.00055
2	0.10	12.73	3	0.23	150.65	0.00066
2.1	0.10	11.92	3	0.24	144.57	0.00069
2.17	0.07	12.32	3	0.25	147.59	0.00047
2.25	0.08	11.6	3	0.26	142.12	0.00056
2.33	0.08	9.84	3	0.27	128.19	0.00062
2.42	0.09	8.39	3	0.28	115.99	0.00078
2.49	0.07	7.92	3	0.29	111.88	0.00063
2.57	0.08	7.61	3	0.3	109.11	0.00073
2.65	0.08	7.28	3	0.31	106.12	0.00075
2.73	0.08	6.06	3	0.32	94.59	0.00085
2.81	0.08	6.31	3	0.33	97.02	0.00082
2.89	0.08	5.98	3	0.34	93.81	0.00085
2.97	0.08	6.55	3	0.35	99.32	0.00081
3.05	0.08	7.83	3	0.36	111.08	0.00072
3.13	0.08	9.75	3	0.37	127.45	0.00063
3.22	0.09	12.65	3	0.38	150.06	0.00060
3.29	0.07	15.48	3	0.39	170.30	0.00041
3.38	0.09	15.7	3	0.4	171.82	0.00052
3.46	0.08	15.15	3	0.41	168.02	0.00048

Project Name	SR-4 Expansion	Calc by:	Jim Wetenkamp
Project Number	111904.G01	Checked by:	Carl Henderson
Date	April 28, 2011		
CPT Number	CPT-11-005	Average small strain sheer wave velocity in meters per second	*Vs30= 255*
CPT Max Depth (ft)	90.01		
Total d_i/V_i	0.36593		

1. For cohesive soils, Mayne and Rix (1995) is used to calculate V_i. $V_S = 1.75(q_t)^{0.627}$
For cohesionless soils, Mayne (2007) is used to calculate V_i. $V_S=277(q_t)^{0.13}(s'_{vo})^{0.27}$

2. If CPT depth is less than 100 feet, Boore (2004) is used to extrapolate to 100 feet.
V_{S30} = [1.45-(0.015d)]$V_{s(d)}$, where d = CPT depth (m), $V_{s(d)}$ = time averaged velocity (m/sec)

Depth (ft)	Layer Thickness (ft)	qt (tsf)	Soil Type	s'_{vo} (ksf)	Shear Wave Velocity (V_i) (m/sec)	d_i/V_i
0.51	0.51	19.29	4	0.06	195.50	0.00261
0.56	0.05	18.4	3	0.07	189.79	0.00026
0.62	0.06	18.24	4	0.07	188.76	0.00032
0.67	0.05	17.66	4	0.08	184.97	0.00027
0.72	0.05	17.2	4	0.09	181.94	0.00027
0.77	0.05	15.66	4	0.09	171.54	0.00029
0.83	0.06	13.24	4	0.1	154.41	0.00039
0.89	0.06	11.06	3	0.11	137.94	0.00043
0.94	0.05	9.36	3	0.11	124.23	0.00040
0.99	0.05	8.41	3	0.12	116.17	0.00043
1.04	0.05	7.2	3	0.12	105.39	0.00047
1.09	0.05	8.01	3	0.13	112.67	0.00044
1.15	0.06	8.57	3	0.14	117.55	0.00051
1.2	0.05	10.51	3	0.14	133.59	0.00037
1.26	0.06	12.85	3	0.15	151.54	0.00040
1.31	0.05	13.8	3	0.16	158.47	0.00032
1.38	0.07	17.1	3	0.17	181.27	0.00039
1.43	0.05	19.6	3	0.17	197.46	0.00025
1.48	0.05	21.2	3	0.18	207.42	0.00024
1.53	0.05	23.39	3	0.19	220.61	0.00023
1.58	0.05	23.39	3	0.19	220.61	0.00023
1.64	0.06	25.28	3	0.2	231.62	0.00026
1.69	0.05	25.67	3	0.21	233.86	0.00021
1.75	0.06	23	3	0.21	218.30	0.00027
1.79	0.04	24.86	3	0.22	229.20	0.00017
1.84	0.05	26.16	3	0.23	236.65	0.00021
1.9	0.06	26.07	3	0.24	236.14	0.00025
1.95	0.05	26.54	3	0.24	238.80	0.00021
2.01	0.06	28	3	0.25	246.95	0.00024
2.08	0.07	29.21	3	0.26	253.59	0.00028
2.15	0.07	26.72	3	0.27	239.81	0.00029
2.22	0.07	25.9	3	0.28	235.17	0.00030
2.28	0.06	23.16	3	0.29	219.25	0.00027
2.36	0.08	24.3	3	0.3	225.95	0.00035
2.43	0.07	24.53	3	0.31	227.29	0.00031

Project Name	SR-4 Expansion	Calc by:	Jim Wetenkamp
Project Number	111904.G01	Checked by:	Carl Henderson
Date	April 28, 2011		
CPT Number	CPT-11-006	Average small strain sheer wave velocity in meters per second	*Vs30= 244*
CPT Max Depth (ft)	90.03		
Total d_i/V_i	0.38325		

1. For cohesive soils, Mayne and Rix (1995) is used to calculate V_i. $V_S = 1.75(q_t)^{0.627}$
For cohesionless soils, Mayne (2007) is used to calculate V_i. $V_S=277(q_t)^{0.13}(s'_{vo})^{0.27}$

2. If CPT depth is less than 100 feet, Boore (2004) is used to extrapolate to 100 feet.
$V_{S30} = [1.45-(0.015d)]V_{s(d)}$, where d = CPT depth (m), $V_{s(d)}$ = time averaged velocity (m/sec)

Depth (ft)	Layer Thickness (ft)	qt (tsf)	Soil Type	s'_{vo} (ksf)	Shear Wave Velocity (V_i) (m/sec)	d_i/V_i
0.54	0.54	49.97	5	0.06	355.08	0.00152
0.62	0.08	42.44	5	0.07	320.52	0.00025
0.7	0.08	36.76	4	0.08	292.91	0.00027
0.78	0.08	34.61	4	0.09	282.05	0.00028
0.85	0.07	33.92	5	0.11	278.51	0.00025
0.93	0.08	36.66	5	0.12	292.41	0.00027
1.01	0.08	42.53	6	0.13	320.95	0.00025
1.09	0.08	44.07	6	0.14	328.18	0.00024
1.16	0.07	40.51	6	0.15	311.30	0.00022
1.24	0.08	35.62	6	0.16	287.18	0.00028
1.31	0.07	36.14	6	0.17	289.80	0.00024
1.39	0.08	30.24	6	0.18	259.16	0.00031
1.46	0.07	27.36	6	0.19	243.40	0.00029
1.55	0.09	22.18	5	0.2	213.38	0.00042
1.62	0.07	20.05	5	0.21	200.29	0.00035
1.7	0.08	17.36	5	0.22	182.99	0.00044
1.77	0.07	16.96	5	0.23	180.34	0.00039
1.84	0.07	17.03	5	0.23	180.81	0.00039
1.94	0.10	16.79	5	0.25	179.20	0.00056
2.01	0.07	16.8	5	0.26	179.27	0.00039
2.08	0.07	15.27	4	0.26	168.85	0.00041
2.16	0.08	13.9	4	0.27	159.19	0.00050
2.23	0.07	11.15	3	0.28	138.64	0.00050
2.3	0.07	9.06	3	0.29	121.72	0.00058
2.38	0.08	7.13	3	0.3	104.74	0.00076
2.46	0.08	5.85	3	0.31	92.52	0.00086
2.53	0.07	4.82	3	0.32	81.94	0.00085
2.62	0.09	4.58	3	0.32	79.36	0.00113
2.69	0.07	5.72	3	0.33	91.23	0.00077
2.77	0.08	6.61	3	0.34	99.89	0.00080
2.86	0.09	8.15	3	0.35	113.90	0.00079
2.93	0.07	8.52	3	0.36	117.12	0.00060
3.02	0.09	9.3	3	0.37	123.73	0.00073
3.09	0.07	12.62	3	0.38	149.83	0.00047
3.18	0.09	14.65	3	0.39	164.52	0.00055

Project Name	SR-4 Expansion	Calc by:	Jim Wetenkamp
Project Number	111904.G01	Checked by:	Carl Henderson
Date	April 28, 2011		
CPT Number	CPT-11-007	Average small strain sheer wave velocity in meters per second	*Vs30= 250*
CPT Max Depth (ft)	90.04		
Total d_i/V_i	0.37341		

1. For cohesive soils, Mayne and Rix (1995) is used to calculate V_i. $V_S = 1.75(q_t)^{0.627}$
For cohesionless soils, Mayne (2007) is used to calculate V_i. $V_S=277(q_t)^{0.13}(s'_{vo})^{0.27}$

2. If CPT depth is less than 100 feet, Boore (2004) is used to extrapolate to 100 feet.
V_{S30} = [1.45-(0.015d)]$V_{s(d)}$, where d = CPT depth (m), $V_{s(d)}$ = time averaged velocity (m/sec)

Depth (ft)	Layer Thickness (ft)	qt (tsf)	Soil Type	s'_{vo} (ksf)	Shear Wave Velocity (V_i) (m/sec)	d_i/V_i
0.54	0.54	156.23	9	0.06	81.09	0.00666
0.62	0.08	149	8	0.07	84.02	0.00095
0.7	0.08	125.39	8	0.08	85.17	0.00094
0.77	0.07	107.28	7	0.09	86.16	0.00081
0.84	0.07	76.54	6	0.1	463.92	0.00015
0.91	0.07	53.52	5	0.11	370.70	0.00019
0.99	0.08	38.54	4	0.12	301.72	0.00027
1.06	0.07	30.17	3	0.13	258.78	0.00027
1.13	0.07	23.23	3	0.14	219.66	0.00032
1.21	0.08	21.24	3	0.15	207.67	0.00039
1.29	0.08	20.64	3	0.16	203.97	0.00039
1.36	0.07	25.58	3	0.17	233.34	0.00030
1.42	0.06	39.48	6	0.18	306.32	0.00020
1.5	0.08	35.66	6	0.19	287.38	0.00028
1.58	0.08	19.54	3	0.2	197.08	0.00041
1.65	0.07	17.16	3	0.21	181.67	0.00039
1.71	0.06	16.75	3	0.22	178.94	0.00034
1.79	0.08	17.9	4	0.23	186.54	0.00043
1.86	0.07	16.12	3	0.24	174.69	0.00040
1.93	0.07	14.58	3	0.25	164.03	0.00043
2.01	0.08	13.92	3	0.26	159.33	0.00050
2.09	0.08	12.79	3	0.27	151.09	0.00053
2.16	0.07	12.23	3	0.28	146.91	0.00048
2.23	0.07	12.33	3	0.28	147.66	0.00047
2.31	0.08	11.77	3	0.29	143.42	0.00056
2.38	0.07	11.28	3	0.3	139.65	0.00050
2.45	0.07	11.03	3	0.31	137.70	0.00051
2.53	0.08	11.44	3	0.32	140.89	0.00057
2.6	0.07	11.84	3	0.33	143.96	0.00049
2.67	0.07	11.75	3	0.34	143.27	0.00049
2.74	0.07	12.41	3	0.35	148.26	0.00047
2.82	0.08	12.98	3	0.36	152.50	0.00052
2.9	0.08	12.4	3	0.37	148.19	0.00054
2.98	0.08	12.71	3	0.38	150.50	0.00053
3.06	0.08	13.21	3	0.39	154.19	0.00052

Project Name	SR-4 Expansion	Calc by:	Jim Wetenkamp
Project Number	111904.G01	Checked by:	Carl Henderson
Date	April 28, 2011		
CPT Number	CPT-11-008	Average small strain sheer wave velocity in meters per second	*Vs30= 248*
CPT Max Depth (ft)	90.04		
Total d_i/V_i	0.37652		

1. For cohesive soils, Mayne and Rix (1995) is used to calculate V_i. $V_S = 1.75(q_t)^{0.627}$
For cohesionless soils, Mayne (2007) is used to calculate V_i. $V_S=277(q_t)^{0.13}(s'_{vo})^{0.27}$

2. If CPT depth is less than 100 feet, Boore (2004) is used to extrapolate to 100 feet.
$V_{S30} = [1.45-(0.015d)]V_{s(d)}$, where d = CPT depth (m), $V_{s(d)}$ = time averaged velocity (m/sec)

Depth (ft)	Layer Thickness (ft)	qt (tsf)	Soil Type	s'_{vo} (ksf)	Shear Wave Velocity (V_i) (m/sec)	d_i/V_i
0.56	0.56	3.58	3	0.06	68.00	0.00823
0.61	0.05	3.26	3	0.07	64.13	0.00078
0.67	0.06	3.42	3	0.07	66.08	0.00091
0.73	0.06	3.57	3	0.08	67.88	0.00088
0.79	0.06	5.67	3	0.08	90.73	0.00066
0.84	0.05	5.43	3	0.09	88.30	0.00057
0.9	0.06	7.2	3	0.1	105.39	0.00057
0.96	0.06	10.18	4	0.1	130.95	0.00046
1.01	0.05	16.16	5	0.11	174.96	0.00029
1.07	0.06	21.46	6	0.12	209.01	0.00029
1.13	0.06	24.28	6	0.12	225.84	0.00027
1.19	0.06	13.26	4	0.13	154.55	0.00039
1.26	0.07	12.2	5	0.14	146.69	0.00048
1.32	0.06	10.83	5	0.15	136.13	0.00044
1.38	0.06	10.91	5	0.15	136.76	0.00044
1.44	0.06	9.46	5	0.16	125.06	0.00048
1.51	0.07	9.38	5	0.17	124.40	0.00056
1.57	0.06	8.58	5	0.17	117.64	0.00051
1.63	0.06	8.49	5	0.18	116.86	0.00051
1.7	0.07	9.13	5	0.19	122.31	0.00057
1.76	0.06	9.13	5	0.19	122.31	0.00049
1.83	0.07	9.46	5	0.2	125.06	0.00056
1.89	0.06	9.86	5	0.21	128.35	0.00047
1.95	0.06	9.46	5	0.21	125.06	0.00048
2.03	0.08	8.73	5	0.22	118.92	0.00067
2.09	0.06	9.62	5	0.23	126.38	0.00047
2.16	0.07	9.62	5	0.24	126.38	0.00055
2.23	0.07	9.22	5	0.24	123.06	0.00057
2.29	0.06	8.25	4	0.25	114.78	0.00052
2.36	0.07	8.09	3	0.26	113.38	0.00062
2.42	0.06	7.93	3	0.26	111.97	0.00054
2.49	0.07	7.77	3	0.27	110.54	0.00063
2.55	0.06	7.85	3	0.28	111.26	0.00054
2.63	0.08	8.02	3	0.29	112.76	0.00071
2.69	0.06	6.56	3	0.29	99.41	0.00060

Project Name	SR-4 Expansion	Calc by:	Jim Wetenkamp
Project Number	111904.G01	Checked by:	Carl Henderson
Date	April 28, 2011		
CPT Number	CPT-11-009	Average small strain sheer wave velocity in meters per second	*Vs30= 258*
CPT Max Depth (ft)	50.05		
Total d_i/V_i	0.23723		

1. For cohesive soils, Mayne and Rix (1995) is used to calculate V_i. $V_S = 1.75(q_t)^{0.627}$
For cohesionless soils, Mayne (2007) is used to calculate V_i. $V_S=277(q_t)^{0.13}(s'_{vo})^{0.27}$

2. If CPT depth is less than 100 feet, Boore (2004) is used to extrapolate to 100 feet.
$V_{S30} = [1.45-(0.015d)]V_{s(d)}$, where d = CPT depth (m), $V_{s(d)}$ = time averaged velocity (m/sec)

Depth (ft)	Layer Thickness (ft)	qt (tsf)	Soil Type	s'_{vo} (ksf)	Shear Wave Velocity (V_i) (m/sec)	d_i/V_i
0.55	0.55	26.56	6	0.06	238.91	0.00230
0.64	0.09	36.01	6	0.07	289.15	0.00031
0.73	0.09	43.39	7	0.09	76.59	0.00118
0.81	0.08	49.99	7	0.1	80.27	0.00100
0.9	0.09	62.08	7	0.11	84.71	0.00106
0.99	0.09	74.99	7	0.12	88.88	0.00101
1.07	0.08	85.36	8	0.13	92.36	0.00087
1.16	0.09	94.14	8	0.14	95.44	0.00094
1.23	0.07	106.03	8	0.15	98.75	0.00071
1.32	0.09	128.73	8	0.16	103.05	0.00087
1.39	0.07	174.2	9	0.17	108.95	0.00064
1.48	0.09	215.91	9	0.18	113.78	0.00079
1.56	0.08	227.8	9	0.19	116.26	0.00069
1.64	0.08	225.64	9	0.21	119.29	0.00067
1.71	0.07	216.94	8	0.22	120.19	0.00058
1.79	0.08	206.72	8	0.23	120.88	0.00066
1.87	0.08	197.53	7	0.24	121.55	0.00066
1.94	0.07	188.97	9	0.25	122.19	0.00057
2.02	0.08	181.26	9	0.26	122.83	0.00065
2.1	0.08	171.73	8	0.27	123.22	0.00065
2.17	0.07	158.96	8	0.28	123.19	0.00057
2.23	0.06	131.49	7	0.28	120.19	0.00050
2.3	0.07	118.54	7	0.29	119.71	0.00058
2.37	0.07	99	7	0.3	118.01	0.00059
2.44	0.07	79.66	6	0.31	475.68	0.00015
2.51	0.07	60.63	6	0.32	400.85	0.00017
2.59	0.08	44	5	0.33	327.86	0.00024
2.67	0.08	37.69	4	0.34	297.53	0.00027
2.74	0.07	31.3	3	0.35	264.82	0.00026
2.82	0.08	26.99	3	0.36	241.33	0.00033
2.89	0.07	25.13	3	0.38	230.76	0.00030
2.96	0.07	24.47	3	0.38	226.94	0.00031
3.04	0.08	27.28	4	0.39	242.95	0.00033
3.11	0.07	29.92	5	0.4	257.44	0.00027
3.19	0.08	29.21	4	0.41	253.59	0.00032

Project Name	SR-4 Expansion	Calc by:	Jim Wetenkamp
Project Number	111904.G01	Checked by:	Carl Henderson
Date	April 28, 2011		
CPT Number	CPT-11-010	Average small strain sheer wave velocity in meters per second	*Vs30= 253*
CPT Max Depth (ft)	100		
Total d_i/V_i	0.39581		

1. For cohesive soils, Mayne and Rix (1995) is used to calculate V_i. $V_S = 1.75(q_t)^{0.627}$

For cohesionless soils, Mayne (2007) is used to calculate V_i. $V_S=277(q_t)^{0.13}(s'_{vo})^{0.27}$

2. If CPT depth is less than 100 feet, Boore (2004) is used to extrapolate to 100 feet.
$V_{S30} = [1.45-(0.015d)]V_{s(d)}$, where d = CPT depth (m), $V_{s(d)}$ = time averaged velocity (m/sec)

Depth (ft)	Layer Thickness (ft)	qt (tsf)	Soil Type	s'_{vo} (ksf)	Shear Wave Velocity (V_i) (m/sec)	d_i/V_i
0.54	0.54	15.05	3	0.06	167.32	0.00323
0.59	0.05	15.14	3	0.07	167.95	0.00030
0.63	0.04	15.06	3	0.07	167.39	0.00024
0.68	0.05	14.97	3	0.08	166.76	0.00030
0.72	0.04	15.14	3	0.09	167.95	0.00024
0.77	0.05	15.63	3	0.09	171.34	0.00029
0.82	0.05	16.91	3	0.1	180.01	0.00028
0.87	0.05	18.12	3	0.1	187.98	0.00027
0.92	0.05	13.35	3	0.11	155.21	0.00032
0.96	0.04	13.2	3	0.12	154.11	0.00026
1.01	0.05	13.12	3	0.12	153.53	0.00033
1.06	0.05	13.04	3	0.13	152.94	0.00033
1.11	0.05	13.03	3	0.13	152.87	0.00033
1.15	0.04	13.2	3	0.14	154.11	0.00026
1.2	0.05	14.9	4	0.15	166.28	0.00030
1.25	0.05	16.91	5	0.15	180.01	0.00028
1.3	0.05	20.06	5	0.16	200.36	0.00025
1.35	0.05	27.23	6	0.16	242.67	0.00021
1.4	0.05	32.15	6	0.17	269.30	0.00019
1.43	0.03	38.78	6	0.17	302.90	0.00010
1.47	0.04	44.42	6	0.18	329.82	0.00012
1.51	0.04	49.03	6	0.19	350.88	0.00011
1.55	0.04	51.53	6	0.19	361.99	0.00011
1.59	0.04	54.09	6	0.2	373.17	0.00011
1.63	0.04	55.56	6	0.2	379.50	0.00011
1.66	0.03	54.5	6	0.21	374.94	0.00008
1.69	0.03	54.32	6	0.21	374.16	0.00008
1.74	0.05	51.44	6	0.22	361.60	0.00014
1.79	0.05	48.72	6	0.22	349.49	0.00014
1.83	0.04	46.21	5	0.23	338.09	0.00012
1.88	0.05	43.7	5	0.24	326.45	0.00015
1.93	0.05	42.5	5	0.24	320.80	0.00016
1.97	0.04	37.08	5	0.25	294.50	0.00014
2.02	0.05	46.35	6	0.25	338.73	0.00015
2.08	0.06	52.41	6	0.26	365.86	0.00016

Project Name	SR-4 Expansion	Calc by:	Jim Wetenkamp
Project Number	111904.G01	Checked by:	Carl Henderson
Date	April 28, 2011		
CPT Number	CPT-11-011	Average small strain sheer wave velocity in meters per second	*Vs30= 277*
CPT Max Depth (ft)	50.01		
Total d_i/V_i	0.22026		

1. For cohesive soils, Mayne and Rix (1995) is used to calculate V_i. $V_S = 1.75(q_t)^{0.627}$
For cohesionless soils, Mayne (2007) is used to calculate V_i. $V_S=277(q_t)^{0.13}(s'_{vo})^{0.27}$

2. If CPT depth is less than 100 feet, Boore (2004) is used to extrapolate to 100 feet.
$V_{S30} = [1.45-(0.015d)]V_{s(d)}$, where d = CPT depth (m), $V_{s(d)}$ = time averaged velocity (m/sec)

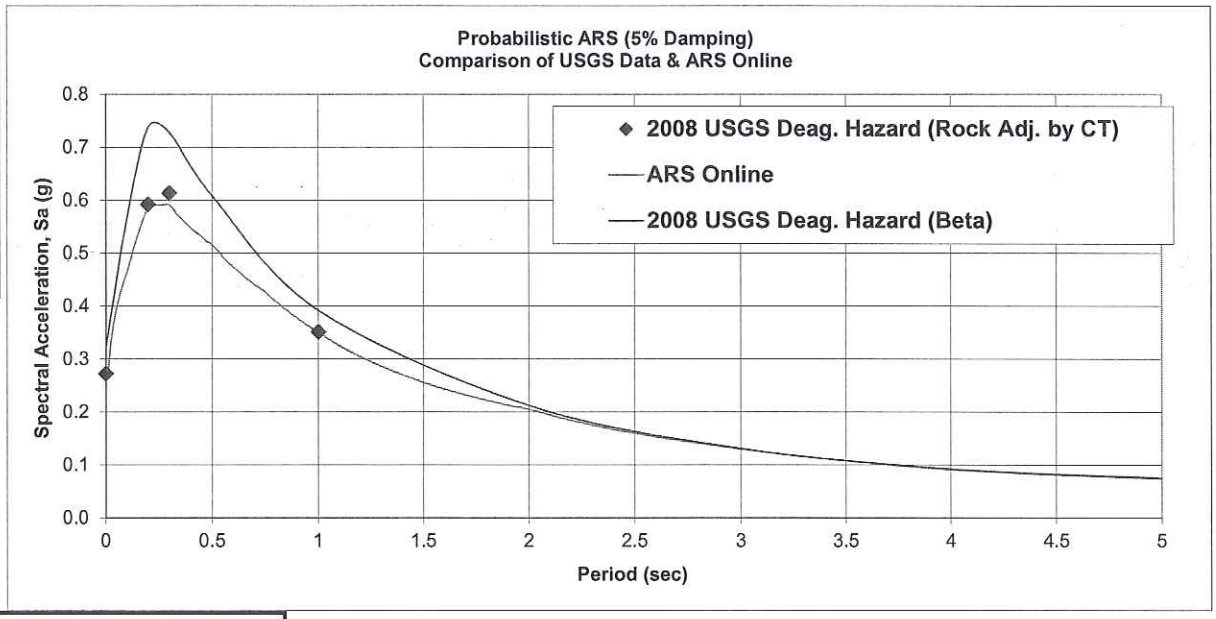
Depth (ft)	Layer Thickness (ft)	qt (tsf)	Soil Type	s'_{vo} (ksf)	Shear Wave Velocity (V_i) (m/sec)	d_i/V_i
0.52	0.52	69.44	5	0.06	436.45	0.00119
0.55	0.03	71.02	5	0.06	442.65	0.00007
0.59	0.04	66.07	5	0.07	423.04	0.00009
0.62	0.03	63.04	4	0.07	410.77	0.00007
0.67	0.05	60.65	11	0.08	77.50	0.00065
0.72	0.05	57.89	11	0.09	79.52	0.00063
0.76	0.04	55.41	3	0.09	378.85	0.00011
0.81	0.05	53.98	3	0.1	372.69	0.00013
0.87	0.06	52.37	3	0.11	365.68	0.00016
0.91	0.04	51.31	3	0.11	361.02	0.00011
0.97	0.06	59.26	11	0.12	86.20	0.00070
1.01	0.04	55.4	3	0.13	378.81	0.00011
1.05	0.04	57.12	3	0.13	386.14	0.00010
1.11	0.06	61.06	11	0.14	90.21	0.00067
1.16	0.05	56.28	4	0.15	382.57	0.00013
1.22	0.06	53.01	4	0.16	368.48	0.00016
1.28	0.06	55.75	4	0.16	380.31	0.00016
1.34	0.06	53.82	4	0.17	372.00	0.00016
1.39	0.05	50.99	4	0.18	359.61	0.00014
1.45	0.06	49.24	4	0.19	351.82	0.00017
1.5	0.05	53.79	5	0.19	371.87	0.00013
1.56	0.06	53.75	5	0.2	371.70	0.00016
1.61	0.05	55.73	5	0.21	380.22	0.00013
1.67	0.06	57.7	6	0.22	388.60	0.00015
1.73	0.06	57.06	6	0.22	385.89	0.00016
1.78	0.05	54.45	6	0.23	374.72	0.00013
1.84	0.06	52.58	6	0.24	366.60	0.00016
1.89	0.05	50.81	6	0.25	358.82	0.00014
1.93	0.04	46.95	6	0.25	341.47	0.00012
1.99	0.06	46.55	6	0.26	339.65	0.00018
2.04	0.05	45.07	6	0.27	332.83	0.00015
2.1	0.06	44.55	6	0.27	330.42	0.00018
2.16	0.06	42.19	6	0.28	319.34	0.00019
2.22	0.06	42.67	5	0.29	321.61	0.00019
2.27	0.05	40.81	5	0.3	312.75	0.00016

Development of Probabilistic and Deterministic Response Spectrums

Comparison spreadsheet of the 2008 USGS Probabilistic Seismic Hazard Data and ARS Online Probabilistic Data (unlock spreadsheet "shmi")
Spectral Accelerations Points from USGS Website at http://earthquake.usgs.gov/research/hazmaps/products_data/2008/data/

*** Note: This spreadsheet uses the given latitude and longitude data provided by the user to estimate spectral acceleration values with a probability of exceedence 5% in 50 yrs (or 975 yr return period). The four spectral acceleration data points plotted on the graph are from the USGS website and are based on a 0.05 degree grid. Basic interpolation is used to estimate intermediate values inside each grid. Raw Data points are provided in the tabs of this spreadsheet. Corner grid spectral acceleration data are shown in the "calculation" tab.**

Input Site Information	
Latitude	Longitude
37.9400	-121.3173
V_{S30} (m/s) =	249
Near Fault Factor, Derived from USGS Deagg. Dist (km) =	29.4
$Z_{1.0}$ (m) =	0
$Z_{2.5}$ (km) =	0



Place ARS Online Probabilistic Data Here "Paste"

T (sec)	Base Spectrum S(a)	Basin Factor	Near Fault Factor	Final Adj. Spectrum S(a)
0.01	0.273	1	1	0.273
0.02	0.321	1	1	0.321
0.022	0.328	1	1	0.328
0.025	0.338	1	1	0.338
0.029	0.349	1	1	0.349
0.03	0.352	1	1	0.352
0.032	0.357	1	1	0.357
0.035	0.365	1	1	0.365
0.036	0.367	1	1	0.367
0.04	0.376	1	1	0.376
0.042	0.381	1	1	0.381
0.044	0.385	1	1	0.385
0.045	0.387	1	1	0.387
0.046	0.389	1	1	0.389
0.048	0.392	1	1	0.392
0.05	0.396	1	1	0.396
0.055	0.405	1	1	0.405
0.06	0.413	1	1	0.413
0.065	0.421	1	1	0.421
0.067	0.424	1	1	0.424
0.07	0.428	1	1	0.428
0.075	0.435	1	1	0.435
0.08	0.442	1	1	0.442
0.085	0.448	1	1	0.448
0.09	0.454	1	1	0.454
0.095	0.46	1	1	0.46
0.1	0.465	1	1	0.465
0.11	0.48	1	1	0.48
0.12	0.495	1	1	0.495
0.13	0.509	1	1	0.509
0.133	0.513	1	1	0.513
0.14	0.522	1	1	0.522
0.15	0.534	1	1	0.534
0.16	0.546	1	1	0.546
0.17	0.558	1	1	0.558
0.18	0.569	1	1	0.569
0.19	0.58	1	1	0.58
0.2	0.59	1	1	0.59
0.22	0.59	1	1	0.59
0.24	0.591	1	1	0.591
0.25	0.591	1	1	0.591

Analysis of ARS Online Results vs USGS Deaggregation Hazard (Adj. By CT)

Period (sec)	USGS Interpolated Spectral Accel.	Adj. for Near Fault Effect	Adj. for Soil Amplification	Adj. For Basin Effect	Final Adj. USGS Spec Accel	ARS Online Final Adj. Spect. Accel.	% Difference (bet. USGS & ARS Online)
0	0.243	1.000	1.122	1.000	0.272	0.273	-0.3%
0.2	0.586	1.000	1.011	1.000	0.592	0.59	0.4%
0.3	0.485	1.000	1.265	1.000	0.614	0.592	3.5%
1	0.180	1.000	1.945	1.000	0.351	0.35	0.3%

Max % Difference = 3.5%

USGS Deaggregation Hazard (Beta) with Near Field and Basin Factors

Period (sec)	INPUT USGS Deagg. Spec Accel	Adj. for Near Fault Effect	Adj. For Basin Effect	Final Adj. USGS Deagg Spec Accel	ARS Online Final Adj. Spect. Accel.	% Difference (bet. USGS & ARS Online)
0	0.3244	1.000	1.000	0.324	0.273	[illegible]
0.1	0.5696	1.000	1.000	0.570	0.465	[illegible]
0.2	0.7382	1.000	1.000	0.738	0.59	[illegible]
0.3	0.7281	1.000	1.000	0.728	0.592	[illegible]
0.5	0.6083	1.000	1.000	0.608	0.516	15.2%
1	0.3919	1.000	1.000	0.392	0.35	10.7%
2	0.2124	1.000	1.000	0.212	0.205	3.5%
3	0.13101	1.000	1.000	0.131	0.13	0.8%
4	0.09244	1.000	1.000	0.092	0.091	1.6%
5	0.07534	1.000	1.000	0.075	0.074	1.8%

Max % Difference = 15.2%

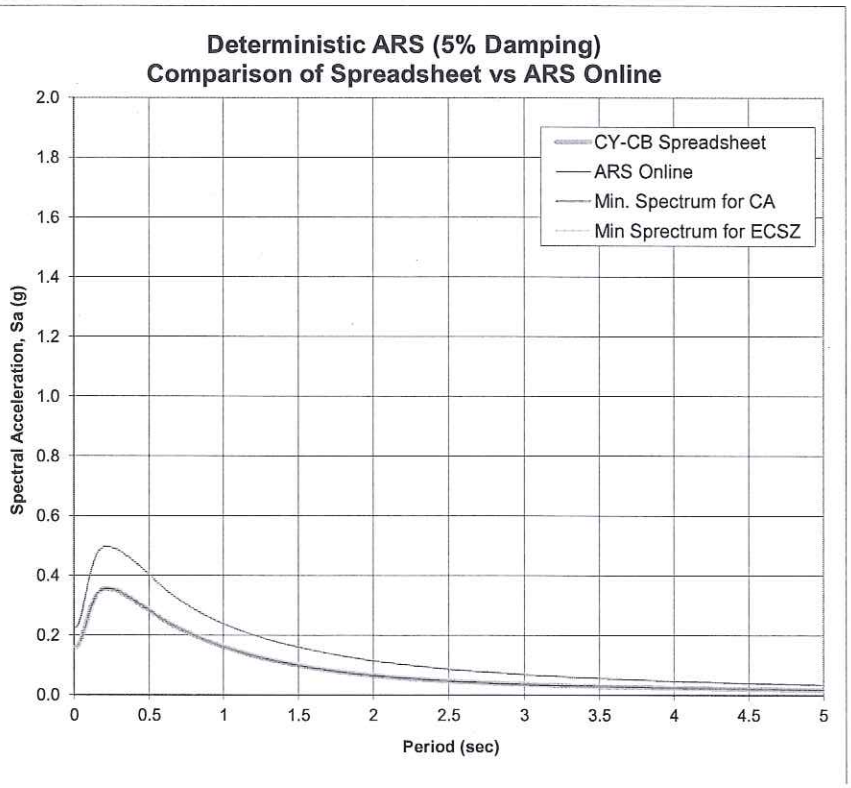
Comparison of ARS Curves
(unlock sheet with "shmi")

Model Inputs

Fault		
Magnitude	6.7	(5 to 8.5)
F_{RV}	1	(input 1 = Rev)
F_{NM}	0	(input 1 = Normal)
Dip (degree)	15	(0 to 90)
Z_{TOR} (km)	7	
Distance		
R_{RUP} (km)	30.1	
R_{JB} (km)	29.3	
R_x (km)	29.1	
Hanging Wall?	☐ Yes?	
Near-Field Factor?	☐ Yes?	
Site		
V_{S30} (m/sec)	249	(270 to 1500 m/s)
$Z_{1.0}$ (m)	0	(0 - No Basin)
$Z_{2.5}$ (km)	0	(0 - No Basin)
No. Cal. Basin?	☐ Yes ?	(Check only for sites located within a Basin)
So. Cal. Basin?	☐ Yes ?	

Analysis

ARS Online vs CY-CB Spreadsheet Results
MAX. % Diff. = 2%



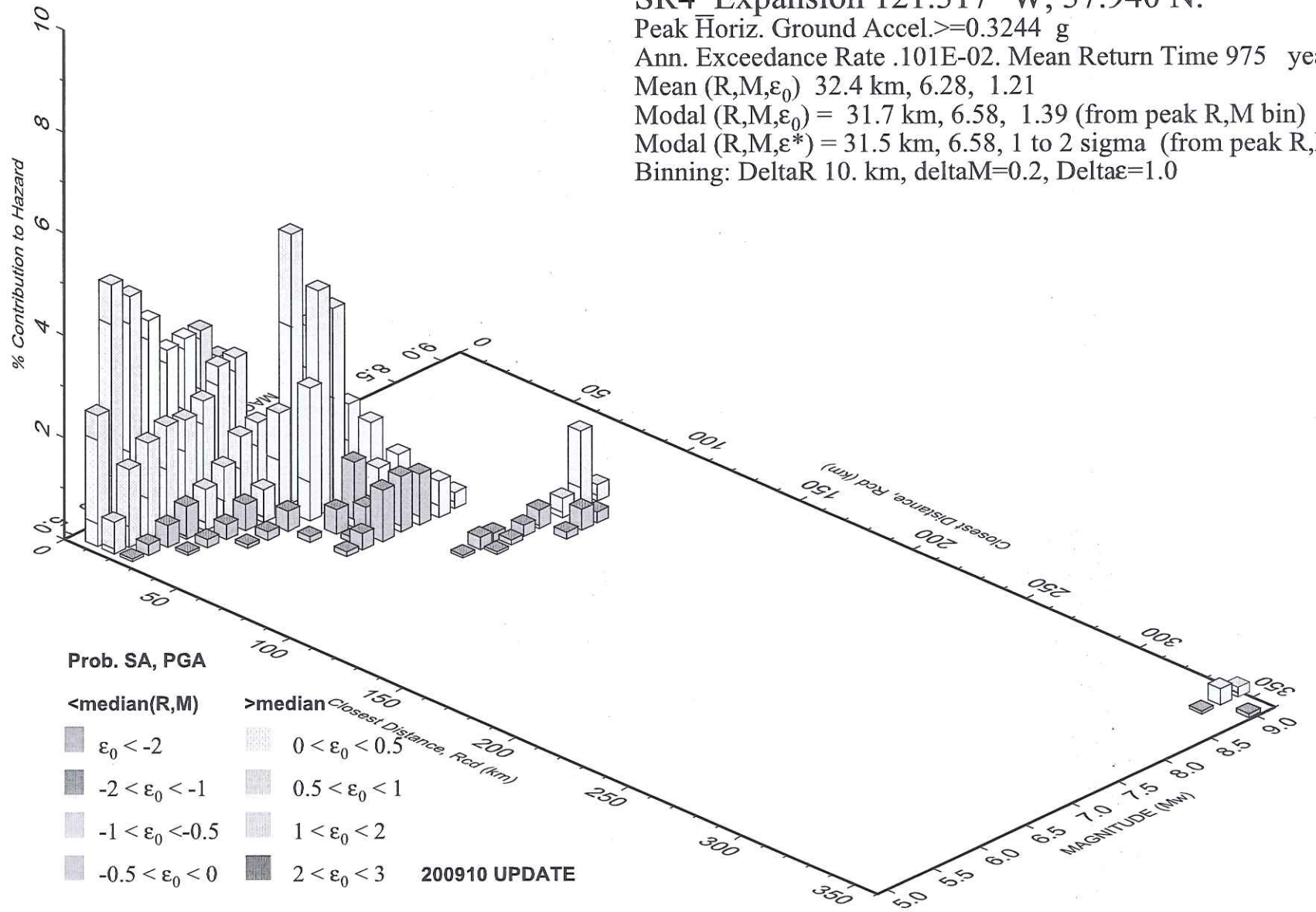
CY-CB Spreadsheet Results	
T (sec)	CB-CY S(a)
0.010	0.15919
0.020	0.16067
0.022	0.16203
0.025	0.16389
0.029	0.16616
0.030	0.16684
0.032	0.16895
0.035	0.17199
0.036	0.17301
0.040	0.17705
0.042	0.17922
0.044	0.18144
0.045	0.18261
0.046	0.18374
0.048	0.18606
0.050	0.18834
0.055	0.19643
0.060	0.20462
0.065	0.21285
0.067	0.21615
0.070	0.22109
0.075	0.22927
0.080	0.23822
0.085	0.24701
0.090	0.25574
0.095	0.26432
0.100	0.27266
0.110	0.28741
0.120	0.30105
0.130	0.31338
0.133	0.31676
0.140	0.32424
0.150	0.33401
0.160	0.33974
0.170	0.34455
0.180	0.34862

Place ARS Online Deterministic Data Here "Paste"					
T (sec)	Base S(a)	Basin Factor	Near Fault Factor	Final Adj. S(a)	Diff. (%)
0.01	0.159	1	1	0.159	0%
0.02	0.161	1	1	0.161	0%
0.022	0.162	1	1	0.162	0%
0.025	0.164	1	1	0.164	0%
0.029	0.166	1	1	0.166	0%
0.03	0.167	1	1	0.167	0%
0.032	0.169	1	1	0.169	0%
0.035	0.172	1	1	0.172	0%
0.036	0.173	1	1	0.173	0%
0.04	0.177	1	1	0.177	0%
0.042	0.179	1	1	0.179	0%
0.044	0.181	1	1	0.181	0%
0.045	0.183	1	1	0.183	0%
0.046	0.184	1	1	0.184	0%
0.048	0.186	1	1	0.186	0%
0.05	0.188	1	1	0.188	0%
0.055	0.196	1	1	0.196	0%
0.06	0.205	1	1	0.205	0%
0.065	0.213	1	1	0.213	0%
0.067	0.216	1	1	0.216	0%
0.07	0.221	1	1	0.221	0%
0.075	0.229	1	1	0.229	0%
0.08	0.238	1	1	0.238	0%
0.085	0.247	1	1	0.247	0%
0.09	0.256	1	1	0.256	0%
0.095	0.264	1	1	0.264	0%
0.1	0.273	1	1	0.273	0%
0.11	0.287	1	1	0.287	0%
0.12	0.301	1	1	0.301	0%
0.13	0.313	1	1	0.313	0%
0.133	0.317	1	1	0.317	0%
0.14	0.324	1	1	0.324	0%
0.15	0.334	1	1	0.334	0%
0.16	0.34	1	1	0.34	0%
0.17	0.345	1	1	0.345	0%
0.18	0.349	1	1	0.349	0%

For Comparsion Plots of Min. Sprectra, Paste Special into Cells			
Min. Spectrum for CA		Min Sprectrum for ECSZ	
T (sec)	S (a)	T (sec)	S (a)
0.01	0.225		
0.02	0.228		
0.022	0.23		
0.025	0.233		
0.029	0.237		
0.03	0.238		
0.032	0.241		
0.035	0.246		
0.036	0.248		
0.04	0.254		
0.042	0.257		
0.044	0.26		
0.045	0.262		
0.046	0.264		
0.048	0.267		
0.05	0.271		
0.055	0.283		
0.06	0.295		
0.065	0.307		
0.067	0.311		
0.07	0.319		
0.075	0.331		
0.08	0.343		
0.085	0.355		
0.09	0.368		
0.095	0.379		
0.1	0.391		
0.11	0.41		
0.12	0.428		
0.13	0.444		
0.133	0.448		
0.14	0.458		
0.15	0.471		
0.16	0.478		
0.17	0.484		
0.18	0.489		

PSH Deaggregation on NEHRP D soil
SR4_Expansion 121.317° W, 37.940 N.

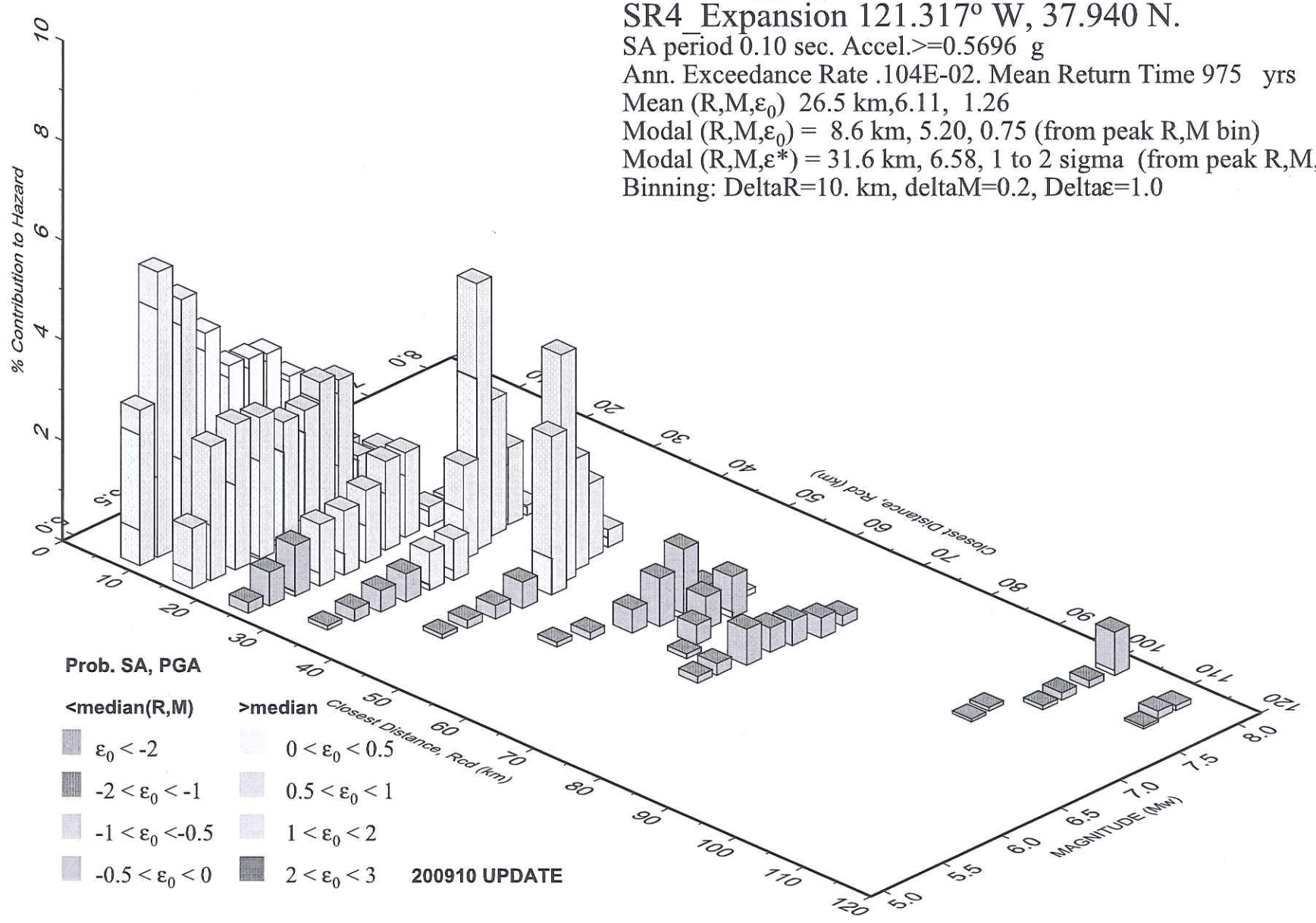
Peak Horiz. Ground Accel.>=0.3244 g
Ann. Exceedance Rate .101E-02. Mean Return Time 975 years
Mean (R,M,ε_0) 32.4 km, 6.28, 1.21
Modal (R,M,ε_0) = 31.7 km, 6.58, 1.39 (from peak R,M bin)
Modal (R,M,ε^*) = 31.5 km, 6.58, 1 to 2 sigma (from peak R,M,ε bin)
Binning: DeltaR 10. km, deltaM=0.2, Deltaε=1.0



GMT 2011 Apr 29 20:35:25 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 249. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

PSH Deaggregation on NEHRP D soil
SR4_Expansion 121.317° W, 37.940 N.

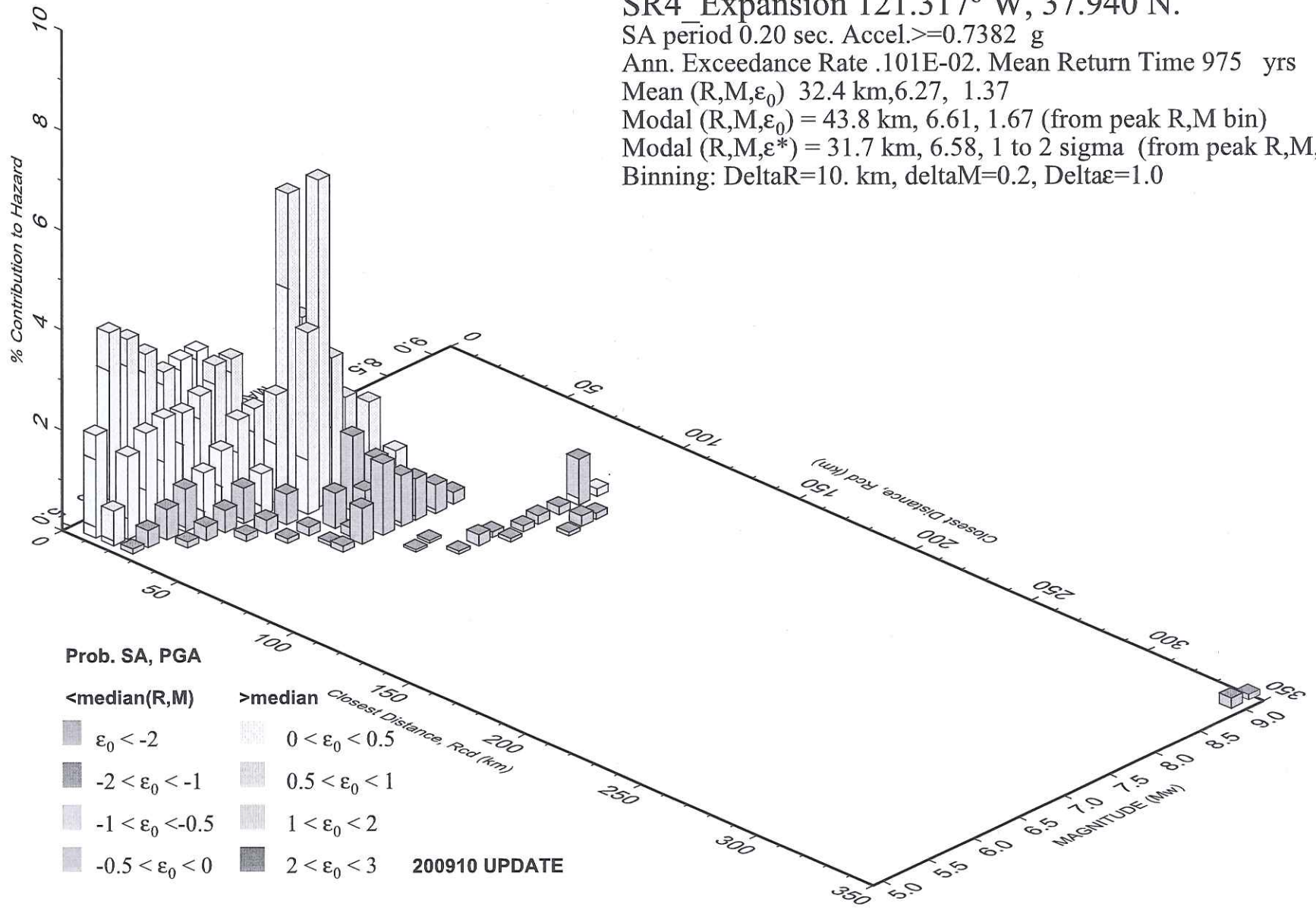
SA period 0.10 sec. Accel.>=0.5696 g
Ann. Exceedance Rate .104E-02. Mean Return Time 975 yrs
Mean (R,M,ε_0) 26.5 km,6.11, 1.26
Modal (R,M,ε_0) = 8.6 km, 5.20, 0.75 (from peak R,M bin)
Modal (R,M,ε^*) = 31.6 km, 6.58, 1 to 2 sigma (from peak R,M,ε bin)
Binning: DeltaR=10. km, deltaM=0.2, Deltaε=1.0



GMT 2011 Apr 29 20:37:27 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 249. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

PSH Deaggregation on NEHRP D soil
SR4_Expansion 121.317° W, 37.940 N.

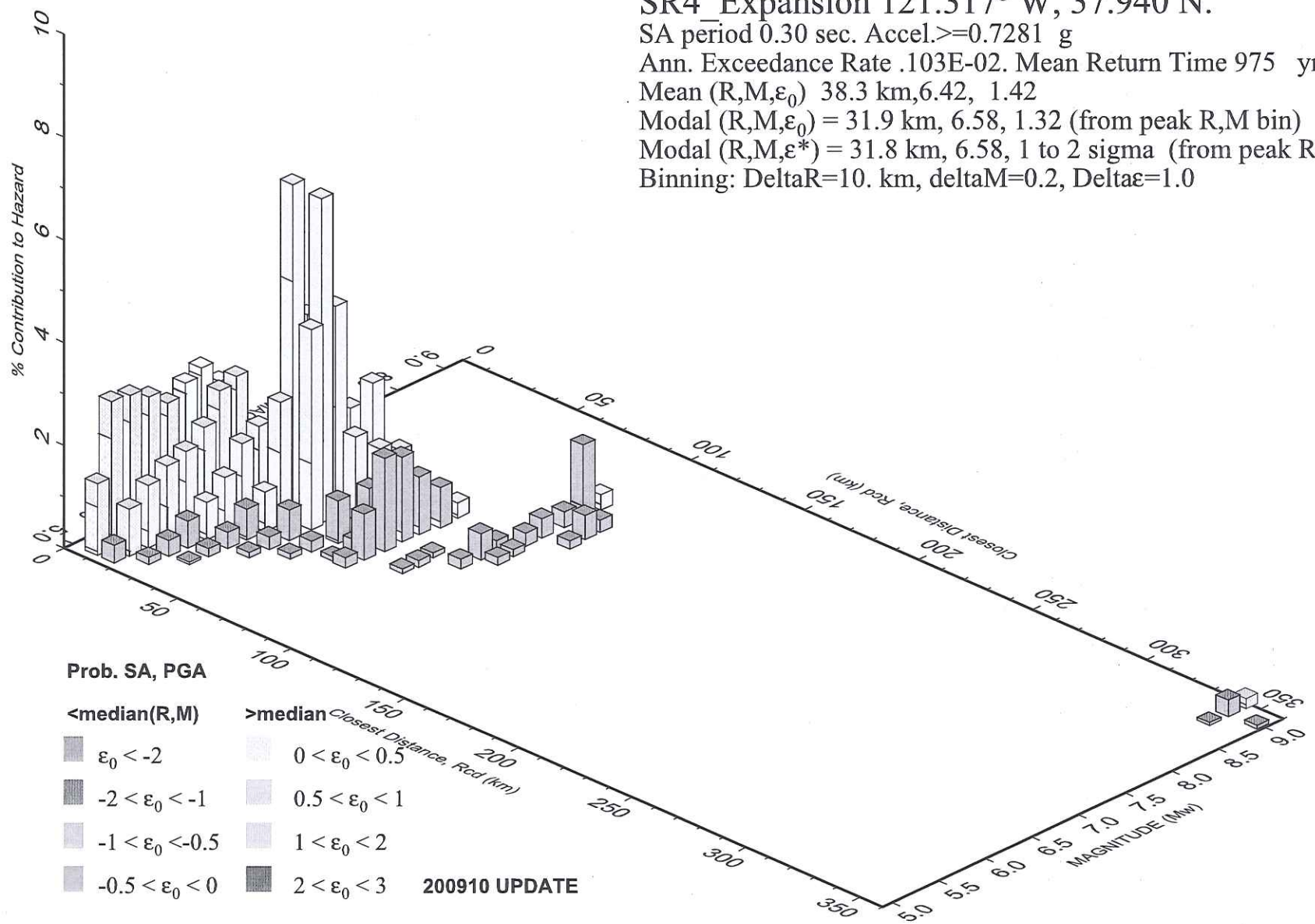
SA period 0.20 sec. Accel.>=0.7382 g
Ann. Exceedance Rate .101E-02. Mean Return Time 975 yrs
Mean (R,M,ε_0) 32.4 km,6.27, 1.37
Modal (R,M,ε_0) = 43.8 km, 6.61, 1.67 (from peak R,M bin)
Modal (R,M,ε^*) = 31.7 km, 6.58, 1 to 2 sigma (from peak R,M,ε bin)
Binning: DeltaR=10. km, deltaM=0.2, Deltaε=1.0



GMT 2011 Apr 29 20:38:07 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 249. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

PSH Deaggregation on NEHRP D soil
SR4_Expansion 121.317° W, 37.940 N.

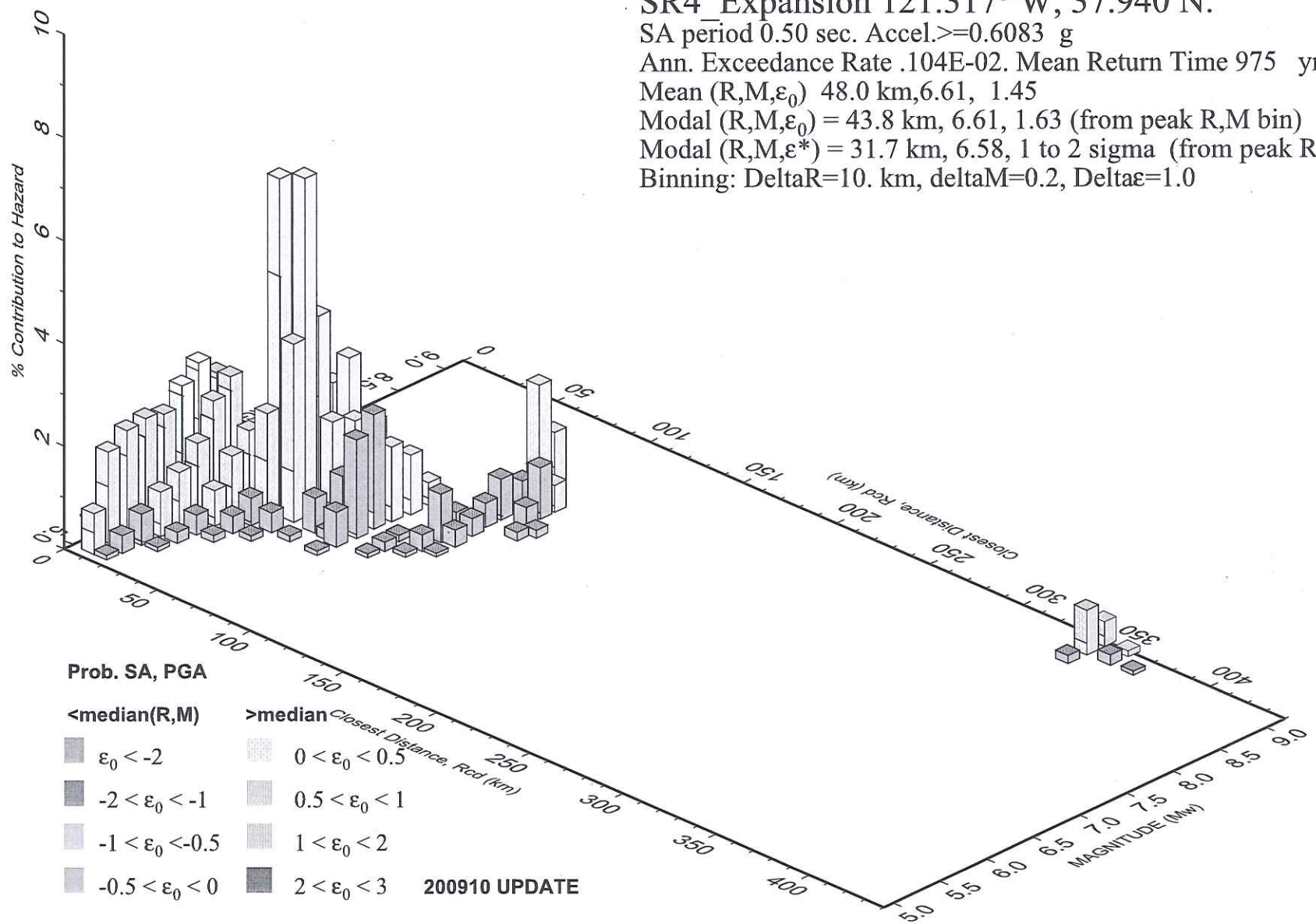
SA period 0.30 sec. Accel.>=0.7281 g
Ann. Exceedance Rate .103E-02. Mean Return Time 975 yrs
Mean (R,M,ε_0) 38.3 km,6.42, 1.42
Modal (R,M,ε_0) = 31.9 km, 6.58, 1.32 (from peak R,M bin)
Modal (R,M,ε^*) = 31.8 km, 6.58, 1 to 2 sigma (from peak R,M,ε bin)
Binning: DeltaR=10. km, deltaM=0.2, Deltaε=1.0



GMT 2011 Apr 29 20:38:47 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 249. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

PSH Deaggregation on NEHRP D soil
SR4_Expansion 121.317° W, 37.940 N.

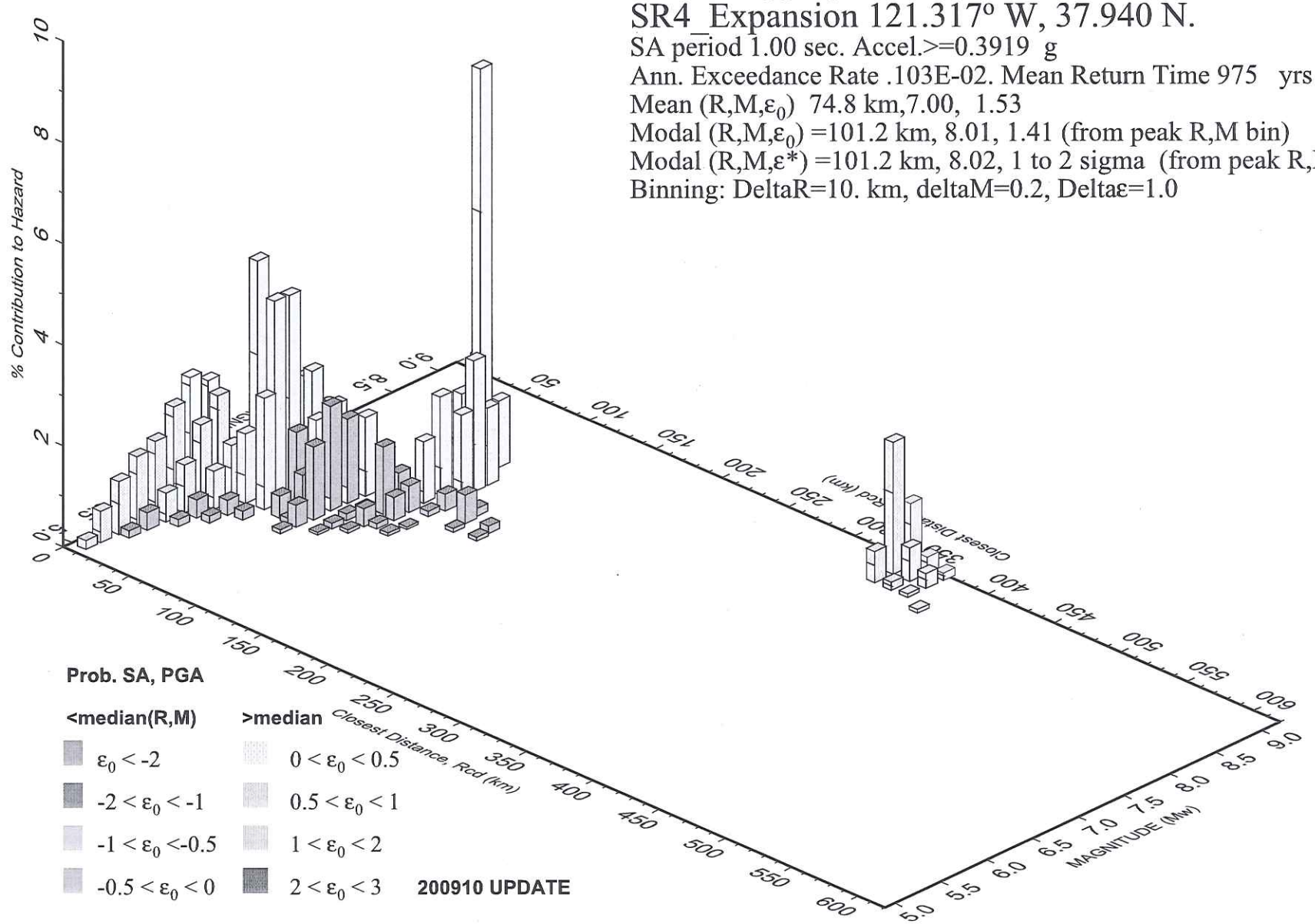
SA period 0.50 sec. Accel.>=0.6083 g
Ann. Exceedance Rate .104E-02. Mean Return Time 975 yrs
Mean (R,M,ε_0) 48.0 km,6.61, 1.45
Modal (R,M,ε_0) = 43.8 km, 6.61, 1.63 (from peak R,M bin)
Modal (R,M,ε^*) = 31.7 km, 6.58, 1 to 2 sigma (from peak R,M,ε bin)
Binning: DeltaR=10. km, deltaM=0.2, Deltaε=1.0



GMT 2011 Apr 29 20:39:19 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 249. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

PSH Deaggregation on NEHRP D soil
SR4_Expansion 121.317° W, 37.940 N.

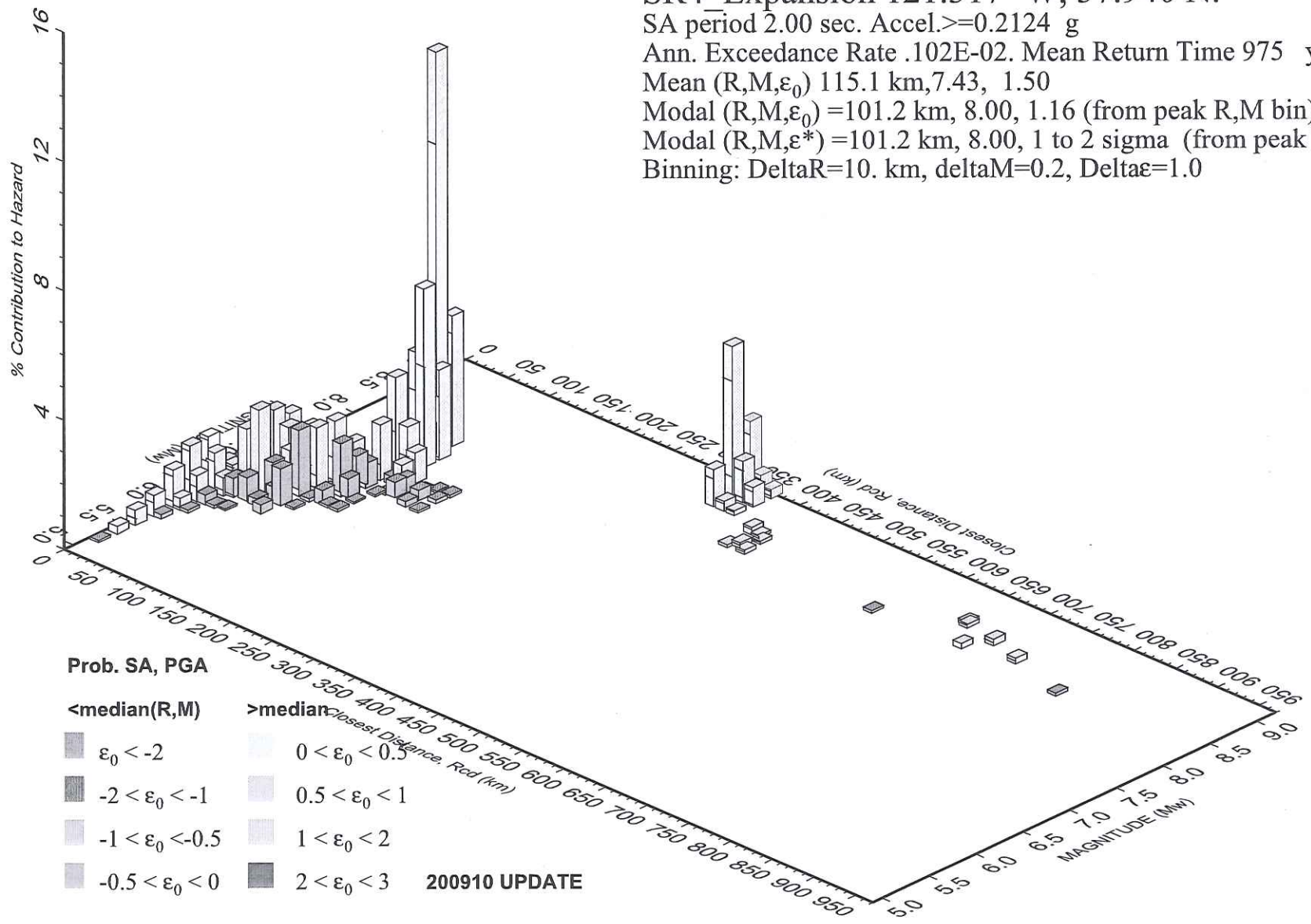
SA period 1.00 sec. Accel.>=0.3919 g
Ann. Exceedance Rate .103E-02. Mean Return Time 975 yrs
Mean (R,M,ε_0) 74.8 km,7.00, 1.53
Modal (R,M,ε_0) =101.2 km, 8.01, 1.41 (from peak R,M bin)
Modal (R,M,ε^*) =101.2 km, 8.02, 1 to 2 sigma (from peak R,M,ε bin)
Binning: DeltaR=10. km, deltaM=0.2, Deltaε=1.0



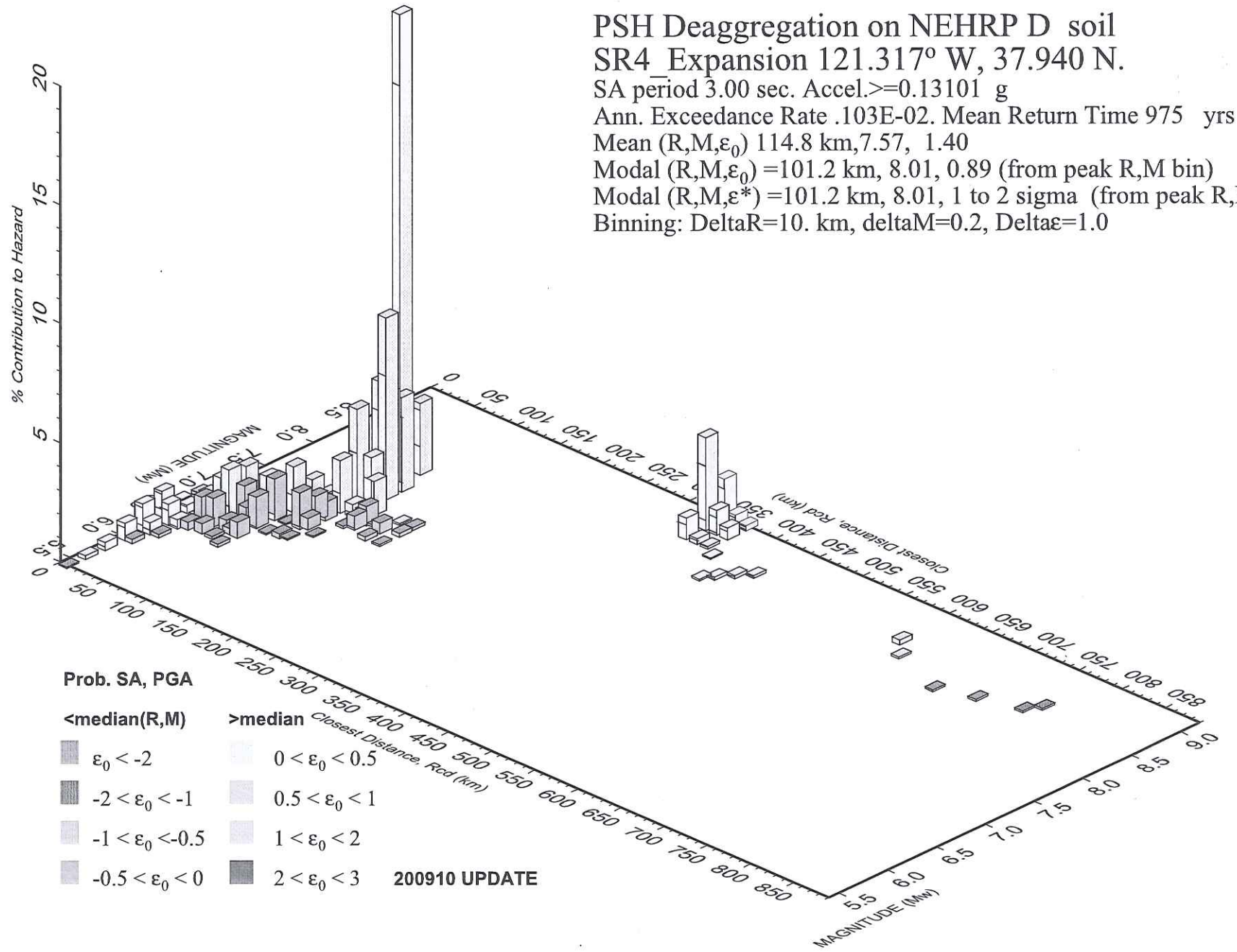
GMT 2011 Apr 29 20:39:52 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 249. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

PSH Deaggregation on NEHRP D soil
SR4_Expansion 121.317° W, 37.940 N.

SA period 2.00 sec. Accel.>=0.2124 g
Ann. Exceedance Rate .102E-02. Mean Return Time 975 yrs
Mean (R,M,ε_0) 115.1 km,7.43, 1.50
Modal (R,M,ε_0) =101.2 km, 8.00, 1.16 (from peak R,M bin)
Modal (R,M,ε^*) =101.2 km, 8.00, 1 to 2 sigma (from peak R,M,ε bin)
Binning: DeltaR=10. km, deltaM=0.2, Deltaε=1.0



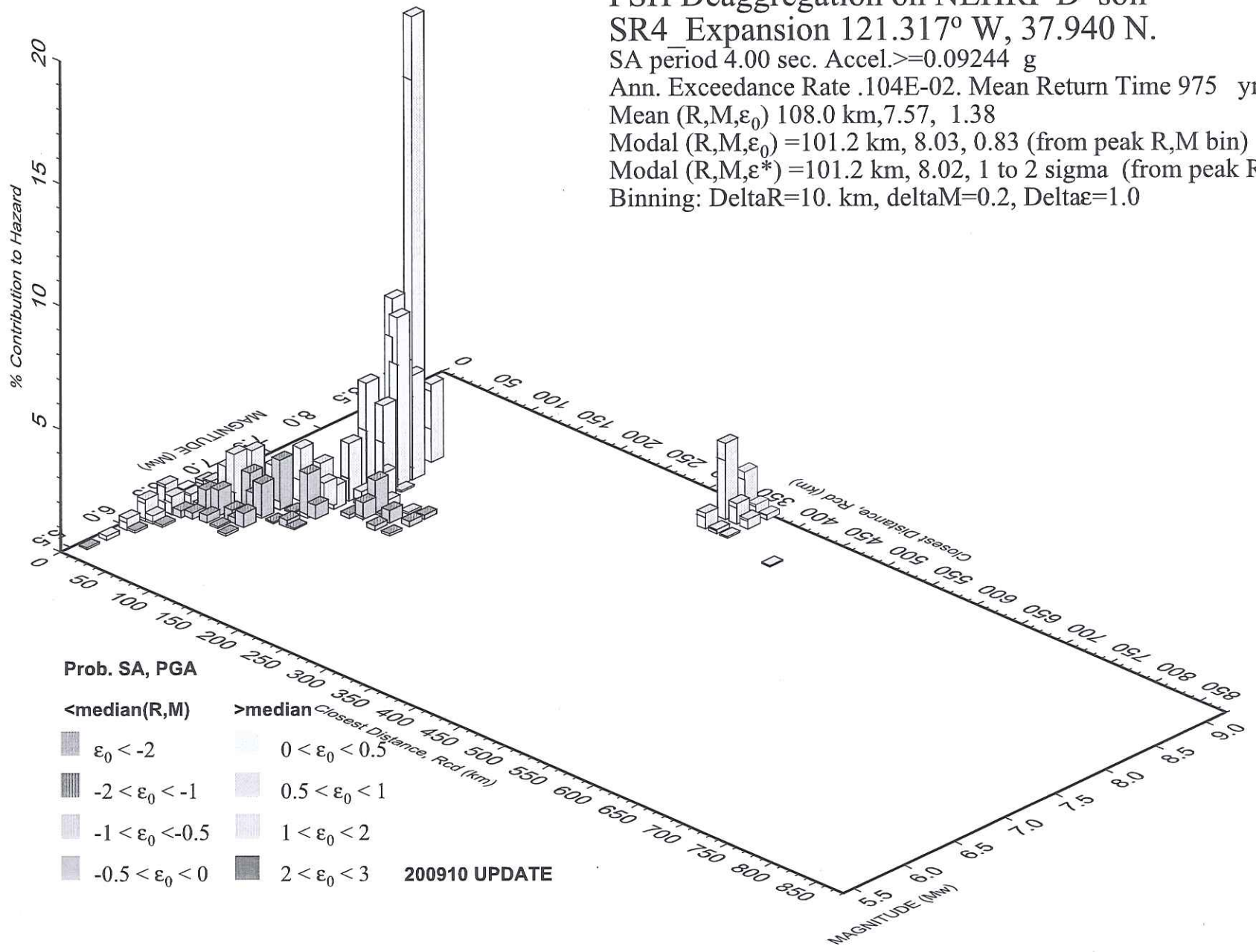
GMT 2011 Apr 29 20:40:28 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 249. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted



GMT 2011 Apr 29 20:40:57 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 249. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

PSH Deaggregation on NEHRP D soil
SR4_Expansion 121.317° W, 37.940 N.

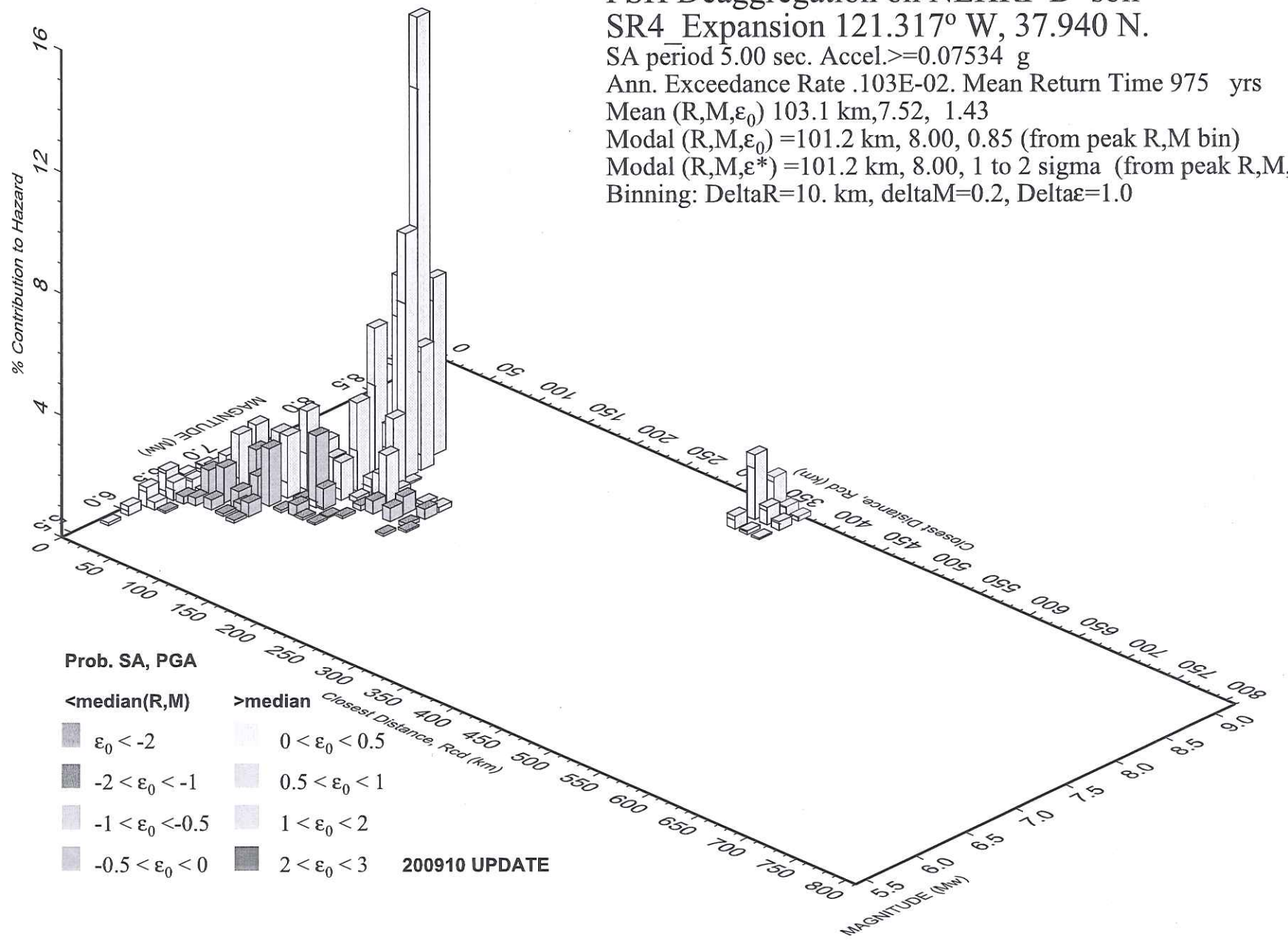
SA period 4.00 sec. Accel.>=0.09244 g
Ann. Exceedance Rate .104E-02. Mean Return Time 975 yrs
Mean (R,M,ε_0) 108.0 km,7.57, 1.38
Modal (R,M,ε_0) =101.2 km, 8.03, 0.83 (from peak R,M bin)
Modal (R,M,ε*) =101.2 km, 8.02, 1 to 2 sigma (from peak R,M,ε bin)
Binning: DeltaR=10. km, deltaM=0.2, Deltaε=1.0



GMT 2011 Apr 29 20:41:26 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 249. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

PSH Deaggregation on NEHRP D soil
SR4_Expansion 121.317° W, 37.940 N.

SA period 5.00 sec. Accel.>=0.07534 g
Ann. Exceedance Rate .103E-02. Mean Return Time 975 yrs
Mean (R,M,ε_0) 103.1 km,7.52, 1.43
Modal (R,M,ε_0) =101.2 km, 8.00, 0.85 (from peak R,M bin)
Modal (R,M,ε*) =101.2 km, 8.00, 1 to 2 sigma (from peak R,M,ε bin)
Binning: DeltaR=10. km, deltaM=0.2, Deltaε=1.0



APPENDIX F
LIQUEFACTION ANALYSIS RESULTS

LIST OF ATTACHMENTS

The following are attached and complete this appendix.

Example of Input Data for Liquefaction Assessment Spreadsheet
Example of Output Data from Liquefaction Assessment Spreadsheet
Liquefaction Assessment Results/Graphs

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Example of Input Data for Liquefaction Assessment Spreadsheet

STEP 1:

Project Name: Crosstown Freeway
Project No.: 111904.G01
Project Location: Stockton, CA

Analyzed by: Hamid Parsa Date: 4/28/2011
Checked by: Carl henderson Date:

STEP 2:

Mw= 6.7 5.5≤Mw≤8.5
PGA= 0.32

Field Ground Water Depth (ft)= 11.3
Design Ground Water Depth (ft)= 11

Existing Ground Surface Elevation (ft)= 0
New Fill Height (ft)= 0
Unit Weight of New Fill (pcf)= 120

STEP 3:

Probability to Liquefy (PL) (%)= 15 for Seed et al. (2003) approach

Factor of Safety (FOS) for Liquefaction = 1.00
Factor of Safety (FOS) for Settlement Calculation = 1.30

STEP 4:

CPT I.D.: CPT-1

Depth ft	Depth (m)	q_c T/ft^2	f_s T/ft^2	u_2 lb/in^2	R_f %	q_t T/ft^2	σ_{vo} T/ft^2	SBT	S_u(qc) T/ft^2	SPT N_{60} blows/ft	UW lb/ft^3
0.57		157.14	4.3		2.7			7			135
0.64		153.52	4.97		3.2			6			135
0.71		153.86	4.76		3.1			7			135
0.79		133.07	4.62		3.5			6			135
0.87		117.91	4.34		3.7			6			135
0.94		103.46	4.51		4.4			11			135
1.02		114.58	4.63		4			11			135
1.1		98.93	4.37		4.4			11			135
1.18		86.15	3.72		4.3			11			135
1.27		74.8	3.86		5.2			11			135
1.36		58.67	4.42		7.5			11			135
1.43		90.12	4.84		5.4			11			135
1.52		149.54	5.62		3.8			12			140
1.59		155.54	6.01		3.9			12			140
1.68		158.17	4.8		3			7			135
1.75		146.99	4.38		3			7			135
1.84		147.59	4.13		2.8			7			135
1.91		145	3.77		2.6			7			135
1.97		137.15	3.71		2.7			7			135
2.05		117.48	3.72		3.2			6			135
2.13		96.67	3.78		3.9			5			135
2.21		69.74	3.56		5.1			11			135
2.3		51.86	3.28		6.3			3			135
2.38		45.38	3.03		6.7			3			135
2.46		38.34	2.74		7.1			3			135
2.54		34.06	2.46		7.2			3			135
2.62		29.1	2.27		7.8			3			135
2.7		25.22	2.08		8.2			3			135
2.78		25.37	1.97		7.8			3			135
2.87		24.23	1.83		7.6			3			135
2.95		21.75	1.69		7.8			3			135
3.03		20.93	1.59		7.6			3			135
3.11		21.1	1.62		7.7			3			135
3.2		21.89	1.6		7.3			3			135
3.28		24.55	1.54		6.3			3			135
3.37		25.04	1.54		6.2			3			135
3.45		19.96	1.51		7.6			3			135
3.53		17.38	1.41		8.1			3			135
3.61		15.93	1.28		8.1			3			135
3.7		17.55	1.21		6.9			3			135
3.78		15.93	1.18		7.4			3			135
3.87		14.8	1.16		7.9			3			135
3.95		15.93	1.16		7.3			3			135
4.04		15.86	1.15		7.3			3			135
4.12		18.84	1.13		6			3			135
4.21		18.04	1.14		6.3			3			135
4.3		18.59	1.14		6.1			3			135
4.39		18.44	1.12		6.1			3			135
4.47		16.83	1.12		6.7			3			135
4.56		15.61	1.12		7.2			3			135
4.64		14.64	1.1		7.5			3			135
4.74		14.82	1.08		7.3			3			125
4.82		16.19	1.08		6.7			3			135
4.91		14.65	1.04		7.1			3			125
5		14.65	0.95		6.5			3			125
5.09		13.93	0.94		6.8			3			125
5.17		15.22	0.96		6.3			3			125
5.33		15.95	0.96		6			3			125
5.42		13.68	0.93		6.8			3			125
5.5		14.72	0.92		6.3			3			125
5.6		13.93	0.99		7.1			3			125
5.68		16.5	0.96		5.8			3			125
5.77		14.48	0.9		6.2			3			125
5.85		10.86	0.87		8			3			125
5.94		10.78	0.86		8			3			125
6.03		10.21	0.79		7.7			3			125
6.12		9.73	0.78		8			3			125

6.2	9.65	0.77	8	3	125
6.29	8.28	0.73	8.8	3	125
6.38	7.95	0.73	9.2	3	125
6.47	9.89	0.78	7.9	3	125
6.55	11.5	0.77	6.7	3	125
6.65	12.71	0.82	6.4	3	125
6.73	20.7	0.86	4.2	4	135
6.82	22.53	0.87	3.9	4	135
6.9	24.41	0.88	3.6	5	135
6.99	21.74	0.86	4	4	135
7.08	20.21	0.87	4.3	4	135
7.17	18.04	0.85	4.7	3	125
7.25	14.56	0.83	5.7	3	125
7.35	12.64	0.8	6.4	3	125
7.43	13.44	0.76	5.7	3	125
7.52	13.76	0.68	4.9	3	125
7.6	12.55	0.62	5	3	125
7.69	9.89	0.6	6.1	3	125
7.78	9.42	0.57	6	3	125
7.86	8.84	0.51	5.8	3	115
7.96	9.9	0.54	5.5	3	125
8.04	11.35	0.67	5.9	3	125
8.13	19.33	0.72	3.7	4	125
8.22	14.81	0.73	4.9	3	125
8.29	10.68	0.73	6.9	3	125
8.33	11.1	0.71	6.4	3	125
8.41	10.62	0.63	5.9	3	125
8.49	13.2	0.53	4	3	125
8.58	14.32	0.5	3.5	4	125
8.66	15.21	0.56	3.7	4	125
8.76	16.43	0.61	3.7	4	125
8.84	12.95	0.57	4.4	3	125
8.93	9.32	0.51	5.5	3	115
9.02	7.31	0.46	6.3	3	115
9.11	5.86	0.41	6.9	3	115
9.19	5.21	0.36	6.9	3	105
9.27	5.13	0.4	7.7	3	105
9.37	6.26	0.43	6.8	3	115
9.45	7.55	0.47	6.2	3	115
9.55	10.86	0.5	4.6	3	125
9.64	12.39	0.59	4.7	3	125
9.72	12.06	0.77	6.4	3	125
9.82	20.39	1.03	5	3	135
9.9	30.99	1.21	3.9	5	135
9.99	35.44	1.45	4.1	4	135
10.07	43.91	1.69	3.8	5	135
10.15	53.74	1.86	3.5	5	135
10.24	55.98	2.04	3.7	5	135
10.33	51.09	2.25	4.4	4	135
10.42	47.71	2.44	5.1	3	135
10.5	46.4	2.61	5.6	3	135
10.6	50.27	2.89	5.7	3	135
10.68	61.6	3.24	5.2	11	135
10.77	78.38	3.65	4.6	11	135
10.85	78.8	4.06	5.1	11	135
10.93	76.9	4.37	5.7	11	135
11	82.96	4.48	5.4	11	135
11.09	82.93	4.53	5.5	11	135
11.16	77.31	4.5	5.8	11	135
11.26	69.58	4.28	6.1	11	135
11.35	64.48	3.64	5.6	11	135
11.43	59.39	3.44	5.8	11	135
11.52	50.11	3.24	6.5	3	135
11.58	48.55	3.05	6.3	3	135
11.63	47.85	2.96	6.2	3	135
11.71	43.91	2.99	6.8	3	135
11.8	48.13	3.12	6.5	3	135
11.88	57.13	3.28	5.7	3	135
11.96	60.19	3.41	5.7	11	135
12.05	62.13	3.46	5.6	11	135
12.13	57.17	3.29	5.7	3	135
12.21	47.09	2.89	6.1	3	135
12.3	38.98	2.54	6.5	3	135
12.38	32.45	2.16	6.7	3	135
12.46	27.23	1.85	6.8	3	135
12.55	24.97	1.68	6.7	3	135
12.63	28.01	1.58	5.6	3	135
12.72	32.45	1.56	4.8	3	135
12.8	31.16	1.62	5.2	3	135
12.88	26.49	1.61	6.1	3	135
12.96	20.77	1.44	6.9	3	135
13.05	16.33	1.11	6.8	3	135
13.14	13.75	0.83	6	3	125
13.22	12.87	0.64	5	3	125

13.3	11.9	0.54	4.5	3	125
13.38	12.48	0.51	4.1	3	125
13.47	12.06	0.53	4.4	3	125
13.56	13.43	0.6	4.4	3	125
13.64	15.44	0.79	5.1	3	125
13.73	26.57	1.04	3.9	4	135
13.81	33.74	1.17	3.5	5	135
13.9	29.01	1.16	4	4	135
13.98	23.35	1.08	4.6	3	135
14.07	19	0.91	4.8	3	135
14.17	16.17	0.69	4.3	3	125
14.25	14.4	0.58	4	3	125
14.34	11.66	0.52	4.5	3	125
14.42	9.24	0.5	5.3	3	115
14.51	8.59	0.48	5.5	3	115
14.61	9.32	0.48	5.1	3	115
14.69	10.3	0.48	4.6	3	115
14.78	10.78	0.42	3.9	3	115
14.87	9.58	0.4	4.1	3	115
14.97	10.86	0.39	3.6	3	115
15.06	10.85	0.37	3.4	4	115
15.15	11.58	0.34	2.9	4	115
15.24	12.79	0.36	2.8	5	115
15.33	12.31	0.36	2.9	4	115
15.42	10.94	0.38	3.5	4	115
15.51	10.05	0.4	4	3	115
15.59	9.88	0.4	4	3	115
15.68	10.86	0.34	3.1	4	115
15.78	9.65	0.31	3.2	4	115
15.87	7.8	0.28	3.5	3	105
15.96	6.18	0.24	3.8	3	105
16.05	5.29	0.18	3.4	3	95
16.13	4.33	0.16	3.5	3	95
16.23	4.33	0.14	3.2	3	95
16.31	4.25	0.13	3	3	95
16.41	4.25	0.13	2.9	3	95
16.5	4.65	0.13	2.7	3	95
16.58	4.81	0.13	2.6	3	95
16.67	4.65	0.14	2.9	3	95
16.76	5.05	0.15	2.9	3	95
16.85	5.3	0.15	2.9	3	95
16.94	5.54	0.15	2.7	3	95
17.02	5.7	0.15	2.6	3	95
17.11	6.1	0.16	2.5	4	95
17.2	6.26	0.16	2.4	4	95
17.29	6.5	0.15	2.3	4	95
17.38	7.07	0.16	2.2	4	95
17.47	7.15	0.17	2.4	4	105
17.56	7.31	0.18	2.5	4	105
17.65	7.31	0.19	2.6	4	105
17.73	7.79	0.2	2.5	4	105
17.82	8.36	0.26	3.1	4	105
17.91	9.09	0.29	3.1	4	115
18	10.78	0.31	2.9	4	115
18.08	10.21	0.33	3.2	4	115
18.12	11.9	0.32	2.7	5	115
18.2	10.86	0.31	2.8	4	115
18.3	10.87	0.3	2.7	4	115
18.38	10.37	0.3	2.9	4	115
18.47	11.43	0.31	2.7	5	115
18.56	12.24	0.32	2.6	5	115
18.64	12.54	0.35	2.8	5	115
18.74	11.98	0.38	3.1	4	115
18.82	11.67	0.38	3.2	4	115
18.91	11.18	0.35	3.1	4	115
19	11.09	0.34	3.1	4	115
19.08	12.47	0.37	2.9	4	115
19.17	13.99	0.44	3.1	4	125
19.27	17.95	0.52	2.9	5	125
19.36	20.54	0.68	3.3	5	125
19.44	20.12	0.82	4	4	125
19.54	24.91	0.85	3.4	5	135
19.63	26.5	0.86	3.2	5	135
19.72	26.33	0.84	3.2	5	135
19.81	24.8	0.81	3.3	5	135
19.89	23.28	0.78	3.3	5	135
19.98	20.04	0.7	3.4	5	125
20.08	20.14	0.64	3.1	5	125
20.16	18.51	0.63	3.3	5	125
20.26	19.5	0.65	3.3	5	125
20.35	19.87	0.65	3.2	5	125
20.43	22.3	0.69	3.1	5	125
20.52	24.57	0.78	3.2	5	135
20.61	29.32	0.94	3.2	5	135

20.7	37.64	1.06	2.8	6	135
20.78	35.99	1.1	3	5	135
20.87	33.2	1.14	3.4	5	135
20.96	29.81	1.09	3.6	5	135
21.05	27.71	0.94	3.3	5	135
21.14	26.92	0.84	3.1	5	135
21.23	26.57	0.82	3	5	135
21.31	26.66	0.8	3	5	135
21.4	25.93	0.78	3	5	135
21.49	26.25	0.74	2.8	5	135
21.56	24	0.69	2.8	5	125
21.65	22.71	0.63	2.7	5	125
21.74	20.53	0.59	2.8	5	125
21.82	20.63	0.6	2.8	5	125
21.92	22.05	0.63	2.8	5	125
22	23.25	0.67	2.8	5	125
22.09	25.06	0.72	2.8	5	125
22.17	25.53	0.77	2.9	5	135
22.27	27.8	0.85	3	5	135
22.35	30.69	0.94	3	5	135
22.44	33.17	1.05	3.1	5	135
22.54	35.69	1.17	3.2	5	135
22.62	37.37	1.24	3.3	5	135
22.71	37.21	1.33	3.5	5	135
22.8	37.07	1.38	3.7	5	135
22.89	38.73	1.41	3.6	5	135
22.97	41.58	1.49	3.5	5	135
23.06	44.25	1.59	3.6	5	135
23.14	43.35	1.66	3.8	5	135
23.23	38.82	1.66	4.3	4	135
23.32	36.58	1.6	4.3	4	135
23.41	34.24	1.44	4.2	4	135
23.49	34.16	1.3	3.8	5	135
23.58	33.83	1.16	3.4	5	135
23.66	34.32	1.1	3.2	5	135
23.76	33.97	1.08	3.1	5	135
23.84	32.96	1.09	3.3	5	135
23.93	31.72	1.2	3.7	5	135
24.01	33.56	1.28	3.8	5	135
24.1	41.46	1.34	3.2	5	135
24.18	63.03	1.32	2.1	7	135
24.27	71.13	1.17	1.6	7	135
24.35	57.13	1.35	2.4	6	135
24.44	42.26	1.53	3.6	5	135
24.53	35.01	1.58	4.5	4	135
24.62	32.38	1.36	4.2	4	135
24.65	43.5	1.36	3.1	6	135
24.73	66.67	1.36	2	7	135
24.81	95.04	1.32	1.4	8	135
24.9	114.25	1.22	1.1	8	125
24.99	129.2	1.17	0.9	9	125
25.07	136.62	1.19	0.9	9	125
25.16	130.97	1.38	1.1	8	125
25.24	116.73	1.79	1.5	8	135
25.33	93.99	2.24	2.4	7	135
25.42	78.15	2.22	2.8	6	135
25.51	62.45	1.93	3.1	6	135
25.59	49.05	1.85	3.8	5	135
25.68	39.79	1.83	4.6	4	135
25.76	38.36	1.83	4.8	4	135
25.85	42.05	1.91	4.5	4	135
25.93	42.42	2	4.7	4	135
26.02	54.07	2.06	3.8	5	135
26.1	89.01	1.97	2.2	7	135
26.18	143.21	1.91	1.3	8	135
26.26	179.98	2.02	1.1	9	125
26.34	211.78	2.28	1.1	9	125
26.42	229.73	2.62	1.1	9	125
26.5	239.02	3.1	1.3	9	135
26.58	232.96	3.51	1.5	9	135
26.66	214.57	3.79	1.8	9	135
26.74	191.51	4.14	2.2	7	135
26.83	166.94	4.66	2.8	7	135
26.9	139.4	4.91	3.5	6	135
27	102.23	4.78	4.7	11	135
27.07	68.8	4.38	6.4	11	135
27.16	50.77	3.73	7.3	3	135
27.24	42.57	3.27	7.7	3	135
27.32	41.4	2.78	6.7	3	135
27.4	42.12	2.57	6.1	3	135
27.48	47.22	2.7	5.7	3	135
27.56	64.48	2.36	3.7	5	135
27.64	82.12	2.56	3.1	6	135
27.71	87.89	2.78	3.2	6	135

27.79	103.47	2.85	2.8	6	135
27.85	116.84	2.89	2.5	7	135
27.93	122.1	3.15	2.6	7	135
28	116.74	3.53	3	6	135
28.08	111.4	3.74	3.4	6	135
28.16	98.4	3.68	3.7	6	135
28.24	82.53	3.58	4.3	5	135
28.32	86.31	3.35	3.9	5	135
28.4	102.07	3.05	3	6	135
28.48	108.35	2.8	2.6	7	135
28.57	106.08	2.74	2.6	7	135
28.64	101.06	2.9	2.9	6	135
28.73	86.11	2.94	3.4	6	135
28.81	68.61	2.88	4.2	5	135
28.89	67.82	2.85	4.2	5	135
28.97	66.01	2.74	4.1	5	135
29.05	73.4	2.8	3.8	5	135
29.14	76.19	3.05	4	5	135
29.22	86.82	3.22	3.7	5	135
29.3	82.57	3.34	4	5	135
29.38	76.38	3.44	4.5	11	135
29.46	81.7	3.6	4.4	11	135
29.54	85.93	3.59	4.2	5	135
29.62	89.96	3.44	3.8	5	135
29.7	90.99	3.27	3.6	6	135
29.79	90.44	3.1	3.4	6	135
29.87	90.56	2.92	3.2	6	135
29.95	87.54	2.7	3.1	6	135
30.03	83.54	2.45	2.9	6	135
30.11	78.25	2.42	3.1	6	135
30.19	74.35	2.57	3.5	6	135
30.28	67.54	2.61	3.9	5	135
30.35	60.42	2.37	3.9	5	135
30.44	65.64	2.02	3.1	6	135
30.52	81.06	1.73	2.1	7	135
30.59	87.73	1.52	1.7	7	135
30.68	87.34	1.4	1.6	7	135
30.77	83.76	1.53	1.8	7	135
30.85	82.76	1.48	1.8	7	135
30.93	75.86	1.69	2.2	7	135
31.02	60.74	2.14	3.5	5	135
31.08	43.91	2.29	5.2	3	135
31.18	42.78	2.21	5.2	3	135
31.26	34.73	2.04	5.9	3	135
31.35	30.53	1.8	5.9	3	135
31.43	25.3	1.65	6.5	3	135
31.52	24.91	1.59	6.4	3	135
31.6	30.09	1.57	5.2	3	135
31.68	40.92	1.61	3.9	5	135
31.76	48.33	1.73	3.6	5	135
31.85	43.14	1.77	4.1	5	135
31.93	33.74	1.69	5	3	135
32.02	25.84	1.65	6.4	3	135
32.1	23.87	1.56	6.5	3	135
32.19	23.6	1.4	5.9	3	135
32.27	22.25	1.27	5.7	3	135
32.36	20.87	1.31	6.3	3	135
32.45	22.25	1.31	5.9	3	135
32.53	23.42	1.17	5	3	135
32.62	23.35	1.15	4.9	3	135
32.7	23.09	1.14	4.9	3	135
32.78	22.14	1.07	4.8	3	135
32.87	19.82	0.97	4.9	3	135
32.95	16.1	0.84	5.2	3	125
33.04	14.64	0.66	4.5	3	125
33.12	13.36	0.56	4.1	3	125
33.21	12.38	0.5	4	3	125
33.29	12.79	0.47	3.7	4	125
33.38	14.16	0.55	3.9	4	125
33.47	16.59	0.71	4.3	3	125
33.56	20.36	0.92	4.5	3	135
33.64	30.71	1.12	3.6	5	135
33.73	34.57	1.29	3.7	5	135
33.81	33.03	1.42	4.3	4	135
33.89	31.58	1.45	4.6	4	135
33.98	29.81	1.42	4.8	3	135
34.06	29.72	1.4	4.7	3	135
34.15	28.61	1.36	4.7	3	135
34.4	29.26	1.2	4.1	4	135
34.49	25.94	1.17	4.5	4	135
34.57	24.66	1.16	4.6	3	135
34.66	23.13	1.04	4.5	3	135
34.75	18.6	0.9	4.8	3	125
34.84	16.84	0.78	4.6	3	125

34.92	15.7	0.68	4.3	3	125
35.01	14.33	0.59	4.1	3	125
35.1	13.77	0.57	4.1	3	125
35.19	14.01	0.55	3.9	4	125
35.28	13.36	0.52	3.8	4	125
35.36	13.13	0.53	4	3	125
35.45	12.95	0.56	4.3	3	125
35.53	13.12	0.58	4.3	3	125
35.62	14.49	0.55	3.8	4	125
35.71	13.93	0.51	3.6	4	125
35.8	12.24	0.49	3.9	3	125
35.88	11.68	0.47	4	3	125
35.97	10.94	0.42	3.8	3	115
36.05	11.19	0.41	3.6	4	115
36.15	10.37	0.38	3.6	3	115
36.23	9.41	0.34	3.5	3	115
36.32	8.68	0.3	3.4	3	115
36.41	8.53	0.29	3.3	3	115
36.5	7.71	0.34	4.2	3	115
36.58	7.31	0.35	4.6	3	115
36.66	8.28	0.32	3.8	3	115
36.73	10.3	0.31	2.9	4	115
36.82	10.06	0.31	3	4	115
36.9	9.42	0.32	3.3	4	115
36.99	9.66	0.32	3.2	4	115
37.09	10.94	0.32	2.9	4	115
37.17	10.06	0.39	3.8	3	115
37.25	11.03	0.49	4.3	3	125
37.34	16.11	0.52	3.2	5	125
37.44	13.69	0.57	4.1	3	125
37.51	12.64	0.58	4.5	3	125
37.6	14.66	0.54	3.6	4	125
37.68	15.05	0.46	3	5	125
37.76	13.04	0.43	3.2	4	125
37.85	13.68	0.41	2.9	5	125
37.94	13.2	0.36	2.7	5	115
38.02	10.06	0.32	3.1	4	115
38.11	7.23	0.27	3.6	3	105
38.2	6.59	0.26	3.8	3	105
38.28	6.83	0.29	4.1	3	105
38.37	7.56	0.32	4.1	3	115
38.46	9.65	0.35	3.5	3	115
38.55	11.51	0.41	3.5	4	115
38.63	12.24	0.41	3.3	4	115
38.72	13.12	0.43	3.2	4	125
38.81	18.76	0.49	2.6	5	125
38.89	17.88	0.53	2.9	5	125
38.98	12.88	0.55	4.2	3	125
39.07	12.15	0.56	4.6	3	125
39.16	21.68	0.48	2.2	6	125
39.24	28.93	0.45	1.6	6	125
39.33	30.48	0.54	1.8	6	125
39.42	27.65	0.65	2.4	6	125
39.5	25.14	0.7	2.8	5	125
39.59	22.47	0.67	3	5	125
39.68	18.67	0.67	3.6	4	125
39.76	15.3	0.73	4.8	3	125
39.85	11.75	0.82	6.9	3	125
39.94	13.84	0.84	6	3	125
40.03	22.8	0.78	3.4	5	135
40.12	32.7	0.69	2.1	6	125
40.2	37.39	0.59	1.6	7	125
40.29	40.9	0.49	1.2	7	125
40.38	41.63	0.58	1.4	7	125
40.46	38.68	0.76	2	6	135
40.55	27.07	0.82	3	5	135
40.64	18.44	0.82	4.4	3	125
40.73	14.48	0.77	5.3	3	125
40.8	10.61	0.75	7.1	3	125
40.85	14.33	0.76	5.2	3	125
40.93	18.77	0.72	3.8	4	125
41.01	22.23	0.65	2.9	5	125
41.1	27.54	0.63	2.3	6	125
41.18	30.76	0.67	2.2	6	125
41.27	32.05	0.63	1.9	6	125
41.35	34.81	0.53	1.5	7	125
41.44	38.52	0.57	1.5	7	125
41.52	38.86	0.69	1.8	6	125
41.61	32.49	0.84	2.6	6	135
41.7	28.21	0.92	3.3	5	135
41.79	25.78	0.85	3.3	5	135
41.87	25.56	0.68	2.7	5	125
41.96	25.22	0.52	2	6	125
42.04	26.11	0.48	1.8	6	125

42.13	24.63	0.61	2.5	6	125
42.22	21.18	0.67	3.2	5	125
42.31	16.74	0.68	4	4	125
42.39	14.39	0.69	4.8	3	125
42.48	13.92	0.62	4.4	3	125
42.57	13.43	0.51	3.8	4	125
42.65	12.64	0.4	3.2	4	115
42.74	10.29	0.3	2.9	4	115
42.82	8.76	0.24	2.7	4	105
42.91	7.72	0.23	2.9	4	105
43	6.34	0.27	4.2	3	105
43.08	6.99	0.34	4.8	3	115
43.17	9.09	0.37	4	3	115
43.26	15.28	0.39	2.5	5	125
43.34	21.82	0.4	1.8	6	125
43.43	22.25	0.41	1.8	6	125
43.52	18.59	0.43	2.3	5	125
43.6	14.24	0.43	3	5	125
43.69	10.96	0.43	3.9	3	115
43.78	8.36	0.28	3.3	3	105
43.87	8.76	0.33	3.7	3	115
43.96	12.14	0.41	3.3	4	115
44.04	15.96	0.38	2.3	5	125
44.1	10.85	0.37	3.2	4	115
44.2	16.59	0.36	2.2	5	125
44.28	14.2	0.36	2.5	5	115
44.37	10.62	0.35	3.2	4	115
44.46	9.32	0.32	3.3	4	115
44.54	10.29	0.27	2.5	5	115
44.63	10.23	0.21	2	5	105
44.72	8.36	0.2	2.3	4	105
44.8	7.4	0.18	2.3	4	105
44.9	7.89	0.19	2.3	4	105
44.98	7.55	0.2	2.4	4	105
45.07	7.55	0.21	2.6	4	105
45.15	8.29	0.24	2.7	4	105
45.25	7.63	0.31	3.8	3	105
45.33	7.71	0.35	4.2	3	115
45.42	12.64	0.47	3.5	4	125
45.5	38.62	0.79	2	6	135
45.59	57.13	1.05	1.8	7	135
45.67	47.5	1.04	2.2	6	135
45.76	41.08	1.14	2.7	6	135
45.85	34.35	1.3	3.7	5	135
45.93	34.82	1.45	4.1	4	135
46.02	35.41	1.57	4.3	4	135
46.11	34.22	1.56	4.5	4	135
46.19	31.73	1.44	4.5	4	135
46.28	28.77	1.29	4.4	4	135
46.36	28.18	1.16	4.1	4	135
46.45	28.43	1.12	3.9	4	135
46.54	30.61	1.18	3.8	5	135
46.62	37.71	1.35	3.5	5	135
46.71	48.42	1.62	3.3	5	135
46.79	55.84	1.86	3.3	6	135
46.87	55.15	2.11	3.8	5	135
46.96	54.46	2.68	4.9	4	135
47.04	54.37	2.61	4.7	4	135
47.12	51.3	2.36	4.5	4	135
47.19	50.77	2.2	4.3	5	135
47.23	49.57	2.11	4.2	5	135
47.31	45.06	1.93	4.2	5	135
47.4	43.1	1.76	4	5	135
47.48	41.01	1.62	3.9	5	135
47.56	38.59	1.52	3.9	5	135
47.65	38.36	1.45	3.7	5	135
47.73	36.48	1.38	3.7	5	135
47.82	35.54	1.36	3.7	5	135
47.9	32.87	1.35	4	4	135
47.99	31.01	1.31	4.1	4	135
48.07	30.21	1.28	4.2	4	135
48.15	30.06	1.35	4.4	4	135
48.24	31.89	1.42	4.4	4	135
48.33	33.93	1.36	3.9	5	135
48.41	33.11	1.22	3.6	5	135
48.49	32.07	1.11	3.4	5	135
48.58	28.61	1.05	3.6	5	135
48.67	25.71	1.04	4	4	135
48.76	24.01	1.04	4.3	4	135
48.84	22.48	1.03	4.5	3	135
48.93	21.84	0.98	4.4	3	135
49.01	20.77	0.95	4.5	3	135
49.09	20.72	0.92	4.4	3	135
49.18	20.29	0.9	4.4	3	135

Example of Output Data From Liquefaction Assessment Spreadsheet

CPT-Based Liquefaction Triggering Analysis

Per recommendations by Youd et al. (2001), "LIQUEFACTION RESISTANCE OF SOILS: SUMMARY REPORT FROM THE 1996 NCEER AND 1998 NCEER/NSF WORKSHOPS ON EVALUATION OF LIQUEFACTION RESISTANCE OF SOILS", ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 132, No.8

Project Name: Crosstown Freeway — Analyzed by: Hamid Parsa — Mw= 6.7 — Field Ground Water Depth (feet) = 11.3 — Fill Height (feet) = 0 — Factor of Safety for Liquefaction = 1.0
Project No.: 111904.G01 — Date : 4/28/2011 — PGA= 0.32 — Design Ground Water Depth 11 — Unit Weight of the Fill (pcf) = 120 — Factor of Safety for Settlement Calculation = 1.3
Project Location: Stockton, CA — Checked by: Carl
CPT I.D.: CPT-1 — Date: — Total Settlement = 0.58 Inches

Depth (feet)	Depth (meters) From	To	Thickness (meter)	Design Depth to Water (m)	Field Depth to Water (m)	Unit Weight (pcf)	SBT Zone	SBT Classification	Unit Wt. (kN/m³)	σ_{vo} (Kpa)	Design σ_{vo}' (Kpa)	Field σ_{vo}' (Kpa)	σ_{vo}+fill (Kpa)	Dolsgn σ_{vo}'+fill (Kpa)	Field σ_{vo}'+fill (Kpa)	f_s (tsf)	q_c (tsf)	f_s (Kpa)	q_c (Mpa)	n Factor	Normalization Factor C_N	CSR	K_σ	K_α	CSR $_{SF=1,M_w=7.5}$	CRR $_{SF=1,M_w=7.5}$	FOS	Notes	N_{60}	C_E	$(N_1)_{60}$	$(N_1)_{60cs}$ for S_c	S_c (psf)	$(N_1)_{60cs}$ for Settlement	Volumetric Strain (%)	Settlement (inch)	Accumulative Settlement (inch)
0.57	0.17	0.20	0.02	3.4	3.4	135.0	7	Silty Sand to Sandy Silt	21.2	3.7	3.7	3.7	3.68	3.68	3.68	4.30	157.14	411.8	15.05	0.5	1.70	252.5	778.7	2.7	1.7	7.0	270.5	1.00	0.208	1.00	1.00	0.156	1.00	NL	above WT	29.8	1.70	50.7	50.7	>1600	51.1	0.0000	0.000	0.58
0.64	0.20	0.22	0.02	3.4	3.4	135.0	6	Sandy Silt to Clayey Silt	21.2	4.1	4.1	4.1	4.14	4.14	4.14	4.97	153.52	475.9	14.70	0.5	1.70	246.7	718.0	3.2	1.8	8.7	278.7	1.00	0.208	1.00	1.00	0.156	1.00	NL	above WT	30.0	1.70	51.0	51.0	>1600	51.8	0.0000	0.000	0.58
0.71	0.22	0.24	0.02	3.4	3.4	135.0	7	Silty Sand to Sandy Silt	21.2	4.6	4.6	4.6	4.59	4.59	4.59	4.76	153.86	455.8	14.73	0.5	1.70	247.2	683.1	3.1	1.8	8.6	278.3	1.00	0.208	1.00	1.00	0.156	1.00	NL	above WT	30.0	1.70	51.0	51.0	>1600	51.7	0.0000	0.000	0.58
0.79	0.24	0.27	0.02	3.4	3.4	135.0	6	Sandy Silt to Clayey Silt	21.2	5.1	5.1	5.1	5.10	5.10	5.10	4.02	123.07	442.4	12.74	0.5	1.70	213.8	560.1	3.5	1.9	10.5	255.4	1.00	0.208	1.00	1.00	0.156	1.00	NL	above WT	26.7	1.70	45.4	45.4	>1600	46.5	0.0000	0.000	0.58
0.87	0.27	0.29	0.02	3.4	3.4	135.0	6	Sandy Silt to Clayey Silt	21.2	5.6	5.6	5.6	5.62	5.62	5.62	4.34	117.91	415.6	11.29	0.5	1.70	189.4	472.8	3.7	2.0	11.7	236.4	1.00	0.208	1.00	1.00	0.156	1.00	NL	above WT	24.1	1.70	41.0	42.0	>1600	42.2	0.0000	0.000	0.58
0.94	0.29	0.31	0.02	3.4	3.4	135.0	11	Very Stiff Fine Grained	21.2	6.1	6.1	6.1	6.07	6.07	6.07	4.51	103.40	431.9	9.91	0.5	1.70	166.2	399.1	4.4	2.1	14.4	229.5	1.00	0.208	1.00	1.00	0.156	1.00	NL	above WT	22.0	1.70	37.4	38.4	>1600	38.7	0.0000	0.000	0.58
1.02	0.31	0.34	0.02	3.4	3.4	135.0	11	Very Stiff Fine Grained	21.2	6.6	6.6	6.6	6.59	6.59	6.59	4.63	114.58	483.4	10.97	0.5	1.70	184.1	424.3	4.0	2.0	13.1	241.1	1.00	0.208	1.00	1.00	0.156	1.00	NL	above WT	23.9	1.70	40.7	41.7	>1600	41.9	0.0000	0.000	0.58
1.10	0.34	0.36	0.02	3.4	3.4	135.0	11	Very Stiff Fine Grained	21.2	7.1	7.1	7.1	7.11	7.11	7.11	4.37	99.93	418.5	9.47	0.5	1.70	158.9	352.7	4.4	2.1	15.1	225.5	1.00	0.207	1.00	1.00	0.155	1.00	NL	above WT	21.2	1.70	36.1	37.1	>1600	37.4	0.0000	0.000	0.58
1.18	0.36	0.39	0.03	3.4	3.4	135.0	11	Very Stiff Fine Grained	21.2	7.6	7.6	7.6	7.63	7.63	7.63	3.72	86.15	356.2	8.25	0.5	1.70	138.4	296.5	4.3	2.1	15.9	202.6	1.00	0.207	1.00	1.00	0.155	1.00	NL	above WT	18.7	1.70	31.8	32.8	>1600	33.2	0.0000	0.000	0.58
1.27	0.39	0.41	0.03	3.4	3.4	135.0	11	Very Stiff Fine Grained	21.2	8.2	8.2	8.2	8.21	8.21	8.21	3.68	74.60	369.8	7.16	0.5	1.70	120.2	248.1	5.2	2.2	19.4	204.7	1.00	0.207	1.00	1.00	0.155	1.00	NL	above WT	17.0	1.70	28.8	29.8	>1600	30.5	0.0000	0.000	0.58
1.30	0.41	0.44	0.02	3.4	3.4	135.0	11	Very Stiff Fine Grained	21.2	8.8	8.8	8.8	8.79	8.79	8.79	4.42	88.67	423.3	5.62	0.5	1.70	94.3	184.0	7.5	2.4	26.7	222.8	1.00	0.207	1.00	1.00	0.155	1.00	NL	above WT	14.5	1.70	24.7	26.7	>1600	26.8	0.0000	0.000	0.58
1.43	0.44	0.46	0.03	3.4	3.4	135.0	11	Very Stiff Fine Grained	21.2	9.2	9.2	9.2	9.24	9.24	9.24	4.84	90.12	465.5	8.63	0.5	1.70	144.8	281.7	5.4	2.2	19.0	242.3	1.00	0.207	1.00	1.00	0.155	1.00	NL	above WT	20.3	1.70	34.6	35.6	>1600	36.2	0.0000	0.000	0.58
1.52	0.46	0.48	0.02	3.4	3.4	140.0	12	Sand to Clayey Sand	22.0	9.8	9.8	9.8	9.84	9.84	9.84	5.62	149.54	535.2	14.32	0.5	1.70	240.3	453.1	3.8	2.0	12.2	304.9	1.00	0.207	1.00	1.00	0.155	1.00	NL	above WT	30.8	1.70	52.4	53.4	>1600	53.6	0.0000	0.000	0.58
1.59	0.48	0.51	0.03	3.4	3.4	140.0	12	Sand to Clayey Sand	22.0	10.3	10.3	10.3	10.31	10.31	10.31	6.01	155.54	575.5	14.89	0.5	1.70	249.9	460.4	3.9	2.0	12.4	319.4	1.00	0.207	1.00	1.00	0.155	1.00	NL	above WT	32.2	1.70	54.7	55.7	>1600	55.8	0.0000	0.000	0.58
1.68	0.51	0.53	0.02	3.4	3.4	135.0	7	Silty Sand to Sandy Silt	21.2	10.9	10.9	10.9	10.89	10.89	10.89	4.60	158.17	459.6	15.15	0.5	1.70	254.1	455.6	3.0	1.9	9.9	297.7	1.00	0.207	1.00	1.00	0.155	1.00	NL	above WT	31.5	1.70	53.5	53.5	>1600	54.5	0.0000	0.000	0.58
1.75	0.53	0.56	0.03	3.4	3.4	135.0	7	Silty Sand to Sandy Silt	21.2	11.3	11.3	11.3	11.35	11.35	11.35	4.38	146.99	419.4	14.08	0.5	1.70	236.2	414.8	3.0	1.9	10.3	280.5	1.00	0.207	1.00	1.00	0.155	1.00	NL	above WT	29.5	1.70	50.1	51.1	>1600	51.1	0.0000	0.000	0.58
1.84	0.56	0.58	0.02	3.4	3.4	135.0	7	Silty Sand to Sandy Silt	21.2	11.9	11.9	11.9	11.93	11.93	11.93	4.13	147.59	395.5	14.13	0.5	1.70	237.1	396.2	2.8	1.9	9.8	276.8	1.00	0.207	1.00	1.00	0.155	1.00	NL	above WT	29.3	1.70	49.9	49.9	>1600	50.8	0.0000	0.000	0.58
1.91	0.58	0.60	0.02	3.4	3.4	135.0	7	Silty Sand to Sandy Silt	21.2	12.4	12.4	12.4	12.38	12.38	12.38	3.77	145.00	361.0	13.89	0.5	1.70	233.0	391.7	2.6	1.9	9.3	267.8	1.00	0.207	1.00	1.00	0.155	1.00	NL	above WT	28.6	1.70	48.6	48.6	>1600	49.5	0.0000	0.000	0.58
1.97	0.60	0.63	0.02	3.4	3.4	135.0	7	Silty Sand to Sandy Silt	21.2	12.8	12.8	12.8	12.77	12.77	12.77	3.71	137.16	355.3	13.13	0.5	1.70	220.3	364.8	2.7	1.9	9.9	258.8	1.00	0.207	1.00	1.00	0.155	1.00	NL	above WT	27.3	1.70	46.5	46.5	>1600	47.5	0.0000	0.000	0.58
2.05	0.63	0.65	0.02	3.4	3.4	135.0	6	Sandy Silt to Clayey Silt	21.2	13.3	13.3	13.3	13.29	13.29	13.29	3.72	117.48	356.2	11.25	0.5	1.70	188.7	306.3	3.2	2.0	12.5	242.0	1.00	0.207	1.00	1.00	0.155	1.00	NL	above WT	24.3	1.70	41.3	42.3	>1600	42.5	0.0000	0.000	0.58
2.13	0.65	0.67	0.02	3.4	3.4	135.0	5	Clayey Silt to Silty Clay	21.2	13.8	13.8	13.8	13.80	13.80	13.80	3.78	99.67	362.0	9.26	0.5	1.70	155.3	247.2	3.9	2.1	15.9	228.0	1.00	0.207	1.00	1.00	0.155	1.00	NL	above WT	21.0	1.70	35.7	36.7	>1600	37.1	0.0000	0.000	0.58
2.21	0.67	0.70	0.03	3.4	3.4	135.0	11	Very Stiff Fine Grained	21.2	14.3	14.3	14.3	14.32	14.32	14.32	3.66	69.74	340.9	6.68	0.5	1.70	112.0	175.0	5.1	2.3	21.8	233.1	1.00	0.207	1.00	1.00	0.155	1.00	NL	above WT	16.3	1.70	27.7	28.7	>1600	29.5	0.0000	0.000	0.58
2.30	0.70	0.73	0.02	3.4	3.4	135.0	3	Clay	21.2	14.9	14.9	14.9	14.90	14.90	14.90	3.28	51.66	314.1	4.97	0.5	1.70	83.3	127.4	6.3	2.4	27.8	205.9	0.99	0.207	1.00	1.00	0.155	1.00	NL	above WT	13.0	1.70	22.1	24.1	>1600	24.3	0.0000	0.000	0.58
2.38	0.73	0.75	0.02	3.4	3.4	135.0	3	Clay	21.2	15.4	15.4	15.4	15.42	15.42	15.42	3.03	45.38	292.2	4.35	0.5	1.70	72.9	110.6	6.7	2.5	30.3	190.3	0.99	0.207	1.00	1.00	0.155	1.00	NL	above WT	11.7	1.70	19.8	21.8	>1600	22.3	0.0000	0.000	0.58
2.46	0.75	0.77	0.02	3.4	3.4	135.0	3	Clay	21.2	15.9	15.9	15.9	15.93	15.93	15.93	2.74	38.34	262.4	3.67	0.5	1.70	61.6	91.0	7.1	2.6	33.4	181.7	0.99	0.207	1.00	1.00	0.155	1.00	NL	above WT	10.2	1.70	17.3	19.3	>1600	20.0	0.0000	0.000	0.58
2.54	0.77	0.80	0.02	3.4	3.4	135.0	3	Clay	21.2	16.5	16.5	16.5	16.45	16.45	16.45	2.46	34.06	235.6	3.26	0.7	1.70	54.7	114.3	7.2	2.5	31.0	154.8	0.99	0.207	1.00	1.00	0.155	0.42	NL	above WT	9.8	1.70	15.0	17.0	>1600	17.5	0.0000	0.000	0.58
2.62	0.80	0.82	0.02	3.4	3.4	135.0	3	Clay	21.2	17.0	17.0	17.0	16.97	16.97	16.97	2.27	29.10	217.4	2.79	0.7	1.70	46.8	95.5	7.8	2.6	34.4	151.2	0.99	0.207	1.00	1.00	0.155	0.49	NL	above WT	7.8	1.70	13.3	15.3	>1600	16.0	0.0000	0.000	0.58
2.70	0.82	0.85	0.02	3.4	3.4	135.0	3	Clay	21.2	17.5	17.5	17.5	17.49	17.49	17.49	2.08	26.22	199.2	2.42	0.7	1.70	40.3	80.9	8.2	2.6	37.3	146.2	0.99	0.207	1.00	1.00	0.155	NL	NL	above WT	7.0	1.70	11.9	13.9	550.0	14.9	0.0000	0.000	0.58
2.78	0.85	0.88	0.03	3.4	3.4	135.0	3	Clay	21.2	18.0	18.0	18.0	18.00	18.00	18.00	1.97	25.27	185.6	2.43	0.7	1.70	40.8	79.8	7.8	2.6	36.6	145.5	0.99	0.207	1.00	1.00	0.155	NL	NL	above WT	7.0	1.70	11.9	13.9	550.0	14.8	0.0000	0.000	0.58
2.87	0.88	0.90	0.02	3.4	3.4	135.0	3	Clay	21.2	18.6	18.6	18.6	18.58	18.58	18.58	1.83	24.23	175.2	2.32	0.7	1.70	39.9	74.5	7.6	2.6	37.1	139.3	0.99	0.207	1.00	1.00	0.155	NL	NL	above WT	6.7	1.70	11.4	13.4	500.0	14.3	0.0000	0.000	0.58
2.95	0.90	0.92	0.02	3.4	3.4	135.0	3	Clay	21.2	19.1	19.1	19.1	19.10	19.10	19.10	1.69	23.75	161.8	2.08	0.7	1.70	34.9	65.5	7.8	2.7	39.3	135.3	0.99	0.207	1.00	1.00	0.155	NL	NL	above WT	6.1	1.70	10.4	12.4	450.0	13.6	0.0000	0.000	0.58
3.03	0.92	0.95	0.02	3.4	3.4	135.0	3	Clay	21.2	19.6	19.6	19.6	19.62	19.62	19.62	1.59	23.93	152.3	2.00	0.7	1.70	31.6	61.8	7.6	2.7	39.6	131.8	0.99	0.207	1.00	1.00	0.155	NL	NL	above WT	5.9	1.70	10.1	12.1	400.0	13.2	0.0000	0.000	0.58
3.11	0.95	0.98	0.03	3.4	3.4	135.0	3	Clay	21.2	20.1	20.1	20.1	20.13	20.13	20.13	1.62	21.10	155.1	2.02	0.7	1.70	33.9	61.2	7.7	2.7	40.0	134.6	0.99	0.206	1.00	1.00	0.155	NL	NL	above WT	6.0	1.70	10.2	12.2	400.0	13.4	0.0000	0.000	0.58
3.20	0.98	1.00	0.02	3.4	3.4	135.0	3	Clay	21.2	20.7	20.7	20.7	20.72	20.72	20.72	1.60	21.69	153.2	2.10	0.7	1.70	35.2	62.2	7.3	2.7	39.8	134.0	0.99	0.206	1.00	1.00	0.155	NL	NL	above WT	6.2	1.70	10.5	12.5	400.0	13.6	0.0000	0.000	0.58
3.29	1.00	1.03	0.03	3.4	3.4	135.0	3	Clay	21.2	21.2	21.2	21.2	21.23	21.23	21.23	1.54	24.59	147.5	2.47	0.7	1.70	39.4	69.7	6.3	2.6	35.0	130.5	0.99	0.206	1.00	1.00	0.155	0.29	NL	above WT	6.6	1.70	11.3	13.3	500.0	14.2	0.0000	0.000	0.58
3.37	1.03	1.05	0.02	3.4	3.4	135.0	3	Clay	21.2	21.8	21.8	21.8	21.81	21.81	21.81	1.54	25.04	147.5	2.40	0.7	1.70	40.2	68.7	6.2	2.6	34.7	131.8	0.99	0.206	1.00	1.00	0.155	0.29	NL	above WT	6.7	1.70	11.5	13.5	500.0	14.2	0.0000	0.000	0.58
3.45	1.05	1.08	0.03	3.4	3.4	135.0	3	Clay	21.2	22.3	22.3	22.3	22.33	22.33	22.33	1.51	25.06	144.8	2.51	0.7	1.70	42.1	71.1	5.9	2.5	32.1	128.6	0.99	0.206	1.00	1.00	0.155	NL	NL	above WT	6.4	1.70	11.0	13.0	500.0	13.8	0.0000	0.000	0.58
3.53	1.08	1.10	0.02	3.4	3.4	135.0	3	Clay	21.2	22.8	22.8	22.8	22.85	22.85	22.85	1.41	17.38	135.0	1.66	1.0	1.70	27.9	71.8	8.1	2.7	38.7	105.8	0.99	0.206	1.00	1.00	0.155	NL	NL	above WT	4.9	1.70	8.3	10.3	300.0	11.4	0.0000	0.000	0.58
3.61	1.10	1.13	0.03	3.4	3.4	135.0	3	Clay	21.2	23.4	23.4	23.4	23.37	23.37	23.37	1.28	15.93	122.6	1.53	1.0	1.70	25.6	64.3	8.1	2.7	40.2	102.4	0.99	0.206	1.00	1.00	0.155	NL	NL	above WT	4.5	1.70	7.7	9.7	275.0	10.9	0.0000	0.000	0.58
3.70	1.13	1.15	0.02	3.4	3.4	135.0	3	Clay	21.2	23.9	23.9	23.9	23.90	23.90	23.90	1.21	17.55	115.9	1.63	1.0	1.70	28.2	66.2	6.9	2.6	36.4	98.4	0.99	0.206	1.00	1.00	0.155	NL	NL	above WT	4.5	1.70	8.2	10.2	300.0	11.1	0.0000	0.000	0.58
3.78	1.15	1.18	0.03	3.4	3.4	135.0	3	Clay	21.2	24.5	24.5	24.5	24.46	24.46	24.46	1.18	15.93	113.0	1.53	1.0	1.70	25.6	61.4	7.4	2.7	39.3	99.0	0.99	0.206	1.00	1.00	0.155	NL	NL	above WT	4.5	1.70	7.6	9.6	275.0	10.8	0.0000	0.000	0.58
3.87	1.18	1.20	0.02	3.4	3.4	135.0	3	Clay	21.2	25.0	25.0	25.0	25.05	25.05	25.05	1.15	14.60	111.1	1.42	1.0	1.70	23.8	55.6	7.9	2.7	41.9	100.6	0.99	0.206	1.00	1.00	0.154	NL	NL	above WT	4.3	1.70	7.3	9.3	250.0	10.6	0.0000	0.000	0.58
3.95	1.20	1.23	0.03	3.4	3.4	135.0	3	Clay	21.2	25.6	25.6	25.6	25.56	25.56	25.56	1.16	15.93	111.1	1.53	1.0	1.70	25.6	58.7	7.3	2.7	39.7	100.4	0.99	0.206	1.00	1.00	0.154	NL	NL	above WT	4.5	1.70	7.7	9.7	275.0	10.8	0.0000	0.000	0.58
4.04	1.23	1.26	0.02	3.4	3.4	135.0	3	Clay	21.2	26.1	26.1	26.1	26.14	26.14	26.14	1.15	15.86	110.1	1.52	1.0	1.70	25.5	57.1	7.3	2.7	40.1	101.3	0.99	0.206	1.00	1.00	0.154	NL	NL	above WT	4.5	1.70	7.7	9.7	275.0	10.9	0.0000	0.000	0.58
4.12	1.26	1.28	0.03	3.4	3.4	135.0	3	Clay	21.2	26.7	26.7	26.7	26.66	26.66	26.66	1.13	18.44	108.2	1.80	0.7	1.70	30.1	44.7	6.0	2.5	30.3	121.6	0.99	0.206	1.00	1.00	0.154	NL	NL	above WT	5.4	1.70	9.1	11.1	350.0	12.4	0.0000	0.000	0.58
4.21	1.28	1.31	0.03	3.4	3.4	135.0	3	Clay	21.2	27.2	27.2	27.2	27.24	27.24	27.24	1.14	18.04	109.2	1.73	1.0	1.70	29.0	62.4	6.3	2.6	36.3	100.7	0.99	0.206	1.00	1.00	0.154	NL	NL	above WT	5.1	1.70	8.6	10.6	325.0	11.4	0.0000	0.000	0.58
4.30	1.31	1.34	0.03	3.4	3.4	135.0	3	Clay	21.2	27.8	27.8	27.8	27.82	27.82	27.82	1.14	18.59	106.2	1.78	1.0	1.70	29.9	63.0	6.1	2.6	35.6	101.2	0.99	0.206	1.00	1.00	0.154	NL	NL	above WT	5.1	1.70	8.6	10.6	325.0	11.4	0.0000	0.000	0.58
4.39	1.34	1.36	0.02	3.4	3.4	135.0	3	Clay	21.2	28.4	28.4	28.4	28.41	28.41	28.41	1.12	18.44	107.3	1.77	1.0	1.70	29.6	61.2	6.1	2.6	36.0	100.9	0.99	0.206	1.00	1.00	0.154	NL	NL	above WT	5.0	1.70	8.6	10.6	325.0	11.4	0.0000	0.000	0.58
4.47	1.36	1.39	0.03	3.4	3.4	135.0	3	Clay	21.2	28.9	28.9	28.9	28.92	28.92	28.92	1.12	18.63	107.3	1.61	1.0	1.70	27.0	54.7	6.7	2.7	39.2	104.3	0.99	0.206	1.00	1.00	0.154	NL	NL	above WT	4.7	1.70	8.1	10.1	300.0	11.2	0.0000	0.000	0.58
4.56	1.39	1.41	0.02	3.4	3.4	135.0	3	Clay	21.2	29.5	29.5	29.5	29.50	29.50	29.50	1.12	18.61	107.3	1.49	1.0	1.70	25.1	49.7	7.3	2.7	41.9	106.2	0.99	0.206	1.00	1.00	0.154	NL	NL	above WT	4.7	1.70	7.7	9.7	275.0	11.0	0.0000	0.000	0.58
4.64	1.41	1.45	0.03	3.4	3.4	135.0	3	Clay	21.2	30.0	30.0	30.0	30.02	30.02	30.02	1.10	14.64	103.4	1.40	1.0	1.70	23.5	45.7	7.5	2.8	43.9	106.4	0.99	0.206	1.00	1.00	0.154	NL	NL	above WT	4.3	1.70	7.4	9.4	250.0	10.9	0.0000	0.000	0.58
4.74	1.45	1.47	0.02	3.4	3.4	135.0	3	Clay	21.2	30.6	30.6	30.6	30.62	30.62	30.62	1.08	14.62	103.4	1.42	1.0	1.70	23.8	45.3	7.3	2.8	43.6	106.4	0.99	0.206	1.00	1.00	0.154	NL	NL	above WT	4.4	1.70	7.4	9.4	250.0	10.9	0.0000	0.000	0.58
4.82	1.47	1.50	0.03	3.4	3.4	135.0	3	Clay	21.2	31.1	31.1	31.1	31.14	31.14	31.14	1.08	14.59	103.4	1.55	1.0	1.70	26.0	48.4	6.7	2.7	40.9	104.4	0.99	0.206	1.00	1.00	0.154	NL	NL	above WT	4.6	1.70	7.9	9.9	275.0	11.2	0.0000	0.000	0.58
4.91	1.50	1.52	0.03	3.4	3.4	125.0	3	Clay	21.2	31.7	31.7	31.7	31.68	31.68	31.68	1.04	14.65	99.6	1.49	1.0	1.70	25.5	48.3	7.1	2.8	43.8	106.0	0.99	0.206	1.00	1.00	0.154	NL	NL	above WT	4.3	1.70	7.4	9.4	250.0	10.9	0.0000	0.000	0.58
5.00	1.52	1.55	0.03	3.4	3.4	125.0	3	Clay	19.6	32.2	32.2	32.2	32.23	32.23	32.23	0.96	14.65	91.9	1.49	1.0	1.70	25.5	48.3	6.5	2.7	42.5	101.6	0.99	0.206	1.00	1.00	0.154	NL	NL	above WT	4.3	1.70	7.3	9.3	250.0	10.7	0.0000	0.000	0.58
5.09	1.55	1.58	0.02	3.4	3.4	125.0	3	Clay	19.6	32.8	32.8	32.8	32.75	32.75	32.75	0.94	13.93	90.0	1.33	1.0	1.70	22.4	39.7	6.8	2.8	44.4	102.8	0.99	0.206	1.00	1.00	0.154	NL	NL	above WT	4.1	1.70	7.0	9.0	250.0	10.6	0.0000	0.000	0.58
5.17	1.58	1.63	0.05	3.4	3.4	125.0	3	Clay	19.6	33.2	33.2	33.2	33.23	33.23	33.23	0.96	15.

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Liquefaction Assessment Results/Graphs

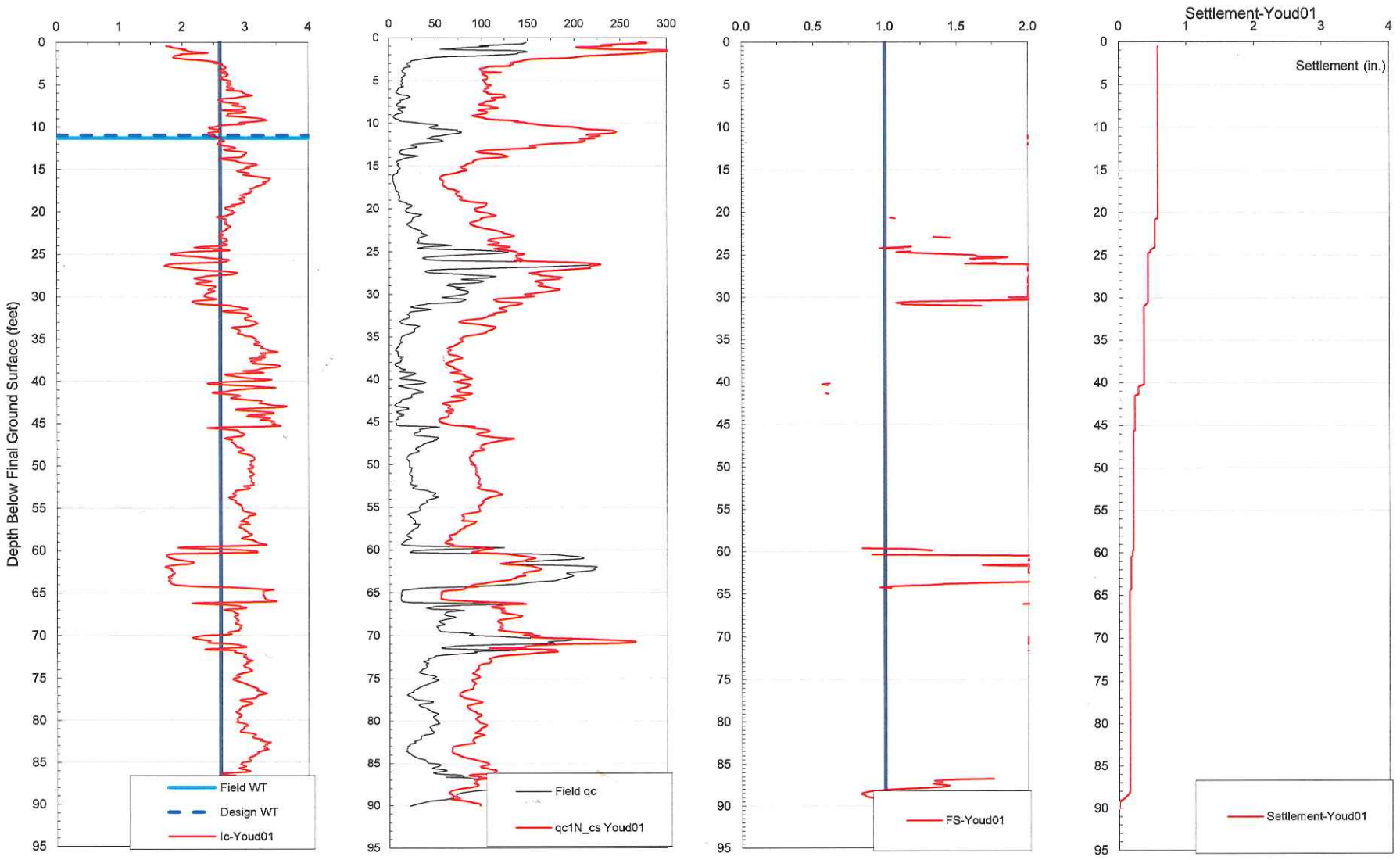
CPT ID: **CPT-11-001**

M_w= **6.7**
PGA= **0.32g**

Field Ground Water Depth (ft)= **11.3 ft**
Design Ground Water Depth (ft)= **11.0 ft**

Existing Ground Elevation= **0.0 ft**
Final Ground Elevation= **0.0 ft**

Ana. by: Hamid Parsa
Checked by: Carl henderson



Note: Factor of safety is capped at 2.0 for liquefiable soils and not plotted for non-liquefiable soils & soils above the ground water; $CRR_{M=7.5,\ \sigma v'=1tsf}$ is capped at 1.0 for liqeufiable soils and not plotted for non-liquefiable soils and soils above the ground water.

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2001 Arch Airport Road, #100
Stockton, California 95206
t: 209 948 1345 f: 209 234 4700

Project Name: Crosstown Freeway
Project No.: 111904.G01
Project Location: Stockton, CA

LIQUEFACTION ANALYSIS

Date 4/28/2011

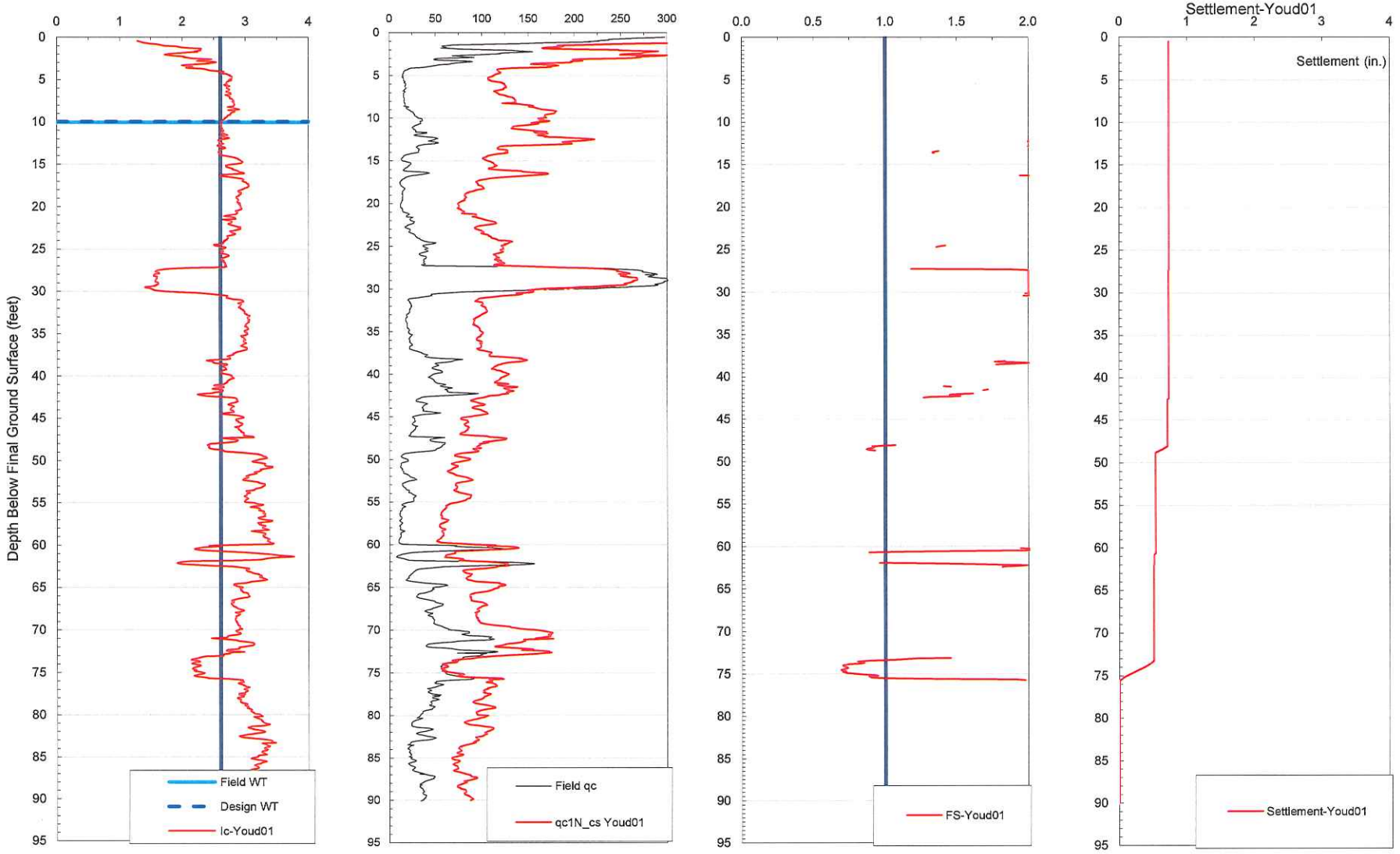
CPT ID: **CPT-11-002**

M_w= **6.7**
PGA= **0.32g**

Field Ground Water Depth (ft)= **10.1 ft**
Design Ground Water Depth (ft)= **10.0 ft**

Existing Ground Elevation= **0.0 ft**
Final Ground Elevation= **0.0 ft**

Ana. by: Hamid Parsa
Checked by: Carl Henderson



Note: Factor of safety is capped at 2.0 for liquefiable soils and not plotted for non-liquefiable soils & soils above the ground water; $CRR_{M=7.5,\ \sigma v'=\ 1tsf}$ is capped at 1.0 for liqeufiable soils and not plotted for non-liquefiable soils and soils above the ground water.

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t: 209 948 1345 f: 209 234 4700

Project Name: Crosstown Freeway
Project No.: 111904.G01
Project Location: Stockton, CA

LIQUEFACTION ANALYSIS

Date 4/28/2011

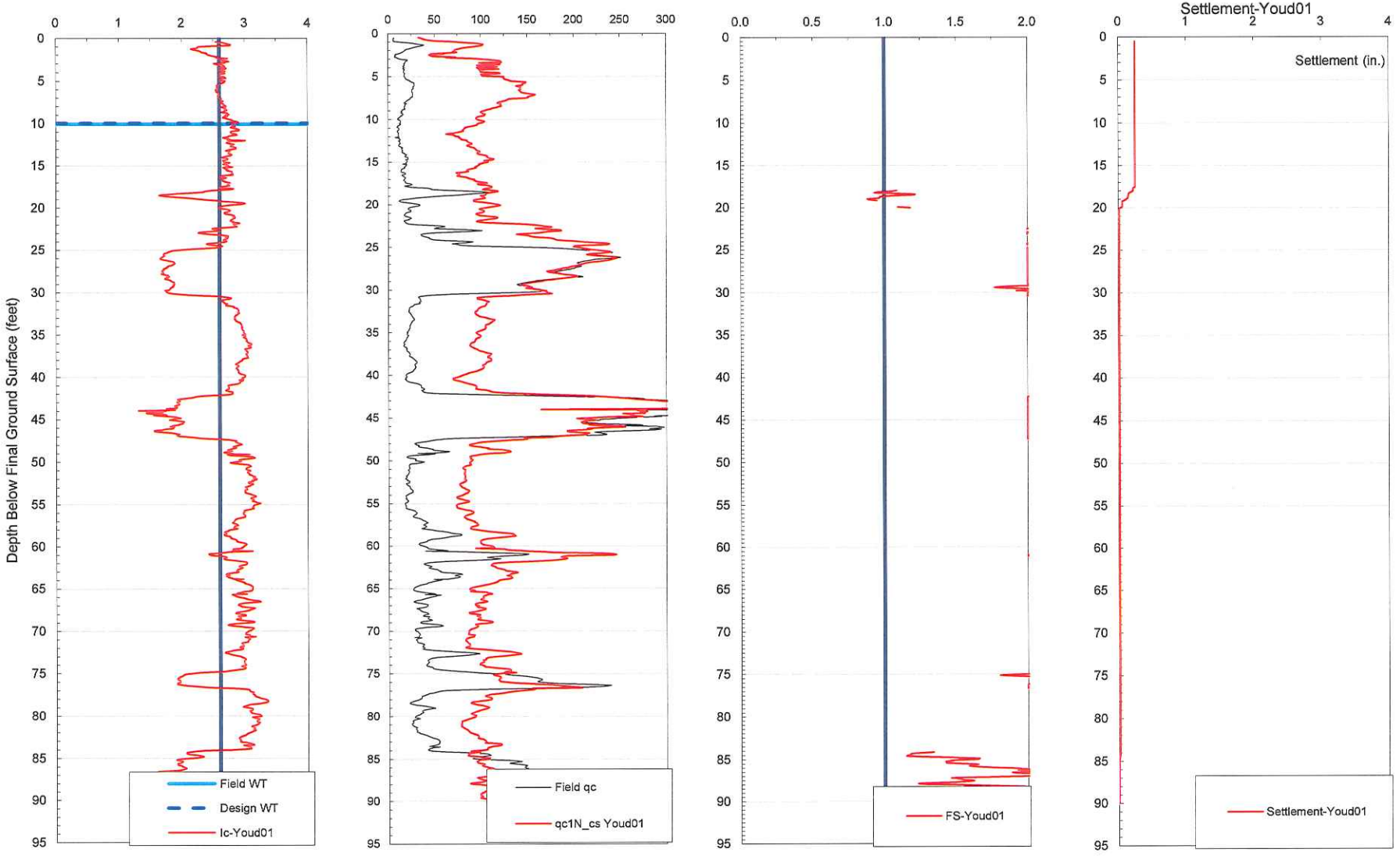
CPT ID: **CPT-11-003**

M_w= **6.7**
PGA= **0.32g**

Field Ground Water Depth (ft)= **10.1 ft**
Design Ground Water Depth (ft)= **10.0 ft**

Existing Ground Elevation= **0.0 ft**
Final Ground Elevation= **0.0 ft**

Ana. by: Hamid Parsa
Checked by: Carl Henderson



Note: Factor of safety is capped at 2.0 for liquefiable soils and not plotted for non-liquefiable soils & soils above the ground water; $CRR_{M=7.5,\ \sigma v'=1tsf}$ is capped at 1.0 for liqeufiable soils and not plotted for non-liquefiable soils and soils above the ground water.

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Project Name: Crosstown Freeway
Project No.: 111904.G01
Project Location: Stockton, CA

LIQUEFACTION ANALYSIS

Date 4/28/2011

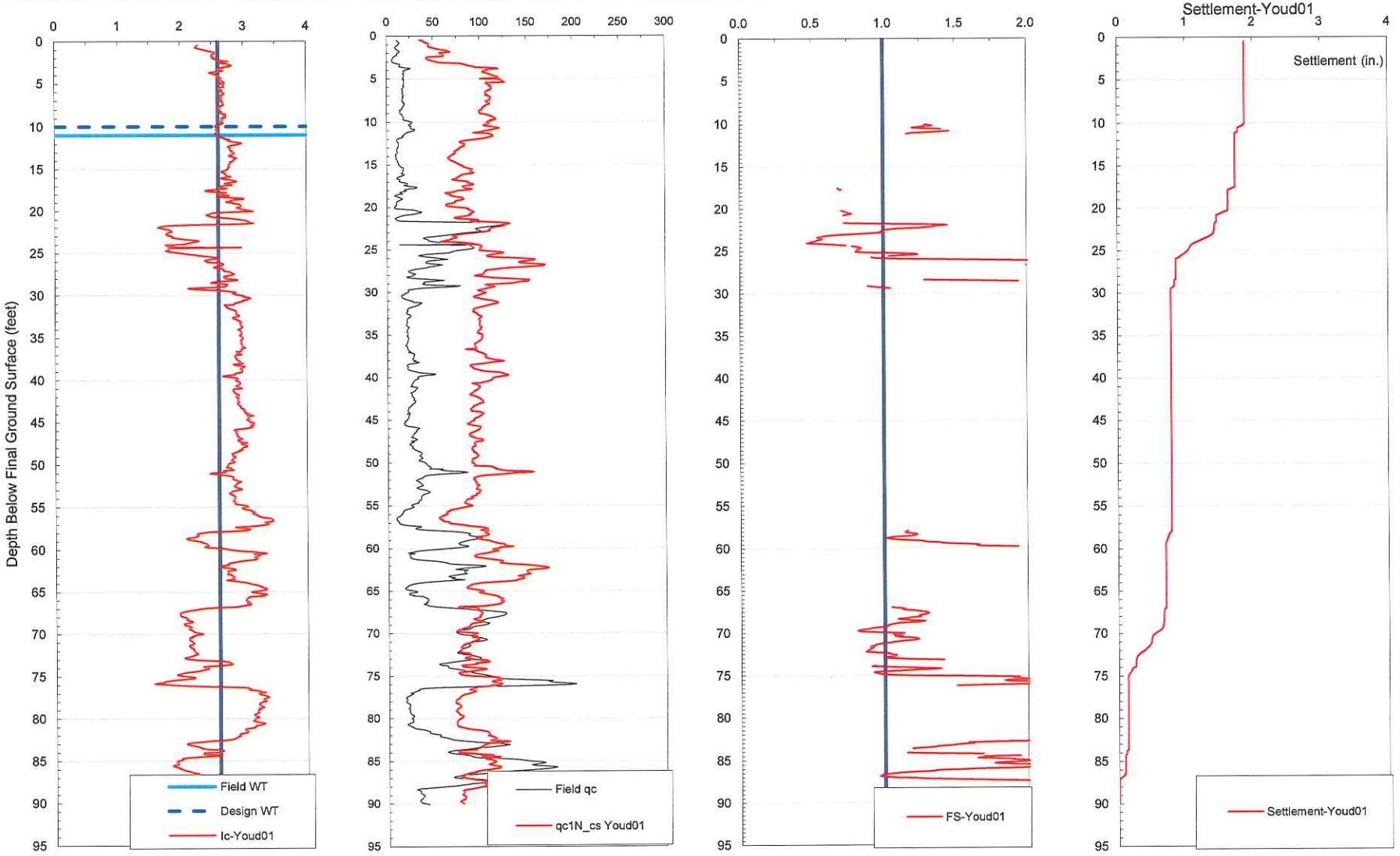
CPT ID: **CPT-11-004**

M_w= **6.7**
PGA= **0.32g**

Field Ground Water Depth (ft)= **11.0 ft**
Design Ground Water Depth (ft)= **10.0 ft**

Existing Ground Elevation= **0.0 ft**
Final Ground Elevation= **0.0 ft**

Ana. by: Hamid Parsa
Checked by: Carl Henderson



Note: Factor of safety is capped at 2.0 for liquefiable soils and not plotted for non-liquefiable soils & soils above the ground water; $CRR_{M=7.5,\ \sigma v'=\ 1tsf}$ is capped at 1.0 for liqeufiable soils and not plotted for non-liquefiable soils and soils above the ground water.

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Project Name: Crosstown Freeway
Project No.: 111904.G01
Project Location: Stockton, CA

LIQUEFACTION ANALYSIS

Date 4/29/2011

CPT ID: **CPT-11-005**

M_w= **6.7**

PGA= **0.32g**

Field Ground Water Depth (ft)= **14.1 ft**

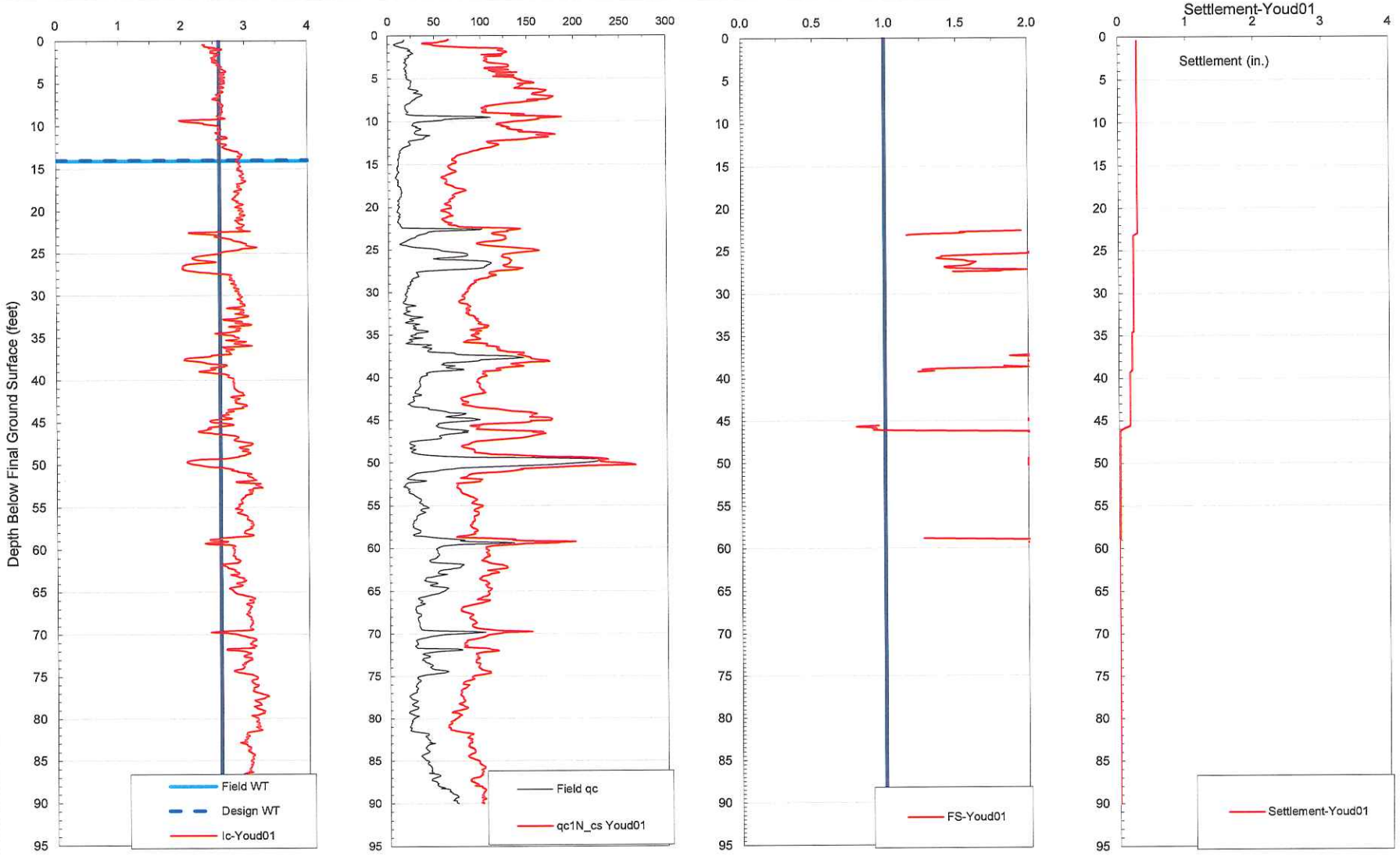
Design Ground Water Depth (ft)= **14.0 ft**

Existing Ground Elevation= **0.0 ft**

Final Ground Elevation= **0.0 ft**

Ana. by: Hamid Parsa

Checked by: Carl Henderson



Note: Factor of safety is capped at 2.0 for liquefiable soils and not plotted for non-liquefiable soils & soils above the ground water; $CRR_{M=7.5,\ \sigma v'=1tsf}$ is capped at 1.0 for liqeufiable soils and not plotted for non-liquefiable soils and soils above the ground water.

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Project Name: Crosstown Freeway
Project No.: 111904.G01
Project Location: Stockton, CA

LIQUEFACTION ANALYSIS

Date 4/28/2011

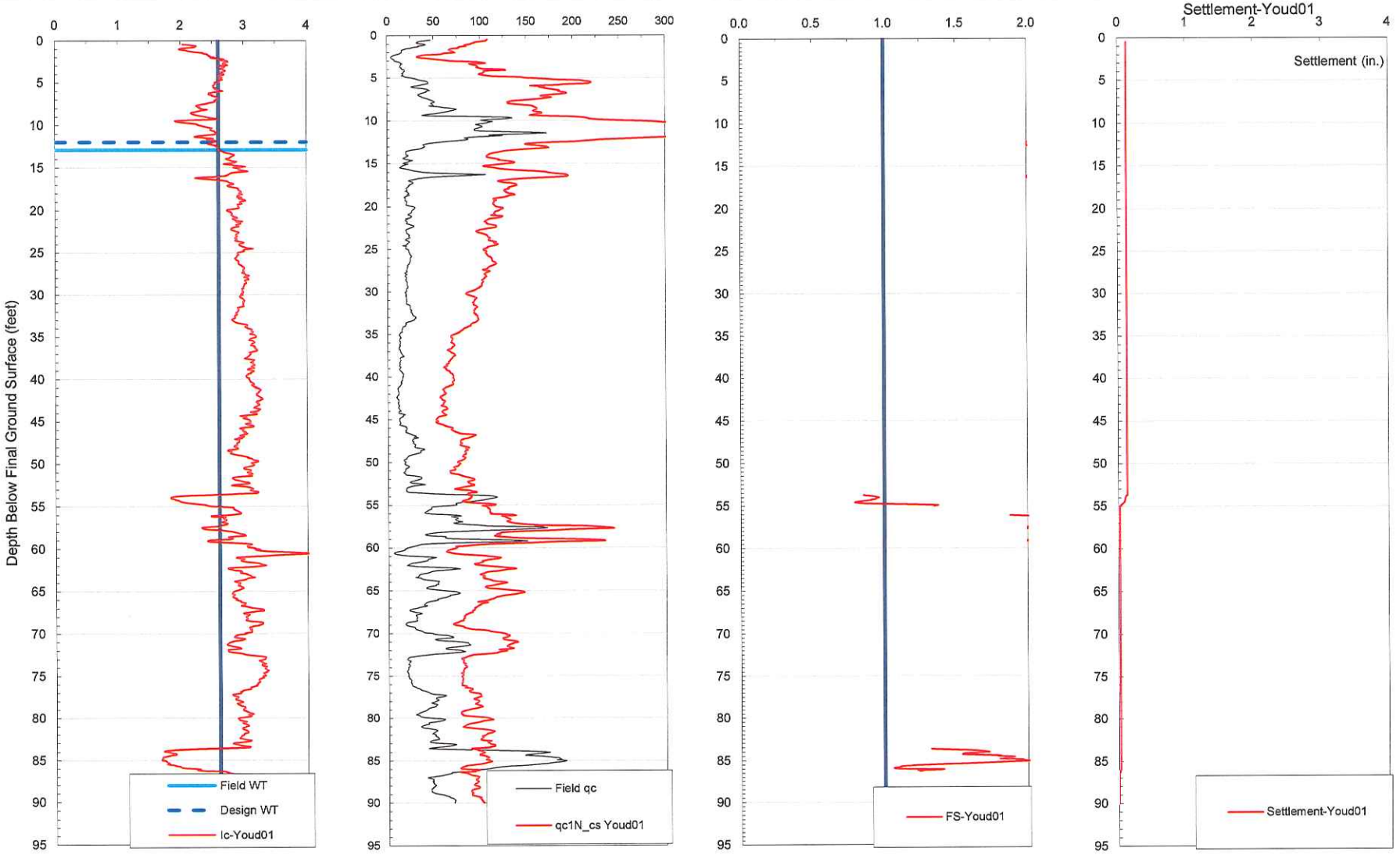
CPT ID: **CPT-11-006**

M_w= **6.7**
PGA= **0.32g**

Field Ground Water Depth (ft)= **12.9 ft**
Design Ground Water Depth (ft)= **12.0 ft**

Existing Ground Elevation= **0.0 ft**
Final Ground Elevation= **0.0 ft**

Ana. by: Hamid Parsa
Checked by: Carl Henderson



Note: Factor of safety is capped at 2.0 for liquefiable soils and not plotted for non-liquefiable soils & soils above the ground water; $CRR_{M=7.5,\ \sigma v'=\ 1tsf}$ is capped at 1.0 for liqeufiable soils and not plotted for non-liquefiable soils and soils above the ground water.

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Project Name: Crosstown Freeway
Project No.: 111904.G01
Project Location: Stockton, CA

LIQUEFACTION ANALYSIS

Date 4/28/2011

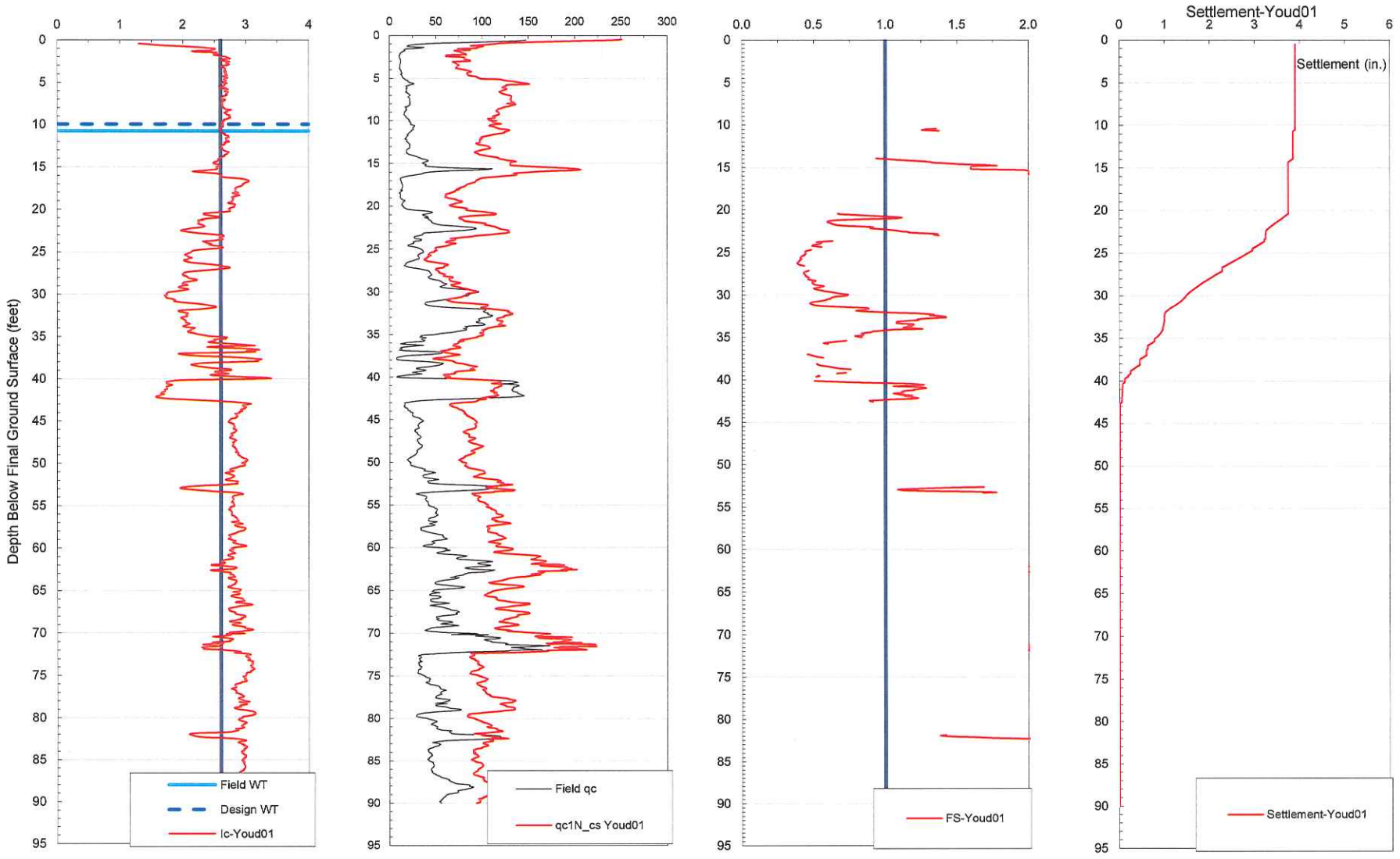
CPT ID: **CPT-11-007**

M_w= **6.7**
PGA= **0.32g**

Field Ground Water Depth (ft)= **10.8 ft**
Design Ground Water Depth (ft)= **10.0 ft**

Existing Ground Elevation= **0.0 ft**
Final Ground Elevation= **0.0 ft**

Ana. by: Hamid Parsa
Checked by: Carl Henderson



Note: Factor of safety is capped at 2.0 for liquefiable soils and not plotted for non-liquefiable soils & soils above the ground water; $CRR_{M=7.5,\ \sigma v'=\ 1tsf}$ is capped at 1.0 for liqeufiable soils and not plotted for non-liquefiable soils and soils above the ground water.

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Project Name: Crosstown Freeway
Project No.: 111904.G01
Project Location: Stockton, CA

LIQUEFACTION ANALYSIS

Date 4/29/2011

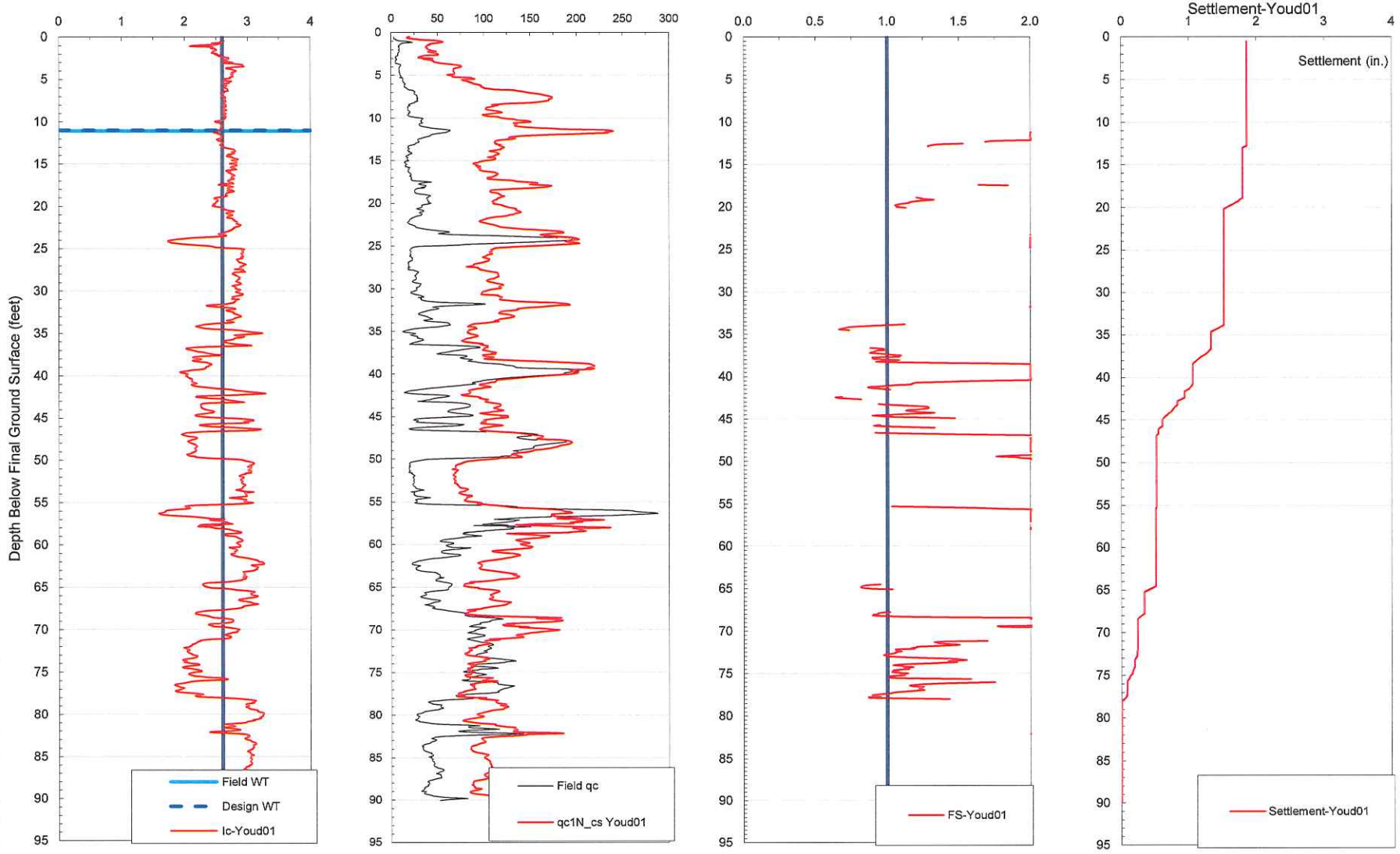
CPT ID: **CPT-11-008**

M_w= **6.7**
PGA= **0.32g**

Field Ground Water Depth (ft)= **11.1 ft**
Design Ground Water Depth (ft)= **11.0 ft**

Existing Ground Elevation= **0.0 ft**
Final Ground Elevation= **0.0 ft**

Ana. by: Hamid Parsa
Checked by: Carl Henderson



Note: Factor of safety is capped at 2.0 for liquefiable soils and not plotted for non-liquefiable soils & soils above the ground water; $CRR_{M=7.5,\ \sigma v'=1tsf}$ is capped at 1.0 for liqeufiable soils and not plotted for non-liquefiable soils and soils above the ground water.

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2001 Arch Airport Road, #100
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Project Name: Crosstown Freeway
Project No.: 111904.G01
Project Location: Stockton, CA

LIQUEFACTION ANALYSIS

Date 4/29/2011

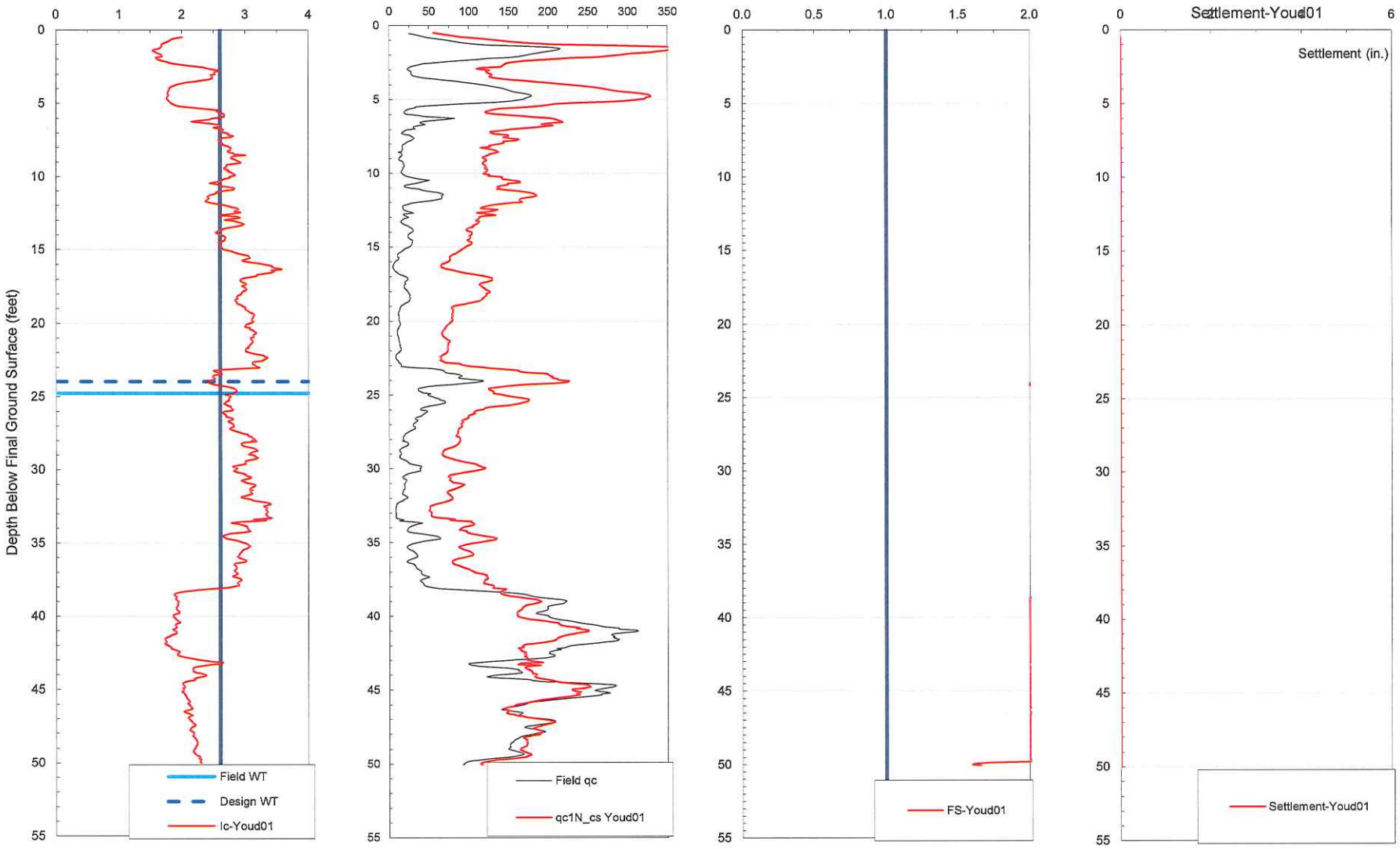
CPT ID: **CPT-11-009**

M_w= **6.7**
PGA= **0.32g**

Field Ground Water Depth (ft)= **24.8 ft**
Design Ground Water Depth (ft)= **24.0 ft**

Existing Ground Elevation= **0.0 ft**
Final Ground Elevation= **0.0 ft**

Ana. by: Hamid Parsa
Checked by: Cral Henderson



Note: Factor of safety is capped at 2.0 for liquefiable soils and not plotted for non-liquefiable soils & soils above the ground water; $CRR_{M=7.5,\ \sigma v=\ 1tsf}$ is capped at 1.0 for liqeufiable soils and not plotted for non-liquefiable soils and soils above the ground water.

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Project Name: Crosstown Freeway
Project No.: 111904.G01
Project Location: Stockton, CA

LIQUEFACTION ANALYSIS

Date 5/2/2011

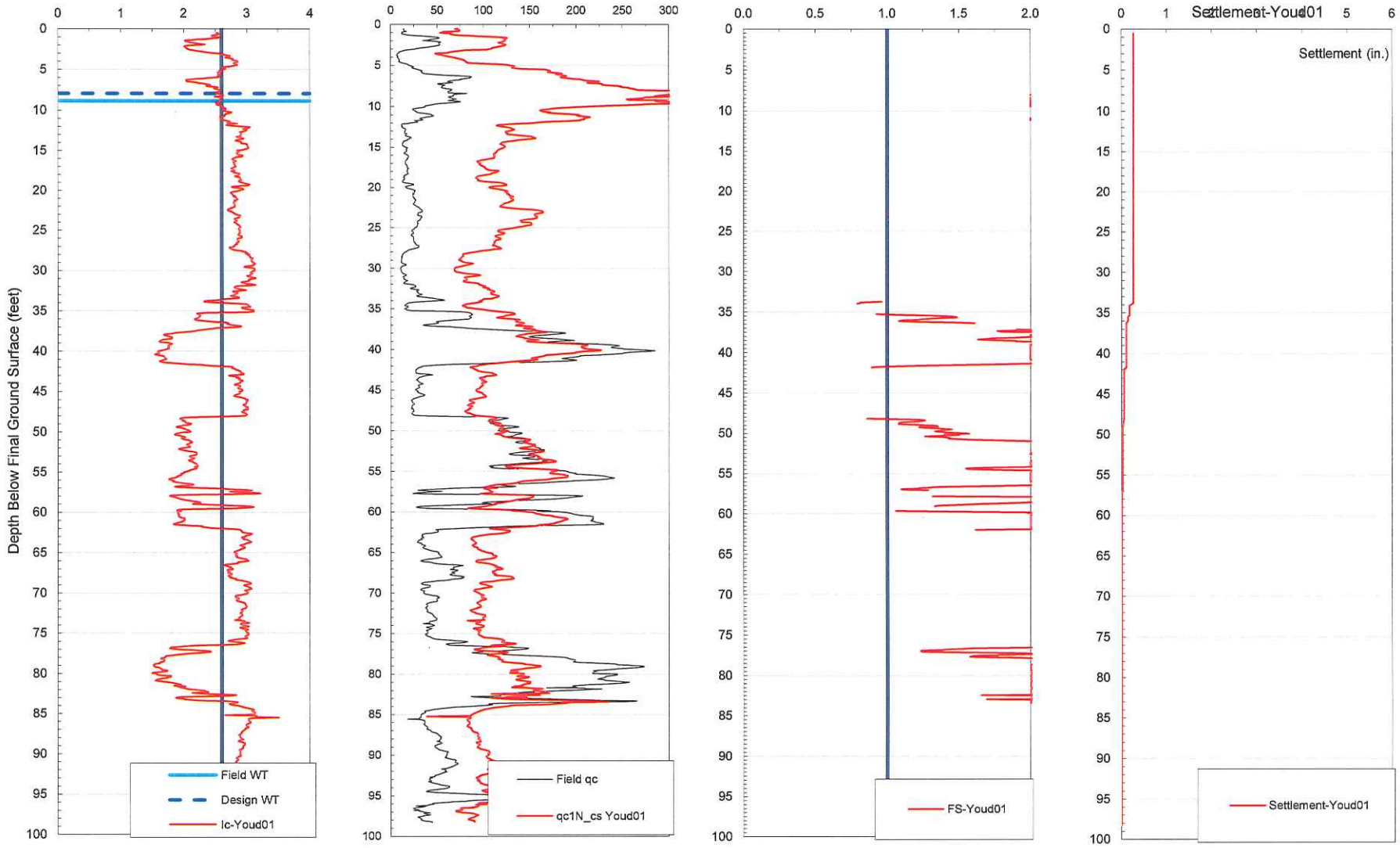
CPT ID: **CPT-11-10**

M_w= **6.7**
PGA= **0.32g**

Field Ground Water Depth (ft)= **8.9 ft**
Design Ground Water Depth (ft)= **8.0 ft**

Existing Ground Elevation= **0.0 ft**
Final Ground Elevation= **0.0 ft**

Ana. by: Hamid Parsa
Checked by: Carl Henderson



Note: Factor of safety is capped at 2.0 for liquefiable soils and not plotted for non-liquefiable soils & soils above the ground water; $CRR_{M=7.5,\ \sigma v'=1tsf}$ is capped at 1.0 for liqeufiable soils and not plotted for non-liquefiable soils and soils above the ground water.

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Project Name: Crosstown Freeway
Project No.: 111904.G01
Project Location: Stockton, CA

LIQUEFACTION ANALYSIS

Date 5/2/2011

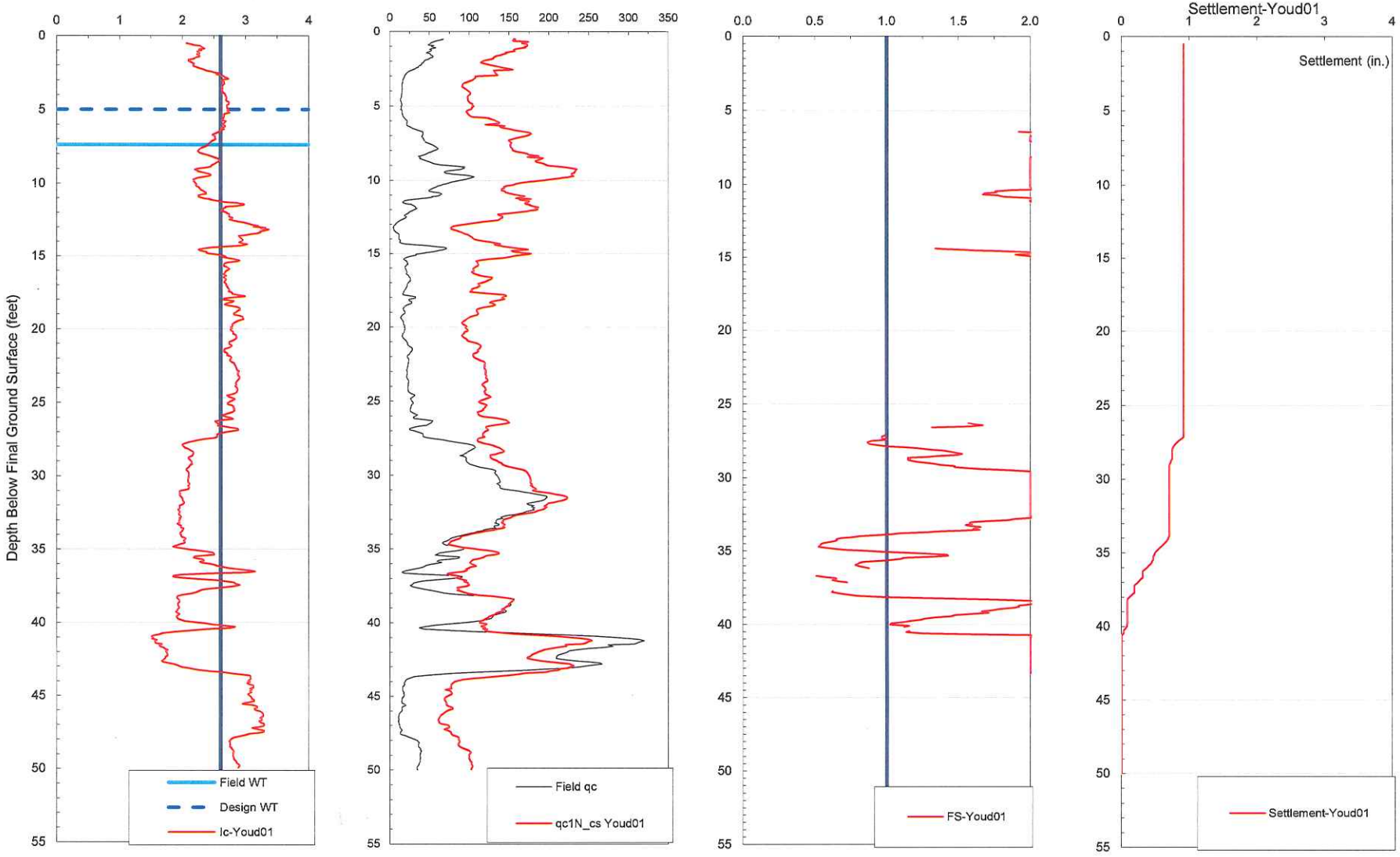
CPT ID: **CPT-11-011**

M_w= **6.7**
PGA= **0.32g**

Field Ground Water Depth (ft)= **7.4 ft**
Design Ground Water Depth (ft)= **5.0 ft**

Existing Ground Elevation= **0.0 ft**
Final Ground Elevation= **0.0 ft**

Ana. by: Hamid Parsa
Checked by: Carl Henderson



Note: Factor of safety is capped at 2.0 for liquefiable soils and not plotted for non-liquefiable soils & soils above the ground water; $CRR_{M=7.5,\ \sigma v'=1tsf}$ is capped at 1.0 for liqeufiable soils and not plotted for non-liquefiable soils and soils above the ground water.

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2001 Arch Airport Road, #100
Stockton, California 95206
t: 209 948 1345 f: 209 234 4700

Project Name: Crosstown Freeway
Project No.: 111904.G01
Project Location: Stockton, CA

LIQUEFACTION ANALYSIS

Date 5/2/2011

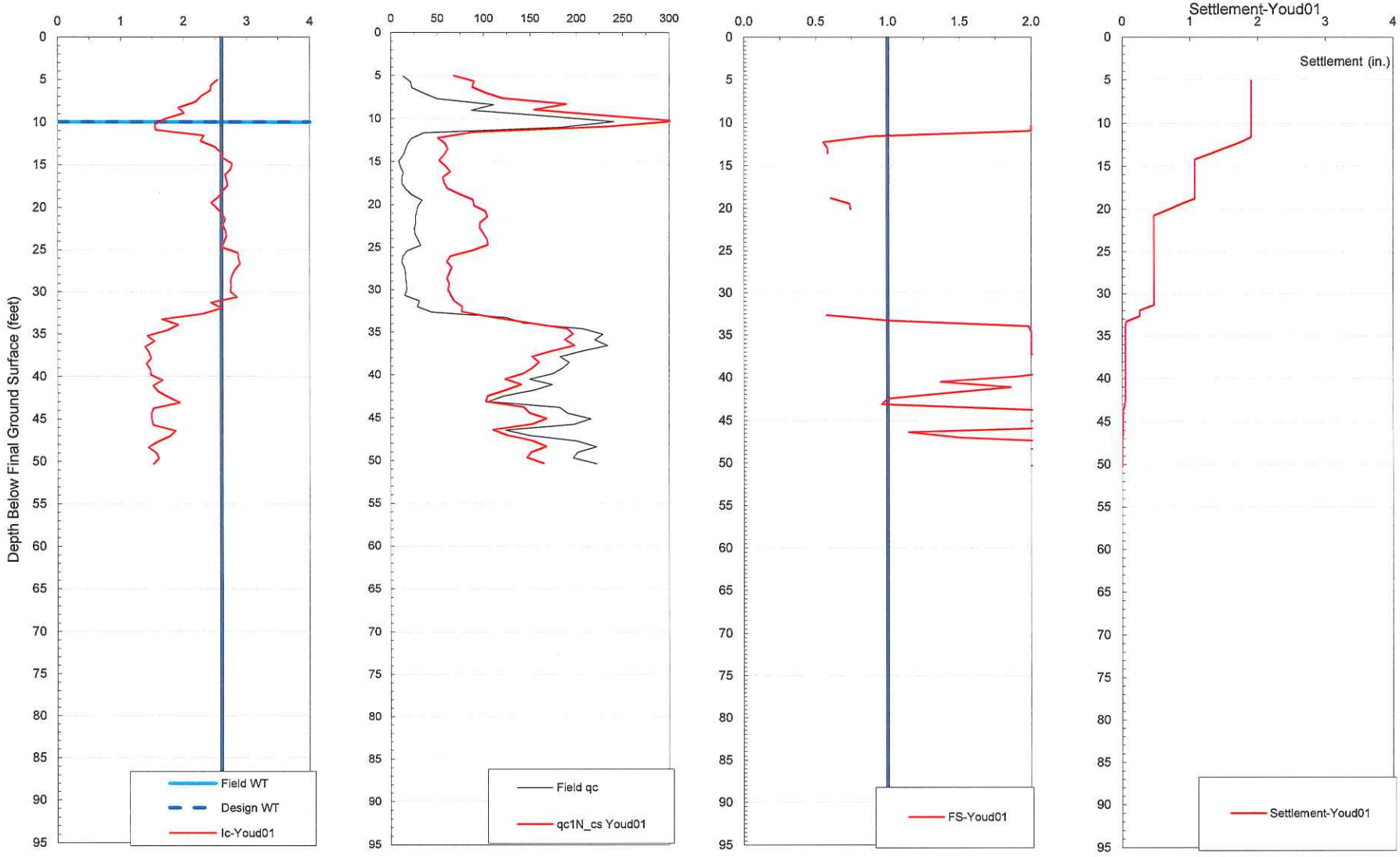
CPT ID: **CPT-09-001**

M_w= **6.7**
PGA= **0.32g**

Field Ground Water Depth (ft)= **10.0 ft**
Design Ground Water Depth (ft)= **10.0 ft**

Existing Ground Elevation= **0.0 ft**
Final Ground Elevation= **0.0 ft**

Ana. by: Carl Henderson
Checked by: Hamid Parsa



Note: Factor of safety is capped at 2.0 for liquefiable soils and not plotted for non-liquefiable soils & soils above the ground water; $CRR_{M=7.5,\ \sigma v'=\ 1tsf}$ is capped at 1.0 for liqeufiable soils and not plotted for non-liquefiable soils and soils above the ground water.

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2001 Arch Airport Road, #100
Stockton, California 95206
t: 209 948 1345 f: 209 234 4700

Project Name: Crosstown Freeway
Project No.: 111904.G01
Project Location: Stockton, CA

LIQUEFACTION ANALYSIS

Date 5/5/2011

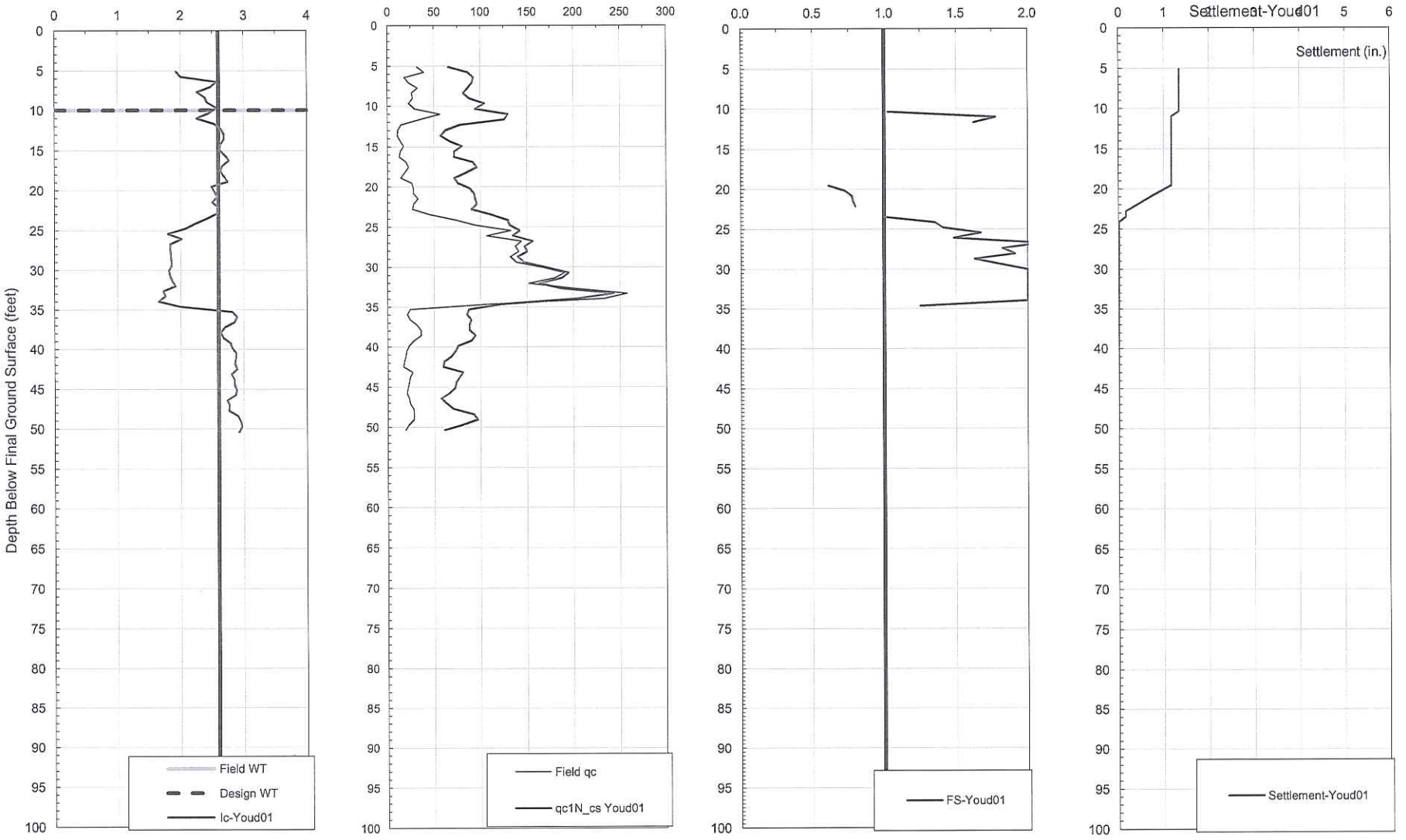
CPT ID: **CPT-09-002**

M_w= **6.7**
PGA= **0.32g**

Field Ground Water Depth (ft)= **10.0 ft**
Design Ground Water Depth (ft)= **10.0 ft**

Existing Ground Elevation= **0.0 ft**
Final Ground Elevation= **0.0 ft**

Ana. by: Hamid Parsa
Checked by: Carl Henderson



Note: Factor of safety is capped at 2.0 for liquefiable soils and not plotted for non-liquefiable soils & soils above the ground water; $CRR_{M=7.5,\ \sigma v'=\ 1tsf}$ is capped at 1.0 for liqeufiable soils and not plotted for non-liquefiable soils and soils above the ground water.

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2001 Arch Airport Road, #100
Stockton, California 95206
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Project Name: Crosstown Freeway
Project No.: 111904.G01
Project Location: Stockton, CA

LIQUEFACTION ANALYSIS

Date 7/13/2011

APPENDIX G
SOIL PROFILE MODELS FOR PILE ANALYSES

LIST OF ATTACHMENTS

The following are attached and complete this appendix.

Undrained Shear Strength Assessment for Axial Pile Capacity Analysis
Generalized Soil Profile and Parameters for Axial and Lateral Pile Capacity Analysis

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Undrained Shear Strength Assessment for Axial Pile Capacity Analysis

MODEL #1

Undrained Shear Strength Assessment

Performed by: HP Date: 4/14/2011

Boring No. : **A-09-001** Checked by: CH Date: 4/15/2011

Depth (feet)	Blow Counts/ft	Undrained Shear Strength (psf) Based on			
		*Pocket Penetrometer data	Unconfined Compression Test	CPT-11-001 Data	CPT-11-002 Data
1.5	NT	NT	NT	3,600	N/A
5	11	NT	NT	2,100	2,300
10	14	NT	**1,100**	4,900	3,500
15	11	NT	NT	1,700	3,700
20	20	NT	**600**	2,400	2,100
25	50/6"	NT			4,500
30	10	NT			
35	11	NT	NT	2,200	2,800
40	16	NT	NT	2,600	6,400
45	13	NT	**800**	2,600	3,400
50	36	NT	NT	3,100	2,000
55	16	NT	NT	4,400	1,900
60	63/6"	NT	NT		1,600
65	48	NT	NT	4,300	5,400
70	46	NT	NT	8,000	>8,000
80	24	NT	NT	4,800	3,800
90	50	NT	NT	2,360	4,500
100	77	NT	NT		N/T
110	41	NT	NT	N/T	N/T
120	48	NT	NT	N/T	N/T

NT = Not Tested

= Sand

* Highest reading on pocket penetrometer for unconfined compression strength is 9,000 psf (4.5 tsf). Strength values are actually higher in most cases below 60 feet depth.

MODEL #2

Undrained Shear Strength Assessment

Performed by: HP Date: 4/14/2011

Boring No. : **R-11-001** Checked by: CH Date: 4/15/2011

Depth (feet)	Blow Counts/ft	Undrained Shear Strength (psf) Based on			
		*Pocket Penetrometer data	Unconfined Compression Test	CPT-11-002 Data	CPT-11-010 Data
1	25	4,000	NT	N/A	5,100
5	20	4,500	NT	2,200	3,500
10	27	4,500	NT	4,200	6,500
15	16	3,000	**2,540**	2,800	2,400
20	20	4,500	NT	2,500	3,000
25	18	4,500	NT		3,300
30	37	NT	NT		1,800
35	28	NT	NT	2,800	
40	38	NT	NT	6,400	
45	29	NT	NT	3,400	3,900
50	30	NT	NT	2,000	
55	40	NT	NT	1,900	
60	28	NT	NT	3,000	
65	27	NT	NT	5,400	6,300
70	43	4,500	NT	>8,000	5,500
75	46	4,500	NT	6,300	8,000
80	57	4,500	**1,230**	3,800	
85	56	4,500	NT	2,900	4,000
90	53	4,500	NT	4,500	>8,000

NT= Not Tested

= Sand

* Highest reading on pocket penetrometer for unconfined compression strength is 9,000 psf (4.5 tsf). Strength values are actually higher in most cases below 60 feet depth.

MODEL #3
Undrained Shear Strength Assessment

Prepared By: HP Date: 4/14/2011

Boring No. : **R-11-002** Checked By: CH Date: 4/15/2011

Depth (feet)	Blow Counts/ft	Undrained Shear Strength (psf) Based on			
		*Pocket Penetrometer data	Unconfined Compression Test	CPT-11-002 Data	CPT-09-001 Data
2	17	4,000	NT	2,000	N/A
5	27	3,500	NT	2,500	2,500
10	11	3,000	NT	2,400	4,900
15	17	4,500	NT	2,800	1,600
20	26	4,500	NT	3,000	3,800
25	36	4,000	NT	3,200	2,300
30	37	3,000	**720**	2,500	2,300
35	23	NT	NT	2,800	
40	22	NT	NT	6,400	
45	49	NT	NT	3,400	
50	46	NT	NT	2,000	
55	42	NT	NT	1,900	4,400
60	42	4,500	NT	3,000	4,300
65	24	4,000	NT	5,400	7,600
70	48	4,500	NT	>8,000	5,800
80	57	4,500	NT	3,800	
90	63	4,500	**2,350**	4,500	6,400

NT= Not Tested

= Sand

* Highest reading on pocket penetrometer for unconfined compression strength is 9,000 psf (4.5 tsf). Strength values are actually higher in most cases below 60 feet depth.

MODEL #4

Undrained Shear Strength Assessment

Prepared By: HP Date: 4/14/2011

Boring No. : **R-11-003** Checked By: CH Date: 4/15/2011

Depth (feet)	Blow Counts/ft	Undrained Shear Strength (psf) Based on			
		*Pocket Penetrometer data	Unconfined Compression Test	CPT-09-001 Data	CPT-09-002 Data
1.5	16	3,000	NT	N/A	N/A
5	20	3,000	NT	2,500	4,300
10	14	2,500	**1,170**	4,900	5,200
15	28	3,000	NT	1,600	1,800
20	35	3,500	NT	3,800	3,800
25	20	4000 (SC)	NT	2,300	
30	30	NT	NT	2,300	
35	58	NT	NT		3,150
40	26	4,000	NT		2,500
45	49	4,500	NT		2,700
50	49	4,500	**2,380**		2,800
55	55	4,500	NT	4,400	3,500
60	48	4,500	NT	4,300	>8,000
65	31	NT	NT	7,600	6,800
70	48	NT	NT	5,800	6,300
75	41	NT	NT		
80	60	4,500	NT		
90	45	NT	NT	6,400	
100	57	4,500	NT	4,500	>8,000
110	42	4,500	NT	>8,000	6,200
120	41	4,500	**2,330**	6,800	
130	57	NT	NT	NT	NT
140	67	4,500	**3,100**	NT	NT
150	68	4,500	NT	NT	NT

NT= Not Tested

= Sand

SC = Clayey Sand

* Highest reading on pocket penetrometer for unconfined compression strength is 9,000 psf (4.5 tsf). Strength values are actually higher in most cases below 60 feet depth.

MODEL #5

Undrained Shear Strength Assessment

Prepared By: HP Date: 4/14/2011

Boring No. : **R-09-002** Checked By: CH Date: 4/15/2011

Depth (feet)	Blow Counts/ft	Undrained Shear Strength (psf) Based on			
		*Pocket Penetrometer data	Unconfined Compression Test	CPT-09-001 Data	CPT-09-002 Data
1.5	NT	NT	N/A	N/A	N/A
5	17	NT	NT	2,500	4,300
10	2	NT	NT	4,900	5,200
15	5	NT	NT	1,600	1,800
20	33	NT	NT	3,800	3,800
25	NT	NT	NT	2,300	
30	27	NT	NT	2,300	
35	20	NT	NT		3,150
40	20	NT	NT		2,500
45	36	NT	NT		2,700
50	16	NT	NT		2,800
55	32	NT	NT	4,400	3,500
60	NT	NT	NT	4,300	>8,000
65	26	NT	NT	7,600	6,800
70	36	NT	NT	5,800	6,300
80	29	NT	**2,300**		
90	50/6"	NT	NT	6,400	
100	25	NT	NT	4,500	>8,000
110	80	NT	NT	>8,000	6,200
120	74	NT	NT	6,800	

NT= Not Tested

= Sand

* Highest reading on pocket penetrometer for unconfined compression strength is 9,000 psf (4.5 tsf). Strength values are actually higher in most cases below 60 feet depth.

MODEL #6

Undrained Shear Strength Assessment

Boring No. : **R-11-011**

Prepared By: HP Date: 4/14/2011

Checked By: CH Date: 4/15/2011

Depth (feet)	Blow Counts/ft	Undrained Shear Strength (psf) Based on		
		*Pocket Penetrometer data	Unconfined Compression Test	CPT-09-002 Data
1	11	4,500	NT	N/A
5	20	4,000	NT	4,300
10	19	NT	NT	5,200
15	22	NT	NT	1,800
20	28	2,000	NT	3,800
25	31	1,000	NT	
30	30	4,000	NT	
35	11	1,500	**460**	3,150
40	18	3,000	NT	2,500
45	17	2,000	NT	2,700
50	17	NT	NT	2,800
55	19	4,500	NT	3,500
60	32	NT	NT	>8,000
65	41	NT	NT	6,800
70	30	4,500	**1570**	6,300
75	44	4,500	NT	
80	30	NT	NT	
85	23	4,500	NT	
90	37	4,500	NT	
100	48	4,500	NT	>8,000
110	57	4,500	NT	6,200
120	59	4,500	**5,250**	
130	59	4,500	**3,920**	NT
140	63	4,500	NT	NT
150	65	4,500	NT	NT

NT= Not Tested

= Sand

* Highest reading on pocket penetrometer for unconfined compression strength is 9,000 psf (4.5 tsf). Strength values are actually higher in most cases below 60 feet depth.

MODEL #7

Undrained Shear Strength Assessment

Prepared By: HP Date: 4/14/2011

Boring No. : **R-11-004** Checked By: CH Date: 4/15/2011

Depth (feet)	Blow Counts/ft	Undrained Shear Strength (psf) Based on			
		*Pocket Penetrometer data	Unconfined Compression Test	CPT-11-003 data	CPT-09-002 Data
1	17	4,500	NT	2,600	N/A
5	24	4,500	NT	3,300	4,300
10	13	2,000	**830**	1,700	5,200
15	9	NT	NT	2,200	1,800
20	23	3,000	NT	3,300	3,800
25	33	NT	NT		
30	18	4,000	NT	4,900	
35	20	4,500	**2,550**	2,300	3,150
40	16	NT	NT	3,500	2,500
45	50	NT	NT		2,700
50	27	4,500	NT	3,000	2,800
55	36	4,500	NT	2,800	3,500
60	38	4,500	NT	6,000	>8,000
65	27	4,500	**630**	4,600	6,800
70	Shelby	3,500	NT	4,000	6,300
75	40	4,500	NT	7,000	
80	32	4,500	NT	3,000	
85	42	NT	NT		
90	52	NT	NT		

NT= Not Tested

= Sand

* Highest reading on pocket penetrometer for unconfined compression strength is 9,000 psf (4.5 tsf). Strength values are actually higher in most cases below 60 feet depth.

MODEL #8

Undrained Shear Strength Assessment

Prepared By: HP Date: 4/15/2011

Boring No. : **R-11-005** Checked By: CH Date: 4/15/2011

Depth (feet)	Blow Counts/ft	Undrained Shear Strength (psf) Based on			
		*Pocket Penetrometer data	Unconfined Compression Test	CPT-11-003 Data	CPT-11-004 Data
2	10	4,000	NT	2,600	1,700
5	16	4,500	NT	3,300	2,500
10	14	3,000	NT	1,700	3,400
15	26	4,000	NT	2,200	2,100
20	14	3,000	**1,140**	3,300	2,500
25	42	NT	NT		
30	44	4,500	NT	4,900	3,200
35	35	4,500	NT	2,300	2,800
40	22	4,500	NT	3,500	3,700
45	24	4,500	NT		3,200
50	24	4,500	NT	3,000	7,000
55	25	NT	NT	2,800	2,000
60	19	2,500	**1,090**	6,000	5,100
65	30	NT	NT	4,600	4,300
70	35	NT	NT	4,000	
75	60	NT	NT		
80	Shelby	NT	NT	3,000	3,600
85	59	NT	NT		
90	64	NT	NT		

NT= Not Tested

= Sand

* Highest reading on pocket penetrometer for unconfined compression strength is 9,000 psf (4.5 tsf). Strength values are actually higher in most cases below 60 feet depth.

MODEL #9

Undrained Shear Strength Assessment

Prepared By: HP Date: 4/15/2011

Boring No. : **R-11-006** Checked By: CH Date: 4/15/2011

Depth (feet)	Blow Counts/ft	Undrained Shear Strength (psf) Based on			
		*Pocket Penetrometer data	Unconfined Compression Test	CPT-11-004 Data	CPT-11-005 Data
2	11	4,500	NT	1,700	3,900
5	20	4,000	NT	2,500	3,500
10	11	3,000	NT	3,400	4,500
15	18	4,000	**1,700**	2,100	1,700
20	13	4,000	NT	2,500	1,700
25	Shelby	NT	NT		
30	18	4,000	NT	3,200	2,700
35	25	NT	NT	2,800	3,600
40	22	4,000	NT	3,700	4,300
45	18	NT	NT	3,200	
50	16	4,000	**1,240**	4,000	3,700
55	25	NT	NT	2,000	4,200
60	24	NT	NT	5,100	6,000
65	35	NT	NT	4,300	5,400
70	25	4,000	NT		4,800
75	34	4,500	NT		3,600
80	22	4,500	**1,610**	3,600	2,600
85	28	4,500	NT		5,600
90	33	4,500	NT		> 8,000

NT= Not Tested

= Sand

* Highest reading on pocket penetrometer for unconfined compression strength is 9,000 psf (4.5 tsf). Strength values are actually higher in most cases below 60 feet depth.

MODEL #10

Undrained Shear Strength Assessment

Prepared By: HP Date: 4/21/2011

Boring No. : **A-09-003** Checked By: CH Date: 4/21/2011

Depth (feet)	Blow Counts/ft	Undrained Shear Strength (psf) Based on			
		*Pocket Penetrometer data	Unconfined Compression Test	CPT-11-004 Data	CPT-11-005 Data
2	NT	NT	NT	1,700	3,900
5	22	NT	**3,400**	2,500	3,500
10	28	NT	NT	3,400	4,500
15	19	NT	NT	2,100	1,700
20	25	NT	**2,700**	2,500	1,700
25	28	NT	NT		
30	17	NT	NT	3,200	2,700
35	22	NT	**2,100**	2,800	3,600
40	27	NT	NT	3,700	4,300
45	17	NT	NT	3,200	Sand
50	35	NT	NT	4,000	3,700
55	22	NT	NT	2,000	4,200
60	28	NT	NT	5,100	6,000

NT= Not Tested

= Sand

* Highest reading on pocket penetrometer for unconfined compression strength is 9,000 psf (4.5 tsf). Strength values are actually higher in most cases below 60 feet depth.

MODEL #11

Undrained Shear Strength Assessment

Prepared By: HP Date: 4/15/2011

Boring No. : **R-11-007** Checked By: CH Date: 4/15/2011

Depth (feet)	Blow Counts/ft	Undrained Shear Strength (psf) Based on			
		*Pocket Penetrometer data	Unconfined Compression Test	CPT-11-005 Data	CPT-11-006 Data
2	11	4,500	NT	3,900	1,600
5	36	4,000	NT	3,500	4,600
10	35	NT	NT	4,500	4,600
15	Shelby	4,000	**410**	1,700	3,600
20	54	4,500	NT	1,700	3,600
25	19	4,000	NT		3,100
30	87	NT	NT	2,700	2,500
35	38	4,000	NT	3,600	1,500
40	34	NT	NT	4,300	1,700
45	37	4,000	**1,510**		1,800
50	29	4,000	NT	3,700	2,400
55	34	4,000	NT	4,200	7,000
60	30	4,000	NT	6,000	3,000
65	35	4,500	NT	5,400	4,700
70	54	4,000	**2,780**	4,800	3,100
75	36	NT	NT	3,600	3,000
80	53	4,500	NT	2,600	5,800
85	44	4,500	NT	5,600	
90	55	4,000	NT	> 8,000	7,400

NT= Not Tested

= Sand

* Highest reading on pocket penetrometer for unconfined compression strength is 9,000 psf (4.5 tsf). Strength values are actually higher in most cases below 60 feet depth.

MODEL #12

Undrained Shear Strength Assessment

Prepared By: HP Date: 4/15/2011

Boring No. : **R-11-008** Checked By: CH Date: 4/15/2011

Depth (feet)	Blow Counts/ft	Undrained Shear Strength (psf) Based on			
		*Pocket Penetrometer data	Unconfined Compression Test	CPT-11-006 Data (psf)	CPT-11-007 Data (psf)
3	7	3,500	NT	1,600	1,700
5	19	4,000	NT	4,600	2,700
10	26	NT	NT	4,600	3,200
15	41	NT	NT	3,600	6,100
20	9	3,000	**1,300**	3,600	
25	23	3,000	NT	3,100	
30	18	4,000	NT	2,500	
35	18	3,000	NT	1,500	3,700
40	Shelby	3,000	NT	1,700	2,400
45	18	NT	NT	1,800	4,300
50	28	NT	NT	2,400	4,300
55	28	4,000	NT	7,000	6,200
60	30	NT	NT	3,000	7,100
65	47	NT	NT	4,700	6,200
70	47	4,500	NT	3,100	7,200
75	68	NT	NT	3,000	5,800
80	61	4,500	NT	5,800	5,900
85	53	4,000	**6,250**		5,800
90	68	4,000	NT	7,400	7,500

NT= Not Tested

= Sand

* Highest reading on pocket penetrometer for unconfined compression strength is 9,000 psf (4.5 tsf). Strength values are actually higher in most cases below 60 feet depth.

MODEL #13

Undrained Shear Strength Assessment

Prepared By: HP Date: 4/15/2011

Boring No. : **R-11-009** Checked By: CH Date: 4/15/2011

Depth (feet)	Blow Counts/ft	Undrained Shear Strength (psf) Based on			
		*Pocket Penetrometer data	Unconfined Compression Test	CPT-11-008 Data	CPT-11-007 Data
3	NT	NT	NT	1,300	1,700
5	9	3,000	NT	2,000	2,700
10	10	2,000	NT	4,200	3,200
15	8	2,500	**850**	2,600	6,100
20	16	4,000	NT	4,400	
25	28	NT	NT	3,200	
30	33	3,500	NT	4,500	
35	21	2,750	NT	3,400	3,700
40	22	2,750	NT		2,400
45	49	NT	NT		4,300
50	29	4,500	NT	3,000	4,300
55	45	NT	NT		6,200
60	57	NT	NT	8,000	7,100
65	25	NT	NT	5,300	6,200
70	20	NT	NT		7,200
75	56	NT	NT		5,800
80	55	4,000	NT	5,700	5,900
85	22	4,000	NT	6,000	5,800
90	42	4,000	**4,850**	5,500	7,500
95	47	4,000	NT	NT	NT
100	40	4,000	NT	NT	NT
105	47	4,000	NT	NT	NT
110	49	4,000	NT	NT	NT
115	56	4,000	NT	NT	NT
120	58	NT	NT	NT	NT

NT= Not Tested

= Sand

* Highest reading on pocket penetrometer for unconfined compression strength is 9,000 psf (4.5 tsf). Strength values are actually higher in most cases below 60 feet depth.

MODEL #14

Undrained Shear Strength Assessment

Prepared By: HP Date: 4/15/2011

Boring No. : **R-11-010** Checked By: CH Date: 4/15/2011

Depth (feet)	Blow Counts/ft	Undrained Shear Strength (psf) Based on			
		*Pocket Penetrometer data	Unconfined Compression Test	CPT-11-009 Data	
3	NT	NT	NT		
5	15	NT	NT		
10	22	4,000	NT	3,300	
15	5	NT	NT	2,000	
20	21	3,500	NT	1,600	
25	12	NT	**2,000**	6,000	
30	21	NT	NT	3,000	
35	24	4,000	NT	4,400	
40	21	4,500	NT		
45	30	NT	NT		
50	21	NT	NT		

NT= Not Tested

= Sand

* Highest reading on pocket penetrometer for unconfined compression strength is 9,000 psf (4.5 tsf). Strength values are actually higher in most cases below 60 feet depth.

Generalized Soil Profile and Parameters for Axial and Lateral Pile Capacity Analysis

CROSSTOWN FREEWAY SR-4 IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
MODEL #1

Calcs. By: HP Date: **4/27/2011**
Reviewed By: CFH Date: 4/27/2011

Idealized Layer	Approx. Depth (ft)	Generalized Vertical Soil Profile	Undrained Shear Strength (psf)	Friction Angle (degrees)	Soil Type	Cyclic Soil Modulus Parameter k_c (pci)	Static Soil Modulus Parameter k_s (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight (pcf)	Remark
1	0 - 8	Sandy Lean Clay (stiff to very stiff)	2,000	N/A	3, Stiff Clay without Free Water	300	750	0.005	120	
2	8 - 25	Lean Clay, Silt (stiff to very stiff)	2,000	N/A	3, Stiff Clay without Free Water	300	750	0.005	58	
3	25-35	Sand (medium dense to dense)	N/A	33	4, Sand below water	--	90	N/A	62	
4	35 - 65	Lean Clay with Sand (stiff to very stiff)	3,000	N/A	3, Stiff clay without free water	400	1000	0.005	63	
5	65-75	Silty Sand (medium dense to dense)	N/A	38	4, Sand below water	--	125	N/A	63	
6	75-95	Silt (stiff to very stiff)	5,000	N/A	3, Stiff clay without free water	600	1500	0.004	63	
7	95-105	Silty Sand (medium dense to dense)	N/A	38	4, Sand below water	--	125	N/A	63	
8	105-120	Lean Clay (stiff to very stiff)	6,000	N/A	3, Stiff clay without free water	800	2000	0.004	63	IGM Cohesive

CROSSTOWN FREEWAY SR-4 IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
MODEL #2

Calcs. By: HP Date: **4/27/2011**
Reviewed By: CFH Date: 4/27/2011

Idealized Layer	Approx. Depth (ft)	Generalized Vertical Soil Profile	Undrained Shear Strength (psf)	Friction Angle (degrees)	Soil Type	Cyclic Soil Modulus Parameter k_c (pci)	Static Soil Modulus Parameter k_s (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight (pcf)	Remark
1	0 -8	Lean Clay, Fat Clay (stiff to very stiff)	3,000	N/A	3, Stiff Clay without Free Water	400	1000	0.005	120	
2	8-30	Lean Clay, Fat Clay (stiff to very stiff)	3,000	N/A	3, Stiff Clay without free water	400	1,000	0.005	63	
3	30 -70	Sand, Silty Sand, Clayey Sand (medium dense to dense)	N/A	38	4, Sand below water	--	125	N/A	63	
4	70 - 90	Lean Clay (stiff to very stiff)	5,000	N/A	3, Stiff Clay without free water	800	2,000	0.004	63	IGM Cohesive
5	90-100	Silty Sand (dense)	N/A	38	4, Sand below water	--	125	N/A	63	
6	100 - 120	Lean Clay, Sandy Silt (stiff to very stiff)	6,000	N/A	3, Stiff Clay without Free Water	800	2,000	0.004	63	IGM Cohesive

CROSSTOWN FREEWAY SR-4 IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
MODEL #3

Calcs. By: HP Date: **4/28/2011**
Reviewed By: CFH Date: 4/28/2011

Idealized Layer	Approx. Depth (ft)	Generalized Vertical Soil Profile	Undrained Shear Strength (psf)	Friction Angle (degrees)	Soil Type	Cyclic Soil Modulus Parameter k_c (pci)	Static Soil Modulus Parameter k_s (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight (pcf)	Remark
1	0 -8	Lean Clay (stiff to very stiff)	2,500	N/A	3, Stiff Clay without Free Water	300	750	0.005	120	
2	8-35	Lean Clay (stiff to very stiff)	3,000	N/A	3, Stiff Clay without free water	400	1,000	0.005	63	
3	35 -60	Sand, Clayey Sand (medium dense to dense)	N/A	36	4, Sand below water	--	125	N/A	63	
4	60 - 90	Lean Clay, Sandy Silt (stiff to very stiff)	5,000	N/A	3, Stiff Clay without free water	800	2,000	0.004	63	IGM Cohesive
5	90-100	Silty Sand (dense)	N/A	38	4, Sand below water	--	125	N/A	63	
6	100 - 120	Lean Clay, Sandy Silt (stiff to very stiff)	6,000	N/A	3, Stiff Clay without Free Water	800	2,000	0.004	63	IGM Cohesive

CROSSTOWN FREEWAY SR-4 IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
MODEL #4

Calcs. By: HP Date: **4/22/2011**
Reviewed By: CFH Date: 4/22/2011

Idealized Layer	Approx. Depth (ft)	Generalized Vertical Soil Profile	Undrained Shear Strength (psf)	Friction Angle (degrees)	Soil Type	Cyclic Soil Modulus Parameter k (pci)	Static Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight (pcf)	Remark
1	0 - 8	Lean Clay, Fat Clay (stiff to very stiff)	2,000	N/A	3, Stiff Clay without Free Water	300	750	0.005	120	
2	8 - 29	Lean Clay, Fat Clay, Sandy Silt (stiff to very stiff)	3,000	N/A	3, Stiff Clay without Free Water	400	1000	0.005	63	
3	29 - 38	Sand, Silty Sand, Clayey Sand (medium dense to dense)	N/A	33	4, Sand below water	--	90	N/A	63	
4	38 - 65	Lean Clay, Sandy Silt (stiff to very stiff)	4,000	N/A	3, Stiff Clay without Free Water	400	1000	0.005	63	
5	65 - 78	Sand, Sand with Silt (medium dense to dense)	N/A	38	4, Sand below water	--	125	N/A	63	
6	78 - 89	Lean Clay (stiff to very stiff)	6,000	N/A	3, Stiff Clay without Free Water	800	2000	0.004	63	IGM Cohesive
7	89 - 99	Sand (dense)	N/A	38	4, Sand below water	--	125	N/A	63	
8	99 - 129	Lean Clay, Sandy Lean Clay (stiff to very stiff)	6,000	N/A	3, Stiff Clay without Free Water	800	2,000	0.004	63	IGM Cohesive
9	129 -139	Silty Sand (dense)	N/A	38	4, Sand below water	--	125	N/A	63	
10	139 - 150	Lean Clay (stiff to very stiff)	6,000	N/A	3, Stiff Clay without Free Water	800	2,000	0.004	63	IGM Cohesive

CROSSTOWN FREEWAY SR-4 IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
MODEL #5

Calcs. By: HP Date: **4/22/2011**
Reviewed By: CFH Date: 4/22/2011

Idealized Layer	Approx. Depth (ft)	Generalized Vertical Soil Profile	Undrained Shear Strength (psf)	Friction Angle (degrees)	Soil Type	Cyclic Soil Modulus Parameter k_c (pci)	Static Soil Modulus Parameter k_s (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight (pcf)	Remark
1	0 - 8	Lean Clay (stiff to very stiff)	1,500	N/A	3, Stiff Clay without Free Water	200	500	0.005	120	
2	8 - 20	Lean Clay, Sandy Silt (soft to stiff)	2,500	N/A	3, Stiff Clay without Free Water	400	1000	0.005	63	Data from R-11-003, CPT-09-001, and CPT-09-002 more representative than A-09-002
3	20- 60	Lean Clay (stiff to very stiff)	3,000	N/A	3, Stiff Clay without Free Water	400	1000	0.005	63	
4	60 - 90	Lean Clay, Sandy Silt (stiff to very stiff)	5,000	N/A	3, Stiff Clay without Free Water	800	2000	0.004	63	IGM Cohesive
5	90 - 150	Lean Clay, Sandy Silt (stiff to very stiff)	6,000	N/A	3, Stiff Clay without Free Water	800	2000	0.004	63	

CROSSTOWN FREEWAY SR-4 IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
MODEL #6

Calcs. By: HP Date: **4/22/2011**
Reviewed By: CFH Date: 4/22/2011

Idealized Layer	Approx. Depth (ft)	Generalized Vertical Soil Profile	Undrained Shear Strength (psf)	Friction Angle (degrees)	Soil Type	Cyclic Soil Modulus Parameter k (pci)	Static Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight (pcf)	Remark
1	0 - 10	Lean Clay, Sandy Silt (stiff to very stiff)	2,000	N/A	3, Stiff Clay without Free Water	300	750	0.005	120	
2	10- 20	Sand, Silty Sand, Clayey Sand (medium dense to dense)	N/A	33	4, Sand below water	--	60	N/A	63	
3	20 - 60	Lean Clay, Sandy Lean Clay (stiff to very stiff)	2,750	N/A	3, Stiff Clay without Free Water	400	1000	0.005	63	
4	60 - 70	Sand, Silty Sand, Clayey Sand (medium dense to dense)	N/A	35	4, Sand below water	--	90	N/A	63	
5	70 - 90	Lean Clay, Sandy Silt (stiff to very stiff)	4,500	N/A	3, Stiff Clay without Free Water	800	2000	N/A	63	
6	90 - 150	Lean Clay (stiff to very stiff)	6,000	N/A	3, Stiff clay without free water	800	2000	N/A	63	IGM Cohesive

CROSSTOWN FREEWAY SR-4 IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
MODEL #7

Calcs. By: HP Date: **4/22/2011**
Reviewed By: CFH Date: 4/27/2011

Idealized Layer	Approx. Depth (ft)	Generalized Vertical Soil Profile	Undrained Shear Strength (psf)	Friction Angle (degrees)	Soil Type	Cyclic Soil Modulus Parameter k (pci)	Static Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight (pcf)	Remark
1	0 - 15	Lean Clay, Fat Clay (stiff to very stiff)	2,000	N/A	3, Stiff Clay without Free Water	300	750	0.005	120	
2	15-23	Lean Clay, Sandy Clay (stiff to very stiff)	2,000	N/A	3, Stiff Clay without Free Water	300	750	0.005	63	
3	23- 29	Sand (medium dense to dense)	N/A	35	4, Sand below water	--	90	N/A	63	
4	29 - 39	Lean Clay (stiff to very stiff)	3,000	N/A	3, Stiff Clay without Free Water	400	1000	0.005	63	
5	39 - 50	Sand (medium dense to dense)	N/A	36	4, Sand below water	--	125	N/A	63	
6	50 - 70	Lean Clay (stiff to very stiff)	4,000	N/A	3, Stiff Clay without Free Water	800	2,000	0.004	63	
7	70 - 90	Lean Clay, Sandy Lean Clay (stiff to very stiff)	5,000	N/A	3, Stiff Clay without Free Water	800	2,000	0.004	63	
8	90-120	Lean Clay, Sandy Silt (stiff to very stiff)	6,000	N/A	3, Stiff Clay without Free Water	800	2,000	0.004	63	IGM Cohesive

CROSSTOWN FREEWAY SR-4 IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
MODEL #8

Calcs. By: HP Date: **4/27/2011**
Reviewed By: CFH Date: **4/27/2011**

Idealized Layer	Approx. Depth (ft)	Generalized Vertical Soil Profile	Undrained Shear Strength (psf)	Friction Angle (degrees)	Soil Type	Cyclic Soil Modulus Parameter k (pci)	Static Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight (pcf)	Remark
1	0 - 15	Lean Clay (stiff to very stiff)	2,500	N/A	3, Stiff Clay without Free Water	300	750	0.005	120	
2	15 - 24	Lean Clay (stiff to very stiff)	2,500	N/A	3, Stiff Clay without Free Water	300	750	0.005	63	
3	24- 29	Sand (dense)	N/A	36	4, Sand below water	--	125	N/A	63	
4	29 - 69	Lean Clay, Sandy Lean Clay (stiff to very stiff)	4,000	N/A	3, Stiff Clay without Free Water	400	1000	0.005	63	
5	69 - 90	Sand (medium dense to very dense)	N/A	38	4, Sand below water	--	125	N/A	63	

CROSSTOWN FREEWAY SR-4 IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
MODEL #9

Calcs. By: HP Date: **4/27/2011**
Reviewed By: CFH Date:

Idealized Layer	Approx. Depth (ft)	Generalized Vertical Soil Profile	Undrained Shear Strength (psf)	Friction Angle (degrees)	Soil Type	Cyclic Soil Modulus Parameter k (pci)	Static Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight (pcf)	Remark
1	0 - 15	Fat Clay, Lean Clay (stiff to very stiff)	2,500	N/A	3, Stiff Clay without Free Water	400	1,000	0.005	120	
2	15 - 25	Fat Clay, Lean Clay (stiff to very stiff)	2,500	N/A	3, Stiff Clay without Free Water	400	1,000	0.005	63	
3	25- 40	Sand, Silty Sand, Clayey Sand (medium dense)	N/A	35	4, Sand below water	--	90	N/A	63	
4	40 - 60	Lean Clay, Sandy Silt (stiff to very stiff)	3,500	N/A	3, Stiff Clay without Free Water	400	1,000	0.005	63	
5	60 - 90	Lean Clay (stiff to very stiff)	5,000	N/A	3, Stiff Clay without Free Water	800	2,000	0.005	63	

CROSSTOWN FREEWAY SR-4 IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
MODEL #10

Calcs. By: HP Date: **4/27/2011**
Reviewed By: CFH Date: 4/27/2011

Idealized Layer	Approx. Depth (ft)	Generalized Vertical Soil Profile	Undrained Shear Strength (psf)	Friction Angle (degrees)	Soil Type	Cyclic Soil Modulus Parameter k (pci)	Static Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight (pcf)	Remark
1	0 - 10	Lean Clay, Silt (stiff to very stiff)	2,000	N/A	3, Stiff Clay without Free Water	300	750	0.005	120	
2	10-25	Lean Clay (stiff to very stiff)	2,000	N/A	3, Stiff Clay without Free Water	300	750	0.005	60	
3	25- 30	Sand, Silty Sand (medium dense)	N/A	33	4, Sand below water	--	90	N/A	63	
4	30 - 45	Lean Clay, Sandy Silt (stiff to very stiff)	3,000	N/A	3, Stiff Clay without Free Water	400	1,000	0.005	63	
5	45 - 60	Sand, Silty Sand (medium dense to dense)	N/A	35	4, Sand below water	--	90	N/A	63	
6	60 - 90	Lean Clay, Sandy Silt (stiff to very stiff)	4,000	N/A	3, Stiff Clay without Free Water	600	1,500	0.004	63	

CROSSTOWN FREEWAY SR-4 IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
MODEL #11

Calcs. By: HP Date: **4/27/2011**
Reviewed By: CFH Date: 4/27/2011

Idealized Layer	Approx. Depth (ft)	Generalized Vertical Soil Profile	Undrained Shear Strength (psf)	Friction Angle (degrees)	Soil Type	Cyclic Soil Modulus Parameter k (pci)	Static Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight (pcf)	Remark
1	0 - 9	Lean Clay, Sandy Silt (stiff to very stiff)	3,000	N/A	3, Stiff Clay without Free Water	300	750	0.005	120	
2	9-15	Sand (medium dense to dense)	N/A	35	4, Sand below water	--	125	N/A	58	
3	15 - 29	Lean Clay (stiff to very stiff)	3,000	N/A	3, Stiff Clay without Free Water	400	1,000	0.005	63	
4	29- 40	Sand, Silty Sand (medium dense to dense)	N/A	38	4, Sand below water	--	125	N/A	63	
5	40 - 55	Lean Clay (stiff to very stiff)	3,500	N/A	3, Stiff Clay without Free Water	400	1,000	0.005	63	
6	55-75	Lean Clay (stiff to very stiff)	4,000	N/A	3, Stiff Clay without Free Water	600	1,500	0.005	63	
7	75 - 90	Lean Clay (stiff to very stiff)	5,000	N/A	3, Stiff Clay without Free Water	800	2,000	0.004	63	

CROSSTOWN FREEWAY SR-4 IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
MODEL #12

Calcs. By: HP Date: **4/27/2011**
Reviewed By: CFH Date: 4/27/2011

Idealized Layer	Approx. Depth (ft)	Generalized Vertical Soil Profile	Undrained Shear Strength (psf)	Friction Angle (degrees)	Soil Type	Cyclic Soil Modulus Parameter k (pci)	Static Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight (pcf)	Remark
1	0 - 14	Lean Clay (stiff to very stiff)	2,500	N/A	3, Stiff Clay without Free Water	300	750	0.005	120	
2	14-20	Sand (dense)	N/A	35	4, Sand below water	--	125	N/A	63	
3	20 - 45	Lean Clay, Fat Clay (stiff to very stiff)	2,500	N/A	3, Stiff Clay without Free Water	300	750	0.005	63	
4	45 - 75	Sand (medium dense to dense)	N/A	38	4, Sand below water	--	125	N/A	63	
5	75 - 90	Lean Clay, Fat Clay (stiff to very stiff)	6,000	N/A	3, Stiff Clay without Free Water	800	2,000	0.004	63	IGM Cohesive

CROSSTOWN FREEWAY SR-4 IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
MODEL #13

Calcs. By: HP Date: **4/27/2011**
Reviewed By: CFH Date: 4/27/2011

Idealized Layer	Approx. Depth (ft)	Generalized Vertical Soil Profile	Undrained Shear Strength (psf)	Friction Angle (degrees)	Soil Type	Cyclic Soil Modulus Parameter k (pci)	Static Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight (pcf)	Remark
1	0-5	Lean Clay (stiff to very stiff)	1,500	N/A	3, Stiff Clay without Free Water	200	500	0.005	120	
2	5-15	Lean Clay, Fat Clay (stiff to very stiff)	2,000	N/A	3, Stiff Clay without Free Water	300	750	0.005	125	
3	15-40	Lean Clay, Sandy Silt (stiff to very stiff)	3,000	N/A	3, Stiff Clay without Free Water	400	1,000	0.005	63	
4	40 - 70	Sand, Silty Sand, Clayey Sand (medium dense to dense)	N/A	36	4, Sand below water	--	125	N/A	63	
5	70 - 80	Sand, Silty Sand, Clayey Sand (medium dense to dense)	N/A	38	4, Sand below water	--	125	N/A	63	
6	80 - 120	Lean Clay, Sandy Silt (stiff to very stiff)	6,000	N/A	3, Stiff Clay without Free Water	800	2,000	0.004	63	IGM Cohesive

CROSSTOWN FREEWAY SR-4 IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
MODEL #14

Calcs. By: HP Date: **4/29/2011**
Reviewed By: CFH Date: 4/29/2011

Idealized Layer	Approx. Depth (ft)	Generalized Vertical Soil Profile	Undrained Shear Strength (psf)	Friction Angle (degrees)	Soil Type	Cyclic Soil Modulus Parameter k (pci)	Static Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight (pcf)	Remark
1	0-20	Silty Sand, Clayey Sand (medium dense to dense)	N/A	33	3, Stiff Clay without Free Water	--	90	N/A	120	
2	20 - 35	Sandy Lean Clay (stiff to very stiff)	2,500	N/A	3, Stiff Clay without Free Water	300	750	0.005	125	
3	35-45	Lean Clay, Sandy Silt (stiff to very stiff)	3,000	N/A	3, Stiff Clay without Free Water	400	1,000	0.005	63	
4	45 - 50	Sand, Silty Sand, Clayey Sand (medium dense to dense)	N/A	36	4, Sand below water	--	90	N/A	63	

CROSSTOWN FREEWAY SR-4 IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
MODEL #14

Calcs. By: HP Date: **4/29/2011**
Reviewed By: CFH Date: 4/29/2011

Idealized Layer	Approx. Depth (ft)	Generalized Vertical Soil Profile	Undrained Shear Strength (psf)	Friction Angle (degrees)	Soil Type	Cyclic Soil Modulus Parameter k (pci)	Static Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight (pcf)	Remark
1	0-20	Silty Sand, Clayey Sand (medium dense to dense)	N/A	33	3, Stiff Clay without Free Water	--	90	N/A	120	
2	20 - 35	Sandy Lean Clay (stiff to very stiff)	2,500	N/A	3, Stiff Clay without Free Water	300	750	0.005	125	
3	35-45	Lean Clay, Sandy Silt (stiff to very stiff)	3,000	N/A	3, Stiff Clay without Free Water	400	1,000	0.005	63	
4	45 - 50	Sand, Silty Sand, Clayey Sand (medium dense to dense)	N/A	36	4, Sand below water	--	90	N/A	63	

KLEINFELDER
Bright People. Right Solutions.

APPENDIX H
AXIAL CAPACITY ANALYSIS RESULTS FOR CIDH PILES

LIST OF ATTACHMENTS

The following are attached and complete this appendix.

Letter Addressing Historical Axial Pile Capacity Assessments in Stockton Area
Axial Pile Capacity Analysis – 6 foot diameter shafts
Axial Pile Capacity Analysis – 8 foot diameter shafts
Axial Pile Capacity Analysis – 9 foot diameter shafts
Axial Pile Capacity Curves – 6 foot diameter shafts
Axial Pile Capacity Curves – 8 foot diameter shafts
Axial Pile Capacity Curves – 9 foot diameter shafts

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Letter Addressing Historical Axial Pile Capacity Assessments in Stockton Area

KLEINFELDER
Bright People. Right Solutions.

File No. 20-1147-71.M01
August 30, 1999

Mr. Wayne Smith
City of Stockton
Public Works Department
425 North El Dorado Street
Stockton, California 95202

Subject: **COMMENTS REGARDING PILE DESIGN**
ESSENTIAL SERVICES BUILDING
STOCKTON, CALIFORNIA

Dear Wayne:

On August 26, 1999, Mr. Richard Toy from our firm had an opportunity to review the plans, specifications, and geotechnical report for the Essential Services Building in Stockton, California for the purpose of preparing an estimate for the materials testing services on the project. He noted that very long piles, on the order of 80 feet in length, were specified for the project and that the design friction values were apparently 500 pounds per square foot (psf) to a depth of 35 feet and 800 psf below that level. Understanding that the project was design/build, I contacted Harry Smith of Turner Construction on the same day to obtain more information regarding the actual foundation design, and to suggest that modifications to the pile lengths and/or design could provide significant cost savings. Mr. Smith indicated that he would discuss this opportunity with City staff. On August 27, 1999, you called and arranged for a conference call between City staff, Kleinfelder, Inc., and Geomatrix. Following two conference calls that day, Kleinfelder was authorized to summarize our experience with deep foundations in the downtown Stockton area.

As you are aware, Kleinfelder began in Stockton in 1961, and has completed virtually all of the major building projects in downtown Stockton since that time. For either drilled or driven pile foundations our firm has typically used friction values in the range of 1,000 to 1,500 psf. On many projects, depth versus capacity curves were furnished which used lower friction values in the upper elevations and higher friction values in the lower elevations. The majority of the drilled or driven pile foundations in downtown Stockton have lengths typically in the 30 to 40 foot range. In addition, in the past approximate 20 years, deep foundations in this area have consisted primarily of drilled cast-in-place piers versus driven piles. Our firm has records of 12 pile load tests performed in downtown Stockton and in the Port of Stockton, the majority of which have been performed on driven pile foundations. Pile load tests in the downtown area were performed at the Pacific Telephone and Telegraph Company (PacTel) at the corner of Lindsay and Hunter Streets in 1966, the San Joaquin First Federal Building (now Pacific States Bank building) in 1973, and the San Joaquin County Courthouse Annex Building (now the McCanlis building). The PacTel project is located several blocks from the Essential Services building. Pile foundations consisted of Raymond Step Taper piles and were tested to near refusal. All piles had an 11-3/8 inch tip diameter and a 16-3/8 inch butt diameter. The pile length, and ultimate load information is presented below.

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2001 Arch-Airport Road, Suite 100, Stockton, CA 95206 **p** | 209.948.1345 **f** | 209.234.4700

Test Pile No.	Length (feet)	Ultimate Load, (tons)
A	67	375
B	59	296
C	68	260
D	67	254

Our calculations indicate that the skin friction values at this location were on the order of 2,800 to 3,000 psf.

Three pile load tests were performed for the San Joaquin First Federal Building, which is closest (one block) to the project. We have included a copy of these pile tests for your information. The piles also consisted of 12-inch square pre-stressed, pre-cast concrete piles. The pile lengths for tests 1, 2, and 3 were 42 feet, 49 feet, and 53 feet, respectively. As the attached report indicates on Page 2, the test pile results for tests 1 and 2 were still in the elastic range at the maximum test loads of 200 and 187.5 tons, respectively. Test pile number 3 was on the verge of failure at a load of 342 tons. Test piles 1 and 2, indicated minimal settlement with friction values on the order of 2,000 to 2,500 psf. Pile test 3 was near failure at an average friction value 2,500 psf. For your information, we have also attached a copy of the boring logs for this project. Please note that the logs were originally presented on a larger sheet and for convenience have been copied to 8½ x 11 size. In our opinion, the blow count information shown on the logs is comparable or even lower than the blow count information from the two logs prepared by Geomatrix.

We understand that Geomatrix will remain the geotechnical engineer of record on this project. We also understand that a series of 12 indicator piles will be driven after which the actual length of production piles will be furnished to the contractor, Turner Construction. With the indicator pile program still several weeks away, we suggest that modifications be considered in the pile design for this project. Based on our experience and the review of the pile load test information by our firm, we suggest a design friction value of 1,000 psf be used below the level of debris to a depth of approximately 35 feet. Below that depth, a design friction value of 1,500 psf can be considered. In our opinion, these parameters would provide a factor of safety of at least 2. We also suggest that the structural engineer be contacted to evaluate what factor of safety, if any, was applied to the design loads for the project. In other words, design loads often include loads or forces mandated by codes and other factors, and the actual dead plus live applied load on the pile foundation on projects are often considerably less than the design loads furnished to the geotechnical engineer. The net effect may be that the actual factor of safety is much higher than 2.

In summary, it is our opinion that modifications to the current pile design could result in a decrease of approximately one-half the pile length. There would be no need to change other portions of the pile foundation design, such as pile cap, tie beams, etc., Therefore, there should be no need for a design review or any other loss or delay in construction. Reducing the pile lengths would also require less installation time. In our opinion, additional cost savings could be realized if drilled cast-in-place piers would be considered for this project. This would, however, require modifications to the pile cap design as well as configuration of the pile layout, since drill shafts would be larger diameter than pre-cast driven piles.

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We trust this furnishes the information discussed during our conference calls on Friday August 27, 1999. If needed, the results of the other pile load tests referenced in this letter can be furnished, however, there are more distant or use a different pile type, such as the Courthouse Annex project which used cast-in-place piers. If you have any questions or need further information, please contact us.

Respectfully submitted,

KLEINFELDER, INC.

Ron Heinzen, G.E.
Regional Manager/Senior Principal

RH:md
Attachments

cc: Geomatrix

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Axial Pile Capacity Analysis – 6 foot diameter shafts

Model #1
Project Name Crosstwon Freeway SR-4
Project No. 111904.G01
Subject Preliminary Estimation for Embedment Length for Drilled Shafts
Prepared By HP
Checked By CFH
Date 5/4/2011

Reference:
FHWA, 1999. Drilled Shafts: Construction Procedures and Design Methods. FHWA report No. FHWA-IF-99-025

6 Shaft Diameter (D), ft
18.84 Shaft Perimeter (P), ft
28.27 Cross Sectional Area (A), ft2

End Bearing Capacity (FHWA, 2010, page B-2)

SPT N value	Embedment Length, ft	End Bearing Factor	Unit End Bearing, ksf	Nominal End Bearing Capacity, kips	Resistance Factor for End Bearing	Factored End Bearing Capacity, kips
N		F	q = F*N	Qn=q*A	q1	Q=(q1)*Qn
60		0.92	55.2	1,561	0.5	780

Skin Friction Capacity ((FHWA, 2010, page B-2)

Layer No.	Undrained Shear Strength, psf	Required Embedment Length, ft	Skin Friction Factor	Unit Skin Friction, ksf	Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	Factored Total Resistance capacity, kips
	Su	L	Alpha	fs = (Alpha) Su	Rn=P*L* fs	F1	F2	R1 = F1*F2*Rn	
1	2,000	8	0.55	1.1	62	0.70	1.00	44	
2	2,000	17	0.55	1.1	352	0.70	1.00	247	
4	3,000	30	0.55	1.7	933	0.70	1.00	653	
6	5,000	20	0.46	2.3	867	0.70	1.00	607	
				SUM	2,214			1,550	
COHESIVE IGM (FHWA, 2010, page B-3)									
Layer No.	SPT N Value	Length	Skin Friction Factor	Unit Skin Friction, ksf	Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	
	N	L	F3	fs = N*F3	Rn=P*L*fs	F4	F5	R2=F4*F5*Rn	
8	55	15	0.075	4.1	1,166	0.70	1.00	816	
TOTAL		15		SUM	1,166			816	2,539

Total Skin Friction for cohesive 2,513 kips

Skin Friction Capacity-Cohesionless Soil

Layer No.	phi (degrees)	Required Embedment Length, ft	SPT N Value	Effective Stress (ksf)	m	σ'p	OCR	Ko	Skin Friction Factor	Nominal Skin Friction Capacity, kips	Side Resistance Factor	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	Factored Total Resistance capacity, kips
									Betta	Rn=P*L*fs	F1	F2	R1 = F1*F2*Rn	
3	33	10	30	2.3	0.8	15.11	6.68	1.28	0.83	354	0.70	1.00	248	
5	41	10	50	4.8	0.7	15.38	3.22	0.74	0.64	580	0.70	1.00	406	
7	41	10	60	6.7	0.7	17.47	2.62	0.65	0.56	707	0.70	1.00	495	
				SUM						1,641			1,149	
														4,294
							SUM			5,020			3,514	

Depth 120

CIDH Embedmen	Nominal	Factored
10	104	73
15	207	145
20	311	218
25	414	290
30	592	414
35	769	538
40	924	647
45	1,080	756
50	1,235	865
55	1,391	973
60	1,546	1,082
65	1,702	1,191
70	1,991	1,394
75	2,281	1,597
80	2,498	1,748
85	2,714	1,900
90	2,931	2,052
95	3,148	2,203
100	3,501	2,451
105	3,854	2,698
110	4,243	2,970
115	4,632	3,242
120	5,020	3,514

Model #2
Project Name Crosstwon Freeway SR-4
Project No. 111904.G01
Subject Preliminary Estimation for Embedment Length for Drilled Shafts
Prepared By HP
Checked By CFH
Date 5/4/2011

Reference:
FHWA, 1999. Drilled Shafts: Construction Procedures and Design Methods. FHWA report No. FHWA-IF-99-025

6 Shaft Diameter (D), ft
18.84 Shaft Perimeter (P), ft
28.27 Cross Sectional Area (A), ft2

End Bearing Capacity (FHWA, 2010, page B-2)

SPT N value	Embedment Length, ft	End Bearing Factor	Unit End Bearing, ksf	Nominal End Bearing Capacity, kips	Resistance Factor for End Bearing	Factored End Bearing Capacity, kips
N		F	q = F*N	Qn=q*A	q1	Q=(q1)*Qn
60		0.92	55.2	1,561	0.5	780

Skin Friction Capacity ((FHWA, 2010, page B-2)

Layer No.	Undrained Shear Strength, psf	Required Embedment Length, ft	Skin Friction Factor	Unit Skin Friction, ksf	Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	Factored Total Resistance capacity, kips
	Su	L	Alpha	fs = (Alpha) Su	Rn=P*L* fs	F1	F2	R1 = F1*F2*Rn	
1	3,000	8	0.55	1.7	93	0.70	1.00	65	
2	3,000	22	0.55	1.7	684	0.70	1.00	479	
				SUM	777			544	

COHESIVE IGM (FHWA, 2010, page B-3)

Layer No.	SPT N Value	Length	Skin Friction Factor	Unit Skin Friction, ksf	Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	
	N	L	F3	fs = N*F3	Rn=P*L*fs	F4	F5	R2=F4*F5*Rn	
4	55	20	0.075	4.1	1,554	0.70	1.00	1,088	
6	60	20	0.075	4.5	1,696	0.70	1.00	1,187	
TOTAL		40		SUM	3,250			2,275	2,412

Total Skin Friction for cohesive 2,331 kips

Skin Friction Capacity-Cohesionless Soil(FHWA, 2010, Section 13.3.5.1)

Layer No.	phi (degrees)	Required Embedment Length, ft	SPT N Value	Effective Stress (ksf)	m	$\sigma'p$	OCR	Ko	Skin Friction Factor	Nominal Skin Friction Capacity, kips	Side Resistance Factor	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	Factored Total Resistance capacity, kips
									Betta	Rn=P*L* fs	F1	F2	R1 = F1*F2*Rn	
3	38	40	30	3.6	0.65	9.07	2.52	0.68	0.53	1,440	0.70	1.00	1,008	
5	38	10	50	6.4	0.75	18.70	2.90	0.74	0.58	702	0.70	1.00	492	
				SUM						2,142			1,500	
														5,099
							SUM			6,169			4,319	

Depth 120

CIDH Embedment	Nominal	Factored
10	155	109
15	311	218
20	466	326
25	622	435
30	777	544
35	957	670
40	1,137	796
45	1,317	922
50	1,497	1,048
55	1,677	1,174
60	1,857	1,300
65	2,037	1,426
70	2,217	1,552
75	2,606	1,824
80	2,994	2,096
85	3,383	2,368
90	3,772	2,640
95	4,123	2,886
100	4,474	3,132
105	4,898	3,428
110	5,322	3,725
115	5,746	4,022
120	6,169	4,319

Model # 3
Project Name Crosstwon Freeway SR-4
Project No. 111904.G01
Subject Preliminary Estimation for Embedment Length for Drilled Shafts
Prepared By HP
Checked By CFH
Date 5/5/2011

Reference:
FHWA, 1999. Drilled Shafts: Construction Procedures and Design Methods. FHWA report No. FHWA-IF-99-025

6 Shaft Diameter (D), ft
18.84 Shaft Perimeter (P), ft
28.27 Cross Sectional Area (A), ft2

End Bearing Capacity (FHWA, 2010, page B-2)

SPT N value	Embedment Length, ft	End Bearing Factor	Unit End Bearing, ksf	Nominal End Bearing Capacity, kips	Resistance Factor for End Bearing	Factored End Bearing Capacity, kips
N		F	q = F*N	Qn=q*A	q1	Q=(q1)*Qn
60		0.92	55.2	1,561	0.5	780

Skin Friction Capacity ((FHWA, 2010, page B-2)

Layer No.	Undrained Shear Strength, psf	Required Embedment Length, ft	Skin Friction Factor	Unit Skin Friction, ksf	†Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	Factored Total Resistance capacity, kips
	Su	L	Alpha	fs = (Alpha) Su	Rn=P*L* fs	F1	F2	R1 = F1*F2*Rn	
1	2,500	8	0.55	1.4	78	0.70	1.00	54	
2	3,000	27	0.55	1.7	839	0.70	1.00	588	
				SUM	917			642	

COHESIVE IGM (FHWA, 2010, page B-3)

Layer No.	SPT N Value	Length	Skin Friction Factor	Unit Skin Friction, ksf	Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	
	N	L	F3	fs = N*F3	Rn=P*L*fs	F4	F5	R2=F4*F5*Rn	
4	55	30	0.075	4.1	2,331	0.70	1.00	1,632	
6	60	20	0.075	4.5	1,696	0.70	1.00	1,187	
TOTAL		50		SUM	4,027			2,819	3,054

Total Skin Friction for cohesive 3,248 kips

Skin Friction Capacity-Cohesionless Soil

Layer No.	phi (degrees)	Required Embedment Length, ft	SPT N Value	Effective Stress (ksf)	m	σ'p	OCR	Ko	Skin Friction Factor	Nominal Skin Friction Capacity, kips	Side Resistance Factor	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	Factored Total Resistance capacity, kips
									Betta	Rn=P*L* fs	F1	F2	R1 = F1*F2*Rn	
3	36	25	30	3.4	0.65	9.07	2.63	0.73	0.53	859	0.70	1.00	601	
5	38	10	50	5.8	0.75	18.70	3.22	0.79	0.62	675	0.70	1.00	473	
				SUM						1,534			1,074	
														5,315

† α = 0 between the ground surface and a depth of 5 ft or to the depth of seasonal moisture

6,478 SUM 4,535

Depth 120

CIDH Embedment	Nominal	Factored
10	140	98
15	295	207
20	451	316
25	606	424
30	762	533
35	917	642
40	1,089	762
45	1,261	882
50	1,432	1,003
55	1,604	1,123
60	1,776	1,243
65	2,165	1,515
70	2,553	1,787
75	2,942	2,059
80	3,330	2,331
85	3,719	2,603
90	4,107	2,875
95	4,445	3,111
100	4,783	3,348
105	5,206	3,645
110	5,630	3,941
115	6,054	4,238
120	6,478	4,535

Model # 7
Project Name Crosstwon Freeway SR-4
Project No. 111904.G01
Subject Preliminary Estimation for Embedment Length for Drilled Shafts
Prepared By HP
Checked By CFH
Date 5/4/2011

Reference:
FHWA, 1999. Drilled Shafts: Construction Procedures and Design Methods. FHWA report No. FHWA-IF-99-025

6 Shaft Diameter (D), ft
18.84 Shaft Perimeter (P), ft
28.27 Cross Sectional Area (A), ft2

End Bearing Capacity (FHWA, 2010, page B-2)

SPT N value	Embedment Length, ft	End Bearing Factor	Unit End Bearing, ksf	Nominal End Bearing Capacity, kips	Resistance Factor for End Bearing	Factored End Bearing Capacity, kips
N		F	q = F*N	Qn=q*A	q1	Q=(q1)*Qn
60		0.92	55.2	1,561	0.5	780

Skin Friction Capacity ((FHWA, 2010, page B-2)

Layer No.	Undrained Shear Strength, psf	Required Embedment Length, ft	Skin Friction Factor	Unit Skin Friction, ksf	†Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	Factored Total Resistance capacity, kips
	Su	L	Alpha	fs = (Alpha) Su	Rn=P*L* fs	F1	F2	R1 = F1*F2*Rn	
1	2,000	15	0.55	1.1	207	0.70	1.00	145	
2	2,000	8	0.55	1.1	166	0.70	1.00	116	
4	3,000	10	0.55	1.7	311	0.70	1.00	218	
6	4,000	20	0.51	2.0	769	0.70	1.00	538	
7	5,000	20	0.464	2.3	874	0.70	1.00	612	
				SUM	2,327			1,629	

COHESIVE IGM (FHWA, 2010, page B-3)

Layer No.	SPT N Value	Length	Skin Friction Factor	Unit Skin Friction, ksf	Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	
	N	L	F3	fs = N*F3	Rn=P*L*fs	F4	F5	R2=F4*F5*Rn	
8	55	30	0.075	4.1	2,331	0.70	1.00	1,632	
TOTAL		30		SUM	2,331			1,632	3,503

Total Skin Friction for cohesive 3,015 kips

Skin Friction Capacity-Cohesionless Soil(FHWA, 2010, Section 13.3.5.1)

Layer No.	phi (degrees)	Required Embedment Length, ft	SPT N Value	Effective Stress (ksf)	m	σ'_p	OCR	Ko	Skin Friction Factor	Nominal Skin Friction Capacity, kips	Side Resistance Factor	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	Factored Total Resistance capacity, kips
									Betta	Rn=P*L* fs	F1	F2	R1 = F1*F2*Rn	
3	35	6	30	2.5	0.65	9.07	3.64	0.89	0.63	177	0.70	1.00	124	
5	36	11	50	3.7	0.65	12.65	3.46	0.85	0.62	471	0.70	1.00	330	
				SUM						647			453	
														4,494

† α = 0 between the ground surface and a depth of 5 ft or to the depth of seasonal moisture

5,305 SUM 3,714

Depth 120

CIDH Embedment	Nominal	Factored
10	104	73
15	207	145
20	311	218
25	432	302
30	581	406
35	736	515
40	903	632
45	1,117	782
50	1,331	932
55	1,523	1,066
60	1,715	1,201
65	1,908	1,335
70	2,100	1,470
75	2,318	1,623
80	2,537	1,776
85	2,755	1,929
90	2,974	2,082
95	3,363	2,354
100	3,751	2,626
105	4,140	2,898
110	4,528	3,170
115	4,917	3,442
120	5,305	3,714

Model #8
Project Name Crosstwon Freeway SR-4
Project No. 111904.G01
Subject Preliminary Estimation for Embedment Length for Drilled Shafts
Prepared By HP
Checked By CFH
Date 5/4/2011

Reference:
FHWA, 1999. Drilled Shafts: Construction Procedures and Design Methods. FHWA report No. FHWA-IF-99-025

6 Shaft Diameter (D), ft
18.84 Shaft Perimeter (P), ft
28.27 Cross Sectional Area (A), ft2

End Bearing Capacity (FHWA, 2010, page B-2)

SPT N value	Embedment Length, ft	End Bearing Factor	Unit End Bearing, ksf	Nominal End Bearing Capacity, kips	Resistance Factor for End Bearing	Factored End Bearing Capacity, kips
N		F	q = F*N	Qn=q*A	q1	Q=(q1)*Qn
60		0.92	55.2	1,561	0.5	780

Skin Friction Capacity ((FHWA, 2010, page B-2)

Layer No.	Undrained Shear Strength, psf	Required Embedment Length, ft	Skin Friction Factor	Unit Skin Friction, ksf	†Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	Factored Total Resistance capacity, kips
	Su	L	Alpha	fs = (Alpha) Su	Rn=P*L* fs	F1	F2	R1 = F1*F2*Rn	
1	2,500	15	0.55	1.4	259	0.70	1.00	181	
2	2,500	9	0.55	1.4	233	0.70	1.00	163	
4	4,000	40	0.51	2.0	1,540	0.70	1.00	1,078	
				SUM	2,033			1,423	

COHESIVE IGM (FHWA, 2010, page B-3)

Layer No.	SPT N Value	Length	Skin Friction Factor	Unit Skin Friction, ksf	Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	
	N	L	F3	fs = N*F3	Rn=P*L*fs	F4	F5	R2=F4*F5*Rn	
6	55	30	0.075	4.1	2,331	0.70	1.00	1,632	
TOTAL		30		SUM	2,331			1,632	

Total Skin Friction for cohesive 4,364 kips

Skin Friction Capacity-Cohesionless Soil(FHWA, 2010, Section 13.3.5.1)

Layer No.	phi (degrees)	Required Embedment Length, ft	SPT N Value	Effective Stress (ksf)	m	σ'_p	OCR	Ko	Skin Friction Factor	Nominal Skin Friction Capacity, kips	Side Resistance Factor	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips
									Betta	Rn=P*L* fs	F1	F2	R1 = F1*F2*Rn
3	36	5	40	2.5	0.65	10.94	4.33	0.98	0.71	169	0.70	1.00	118
5	40	21	50	5.9	0.65	12.65	2.16	0.59	0.49	1,140	0.70	1.00	798
				SUM						1,308			916

†α = 0 between the ground surface and a depth of 5 ft or to the depth of seasonal moisture

5,672 SUM 3,971

Depth 120

CIDH Embedmen	Nominal	Factored
10	130	91
15	259	181
20	389	272
25	526	368
30	699	490
35	892	624
40	1,084	759
45	1,277	894
50	1,470	1,029
55	1,662	1,163
60	1,855	1,298
65	2,047	1,433
70	2,256	1,579
75	2,527	1,769
80	2,798	1,959
85	3,070	2,149
90	3,341	2,339
95	3,729	2,611
100	4,118	2,883
105	4,507	3,155
110	4,895	3,427
115	5,284	3,699
120	5,672	3,971

Model #9
Project Name Crosstwon Freeway SR-4
Project No. 111904.G01
Subject Preliminary Estimation for Embedment Length for Drilled Shafts
Prepared By HP
Checked By CFH
Date 5/4/2011

Reference:
FHWA, 1999. Drilled Shafts: Construction Procedures and Design Methods. FHWA report No. FHWA-IF-99-025

6 Shaft Diameter (D), ft
18.84 Shaft Perimeter (P), ft
28.27 Cross Sectional Area (A), ft2

End Bearing Capacity (FHWA, 2010, page B-2)

SPT N value	Embedment Length, ft	End Bearing Factor	Unit End Bearing, ksf	Nominal End Bearing Capacity, kips	Resistance Factor for End Bearing	Factored End Bearing Capacity, kips
N		F	q = F*N	Qn=q*A	q1	Q=(q1)*Qn
60		0.92	55.2	1,561	0.5	780

Skin Friction Capacity ((FHWA, 2010, page B-2)

Layer No.	Undrained Shear Strength, psf	Required Embedment Length, ft	Skin Friction Factor	Unit Skin Friction, ksf	†Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	Factored Total Resistance capacity, kips
	Su	L	Alpha	fs = (Alpha) Su	Rn=P*L* fs	F1	F2	R1 = F1*F2*Rn	
1	2,500	15	0.55	1.4	259	0.70	1.00	181	
2	2,500	10	0.55	1.4	259	0.70	1.00	181	
4	3,500	20	0.535	1.9	706	0.70	1.00	494	
5	5,000	30	0.464	2.3	1,311	0.70	1.00	918	
				SUM	2,535			1,774	

COHESIVE IGM (FHWA, 2010, page B-3)

Layer No.	SPT N Value	Length	Skin Friction Factor	Unit Skin Friction, ksf	Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	
	N	L	F3	fs = N*F3	Rn=P*L*fs	F4	F5	R2=F4*F5*Rn	
6	55	30	0.075	4.1	2,331	0.70	1.00	1,632	
TOTAL		30		SUM	2,331			1,632	

Total Skin Friction for cohesive 3,555 kips

Skin Friction Capacity-Cohesionless Soil(FHWA, 2010, Section 13.3.5.1)

Layer No.	phi (degrees)	Required Embedment Length, ft	SPT N Value	Effective Stress (ksf)	m	σ'p	OCR	Ko	Skin Friction Factor	Nominal Skin Friction Capacity, kips	Side Resistance Factor	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	Factored Total Resistance capacity, kips
									Betta	Rn=P*L* fs	F1	F2	R1 = F1*F2*Rn	
3	35	15	25	2.9	0.75	11.12	3.83	0.92	0.65	529	0.70	1.00	370	
				SUM						529			370	
														4,557

† α = 0 between the ground surface and a depth of 5 ft or to the depth of seasonal moisture

5,396 SUM 3,777

Depth 120

CIDH Embedment	Nominal	Factored
10	130	91
15	259	181
20	389	272
25	518	363
30	694	486
35	871	610
40	1,047	733
45	1,224	857
50	1,400	980
55	1,576	1,103
60	1,753	1,227
65	1,971	1,380
70	2,190	1,533
75	2,408	1,686
80	2,627	1,839
85	2,846	1,992
90	3,064	2,145
95	3,453	2,417
100	3,841	2,689
105	4,230	2,961
110	4,618	3,233
115	5,007	3,505
120	5,396	3,777

Model # 10
Project Name Crosstwon Freeway SR-4
Project No. 111904.G01
Subject Preliminary Estimation for Embedment Length for Drilled Shafts
Prepared By HP
Checked By CFH
Date 5/4/2011

Reference:
FHWA, 1999. Drilled Shafts: Construction Procedures and Design Methods. FHWA report No. FHWA-IF-99-025

6 Shaft Diameter (D), ft
18.84 Shaft Perimeter (P), ft
28.27 Cross Sectional Area (A), ft2

End Bearing Capacity (FHWA, 2010, page B-2)

SPT N value	Embedment Length, ft	End Bearing Factor	Unit End Bearing, ksf	Nominal End Bearing Capacity, kips	Resistance Factor for End Bearing	Factored End Bearing Capacity, kips
N		F	q = F*N	Qn=q*A	q1	Q=(q1)*Qn
60		0.92	55.2	1,561	0.5	780

Skin Friction Capacity ((FHWA, 2010, page B-2)

Layer No.	Undrained Shear Strength, psf	Required Embedment Length, ft	Skin Friction Factor	Unit Skin Friction, ksf	†Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	Factored Total Resistance capacity, kips
	Su	L	Alpha	fs = (Alpha) Su	Rn=P*L* fs	F1	F2	R1 = F1*F2*Rn	
1	2,000	10	0.55	1.1	104	0.70	1.00	73	
2	2,000	15	0.55	1.1	311	0.70	1.00	218	
4	3,000	15	0.55	1.7	466	0.70	1.00	326	
6	4,000	30	0.51	2.0	1,155	0.70	1.00	809	
				SUM	2,036			1,425	

COHESIVE IGM (FHWA, 2010, page B-3)

Layer No.	SPT N Value	Length	Skin Friction Factor	Unit Skin Friction, ksf	Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	
	N	L	F3	fs = N*F3	Rn=P*L*fs	F4	F5	R2=F4*F5*Rn	
7	60	30	0.075	4.5	2,543	0.70	1.00	1,780	
TOTAL		30		SUM	2,543			1,780	3,177

Total Skin Friction for cohesive 3,424 kips

Skin Friction Capacity-Cohesionless Soil(FHWA, 2010, Section 13.3.5.1)

Layer No.	phi (degrees)	Required Embedment Length, ft	SPT N Value	Effective Stress (ksf)	m	σ'p	OCR	Ko	Skin Friction Factor	Nominal Skin Friction Capacity, kips	Side Resistance Factor	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	Factored Total Resistance capacity, kips
									Betta	Rn=P*L* fs	F1	F2	R1 = F1*F2*Rn	
3	33	5	30	2.3	0.65	9.07	4.02	0.97	0.63	134	0.70	1.00	94	
5	35	15	35	3.8	0.75	14.31	3.73	0.91	0.64	689	0.70	1.00	482	
				SUM						823			576	
														4,562

† α = 0 between the ground surface and a depth of 5 ft or to the depth of seasonal moisture

5,402 SUM 3,782

Depth 120

CIDH Embedment	Nominal	Factored
10	104	73
15	207	145
20	311	218
25	414	290
30	549	384
35	704	493
40	859	602
45	1,015	710
50	1,244	871
55	1,474	1,032
60	1,703	1,192
65	1,896	1,327
70	2,089	1,462
75	2,281	1,597
80	2,474	1,732
85	2,666	1,866
90	2,859	2,001
95	3,283	2,298
100	3,707	2,595
105	4,130	2,891
110	4,554	3,188
115	4,978	3,485
120	5,402	3,782

Model # 11
Project Name Crosstwon Freeway SR-4
Project No. 111904.G01
Subject Preliminary Estimation for Embedment Length for Drilled Shafts
Prepared By HP
Checked By CFH
Date 5/4/2011

Reference:
FHWA, 1999. Drilled Shafts: Construction Procedures and Design Methods. FHWA report No. FHWA-IF-99-025

6 Shaft Diameter (D), ft
18.84 Shaft Perimeter (P), ft
28.27 Cross Sectional Area (A), ft2

End Bearing Capacity (FHWA, 2010, page B-2)

SPT N value	Embedment Length, ft	End Bearing Factor	Unit End Bearing, ksf	Nominal End Bearing Capacity, kips	Resistance Factor for End Bearing	Factored End Bearing Capacity, kips
N		F	q = F*N	Qn=q*A	q1	Q=(q1)*Qn
60		0.92	55.2	1,561	0.5	780

Skin Friction Capacity ((FHWA, 2010, page B-2)

Layer No.	Undrained Shear Strength, psf	Required Embedment Length, ft	Skin Friction Factor	Unit Skin Friction, ksf	†Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	Factored Total Resistance capacity, kips
	Su	L	Alpha	fs = (Alpha) Su	Rn=P*L* fs	F1	F2	R1 = F1*F2*Rn	
1	3,000	9	0.55	1.7	124	0.70	1.00	87	
3	3,000	14	0.55	1.7	435	0.70	1.00	305	
5	3,500	15	0.53	1.9	529	0.70	1.00	370	
6	4,000	20	0.51	2.0	770	0.70	1.00	539	
7	5,000	15	0.46	2.3	655	0.70	1.00	459	
				SUM	2,514			1,760	
COHESIVE IGM (FHWA, 2010, page B-3)									
Layer No.	SPT N Value	Length	Skin Friction Factor	Unit Skin Friction, ksf	Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	
	N	L	F3	fs = N*F3	Rn=P*L*fs	F4	F5	R2=F4*F5*Rn	
8	60	30	0.075	4.5	2,543	0.70	1.00	1,780	
TOTAL		30		SUM	2,543			1,780	3,781

Total Skin Friction for cohesive 3,632 kips

Skin Friction Capacity-Cohesionless Soil(FHWA, 2010, Section 13.3.5.1)

Layer No.	phi (degrees)	Required Embedment Length, ft	SPT N Value	Effective Stress (ksf)	m	$\sigma'p$	OCR	Ko	Skin Friction Factor	Nominal Skin Friction Capacity, kips	Side Resistance Factor	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	Factored Total Resistance capacity, kips
									Betta	Rn=P*L* fs	F1	F2	R1 = F1*F2*Rn	
2	35	6	35	1.3	0.7	11.98	9.55	1.56	1.09	154	0.70	1.00	108	
4	40	11	45	2.7	0.65	11.81	4.45	0.93	0.78	431	0.70	1.00	301	
				SUM						585			409	
														4,730

† α = 0 between the ground surface and a depth of 5 ft or to the depth of seasonal moisture

5,642 SUM 3,950

Depth 120

CIDH Embedment	Nominal	Factored
10	150	105
15	279	195
20	434	304
25	590	413
30	753	527
35	949	664
40	1,145	801
45	1,321	925
50	1,497	1,048
55	1,673	1,171
60	1,866	1,306
65	2,058	1,441
70	2,251	1,576
75	2,444	1,710
80	2,662	1,863
85	2,880	2,016
90	3,099	2,169
95	3,523	2,466
100	3,947	2,763
105	4,371	3,059
110	4,795	3,356
115	5,218	3,653
120	5,642	3,950

Model #12
Project Name Crosstwon Freeway SR-4
Project No. 111904.G01
Subject Preliminary Estimation for Embedment Length for Drilled Shafts
Prepared By HP
Checked By CFH
Date 5/4/2011

Reference:
FHWA, 1999. Drilled Shafts: Construction Procedures and Design Methods. FHWA report No. FHWA-IF-99-025

6 Shaft Diameter (D), ft
18.84 Shaft Perimeter (P), ft
28.27 Cross Sectional Area (A), ft2

End Bearing Capacity (FHWA, 2010, page B-2)

SPT N value	Embedment Length, ft	End Bearing Factor	Unit End Bearing, ksf	Nominal End Bearing Capacity, kips	Resistance Factor for End Bearing	Factored End Bearing Capacity, kips
N		F	q = F*N	Qn=q*A	q1	Q=(q1)*Qn
60		0.92	55.2	1,561	0.5	780

Skin Friction Capacity ((FHWA, 2010, page B-2)

Layer No.	Undrained Shear Strength, psf	Required Embedment Length, ft	Skin Friction Factor	Unit Skin Friction, ksf	†Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	Factored Total Resistance capacity, kips
	Su	L	Alpha	fs = (Alpha) Su	Rn=P*L* fs	F1	F2	R1 = F1*F2*Rn	
1	2,500	14	0.55	1.4	233	0.70	1.00	163	
3	2,500	25	0.55	1.4	648	0.70	1.00	453	
				0.0	0	0.70	1.00	0	
					0	0.70	1.00	0	
				SUM	881			617	
COHESIVE IGM (FHWA, 2010, page B-3)									
Layer No.	SPT N Value	Length	Skin Friction Factor	Unit Skin Friction, ksf	Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	
	N	L	F3	fs = N*F3	Rn=P*L*fs	F4	F5	R2=F4*F5*Rn	
5	60	45	0.075	4.5	3,815	0.70	1.00	2,671	
TOTAL		45		SUM	3,815			2,671	4,067

Total Skin Friction for cohesive 4,696 kips

Skin Friction Capacity-Cohesionless Soil(FHWA, 2010, Section 13.3.5.1)

Layer No.	phi (degrees)	Required Embedment Length, ft	SPT N Value	Effective Stress (ksf)	m	σ'_p	OCR	Ko	Skin Friction Factor	Nominal Skin Friction Capacity, kips	Side Resistance Factor	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	Factored Total Resistance capacity, kips
									Betta	Rn=P*L* fs	F1	F2	R1 = F1*F2*Rn	
2	36	6	41	1.9	0.7	13.38	7.16	1.31	0.95	201	0.70	1.00	141	
4	38	30	50	4.6	0.65	12.65	2.76	0.72	0.56	1,452	0.70	1.00	1,017	
				SUM						1,654			1,158	
														5,225

† α = 0 between the ground surface and a depth of 5 ft or to the depth of seasonal moisture

6,349 SUM 4,445

Depth 120

CIDH Embedment	Nominal	Factored
10	130	91
15	267	187
20	434	304
25	564	395
30	693	485
35	823	576
40	953	667
45	1,082	757
50	1,324	927
55	1,566	1,096
60	1,808	1,266
65	2,050	1,435
70	2,292	1,605
75	2,534	1,774
80	2,958	2,071
85	3,382	2,368
90	3,806	2,664
95	4,230	2,961
100	4,654	3,258
105	5,078	3,554
110	5,502	3,851
115	5,926	4,148
120	6,349	4,445

Model #13
Project Name Crosstwon Freeway SR-4
Project No. 111904.G01
Subject Preliminary Estimation for Embedment Length for Drilled Shafts
Prepared By HP
Checked By CFH
Date 5/5/2011

Reference:
FHWA, 1999. Drilled Shafts: Construction Procedures and Design Methods. FHWA report No. FHWA-IF-99-025

6 Shaft Diameter (D), ft
18.84 Shaft Perimeter (P), ft
28.27 Cross Sectional Area (A), ft2

End Bearing Capacity (FHWA, 2010, page B-2)

SPT N value	Embedment Length, ft	End Bearing Factor	Unit End Bearing, ksf	Nominal End Bearing Capacity, kips	Resistance Factor for End Bearing	Factored End Bearing Capacity, kips
N		F	q = F*N	Qn=q*A	q1	Q=(q1)*Qn
60		0.92	55.2	1,561	0.5	780

Skin Friction Capacity ((FHWA, 2010, page B-2)

Layer No.	Undrained Shear Strength, psf	Required Embedment Length, ft	Skin Friction Factor	Unit Skin Friction, ksf	†Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	Factored Total Resistance capacity, kips
	Su	L	Alpha	fs = (Alpha) Su	Rn=P*L* fs	F1	F2	R1 = F1*F2*Rn	
1	1,500	5	0.55	0.8	0	0.70	1.00	0	
2	2,000	10	0.55	1.1	207	0.70	1.00	145	
3	3,000	25	0.55	1.7	777	0.70	1.00	544	
				SUM	984			689	
COHESIVE IGM (FHWA, 2010, page B-3)									
Layer No.	SPT N Value	Length	Skin Friction Factor	Unit Skin Friction, ksf	Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	
	N	L	F3	fs = N*F3	Rn=P*L*fs	F4	F5	R2=F4*F5*Rn	
6	50	40	0.075	3.8	2,826	0.70	1.00	1,978	
TOTAL		40		SUM	2,826			1,978	3,448

Total Skin Friction for cohesive 3,810 kips

Skin Friction Capacity-Cohesionless Soil(FHWA, 2010, Section 13.3.5.1)

Layer No.	phi (degrees)	Required Embedment Length, ft	SPT N Value	Effective Stress (ksf)	m	σ'_p	OCR	Ko	Skin Friction Factor	Nominal Skin Friction Capacity, kips	Side Resistance Factor	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	Factored Total Resistance capacity, kips
									Betta	Rn=P*L* fs	F1	F2	R1 = F1*F2*Rn	
4	36	30	30	4.4	0.8	15.11	3.46	0.85	0.62	1,534	0.70	1.00	1,074	
5	38	10	41	5.6	0.8	19.40	3.45	0.82	0.64	682	0.70	1.00	478	
				SUM						2,216			1,551	
														4,999

† α = 0 between the ground surface and a depth of 5 ft or to the depth of seasonal moisture

6,026 SUM 4,219

Depth 120

CIDH Embedment	Nominal	Factored
10	104	73
15	207	145
20	363	254
25	518	363
30	674	471
35	829	580
40	984	689
45	1,240	868
50	1,496	1,047
55	1,751	1,226
60	2,007	1,405
65	2,263	1,584
70	2,518	1,763
75	2,859	2,002
80	3,200	2,240
85	3,554	2,488
90	3,907	2,735
95	4,260	2,982
100	4,613	3,229
105	4,967	3,477
110	5,320	3,724
115	5,673	3,971
120	6,026	4,219

Axial Pile Capacity Analysis – 8 foot diameter shafts

Model #2
Project Name Crosstwon Freeway SR-4
Project No. 111904.G01
Subject Preliminary Estimation for Embedment Length for Drilled Shafts
Prepared By HP
Checked By CFH
Date 2/14/2011

Reference:
FHWA, 1999. Drilled Shafts: Construction Procedures and Design Methods. FHWA report No. FHWA-IF-99-025

8 Shaft Diameter (D), ft
25.12 Shaft Perimeter (P), ft
50.27 Cross Sectional Area (A), ft2

End Bearing Capacity (FHWA, 2010, page B-2)

SPT N value	Embedment Length, ft	End Bearing Factor	Unit End Bearing, ksf	Nominal End Bearing Capacity, kips	Resistance Factor for End Bearing	Factored End Bearing Capacity, kips
N		F	q = F*N	Qn=q*A	q1	Q=(q1)*Qn
60		0.92	55.2	2,775	0.5	1,387

Skin Friction Capacity ((FHWA, 2010, page B-2)

Layer No.	Undrained Shear Strength, psf	Required Embedment Length, ft	Skin Friction Factor	Unit Skin Friction, ksf	Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	Factored Total Resistance capacity, kips
	Su	L	Alpha	fs = (Alpha) Su	Rn=P*L* fs	F1	F2	R1 = F1*F2*Rn	
1	3,000	8	0.55	1.7	124	0.70	1.00	87	
2	3,000	12	0.55	1.7	497	0.70	1.00	348	
				SUM	622			435	
COHESIVE IGM (FHWA, 2010, page B-3)									
Layer No.	SPT N Value	Length	Skin Friction Factor	Unit Skin Friction, ksf	Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	
	N	L	F3	fs = N*F3	Rn=P*L*fs	F4	F5	R2=F4*F5*Rn	
4	55	20	0.075	4.1	2,072	0.70	1.00	1,451	
6	60	20	0.075	4.5	2,261	0.70	1.00	1,583	
TOTAL		40		SUM	4,333			3,033	3,273

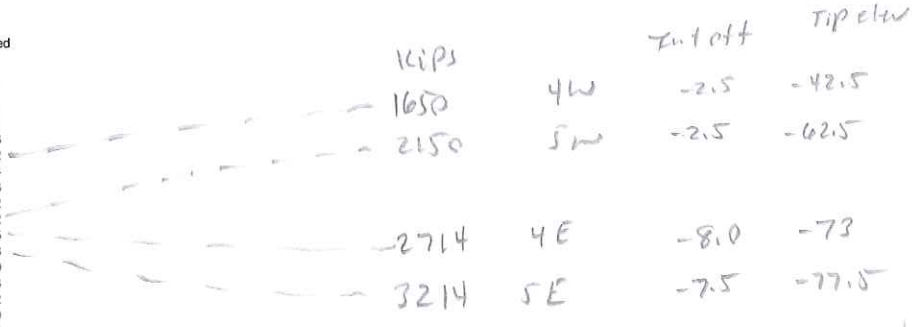
Total Skin Friction for cohesive 2,694 kips

Skin Friction Capacity-Cohesionless Soil(FHWA, 2010, Section 13.3.5.1)

Layer No.	phi (degrees)	Required Embedment Length, ft	SPT N Value	Effective Stress (ksf)	Effective Stress (ksf) bottom of the layer	m	σ'p	OCR	Ko	Skin Friction Factor	Nominal Skin Friction Capacity, kips	Side Resistance Factor	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	Factored Total Resistance capacity, kips
										Betta	Rn=P*L* fs	F1	F2	R1 = F1*F2*Rn	
3	38	40	30	3.6	4.2	0.65	9.07	2.14	0.61	0.48	1,739	0.70	1.00	1,217	
5	38	10	50	6.4	6.1	0.75	18.70	3.05	0.76	0.60	966	0.70	1.00	676	
				SUM							2,705			1,893	
															6,749
								SUM			7,660			5,362	

Depth 110

CIDH Embedment	Nominal	Factored
10	207	145
15	414	290
20	622	435
25	963	674
30	1,247	873
35	1,490	1,043
40	1,702	1,192
45	1,891	1,324
50	2,062	1,443
55	2,217	1,552
60	2,361	1,652
65	2,879	2,015
70	3,397	2,378
75	3,915	2,740
80	4,433	3,103
85	4,932	3,452
90	5,399	3,779
95	5,964	4,175
100	6,529	4,570
105	7,094	4,966
110	7,660	5,362



Model # 3
Project Name Crosstwon Freeway SR-4
Project No. 111904.G01
Subject Preliminary Estimation for Embedment Length for Drilled Shafts
Prepared By HP
Checked By CFH
Date 6/2/2011

Reference:
FHWA, 1999. Drilled Shafts: Construction Procedures and Design Methods. FHWA report No. FHWA-IF-99-025

8 Shaft Diameter (D), ft
25.12 Shaft Perimeter (P), ft
50.27 Cross Sectional Area (A), ft2

End Bearing Capacity (FHWA, 2010, page B-2)

SPT N value	Embedment Length, ft	End Bearing Factor	Unit End Bearing, ksf	Nominal End Bearing Capacity, kips	Resistance Factor for End Bearing	Factored End Bearing Capacity, kips
N		F	q = F*N	Qn=q*A	q1	Q=(q1)*Qn
60		0.92	55.2	2,775	0.5	1,387

Skin Friction Capacity ((FHWA, 2010, page B-2)

Layer No.	Undrained Shear Strength, psf	Required Embedment Length, ft	Skin Friction Factor	Unit Skin Friction, ksf	†Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	Factored Total Resistance capacity, kips
	Su	L	Alpha	fs = (Alpha) Su	Rn=P*L* fs	F1	F2	R1 = F1*F2*Rn	
1	2,500	8	0.55	1.4	104	0.70	1.00	73	
2	3,000	17	0.55	1.7	705	0.70	1.00	493	
				SUM	808			566	

COHESIVE IGM (FHWA, 2010, page B-3)

Layer No.	SPT N Value	Length	Skin Friction Factor	Unit Skin Friction, ksf	Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	
	N	L	F3	fs = N*F3	Rn=P*L*fs	F4	F5	R2=F4*F5*Rn	
4	55	30	0.075	4.1	3,109	0.70	1.00	2,176	
6	60	20	0.075	4.5	2,261	0.70	1.00	1,583	
TOTAL		50		SUM	5,369			3,759	4,129

Total Skin Friction for cohesive 3,917 kips

Skin Friction Capacity-Cohesionless Soil(FHWA, 2010, Section 13.3.5.1)

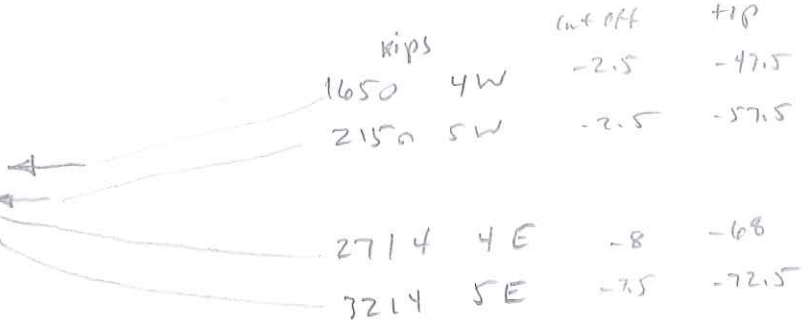
Layer No.	phi (degrees)	Required Embedment Length, ft	SPT N Value	Effective Stress (ksf)	Effective Stress (ksf) Bottom of the Layer	m	σ'p	OCR	Ko	Skin Friction Factor	Nominal Skin Friction Capacity, kips	Side Resistance Factor	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	Factored Total Resistance capacity, kips
										Betta	Rn=P*L* fs	F1	F2	R1 = F1*F2*Rn	
3	36	25	30	3.4	3.6	0.65	9.07	2.52	0.71	0.52	1,116	0.70	1.00	781	
5	38	10	50	5.8	6.1	0.75	18.70	3.05	0.76	0.60	871	0.70	1.00	610	
				SUM							1,987			1,391	
															7,103

†α = 0 between the ground surface and a depth of 5 ft or to the depth of seasonal moisture□change, whichever is greater

8,165 SUM 5,715

Depth 110

CIDH Embedment	Nominal	Factored
10	187	131
15	394	276
20	601	421
25	808	566
30	1,095	767
35	1,342	939
40	1,558	1,090
45	1,750	1,225
50	1,924	1,347
55	2,442	1,709
60	2,960	2,072
65	3,478	2,435
70	3,996	2,797
75	4,514	3,160
80	5,032	3,523
85	5,482	3,838
90	5,904	4,133
95	6,469	4,528
100	7,034	4,924
105	7,599	5,320
110	8,165	5,715



Model # 7
Project Name Crosstwon Freeway SR-4
Project No. 111904.G01
Subject Preliminary Estimation for Embedment Length for Drilled Shafts
Prepared By HP
Checked By CFH
Date 6/6/2011

Reference:
FHWA, 1999. Drilled Shafts: Construction Procedures and Design Methods. FHWA report No. FHWA-IF-99-025

8 Shaft Diameter (D), ft
25.12 Shaft Perimeter (P), ft
50.27 Cross Sectional Area (A), ft2

End Bearing Capacity (FHWA, 2010, page B-2)

SPT N value	Embedment Length, ft	End Bearing Factor	Unit End Bearing, ksf	Nominal End Bearing Capacity, kips	Resistance Factor for End Bearing	Factored End Bearing Capacity, kips
N		F	q = F*N	Qn=q*A	q1	Q=(q1)*Qn
60		0.92	55.2	2,775	0.5	1,387

Skin Friction Capacity ((FHWA, 2010, page B-2)

Layer No.	Undrained Shear Strength, psf	Required Embedment Length, ft	Skin Friction Factor	Unit Skin Friction, ksf	†Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	Factored Total Resistance capacity, kips
	Su	L	Alpha	fs = (Alpha) Su	Rn=P*L* fs	F1	F2	R1 = F1*F2*Rn	
1	2,000	15	0.55	1.1	276	0.70	1.00	193	
2	2,000	8	0.55	1.1	221	0.70	1.00	155	
4	3,000	10	0.55	1.7	414	0.70	1.00	290	
6	4,000	20	0.51	2.0	1,025	0.70	1.00	717	
7	5,000	20	0.464	2.3	1,166	0.70	1.00	816	
				SUM	3,102			2,172	
COHESIVE IGM (FHWA, 2010, page B-3)									
Layer No.	SPT N Value	Length	Skin Friction Factor	Unit Skin Friction, ksf	Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	
	N	L	F3	fs = N*F3	Rn=P*L*fs	F4	F5	R2=F4*F5*Rn	
8	55	30	0.075	4.1	3,109	0.70	1.00	2,176	
TOTAL		30		SUM	3,109			2,176	5,018

Total Skin Friction for cohesive 4,020 kips

Skin Friction Capacity-Cohesionless Soil(FHWA, 2010, Section 13.3.5.1)

Layer No.	phi (degrees)	Required Embedment Length, ft	SPT N Value	Effective Stress (ksf)	Effective Stress (ksf) Bottom of the Layer	m	σ'_p	OCR	Ko	Skin Friction Factor	Nominal Skin Friction Capacity, kips	Side Resistance Factor	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	Factored Total Resistance capacity, kips
										Betta	Rn=P*L* fs	F1	F2	R1 = F1*F2*Rn	
3	35	6	30	2.5	2.7	0.65	9.07	3.38	0.86	0.60	226	0.70	1.00	158	
5	36	11	50	3.7	4.0	0.65	12.65	3.16	0.81	0.59	595	0.70	1.00	417	
				SUM							821			575	
															6,310

† α = 0 between the ground surface and a depth of 5 ft or to the depth of seasonal moisture□change, whichever is greater

7,032 SUM 4,922

Depth 120

CIDH Embedment	Nominal	Factored
10	138	97
15	276	193
20	414	290
25	577	404
30	765	535
35	972	680
40	1,197	838
45	1,478	1,035
50	1,733	1,213
55	1,989	1,392
60	2,245	1,572
65	2,501	1,751
70	2,758	1,930
75	3,049	2,134
80	3,340	2,338
85	3,632	2,542
90	3,923	2,746
95	4,441	3,109
100	4,959	3,472
105	5,477	3,834
110	5,996	4,197
115	6,514	4,560
120	7,032	4,922

Model #8
Project Name Crosstwon Freeway SR-4
Project No. 111904.G01
Subject Preliminary Estimation for Embedment Length for Drilled Shafts
Prepared By HP
Checked By CFH
Date 5/4/2011

Reference:
FHWA, 1999. Drilled Shafts: Construction Procedures and Design Methods. FHWA report No. FHWA-IF-99-025

8 Shaft Diameter (D), ft
25.12 Shaft Perimeter (P), ft
50.27 Cross Sectional Area (A), ft2

End Bearing Capacity (FHWA, 2010, page B-2)

SPT N value	Embedment Length, ft	End Bearing Factor	Unit End Bearing, ksf	Nominal End Bearing Capacity, kips	Resistance Factor for End Bearing	Factored End Bearing Capacity, kips
N		F	q = F*N	Qn=q*A	q1	Q=(q1)*Qn
60		0.92	55.2	2,775	0.5	1,387

Skin Friction Capacity ((FHWA, 2010, page B-2)

Layer No.	Undrained Shear Strength, psf	Required Embedment Length, ft	Skin Friction Factor	Unit Skin Friction, ksf	†Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	Factored Total Resistance capacity, kips
	Su	L	Alpha	fs = (Alpha) Su	Rn=P*L* fs	F1	F2	R1 = F1*F2*Rn	
1	2,500	15	0.55	1.4	345	0.70	1.00	242	
2	2,500	9	0.55	1.4	311	0.70	1.00	218	
4	4,000	40	0.51	2.0	2,054	0.70	1.00	1,438	
				SUM	2,710			1,897	

COHESIVE IGM (FHWA, 2010, page B-3)

Layer No.	SPT N Value	Length	Skin Friction Factor	Unit Skin Friction, ksf	Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	
	N	L	F3	fs = N*F3	Rn=P*L*fs	F4	F5	R2=F4*F5*Rn	
6	55	30	0.075	4.1	3,109	0.70	1.00	2,176	
TOTAL		30		SUM	3,109			2,176	

Total Skin Friction for cohesive 5,819 kips

Skin Friction Capacity-Cohesionless Soil(FHWA, 2010, Section 13.3.5.1)

Layer No.	phi (degrees)	Required Embedment Length, ft	SPT N Value	Effective Stress (ksf)	Effective Stress (ksf)	m	σ'_p	OCR	Ko	Skin Friction Factor	Nominal Skin Friction Capacity, kips	Side Resistance Factor	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	Factored Total Resistance capacity, kips
										Betta	Rn=P*L* fs	F1	F2	R1 = F1*F2*Rn	
3	36	5	40	2.5	2.7	0.65	10.94	4.08	0.94	0.68	217	0.70	1.00	152	
5	40	21	50	5.9	6.5	0.65	12.65	1.94	0.55	0.46	1,419	0.70	1.00	993	
				SUM							1,636			1,145	
															6,605

†α = 0 between the ground surface and a depth of 5 ft or to the depth of seasonal moisture□change, whichever is greater

7,454 SUM 5,218

Depth 120

CIDH Embedmen	Nominal	Factored
10	173	121
15	345	242
20	518	363
25	702	492
30	925	647
35	1,181	827
40	1,438	1,007
45	1,695	1,186
50	1,952	1,366
55	2,208	1,546
60	2,465	1,726
65	2,722	1,905
70	3,005	2,103
75	3,375	2,363
80	3,720	2,604
85	4,043	2,830
90	4,346	3,042
95	4,864	3,405
100	5,382	3,767
105	5,900	4,130
110	6,418	4,493
115	6,936	4,855
120	7,454	5,218

Model #9
Project Name Crosstwon Freeway SR-4
Project No. 111904.G01
Subject Preliminary Estimation for Embedment Length for Drilled Shafts
Prepared By HP
Checked By CFH
Date 6/2/2011

Reference:
FHWA, 1999. Drilled Shafts: Construction Procedures and Design Methods. FHWA report No. FHWA-IF-99-025

8 Shaft Diameter (D), ft
25.12 Shaft Perimeter (P), ft
50.27 Cross Sectional Area (A), ft2

End Bearing Capacity (FHWA, 2010, page B-2)

SPT N value	Embedment Length, ft	End Bearing Factor	Unit End Bearing, ksf	Nominal End Bearing Capacity, kips	Resistance Factor for End Bearing	Factored End Bearing Capacity, kips
N		F	q = F*N	Qn=q*A	q1	Q=(q1)*Qn
60		0.92	55.2	2,775	0.5	1,387

Skin Friction Capacity ((FHWA, 2010, page B-2)

Layer No.	Undrained Shear Strength, psf	Required Embedment Length, ft	Skin Friction Factor	Unit Skin Friction, ksf	†Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	Factored Total Resistance capacity, kips
	Su	L	Alpha	fs = (Alpha) Su	Rn=P*L* fs	F1	F2	R1 = F1*F2*Rn	
1	2,500	15	0.55	1.4	345	0.70	1.00	242	
2	2,500	10	0.55	1.4	345	0.70	1.00	242	
4	3,500	20	0.535	1.9	941	0.70	1.00	659	
5	5,000	30	0.464	2.3	1,748	0.70	1.00	1,224	
				SUM	3,380			2,366	

COHESIVE IGM (FHWA, 2010, page B-3)

Layer No.	SPT N Value	Length	Skin Friction Factor	Unit Skin Friction, ksf	Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	
	N	L	F3	fs = N*F3	Rn=P*L*fs	F4	F5	R2=F4*F5*Rn	
6	55	30	0.075	4.1	3,109	0.70	1.00	2,176	
TOTAL		30		SUM	3,109			2,176	

Total Skin Friction for cohesive 4,740 kips

Skin Friction Capacity-Cohesionless Soil(FHWA, 2010, Section 13.3.5.1)

Layer No.	phi (degrees)	Required Embedment Length, ft	SPT N Value	Effective Stress (ksf)	Effective Stress (ksf) bottom of the Layer	m	σ'_p	OCR	Ko	Skin Friction Factor	Nominal Skin Friction Capacity, kips	Side Resistance Factor	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	Factored Total Resistance capacity, kips
										Betta	Rn=P*L* fs	F1	F2	R1 = F1*F2*Rn	
3	35	15	25	2.9	3.4	0.75	11.12	3.29	0.84	0.59	647	0.70	1.00	453	
				SUM							647			453	
															6,382

† α = 0 between the ground surface and a depth of 5 ft or to the depth of seasonal moisture change, whichever is greater

7,136 SUM 4,995

Depth 120

CIDH Embedment	Nominal	Factored
10	173	121
15	345	242
20	518	363
25	691	484
30	934	654
35	1,147	803
40	1,338	937
45	1,573	1,101
50	1,808	1,266
55	2,043	1,430
60	2,279	1,595
65	2,570	1,799
70	2,861	2,003
75	3,153	2,207
80	3,444	2,411
85	3,736	2,615
90	4,027	2,819
95	4,545	3,182
100	5,063	3,544
105	5,581	3,907
110	6,099	4,270
115	6,617	4,632
120	7,136	4,995

Model # 10
Project Name Crosstwon Freeway SR-4
Project No. 111904.G01
Subject Preliminary Estimation for Embedment Length for Drilled Shafts
Prepared By HP
Checked By CFH
Date 6/2/2011

Reference:
FHWA, 1999. Drilled Shafts: Construction Procedures and Design Methods. FHWA report No. FHWA-IF-99-025

8 Shaft Diameter (D), ft
25.12 Shaft Perimeter (P), ft
50.27 Cross Sectional Area (A), ft2

End Bearing Capacity (FHWA, 2010, page B-2)

SPT N value	Embedment Length, ft	End Bearing Factor	Unit End Bearing, ksf	Nominal End Bearing Capacity, kips	Resistance Factor for End Bearing	Factored End Bearing Capacity, kips
N		F	q = F*N	Qn=q*A	q1	Q=(q1)*Qn
60		0.92	55.2	2,775	0.5	1,387

Skin Friction Capacity ((FHWA, 2010, page B-2)

Layer No.	Undrained Shear Strength, psf	Required Embedment Length, ft	Skin Friction Factor	Unit Skin Friction, ksf	†Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	Factored Total Resistance capacity, kips
	Su	L	Alpha	fs = (Alpha) Su	Rn=P*L* fs	F1	F2	R1 = F1*F2*Rn	
1	2,000	10	0.55	1.1	138	0.70	1.00	97	
2	2,000	15	0.55	1.1	414	0.70	1.00	290	
4	3,000	15	0.55	1.7	622	0.70	1.00	435	
6	4,000	30	0.51	2.0	1,540	0.70	1.00	1,078	
				SUM	2,715			1,900	
COHESIVE IGM (FHWA, 2010, page B-3)									
Layer No.	SPT N Value	Length	Skin Friction Factor	Unit Skin Friction, ksf	Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	
	N	L	F3	fs = N*F3	Rn=P*L*fs	F4	F5	R2=F4*F5*Rn	
7	60	30	0.075	4.5	3,391	0.70	1.00	2,374	
TOTAL		30		SUM	3,391			2,374	4,583

Total Skin Friction for cohesive 4,566 kips

Skin Friction Capacity-Cohesionless Soil(FHWA, 2010, Section 13.3.5.1)

Layer No.	phi (degrees)	Required Embedment Length, ft	SPT N Value	Effective Stress (ksf)	Effective Stress (ksf) Bottom of the Layer	m	σ'p	OCR	Ko	Skin Friction Factor	Nominal Skin Friction Capacity, kips	Side Resistance Factor	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	Factored Total Resistance capacity, kips
										Betta	Rn=P*L* fs	F1	F2	R1 = F1*F2*Rn	
3	33	5	30	2.3	2.4	0.65	9.07	3.76	0.94	0.61	172	0.70	1.00	121	
5	35	15	35	3.8	4.3	0.75	14.31	3.32	0.85	0.59	859	0.70	1.00	601	
				SUM							1,031			722	
															6,383

† α = 0 between the ground surface and a depth of 5 ft or to the depth of seasonal moisturemchange, whichever is greater

7,137 SUM 4,996

Depth 120

CIDH Embedment	Nominal	Factored
10	138	97
15	276	193
20	414	290
25	553	387
30	725	508
35	932	653
40	1,140	798
45	1,347	943
50	1,660	1,162
55	1,945	1,361
60	2,206	1,544
65	2,462	1,724
70	2,719	1,903
75	2,976	2,083
80	3,233	2,263
85	3,489	2,443
90	3,746	2,622
95	4,311	3,018
100	4,876	3,414
105	5,442	3,809
110	6,007	4,205
115	6,572	4,600
120	7,137	4,996

Axial Pile Capacity Analysis – 9 foot diameter shafts

Model #4
Project Nam Crosstwon Freeway SR-4
Project No. 111904.G01
Subject Preliminary Estimation for Embedment Length for Drilled Shafts
Prepared By HP
Checked By CFH
Date 4/20/2011

Reference:
FHWA, 1999. Drilled Shafts: Construction Procedures and Design Methods. FHWA report No. FHWA-IF-99-025

9 Shaft Diameter (D), ft
28.26 Shaft Perimeter (P), ft
63.62 Cross Sectional Area (A), ft2

End Bearing Capacity (FHWA, 2010, page B-2)

SPT N value	Embedment Length, ft	End Bearing Factor	Unit End Bearing, ksf	Nominal End Bearing Capacity, kips	Resistance Factor for End Bearing	Factored End Bearing Capacity, kips
N		F	q = F*N	Qn=q*A	q1	Q=(q1)*Qn
60		0.92	55.2	3,512	0.5	1,756

Skin Friction Capacity ((FHWA, 2010, page B-2)

Layer No.	Undrained Shear Strength, psf	Required Embedment Length, ft	Skin Friction Factor	Unit Skin Friction, ksf	Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	Factored Total Resistance capacity, kips
	Su	L	Alpha	fs = (Alpha) Su	Rn=P*L* fs	F1	F2	R1 = F1*F2*Rn	
1	2,000	8	0.55	1.1	93	0.70	0.75	49	
2	3,000	12	0.55	1.7	560	0.70	0.75	294	
2a	3,000	9	0.55	1.7	420	0.70	1.00	294	
4	4,000	27	0.51	2.0	1,557	0.70	1.00	1,090	
				SUM	2,629			1,726	

COHESIVE IGM (FHWA, 2010, page B-3)

Layer No.	SPT N Value	Length	Skin Friction Factor	Unit Skin Friction, ksf	Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	
	N	L	F3	fs = N*F3	Rn=P*L*fs	F4	F5	R2=F4*F5*Rn	
6	55	11	0.075	4.1	1,282	0.70	1.00	898	
8	55	30	0.075	4.1	3,497	0.70	1.00	2,448	
10	60	11	0.075	4.5	1,399	0.70	1.00	979	
TOTAL		52		SUM	6,178			4,325	

Total Skin Friction for cohesive 3,492 kips

Skin Friction Capacity-Cohesionless Soil

Layer No.	phi (degrees)	Required Embedment Length, ft	SPT N Value	Effective Stress (ksf)	m	σp	OCR	Ko	Skin Friction Factor	Nominal Skin Friction Capacity, kips	Side Resistance Factor	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	Factored Total Resistance capacity, kips
									Betta	Rn=P*L* fs	F1	F2	R1 = F1*F2*Rn	
3	33	9	36	2.6	0.8	17.48	6.81	1.29	0.84	549	0.70	0.75	288	
5	38	13	40	4.3	0.7	13.15	3.04	0.76	0.60	947	0.70	1.00	663	
7	40	10	45	6.4	0.7	14.28	2.24	0.60	0.50	907	0.70	1.00	635	
9	40	10	57	8.9	0.75	20.63	2.32	0.61	0.51	1,294	0.70	1.00	906	
				SUM						3,697			2,492	
														10,299

SUM 12,504 8,543

Depth 150

CIDH Embedment	Nominal	Factored
10	187	98
15	420	220
20	653	343
25	886	506
30	1,133	669
35	1,438	829
40	1,737	1,005
45	2,025	1,207
50	2,313	1,409
55	2,601	1,611
60	2,890	1,812
65	3,178	2,014
70	3,542	2,269
75	3,906	2,524
80	4,358	2,840
85	4,941	3,248
90	5,498	3,638
95	5,951	3,956
100	6,431	4,291
105	7,014	4,699
110	7,596	5,107
115	8,179	5,515
120	8,762	5,923
125	9,345	6,331
130	9,941	6,748
135	10,588	7,201
140	11,233	7,653
145	11,868	8,098
150	12,504	8,543

Model #5
Project Name Crosstwon Freeway SR-4
Project No. 111904.G01
Subject Preliminary Estimation for Embedment Length for Drilled Shafts
Prepared By HP
Checked By CFH
Date 4/20/2011

Reference:
FHWA, 1999. Drilled Shafts: Construction Procedures and Design Methods. FHWA report No. FHWA-IF-99-025

9 Shaft Diameter (D), ft
28.26 Shaft Perimeter (P), ft
63.62 Cross Sectional Area (A), ft2

End Bearing Capacity (FHWA, 2010, page B-2)

SPT N value	Embedment Length, ft	End Bearing Factor	Unit End Bearing, ksf	Nominal End Bearing Capacity, kips	Resistance Factor for End Bearing	Factored End Bearing Capacity, kips
N		F	q = F*N	Qn=q*A	q1	Q=(q1)*Qn
60		0.92	55.2	3,512	0.5	1,756

Skin Friction Capacity ((FHWA, 2010, page B-2)

Layer No.	Undrained Shear Strength, psf	Required Embedment Length, ft	Skin Friction Factor	Unit Skin Friction, ksf	Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	Factored Total Resistance capacity, kips
	Su	L	Alpha	fs = (Alpha) Su	Rn=P*L* fs	F1	F2	R1 = F1*F2*Rn	
1	1,500	8	0.55	0.8	70	0.70	0.75	37	
2	2,500	12	0.55	1.4	466	0.70	0.75	245	
3	3,500	40	0.55	1.9	2,176	0.70	1.00	1,523	
4	5,000	30	0.46	2.3	1,950	0.70	1.00	1,365	
				SUM	4,662			3,170	
COHESIVE IGM (FHWA, 2010, page B-3)									
Layer No.	SPT N Value	Length	Skin Friction Factor	Unit Skin Friction, ksf	Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	
	N	L	F3	fs = N*F3	Rn=P*L*fs	F4	F5	R2=F4*F5*Rn	
5	60	60	0.075	4.5	7,630	0.70	1.00	5,341	
TOTAL		60						5,341	10,267

Total Skin Friction for cohesive 10,342 kips

12,292 SUM 8,511

Depth 150

CIDH Embedment	Nominal	Factored
10	148	78
15	342	180
20	536	282
25	808	472
30	1,080	662
35	1,352	853
40	1,624	1,043
45	1,896	1,234
50	2,168	1,424
55	2,440	1,614
60	2,712	1,805
65	3,037	2,032
70	3,362	2,260
75	3,687	2,487
80	4,012	2,715
85	4,337	2,942
90	4,662	3,170
95	5,298	3,615
100	5,934	4,060
105	6,570	4,505
110	7,206	4,950
115	7,841	5,395
120	8,477	5,840
125	9,113	6,285
130	9,749	6,730
135	10,385	7,176
140	11,021	7,621
145	11,657	8,066
150	12,292	8,511

Model #6
Project Name Crosstwon Freeway SR-4
Project No. 111904.G01
Subject Preliminary Estimation for Embedment Length for Drilled Shafts
Prepared By HP
Checked By CFH
Date 4/20/2011

Reference:
FHWA, 1999. Drilled Shafts: Construction Procedures and Design Methods. FHWA report No. FHWA-IF-99-025

9 Shaft Diameter (D), ft
28.26 Shaft Perimeter (P), ft
63.62 Cross Sectional Area (A), ft2

End Bearing Capacity (FHWA, 2010, page B-2)

SPT N value	Embedment Length, ft	End Bearing Factor	Unit End Bearing, ksf	Nominal End Bearing Capacity, kips	Resistance Factor for End Bearing	Factored End Bearing Capacity, kips
N		F	q = F*N	Qn=q*A	q1	Q=(q1)*Qn
60		0.92	55.2	3,512	0.5	1,756

Skin Friction Capacity ((FHWA, 2010, page B-2)

Layer No.	Undrained Shear Strength, psf	Required Embedment Length, ft	Skin Friction Factor	Unit Skin Friction, ksf	Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	Factored Total Resistance capacity, kips
	Su	L	Alpha	fs = (Alpha) Su	Rn=P*L* fs	F1	F2	R1 = F1*F2*Rn	
1	2,000	10	0.55	1.1	155	0.70	0.75	82	
3	3,250	40	0.55	1.8	2,021	0.70	1.00	1,414	
5	5,000	20	0.46	2.3	1,300	0.70	1.00	910	
				SUM	3,476			2,406	
COHESIVE IGM (FHWA, 2010, page B-3)									
Layer No.	SPT N Value	Length	Skin Friction Factor	Unit Skin Friction, ksf	Nominal Skin Friction Capacity, kips	Resistance Factor for Skin Friction	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	
	N	L	F3	fs = N*F3	Rn=P*L*fs	F4	F5	R2=F4*F5*Rn	
6	60	60	0.075	4.5	7,630	0.70	1.00	5,341	
TOTAL		60						5,341	9,503

Total Skin Friction for cohesive 11,106 kips

Skin Friction Capacity-Cohesionless Soil(FHWA, 2010, Section 13.3.5.1)

Layer No.	phi (degrees)	Required Embedment Length, ft	SPT N value	Effective Stress (ksf) σ'v	m	σ'p	OCR	Ko	Skin Friction Factor	Nominal Skin Friction Capacity, kips	Side Resistance Factor	Reduction Factor Due to Steel Casing	Factored Skin Friction Capacity, kips	Factored Total Resistance capacity, kips
									Betta	Rn=P*L* fs	F1	F2	R1 = F1*F2*Rn	
2	36	10	21	1.5	0.75	9.76	6.44	1.23	0.90	383	0.70	0.75	201	
4	40	10	35	4.7	0.7	11.98	2.57	0.65	0.55	725	0.70	1.00	507	
				SUM						1,108			708	
														10,211
							SUM			12,214			8,455	

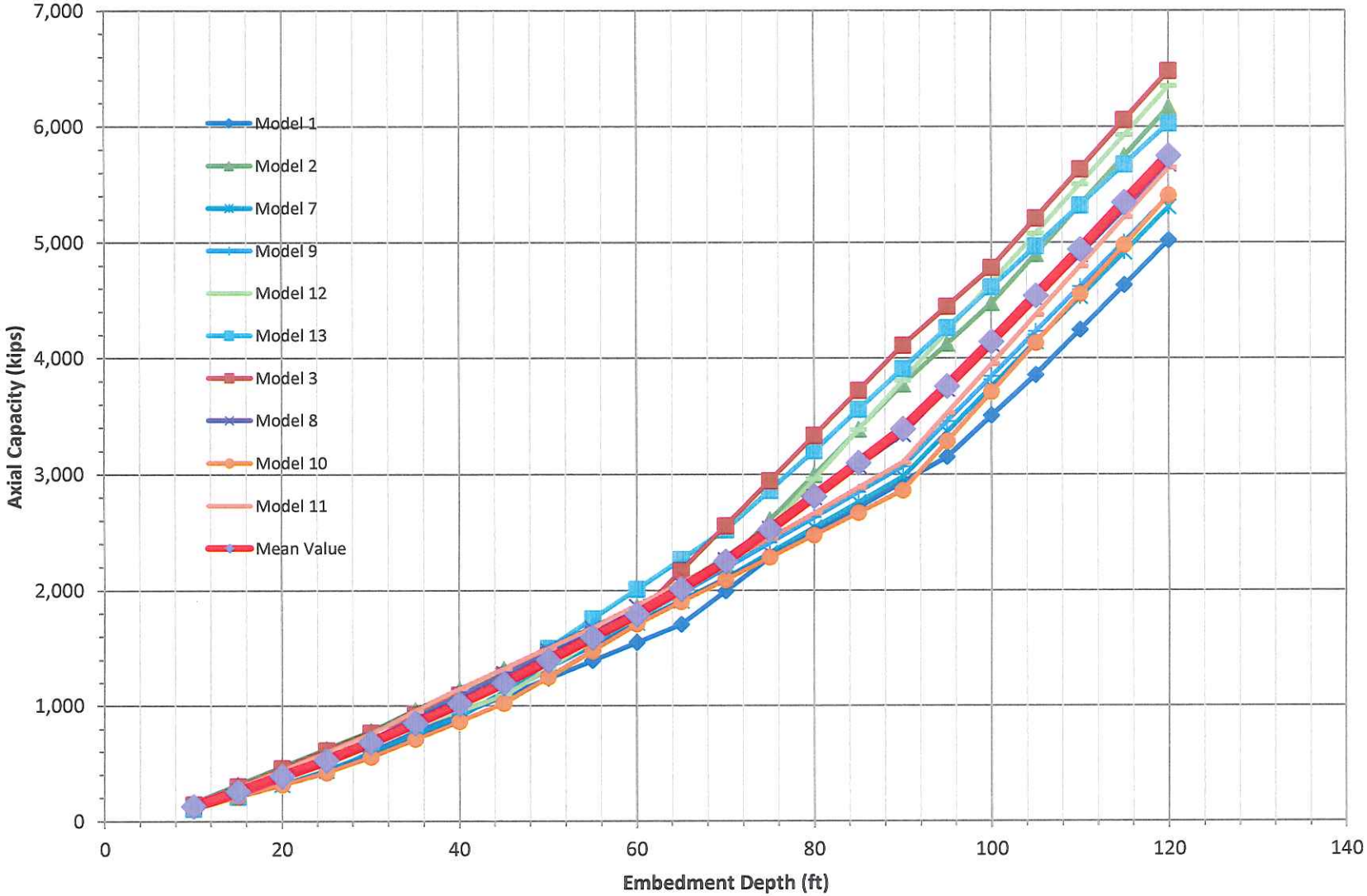
Depth 150

CIDH Embedment	Nominal	Factored
10	155	82
15	347	182
20	539	283
25	791	460
30	1,044	636
35	1,296	813
40	1,549	990
45	1,801	1,167
50	2,054	1,344
55	2,307	1,520
60	2,559	1,697
65	2,921	1,951
70	3,284	2,204
75	3,609	2,432
80	3,934	2,659
85	4,259	2,887
90	4,584	3,114
95	5,220	3,559
100	5,855	4,005
105	6,491	4,450
110	7,127	4,895
115	7,763	5,340
120	8,399	5,785
125	9,035	6,230
130	9,670	6,675
135	10,306	7,120
140	10,942	7,565
145	11,578	8,010
150	12,214	8,455

Axial Pile Capacity Curves – 6 foot diameter shafts

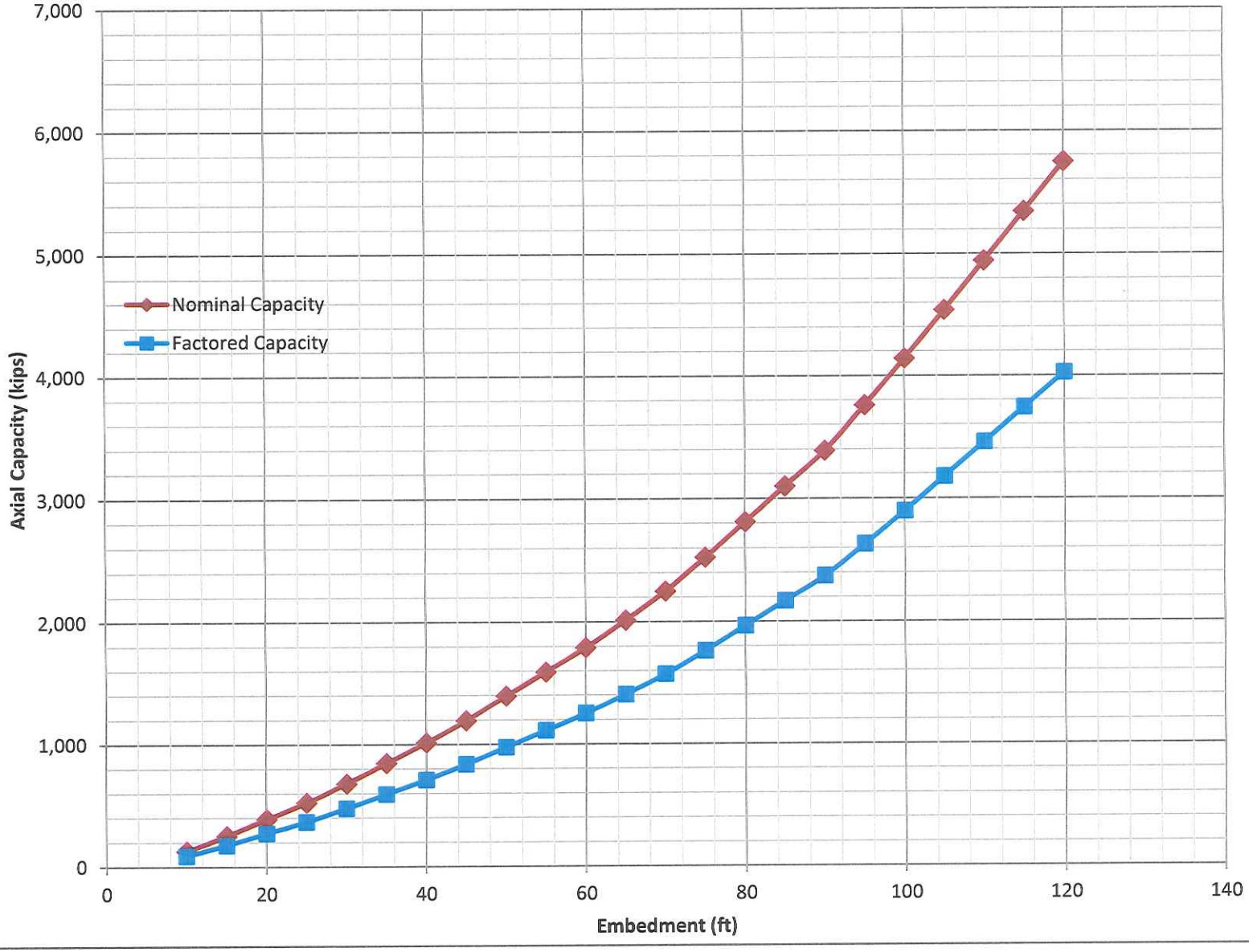
Crosstown Freeway SR 4

6- foot Dia. CIDH Nominal Axial Capacity vs. Embedment Depth



Crosstown Freeway SR 4

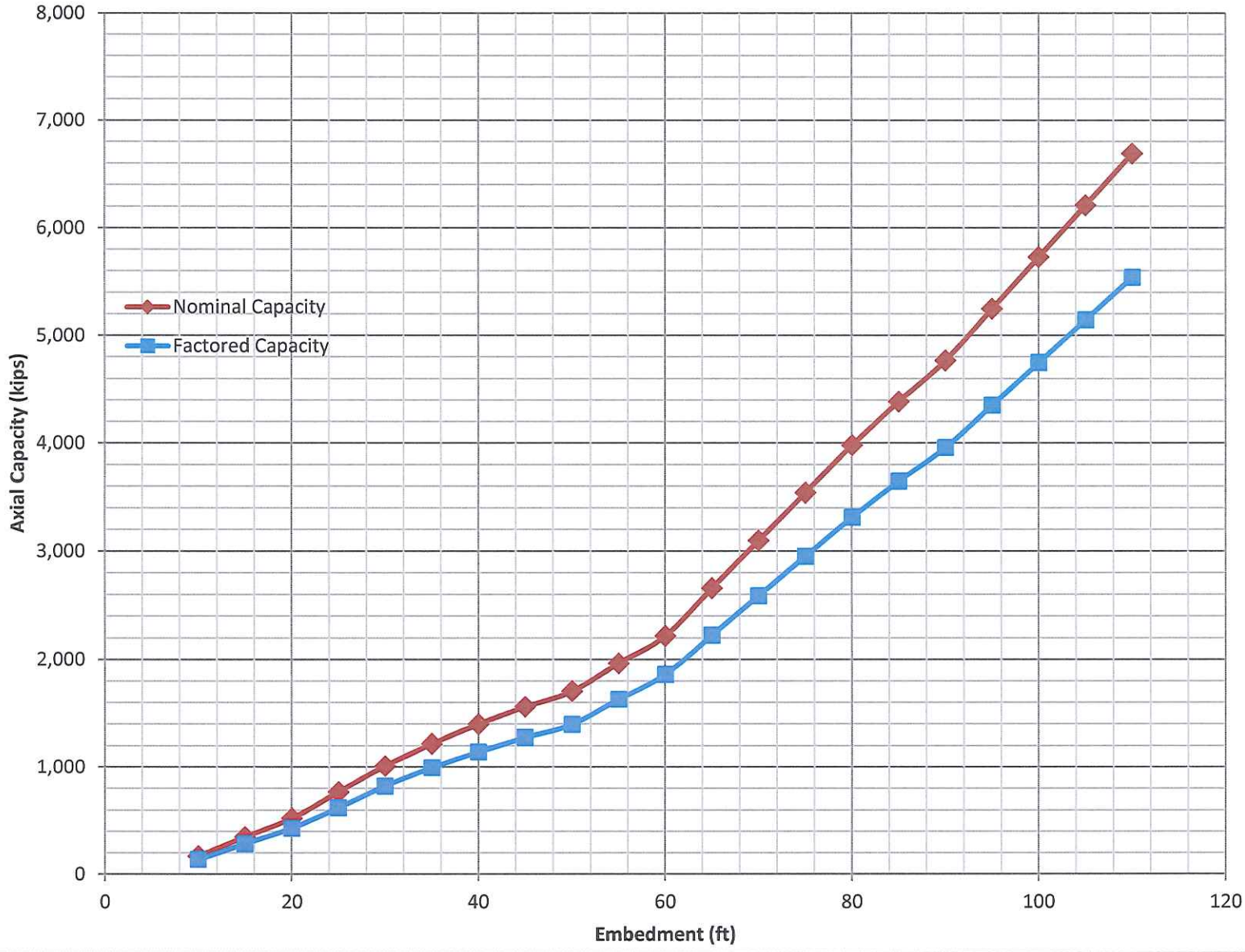
6-foot Dia. CIDH Axial Capacity vs. Embedment Depth



Axial Pile Capacity Curves – 8 foot diameter shafts

Crosstown Freeway SR 4

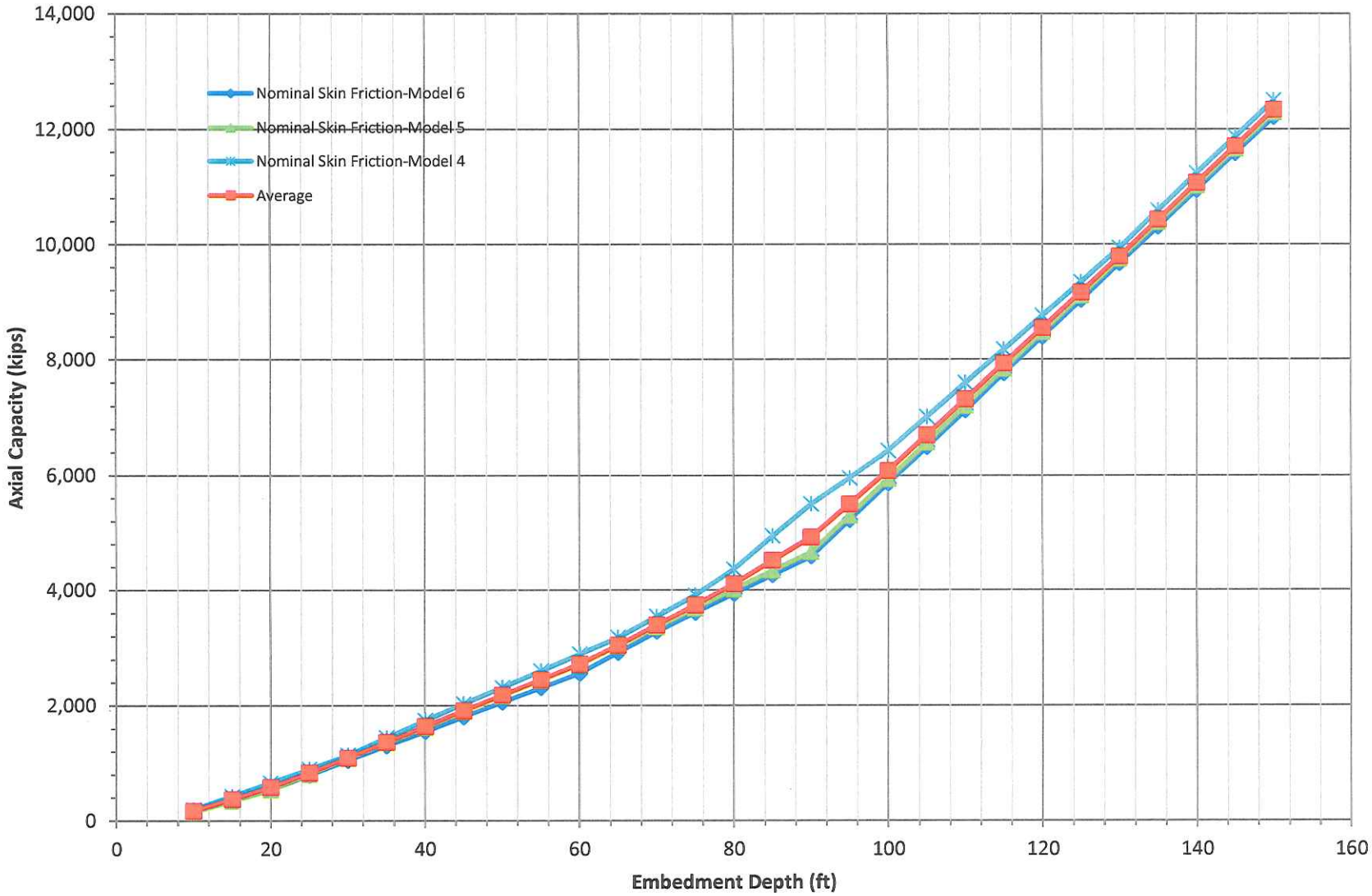
8-foot Dia. CIDH Axial Capacity vs. Embedment Depth



Axial Pile Capacity Curves – 9 foot diameter shafts

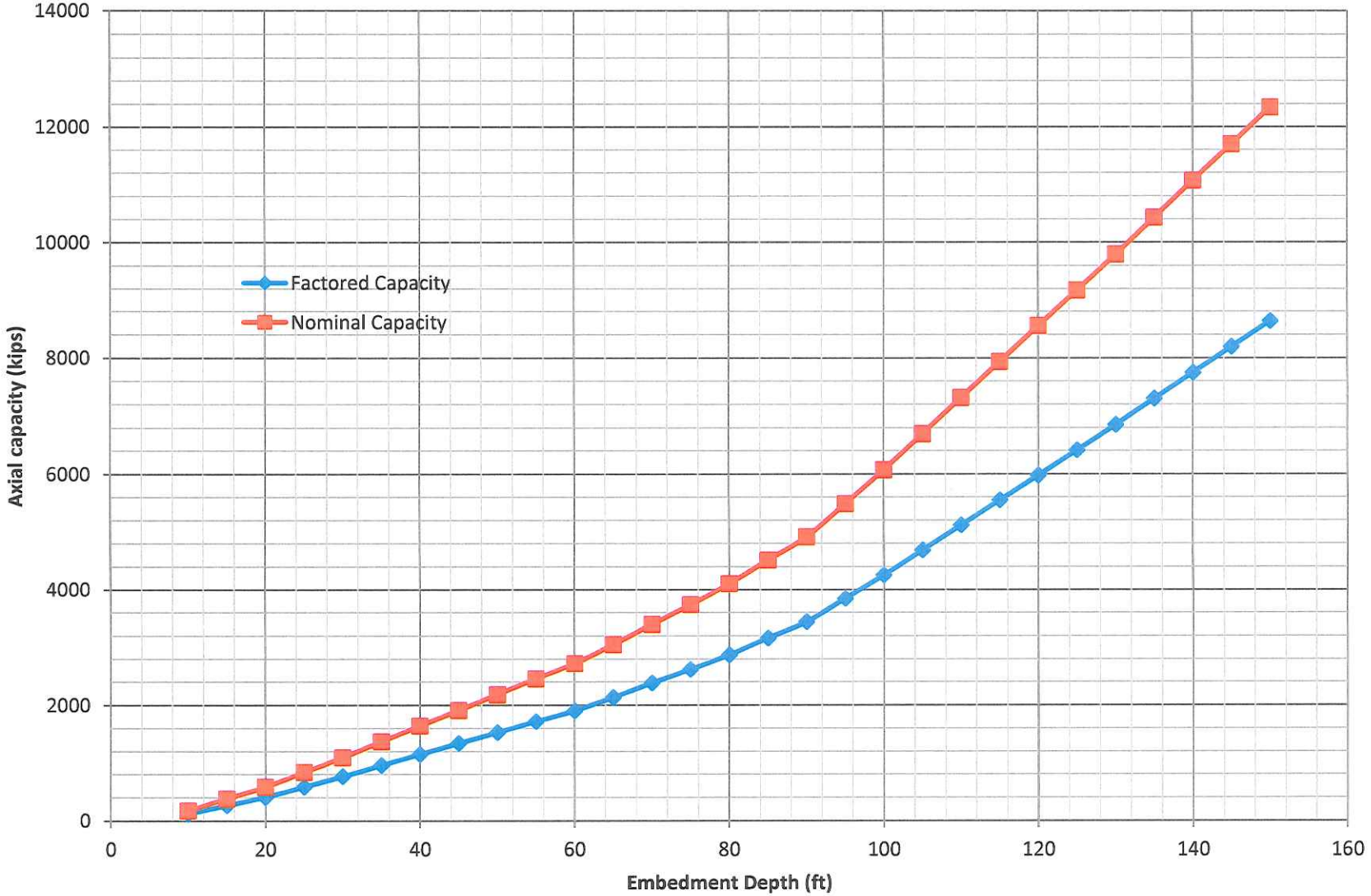
Crosstown Freeway SR 4

9- foot Dia. CIDH Nominal Axial Capacity vs. Embedment Depth



Croostown Freeway SR 4

9-Foot Dia. CIDH Axial Capacity vs. Embedment Depth



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Bright People. Right Solutions.

APPENDIX I
L-PILE ANALYSIS RESULTS FOR CIDH PILES

LIST OF ATTACHMENTS

The following are attached and complete this appendix.

Summary of Max Deflection and Rotation of Piles Under Applied Shear and Moment
L-Pile Input for 6 foot Diameter Piles – Shear Force Applied at Pile Head
L-Pile Input for 6 foot Diameter Piles – Moment Applied at Pile Head
L-Pile Input for 8 foot Diameter Piles – Shear Force Applied at Pile Head
L-Pile Input for 8 foot Diameter Piles – Moment Applied at Pile Head
L-Pile Input for 9 foot Diameter Piles – Shear Force Applied at Pile Head
L-Pile Input for 9 foot Diameter Piles – Moment Applied at Pile Head
L-Pile Input for 9 foot Diameter Piles – Shear Force & Moment Applied at Pile Head
Deflection and Moment Curves for 6 foot Diameter Piles – Shear Force Applied
Deflection and Moment Curves for 6 foot Diameter Piles – Moment Applied
Deflection and Moment Curves for 8 foot Diameter Piles – Shear Force Applied
Deflection and Moment Curves for 8 foot Diameter Piles – Moment Applied
Deflection and Moment Curves for 9 foot Diameter Piles – Shear Force Applied
Deflection and Moment Curves for 9 foot Diameter Piles – Moment Applied
Deflection and Moment Curves for foot Diameter Piles – Shear & Moment Applied

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Summary of Max Deflection and Rotation of Piles Under Applied Shear and Moment

L-Pile Analysis Results for 6-foot Diameter Shafts

Crosstown Freeway SR-4 Improvement

Applicable Substructures	Loading Condition	Model #	Pile Head	
			Max Deflection (inch)	Rotation (Radians)
Abutment 1 and Bents 2 and 3	Shear only (Max 300 KIPS)	1	0.48	0.0023
	Shear only (Min 247 KIPS)	1	0.312	0.0016
	Moment Only (Max 4,529 FT-KIPS)	1	0.322	0.0029
	Moment Only (Min 4,068 FT-KIPS)	1	0.2743	0.0025
	Shear only (Max 300 KIPS)	2	0.274	0.0016
	Shear only (Min 247 KIPS)	2	0.179	0.0011
	Moment Only (Max 4,529 FT-KIPS)	2	0.244	0.0025
	Moment Only (Min 4,068 FT-KIPS)	2	0.206	0.0022
Bents 16 through 21	Shear only (Max 547 KIPS)	12	1.00	0.0046
	Shear only (Ave 416 KIPS)	12	0.62	0.003
	Shear only (Min 284 KIPS)	12	0.317	0.0017
	Moment Only (Max 4,803 FT-KIPS)	12	0.303	0.0028
	Moment Only (Ave. 4,645.5 FT-KIPS)	12	0.287	0.0027
	Moment Only (Min 4,488 FT-KIPS)	12	0.271	0.0026
	Shear only (Max 547 KIPS)	13	1.678	0.0066
	Shear only (Ave 416 KIPS)	13	0.975	0.0042
	Shear only (Min 284 KIPS)	13	0.451	0.0022
	Moment Only (Max 4,803 FT-KIPS)	13	0.379	0.0032
	Moment Only (Ave. 4,645.5 FT-KIPS)	13	0.36	0.003
	Moment Only (Min 4,488 FT-KIPS)	13	0.342	0.0029

Applicable Substructures	Loading Condition	Model #	Pile Head	
			Max Deflection (inch)	Moment (kips-ft)
Abutment 22	Lateral (115 KIPS), Axial (620 KIPS)	13	0.017	752

		Date:
Prepared By:	HP	5/23/2012
Checked By:	CFH	5/23/2012

L-Pile Analysis Results for 8-foot Diameter Shafts

Crosstown Freeway SR-4 Improvement

Applicable Substructures	Loading Condition	Model #	Pile Head	
			Max Deflection (inch)	Rotation (Radians)
Bents 4 and 5	Shear only (Max 493 KIPS)	2	0.48	0.0019
	Shear only (Min 427 KIPS)	2	0.373	0.0019
	Moment Only (Max 11,184 FT-KIPS)	2	0.448	0.003
	Moment Only (Min 10,526 FT-KIPS)	2	0.408	0.0028
	Shear only (Max 493 KIPS)	3	0.517	0.002
	Shear only (Min 427 KIPS)	3	0.393	0.0016
	Moment Only (Max 11,184 FT-KIPS)	3	0.467	0.003
	Moment Only (Min 10,526 FT-KIPS)	3	0.429	0.0029
Bents 10 through 15	Shear only (Max 617 KIPS)	7	0.775	0.0028
	Shear only (Min 533 KIPS)	7	0.592	0.0022
	Moment Only (Max 11,284FT-KIPS)	7	0.446	0.003
	Moment Only (Min 10,687 FT-KIPS)	7	0.408	0.0028
	Shear only (Max 617 KIPS)	8	0.597	0.0023
	Shear only (Min 533 KIPS)	8	0.447	0.0018
	Moment Only (Max 11,284FT-KIPS)	8	0.38	0.0028
	Moment Only (Min 10,687 FT-KIPS)	8	0.35	0.0026
	Shear only (Max 617 KIPS)	9	0.629	0.0023
	Shear only (Min 533 KIPS)	9	0.464	0.0018
	Moment Only (Max 11,284FT-KIPS)	9	0.384	0.0028
	Moment Only (Min 10,687 FT-KIPS)	9	0.354	0.0026
	Shear only (Max 617 KIPS)	10	0.804	0.0028
	Shear only (Min 533 KIPS)	10	0.607	0.0022
	Moment Only (Max 11,284FT-KIPS)	10	0.449	0.003
	Moment Only (Min 10,687 FT-KIPS)	10	0.413	0.0028

Date:

Prepared By: HP 5/23/2012

Checked By: CFH

L-Pile Analysis Results Results for 9-foot Diameter Shafts

Loading Condition	Model #	Pile Head	
		Max Deflection (inch)	Rotation (Radians)
Shear only (Max 661 KIPS)	4	0.502	0.0018
Shear only (Ave 484 KIPS)	4	0.26	0.00104
Shear only (Min 307 KIPS)	4	0.1	0.00048
Shear only (Max 661 KIPS)	5	0.616	0.00209
Shear only (Ave 484 KIPS)	5	0.329	0.00124
Shear only (Min 307 KIPS)	5	0.132	0.00058
Shear only (Max 661 KIPS)	6	0.543	0.0019
Shear only (Ave 484 KIPS)	6	0.338	0.0012
Shear only (Min 307 KIPS)	6	0.16	0.00064
Moment Only (Max 21,960 FT-KIPS)	4	0.718	0.0042
Moment Only (Ave. 15,812 FT-KIPS)	4	0.438	0.0028
Moment Only (Min 9,665 FT-KIPS)	4	0.209	0.0015
Moment Only (Max 21,960 FT-KIPS)	5	0.831	0.0045
Moment Only (Ave. 15,812 FT-KIPS)	5	0.507	0.003
Moment Only (Min 9,665 FT-KIPS)	5	0.241	0.0016
Moment Only (Max 21,960 FT-KIPS)	6	0.781	0.00433
Moment Only (Ave. 15,812 FT-KIPS)	6	0.51	0.00296
Moment Only (Min 9,665 FT-KIPS)	6	0.262	0.00165
661 KIPS and 21,960 FT-KIPS	4	1.986	0.0078
661 KIPS and 21,960 FT-KIPS	5	2.32	0.0085
661 KIPS and 21,960 FT-KIPS	6	1.605	0.00698

L-Pile Input for 6 foot Diameter Piles – Shear Force Applied at Pile Head

L-Pile-6 foot- Q1-Abut1-Bent3-Model 1.lpo

LPILE Plus for Windows, Version 5.0 (5.0.46)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

(c) 1985-2010 by Ensoft, Inc.
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This program is licensed to:

Hamid Parsa
Kleinfelder

Files Used for Analysis

Path to file locations: U:\SParsa\ONGOING PROJECTS\Crosstown #4\Pile Analysis\Revised 6 and 8 and abutment\
Name of input data file: L-Pile-6 foot- Q1-Abut1-Bent3-Model 1.lpd
Name of output file: L-Pile-6 foot- Q1-Abut1-Bent3-Model 1.lpo
Name of plot output file: L-Pile-6 foot- Q1-Abut1-Bent3-Model 1.lpp
Name of runtime file: L-Pile-6 foot- Q1-Abut1-Bent3-Model 1.lpr

Time and Date of Analysis

Date: May 22, 2012 Time: 13:56:57

Problem Title

SR-4-6 ft- Model #1-Shear Only-Abutment 1 to Bent 3

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- Analysis includes computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 1080.00 in

Depth of ground surface below top of pile = 0.00 in

Slope angle of ground surface = 0.00 deg.

Structural properties of pile defined using 2 points

Point No.	Point Depth in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	72.00000000	1319167.	4072.0000	3000000.
2	1080.0000	72.00000000	1319167.	4072.0000	3000000.

Soil and Rock Layering Information

The soil profile is modelled using 8 layers

Layer 1 is stiff clay without free water
Distance from top of pile to top of layer = 0.000 in
Distance from top of pile to bottom of layer = 96.000 in

Layer 2 is stiff clay without free water
Distance from top of pile to top of layer = 96.000 in
Distance from top of pile to bottom of layer = 300.000 in

Layer 3 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 300.000 in
Distance from top of pile to bottom of layer = 420.000 in
p-y subgrade modulus k for top of soil layer = 90.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 90.000 lbs/in**3

Layer 4 is stiff clay without free water
Distance from top of pile to top of layer = 420.000 in
Distance from top of pile to bottom of layer = 780.000 in

Layer 5 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 780.000 in
Distance from top of pile to bottom of layer = 900.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 6 is stiff clay without free water
Distance from top of pile to top of layer = 900.000 in
Distance from top of pile to bottom of layer = 1140.000 in

Layer 7 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 1140.000 in
Distance from top of pile to bottom of layer = 1260.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 8 is stiff clay without free water
Distance from top of pile to top of layer = 1260.000 in
Distance from top of pile to bottom of layer = 1440.000 in

(Depth of lowest layer extends 360.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 16 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	0.00	0.06900
2	96.00	0.06900
3	96.00	0.03400
4	300.00	0.03400
5	300.00	0.03600
6	420.00	0.03600
7	420.00	0.03600
8	780.00	0.03600
9	780.00	0.03600
10	900.00	0.03600
11	900.00	0.03600
12	1140.00	0.03600
13	1140.00	0.03600
14	1260.00	0.03600
15	1260.00	0.03600
16	1440.00	0.03600

Shear Strength of Soils

Shear strength parameters with depth defined using 16 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	0.000	13.89000	0.00	0.00500	0.0
2	96.000	13.89000	0.00	0.00500	0.0
3	96.000	13.89000	0.00	0.00500	0.0
4	300.000	13.89000	0.00	0.00500	0.0
5	300.000	0.00000	33.00	------	------
6	420.000	0.00000	33.00	------	------
7	420.000	20.83000	0.00	0.00500	0.0
8	780.000	20.83000	0.00	0.00500	0.0

9	780.000	0.00000	38.00	------	------
10	900.000	0.00000	38.00	------	------
11	900.000	34.72000	0.00	0.00400	0.0
12	1140.000	34.72000	0.00	0.00400	0.0
13	1140.000	0.00000	38.00	------	------
14	1260.000	0.00000	38.00	------	------
15	1260.000	41.67000	0.00	0.00400	0.0
16	1440.000	41.67000	0.00	0.00400	0.0

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 2

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 247000.000 lbs
Bending moment at pile head = 0.000 in-lbs
Axial load at pile head = 0.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 300000.000 lbs
Bending moment at pile head = 0.000 in-lbs
Axial load at pile head = 0.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (Pile-head Condition Type 1)
Specified shear force at pile head = 247000.000 lbs
Specified moment at pile head = 0.000 in-lbs
Specified axial load at pile head = 0.000 lbs

LPILE Plus for Windows, Version 5.0 (5.0.46)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

(c) 1985-2010 by Ensoft, Inc.
All Rights Reserved

This program is licensed to:

Hamid Parsa
Kleinfelder

Files Used for Analysis

Path to file locations: U:\SParsa\ONGOING PROJECTS\Crosstown #4\Pile Analysis\Revised 6 and 8 and abutment\
Name of input data file: L-Pile-6 foot-Q-1Abut 1-Bent3-Model 2.lpd
Name of output file: L-Pile-6 foot-Q-1Abut 1-Bent3-Model 2.lpo
Name of plot output file: L-Pile-6 foot-Q-1Abut 1-Bent3-Model 2.lpp
Name of runtime file: L-Pile-6 foot-Q-1Abut 1-Bent3-Model 2.lpr

Time and Date of Analysis

Date: May 22, 2012 Time: 14:25:39

Problem Title

SR-4-6 ft- Model #2-Shear Only - Abutment 1 to Bent 3

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- Analysis includes computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 1080.00 in

Depth of ground surface below top of pile = 0.00 in

Slope angle of ground surface = 0.00 deg.

Structural properties of pile defined using 2 points

Point No.	Point Depth in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	72.00000000	1319167.	4072.0000	3000000.
2	1080.0000	72.00000000	1319167.	4062.0000	3000000.

Soil and Rock Layering Information

The soil profile is modelled using 6 layers

Layer 1 is stiff clay without free water
Distance from top of pile to top of layer = 0.000 in
Distance from top of pile to bottom of layer = 96.000 in

Layer 2 is stiff clay without free water
Distance from top of pile to top of layer = 96.000 in
Distance from top of pile to bottom of layer = 360.000 in

Layer 3 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 360.000 in
Distance from top of pile to bottom of layer = 840.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 4 is stiff clay without free water
Distance from top of pile to top of layer = 840.000 in
Distance from top of pile to bottom of layer = 1080.000 in

Layer 5 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 1080.000 in
Distance from top of pile to bottom of layer = 1200.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 6 is stiff clay without free water
Distance from top of pile to top of layer = 1200.000 in
Distance from top of pile to bottom of layer = 1440.000 in

(Depth of lowest layer extends 360.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 12 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	0.00	0.06900
2	96.00	0.06900
3	96.00	0.03600
4	360.00	0.03600
5	360.00	0.03600
6	840.00	0.03600
7	840.00	0.03600
8	1080.00	0.03600
9	1080.00	0.03600
10	1200.00	0.03600
11	1200.00	0.03600
12	1440.00	0.03600

Shear Strength of Soils

Shear strength parameters with depth defined using 12 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	0.000	20.83000	0.00	0.00500	0.0
2	96.000	20.83000	0.00	0.00500	0.0
3	96.000	20.83000	0.00	0.00500	0.0
4	360.000	20.83000	0.00	0.00500	0.0
5	360.000	0.00000	38.00	------	------
6	840.000	0.00000	38.00	------	------
7	840.000	34.72000	0.00	0.00400	0.0
8	1080.000	34.72000	0.00	0.00400	0.0
9	1080.000	0.00000	38.00	------	------
10	1200.000	0.00000	38.00	------	------
11	1200.000	41.67000	0.00	0.00400	0.0
12	1440.000	41.67000	0.00	0.00400	0.0

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 2

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 247000.000 lbs
Bending moment at pile head = 0.000 in-lbs
Axial load at pile head = 0.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 300000.000 lbs
Bending moment at pile head = 0.000 in-lbs
Axial load at pile head = 0.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (Pile-head Condition Type 1)
Specified shear force at pile head = 247000.000 lbs
Specified moment at pile head = 0.000 in-lbs
Specified axial load at pile head = 0.000 lbs

Depth X	Deflect. y	Moment M	Shear V	Slope S	Total Stress	Soil Res. Es*h p
in	in	lbs-in	lbs	Rad.	lbs/in**2	lbs/in F/L lbs/in
0.000	0.179226	-1.8834E-06	247000.	-0.0010963	5.1399E-11	-1502.8663 45280.6076
10.800	0.167386	2579953.	230612.	-0.0010928	70.4068	-1531.9610 98844.2989
21.600	0.155622	4981218.	213927.	-0.0010825	135.9372	-1557.8747 108115.
32.400	0.144005	7200772.	196980.	-0.0010658	196.5087	-1580.4914 118533.
43.200	0.132600	9235978.	179807.	-0.0010434	252.0494	-1599.6930 130292.

L-Pile-6 foot-Q-1- Bent 16 to 21 - Model 12.lpo

LPILE Plus for Windows, Version 5.0 (5.0.46)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

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This program is licensed to:

Hamid Parsa
Kleinfelder

Files Used for Analysis

Path to file locations: U:\SParsa\ONGOING PROJECTS\Crosstown #4\Pile Analysis\Revised 6 and 8 and abutment\
Name of input data file: L-Pile-6 foot-Q-1- Bent 16 to 21 - Model 12.lpd
Name of output file: L-Pile-6 foot-Q-1- Bent 16 to 21 - Model 12.lpo
Name of plot output file: L-Pile-6 foot-Q-1- Bent 16 to 21 - Model 12.lpp
Name of runtime file: L-Pile-6 foot-Q-1- Bent 16 to 21 - Model 12.lpr

Time and Date of Analysis

Date: May 22, 2012 Time: 14:50:27

Problem Title

SR-4-6 ft- Model #12 - Shear Only - Bent 16 to 21

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- Analysis includes computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 1080.00 in

Depth of ground surface below top of pile = 0.00 in

Slope angle of ground surface = 0.00 deg.

Structural properties of pile defined using 2 points

Point No.	Point Depth in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	72.00000000	1319167.	4072.0000	3000000.
2	1080.0000	72.00000000	1319167.	4062.0000	3000000.

Soil and Rock Layering Information

The soil profile is modelled using 5 layers

Layer 1 is stiff clay without free water
Distance from top of pile to top of layer = 0.000 in
Distance from top of pile to bottom of layer = 168.000 in

Layer 2 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 168.000 in
Distance from top of pile to bottom of layer = 240.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 3 is stiff clay without free water
Distance from top of pile to top of layer = 240.000 in
Distance from top of pile to bottom of layer = 540.000 in

Layer 4 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 540.000 in
Distance from top of pile to bottom of layer = 900.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 5 is stiff clay without free water
Distance from top of pile to top of layer = 900.000 in
Distance from top of pile to bottom of layer = 1080.000 in

(Depth of lowest layer extends 0.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 10 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	0.00	0.06900
2	168.00	0.06900
3	168.00	0.03600
4	240.00	0.03600
5	240.00	0.03600
6	540.00	0.03600
7	540.00	0.03600
8	900.00	0.03600
9	900.00	0.03600
10	1080.00	0.03600

Shear Strength of Soils

Shear strength parameters with depth defined using 10 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	0.000	17.36000	0.00	0.00500	0.0
2	168.000	17.36000	0.00	0.00500	0.0
3	168.000	0.00000	35.00	------	------
4	240.000	0.00000	35.00	------	------
5	240.000	17.36000	0.00	0.00500	0.0
6	540.000	17.36000	0.00	0.00500	0.0
7	540.000	0.00000	38.00	------	------
8	900.000	0.00000	38.00	------	------
9	900.000	41.67000	0.00	0.00400	0.0
10	1080.000	41.67000	0.00	0.00400	0.0

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 3

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 284000.000 lbs
Bending moment at pile head = 0.000 in-lbs
Axial load at pile head = 0.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 416000.000 lbs
Bending moment at pile head = 0.000 in-lbs
Axial load at pile head = 0.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Load Case Number 3

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 547000.000 lbs
Bending moment at pile head = 0.000 in-lbs
Axial load at pile head = 0.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (Pile-head Condition Type 1)
Specified shear force at pile head = 284000.000 lbs
Specified moment at pile head = 0.000 in-lbs
Specified axial load at pile head = 0.000 lbs

Depth X	Deflect. y	Moment M	Shear V	Slope S	Total Stress	Soil Res. Es*h p
in	in	lbs-in	lbs	Rad.	lbs/in**2	lbs/in F/L lbs/in
0.000	0.316833	-1.5068E-05	284000.	-0.0016755	4.1119E-10	-1444.2037 24614.5642
10.800	0.298738	2982974.	268214.	-0.0016714	81.4052	-1479.0683 53471.4305
21.600	0.280731	5793430.	252066.	-0.0016594	158.1024	-1511.3367 58142.6711
32.400	0.262894	8427603.	235584.	-0.0016400	229.9888	-1540.9195 63302.7045
43.200	0.245307	1.0882E+07	218797.	-0.0016137	296.9704	-1567.7272

L-Pile-6 foot-Q-1-Bent 16 to 21-Model 13Corrected.lpo

LPILE Plus for Windows, Version 5.0 (5.0.46)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

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This program is licensed to:

Hamid Parsa
Kleinfelder

Files Used for Analysis

Path to file locations: U:\SParsa\ONGOING PROJECTS\Crosstown #4\Pile Analysis\Revised 6 and 8 and abutment\
Name of input data file: L-Pile-6 foot-Q-1-Bent 16 to 21-Model 13Corrected.lpd
Name of output file: L-Pile-6 foot-Q-1-Bent 16 to 21-Model 13Corrected.lpo
Name of plot output file: L-Pile-6 foot-Q-1-Bent 16 to 21-Model 13Corrected.lpp
Name of runtime file: L-Pile-6 foot-Q-1-Bent 16 to 21-Model 13Corrected.lpr

Time and Date of Analysis

Date: May 22, 2012 Time: 16:55:42

Problem Title

SR-4-6 ft- Model #13-Shear Only- Bents 16 to 21

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- Analysis includes computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 1080.00 in

Depth of ground surface below top of pile = 0.00 in

Slope angle of ground surface = 0.00 deg.

Structural properties of pile defined using 2 points

Point No.	Point Depth in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	72.00000000	1319167.	4072.0000	3000000.
2	1080.0000	72.00000000	1319167.	4062.0000	3000000.

Soil and Rock Layering Information

The soil profile is modelled using 6 layers

Layer 1 is stiff clay without free water
Distance from top of pile to top of layer = 0.000 in
Distance from top of pile to bottom of layer = 60.000 in

Layer 2 is stiff clay without free water
Distance from top of pile to top of layer = 60.000 in
Distance from top of pile to bottom of layer = 180.000 in

Layer 3 is stiff clay without free water
Distance from top of pile to top of layer = 180.000 in
Distance from top of pile to bottom of layer = 480.000 in

Layer 4 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 480.000 in
Distance from top of pile to bottom of layer = 840.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 5 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 840.000 in
Distance from top of pile to bottom of layer = 960.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 6 is stiff clay without free water
Distance from top of pile to top of layer = 960.000 in
Distance from top of pile to bottom of layer = 1440.000 in

(Depth of lowest layer extends 360.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 12 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	0.00	0.06900
2	60.00	0.06900
3	60.00	0.07200
4	180.00	0.07200
5	180.00	0.03600
6	480.00	0.03600
7	480.00	0.03600
8	840.00	0.03600
9	840.00	0.03600
10	960.00	0.03600
11	960.00	0.03600
12	1440.00	0.03600

Shear Strength of Soils

Shear strength parameters with depth defined using 12 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	0.000	10.40000	0.00	0.00500	0.0
2	60.000	10.40000	0.00	0.00500	0.0
3	60.000	13.89000	0.00	0.00500	0.0
4	180.000	13.89000	0.00	0.00500	0.0
5	180.000	20.83000	0.00	0.00500	0.0
6	480.000	20.83000	0.00	0.00500	0.0
7	480.000	0.00000	36.00	------	------
8	840.000	0.00000	36.00	------	------
9	840.000	0.00000	38.00	------	------
10	960.000	0.00000	38.00	------	------
11	960.000	41.67000	0.00	0.00400	0.0
12	1440.000	41.67000	0.00	0.00400	0.0

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 3

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 284000.000 lbs
Bending moment at pile head = 0.000 in-lbs
Axial load at pile head = 0.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 416000.000 lbs
Bending moment at pile head = 0.000 in-lbs
Axial load at pile head = 0.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Load Case Number 3

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 547000.000 lbs
Bending moment at pile head = 0.000 in-lbs
Axial load at pile head = 0.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (Pile-head Condition Type 1)
Specified shear force at pile head = 284000.000 lbs
Specified moment at pile head = 0.000 in-lbs
Specified axial load at pile head = 0.000 lbs

Depth X	Deflect. y	Moment M	Shear V	Slope S	Total Stress	Soil Res. Es*h p
in	in	lbs-in	lbs	Rad.	lbs/in**2	lbs/in F/L lbs/in
0.000	0.450862	1.6951E-05	284000.	-0.0021964	4.6259E-10	-944.9570

L-Pile-6 foot-Abut 22-Model 13Corrected.lpo

LPILE Plus for Windows, Version 5.0 (5.0.46)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

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This program is licensed to:

Hamid Parsa
Kleinfelder

Files Used for Analysis

Path to file locations: U:\SParsa\ONGOING PROJECTS\Crosstown #4\Pile Analysis\Revised 6 and 8 and abutment\
Name of input data file: L-Pile-6 foot-Abut 22-Model 13Corrected.lpd
Name of output file: L-Pile-6 foot-Abut 22-Model 13Corrected.lpo
Name of plot output file: L-Pile-6 foot-Abut 22-Model 13Corrected.lpp
Name of runtime file: L-Pile-6 foot-Abut 22-Model 13Corrected.lpr

Time and Date of Analysis

Date: May 22, 2012 Time: 17:12:38

Problem Title

SR-4-6 ft- Model #13-Shear and Fixed Hea

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- Analysis includes computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 1080.00 in

Depth of ground surface below top of pile = 0.00 in

Slope angle of ground surface = 0.00 deg.

Structural properties of pile defined using 2 points

Point No.	Point Depth in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	72.00000000	1319167.	4072.0000	3000000.
2	1080.0000	72.00000000	1319167.	4062.0000	3000000.

Soil and Rock Layering Information

The soil profile is modelled using 6 layers

Layer 1 is stiff clay without free water
Distance from top of pile to top of layer = 0.000 in
Distance from top of pile to bottom of layer = 60.000 in

Layer 2 is stiff clay without free water
Distance from top of pile to top of layer = 60.000 in
Distance from top of pile to bottom of layer = 180.000 in

Layer 3 is stiff clay without free water
Distance from top of pile to top of layer = 180.000 in
Distance from top of pile to bottom of layer = 480.000 in

Layer 4 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 480.000 in
Distance from top of pile to bottom of layer = 840.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 5 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 840.000 in
Distance from top of pile to bottom of layer = 960.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 6 is stiff clay without free water
Distance from top of pile to top of layer = 960.000 in
Distance from top of pile to bottom of layer = 1440.000 in

(Depth of lowest layer extends 360.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 12 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	0.00	0.06900
2	60.00	0.06900
3	60.00	0.07200
4	180.00	0.07200
5	180.00	0.03600
6	480.00	0.03600
7	480.00	0.03600
8	840.00	0.03600
9	840.00	0.03600
10	960.00	0.03600
11	960.00	0.03600
12	1440.00	0.03600

Shear Strength of Soils

Shear strength parameters with depth defined using 12 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	0.000	10.40000	0.00	0.00500	0.0
2	60.000	10.40000	0.00	0.00500	0.0
3	60.000	13.89000	0.00	0.00500	0.0
4	180.000	13.89000	0.00	0.00500	0.0
5	180.000	20.83000	0.00	0.00500	0.0
6	480.000	20.83000	0.00	0.00500	0.0
7	480.000	0.00000	36.00	------	------
8	840.000	0.00000	36.00	------	------
9	840.000	0.00000	38.00	------	------
10	960.000	0.00000	38.00	------	------
11	960.000	41.67000	0.00	0.00400	0.0
12	1440.000	41.67000	0.00	0.00400	0.0

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load Case Number 1

Pile-head boundary conditions are Shear and Slope (BC Type 2)
Shear force at pile head = 115000.000 lbs
Slope at pile head = 0.000 in/in
Axial load at pile head = 620000.000 lbs

(Zero slope for this load indicates fixed-head condition)

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Slope (Pile-head Condition Type 2)
Specified shear force at pile head = 115000.000 lbs
Specified slope at pile head = 0.000E+00 in/in
Specified axial load at pile head = 620000.000 lbs

(Zero slope for this load indicates fixed-head conditions)

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Soil Res. Es*h p lbs/in	F/L lbs/in
0.000	0.017217	-9018980.	115000.	-4.8187E-19	398.3868	-417.5815	130968.
10.800	0.017085	-7801250.	110384.	-2.2951E-05	365.1588	-437.1463	276342.
21.600	0.016722	-6634368.	105567.	-4.2649E-05	333.3184	-455.0691	293914.
32.400	0.016163	-5520443.	100564.	-5.9234E-05	302.9232	-471.3114	314920.
43.200	0.015442	-4461391.	95395.5168	-7.2854E-05	274.0254	-485.8248	339776.
54.000	0.014590	-3458924.	90079.8692	-8.3661E-05	246.6719	-498.5544	369054.
64.800	0.013635	-2514545.	83965.7580	-9.1812E-05	220.9036	-633.6884	501924.
75.600	0.012607	-1644034.	77067.1591	-9.7486E-05	197.1511	-643.8299	551567.
86.400	0.011529	-848589.	70071.9550	-0.0001009	175.4472	-651.5783	

L-Pile Input for 6 foot Diameter Piles – Moment Applied at Pile Head

L-Pile-6 foot-Q2-Abut1-Bent3-Model 1.lpo

LPILE Plus for Windows, Version 5.0 (5.0.46)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

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This program is licensed to:

Hamid Parsa
Kleinfelder

Files Used for Analysis

Path to file locations: U:\SParsa\ONGOING PROJECTS\Crosstown #4\Pile Analysis\Revised 6 and 8 and abutment\
Name of input data file: L-Pile-6 foot-Q2-Abut1-Bent3-Model 1.lpd
Name of output file: L-Pile-6 foot-Q2-Abut1-Bent3-Model 1.lpo
Name of plot output file: L-Pile-6 foot-Q2-Abut1-Bent3-Model 1.lpp
Name of runtime file: L-Pile-6 foot-Q2-Abut1-Bent3-Model 1.lpr

Time and Date of Analysis

Date: May 22, 2012 Time: 14:11:11

Problem Title

SR-4-6 ft- Model #1-Moment Only - Abutment 1 to Bent 3

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- Analysis includes computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 1080.00 in

Depth of ground surface below top of pile = 0.00 in

Slope angle of ground surface = 0.00 deg.

Structural properties of pile defined using 2 points

Point No.	Point Depth in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	72.00000000	1319167.	4072.0000	3000000.
2	1080.0000	72.00000000	1319167.	4072.0000	3000000.

Soil and Rock Layering Information

The soil profile is modelled using 8 layers

Layer 1 is stiff clay without free water
Distance from top of pile to top of layer = 0.000 in
Distance from top of pile to bottom of layer = 96.000 in

Layer 2 is stiff clay without free water
Distance from top of pile to top of layer = 96.000 in
Distance from top of pile to bottom of layer = 300.000 in

Layer 3 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 300.000 in
Distance from top of pile to bottom of layer = 420.000 in
p-y subgrade modulus k for top of soil layer = 90.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 90.000 lbs/in**3

Layer 4 is stiff clay without free water
Distance from top of pile to top of layer = 420.000 in
Distance from top of pile to bottom of layer = 780.000 in

Layer 5 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 780.000 in
Distance from top of pile to bottom of layer = 900.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 6 is stiff clay without free water
Distance from top of pile to top of layer = 900.000 in
Distance from top of pile to bottom of layer = 1140.000 in

Layer 7 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 1140.000 in
Distance from top of pile to bottom of layer = 1260.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 8 is stiff clay without free water
Distance from top of pile to top of layer = 1260.000 in
Distance from top of pile to bottom of layer = 1440.000 in

(Depth of lowest layer extends 360.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 16 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	0.00	0.06900
2	96.00	0.06900
3	96.00	0.03400
4	300.00	0.03400
5	300.00	0.03600
6	420.00	0.03600
7	420.00	0.03600
8	780.00	0.03600
9	780.00	0.03600
10	900.00	0.03600
11	900.00	0.03600
12	1140.00	0.03600
13	1140.00	0.03600
14	1260.00	0.03600
15	1260.00	0.03600
16	1440.00	0.03600

Shear Strength of Soils

Shear strength parameters with depth defined using 16 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	0.000	13.89000	0.00	0.00500	0.0
2	96.000	13.89000	0.00	0.00500	0.0
3	96.000	13.89000	0.00	0.00500	0.0
4	300.000	13.89000	0.00	0.00500	0.0
5	300.000	0.00000	33.00	------	------
6	420.000	0.00000	33.00	------	------
7	420.000	20.83000	0.00	0.00500	0.0
8	780.000	20.83000	0.00	0.00500	0.0

9	780.000	0.00000	38.00	------	------
10	900.000	0.00000	38.00	------	------
11	900.000	34.72000	0.00	0.00400	0.0
12	1140.000	34.72000	0.00	0.00400	0.0
13	1140.000	0.00000	38.00	------	------
14	1260.000	0.00000	38.00	------	------
15	1260.000	41.67000	0.00	0.00400	0.0
16	1440.000	41.67000	0.00	0.00400	0.0

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 2

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 0.000 lbs
Bending moment at pile head = 48816000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 0.000 lbs
Bending moment at pile head = 54348000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (Pile-head Condition Type 1)

Specified shear force at pile head = 0.000 lbs
Specified moment at pile head = 48816000.000 in-lbs
Specified axial load at pile head = 0.000 lbs

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Soil Res. Es*h p lbs/in	F/L lbs/in
0.000	0.273008	4.8816E+07	0.0000	-0.0025023	1332.1861	-1113.3189	22021.0115
10.800	0.246703	4.8751E+07	-12124.8549	-0.0023691	1330.4142	-1132.0246	49556.9444
21.600	0.221835	4.8554E+07	-24435.2667	-0.0022364	1325.0390	-1147.6813	55874.6404
32.400	0.198398	4.8223E+07	-36897.6665	-0.0021043	1316.0106	-1160.1705	63155.0967
43.200	0.176382	4.7757E+07	-49477.1223	-0.0019733	1303.2892	-1169.3584	71600.6600
54.000	0.155774	4.7155E+07	-62137.1560	-0.0018438	1286.8456	-1175.0923	81470.7335
64.800	0.136555	4.6415E+07	-74839.5049	-0.0017162	1266.6616	-1177.1945	93103.0597
75.600	0.118705	4.5538E+07	-87543.8070	-0.0015907	1242.7305	-1175.4540	106945.
86.400	0.102196	4.4524E+07	-100207.	-0.0014678	1215.0578	-1169.6142	123604.
97.200	0.087000	4.3374E+07	-112777.	-0.0013479	1183.6621	-1158.1877	143775.
108.000	0.073082	4.2088E+07	-125165.	-0.0012313	1148.5798	-1135.8903	167860.
118.800	0.060405	4.0670E+07	-137287.	-0.0011183	1109.8818	-1108.8922	198263.
129.600	0.048926	3.9123E+07	-149088.	-0.0010095	1067.6541	-1076.4902	237625.
140.400	0.038601	3.7450E+07	-160505.	-0.0009050	1021.9998	-1037.6563	290324.
151.200	0.029379	3.5656E+07	-171458.	-0.0008052	973.0426	-990.7910	364227.
162.000	0.021208	3.3746E+07	-181848.	-0.0007105	920.9316	-933.1833	475219.
172.800	0.014032	3.1728E+07	-191529.	-0.0006212	865.8501	-859.6180	661644.
183.600	0.007790	2.9609E+07	-200262.	-0.0005375	808.0324	-757.6326	1050335.
194.400	0.002422	2.7402E+07	-207473.	-0.0004597	747.8031	-577.7773	2576632.
205.200	-0.002139	2.5128E+07	-207516.	-0.0003880	685.7347	569.8317	2876924.
216.000	-0.005959	2.2920E+07	-200382.	-0.0003225	625.4801	751.3171	1361566.
226.800	-0.009104	2.0799E+07	-191727.	-0.0002628	567.6170	851.5125	1010111.
237.600	-0.011636	1.8779E+07	-182147.	-0.0002088	512.4644	922.5505	856264.
248.400	-0.013614	1.6865E+07	-171887.	-0.0001602	460.2483	977.3117	775281.
259.200	-0.015096	1.5066E+07	-161096.	-0.0001166	411.1431	1021.1532	730573.
270.000	-0.016133	1.3385E+07	-149875.	-7.7774E-05	365.2884	1056.8368	

L-Pile-6 foot-Q-2-Abut 1-Bent3-Model 2.lpo

LPILE Plus for Windows, Version 5.0 (5.0.46)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

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This program is licensed to:

Hamid Parsa
Kleinfelder

Files Used for Analysis

Path to file locations: U:\SParsa\ONGOING PROJECTS\Crosstown #4\Pile Analysis\Revised 6 and 8 and abutment\
Name of input data file: L-Pile-6 foot-Q-2-Abut 1-Bent3-Model 2.lpd
Name of output file: L-Pile-6 foot-Q-2-Abut 1-Bent3-Model 2.lpo
Name of plot output file: L-Pile-6 foot-Q-2-Abut 1-Bent3-Model 2.lpp
Name of runtime file: L-Pile-6 foot-Q-2-Abut 1-Bent3-Model 2.lpr

Time and Date of Analysis

Date: May 22, 2012 Time: 14:32:11

Problem Title

SR-4-6 ft- Model #2- Moment Only - Abutment 1 to Bent 3

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- Analysis includes computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 1080.00 in

Depth of ground surface below top of pile = 0.00 in

Slope angle of ground surface = 0.00 deg.

Structural properties of pile defined using 2 points

Point No.	Point Depth in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	72.00000000	1319167.	4072.0000	3000000.
2	1080.0000	72.00000000	1319167.	4062.0000	3000000.

Soil and Rock Layering Information

The soil profile is modelled using 6 layers

Layer 1 is stiff clay without free water
Distance from top of pile to top of layer = 0.000 in
Distance from top of pile to bottom of layer = 96.000 in

Layer 2 is stiff clay without free water
Distance from top of pile to top of layer = 96.000 in
Distance from top of pile to bottom of layer = 360.000 in

Layer 3 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 360.000 in
Distance from top of pile to bottom of layer = 840.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 4 is stiff clay without free water
Distance from top of pile to top of layer = 840.000 in
Distance from top of pile to bottom of layer = 1080.000 in

Layer 5 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 1080.000 in
Distance from top of pile to bottom of layer = 1200.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 6 is stiff clay without free water
Distance from top of pile to top of layer = 1200.000 in
Distance from top of pile to bottom of layer = 1440.000 in

(Depth of lowest layer extends 360.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 12 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	0.00	0.06900
2	96.00	0.06900
3	96.00	0.03600
4	360.00	0.03600
5	360.00	0.03600
6	840.00	0.03600
7	840.00	0.03600
8	1080.00	0.03600
9	1080.00	0.03600
10	1200.00	0.03600
11	1200.00	0.03600
12	1440.00	0.03600

Shear Strength of Soils

Shear strength parameters with depth defined using 12 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	0.000	20.83000	0.00	0.00500	0.0
2	96.000	20.83000	0.00	0.00500	0.0
3	96.000	20.83000	0.00	0.00500	0.0
4	360.000	20.83000	0.00	0.00500	0.0
5	360.000	0.00000	38.00	------	------
6	840.000	0.00000	38.00	------	------
7	840.000	34.72000	0.00	0.00400	0.0
8	1080.000	34.72000	0.00	0.00400	0.0
9	1080.000	0.00000	38.00	------	------
10	1200.000	0.00000	38.00	------	------
11	1200.000	41.67000	0.00	0.00400	0.0
12	1440.000	41.67000	0.00	0.00400	0.0

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 2

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 0.000 lbs
Bending moment at pile head = 48816000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 0.000 lbs
Bending moment at pile head = 54348000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (Pile-head Condition Type 1)
Specified shear force at pile head = 0.000 lbs
Specified moment at pile head = 48816000.000 in-lbs
Specified axial load at pile head = 0.000 lbs

Depth X	Deflect. y	Moment M	Shear V	Slope S	Total Stress	Soil Res. p	Es*h F/L
in	in	lbs-in	lbs	Rad.	lbs/in**2	lbs/in	lbs/in
0.000	0.206002	4.8816E+07	0.0000	-0.0021613	1332.1861	-1556.0867	40790.2396
10.800	0.183379	4.8725E+07	-16866.2874	-0.0020282	1329.7096	-1567.2999	92305.1450
21.600	0.162192	4.8452E+07	-33829.6067	-0.0018956	1322.2441	-1574.0555	104813.

L-Pile-6 foot-Q-2- Bent 16 to 21 - Model 12.lpo

LPILE Plus for Windows, Version 5.0 (5.0.46)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

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This program is licensed to:

Hamid Parsa
Kleinfelder

Files Used for Analysis

Path to file locations: U:\SParsa\ONGOING PROJECTS\Crosstown #4\Pile Analysis\Revised 6 and 8 and abutment\
Name of input data file: L-Pile-6 foot-Q-2- Bent 16 to 21 - Model 12.lpd
Name of output file: L-Pile-6 foot-Q-2- Bent 16 to 21 - Model 12.lpo
Name of plot output file: L-Pile-6 foot-Q-2- Bent 16 to 21 - Model 12.lpp
Name of runtime file: L-Pile-6 foot-Q-2- Bent 16 to 21 - Model 12.lpr

Time and Date of Analysis

Date: May 22, 2012 Time: 15:01:51

Problem Title

SR-4-6 ft- Model #12 - Moment Only - Bent 16 to 21

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- Analysis includes computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 1080.00 in

Depth of ground surface below top of pile = 0.00 in

Slope angle of ground surface = 0.00 deg.

Structural properties of pile defined using 2 points

Point No.	Point Depth in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	72.00000000	1319167.	4072.0000	3000000.
2	1080.0000	72.00000000	1319167.	4062.0000	3000000.

Soil and Rock Layering Information

The soil profile is modelled using 5 layers

Layer 1 is stiff clay without free water
Distance from top of pile to top of layer = 0.000 in
Distance from top of pile to bottom of layer = 168.000 in

Layer 2 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 168.000 in
Distance from top of pile to bottom of layer = 240.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 3 is stiff clay without free water
Distance from top of pile to top of layer = 240.000 in
Distance from top of pile to bottom of layer = 540.000 in

Layer 4 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 540.000 in
Distance from top of pile to bottom of layer = 900.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 5 is stiff clay without free water
Distance from top of pile to top of layer = 900.000 in
Distance from top of pile to bottom of layer = 1080.000 in

(Depth of lowest layer extends 0.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 10 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	0.00	0.06900
2	168.00	0.06900
3	168.00	0.03600
4	240.00	0.03600
5	240.00	0.03600
6	540.00	0.03600
7	540.00	0.03600
8	900.00	0.03600
9	900.00	0.03600
10	1080.00	0.03600

Shear Strength of Soils

Shear strength parameters with depth defined using 10 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	0.000	17.36000	0.00	0.00500	0.0
2	168.000	17.36000	0.00	0.00500	0.0
3	168.000	0.00000	35.00	------	------
4	240.000	0.00000	35.00	------	------
5	240.000	17.36000	0.00	0.00500	0.0
6	540.000	17.36000	0.00	0.00500	0.0
7	540.000	0.00000	38.00	------	------
8	900.000	0.00000	38.00	------	------
9	900.000	41.67000	0.00	0.00400	0.0
10	1080.000	41.67000	0.00	0.00400	0.0

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 3

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 0.000 lbs
Bending moment at pile head = 53856000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 0.000 lbs
Bending moment at pile head = 55746000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 3

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 0.000 lbs
Bending moment at pile head = 57636000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (Pile-head Condition Type 1)
Specified shear force at pile head = 0.000 lbs
Specified moment at pile head = 53856000.000 in-lbs
Specified axial load at pile head = 0.000 lbs

Depth	Deflect.	Moment	Shear	Slope	Total	Soil Res.
X	y	M	V	S	Stress	p
in	in	lbs-in	lbs	Rad.	lbs/in**2	lbs/in
0.000	0.270708	5.3856E+07	0.0000	-0.0025973	1469.7275	-1388.5028
10.800	0.243451	5.3775E+07	-15086.5458	-0.0024504	1467.5176	-1405.3019

Es*h	F/L
27697.4679	

L-Pile-6 foot-Q-2-Bent 16 to 21-Model 13Corrected.lpo

LPILE Plus for Windows, Version 5.0 (5.0.46)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

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This program is licensed to:

Hamid Parsa
Kleinfelder

Files Used for Analysis

Path to file locations: U:\SParsa\ONGOING PROJECTS\Crosstown #4\Pile Analysis\Revised 6 and 8 and abutment\
Name of input data file: L-Pile-6 foot-Q-2-Bent 16 to 21-Model 13Corrected.lpd
Name of output file: L-Pile-6 foot-Q-2-Bent 16 to 21-Model 13Corrected.lpo
Name of plot output file: L-Pile-6 foot-Q-2-Bent 16 to 21-Model 13Corrected.lpp
Name of runtime file: L-Pile-6 foot-Q-2-Bent 16 to 21-Model 13Corrected.lpr

Time and Date of Analysis

Date: May 22, 2012 Time: 17:05:46

Problem Title

SR-4-6 ft- Model #13-Moment Only- Bents 16 to 21

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- Analysis includes computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 1080.00 in

Depth of ground surface below top of pile = 0.00 in

Slope angle of ground surface = 0.00 deg.

Structural properties of pile defined using 2 points

Point No.	Point Depth in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	72.00000000	1319167.	4072.0000	3000000.
2	1080.0000	72.00000000	1319167.	4062.0000	3000000.

Soil and Rock Layering Information

The soil profile is modelled using 6 layers

Layer 1 is stiff clay without free water
Distance from top of pile to top of layer = 0.000 in
Distance from top of pile to bottom of layer = 60.000 in

Layer 2 is stiff clay without free water
Distance from top of pile to top of layer = 60.000 in
Distance from top of pile to bottom of layer = 180.000 in

Layer 3 is stiff clay without free water
Distance from top of pile to top of layer = 180.000 in
Distance from top of pile to bottom of layer = 480.000 in

Layer 4 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 480.000 in
Distance from top of pile to bottom of layer = 840.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 5 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 840.000 in
Distance from top of pile to bottom of layer = 960.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 6 is stiff clay without free water
Distance from top of pile to top of layer = 960.000 in
Distance from top of pile to bottom of layer = 1440.000 in

(Depth of lowest layer extends 360.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 12 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	0.00	0.06900
2	60.00	0.06900
3	60.00	0.07200
4	180.00	0.07200
5	180.00	0.03600
6	480.00	0.03600
7	480.00	0.03600
8	840.00	0.03600
9	840.00	0.03600
10	960.00	0.03600
11	960.00	0.03600
12	1440.00	0.03600

Shear Strength of Soils

Shear strength parameters with depth defined using 12 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	0.000	10.40000	0.00	0.00500	0.0
2	60.000	10.40000	0.00	0.00500	0.0
3	60.000	13.89000	0.00	0.00500	0.0
4	180.000	13.89000	0.00	0.00500	0.0
5	180.000	20.83000	0.00	0.00500	0.0
6	480.000	20.83000	0.00	0.00500	0.0
7	480.000	0.00000	36.00	------	------
8	840.000	0.00000	36.00	------	------
9	840.000	0.00000	38.00	------	------
10	960.000	0.00000	38.00	------	------
11	960.000	41.67000	0.00	0.00400	0.0
12	1440.000	41.67000	0.00	0.00400	0.0

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 3

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 0.000 lbs
Bending moment at pile head = 53856000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 0.000 lbs
Bending moment at pile head = 55746000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 3

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 0.000 lbs
Bending moment at pile head = 57636000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (Pile-head Condition Type 1)
Specified shear force at pile head = 0.000 lbs
Specified moment at pile head = 53856000.000 in-lbs
Specified axial load at pile head = 0.000 lbs

Depth	Deflect.	Moment	Shear	Slope	Total	Soil Res.
X	y	M	V	S	Stress	Es*h p

L-Pile Input for 8 foot Diameter Piles – Shear Force Applied at Pile Head

LPILE Plus for Windows, Version 5.0 (5.0.46)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

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This program is licensed to:

Hamid Parsa
Kleinfelder

Files Used for Analysis

Path to file locations: U:\SParsa\ONGOING PROJECTS\Crosstown #4\Pile Analysis\Revised 6 and 8 and abutment\
Name of input data file: L-Pile-8 foot--Model 2-Shear Only.lpd
Name of output file: L-Pile-8 foot--Model 2-Shear Only.lpo
Name of plot output file: L-Pile-8 foot--Model 2-Shear Only.lpp
Name of runtime file: L-Pile-8 foot--Model 2-Shear Only.lpr

Time and Date of Analysis

Date: May 22, 2012 Time: 17:44:59

Problem Title

SR-4-8 ft- Model #2 -Shear Only-Bents 4 and 5

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- Analysis includes computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 1080.00 in

Depth of ground surface below top of pile = 0.00 in

Slope angle of ground surface = 0.00 deg.

Structural properties of pile defined using 2 points

Point No.	Point Depth in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	96.00000000	4169220.	7238.0000	3000000.
2	1080.0000	96.00000000	4169220.	7238.0000	3000000.

Soil and Rock Layering Information

The soil profile is modelled using 8 layers

Layer 1 is stiff clay with water-induced erosion
Distance from top of pile to top of layer = 0.000 in
Distance from top of pile to bottom of layer = 36.000 in
p-y subgrade modulus k for top of soil layer = 500.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 500.000 lbs/in**3

Layer 2 is stiff clay without free water
Distance from top of pile to top of layer = 36.000 in
Distance from top of pile to bottom of layer = 96.000 in

Layer 3 is stiff clay without free water
Distance from top of pile to top of layer = 96.000 in
Distance from top of pile to bottom of layer = 240.000 in

Layer 4 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 240.000 in
Distance from top of pile to bottom of layer = 720.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 5 is stiff clay with water-induced erosion
Distance from top of pile to top of layer = 720.000 in
Distance from top of pile to bottom of layer = 756.000 in
p-y subgrade modulus k for top of soil layer = 1000.000 lbs/in**3

p-y subgrade modulus k for bottom of layer = 1000.000 lbs/in**3

Layer 6 is stiff clay without free water
Distance from top of pile to top of layer = 756.000 in
Distance from top of pile to bottom of layer = 960.000 in

Layer 7 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 960.000 in
Distance from top of pile to bottom of layer = 1080.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 8 is stiff clay without free water
Distance from top of pile to top of layer = 1080.000 in
Distance from top of pile to bottom of layer = 1320.000 in

(Depth of lowest layer extends 240.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 16 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	0.00	0.03600
2	36.00	0.03600
3	36.00	0.03600
4	96.00	0.03600
5	96.00	0.03600
6	240.00	0.03600
7	240.00	0.03600
8	720.00	0.03600
9	720.00	0.03600
10	756.00	0.03600
11	756.00	0.03600
12	960.00	0.03600
13	960.00	0.03600
14	1080.00	0.03600
15	1080.00	0.03600
16	1320.00	0.03600

Shear Strength of Soils

Shear strength parameters with depth defined using 16 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	0.000	7.00000	0.00	0.07000	0.0
2	36.000	7.00000	0.00	0.07000	0.0
3	36.000	20.83000	0.00	0.00500	0.0
4	96.000	20.83000	0.00	0.00500	0.0
5	96.000	20.83000	0.00	0.00500	0.0
6	240.000	20.83000	0.00	0.00500	0.0

7	240.000	0.00000	38.00	------	------
8	720.000	0.00000	38.00	------	------
9	720.000	17.36000	0.00	0.00500	0.0
10	756.000	17.36000	0.00	0.00500	0.0
11	756.000	34.72000	0.00	0.00400	0.0
12	960.000	34.72000	0.00	0.00400	0.0
13	960.000	0.00000	38.00	------	------
14	1080.000	0.00000	38.00	------	------
15	1080.000	41.67000	0.00	0.00400	0.0
16	1320.000	41.67000	0.00	0.00400	0.0

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 2

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 427000.000 lbs
Bending moment at pile head = 0.000 in-lbs
Axial load at pile head = 0.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 493000.000 lbs
Bending moment at pile head = 0.000 in-lbs
Axial load at pile head = 0.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (Pile-head Condition Type 1)
Specified shear force at pile head = 427000.000 lbs
Specified moment at pile head = 0.000 in-lbs

Specified axial load at pile head = 0.000 lbs

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Soil Res. Es*h p lbs/in	F/L lbs/in
0.000	0.373056	-1.1905E-05	427000.	-0.0015149	1.3706E-10	0.0000	0.0000
10.800	0.356695	4611600.	426008.	-0.0015129	53.0931	-183.7694	5564.1570
21.600	0.340378	9201765.	423893.	-0.0015069	105.9394	-207.7925	6593.1481
32.400	0.324145	1.3768E+07	421527.	-0.0014970	158.5067	-230.3708	7675.5802
43.200	0.308042	1.8307E+07	408522.	-0.0014832	210.7646	-2178.0377	76362.3880
54.000	0.292109	2.2592E+07	384870.	-0.0014655	260.0977	-2201.9248	81410.7079
64.800	0.276387	2.6620E+07	360972.	-0.0014443	306.4739	-2223.5701	86887.5721
75.600	0.260913	3.0389E+07	336853.	-0.0014197	349.8642	-2242.9215	92841.6561
86.400	0.245722	3.3896E+07	312538.	-0.0013919	390.2424	-2259.9255	99328.5327
97.200	0.230847	3.7140E+07	288213.	-0.0013612	427.5859	-2244.6065	105012.
108.000	0.216319	4.0121E+07	263903.	-0.0013279	461.9152	-2257.2338	112695.
118.800	0.202165	4.2840E+07	239471.	-0.0012921	493.2133	-2267.3579	121126.
129.600	0.188411	4.5294E+07	214942.	-0.0012540	521.4667	-2274.9193	130402.
140.400	0.175078	4.7483E+07	190347.	-0.0012140	546.6651	-2279.8558	140637.
151.200	0.162189	4.9405E+07	165712.	-0.0011721	568.8020	-2282.1022	151963.
162.000	0.149760	5.1062E+07	141068.	-0.0011288	587.8743	-2281.5889	164537.
172.800	0.137808	5.2452E+07	116445.	-0.0010841	603.8827	-2278.2416	178546.
183.600	0.126345	5.3577E+07	91873.7543	-0.0010383	616.8318	-2271.9793	194210.
194.400	0.115381	5.4437E+07	67386.4155	-0.0009917	626.7299	-2262.7131	211797.
205.200	0.104925	5.5033E+07	43015.9100	-0.0009444	633.5894	-2250.3435	231629.
216.000	0.094982	5.5366E+07	18796.3611	-0.0008967	637.4270	-2234.7581	254104.
226.800	0.085556	5.5439E+07	-5236.7984	-0.0008489	638.2637	-2215.8270	279712.
237.600	0.076646	5.5253E+07	-29046.6123	-0.0008011	636.1248	-2193.3978	309066.
248.400	0.068252	5.4811E+07	-51586.7140	-0.0007536	631.0404	-1980.6951	313420.
259.200	0.060369	5.4139E+07	-72182.9493	-0.0007065	623.2962	-1833.4225	328000.
270.000	0.052991	5.3252E+07	-91160.1840	-0.0006602	613.0900	-1680.8802	342580.
280.800	0.046109	5.2170E+07	-108471.	-0.0006147	600.6265	-1524.8395	

L-Pile-8 foot-Q-1-Model 3-Shear Only.lpo

LPILE Plus for Windows, Version 5.0 (5.0.46)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

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This program is licensed to:

Hamid Parsa
Kleinfelder

Files Used for Analysis

Path to file locations: U:\SParsa\ONGOING PROJECTS\Crosstown #4\Pile Analysis\Revised 6 and 8 and abutment\
Name of input data file: L-Pile-8 foot-Q-1-Model 3-Shear Only.lpd
Name of output file: L-Pile-8 foot-Q-1-Model 3-Shear Only.lpo
Name of plot output file: L-Pile-8 foot-Q-1-Model 3-Shear Only.lpp
Name of runtime file: L-Pile-8 foot-Q-1-Model 3-Shear Only.lpr

Time and Date of Analysis

Date: May 23, 2012 Time: 7:43:33

Problem Title

SR-4-8 ft- Model #3 -Shear Only-Bents 4 and 5

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- Analysis includes computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 1080.00 in

Depth of ground surface below top of pile = 0.00 in

Slope angle of ground surface = 0.00 deg.

Structural properties of pile defined using 2 points

Point No.	Point Depth in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	96.00000000	4169220.	7238.0000	3000000.
2	1080.0000	96.00000000	4169220.	7238.0000	3000000.

Soil and Rock Layering Information

The soil profile is modelled using 8 layers

Layer 1 is stiff clay with water-induced erosion
Distance from top of pile to top of layer = 0.000 in
Distance from top of pile to bottom of layer = 36.000 in
p-y subgrade modulus k for top of soil layer = 500.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 500.000 lbs/in**3

Layer 2 is stiff clay without free water
Distance from top of pile to top of layer = 36.000 in
Distance from top of pile to bottom of layer = 96.000 in

Layer 3 is stiff clay without free water
Distance from top of pile to top of layer = 96.000 in
Distance from top of pile to bottom of layer = 300.000 in

Layer 4 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 300.000 in
Distance from top of pile to bottom of layer = 600.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 5 is stiff clay with water-induced erosion
Distance from top of pile to top of layer = 600.000 in
Distance from top of pile to bottom of layer = 636.000 in
p-y subgrade modulus k for top of soil layer = 1000.000 lbs/in**3

p-y subgrade modulus k for bottom of layer = 1000.000 lbs/in**3

Layer 6 is stiff clay without free water
Distance from top of pile to top of layer = 636.000 in
Distance from top of pile to bottom of layer = 960.000 in

Layer 7 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 960.000 in
Distance from top of pile to bottom of layer = 1080.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 8 is stiff clay without free water
Distance from top of pile to top of layer = 1080.000 in
Distance from top of pile to bottom of layer = 1320.000 in

(Depth of lowest layer extends 240.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 16 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	0.00	0.03600
2	36.00	0.03600
3	36.00	0.03600
4	96.00	0.03600
5	96.00	0.03600
6	300.00	0.03600
7	300.00	0.03600
8	600.00	0.03600
9	600.00	0.03600
10	636.00	0.03600
11	636.00	0.03600
12	960.00	0.03600
13	960.00	0.03600
14	1080.00	0.03600
15	1080.00	0.03600
16	1320.00	0.03600

Shear Strength of Soils

Shear strength parameters with depth defined using 16 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	0.000	7.00000	0.00	0.07000	0.0
2	36.000	7.00000	0.00	0.07000	0.0
3	36.000	17.36000	0.00	0.00500	0.0
4	96.000	17.36000	0.00	0.00500	0.0
5	96.000	20.83000	0.00	0.00500	0.0
6	300.000	20.83000	0.00	0.00500	0.0

7	300.000	0.00000	36.00	------	------
8	600.000	0.00000	36.00	------	------
9	600.000	17.36000	0.00	0.00500	0.0
10	636.000	17.36000	0.00	0.00500	0.0
11	636.000	34.72000	0.00	0.00400	0.0
12	960.000	34.72000	0.00	0.00400	0.0
13	960.000	0.00000	38.00	------	------
14	1080.000	0.00000	38.00	------	------
15	1080.000	41.67000	0.00	0.00400	0.0
16	1320.000	41.67000	0.00	0.00400	0.0

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 2

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 427000.000 lbs
Bending moment at pile head = 0.000 in-lbs
Axial load at pile head = 0.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 493000.000 lbs
Bending moment at pile head = 0.000 in-lbs
Axial load at pile head = 0.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (Pile-head Condition Type 1)
Specified shear force at pile head = 427000.000 lbs
Specified moment at pile head = 0.000 in-lbs

Specified axial load at pile head = 0.000 lbs

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Soil Res. Es*h p lbs/in	F/L lbs/in
0.000	0.393361	9.5242E-05	427000.	-0.0015823	1.0965E-09	0.0000	0.0000
10.800	0.376272	4611600.	425981.	-0.0015803	53.0931	-188.7456	5417.4926
21.600	0.359226	9201185.	423809.	-0.0015744	105.9327	-213.4691	6417.8655
32.400	0.342266	1.3766E+07	421378.	-0.0015645	158.4857	-236.7232	7469.6585
43.200	0.325434	1.8303E+07	410145.	-0.0015506	210.7208	-1843.4291	61176.8537
54.000	0.308773	2.2625E+07	390114.	-0.0015329	260.4804	-1866.0443	65268.9406
64.800	0.292323	2.6729E+07	369849.	-0.0015116	307.7341	-1886.7201	69705.7779
75.600	0.276122	3.0614E+07	349371.	-0.0014869	352.4543	-1905.4081	74526.5922
86.400	0.260206	3.4276E+07	328703.	-0.0014589	394.6157	-1922.0593	79776.1238
97.200	0.244610	3.7714E+07	306235.	-0.0014278	434.1960	-2238.6269	98839.5297
108.000	0.229366	4.0891E+07	281984.	-0.0013938	470.7702	-2252.4321	106059.
118.800	0.214503	4.3805E+07	257596.	-0.0013573	504.3196	-2263.7279	113976.
129.600	0.200049	4.6455E+07	233101.	-0.0013183	534.8291	-2272.4539	122682.
140.400	0.186028	4.8840E+07	208526.	-0.0012772	562.2871	-2278.5475	132283.
151.200	0.172462	5.0959E+07	183899.	-0.0012341	586.6852	-2281.9433	142901.
162.000	0.159372	5.2812E+07	159251.	-0.0011893	608.0190	-2282.5725	154681.
172.800	0.146774	5.4399E+07	134611.	-0.0011430	626.2875	-2280.3621	167795.
183.600	0.134683	5.5719E+07	110010.	-0.0010954	641.4939	-2275.2339	182447.
194.400	0.123112	5.6775E+07	85481.8517	-0.0010469	653.6449	-2267.1032	198881.
205.200	0.112071	5.7566E+07	61057.7568	-0.0009975	662.7515	-2255.8773	217394.
216.000	0.101566	5.8094E+07	36772.1714	-0.0009476	668.8287	-2241.4533	238345.
226.800	0.091603	5.8360E+07	12660.2618	-0.0008973	671.8960	-2223.7152	262177.
237.600	0.082184	5.8367E+07	-11241.4635	-0.0008469	671.9771	-2202.5303	289440.
248.400	0.073310	5.8117E+07	-34894.9450	-0.0007966	669.1005	-2177.7441	320826.
259.200	0.064977	5.7613E+07	-58260.2996	-0.0007467	663.2994	-2149.1734	357220.
270.000	0.057182	5.6859E+07	-81295.4568	-0.0006972	654.6123	-2116.5964	399764.
280.800	0.049917	5.5857E+07	-103956.	-0.0006486	643.0829	-2079.7389	

L-Pile-8 foot-Q-1-Model 7-Shear Only.lpo

LPILE Plus for Windows, Version 5.0 (5.0.46)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

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This program is licensed to:

Hamid Parsa
Kleinfelder

Files Used for Analysis

Path to file locations: U:\SParsa\ONGOING PROJECTS\Crosstown #4\Pile Analysis\Revised 6 and 8 and abutment\
Name of input data file: L-Pile-8 foot-Q-1-Model 7-Shear Only.lpd
Name of output file: L-Pile-8 foot-Q-1-Model 7-Shear Only.lpo
Name of plot output file: L-Pile-8 foot-Q-1-Model 7-Shear Only.lpp
Name of runtime file: L-Pile-8 foot-Q-1-Model 7-Shear Only.lpr

Time and Date of Analysis

Date: May 23, 2012 Time: 8:29:50

Problem Title

SR-4-8 ft- Model #7-Shear Only-Bent 10 through 15

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- Analysis includes computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 960.00 in

Depth of ground surface below top of pile = 0.00 in

Slope angle of ground surface = 0.00 deg.

Structural properties of pile defined using 2 points

Point No.	Point Depth in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	96.00000000	4169220.	7238.0000	3000000.
2	960.0000	96.00000000	4169220.	7238.0000	3000000.

Soil and Rock Layering Information

The soil profile is modelled using 8 layers

Layer 1 is stiff clay without free water
Distance from top of pile to top of layer = 0.000 in
Distance from top of pile to bottom of layer = 180.000 in

Layer 2 is stiff clay without free water
Distance from top of pile to top of layer = 180.000 in
Distance from top of pile to bottom of layer = 276.000 in

Layer 3 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 276.000 in
Distance from top of pile to bottom of layer = 348.000 in
p-y subgrade modulus k for top of soil layer = 90.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 90.000 lbs/in**3

Layer 4 is stiff clay without free water
Distance from top of pile to top of layer = 348.000 in
Distance from top of pile to bottom of layer = 468.000 in

Layer 5 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 468.000 in
Distance from top of pile to bottom of layer = 600.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 6 is stiff clay without free water
Distance from top of pile to top of layer = 600.000 in
Distance from top of pile to bottom of layer = 840.000 in

Layer 7 is stiff clay without free water
Distance from top of pile to top of layer = 840.000 in
Distance from top of pile to bottom of layer = 1080.000 in

Layer 8 is stiff clay without free water
Distance from top of pile to top of layer = 1080.000 in
Distance from top of pile to bottom of layer = 1440.000 in

(Depth of lowest layer extends 480.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 16 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	0.00	0.06900
2	180.00	0.06900
3	180.00	0.03600
4	276.00	0.03600
5	276.00	0.03600
6	348.00	0.03600
7	348.00	0.03600
8	468.00	0.03600
9	468.00	0.03600
10	600.00	0.03600
11	600.00	0.03600
12	840.00	0.03600
13	840.00	0.03600
14	1080.00	0.03600
15	1080.00	0.03600
16	1440.00	0.03600

Shear Strength of Soils

Shear strength parameters with depth defined using 16 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	0.000	13.89000	0.00	0.00500	0.0
2	180.000	13.89000	0.00	0.00500	0.0
3	180.000	13.89000	0.00	0.00500	0.0
4	276.000	13.89000	0.00	0.00500	0.0
5	276.000	0.00000	35.00	------	------
6	348.000	0.00000	35.00	------	------
7	348.000	20.83000	0.00	0.00500	0.0
8	468.000	20.83000	0.00	0.00500	0.0
9	468.000	0.00000	36.00	------	------
10	600.000	0.00000	36.00	------	------

11	600.000	27.78000	0.00	0.00400	0.0
12	840.000	27.78000	0.00	0.00400	0.0
13	840.000	34.72000	0.00	0.00400	0.0
14	1080.000	34.72000	0.00	0.00400	0.0
15	1080.000	41.67000	0.00	0.00400	0.0
16	1440.000	41.67000	0.00	0.00400	0.0

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 2

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 533000.000 lbs
Bending moment at pile head = 0.000 in-lbs
Axial load at pile head = 0.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 617000.000 lbs
Bending moment at pile head = 0.000 in-lbs
Axial load at pile head = 0.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (Pile-head Condition Type 1)
Specified shear force at pile head = 533000.000 lbs
Specified moment at pile head = 0.000 in-lbs
Specified axial load at pile head = 0.000 lbs

Depth	Deflect.	Moment	Shear	Slope	Total	Soil Res. Es*h

L-Pile-8 foot-Q-1-Model 8-Shear Only.lpo

LPILE Plus for Windows, Version 5.0 (5.0.46)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

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This program is licensed to:

Hamid Parsa
Kleinfelder

Files Used for Analysis

Path to file locations: U:\SParsa\ONGOING PROJECTS\Crosstown #4\Pile Analysis\Revised 6 and 8 and abutment\
Name of input data file: L-Pile-8 foot-Q-1-Model 8-Shear Only.lpd
Name of output file: L-Pile-8 foot-Q-1-Model 8-Shear Only.lpo
Name of plot output file: L-Pile-8 foot-Q-1-Model 8-Shear Only.lpp
Name of runtime file: L-Pile-8 foot-Q-1-Model 8-Shear Only.lpr

Time and Date of Analysis

Date: May 23, 2012 Time: 8:44:03

Problem Title

SR-4- 8 ft- Model #8 - Shear Only - Bent 10 through 15

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- Analysis includes computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 960.00 in

Depth of ground surface below top of pile = 0.00 in

Slope angle of ground surface = 0.00 deg.

Structural properties of pile defined using 2 points

Point No.	Point Depth in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	96.00000000	4169220.	7238.0000	3000000.
2	960.0000	96.00000000	4169220.	7238.0000	3000000.

Soil and Rock Layering Information

The soil profile is modelled using 5 layers

Layer 1 is stiff clay without free water
Distance from top of pile to top of layer = 0.000 in
Distance from top of pile to bottom of layer = 180.000 in

Layer 2 is stiff clay without free water
Distance from top of pile to top of layer = 180.000 in
Distance from top of pile to bottom of layer = 288.000 in

Layer 3 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 288.000 in
Distance from top of pile to bottom of layer = 348.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 4 is stiff clay without free water
Distance from top of pile to top of layer = 348.000 in
Distance from top of pile to bottom of layer = 828.000 in

Layer 5 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 828.000 in
Distance from top of pile to bottom of layer = 1080.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

(Depth of lowest layer extends 120.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 10 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	0.00	0.06900
2	180.00	0.06900
3	180.00	0.03600
4	288.00	0.03600
5	288.00	0.03600
6	348.00	0.03600
7	348.00	0.03600
8	828.00	0.03600
9	828.00	0.03600
10	1080.00	0.03600

Shear Strength of Soils

Shear strength parameters with depth defined using 10 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	0.000	17.36000	0.00	0.00500	0.0
2	180.000	17.36000	0.00	0.00500	0.0
3	180.000	17.36000	0.00	0.00500	0.0
4	288.000	17.36000	0.00	0.00500	0.0
5	288.000	0.00000	36.00	------	------
6	348.000	0.00000	36.00	------	------
7	348.000	27.78000	0.00	0.00500	0.0
8	828.000	27.78000	0.00	0.00500	0.0
9	828.000	0.00000	38.00	------	------
10	1080.000	0.00000	38.00	------	------

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 2

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 533000.000 lbs
Bending moment at pile head = 0.000 in-lbs
Axial load at pile head = 0.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 617000.000 lbs
Bending moment at pile head = 0.000 in-lbs
Axial load at pile head = 0.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (Pile-head Condition Type 1)
Specified shear force at pile head = 533000.000 lbs
Specified moment at pile head = 0.000 in-lbs
Specified axial load at pile head = 0.000 lbs

Depth X	Deflect. y	Moment M	Shear V	Slope S	Total Stress	Soil Res. Es*h p
in	in	lbs-in	lbs	Rad.	lbs/in**2	lbs/in
						F/L lbs/in
0.000	0.446665	0.0001582	533000.	-0.0017788	1.8215E-09	-1952.6259 20983.5299
9.600	0.429589	5026823.	514073.	-0.0017768	57.8735	-1990.5128 44481.9083
19.200	0.412550	9870200.	494790.	-0.0017711	113.6351	-2026.7254 47161.7650
28.800	0.395583	1.4527E+07	475168.	-0.0017618	167.2462	-2061.2221 50021.6769
38.400	0.378724	1.8993E+07	455223.	-0.0017489	218.6703	-2093.9611 53078.3324
48.000	0.362004	2.3267E+07	434973.	-0.0017327	267.8726	-2124.9009 56350.2817
57.600	0.345456	2.7345E+07	414434.	-0.0017133	314.8203	-2153.9996 59858.1922
67.200	0.329110	3.1224E+07	393625.	-0.0016908	359.4826	-2181.2152 63625.1445
76.800	0.312994	3.4902E+07	372564.	-0.0016654	401.8305	-2206.5057 67676.9776

L-Pile-8 foot-Q-1-Model 9-Shear Only.lpo

LPILE Plus for Windows, Version 5.0 (5.0.46)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

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This program is licensed to:

Hamid Parsa
Kleinfelder

Files Used for Analysis

Path to file locations: U:\SParsa\ONGOING PROJECTS\Crosstown #4\Pile Analysis\Revised 6 and 8 and abutment\
Name of input data file: L-Pile-8 foot-Q-1-Model 9-Shear Only.lpd
Name of output file: L-Pile-8 foot-Q-1-Model 9-Shear Only.lpo
Name of plot output file: L-Pile-8 foot-Q-1-Model 9-Shear Only.lpp
Name of runtime file: L-Pile-8 foot-Q-1-Model 9-Shear Only.lpr

Time and Date of Analysis

Date: May 23, 2012 Time: 9:29:02

Problem Title

SR-4- 8 ft- Model #9 - Shear Only-Bent 10 through 15

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- Analysis includes computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 960.00 in

Depth of ground surface below top of pile = 0.00 in

Slope angle of ground surface = 0.00 deg.

Structural properties of pile defined using 2 points

Point No.	Point Depth in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	96.00000000	4169220.	7238.0000	3000000.
2	960.0000	96.00000000	4169220.	7238.0000	3000000.

Soil and Rock Layering Information

The soil profile is modelled using 5 layers

Layer 1 is stiff clay without free water
Distance from top of pile to top of layer = 0.000 in
Distance from top of pile to bottom of layer = 180.000 in

Layer 2 is stiff clay without free water
Distance from top of pile to top of layer = 180.000 in
Distance from top of pile to bottom of layer = 300.000 in

Layer 3 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 300.000 in
Distance from top of pile to bottom of layer = 480.000 in
p-y subgrade modulus k for top of soil layer = 90.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 90.000 lbs/in**3

Layer 4 is stiff clay without free water
Distance from top of pile to top of layer = 480.000 in
Distance from top of pile to bottom of layer = 720.000 in

Layer 5 is stiff clay without free water
Distance from top of pile to top of layer = 720.000 in
Distance from top of pile to bottom of layer = 1080.000 in

(Depth of lowest layer extends 120.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 10 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	0.00	0.06900
2	180.00	0.06900
3	180.00	0.03600
4	300.00	0.03600
5	300.00	0.03600
6	480.00	0.03600
7	480.00	0.03600
8	720.00	0.03600
9	720.00	0.03600
10	1080.00	0.03600

Shear Strength of Soils

Shear strength parameters with depth defined using 10 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	0.000	17.36000	0.00	0.00500	0.0
2	180.000	17.36000	0.00	0.00500	0.0
3	180.000	17.36000	0.00	0.00500	0.0
4	300.000	17.36000	0.00	0.00500	0.0
5	300.000	0.00000	35.00	------	------
6	480.000	0.00000	35.00	------	------
7	480.000	24.31000	0.00	0.00500	0.0
8	720.000	24.31000	0.00	0.00500	0.0
9	720.000	34.72000	0.00	0.00500	0.0
10	1080.000	34.72000	0.00	0.00500	0.0

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves.

L-Pile-8 foot-Q-1-Model 9-Shear Only.lpo
Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 2

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 533000.000 lbs
Bending moment at pile head = 0.000 in-lbs
Axial load at pile head = 0.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 617000.000 lbs
Bending moment at pile head = 0.000 in-lbs
Axial load at pile head = 0.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (Pile-head Condition Type 1)
Specified shear force at pile head = 533000.000 lbs
Specified moment at pile head = 0.000 in-lbs
Specified axial load at pile head = 0.000 lbs

Depth X	Deflect. y	Moment M	Shear V	Slope S	Total Stress	Soil Res. Es*h p
in	in	lbs-in	lbs	Rad.	lbs/in**2	lbs/in F/L lbs/in
0.000	0.463869	-0.0002185	533000.	-0.0017976	2.5153E-09	-1971.1562 20397.0335
9.600	0.446611	5025969.	513891.	-0.0017957	57.8637	-2009.9388 43204.0233
19.200	0.429391	9866702.	494417.	-0.0017900	113.5948	-2047.0936 45767.3677
28.800	0.412243	1.4519E+07	474595.	-0.0017806	167.1539	-2082.5829 48497.5570
38.400	0.395203	1.8979E+07	454440.	-0.0017678	218.5032	-2116.3693 51409.4365
48.000	0.378302	2.3244E+07	433969.	-0.0017516	267.6071	-2148.4156 54519.4140
57.600	0.361572	2.7311E+07	413199.	-0.0017322	314.4314	-2178.6853 57845.6594
67.200	0.345044	3.1177E+07	392147.	-0.0017097	358.9440	-2207.1419 61408.3333
76.800	0.328745	3.4840E+07	370830.	-0.0016844	401.1148	-2233.7493 65229.8496
86.400	0.312703	3.8297E+07	349268.	-0.0016563	440.9155	-2258.4718 69335.1789

L-Pile-8 foot-Q-1-Model 10-Shear Only.lpo

LPILE Plus for Windows, Version 5.0 (5.0.46)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

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This program is licensed to:

Hamid Parsa
Kleinfelder

Files Used for Analysis

Path to file locations: U:\SParsa\ONGOING PROJECTS\Crosstown #4\Pile Analysis\Revised 6 and 8 and abutment\
Name of input data file: L-Pile-8 foot-Q-1-Model 10-Shear Only.lpd
Name of output file: L-Pile-8 foot-Q-1-Model 10-Shear Only.lpo
Name of plot output file: L-Pile-8 foot-Q-1-Model 10-Shear Only.lpp
Name of runtime file: L-Pile-8 foot-Q-1-Model 10-Shear Only.lpr

Time and Date of Analysis

Date: May 23, 2012 Time: 9:38:58

Problem Title

SR-4- 8 ft- Model #10 -Shear Only-Bents 10 through 15

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- Analysis includes computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 960.00 in

Depth of ground surface below top of pile = 0.00 in

Slope angle of ground surface = 0.00 deg.

Structural properties of pile defined using 2 points

Point No.	Point Depth in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	96.00000000	4169220.	7238.0000	3000000.
2	960.0000	96.00000000	4169220.	7238.0000	3000000.

Soil and Rock Layering Information

The soil profile is modelled using 6 layers

Layer 1 is stiff clay without free water
Distance from top of pile to top of layer = 0.000 in
Distance from top of pile to bottom of layer = 120.000 in

Layer 2 is stiff clay without free water
Distance from top of pile to top of layer = 120.000 in
Distance from top of pile to bottom of layer = 300.000 in

Layer 3 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 300.000 in
Distance from top of pile to bottom of layer = 360.000 in
p-y subgrade modulus k for top of soil layer = 90.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 90.000 lbs/in**3

Layer 4 is stiff clay without free water
Distance from top of pile to top of layer = 360.000 in
Distance from top of pile to bottom of layer = 540.000 in

Layer 5 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 540.000 in
Distance from top of pile to bottom of layer = 720.000 in
p-y subgrade modulus k for top of soil layer = 90.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 90.000 lbs/in**3

Layer 6 is stiff clay without free water
Distance from top of pile to top of layer = 720.000 in
Distance from top of pile to bottom of layer = 1080.000 in

(Depth of lowest layer extends 120.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 12 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	0.00	0.06900
2	120.00	0.06900
3	120.00	0.03500
4	300.00	0.03500
5	300.00	0.03600
6	360.00	0.03600
7	360.00	0.03600
8	540.00	0.03600
9	540.00	0.03600
10	720.00	0.03600
11	720.00	0.03600
12	1080.00	0.03600

Shear Strength of Soils

Shear strength parameters with depth defined using 12 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	0.000	13.89000	0.00	0.00500	0.0
2	120.000	13.89000	0.00	0.00500	0.0
3	120.000	13.89000	0.00	0.00500	0.0
4	300.000	13.89000	0.00	0.00500	0.0
5	300.000	0.00000	33.00	------	------
6	360.000	0.00000	33.00	------	------
7	360.000	20.83000	0.00	0.00500	0.0
8	540.000	20.83000	0.00	0.00500	0.0
9	540.000	0.00000	35.00	------	------
10	720.000	0.00000	35.00	------	------
11	720.000	27.78000	0.00	0.00400	0.0
12	1080.000	27.78000	0.00	0.00400	0.0

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 2

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 533000.000 lbs
Bending moment at pile head = 0.000 in-lbs
Axial load at pile head = 0.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 617000.000 lbs
Bending moment at pile head = 0.000 in-lbs
Axial load at pile head = 0.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (Pile-head Condition Type 1)
Specified shear force at pile head = 533000.000 lbs
Specified moment at pile head = 0.000 in-lbs
Specified axial load at pile head = 0.000 lbs

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Soil Res. p lbs/in	Es*h F/L lbs/in
0.000	0.606937	0.0001205	533000.	-0.0022007	1.3878E-09	-1686.7824	13340.0241
9.600	0.585811	5039073.	516617.	-0.0021987	58.0146	-1726.3508	28290.6604
19.200	0.564721	9919046.	499861.	-0.0021930	114.1974	-1764.5450	29996.4543
28.800	0.543705	1.4636E+07	482744.	-0.0021836	168.5080	-1801.3306	31805.4434
38.400	0.522796	1.9188E+07	465282.	-0.0021706	220.9074	-1836.6729	33726.4376

L-Pile Input for 8 foot Diameter Piles – Moment Applied at Pile Head

L-Pile-8 foot--Model 2-Moment Only.lpo

LPILE Plus for Windows, Version 5.0 (5.0.46)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

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This program is licensed to:

Hamid Parsa
Kleinfelder

Files Used for Analysis

Path to file locations: U:\SParsa\ONGOING PROJECTS\Crosstown #4\Pile Analysis\Revised 6 and 8 and abutment\
Name of input data file: L-Pile-8 foot--Model 2-Moment Only.lpd
Name of output file: L-Pile-8 foot--Model 2-Moment Only.lpo
Name of plot output file: L-Pile-8 foot--Model 2-Moment Only.lpp
Name of runtime file: L-Pile-8 foot--Model 2-Moment Only.lpr

Time and Date of Analysis

Date: May 22, 2012 Time: 17:49:18

Problem Title

SR-4-8 ft- Model #2 -Moment Only-Bents 4 and 5

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- Analysis includes computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 1080.00 in

Depth of ground surface below top of pile = 0.00 in

Slope angle of ground surface = 0.00 deg.

Structural properties of pile defined using 2 points

Point No.	Point Depth in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	96.00000000	4169220.	7238.0000	3000000.
2	1080.0000	96.00000000	4169220.	7238.0000	3000000.

Soil and Rock Layering Information

The soil profile is modelled using 8 layers

Layer 1 is stiff clay with water-induced erosion
Distance from top of pile to top of layer = 0.000 in
Distance from top of pile to bottom of layer = 36.000 in
p-y subgrade modulus k for top of soil layer = 500.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 500.000 lbs/in**3

Layer 2 is stiff clay without free water
Distance from top of pile to top of layer = 36.000 in
Distance from top of pile to bottom of layer = 96.000 in

Layer 3 is stiff clay without free water
Distance from top of pile to top of layer = 96.000 in
Distance from top of pile to bottom of layer = 240.000 in

Layer 4 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 240.000 in
Distance from top of pile to bottom of layer = 720.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 5 is stiff clay with water-induced erosion
Distance from top of pile to top of layer = 720.000 in
Distance from top of pile to bottom of layer = 756.000 in
p-y subgrade modulus k for top of soil layer = 1000.000 lbs/in**3

p-y subgrade modulus k for bottom of layer = 1000.000 lbs/in**3

Layer 6 is stiff clay without free water
Distance from top of pile to top of layer = 756.000 in
Distance from top of pile to bottom of layer = 960.000 in

Layer 7 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 960.000 in
Distance from top of pile to bottom of layer = 1080.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 8 is stiff clay without free water
Distance from top of pile to top of layer = 1080.000 in
Distance from top of pile to bottom of layer = 1320.000 in

(Depth of lowest layer extends 240.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 16 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	0.00	0.03600
2	36.00	0.03600
3	36.00	0.03600
4	96.00	0.03600
5	96.00	0.03600
6	240.00	0.03600
7	240.00	0.03600
8	720.00	0.03600
9	720.00	0.03600
10	756.00	0.03600
11	756.00	0.03600
12	960.00	0.03600
13	960.00	0.03600
14	1080.00	0.03600
15	1080.00	0.03600
16	1320.00	0.03600

Shear Strength of Soils

Shear strength parameters with depth defined using 16 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	0.000	7.00000	0.00	0.07000	0.0
2	36.000	7.00000	0.00	0.07000	0.0
3	36.000	20.83000	0.00	0.00500	0.0
4	96.000	20.83000	0.00	0.00500	0.0
5	96.000	20.83000	0.00	0.00500	0.0
6	240.000	20.83000	0.00	0.00500	0.0

7	240.000	0.00000	38.00	------	------
8	720.000	0.00000	38.00	------	------
9	720.000	17.36000	0.00	0.00500	0.0
10	756.000	17.36000	0.00	0.00500	0.0
11	756.000	34.72000	0.00	0.00400	0.0
12	960.000	34.72000	0.00	0.00400	0.0
13	960.000	0.00000	38.00	------	------
14	1080.000	0.00000	38.00	------	------
15	1080.000	41.67000	0.00	0.00400	0.0
16	1320.000	41.67000	0.00	0.00400	0.0

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 2

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 0.000 lbs
Bending moment at pile head = 126312000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 0.000 lbs
Bending moment at pile head = 134208000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (Pile-head Condition Type 1)
Specified shear force at pile head = 0.000 lbs
Specified moment at pile head = 126312000.000 in-lbs
Specified axial load at pile head = 0.000 lbs

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Soil Res. p lbs/in	Es*h F/L lbs/in
0.000	0.407856	1.2631E+08	0.0000	-0.0028176	1454.2231	0.0000	0.0000
10.800	0.378015	1.2631E+08	-1021.5794	-0.0027085	1454.2231	-189.1814	5404.9650
21.600	0.349352	1.2629E+08	-3179.9332	-0.0025995	1453.9690	-210.5138	6507.9051
32.400	0.321866	1.2624E+08	-5556.3274	-0.0024905	1453.4323	-229.5592	7702.7012
43.200	0.295558	1.2617E+08	-18436.3227	-0.0023815	1452.5873	-2155.6251	78768.8188
54.000	0.270426	1.2585E+08	-41740.0149	-0.0022727	1448.8476	-2159.8735	86258.7677
64.800	0.246468	1.2527E+08	-65071.5782	-0.0021643	1442.2074	-2160.7864	94683.6311
75.600	0.223678	1.2444E+08	-88394.2351	-0.0020565	1432.6656	-2158.2242	104207.
86.400	0.202049	1.2336E+08	-111670.	-0.0019495	1420.2256	-2152.0274	115031.
97.200	0.181569	1.2203E+08	-134705.	-0.0018435	1404.8956	-2113.8335	125734.
108.000	0.162228	1.2045E+08	-157463.	-0.0017388	1386.7271	-2100.5643	139841.
118.800	0.144010	1.1863E+08	-180054.	-0.0016356	1365.7378	-2083.0229	156215.
129.600	0.126899	1.1656E+08	-202432.	-0.0015341	1341.9513	-2060.8989	175397.
140.400	0.110874	1.1425E+08	-224543.	-0.0014344	1315.3972	-2033.8091	198109.
151.200	0.095915	1.1171E+08	-246332.	-0.0013369	1286.1120	-2001.2695	225343.
162.000	0.081997	1.0893E+08	-267738.	-0.0012416	1254.1394	-1962.6540	258504.
172.800	0.069096	1.0593E+08	-288688.	-0.0011489	1219.5311	-1917.1284	299656.
183.600	0.057182	1.0270E+08	-309104.	-0.0010588	1182.3485	-1863.5416	351968.
194.400	0.046226	9.9250E+07	-328888.	-0.0009716	1142.6633	-1800.2349	420598.
205.200	0.036195	9.5593E+07	-347923.	-0.0008875	1100.5606	-1724.6790	514610.
216.000	0.027056	9.1735E+07	-366053.	-0.0008066	1056.1419	-1632.7106	651724.
226.800	0.018773	8.7687E+07	-383059.	-0.0007291	1009.5307	-1516.6602	872538.
237.600	0.011307	8.3461E+07	-398591.	-0.0006553	960.8829	-1359.5220	1298576.
248.400	0.004619	7.9077E+07	-406656.	-0.0005851	910.4093	-134.0542	313420.
259.200	-0.001331	7.4677E+07	-407162.	-0.0005187	859.7558	40.4173	

L-Pile-8 foot-Q-1-Model 3-Moment Only.lpo

LPILE Plus for Windows, Version 5.0 (5.0.46)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

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This program is licensed to:

Hamid Parsa
Kleinfelder

Files Used for Analysis

Path to file locations: U:\SParsa\ONGOING PROJECTS\Crosstown #4\Pile Analysis\Revised 6 and 8 and abutment\
Name of input data file: L-Pile-8 foot-Q-1-Model 3-Moment Only.lpd
Name of output file: L-Pile-8 foot-Q-1-Model 3-Moment Only.lpo
Name of plot output file: L-Pile-8 foot-Q-1-Model 3-Moment Only.lpp
Name of runtime file: L-Pile-8 foot-Q-1-Model 3-Moment Only.lpr

Time and Date of Analysis

Date: May 23, 2012 Time: 8:14:25

Problem Title

SR-4-8 ft- Model #3 -Moment Only-Bents 4 and 5

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- Analysis includes computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 1080.00 in

Depth of ground surface below top of pile = 0.00 in

Slope angle of ground surface = 0.00 deg.

Structural properties of pile defined using 2 points

Point No.	Point Depth in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	96.00000000	4169220.	7238.0000	3000000.
2	1080.0000	96.00000000	4169220.	7238.0000	3000000.

Soil and Rock Layering Information

The soil profile is modelled using 8 layers

Layer 1 is stiff clay with water-induced erosion
Distance from top of pile to top of layer = 0.000 in
Distance from top of pile to bottom of layer = 36.000 in
p-y subgrade modulus k for top of soil layer = 500.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 500.000 lbs/in**3

Layer 2 is stiff clay without free water
Distance from top of pile to top of layer = 36.000 in
Distance from top of pile to bottom of layer = 96.000 in

Layer 3 is stiff clay without free water
Distance from top of pile to top of layer = 96.000 in
Distance from top of pile to bottom of layer = 300.000 in

Layer 4 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 300.000 in
Distance from top of pile to bottom of layer = 600.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 5 is stiff clay with water-induced erosion
Distance from top of pile to top of layer = 600.000 in
Distance from top of pile to bottom of layer = 636.000 in
p-y subgrade modulus k for top of soil layer = 1000.000 lbs/in**3

p-y subgrade modulus k for bottom of layer = 1000.000 lbs/in**3

Layer 6 is stiff clay without free water
Distance from top of pile to top of layer = 636.000 in
Distance from top of pile to bottom of layer = 960.000 in

Layer 7 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 960.000 in
Distance from top of pile to bottom of layer = 1080.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 8 is stiff clay without free water
Distance from top of pile to top of layer = 1080.000 in
Distance from top of pile to bottom of layer = 1320.000 in

(Depth of lowest layer extends 240.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 16 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	0.00	0.03600
2	36.00	0.03600
3	36.00	0.03600
4	96.00	0.03600
5	96.00	0.03600
6	300.00	0.03600
7	300.00	0.03600
8	600.00	0.03600
9	600.00	0.03600
10	636.00	0.03600
11	636.00	0.03600
12	960.00	0.03600
13	960.00	0.03600
14	1080.00	0.03600
15	1080.00	0.03600
16	1320.00	0.03600

Shear Strength of Soils

Shear strength parameters with depth defined using 16 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	0.000	7.00000	0.00	0.07000	0.0
2	36.000	7.00000	0.00	0.07000	0.0
3	36.000	17.36000	0.00	0.00500	0.0
4	96.000	17.36000	0.00	0.00500	0.0
5	96.000	20.83000	0.00	0.00500	0.0
6	300.000	20.83000	0.00	0.00500	0.0

7	300.000	0.00000	36.00	------	------
8	600.000	0.00000	36.00	------	------
9	600.000	17.36000	0.00	0.00500	0.0
10	636.000	17.36000	0.00	0.00500	0.0
11	636.000	34.72000	0.00	0.00400	0.0
12	960.000	34.72000	0.00	0.00400	0.0
13	960.000	0.00000	38.00	------	------
14	1080.000	0.00000	38.00	------	------
15	1080.000	41.67000	0.00	0.00400	0.0
16	1320.000	41.67000	0.00	0.00400	0.0

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 2

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 0.000 lbs
Bending moment at pile head = 126312000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 0.000 lbs
Bending moment at pile head = 134208000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (Pile-head Condition Type 1)
Specified shear force at pile head = 0.000 lbs
Specified moment at pile head = 126312000.000 in-lbs
Specified axial load at pile head = 0.000 lbs

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Soil Res. Es*h p lbs/in	F/L lbs/in
0.000	0.428588	1.2631E+08	2.7558E-07	-0.0028824	1454.2231	0.0000	0.0000
10.800	0.398046	1.2631E+08	-1048.2994	-0.0027734	1454.2231	-194.1295	5267.2259
21.600	0.368683	1.2629E+08	-3264.4035	-0.0026643	1453.9624	-216.2601	6335.0144
32.400	0.340497	1.2624E+08	-5707.2035	-0.0025553	1453.4113	-236.1102	7489.0287
43.200	0.313488	1.2617E+08	-16844.0633	-0.0024463	1452.5431	-1826.2712	62916.9637
54.000	0.287656	1.2588E+08	-36605.6707	-0.0023375	1449.2225	-1833.2857	68830.3605
64.800	0.262998	1.2538E+08	-56427.9701	-0.0022290	1443.4401	-1837.5105	75457.2291
75.600	0.239509	1.2466E+08	-76280.2535	-0.0021211	1435.1900	-1838.8383	82917.2701
86.400	0.217183	1.2373E+08	-96130.5805	-0.0020138	1424.4707	-1837.1482	91357.1304
97.200	0.196010	1.2258E+08	-117489.	-0.0019075	1411.2843	-2118.0370	116702.
108.000	0.175981	1.2119E+08	-140310.	-0.0018023	1395.2537	-2108.0794	129374.
118.800	0.157081	1.1955E+08	-163001.	-0.0016983	1376.3922	-2094.1066	143978.
129.600	0.139297	1.1767E+08	-185519.	-0.0015959	1354.7186	-2075.8642	160946.
140.400	0.122610	1.1554E+08	-207815.	-0.0014952	1330.2573	-2053.0440	180841.
151.200	0.107000	1.1318E+08	-229838.	-0.0013965	1303.0391	-2025.2650	204419.
162.000	0.092446	1.1058E+08	-251532.	-0.0012999	1273.1013	-1992.0462	232720.
172.800	0.078923	1.0775E+08	-272834.	-0.0012056	1240.4883	-1952.7644	267220.
183.600	0.066405	1.0469E+08	-293674.	-0.0011139	1205.2531	-1906.5887	310084.
194.400	0.054863	1.0140E+08	-313973.	-0.0010249	1167.4575	-1852.3710	364644.
205.200	0.044267	9.7905E+07	-333633.	-0.0009389	1127.1745	-1788.4542	436335.
216.000	0.034584	9.4197E+07	-352537.	-0.0008559	1084.4898	-1712.3062	534726.
226.800	0.025779	9.0290E+07	-370530.	-0.0007763	1039.5057	-1619.7475	678583.
237.600	0.017816	8.6194E+07	-387393.	-0.0007001	992.3465	-1503.0466	911125.
248.400	0.010657	8.1922E+07	-402772.	-0.0006275	943.1689	-1344.9424	1362944.
259.200	0.004262	7.7494E+07	-415911.	-0.0005587	892.1852	-1088.1474	

L-Pile-8 foot-Q-1-Model 7-Moment Only.lpo

LPILE Plus for Windows, Version 5.0 (5.0.46)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

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This program is licensed to:

Hamid Parsa
Kleinfelder

Files Used for Analysis

Path to file locations: U:\SParsa\ONGOING PROJECTS\Crosstown #4\Pile Analysis\Revised 6 and 8 and abutment\
Name of input data file: L-Pile-8 foot-Q-1-Model 7-Moment Only.lpd
Name of output file: L-Pile-8 foot-Q-1-Model 7-Moment Only.lpo
Name of plot output file: L-Pile-8 foot-Q-1-Model 7-Moment Only.lpp
Name of runtime file: L-Pile-8 foot-Q-1-Model 7-Moment Only.lpr

Time and Date of Analysis

Date: May 23, 2012 Time: 8:38:58

Problem Title

SR-4-8 ft- Model #7-Moment Only-Bent 10 through 15

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- Analysis includes computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 960.00 in

Depth of ground surface below top of pile = 0.00 in

Slope angle of ground surface = 0.00 deg.

Structural properties of pile defined using 2 points

Point No.	Point Depth in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	96.00000000	4169220.	7238.0000	3000000.
2	960.0000	96.00000000	4169220.	7238.0000	3000000.

Soil and Rock Layering Information

The soil profile is modelled using 8 layers

Layer 1 is stiff clay without free water
Distance from top of pile to top of layer = 0.000 in
Distance from top of pile to bottom of layer = 180.000 in

Layer 2 is stiff clay without free water
Distance from top of pile to top of layer = 180.000 in
Distance from top of pile to bottom of layer = 276.000 in

Layer 3 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 276.000 in
Distance from top of pile to bottom of layer = 348.000 in
p-y subgrade modulus k for top of soil layer = 90.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 90.000 lbs/in**3

Layer 4 is stiff clay without free water
Distance from top of pile to top of layer = 348.000 in
Distance from top of pile to bottom of layer = 468.000 in

Layer 5 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 468.000 in
Distance from top of pile to bottom of layer = 600.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 6 is stiff clay without free water
Distance from top of pile to top of layer = 600.000 in
Distance from top of pile to bottom of layer = 840.000 in

Layer 7 is stiff clay without free water
Distance from top of pile to top of layer = 840.000 in
Distance from top of pile to bottom of layer = 1080.000 in

Layer 8 is stiff clay without free water
Distance from top of pile to top of layer = 1080.000 in
Distance from top of pile to bottom of layer = 1440.000 in

(Depth of lowest layer extends 480.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 16 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	0.00	0.06900
2	180.00	0.06900
3	180.00	0.03600
4	276.00	0.03600
5	276.00	0.03600
6	348.00	0.03600
7	348.00	0.03600
8	468.00	0.03600
9	468.00	0.03600
10	600.00	0.03600
11	600.00	0.03600
12	840.00	0.03600
13	840.00	0.03600
14	1080.00	0.03600
15	1080.00	0.03600
16	1440.00	0.03600

Shear Strength of Soils

Shear strength parameters with depth defined using 16 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	0.000	13.89000	0.00	0.00500	0.0
2	180.000	13.89000	0.00	0.00500	0.0
3	180.000	13.89000	0.00	0.00500	0.0
4	276.000	13.89000	0.00	0.00500	0.0
5	276.000	0.00000	35.00	------	------
6	348.000	0.00000	35.00	------	------
7	348.000	20.83000	0.00	0.00500	0.0
8	468.000	20.83000	0.00	0.00500	0.0
9	468.000	0.00000	36.00	------	------
10	600.000	0.00000	36.00	------	------

11	600.000	27.78000	0.00	0.00400	0.0
12	840.000	27.78000	0.00	0.00400	0.0
13	840.000	34.72000	0.00	0.00400	0.0
14	1080.000	34.72000	0.00	0.00400	0.0
15	1080.000	41.67000	0.00	0.00400	0.0
16	1440.000	41.67000	0.00	0.00400	0.0

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 2

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 0.000 lbs
Bending moment at pile head = 128244000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 0.000 lbs
Bending moment at pile head = 135408000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (Pile-head Condition Type 1)
Specified shear force at pile head = 0.000 lbs
Specified moment at pile head = 128244000.000 in-lbs

Specified axial load at pile head = 0.000 lbs

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Soil Res. Es*h p lbs/in	F/L lbs/in
0.000	0.408180	1.2824E+08	-3.9239E-07	-0.0027867	1476.4661	-1527.5208	17962.8948
9.600	0.381900	1.2817E+08	-14778.0397	-0.0026883	1475.6557	-1551.2375	38994.1886
19.200	0.356564	1.2796E+08	-29774.0527	-0.0025901	1473.1994	-1572.9319	42349.0400
28.800	0.332171	1.2760E+08	-44968.3911	-0.0024920	1469.0742	-1592.5553	46026.0830
38.400	0.308718	1.2710E+08	-60340.9230	-0.0023942	1463.2592	-1610.0555	50066.8082
48.000	0.286202	1.2644E+08	-75870.9974	-0.0022969	1455.7359	-1625.3767	54519.6289
57.600	0.264617	1.2564E+08	-91537.4043	-0.0022002	1446.4881	-1638.4581	59441.3496
67.200	0.243958	1.2469E+08	-107318.	-0.0021041	1435.5017	-1649.2340	64899.0222
76.800	0.224218	1.2358E+08	-123191.	-0.0020088	1422.7655	-1657.6320	70972.3182
86.400	0.205388	1.2232E+08	-139133.	-0.0019145	1408.2705	-1663.5722	77756.5898
96.000	0.187460	1.2091E+08	-155120.	-0.0018211	1392.0103	-1666.9654	85366.8718
105.600	0.170422	1.1934E+08	-171126.	-0.0017289	1373.9815	-1667.7113	93943.1784
115.200	0.154264	1.1762E+08	-187126.	-0.0016380	1354.1832	-1665.6953	103658.
124.800	0.138973	1.1575E+08	-203094.	-0.0015484	1332.6175	-1660.7859	114724.
134.400	0.124534	1.1372E+08	-218999.	-0.0014604	1309.2896	-1652.8294	127412.
144.000	0.110934	1.1154E+08	-234812.	-0.0013739	1284.2081	-1641.6445	142065.
153.600	0.098155	1.0921E+08	-250502.	-0.0012892	1257.3847	-1627.0129	159129.
163.200	0.086181	1.0674E+08	-266033.	-0.0012063	1228.8350	-1608.6683	179195.
172.800	0.074993	1.0411E+08	-281369.	-0.0011254	1198.5785	-1586.2790	203062.
182.400	0.064573	1.0133E+08	-296460.	-0.0010466	1166.6388	-1557.7553	231590.
192.000	0.054899	9.8415E+07	-311228.	-0.0009699	1133.0463	-1518.9109	265606.
201.600	0.045950	9.5357E+07	-325599.	-0.0008956	1097.8422	-1474.9184	308141.
211.200	0.037704	9.2163E+07	-339517.	-0.0008236	1061.0732	-1424.7927	362769.
220.800	0.030137	8.8838E+07	-352918.	-0.0007541	1022.7924	-1367.0838	435471.
230.400	0.023225	8.5387E+07	-365718.	-0.0006873	983.0611	-1299.5204	537151.
240.000	0.016942	8.1817E+07	-377803.	-0.0006231	941.9510	-1218.2161	690293.
249.600	0.011262	7.8134E+07	-389005.	-0.0005617	899.5483	-1115.5677	

L-Pile-8 foot-Q-1-Model 8-Moment Only.lpo

LPILE Plus for Windows, Version 5.0 (5.0.46)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

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This program is licensed to:

Hamid Parsa
Kleinfelder

Files Used for Analysis

Path to file locations: U:\SParsa\ONGOING PROJECTS\Crosstown #4\Pile Analysis\Revised 6 and 8 and abutment\
Name of input data file: L-Pile-8 foot-Q-1-Model 8-Moment Only.lpd
Name of output file: L-Pile-8 foot-Q-1-Model 8-Moment Only.lpo
Name of plot output file: L-Pile-8 foot-Q-1-Model 8-Moment Only.lpp
Name of runtime file: L-Pile-8 foot-Q-1-Model 8-Moment Only.lpr

Time and Date of Analysis

Date: May 23, 2012 Time: 9:17:50

Problem Title

SR-4- 8 ft- Model #8 - Moment Only - Bent 10 through 15

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- Analysis includes computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 960.00 in

Depth of ground surface below top of pile = 0.00 in

Slope angle of ground surface = 0.00 deg.

Structural properties of pile defined using 2 points

Point No.	Point Depth in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	96.00000000	4169220.	7238.0000	3000000.
2	960.0000	96.00000000	4169220.	7238.0000	3000000.

Soil and Rock Layering Information

The soil profile is modelled using 5 layers

Layer 1 is stiff clay without free water
Distance from top of pile to top of layer = 0.000 in
Distance from top of pile to bottom of layer = 180.000 in

Layer 2 is stiff clay without free water
Distance from top of pile to top of layer = 180.000 in
Distance from top of pile to bottom of layer = 288.000 in

Layer 3 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 288.000 in
Distance from top of pile to bottom of layer = 348.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 4 is stiff clay without free water
Distance from top of pile to top of layer = 348.000 in
Distance from top of pile to bottom of layer = 828.000 in

Layer 5 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 828.000 in
Distance from top of pile to bottom of layer = 1080.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

(Depth of lowest layer extends 120.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 10 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	0.00	0.06900
2	180.00	0.06900
3	180.00	0.03600
4	288.00	0.03600
5	288.00	0.03600
6	348.00	0.03600
7	348.00	0.03600
8	828.00	0.03600
9	828.00	0.03600
10	1080.00	0.03600

Shear Strength of Soils

Shear strength parameters with depth defined using 10 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	0.000	17.36000	0.00	0.00500	0.0
2	180.000	17.36000	0.00	0.00500	0.0
3	180.000	17.36000	0.00	0.00500	0.0
4	288.000	17.36000	0.00	0.00500	0.0
5	288.000	0.00000	36.00	------	------
6	348.000	0.00000	36.00	------	------
7	348.000	27.78000	0.00	0.00500	0.0
8	828.000	27.78000	0.00	0.00500	0.0
9	828.000	0.00000	38.00	------	------
10	1080.000	0.00000	38.00	------	------

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 2

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 0.000 lbs
Bending moment at pile head = 128244000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 0.000 lbs
Bending moment at pile head = 135408000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (Pile-head Condition Type 1)
Specified shear force at pile head = 0.000 lbs
Specified moment at pile head = 128244000.000 in-lbs
Specified axial load at pile head = 0.000 lbs

Depth X	Deflect. y	Moment M	Shear V	Slope S	Total Stress	Soil Res. Es*h p
in	in	lbs-in	lbs	Rad.	lbs/in**2	lbs/in F/L lbs/in
0.000	0.349690	1.2824E+08	3.9239E-07	-0.0025684	1476.4661	-1836.7343 25211.7950
9.600	0.325506	1.2816E+08	-17730.5745	-0.0024700	1475.4917	-1857.1354 54771.6688
19.200	0.302266	1.2790E+08	-35645.3269	-0.0023718	1472.5468	-1875.1047 59553.6009
28.800	0.279968	1.2747E+08	-53720.6157	-0.0022738	1467.6123	-1890.5805 64827.3703
38.400	0.258609	1.2687E+08	-71932.1881	-0.0021762	1460.6719	-1903.4971 70660.9586
48.000	0.238185	1.2609E+08	-90255.1339	-0.0020791	1451.7118	-1913.7833 77134.5254
57.600	0.218691	1.2514E+08	-108664.	-0.0019827	1440.7212	-1921.3620 84343.1787

L-Pile-8 foot-Q-2-Model 9-Moment Only.lpo

LPILE Plus for Windows, Version 5.0 (5.0.46)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

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This program is licensed to:

Hamid Parsa
Kleinfelder

Files Used for Analysis

Path to file locations: U:\SParsa\ONGOING PROJECTS\Crosstown #4\Pile Analysis\Revised 6 and 8 and abutment\
Name of input data file: L-Pile-8 foot-Q-2-Model 9-Moment Only.lpd
Name of output file: L-Pile-8 foot-Q-2-Model 9-Moment Only.lpo
Name of plot output file: L-Pile-8 foot-Q-2-Model 9-Moment Only.lpp
Name of runtime file: L-Pile-8 foot-Q-2-Model 9-Moment Only.lpr

Time and Date of Analysis

Date: May 23, 2012 Time: 9:34:27

Problem Title

SR-4- 8 ft- Model #9 - Moment Only-Bent 10 through 15

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- Analysis includes computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 960.00 in

Depth of ground surface below top of pile = 0.00 in

Slope angle of ground surface = 0.00 deg.

Structural properties of pile defined using 2 points

Point No.	Point Depth in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	96.00000000	4169220.	7238.0000	3000000.
2	960.0000	96.00000000	4169220.	7238.0000	3000000.

Soil and Rock Layering Information

The soil profile is modelled using 5 layers

Layer 1 is stiff clay without free water
Distance from top of pile to top of layer = 0.000 in
Distance from top of pile to bottom of layer = 180.000 in

Layer 2 is stiff clay without free water
Distance from top of pile to top of layer = 180.000 in
Distance from top of pile to bottom of layer = 300.000 in

Layer 3 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 300.000 in
Distance from top of pile to bottom of layer = 480.000 in
p-y subgrade modulus k for top of soil layer = 90.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 90.000 lbs/in**3

Layer 4 is stiff clay without free water
Distance from top of pile to top of layer = 480.000 in
Distance from top of pile to bottom of layer = 720.000 in

Layer 5 is stiff clay without free water
Distance from top of pile to top of layer = 720.000 in
Distance from top of pile to bottom of layer = 1080.000 in

(Depth of lowest layer extends 120.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 10 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	0.00	0.06900
2	180.00	0.06900
3	180.00	0.03600
4	300.00	0.03600
5	300.00	0.03600
6	480.00	0.03600
7	480.00	0.03600
8	720.00	0.03600
9	720.00	0.03600
10	1080.00	0.03600

Shear Strength of Soils

Shear strength parameters with depth defined using 10 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	0.000	17.36000	0.00	0.00500	0.0
2	180.000	17.36000	0.00	0.00500	0.0
3	180.000	17.36000	0.00	0.00500	0.0
4	300.000	17.36000	0.00	0.00500	0.0
5	300.000	0.00000	35.00	------	------
6	480.000	0.00000	35.00	------	------
7	480.000	24.31000	0.00	0.00500	0.0
8	720.000	24.31000	0.00	0.00500	0.0
9	720.000	34.72000	0.00	0.00500	0.0
10	1080.000	34.72000	0.00	0.00500	0.0

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 2

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 0.000 lbs
Bending moment at pile head = 128244000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 0.000 lbs
Bending moment at pile head = 135408000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (Pile-head Condition Type 1)
Specified shear force at pile head = 0.000 lbs
Specified moment at pile head = 128244000.000 in-lbs
Specified axial load at pile head = 0.000 lbs

Depth X	Deflect. y	Moment M	Shear V	Slope S	Total Stress	Soil Res. Es*h p
in	in	lbs-in	lbs	Rad.	lbs/in**2	lbs/in F/L lbs/in
0.000	0.354108	1.2824E+08	3.9239E-07	-0.0026133	1476.4661	-1842.4987 24975.4245
9.600	0.329492	1.2816E+08	-17785.3662	-0.0025150	1475.4886	-1862.7859 54273.6579
19.200	0.305821	1.2790E+08	-35753.5427	-0.0024167	1472.5347	-1880.5842 59033.2912
28.800	0.283092	1.2747E+08	-53880.2926	-0.0023187	1467.5854	-1895.8221 64289.7062
38.400	0.261302	1.2687E+08	-72140.6637	-0.0022211	1460.6245	-1908.4219 70113.6587
48.000	0.240447	1.2609E+08	-90508.9213	-0.0021240	1451.6388	-1918.2984 76589.2056
57.600	0.220521	1.2513E+08	-108958.	-0.0020276	1440.6176	-1925.3573 83816.9381
67.200	0.201518	1.2400E+08	-127462.	-0.0019320	1427.5537	-1929.4932 91918.2113

L-Pile-8 foot-Q-2-Model 10-Moment Only.lpo

LPILE Plus for Windows, Version 5.0 (5.0.46)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

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This program is licensed to:

Hamid Parsa
Kleinfelder

Files Used for Analysis

Path to file locations: U:\SParsa\ONGOING PROJECTS\Crosstown #4\Pile Analysis\Revised 6 and 8 and abutment\
Name of input data file: L-Pile-8 foot-Q-2-Model 10-Moment Only.lpd
Name of output file: L-Pile-8 foot-Q-2-Model 10-Moment Only.lpo
Name of plot output file: L-Pile-8 foot-Q-2-Model 10-Moment Only.lpp
Name of runtime file: L-Pile-8 foot-Q-2-Model 10-Moment Only.lpr

Time and Date of Analysis

Date: May 23, 2012 Time: 9:43:05

Problem Title

SR-4- 8 ft- Model #10 -Moment Only-Bents 10 through 15

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- Analysis includes computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 960.00 in

Depth of ground surface below top of pile = 0.00 in

Slope angle of ground surface = 0.00 deg.

Structural properties of pile defined using 2 points

Point No.	Point Depth in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	96.00000000	4169220.	7238.0000	3000000.
2	960.0000	96.00000000	4169220.	7238.0000	3000000.

Soil and Rock Layering Information

The soil profile is modelled using 6 layers

Layer 1 is stiff clay without free water
Distance from top of pile to top of layer = 0.000 in
Distance from top of pile to bottom of layer = 120.000 in

Layer 2 is stiff clay without free water
Distance from top of pile to top of layer = 120.000 in
Distance from top of pile to bottom of layer = 300.000 in

Layer 3 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 300.000 in
Distance from top of pile to bottom of layer = 360.000 in
p-y subgrade modulus k for top of soil layer = 90.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 90.000 lbs/in**3

Layer 4 is stiff clay without free water
Distance from top of pile to top of layer = 360.000 in
Distance from top of pile to bottom of layer = 540.000 in

Layer 5 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 540.000 in
Distance from top of pile to bottom of layer = 720.000 in
p-y subgrade modulus k for top of soil layer = 90.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 90.000 lbs/in**3

Layer 6 is stiff clay without free water
Distance from top of pile to top of layer = 720.000 in
Distance from top of pile to bottom of layer = 1080.000 in

(Depth of lowest layer extends 120.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 12 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	0.00	0.06900
2	120.00	0.06900
3	120.00	0.03500
4	300.00	0.03500
5	300.00	0.03600
6	360.00	0.03600
7	360.00	0.03600
8	540.00	0.03600
9	540.00	0.03600
10	720.00	0.03600
11	720.00	0.03600
12	1080.00	0.03600

Shear Strength of Soils

Shear strength parameters with depth defined using 12 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	0.000	13.89000	0.00	0.00500	0.0
2	120.000	13.89000	0.00	0.00500	0.0
3	120.000	13.89000	0.00	0.00500	0.0
4	300.000	13.89000	0.00	0.00500	0.0
5	300.000	0.00000	33.00	------	------
6	360.000	0.00000	33.00	------	------
7	360.000	20.83000	0.00	0.00500	0.0
8	540.000	20.83000	0.00	0.00500	0.0
9	540.000	0.00000	35.00	------	------
10	720.000	0.00000	35.00	------	------
11	720.000	27.78000	0.00	0.00400	0.0
12	1080.000	27.78000	0.00	0.00400	0.0

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 2

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 0.000 lbs
Bending moment at pile head = 128244000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 0.000 lbs
Bending moment at pile head = 135408000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (Pile-head Condition Type 1)
Specified shear force at pile head = 0.000 lbs
Specified moment at pile head = 128244000.000 in-lbs
Specified axial load at pile head = 0.000 lbs

Depth X	Deflect. y	Moment M	Shear V	Slope S	Total Stress	Soil Res. p	Es*h F/L
in	in	lbs-in	lbs	Rad.	lbs/in**2	lbs/in	lbs/in
0.000	0.412788	1.2824E+08	-3.9239E-07	-0.0027911	1476.4661	-1531.8161	17812.3357
9.600	0.386466	1.2817E+08	-14820.8272	-0.0026927	1475.6534	-1555.8562	38648.2316
19.200	0.361088	1.2796E+08	-29862.8606	-0.0025944	1473.1900	-1577.9007	41950.5603

L-Pile Input for 9 foot Diameter Piles – Shear Force Applied at Pile Head

LPILE Plus for Windows, Version 5.0 (5.0.46)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

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This program is licensed to:

Hamid Parsa
Kleinfelder

Files Used for Analysis

Path to file locations: U:\SParsa\ONGOING PROJECTS\Crosstown #4\Pile Analysis\Revised-9 ft- L-Pile\
Name of input data file: L-Pile-B-3-Q-1-Model 4.lpd
Name of output file: L-Pile-B-3-Q-1-Model 4.lpo
Name of plot output file: L-Pile-B-3-Q-1-Model 4.lpp
Name of runtime file: L-Pile-B-3-Q-1-Model 4.lpr

Time and Date of Analysis

Date: May 8, 2012 Time: 16:11:04

Problem Title

SR-4-9 ft- Model #4-Shear Only-Revised

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- Analysis includes computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 1500.00 in

Depth of ground surface below top of pile = 0.00 in

Slope angle of ground surface = 0.00 deg.

Structural properties of pile defined using 2 points

Point No.	Point Depth in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	108.00000	6678285.	9161.0000	3000000.
2	1500.0000	108.00000	6678285.	9161.0000	3000000.

Soil and Rock Layering Information

The soil profile is modelled using 10 layers

Layer 1 is stiff clay without free water
Distance from top of pile to top of layer = 0.000 in
Distance from top of pile to bottom of layer = 96.000 in

Layer 2 is stiff clay without free water
Distance from top of pile to top of layer = 96.000 in
Distance from top of pile to bottom of layer = 348.000 in

Layer 3 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 348.000 in
Distance from top of pile to bottom of layer = 456.000 in
p-y subgrade modulus k for top of soil layer = 90.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 90.000 lbs/in**3

Layer 4 is stiff clay without free water
Distance from top of pile to top of layer = 456.000 in
Distance from top of pile to bottom of layer = 780.000 in

Layer 5 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 780.000 in
Distance from top of pile to bottom of layer = 936.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 6 is stiff clay without free water
Distance from top of pile to top of layer = 936.000 in
Distance from top of pile to bottom of layer = 1068.000 in

Layer 7 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 1068.000 in
Distance from top of pile to bottom of layer = 1188.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 8 is stiff clay without free water
Distance from top of pile to top of layer = 1188.000 in
Distance from top of pile to bottom of layer = 1548.000 in

Layer 9 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 1548.000 in
Distance from top of pile to bottom of layer = 1668.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 10 is stiff clay with water-induced erosion
Distance from top of pile to top of layer = 1668.000 in
Distance from top of pile to bottom of layer = 1800.000 in
p-y subgrade modulus k for top of soil layer = 2000.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 2000.000 lbs/in**3

(Depth of lowest layer extends 300.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 20 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	0.00	0.06900
2	96.00	0.06900
3	96.00	0.03600
4	348.00	0.03600
5	348.00	0.03600
6	456.00	0.03600
7	456.00	0.03600
8	780.00	0.03600
9	780.00	0.03600
10	936.00	0.03600
11	936.00	0.03600
12	1068.00	0.03600
13	1068.00	0.03600
14	1188.00	0.03600
15	1188.00	0.03600
16	1548.00	0.03600
17	1548.00	0.03600
18	1668.00	0.03600
19	1668.00	0.03600
20	1800.00	0.03600

Shear Strength of Soils

Shear strength parameters with depth defined using 20 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	0.000	13.90000	0.00	0.00500	0.0
2	96.000	13.90000	0.00	0.00500	0.0
3	96.000	20.80000	0.00	0.00500	0.0
4	348.000	20.80000	0.00	0.00500	0.0
5	348.000	0.00000	33.00	------	------
6	456.000	0.00000	33.00	------	------
7	456.000	27.78000	0.00	0.00500	0.0
8	780.000	27.78000	0.00	0.00500	0.0
9	780.000	0.00000	38.00	------	------
10	936.000	0.00000	38.00	------	------
11	936.000	41.67000	0.00	0.00400	0.0
12	1068.000	41.67000	0.00	0.00400	0.0
13	1068.000	0.00000	38.00	------	------
14	1188.000	0.00000	38.00	------	------
15	1188.000	41.67000	0.00	0.00400	0.0
16	1548.000	41.67000	0.00	0.00400	0.0
17	1548.000	0.00000	38.00	------	------
18	1668.000	0.00000	38.00	------	------
19	1668.000	41.67000	0.00	0.00400	0.0
20	1800.000	41.67000	0.00	0.00400	0.0

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 5

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 307000.000 lbs
Bending moment at pile head = 0.000 in-lbs
Axial load at pile head = 0.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 395500.000 lbs
Bending moment at pile head = 0.000 in-lbs
Axial load at pile head = 0.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Load Case Number 3

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 484000.000 lbs
Bending moment at pile head = 0.000 in-lbs
Axial load at pile head = 0.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Load Case Number 4

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 572500.000 lbs
Bending moment at pile head = 0.000 in-lbs
Axial load at pile head = 0.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Load Case Number 5

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 661000.000 lbs
Bending moment at pile head = 0.000 in-lbs
Axial load at pile head = 0.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (Pile-head Condition Type 1)
Specified shear force at pile head = 307000.000 lbs
Specified moment at pile head = 0.000 in-lbs
Specified axial load at pile head = 0.000 lbs

Depth X	Deflect. y	Moment M	Shear V	Slope S	Total Stress	Soil Res. p	Es*h F/L
in	in	lbs-in	lbs	Rad.	lbs/in**2	lbs/in	lbs/in
0.000	0.099518	-1.2357E-06	307000.	-0.0004777	9.9920E-12	-1173.3962	88431.2449
15.000	0.092352	4472993.	289148.	-0.0004761	36.1682	-1206.9229	196032.
30.000	0.085236	8674428.	270817.	-0.0004711	70.1406	-1237.1202	217712.
45.000	0.078217	1.2598E+07	252060.	-0.0004632	101.8623	-1263.8267	242369.

L-Pile-Q-1-Model #5.lpo

LPILE Plus for Windows, Version 5.0 (5.0.46)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

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This program is licensed to:

Hamid Parsa
Kleinfelder

Files Used for Analysis

Path to file locations: U:\SParsa\ONGOING PROJECTS\Crosstown #4\Pile Analysis\Revised-9 ft- L-Pile\
Name of input data file: L-Pile-Q-1-Model #5.lpd
Name of output file: L-Pile-Q-1-Model #5.lpo
Name of plot output file: L-Pile-Q-1-Model #5.lpp
Name of runtime file: L-Pile-Q-1-Model #5.lpr

Time and Date of Analysis

Date: May 10, 2012 Time: 10:41:51

Problem Title

SR-4-9ft-Model #5-Shear Only

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- Analysis includes computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 1500.00 in

Depth of ground surface below top of pile = 0.00 in

Slope angle of ground surface = 0.00 deg.

Structural properties of pile defined using 2 points

Point No.	Point Depth in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	108.00000	6678285.	9161.0000	3000000.
2	1500.0000	108.00000	6678285.	9161.0000	3000000.

Soil and Rock Layering Information

The soil profile is modelled using 5 layers

Layer 1 is stiff clay without free water
Distance from top of pile to top of layer = 0.000 in
Distance from top of pile to bottom of layer = 96.000 in

Layer 2 is stiff clay without free water
Distance from top of pile to top of layer = 96.000 in
Distance from top of pile to bottom of layer = 240.000 in

Layer 3 is stiff clay without free water
Distance from top of pile to top of layer = 240.000 in
Distance from top of pile to bottom of layer = 720.000 in

Layer 4 is stiff clay without free water
Distance from top of pile to top of layer = 720.000 in
Distance from top of pile to bottom of layer = 1080.000 in

Layer 5 is stiff clay without free water
Distance from top of pile to top of layer = 1080.000 in
Distance from top of pile to bottom of layer = 1800.000 in

(Depth of lowest layer extends 300.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 10 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	0.00	0.06900
2	96.00	0.06900
3	96.00	0.03600
4	240.00	0.03600
5	240.00	0.03600
6	720.00	0.03600
7	720.00	0.03600
8	1080.00	0.03600
9	1080.00	0.03600
10	1800.00	0.03600

Shear Strength of Soils

Shear strength parameters with depth defined using 10 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	0.000	10.40000	0.00	0.00500	0.0
2	96.000	10.40000	0.00	0.00500	0.0
3	96.000	17.36000	0.00	0.00500	0.0
4	240.000	17.36000	0.00	0.00500	0.0
5	240.000	20.83000	0.00	0.00500	0.0
6	720.000	20.83000	0.00	0.00500	0.0
7	720.000	34.72000	0.00	0.00400	0.0
8	1080.000	34.72000	0.00	0.00400	0.0
9	1080.000	41.67000	0.00	0.00400	0.0
10	1800.000	41.67000	0.00	0.00400	0.0

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 5

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 307000.000 lbs
Bending moment at pile head = 0.000 in-lbs
Axial load at pile head = 0.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 395500.000 lbs
Bending moment at pile head = 0.000 in-lbs
Axial load at pile head = 0.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Load Case Number 3

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 484000.000 lbs
Bending moment at pile head = 0.000 in-lbs
Axial load at pile head = 0.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Load Case Number 4

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 572500.000 lbs
Bending moment at pile head = 0.000 in-lbs
Axial load at pile head = 0.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Load Case Number 5

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 661000.000 lbs
Bending moment at pile head = 0.000 in-lbs
Axial load at pile head = 0.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (Pile-head Condition Type 1)
Specified shear force at pile head = 307000.000 lbs
Specified moment at pile head = 0.000 in-lbs
Specified axial load at pile head = 0.000 lbs

Depth	Deflect.	Moment	Shear	Slope	Total	Soil Res.
X	y	M	V	S	Stress	p
Es*h						

LPILE Plus for Windows, Version 5.0 (5.0.46)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

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This program is licensed to:

Hamid Parsa
Kleinfelder

Files Used for Analysis

Path to file locations: U:\SParsa\ONGOING PROJECTS\Crosstown #4\Pile Analysis\Revised-9 ft- L-Pile\
Name of input data file: L-Pile-Q-1-Model #6.lpd
Name of output file: L-Pile-Q-1-Model #6.lpo
Name of plot output file: L-Pile-Q-1-Model #6.lpp
Name of runtime file: L-Pile-Q-1-Model #6.lpr

Time and Date of Analysis

Date: May 10, 2012 Time: 11:34:53

Problem Title

SR-4-9ft-Model #6-Shear Only

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- Analysis includes computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 1500.00 in

Depth of ground surface below top of pile = 0.00 in

Slope angle of ground surface = 0.00 deg.

Structural properties of pile defined using 2 points

Point No.	Point Depth in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	108.00000	6678285.	9161.0000	3000000.
2	1500.0000	108.00000	6678285.	9161.0000	3000000.

Soil and Rock Layering Information

The soil profile is modelled using 6 layers

Layer 1 is stiff clay without free water
Distance from top of pile to top of layer = 0.000 in
Distance from top of pile to bottom of layer = 120.000 in

Layer 2 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 120.000 in
Distance from top of pile to bottom of layer = 240.000 in
p-y subgrade modulus k for top of soil layer = 60.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 60.000 lbs/in**3

Layer 3 is stiff clay without free water
Distance from top of pile to top of layer = 240.000 in
Distance from top of pile to bottom of layer = 720.000 in

Layer 4 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 720.000 in
Distance from top of pile to bottom of layer = 840.000 in
p-y subgrade modulus k for top of soil layer = 90.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 90.000 lbs/in**3

Layer 5 is stiff clay without free water
Distance from top of pile to top of layer = 840.000 in
Distance from top of pile to bottom of layer = 1080.000 in

Layer 6 is stiff clay without free water
Distance from top of pile to top of layer = 1080.000 in
Distance from top of pile to bottom of layer = 1800.000 in

(Depth of lowest layer extends 300.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 12 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	0.00	0.06900
2	120.00	0.06900
3	120.00	0.03600
4	240.00	0.03600
5	240.00	0.03600
6	720.00	0.03600
7	720.00	0.03600
8	840.00	0.03600
9	840.00	0.03600
10	1080.00	0.03600
11	1080.00	0.03600
12	1800.00	0.03600

Shear Strength of Soils

Shear strength parameters with depth defined using 12 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	0.000	13.89000	0.00	0.00500	0.0
2	120.000	13.89000	0.00	0.00500	0.0
3	120.000	0.00000	33.00	------	------
4	240.000	0.00000	33.00	------	------
5	240.000	19.10000	0.00	0.00500	0.0
6	720.000	19.10000	0.00	0.00500	0.0
7	720.000	0.00000	35.00	------	------
8	840.000	0.00000	35.00	------	------
9	840.000	31.25000	0.00	0.00500	0.0
10	1080.000	31.25000	0.00	0.00500	0.0
11	1080.000	41.67000	0.00	0.00400	0.0
12	1800.000	41.67000	0.00	0.00400	0.0

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 5

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 307000.000 lbs
Bending moment at pile head = 0.000 in-lbs
Axial load at pile head = 0.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 395500.000 lbs
Bending moment at pile head = 0.000 in-lbs
Axial load at pile head = 0.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Load Case Number 3

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 484000.000 lbs
Bending moment at pile head = 0.000 in-lbs
Axial load at pile head = 0.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Load Case Number 4

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 572500.000 lbs
Bending moment at pile head = 0.000 in-lbs
Axial load at pile head = 0.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Load Case Number 5

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 661000.000 lbs
Bending moment at pile head = 0.000 in-lbs
Axial load at pile head = 0.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

L-Pile-Q-1-Model #6.lpo
Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (Pile-head Condition Type 1)
Specified shear force at pile head = 307000.000 lbs
Specified moment at pile head = 0.000 in-lbs
Specified axial load at pile head = 0.000 lbs

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Soil Res. Es*h p lbs/in	F/L lbs/in
0.000	0.160263	1.9772E-05	307000.	-0.0006394	1.5987E-10	-1320.8580	61813.7552
15.000	0.150671	4456403.	286871.	-0.0006378	36.0341	-1363.0484	135698.
30.000	0.141129	8606121.	266130.	-0.0006329	69.5883	-1402.3376	149048.
45.000	0.131684	1.2440E+07	244823.	-0.0006250	100.5912	-1438.6091	163870.
60.000	0.122379	1.5951E+07	222996.	-0.0006144	128.9768	-1471.7440	180392.
75.000	0.113253	1.9130E+07	200695.	-0.0006013	154.6849	-1501.6186	198885.
90.000	0.104341	2.1972E+07	177972.	-0.0005859	177.6610	-1528.1028	219679.
105.000	0.095677	2.4469E+07	154879.	-0.0005685	197.8569	-1551.0570	243172.
120.000	0.087287	2.6618E+07	138447.	-0.0005494	215.2310	-639.8565	109958.
135.000	0.079196	2.8623E+07	128759.	-0.0005287	231.4410	-651.8226	123458.
150.000	0.071427	3.0481E+07	118979.	-0.0005066	246.4651	-652.1601	136958.
165.000	0.063999	3.2192E+07	109274.	-0.0004831	260.3027	-641.9461	150458.
180.000	0.056934	3.3759E+07	99791.6620	-0.0004584	272.9724	-622.3151	163958.
195.000	0.050247	3.5186E+07	90665.9137	-0.0004326	284.5099	-594.4513	177458.
210.000	0.043956	3.6479E+07	82010.6609	-0.0004058	294.9659	-559.5824	190958.
225.000	0.038074	3.7646E+07	73921.4942	-0.0003780	304.4038	-518.9732	204458.
240.000	0.032616	3.8697E+07	54681.6908	-0.0003494	312.8976	-2046.3339	941116.
255.000	0.027591	3.9287E+07	24334.6683	-0.0003202	317.6683	-1999.9357	1087266.
270.000	0.023008	3.9427E+07	-5267.2072	-0.0002908	318.8006	-1946.9810	1269317.
285.000	0.018868	3.9129E+07	-34021.5887	-0.0002614	316.3906	-1886.9365	1500115.
300.000	0.015167	3.8406E+07	-61816.8773	-0.0002323	310.5477	-1819.1019	1799064.
315.000	0.011898	3.7274E+07	-88528.9416	-0.0002040	301.3953	-1742.5066	2196893.
330.000	0.009047	3.5750E+07	-114016.	-0.0001767	289.0726	-1655.7205	2745321.

L-Pile Input for 9 foot Diameter Piles – Moment Applied at Pile Head

LPILE Plus for Windows, Version 5.0 (5.0.46)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

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This program is licensed to:

Hamid Parsa
Kleinfelder

Files Used for Analysis

Path to file locations: U:\SParsa\ONGOING PROJECTS\Crosstown #4\Pile Analysis\
Name of input data file: L-Pile-B-3-Q-2-Model #4.lpd
Name of output file: L-Pile-B-3-Q-2-Model #4.lpo
Name of plot output file: L-Pile-B-3-Q-2-Model #4.lpp
Name of runtime file: L-Pile-B-3-Q-2-Model #4.lpr

Time and Date of Analysis

Date: May 8, 2012 Time: 16:41:03

Problem Title

SR-4-9 ft-Model #4-Moment Only

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- Analysis includes computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 1500.00 in

Depth of ground surface below top of pile = 0.00 in

Slope angle of ground surface = 0.00 deg.

Structural properties of pile defined using 2 points

Point No.	Point Depth in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	108.00000	6678285.	9161.0000	3000000.
2	1500.0000	108.00000	6678285.	9161.0000	3000000.

Soil and Rock Layering Information

The soil profile is modelled using 10 layers

Layer 1 is stiff clay without free water
Distance from top of pile to top of layer = 0.000 in
Distance from top of pile to bottom of layer = 96.000 in

Layer 2 is stiff clay without free water
Distance from top of pile to top of layer = 96.000 in
Distance from top of pile to bottom of layer = 348.000 in

Layer 3 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 348.000 in
Distance from top of pile to bottom of layer = 456.000 in
p-y subgrade modulus k for top of soil layer = 90.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 90.000 lbs/in**3

Layer 4 is stiff clay without free water
Distance from top of pile to top of layer = 456.000 in
Distance from top of pile to bottom of layer = 780.000 in

Layer 5 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 780.000 in
Distance from top of pile to bottom of layer = 936.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 6 is stiff clay without free water

Distance from top of pile to top of layer = 936.000 in
Distance from top of pile to bottom of layer = 1068.000 in

Layer 7 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 1068.000 in
Distance from top of pile to bottom of layer = 1188.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 8 is stiff clay without free water
Distance from top of pile to top of layer = 1188.000 in
Distance from top of pile to bottom of layer = 1548.000 in

Layer 9 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 1548.000 in
Distance from top of pile to bottom of layer = 1668.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 10 is stiff clay without free water
Distance from top of pile to top of layer = 1668.000 in
Distance from top of pile to bottom of layer = 1800.000 in

(Depth of lowest layer extends 300.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 20 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	0.00	0.06900
2	96.00	0.06900
3	96.00	0.03600
4	348.00	0.03600
5	348.00	0.03600
6	456.00	0.03600
7	456.00	0.03600
8	780.00	0.03600
9	780.00	0.03600
10	936.00	0.03600
11	936.00	0.03600
12	1068.00	0.03600
13	1068.00	0.03600
14	1188.00	0.03600
15	1188.00	0.03600
16	1548.00	0.03600
17	1548.00	0.03600
18	1668.00	0.03600
19	1668.00	0.03600
20	1800.00	0.03600

Shear Strength of Soils

Shear strength parameters with depth defined using 20 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	0.000	13.89000	0.00	0.00500	0.0
2	96.000	13.89000	0.00	0.00500	0.0
3	96.000	20.83000	0.00	0.00500	0.0
4	348.000	20.83000	0.00	0.00500	0.0
5	348.000	0.00000	33.00	------	------
6	456.000	0.00000	33.00	------	------
7	456.000	27.78000	0.00	0.00500	0.0
8	780.000	27.78000	0.00	0.00500	0.0
9	780.000	0.00000	38.00	------	------
10	936.000	0.00000	38.00	------	------
11	936.000	41.67000	0.00	0.00400	0.0
12	1068.000	41.67000	0.00	0.00400	0.0
13	1068.000	0.00000	38.00	------	------
14	1188.000	0.00000	38.00	------	------
15	1188.000	41.67000	0.00	0.00400	0.0
16	1548.000	41.67000	0.00	0.00400	0.0
17	1548.000	0.00000	38.00	------	------
18	1668.000	0.00000	38.00	------	------
19	1668.000	41.67000	0.00	0.00400	0.0
20	1800.000	41.67000	0.00	0.00400	0.0

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 5

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 0.000 lbs
Bending moment at pile head = 115980000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 0.000 lbs
Bending moment at pile head = 152868000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 3

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 0.000 lbs
Bending moment at pile head = 189744000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 4

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 0.000 lbs
Bending moment at pile head = 226632000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 5

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 0.000 lbs
Bending moment at pile head = 263520000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (Pile-head Condition Type 1)
Specified shear force at pile head = 0.000 lbs
Specified moment at pile head = 115980000.000 in-lbs
Specified axial load at pile head = 0.000 lbs

Depth X	Deflect. y	Moment M	Shear V	Slope S	Total Stress	Soil Res. Es*h p
in	in	lbs-in	lbs	Rad.	lbs/in**2	lbs/in lbs/in
0.000	0.208908	1.1598E+08	0.0000	-0.0015043	937.8036	-1411.3455

LPILE Plus for Windows, Version 5.0 (5.0.46)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

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This program is licensed to:

Hamid Parsa
Kleinfelder

Files Used for Analysis

Path to file locations: U:\SParsa\ONGOING PROJECTS\Crosstown #4\Pile Analysis\Revised-9 ft- L-Pile\
Name of input data file: L-Pile-Q-2-Model #5.lpd
Name of output file: L-Pile-Q-2-Model #5.lpo
Name of plot output file: L-Pile-Q-2-Model #5.lpp
Name of runtime file: L-Pile-Q-2-Model #5.lpr

Time and Date of Analysis

Date: May 10, 2012 Time: 10:56:00

Problem Title

SR-4-9ft-Model #5-Moment Only

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- Analysis includes computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 1500.00 in

Depth of ground surface below top of pile = 0.00 in

Slope angle of ground surface = 0.00 deg.

Structural properties of pile defined using 2 points

Point No.	Point Depth in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	108.00000	6678285.	9161.0000	3000000.
2	1500.0000	108.00000	6678285.	9161.0000	3000000.

Soil and Rock Layering Information

The soil profile is modelled using 5 layers

Layer 1 is stiff clay without free water
Distance from top of pile to top of layer = 0.000 in
Distance from top of pile to bottom of layer = 96.000 in

Layer 2 is stiff clay without free water
Distance from top of pile to top of layer = 96.000 in
Distance from top of pile to bottom of layer = 240.000 in

Layer 3 is stiff clay without free water
Distance from top of pile to top of layer = 240.000 in
Distance from top of pile to bottom of layer = 720.000 in

Layer 4 is stiff clay without free water
Distance from top of pile to top of layer = 720.000 in
Distance from top of pile to bottom of layer = 1080.000 in

Layer 5 is stiff clay without free water
Distance from top of pile to top of layer = 1080.000 in
Distance from top of pile to bottom of layer = 1800.000 in

(Depth of lowest layer extends 300.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 10 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	0.00	0.06900
2	96.00	0.06900
3	96.00	0.03600
4	240.00	0.03600
5	240.00	0.03600
6	720.00	0.03600
7	720.00	0.03600
8	1080.00	0.03600
9	1080.00	0.03600
10	1800.00	0.03600

Shear Strength of Soils

Shear strength parameters with depth defined using 10 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	0.000	10.40000	0.00	0.00500	0.0
2	96.000	10.40000	0.00	0.00500	0.0
3	96.000	17.36000	0.00	0.00500	0.0
4	240.000	17.36000	0.00	0.00500	0.0
5	240.000	20.83000	0.00	0.00500	0.0
6	720.000	20.83000	0.00	0.00500	0.0
7	720.000	34.72000	0.00	0.00400	0.0
8	1080.000	34.72000	0.00	0.00400	0.0
9	1080.000	41.67000	0.00	0.00400	0.0
10	1800.000	41.67000	0.00	0.00400	0.0

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 5

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 0.000 lbs
Bending moment at pile head = 115980000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 0.000 lbs
Bending moment at pile head = 152868000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 3

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 0.000 lbs
Bending moment at pile head = 189744000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 4

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 0.000 lbs
Bending moment at pile head = 226632000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 5

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 0.000 lbs
Bending moment at pile head = 263520000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

LPILE Plus for Windows, Version 5.0 (5.0.46)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

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This program is licensed to:

Hamid Parsa
Kleinfelder

Files Used for Analysis

Path to file locations: U:\SParsa\ONGOING PROJECTS\Crosstown #4\Pile Analysis\Revised-9 ft- L-Pile\
Name of input data file: L-Pile-Q-2-Model #6.lpd
Name of output file: L-Pile-Q-2-Model #6.lpo
Name of plot output file: L-Pile-Q-2-Model #6.lpp
Name of runtime file: L-Pile-Q-2-Model #6.lpr

Time and Date of Analysis

Date: May 10, 2012 Time: 12:27:46

Problem Title

SR-4-9ft-Model #6-Moment Only

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- Analysis includes computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 1500.00 in

Depth of ground surface below top of pile = 0.00 in

Slope angle of ground surface = 0.00 deg.

Structural properties of pile defined using 2 points

Point No.	Point Depth in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	108.00000	6678285.	9161.0000	3000000.
2	1500.0000	108.00000	6678285.	9161.0000	3000000.

Soil and Rock Layering Information

The soil profile is modelled using 6 layers

Layer 1 is stiff clay without free water
Distance from top of pile to top of layer = 0.000 in
Distance from top of pile to bottom of layer = 120.000 in

Layer 2 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 120.000 in
Distance from top of pile to bottom of layer = 240.000 in
p-y subgrade modulus k for top of soil layer = 60.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 60.000 lbs/in**3

Layer 3 is stiff clay without free water
Distance from top of pile to top of layer = 240.000 in
Distance from top of pile to bottom of layer = 720.000 in

Layer 4 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 720.000 in
Distance from top of pile to bottom of layer = 840.000 in
p-y subgrade modulus k for top of soil layer = 90.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 90.000 lbs/in**3

Layer 5 is stiff clay without free water
Distance from top of pile to top of layer = 840.000 in
Distance from top of pile to bottom of layer = 1080.000 in

Layer 6 is stiff clay without free water
Distance from top of pile to top of layer = 1080.000 in
Distance from top of pile to bottom of layer = 1800.000 in

(Depth of lowest layer extends 300.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 12 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	0.00	0.06900
2	120.00	0.06900
3	120.00	0.03600
4	240.00	0.03600
5	240.00	0.03600
6	720.00	0.03600
7	720.00	0.03600
8	840.00	0.03600
9	840.00	0.03600
10	1080.00	0.03600
11	1080.00	0.03600
12	1800.00	0.03600

Shear Strength of Soils

Shear strength parameters with depth defined using 12 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	0.000	13.89000	0.00	0.00500	0.0
2	120.000	13.89000	0.00	0.00500	0.0
3	120.000	0.00000	33.00	------	------
4	240.000	0.00000	33.00	------	------
5	240.000	19.10000	0.00	0.00500	0.0
6	720.000	19.10000	0.00	0.00500	0.0
7	720.000	0.00000	35.00	------	------
8	840.000	0.00000	35.00	------	------
9	840.000	31.25000	0.00	0.00500	0.0
10	1080.000	31.25000	0.00	0.00500	0.0
11	1080.000	41.67000	0.00	0.00400	0.0
12	1800.000	41.67000	0.00	0.00400	0.0

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 5

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 0.000 lbs
Bending moment at pile head = 115980000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 0.000 lbs
Bending moment at pile head = 152868000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 3

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 0.000 lbs
Bending moment at pile head = 189744000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 4

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 0.000 lbs
Bending moment at pile head = 226632000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Load Case Number 5

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 0.000 lbs

Bending moment at pile head = 263520000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (Pile-head Condition Type 1)
Specified shear force at pile head = 0.000 lbs
Specified moment at pile head = 115980000.000 in-lbs
Specified axial load at pile head = 0.000 lbs

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Soil Res. Es*h p lbs/in	F/L lbs/in
0.000	0.262073	1.1598E+08	0.0000	-0.0016458	937.8036	-1493.6546	42745.3146
15.000	0.238037	1.1581E+08	-22663.4323	-0.0015590	936.4449	-1528.1364	96295.9952
30.000	0.215302	1.1530E+08	-45813.2321	-0.0014725	932.3060	-1558.5036	108580.
45.000	0.193861	1.1444E+08	-69386.8038	-0.0013865	925.3317	-1584.6393	122611.
60.000	0.173706	1.1322E+08	-93319.7283	-0.0013013	915.4743	-1606.4173	138719.
75.000	0.154822	1.1164E+08	-117546.	-0.0012171	902.6944	-1623.6998	157313.
90.000	0.137192	1.0969E+08	-141996.	-0.0011343	886.9605	-1636.3333	178910.
105.000	0.120794	1.0738E+08	-166599.	-0.0010530	868.2495	-1644.1437	204167.
120.000	0.105602	1.0469E+08	-184736.	-0.0009736	846.5472	-774.1130	109958.
135.000	0.091585	1.0184E+08	-196196.	-0.0008963	823.4366	-753.7910	123458.
150.000	0.078712	9.8808E+07	-207239.	-0.0008212	798.9546	-718.6815	136958.
165.000	0.066949	9.5619E+07	-217666.	-0.0007484	773.1651	-671.5311	150458.
180.000	0.056259	9.2278E+07	-227314.	-0.0006781	746.1539	-614.9446	163958.
195.000	0.046606	8.8799E+07	-236062.	-0.0006103	718.0238	-551.3774	177458.
210.000	0.037951	8.5196E+07	-243821.	-0.0005452	688.8907	-483.1302	190958.
225.000	0.030252	8.1485E+07	-250537.	-0.0004828	658.8785	-412.3441	204458.
240.000	0.023468	7.7680E+07	-267766.	-0.0004232	628.1162	-1884.9104	1204793.
255.000	0.017556	7.3452E+07	-295302.	-0.0003666	593.9246	-1786.5106	1526401.

L-Pile Input for 9 foot Diameter Piles – Shear Force & Moment Applied at Pile Head

LPILE Plus for Windows, Version 5.0 (5.0.46)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

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This program is licensed to:

Hamid Parsa
Kleinfelder

Files Used for Analysis

Path to file locations: U:\SParsa\ONGOING PROJECTS\Crosstown #4\Pile Analysis\Revised-9 ft- L-Pile\
Name of input data file: L-Pile-B-3-Q-3-Model #4.lpd
Name of output file: L-Pile-B-3-Q-3-Model #4.lpo
Name of plot output file: L-Pile-B-3-Q-3-Model #4.lpp
Name of runtime file: L-Pile-B-3-Q-3-Model #4.lpr

Time and Date of Analysis

Date: May 10, 2012 Time: 10:32:12

Problem Title

SR-4-9ft-Model #4-Moment-Shear

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- Analysis includes computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 1500.00 in

Depth of ground surface below top of pile = 0.00 in

Slope angle of ground surface = 0.00 deg.

Structural properties of pile defined using 2 points

Point No.	Point Depth in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	108.00000	6678285.	9161.0000	3000000.
2	1500.0000	108.00000	6678285.	9161.0000	3000000.

Soil and Rock Layering Information

The soil profile is modelled using 10 layers

Layer 1 is stiff clay without free water
Distance from top of pile to top of layer = 0.000 in
Distance from top of pile to bottom of layer = 96.000 in

Layer 2 is stiff clay without free water
Distance from top of pile to top of layer = 96.000 in
Distance from top of pile to bottom of layer = 348.000 in

Layer 3 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 348.000 in
Distance from top of pile to bottom of layer = 456.000 in
p-y subgrade modulus k for top of soil layer = 90.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 90.000 lbs/in**3

Layer 4 is stiff clay without free water
Distance from top of pile to top of layer = 456.000 in
Distance from top of pile to bottom of layer = 780.000 in

Layer 5 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 780.000 in
Distance from top of pile to bottom of layer = 936.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 6 is stiff clay without free water
Distance from top of pile to top of layer = 936.000 in
Distance from top of pile to bottom of layer = 1068.000 in

Layer 7 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 1068.000 in
Distance from top of pile to bottom of layer = 1188.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 8 is stiff clay without free water
Distance from top of pile to top of layer = 1188.000 in
Distance from top of pile to bottom of layer = 1548.000 in

Layer 9 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 1548.000 in
Distance from top of pile to bottom of layer = 1668.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

Layer 10 is stiff clay without free water
Distance from top of pile to top of layer = 1668.000 in
Distance from top of pile to bottom of layer = 1800.000 in

(Depth of lowest layer extends 300.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 20 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	0.00	0.06900
2	96.00	0.06900
3	96.00	0.03600
4	348.00	0.03600
5	348.00	0.03600
6	456.00	0.03600
7	456.00	0.03600
8	780.00	0.03600
9	780.00	0.03600
10	936.00	0.03600
11	936.00	0.03600
12	1068.00	0.03600
13	1068.00	0.03600
14	1188.00	0.03600
15	1188.00	0.03600
16	1548.00	0.03600
17	1548.00	0.03600
18	1668.00	0.03600
19	1668.00	0.03600
20	1800.00	0.03600

Shear Strength of Soils

Shear strength parameters with depth defined using 20 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	0.000	13.89000	0.00	0.00500	0.0
2	96.000	13.89000	0.00	0.00500	0.0
3	96.000	20.83000	0.00	0.00500	0.0
4	348.000	20.83000	0.00	0.00500	0.0
5	348.000	0.00000	33.00	------	------
6	456.000	0.00000	33.00	------	------
7	456.000	27.78000	0.00	0.00500	0.0
8	780.000	27.78000	0.00	0.00500	0.0
9	780.000	0.00000	38.00	------	------
10	936.000	0.00000	38.00	------	------
11	936.000	41.67000	0.00	0.00400	0.0
12	1068.000	41.67000	0.00	0.00400	0.0
13	1068.000	0.00000	38.00	------	------
14	1188.000	0.00000	38.00	------	------
15	1188.000	41.67000	0.00	0.00400	0.0
16	1548.000	41.67000	0.00	0.00400	0.0
17	1548.000	0.00000	38.00	------	------
18	1668.000	0.00000	38.00	------	------
19	1668.000	41.67000	0.00	0.00400	0.0
20	1800.000	41.67000	0.00	0.00400	0.0

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 661000.000 lbs
Bending moment at pile head = 263520000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (Pile-head Condition Type 1)
Specified shear force at pile head = 661000.000 lbs
Specified moment at pile head = 263520000.000 in-lbs
Specified axial load at pile head = 0.000 lbs

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Soil Res. Es*h p lbs/in
0.000	1.986	2.6352E+08	661000.	-0.0077944	2130.7986	-2478.0262
9360.1536						
15.000	1.870	2.7316E+08	623227.	-0.0075935	2208.7162	-2558.3406
20520.0257						
30.000	1.758	2.8222E+08	584282.	-0.0073856	2281.9793	-2634.3562
22480.4900						
45.000	1.649	2.9068E+08	544230.	-0.0071711	2350.4497	-2705.9614
24621.0917						
60.000	1.543	2.9854E+08	503137.	-0.0069505	2413.9970	-2773.0419
26964.1187						
75.000	1.440	3.0578E+08	461073.	-0.0067243	2472.4993	-2835.4806
29535.2805						
90.000	1.341	3.1238E+08	418108.	-0.0064929	2525.8429	-2893.1573
32364.3454						
105.000	1.245	3.1832E+08	366649.	-0.0062568	2573.9229	-3968.0995
47798.4050						
120.000	1.153	3.2338E+08	306879.	-0.0060166	2614.7836	-4001.2783
52045.9098						
135.000	1.065	3.2753E+08	246662.	-0.0057729	2648.3647	-4027.5583
56738.7400						
150.000	0.980010	3.3078E+08	186102.	-0.0055265	2674.6183	-4047.2151
61946.5544						
165.000	0.898970	3.3311E+08	125295.	-0.0052779	2693.5087	-4060.3629
67750.2431						
180.000	0.821671	3.3453E+08	64339.4005	-0.0050280	2705.0119	-4067.0164
74245.3008						
195.000	0.748130	3.3504E+08	3333.3534	-0.0047774	2709.1160	-4067.1232
81545.8068						
210.000	0.678351	3.3463E+08	-57624.4300	-0.0045267	2705.8205	-4060.5812
89789.4129						
225.000	0.612330	3.3331E+08	-118433.	-0.0042766	2695.1376	-4047.2473
99143.8014						
240.000	0.550052	3.3108E+08	-178990.	-0.0040279	2677.0913	-4026.9404
109815.						
255.000	0.491493	3.2794E+08	-239187.	-0.0037812	2651.7188	-3999.4415
122060.						
270.000	0.436616	3.2391E+08	-298917.	-0.0035372	2619.0699	-3964.4899
136201.						
285.000	0.385377	3.1898E+08	-358064.	-0.0032965	2579.2083	-3921.7761
152647.						
300.000	0.337720	3.1316E+08	-416509.	-0.0030599	2532.2118	-3870.9309
171929.						
315.000	0.293581	3.0648E+08	-474128.	-0.0028279	2478.1727	-3811.5093
194743.						

LPILE Plus for Windows, Version 5.0 (5.0.46)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

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This program is licensed to:

Hamid Parsa
Kleinfelder

Files Used for Analysis

Path to file locations: U:\SParsa\ONGOING PROJECTS\Crosstown #4\Pile Analysis\Revised-9 ft- L-Pile\
Name of input data file: L-Pile-Q-3-Model #5.lpd
Name of output file: L-Pile-Q-3-Model #5.lpo
Name of plot output file: L-Pile-Q-3-Model #5.lpp
Name of runtime file: L-Pile-Q-3-Model #5.lpr

Time and Date of Analysis

Date: May 10, 2012 Time: 11:25:56

Problem Title

SR-4-9ft-Model #5-Moment-Shear

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- Analysis includes computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 1500.00 in

Depth of ground surface below top of pile = 0.00 in

Slope angle of ground surface = 0.00 deg.

Structural properties of pile defined using 2 points

Point No.	Point Depth in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	108.00000	6678285.	9161.0000	3000000.
2	1500.0000	108.00000	6678285.	9161.0000	3000000.

Soil and Rock Layering Information

The soil profile is modelled using 5 layers

Layer 1 is stiff clay without free water
Distance from top of pile to top of layer = 0.000 in
Distance from top of pile to bottom of layer = 96.000 in

Layer 2 is stiff clay without free water
Distance from top of pile to top of layer = 96.000 in
Distance from top of pile to bottom of layer = 240.000 in

Layer 3 is stiff clay without free water
Distance from top of pile to top of layer = 240.000 in
Distance from top of pile to bottom of layer = 720.000 in

Layer 4 is stiff clay without free water
Distance from top of pile to top of layer = 720.000 in
Distance from top of pile to bottom of layer = 1080.000 in

Layer 5 is stiff clay without free water
Distance from top of pile to top of layer = 1080.000 in
Distance from top of pile to bottom of layer = 1800.000 in

(Depth of lowest layer extends 300.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 10 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	0.00	0.06900
2	96.00	0.06900
3	96.00	0.03600
4	240.00	0.03600
5	240.00	0.03600
6	720.00	0.03600
7	720.00	0.03600
8	1080.00	0.03600
9	1080.00	0.03600
10	1800.00	0.03600

Shear Strength of Soils

Shear strength parameters with depth defined using 10 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	0.000	10.40000	0.00	0.00500	0.0
2	96.000	10.40000	0.00	0.00500	0.0
3	96.000	17.36000	0.00	0.00500	0.0
4	240.000	17.36000	0.00	0.00500	0.0
5	240.000	20.83000	0.00	0.00500	0.0
6	720.000	20.83000	0.00	0.00500	0.0
7	720.000	34.72000	0.00	0.00400	0.0
8	1080.000	34.72000	0.00	0.00400	0.0
9	1080.000	41.67000	0.00	0.00400	0.0
10	1800.000	41.67000	0.00	0.00400	0.0

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 661000.000 lbs
Bending moment at pile head = 263520000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (Pile-head Condition Type 1)
Specified shear force at pile head = 661000.000 lbs
Specified moment at pile head = 263520000.000 in-lbs
Specified axial load at pile head = 0.000 lbs

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Soil Res. p lbs/in	Es*h F/L lbs/in
0.000	2.324	2.6352E+08	661000.	-0.0085461	2130.7986	-1929.7766	6228.7840
15.000	2.197	2.7322E+08	631451.	-0.0083451	2209.2149	-2010.0867	13724.4240
30.000	2.073	2.8246E+08	600724.	-0.0081371	2283.9742	-2086.8209	15098.0742
45.000	1.953	2.9124E+08	568874.	-0.0079224	2354.9369	-2159.8870	16590.7387
60.000	1.836	2.9953E+08	535956.	-0.0077012	2421.9701	-2229.1904	18216.3631
75.000	1.722	3.0732E+08	502027.	-0.0074740	2484.9476	-2294.6347	19990.9126
90.000	1.611	3.1459E+08	467146.	-0.0072412	2543.7504	-2356.1208	21932.7256
105.000	1.505	3.2133E+08	423452.	-0.0070032	2598.2667	-3469.7970	34593.6906
120.000	1.401	3.2729E+08	371098.	-0.0067604	2646.4703	-3510.7323	37580.6717
135.000	1.302	3.3247E+08	318178.	-0.0065134	2688.2867	-3545.3203	40853.7758
150.000	1.206	3.3684E+08	264783.	-0.0062628	2723.6530	-3573.9656	44456.8388
165.000	1.114	3.4041E+08	211002.	-0.0060093	2752.5170	-3596.8765	48439.4683
180.000	1.026	3.4317E+08	156919.	-0.0057534	2774.8372	-3614.1437	52859.0608
195.000	0.941224	3.4512E+08	102620.	-0.0054957	2790.5820	-3625.7820	57782.9761
210.000	0.860726	3.4625E+08	48188.1597	-0.0052369	2799.7304	-3631.7530	

LPILE Plus for Windows, Version 5.0 (5.0.46)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

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This program is licensed to:

Hamid Parsa
Kleinfelder

Files Used for Analysis

Path to file locations: U:\SParsa\ONGOING PROJECTS\Crosstown #4\Pile Analysis\Revised-9 ft- L-Pile\
Name of input data file: L-Pile-Q-3-Model #6.lpd
Name of output file: L-Pile-Q-3-Model #6.lpo
Name of plot output file: L-Pile-Q-3-Model #6.lpp
Name of runtime file: L-Pile-Q-3-Model #6.lpr

Time and Date of Analysis

Date: May 10, 2012 Time: 12:37:17

Problem Title

SR-4-9ft-Model #6-Shear and Moment

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- Analysis includes computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 1500.00 in

Depth of ground surface below top of pile = 0.00 in

Slope angle of ground surface = 0.00 deg.

Structural properties of pile defined using 2 points

Point No.	Point Depth in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	108.00000	6678285.	9161.0000	3000000.
2	1500.0000	108.00000	6678285.	9161.0000	3000000.

Soil and Rock Layering Information

The soil profile is modelled using 6 layers

Layer 1 is stiff clay without free water
Distance from top of pile to top of layer = 0.000 in
Distance from top of pile to bottom of layer = 120.000 in

Layer 2 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 120.000 in
Distance from top of pile to bottom of layer = 240.000 in
p-y subgrade modulus k for top of soil layer = 60.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 60.000 lbs/in**3

Layer 3 is stiff clay without free water
Distance from top of pile to top of layer = 240.000 in
Distance from top of pile to bottom of layer = 720.000 in

Layer 4 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 720.000 in
Distance from top of pile to bottom of layer = 840.000 in
p-y subgrade modulus k for top of soil layer = 90.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 90.000 lbs/in**3

Layer 5 is stiff clay without free water
Distance from top of pile to top of layer = 840.000 in
Distance from top of pile to bottom of layer = 1080.000 in

Layer 6 is stiff clay without free water
Distance from top of pile to top of layer = 1080.000 in
Distance from top of pile to bottom of layer = 1800.000 in

(Depth of lowest layer extends 300.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 12 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	0.00	0.06900
2	120.00	0.06900
3	120.00	0.03600
4	240.00	0.03600
5	240.00	0.03600
6	720.00	0.03600
7	720.00	0.03600
8	840.00	0.03600
9	840.00	0.03600
10	1080.00	0.03600
11	1080.00	0.03600
12	1800.00	0.03600

Shear Strength of Soils

Shear strength parameters with depth defined using 12 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	0.000	13.89000	0.00	0.00500	0.0
2	120.000	13.89000	0.00	0.00500	0.0
3	120.000	0.00000	33.00	------	------
4	240.000	0.00000	33.00	------	------
5	240.000	19.10000	0.00	0.00500	0.0
6	720.000	19.10000	0.00	0.00500	0.0
7	720.000	0.00000	35.00	------	------
8	840.000	0.00000	35.00	------	------
9	840.000	31.25000	0.00	0.00500	0.0
10	1080.000	31.25000	0.00	0.00500	0.0
11	1080.000	41.67000	0.00	0.00400	0.0
12	1800.000	41.67000	0.00	0.00400	0.0

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 661000.000 lbs
Bending moment at pile head = 263520000.000 in-lbs
Axial load at pile head = 0.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

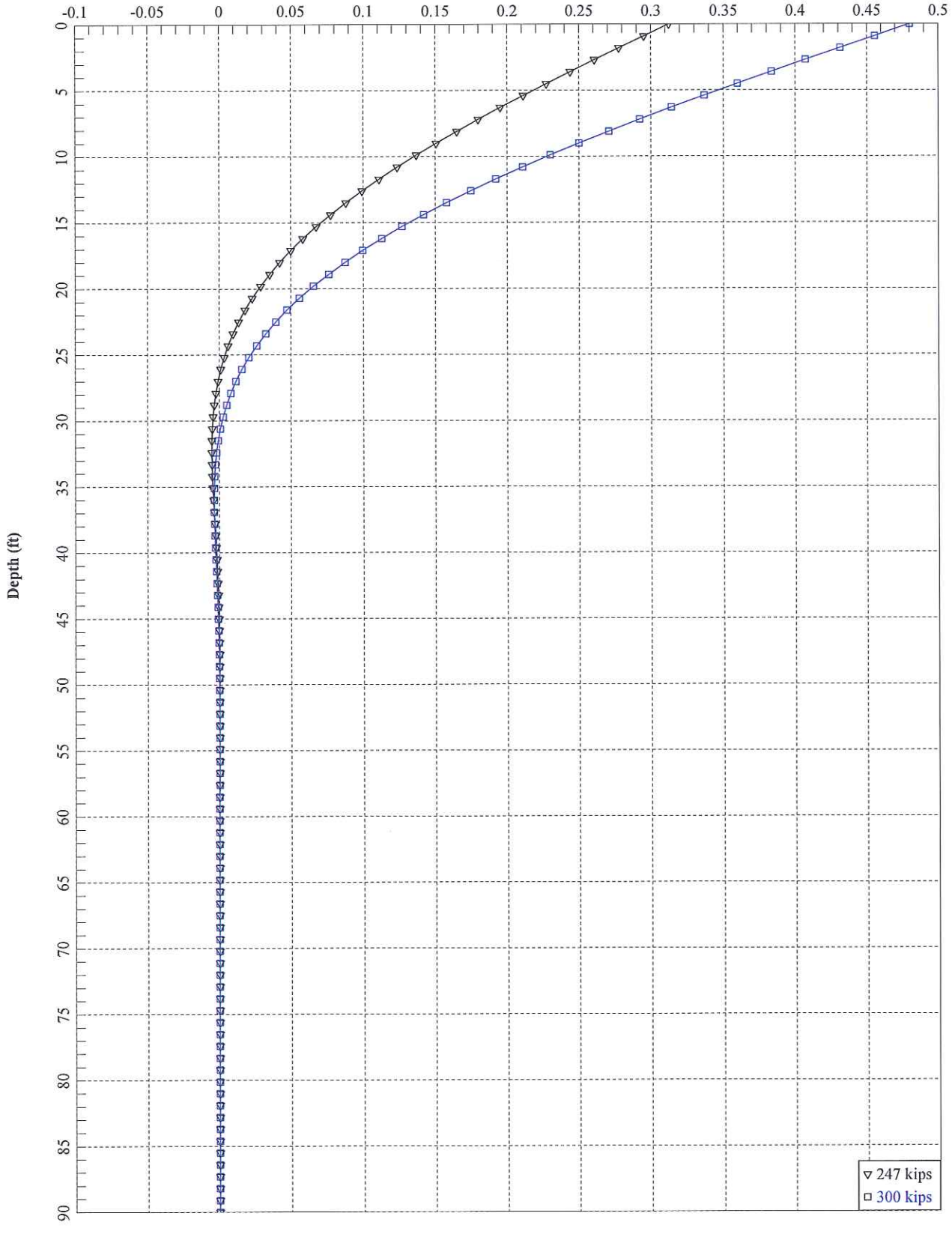
Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (Pile-head Condition Type 1)
Specified shear force at pile head = 661000.000 lbs
Specified moment at pile head = 263520000.000 in-lbs
Specified axial load at pile head = 0.000 lbs

Depth X	Deflect. y	Moment M	Shear V	Slope S	Total Stress	Soil Res. Es*h p
in	in	lbs-in	lbs	Rad.	lbs/in**2	lbs/in F/L lbs/in
0.000	1.605	2.6352E+08	661000.	-0.0069806	2130.7986	-2349.7857 10977.7839
15.000	1.502	2.7317E+08	625212.	-0.0067797	2208.8328	-2421.9644 24185.1424
30.000	1.402	2.8228E+08	588376.	-0.0065717	2282.4607	-2489.5431 26636.0302
45.000	1.305	2.9082E+08	550561.	-0.0063572	2351.5593	-2552.3877 29338.0705
60.000	1.211	2.9879E+08	511840.	-0.0061365	2416.0143	-2610.3609 32326.1181
75.000	1.121	3.0618E+08	472288.	-0.0059100	2475.7202	-2663.3209 35641.0609
90.000	1.034	3.1296E+08	431980.	-0.0056782	2530.5806	-2711.1226 39331.0786
105.000	0.950546	3.1914E+08	390994.	-0.0054416	2580.5086	-2753.6169 43453.2033
120.000	0.870713	3.2469E+08	322579.	-0.0052006	2625.4269	-6368.3627

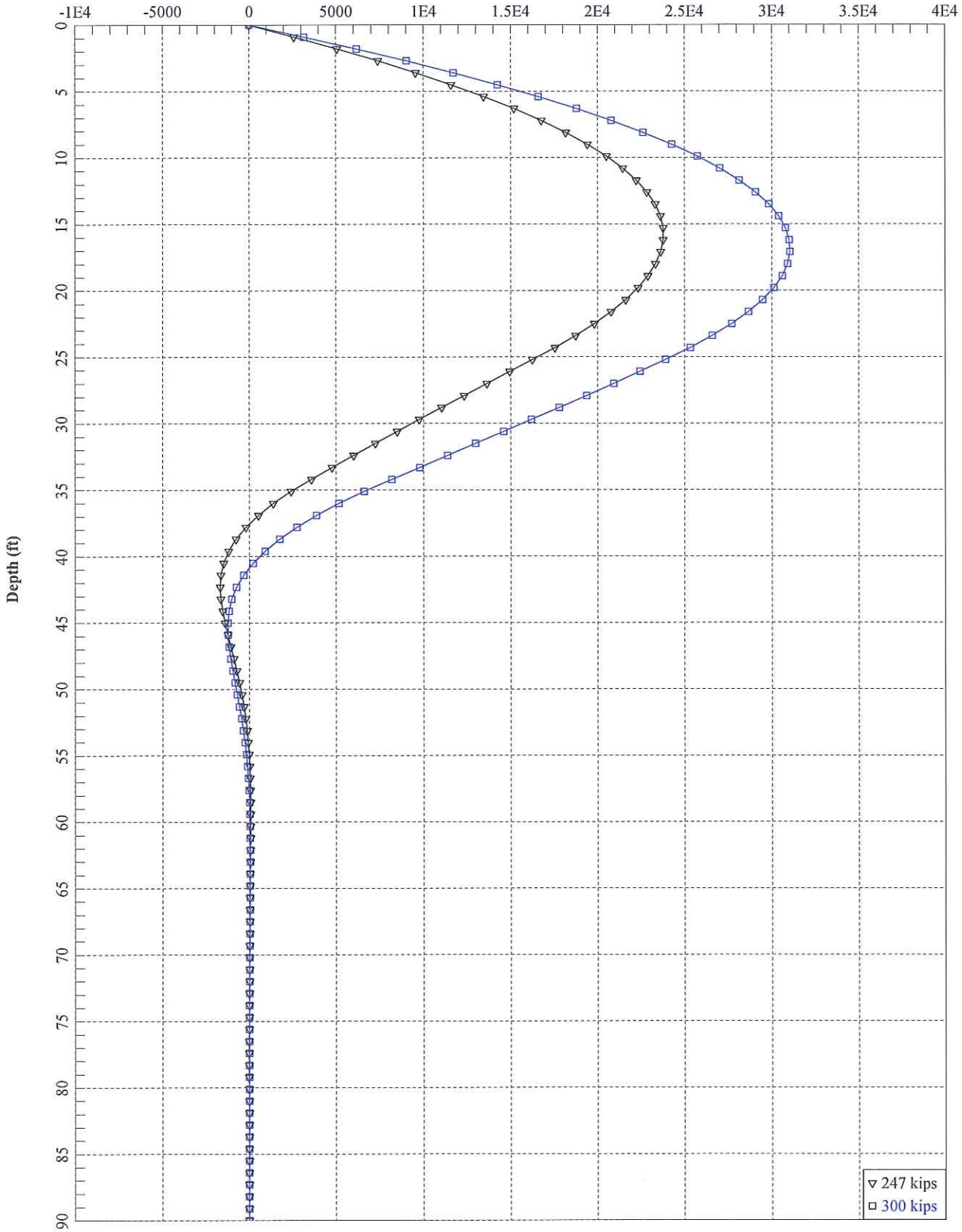
Deflection and Moment Curves for 6 foot Diameter Piles – Shear Force Applied

(6-foot Diameter, Shear Only, Model #1)

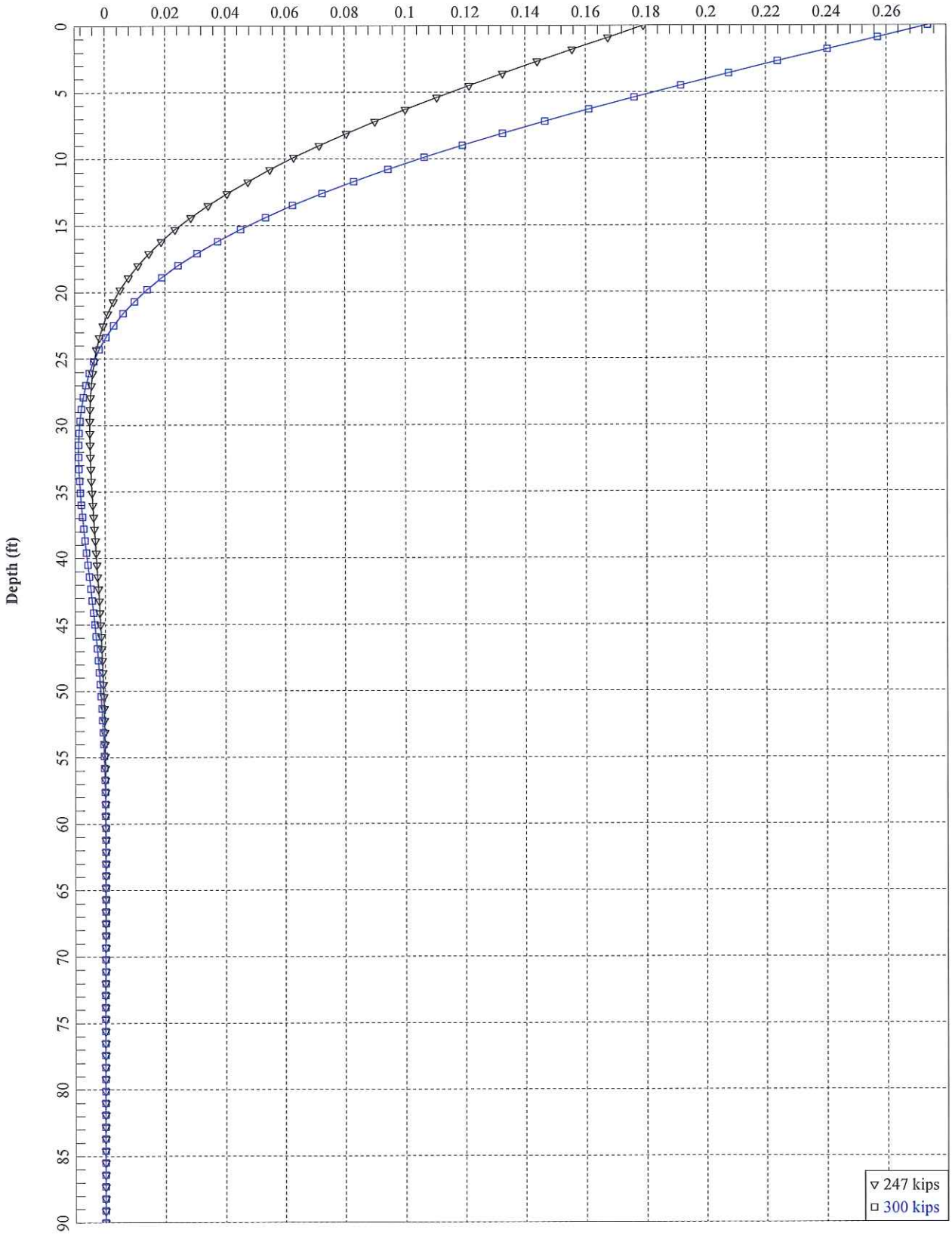


(6-foot Diameter, Shear Only, Model #1)

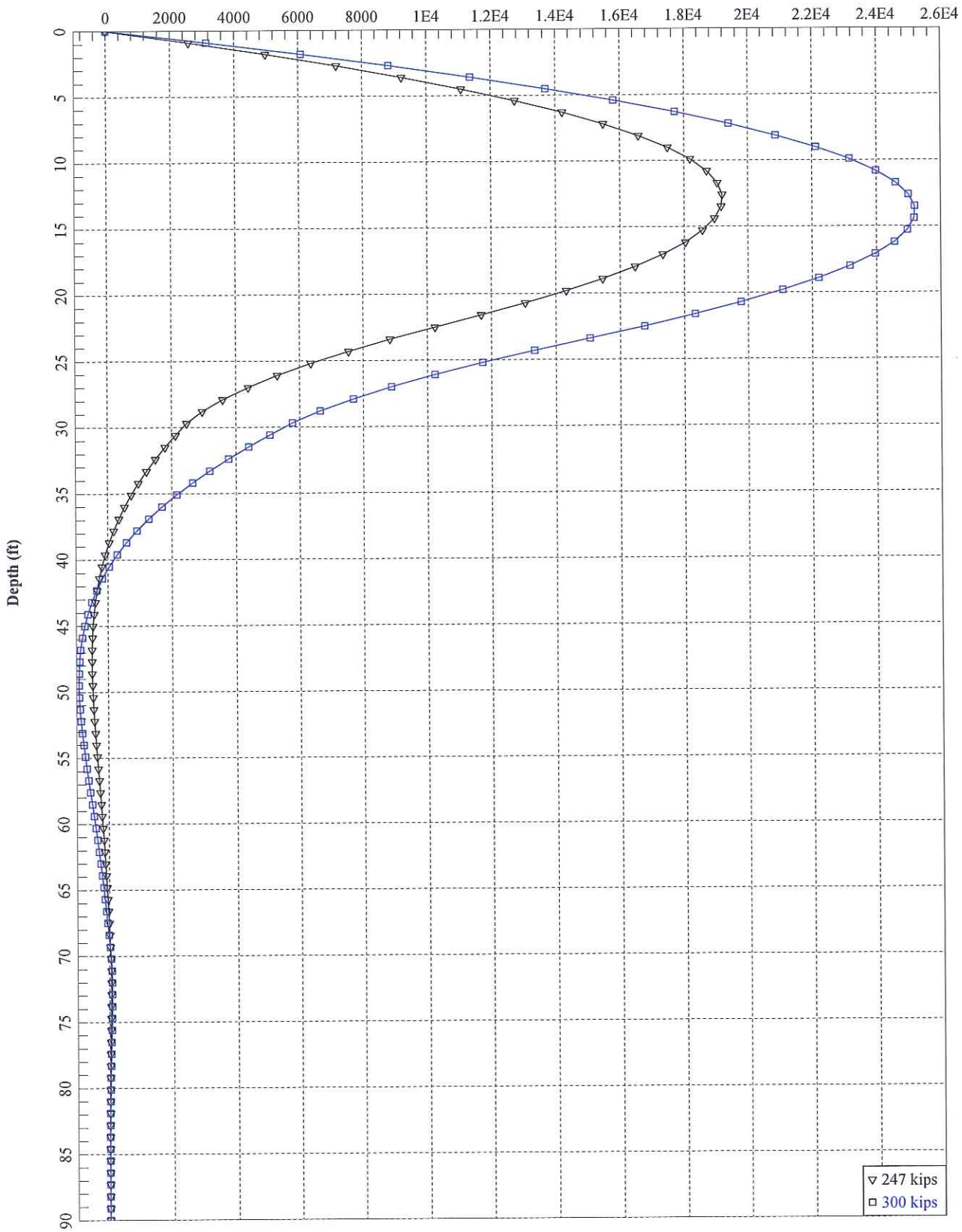
Bending Moment (in-kips)



(6-foot Diameter, Shear Only, Model #2)

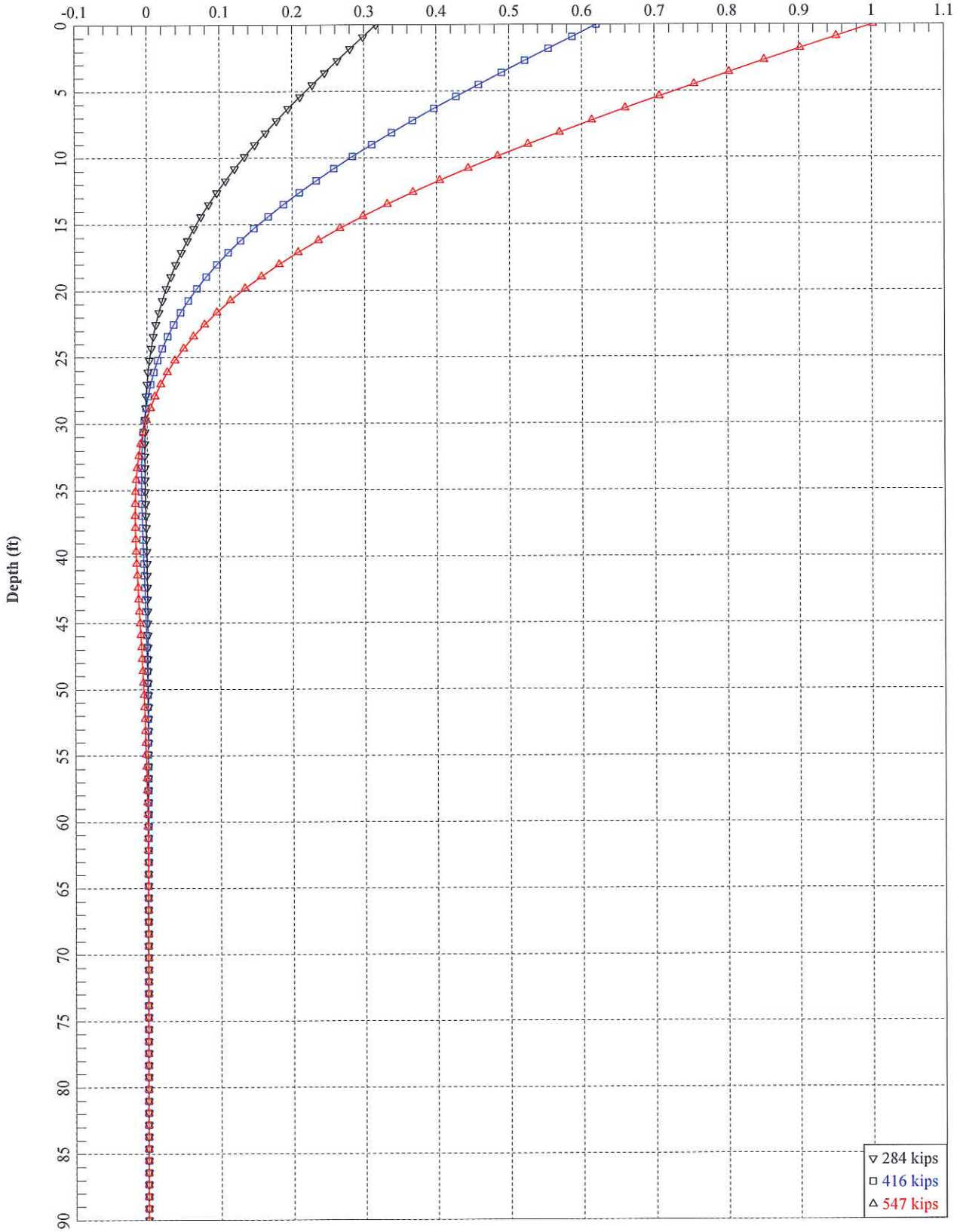


(6-foot Diameter, Shear Only, Model #2)

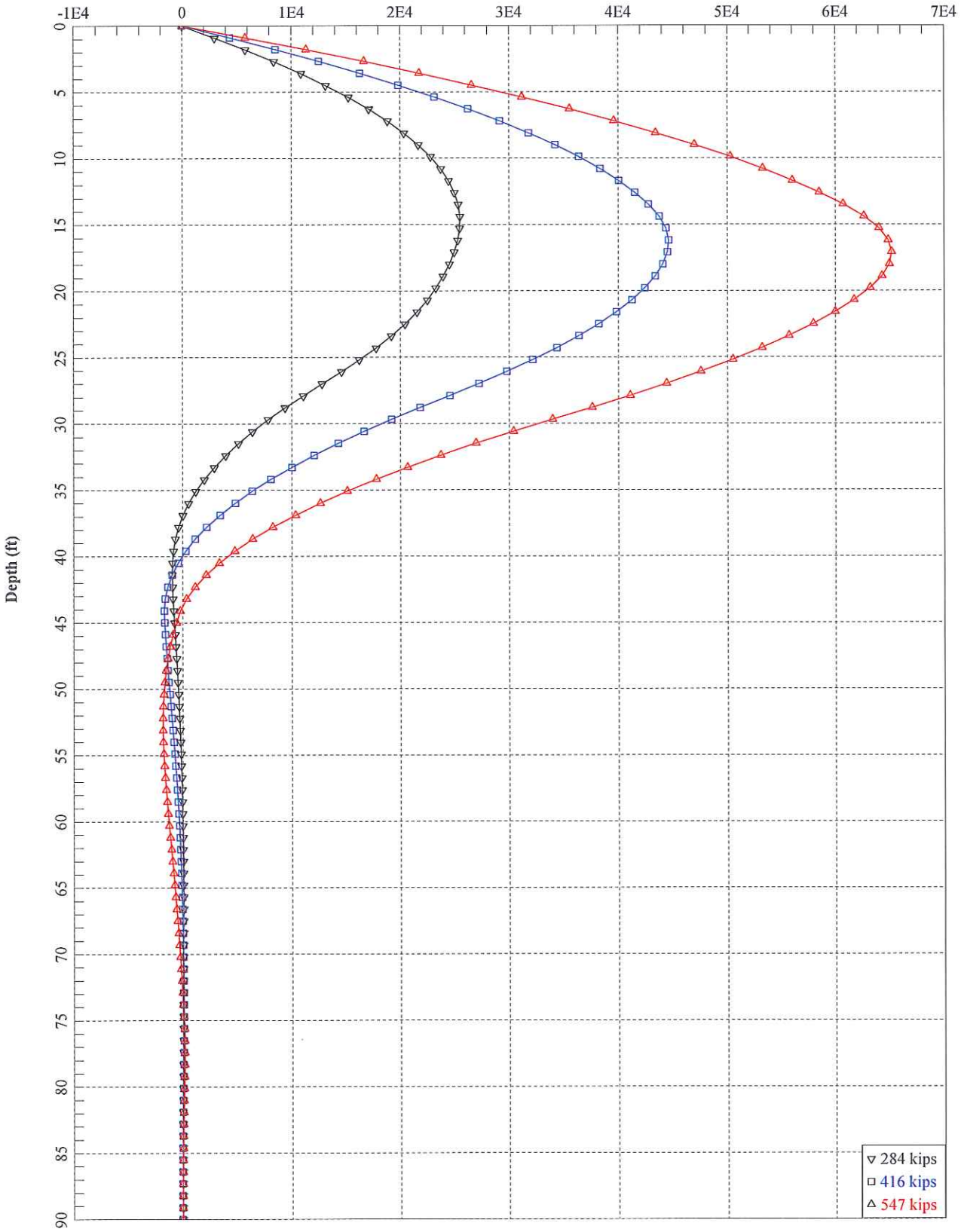


(6-foot Diameter, Shear Only, Model #12)

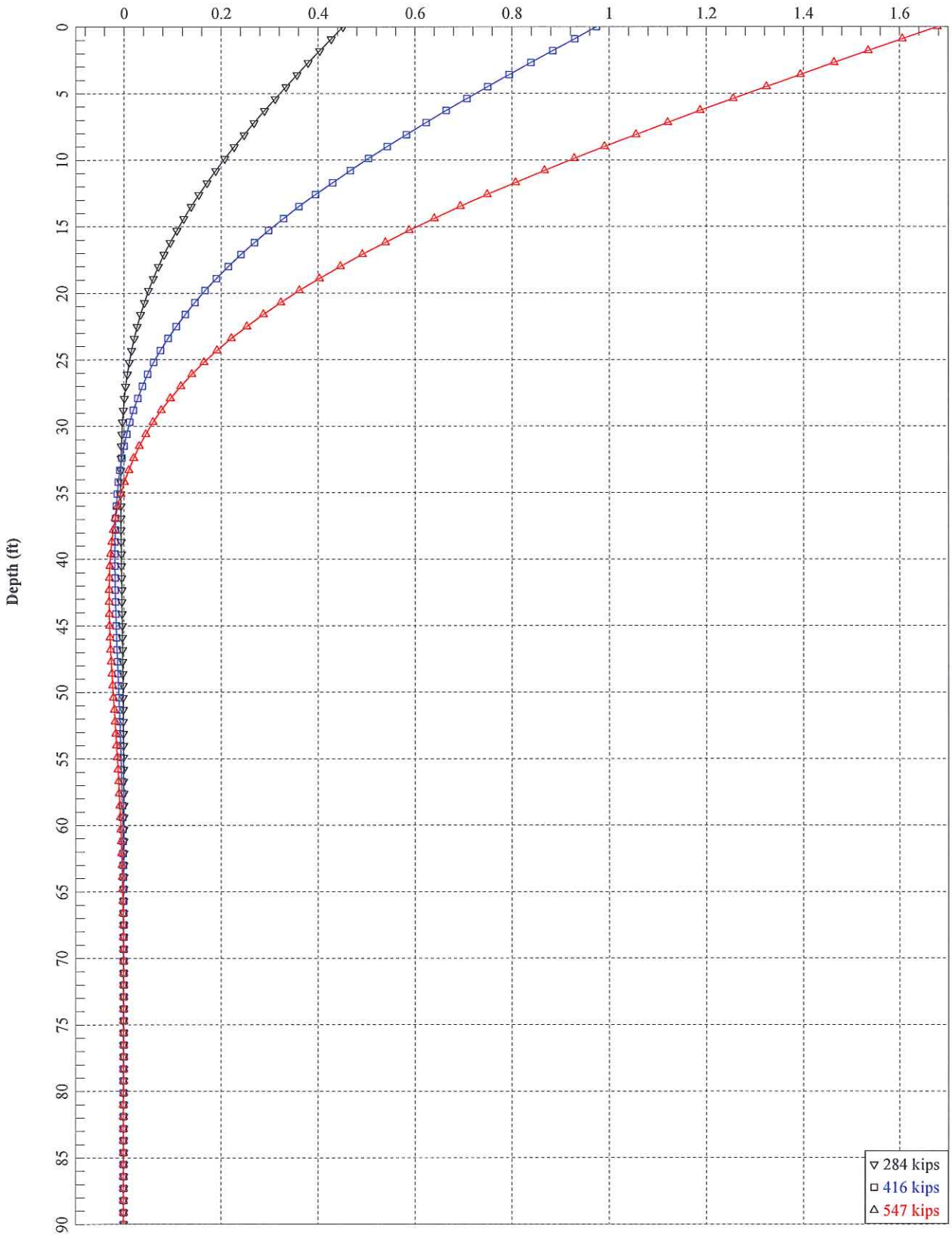
Lateral Deflection (in)



(6-foot Diameter, Shear Only, Model #12)

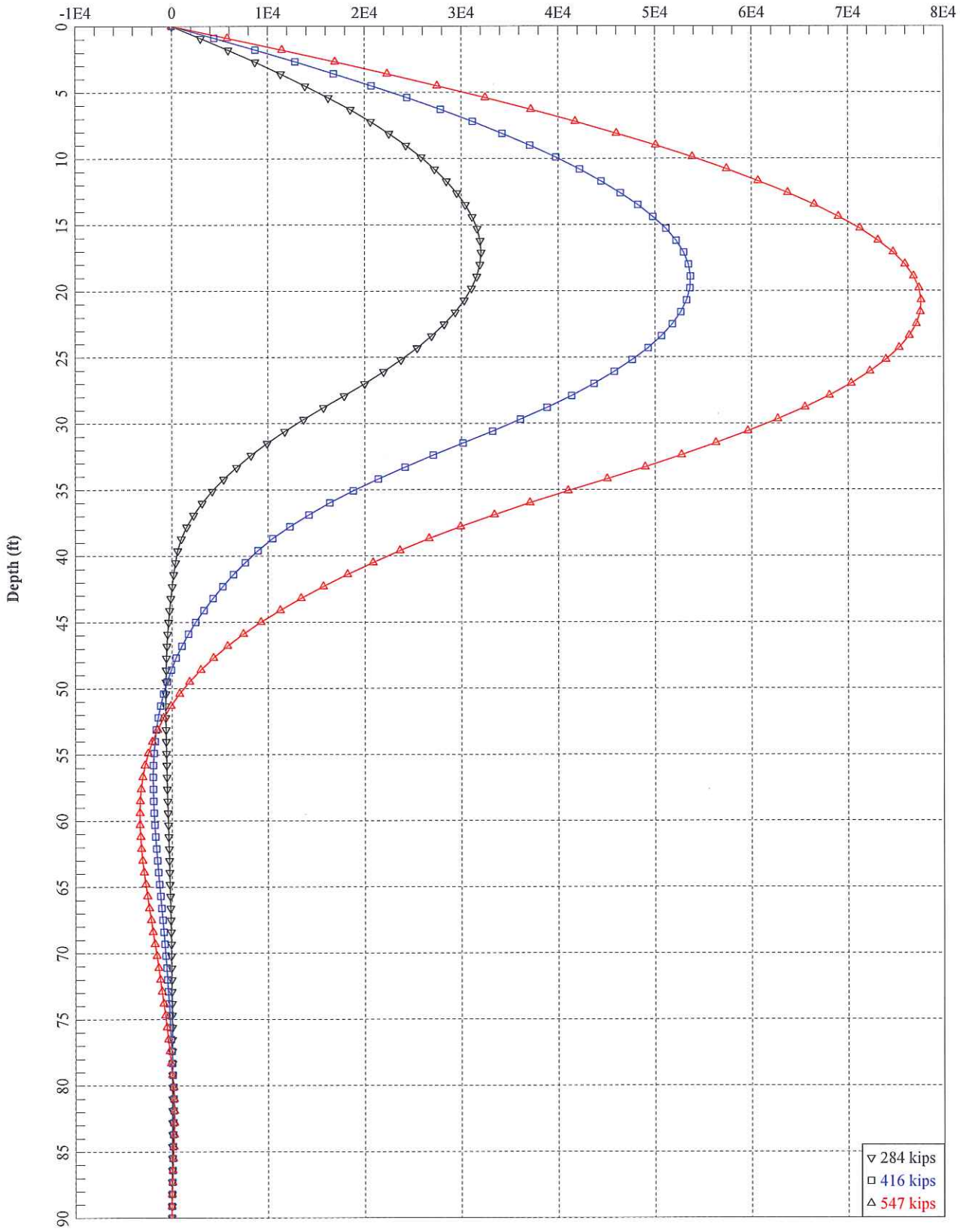


(6-foot Diameter, Shear Only, Model #13)

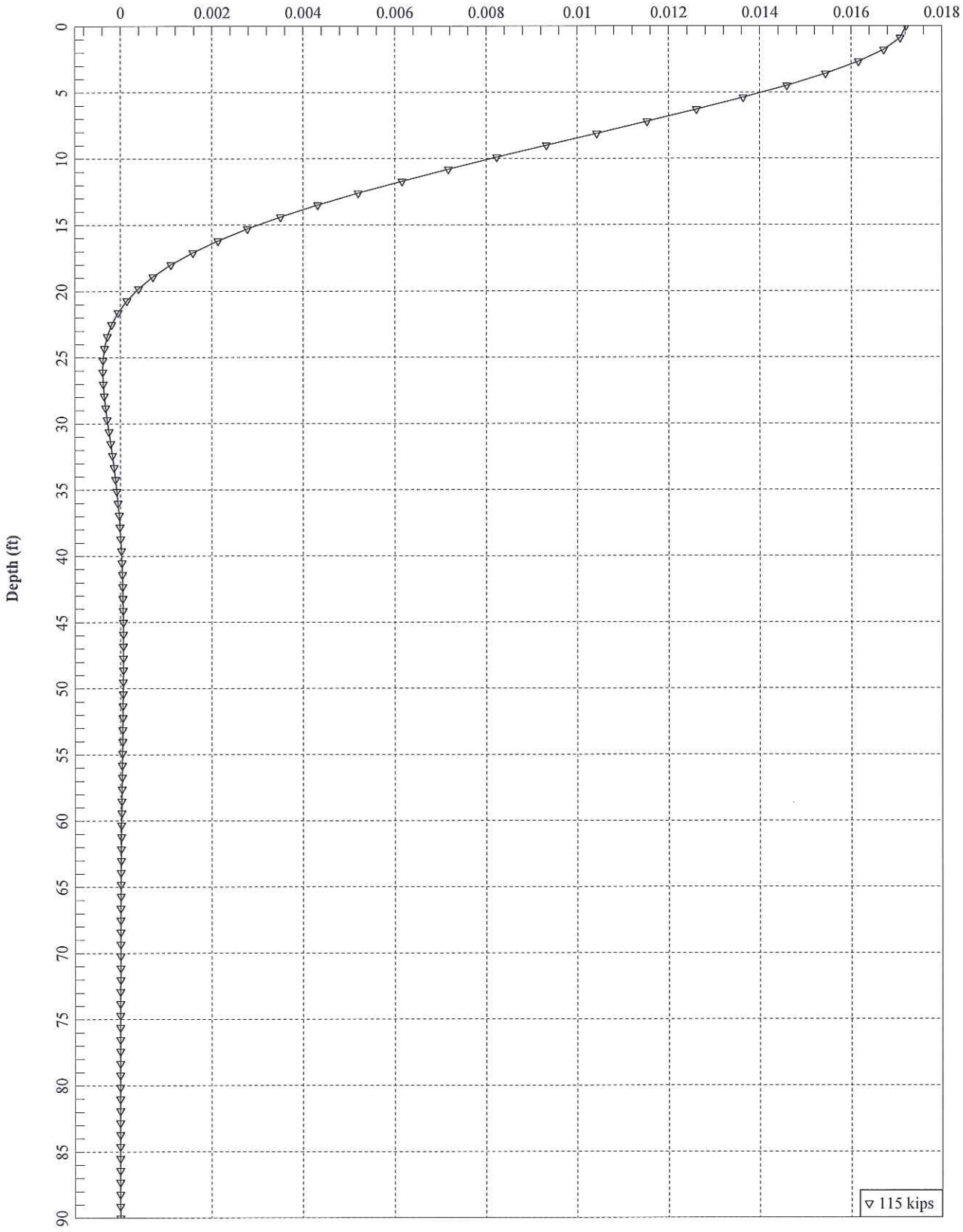


(6-foot Diameter, Shear Only, Model #13)

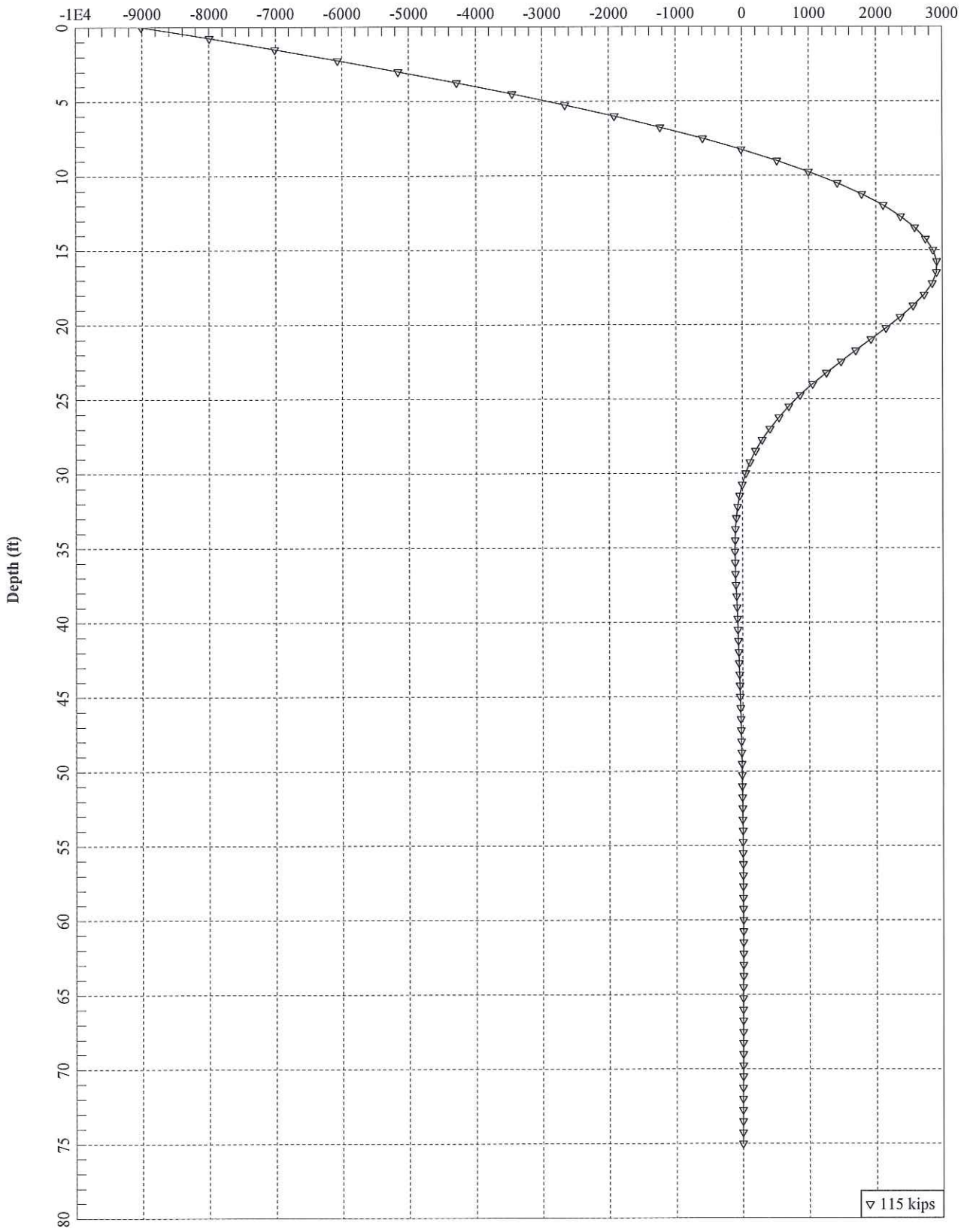
Bending Moment (in-kips)



(6-foot Diameter, Shear Only, Fixed Head, Model #13)

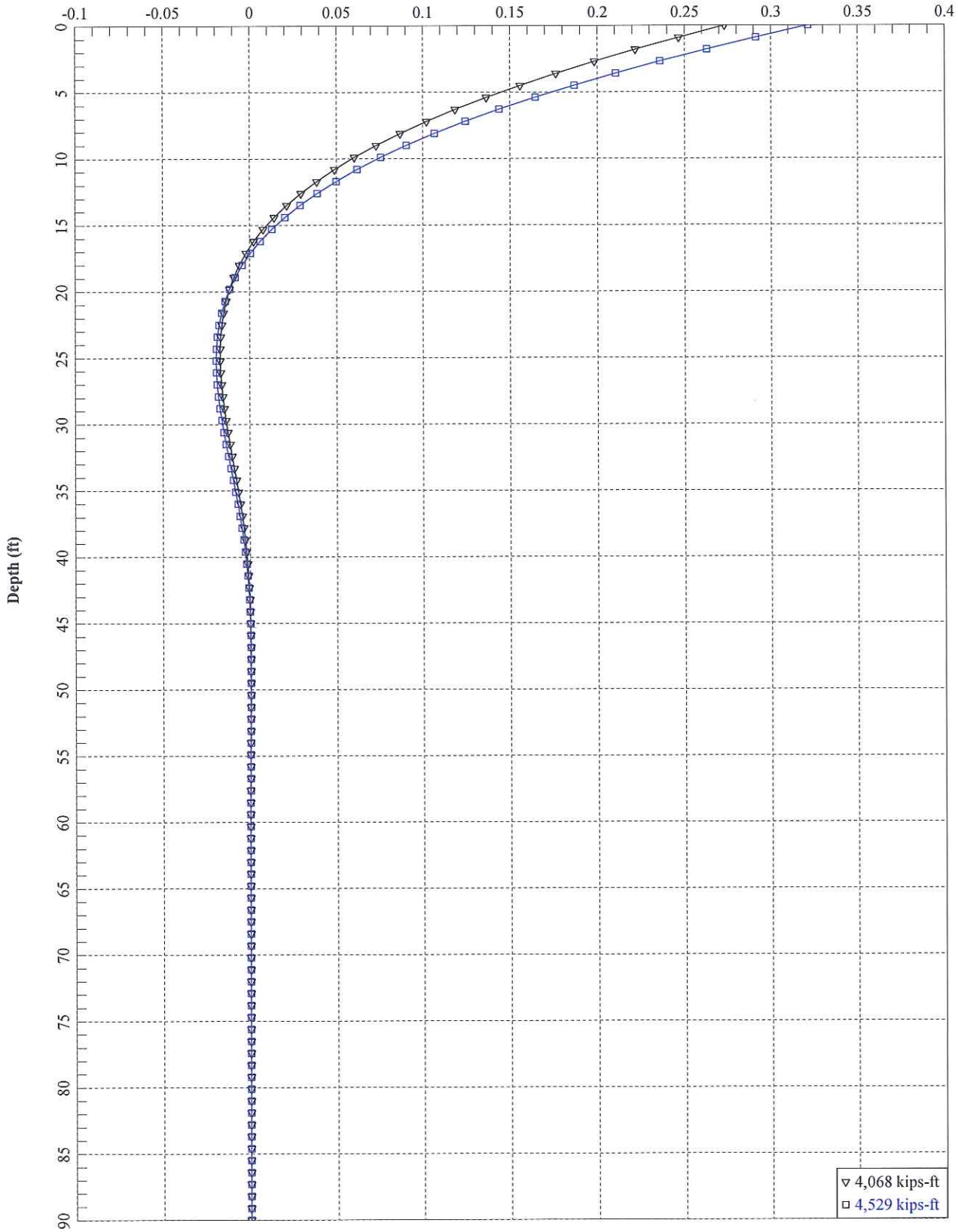


(6-foot Diameter, Shear Only, Fixed Head, Model #13)

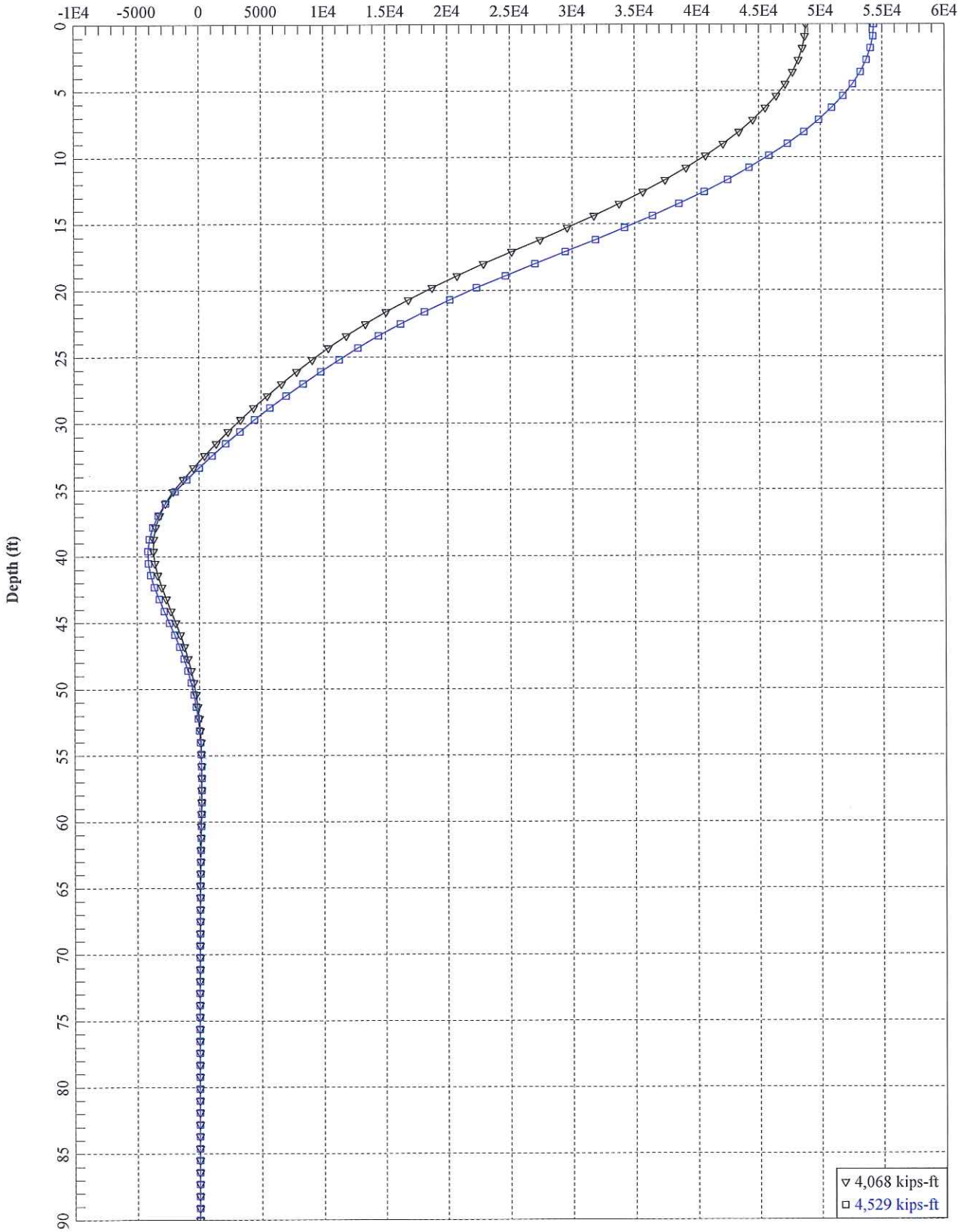


Deflection and Moment Curves for 6 foot Diameter Piles – Moment Applied

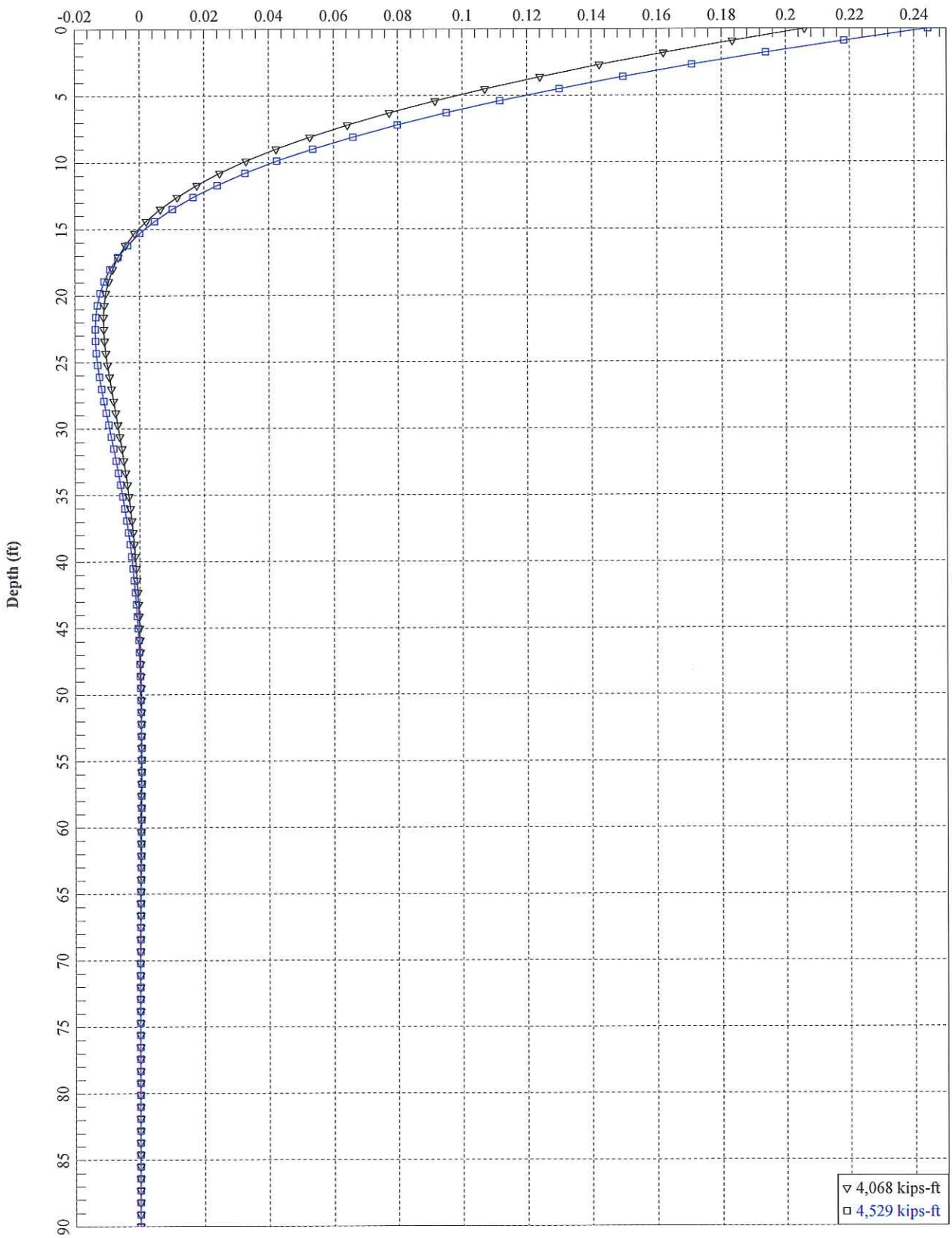
(6-foot Diameter, Moment Only, Model #1)



(6-foot Diameter, Moment Only, Model #1)

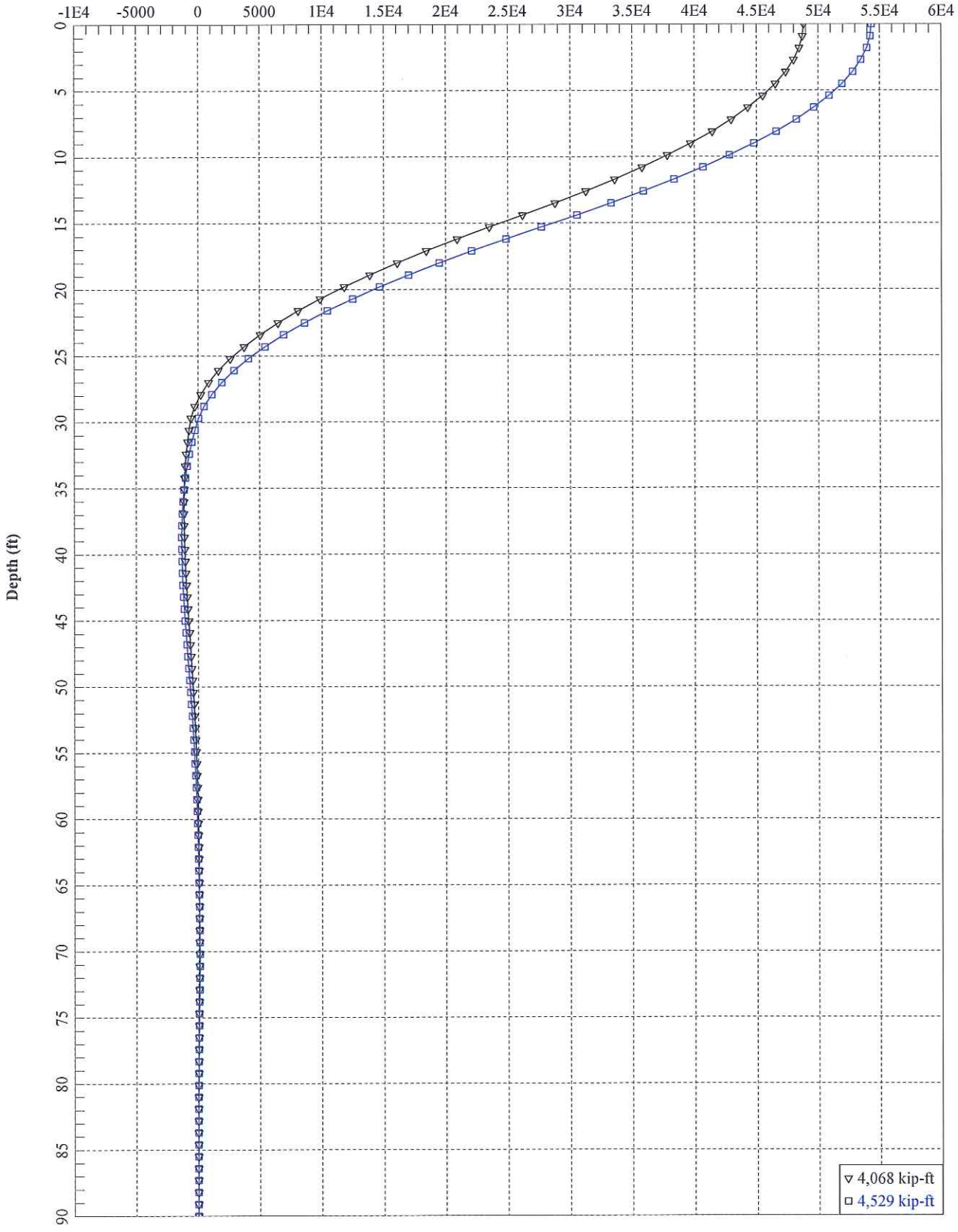


(6-foot Diameter, Moment Only, Model #2)



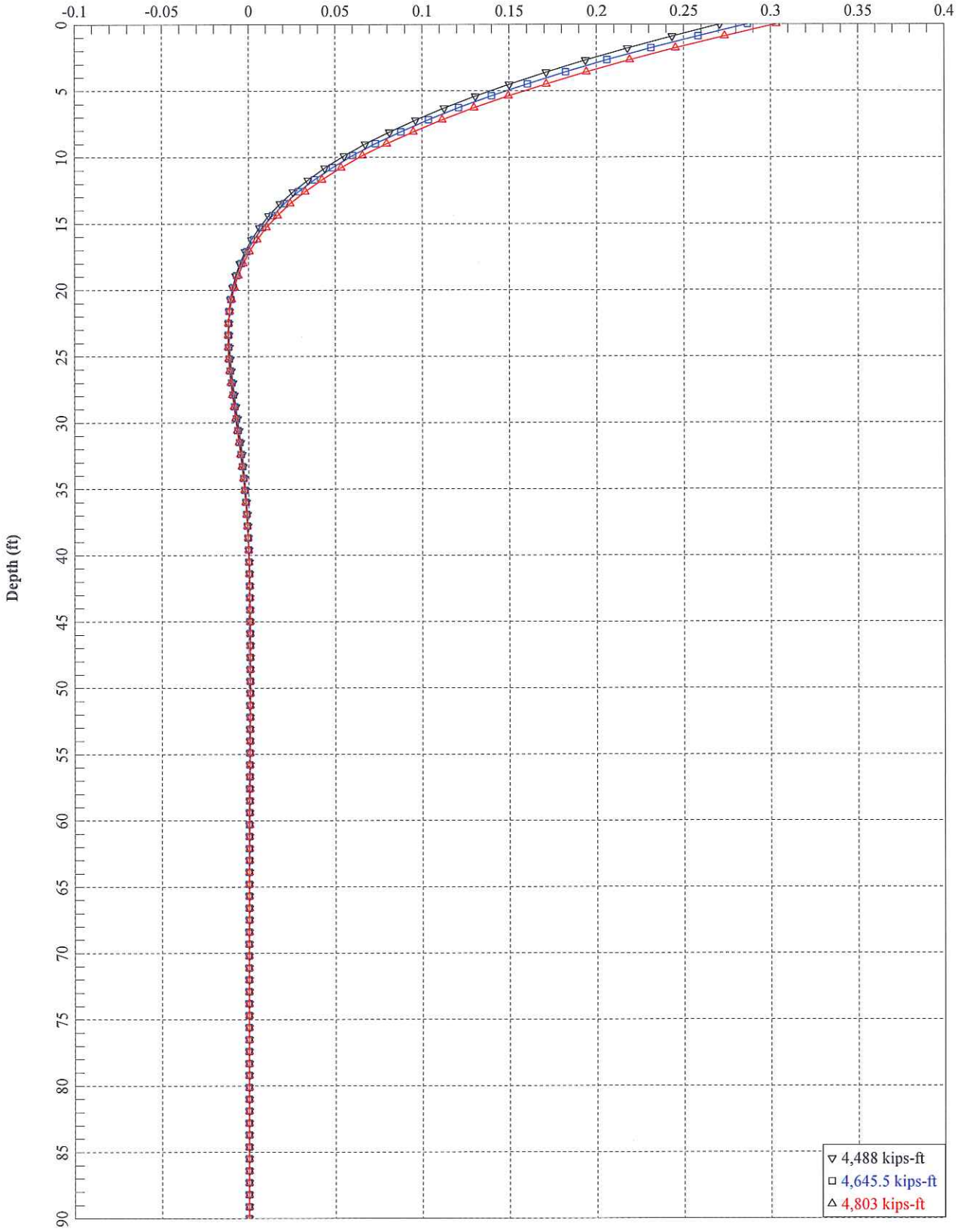
(6-foot Diameter, Moment Only, Model #2)

Bending Moment (in-kips)

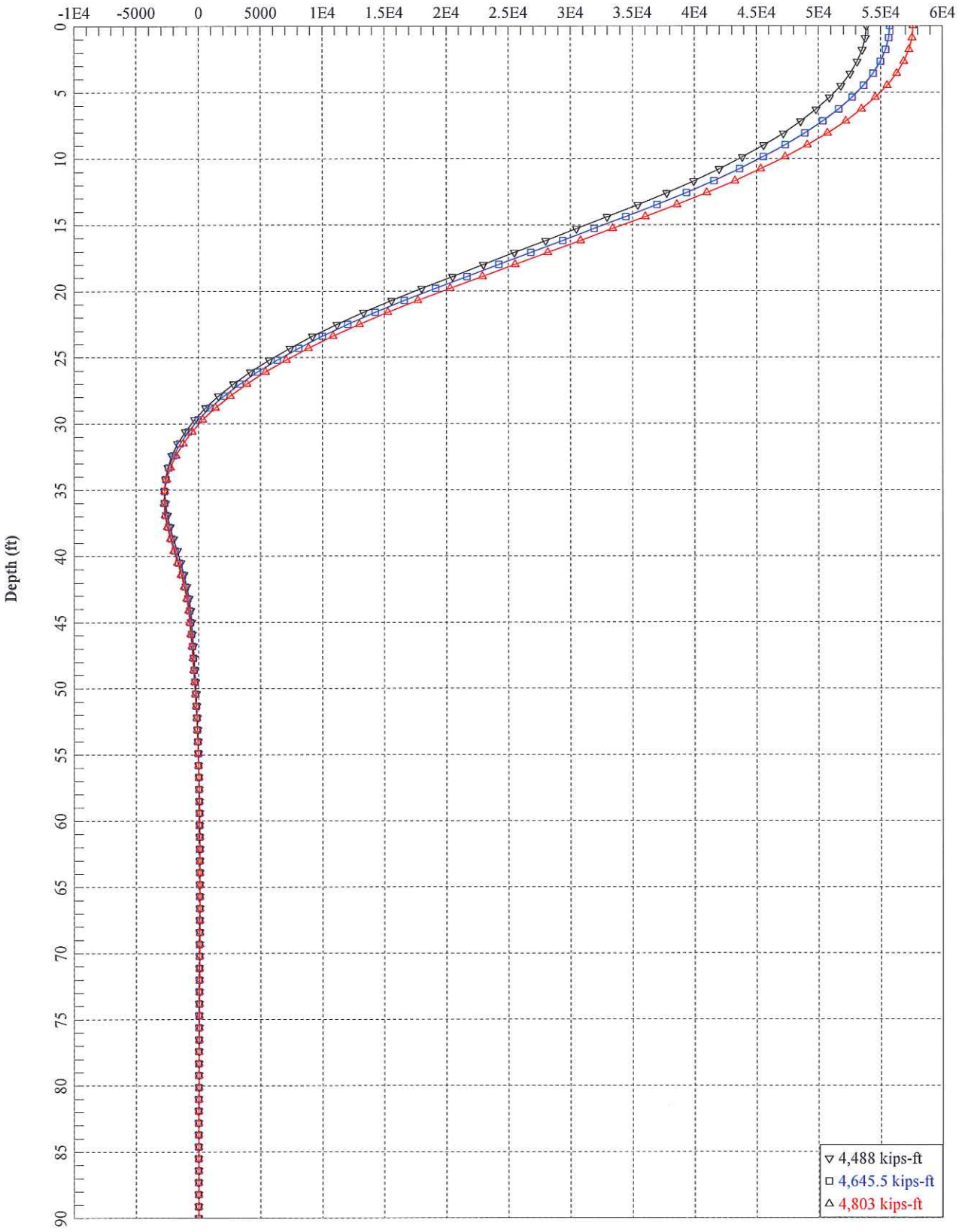


(6-foot Diameter, Moment Only, Model #12)

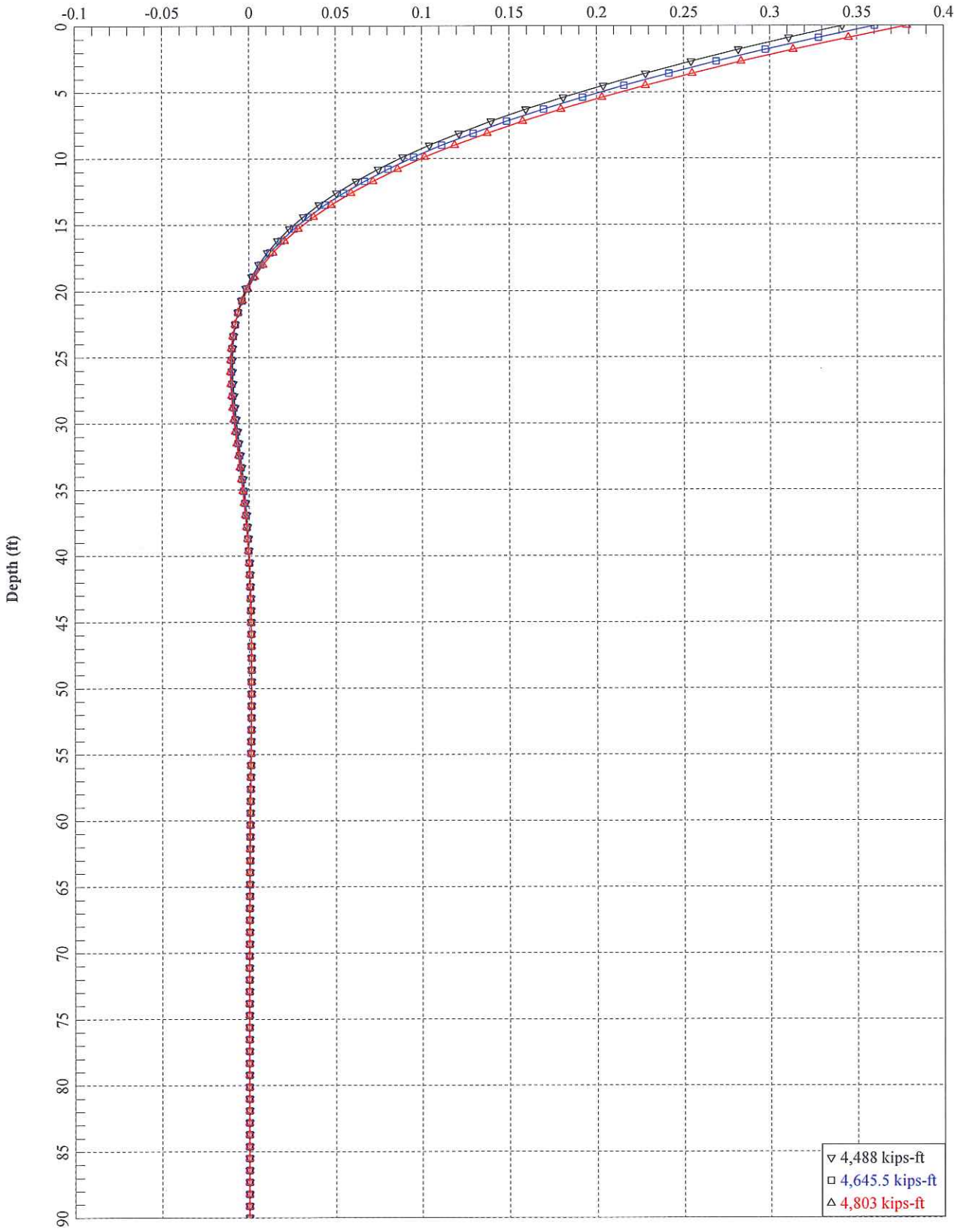
Lateral Deflection (in)



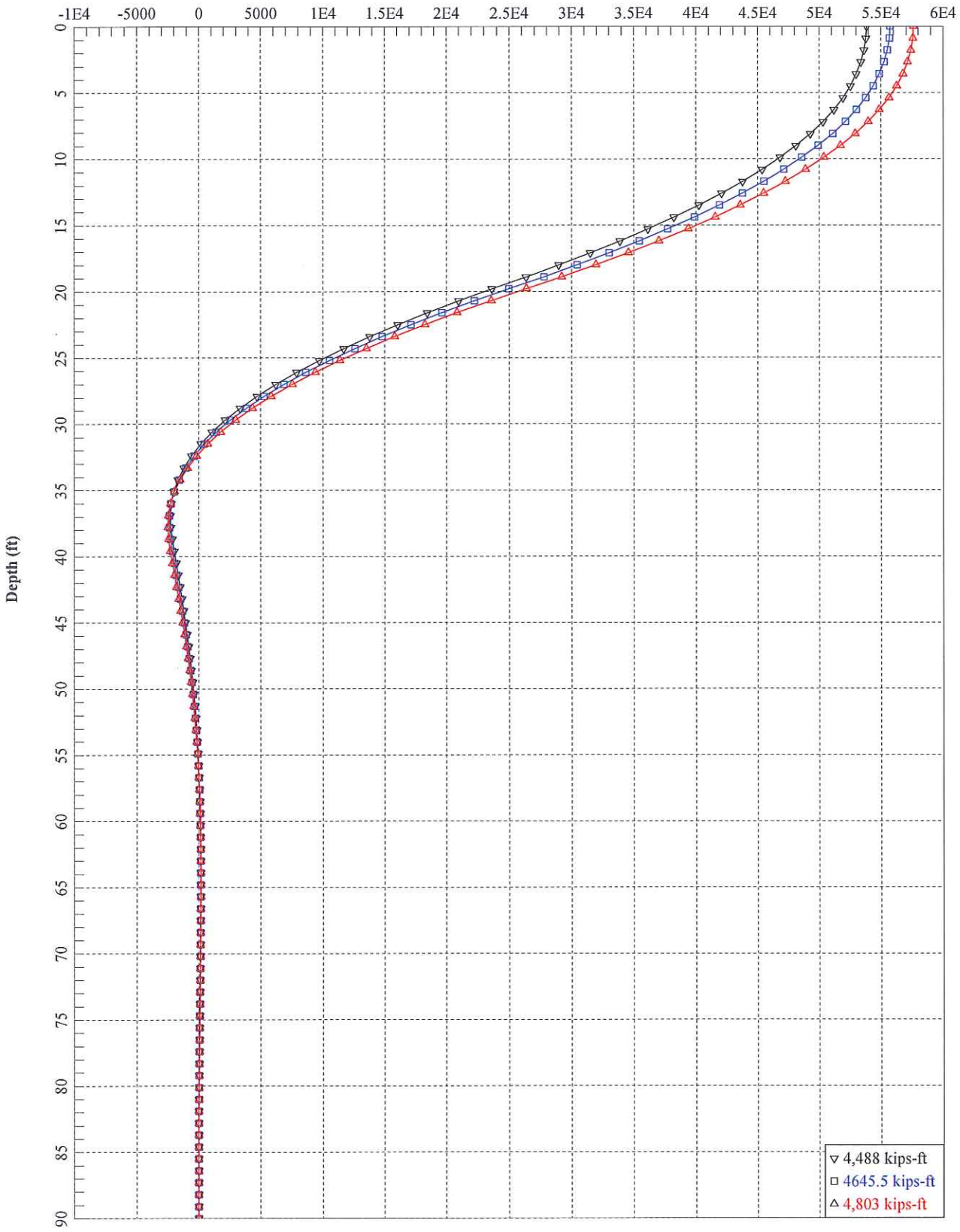
(6-foot Diameter, Moment Only, Model #12)



(6-foot Diameter, Moment Only, Model #13)

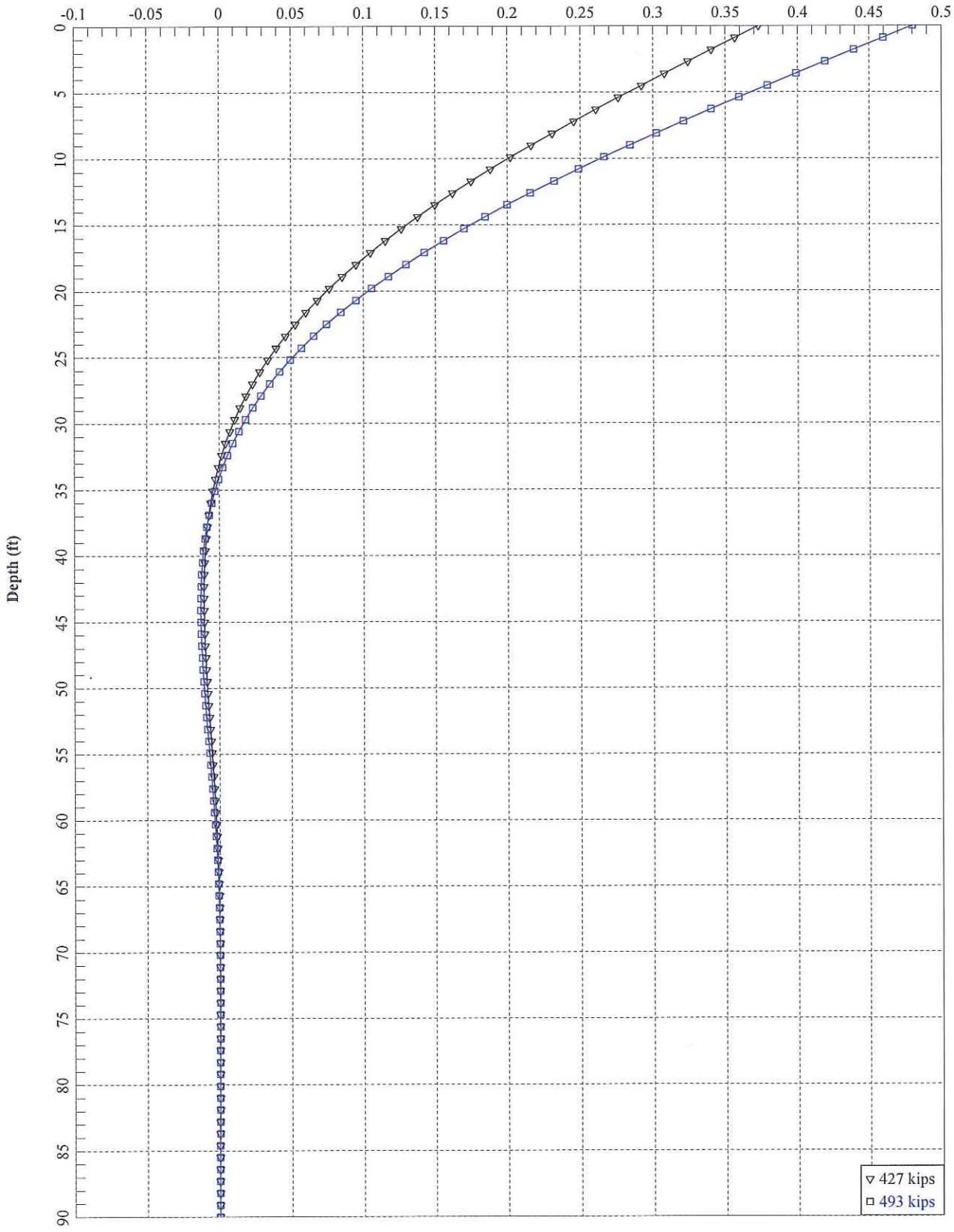


(6-foot Diameter, Moment Only, Model #13)

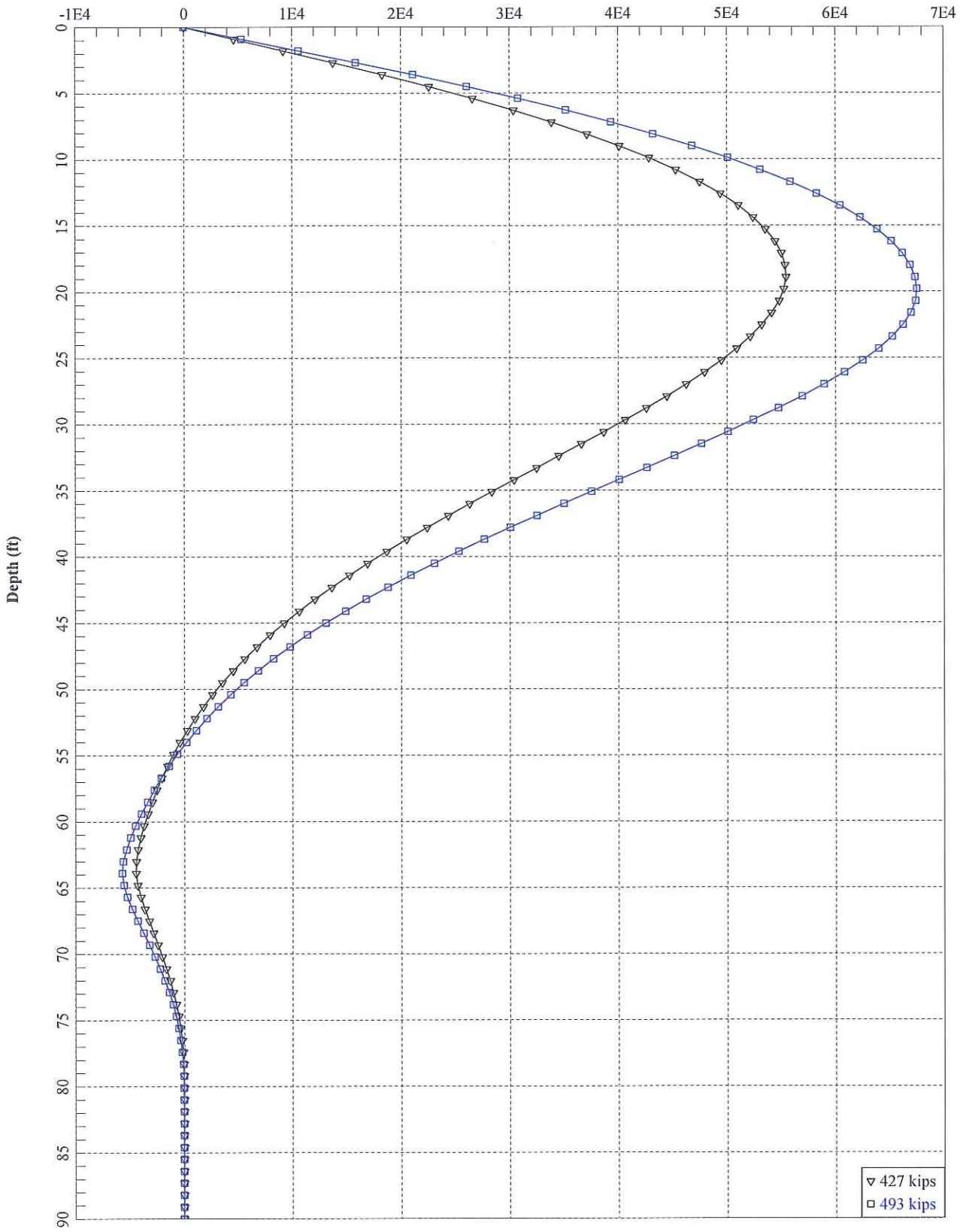


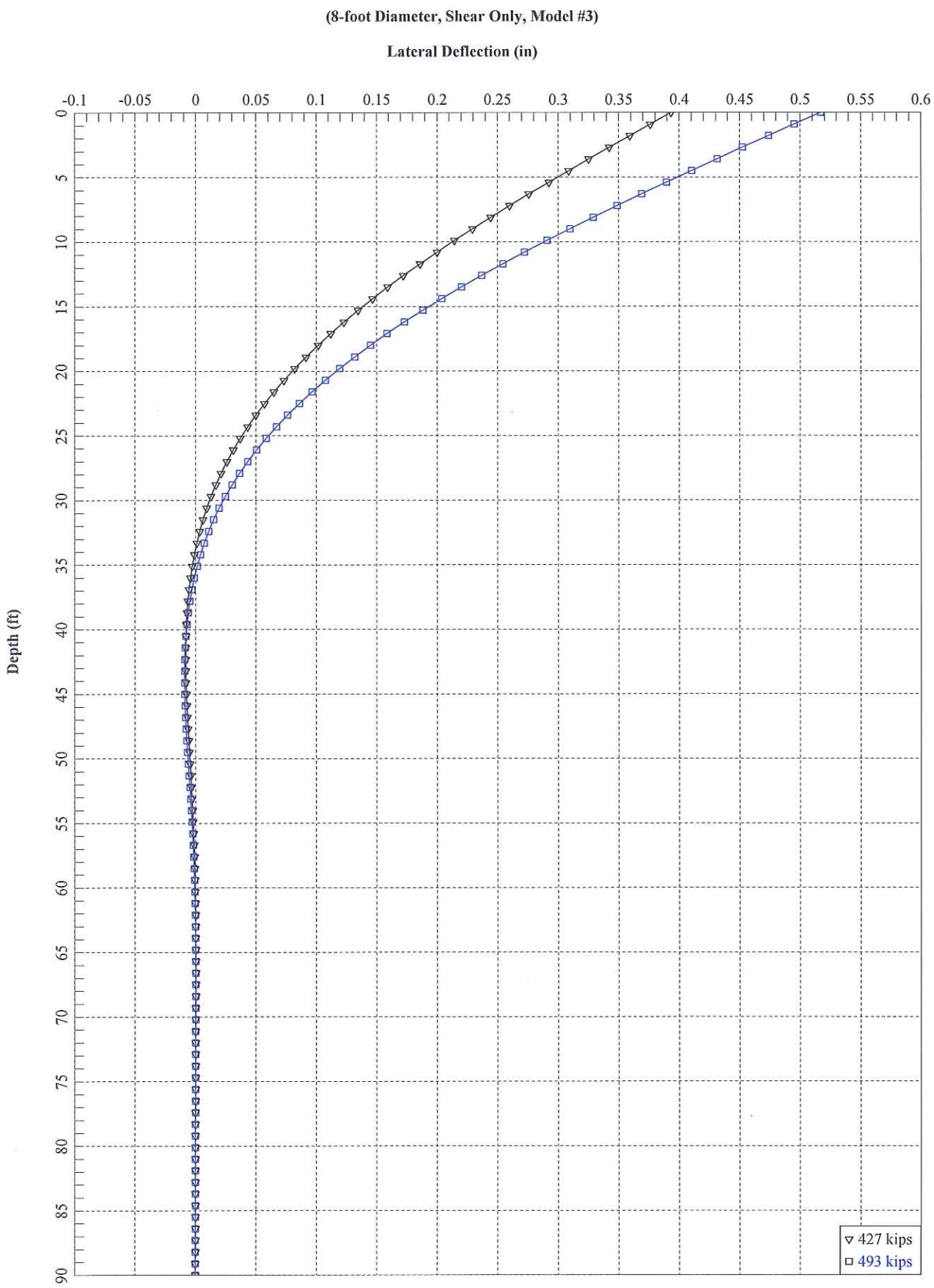
Deflection and Moment Curves for 8 foot Diameter Piles – Shear Force Applied

(8-foot Diameter, Shear Only, Model #2)



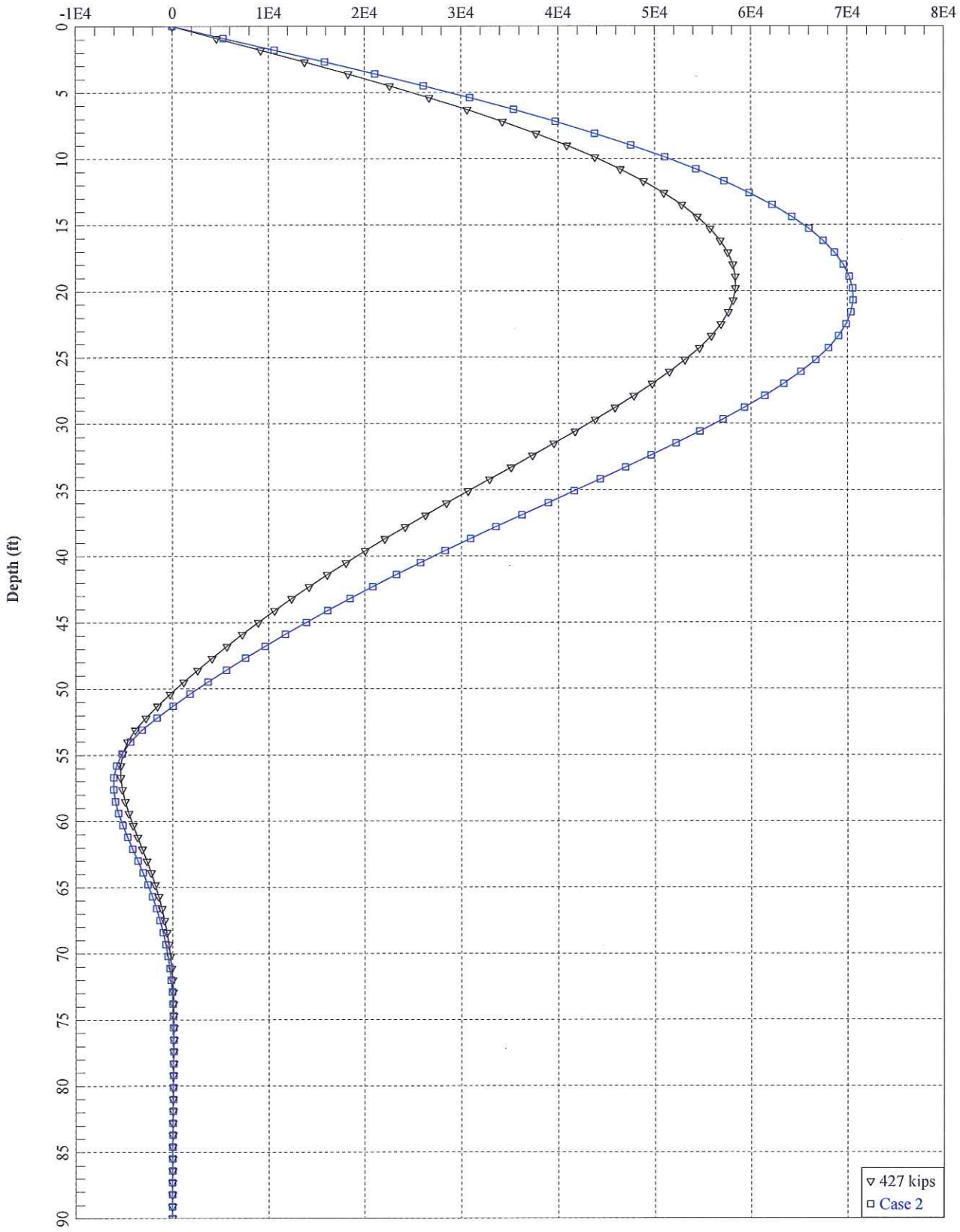
(8-foot Diameter, Shear Only, Model #2)



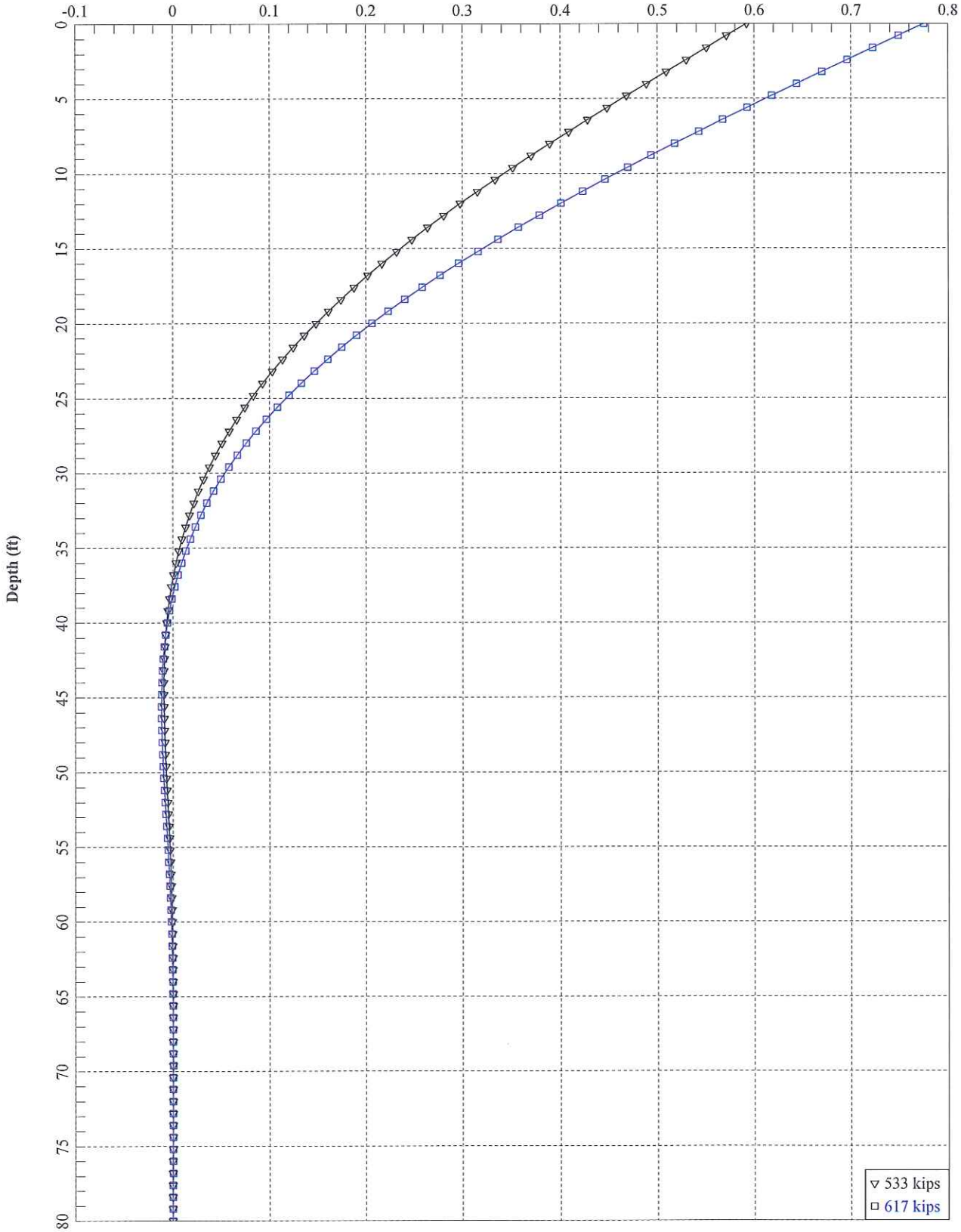

(8-foot Diameter, Shear Only, Model #3)
Lateral Deflection (in)
-0.1
-0.05
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
0.4
0.45
0.5
0.55
0.6
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
85
90
427 kips
493 kips

(8-foot Diameter, Shear Only, Model #3)

Bending Moment (in-kips)

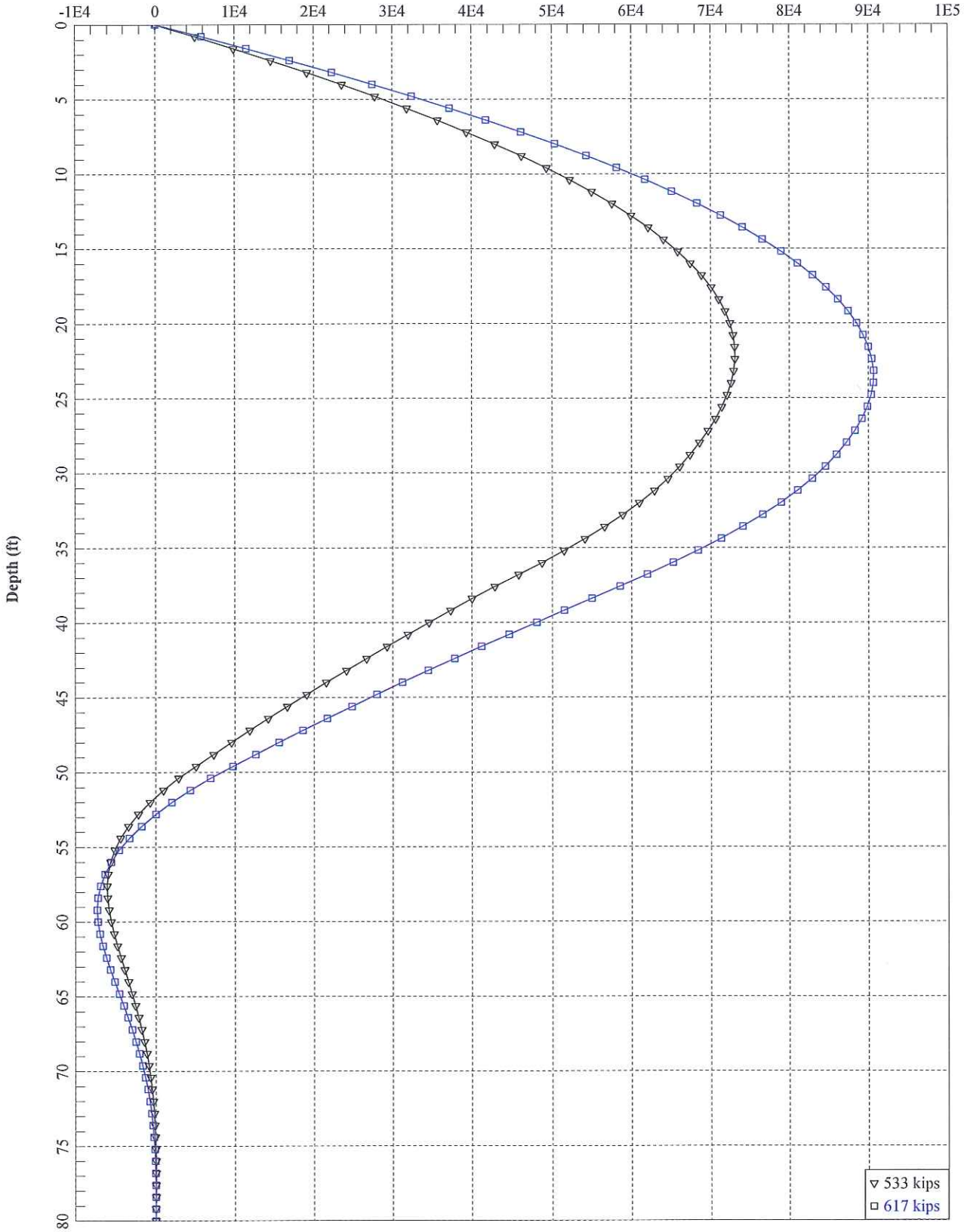


(8-foot Diameter, Shear Only, Model #7)

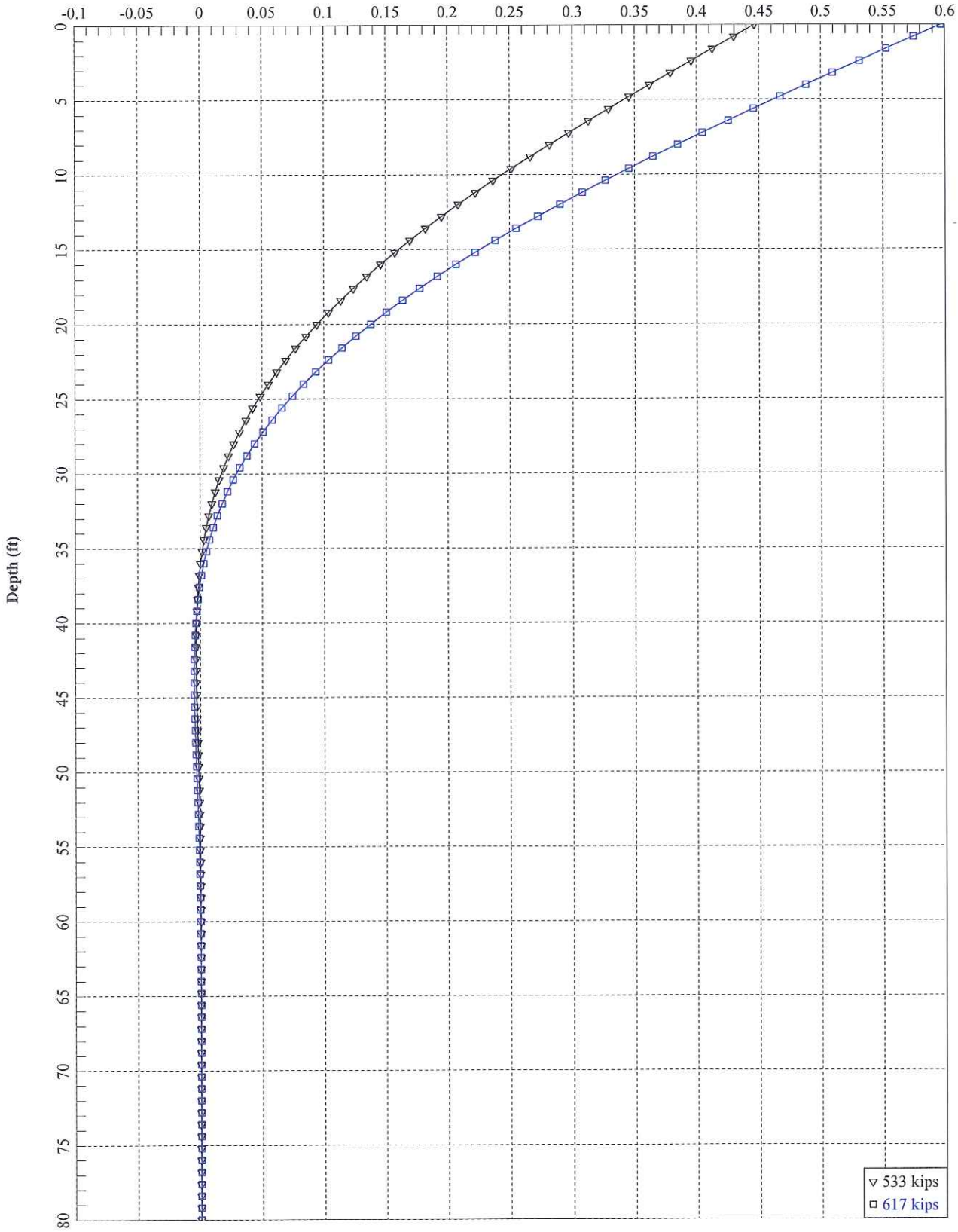


(8-foot Diameter, Shear Only, Model #7)

Bending Moment (in-kips)

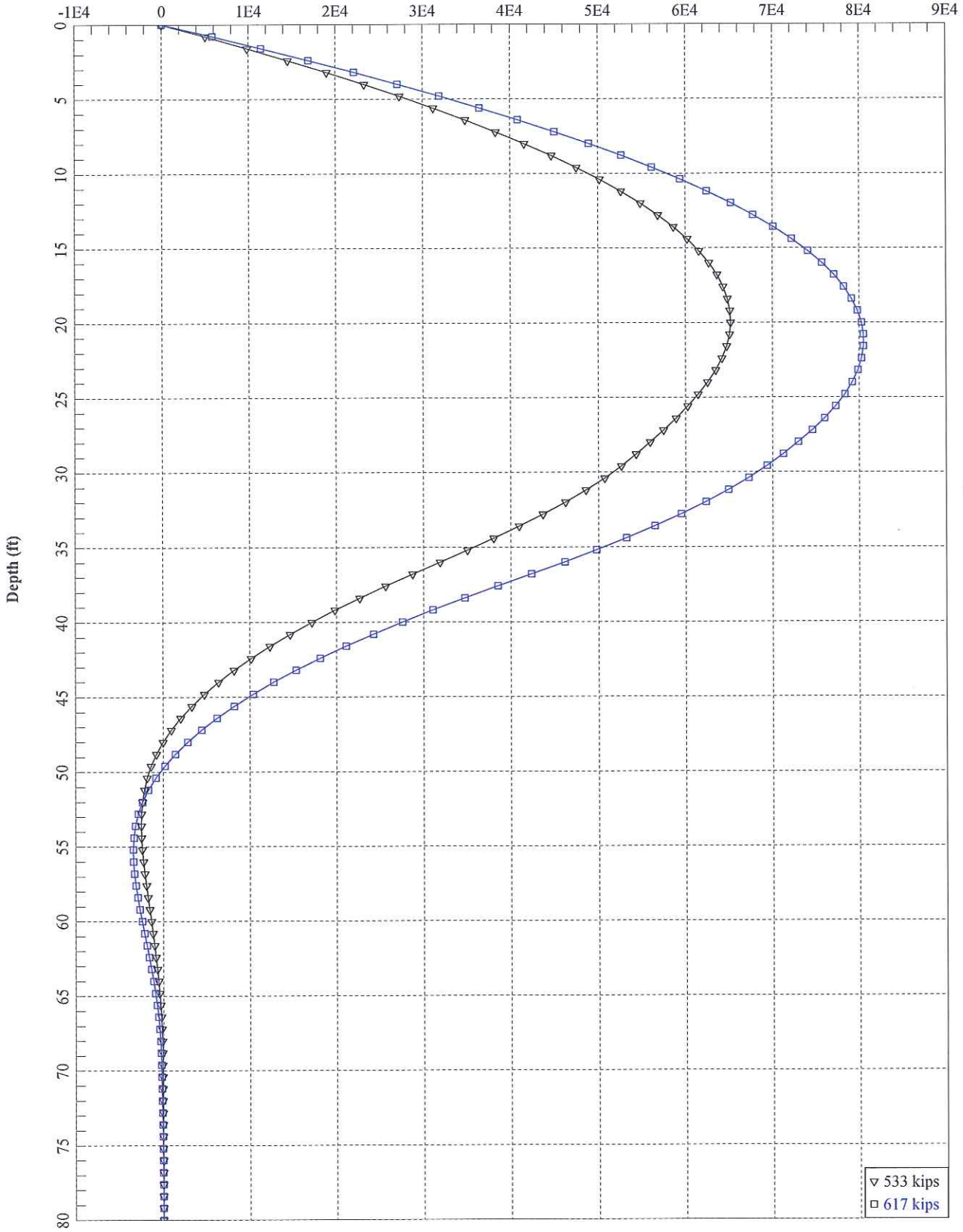


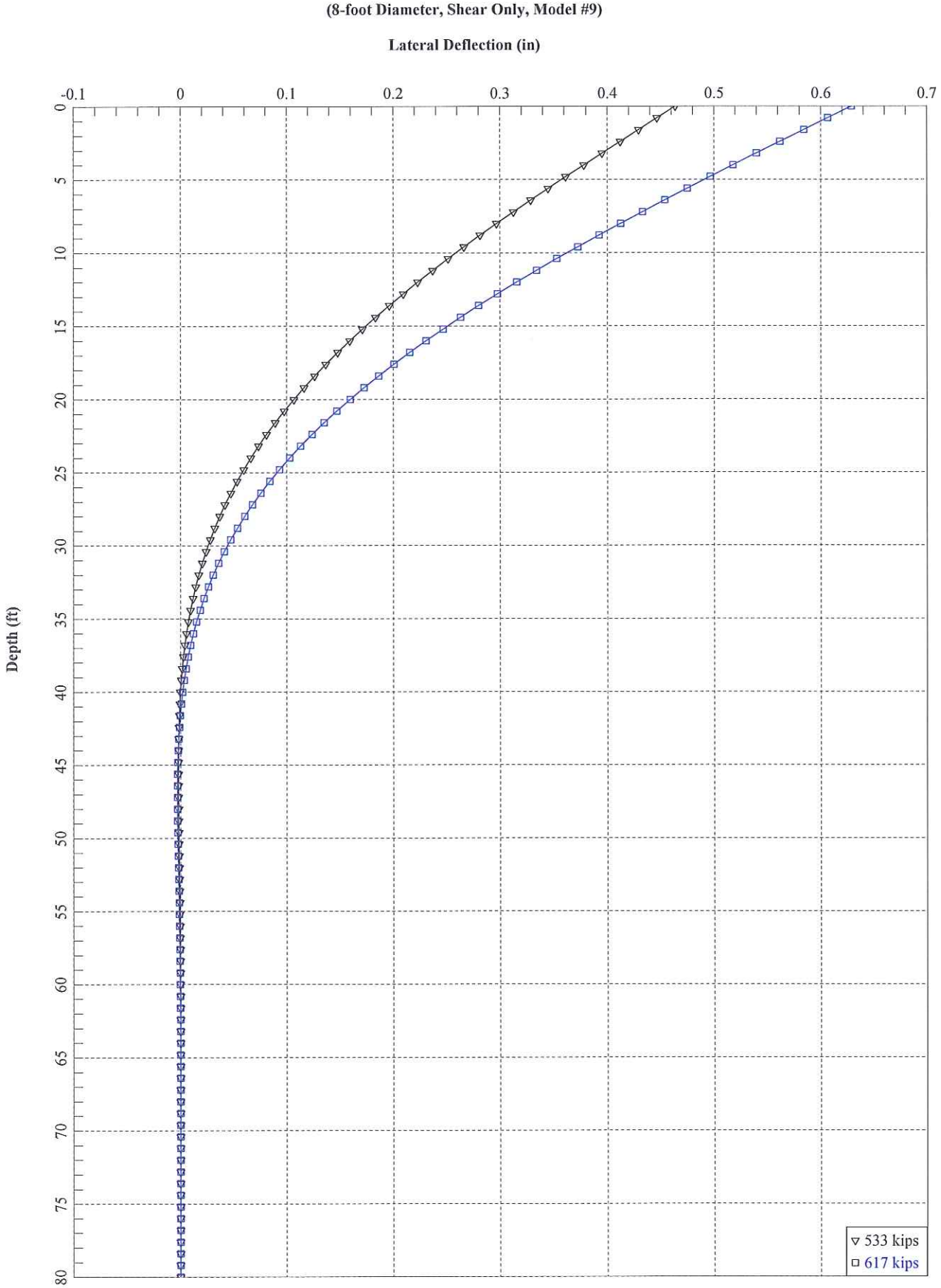
(8-foot Diameter, Shear Only, Model #8)



(8-foot Diameter, Shear Only, Model #8)

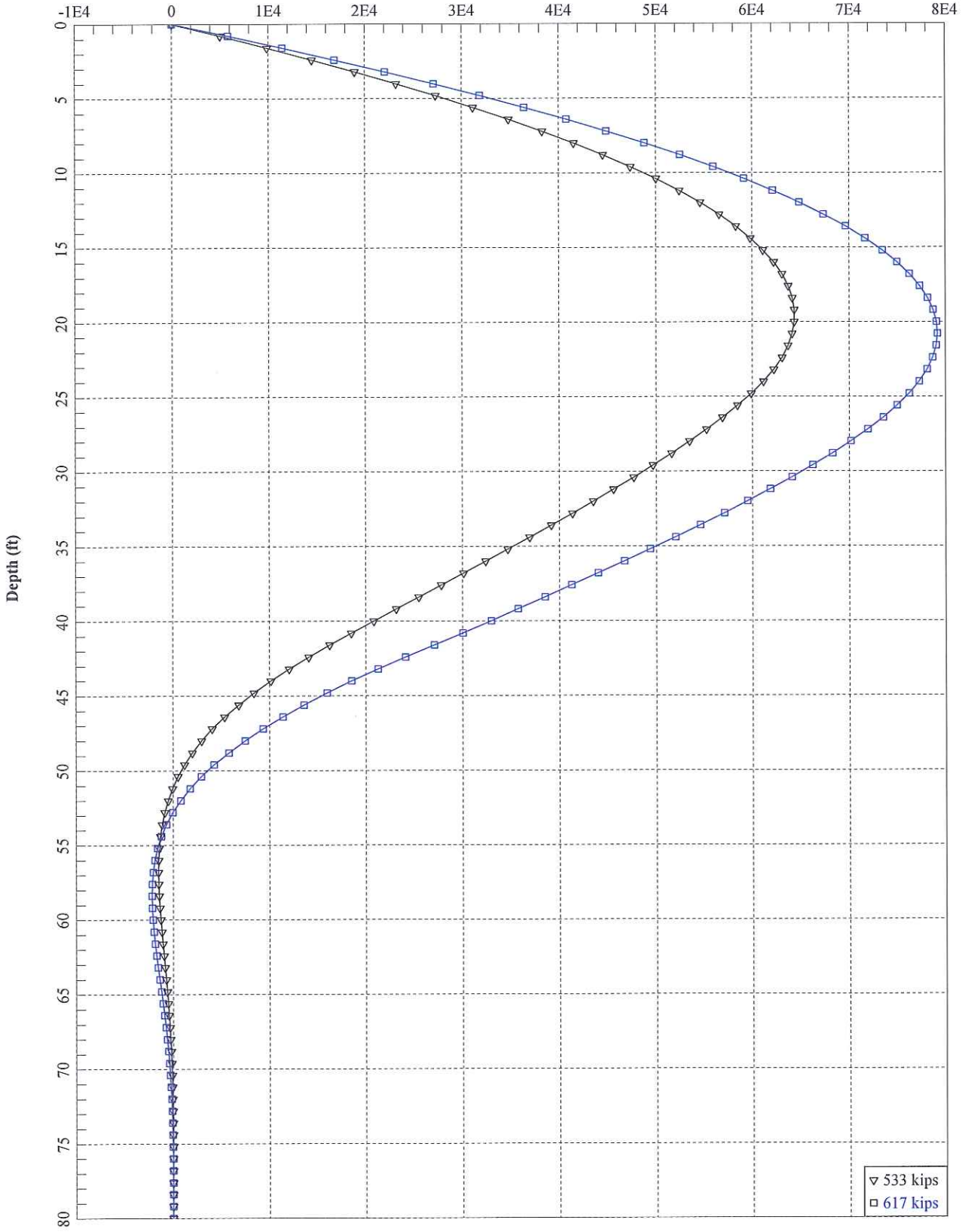
Bending Moment (in-kips)



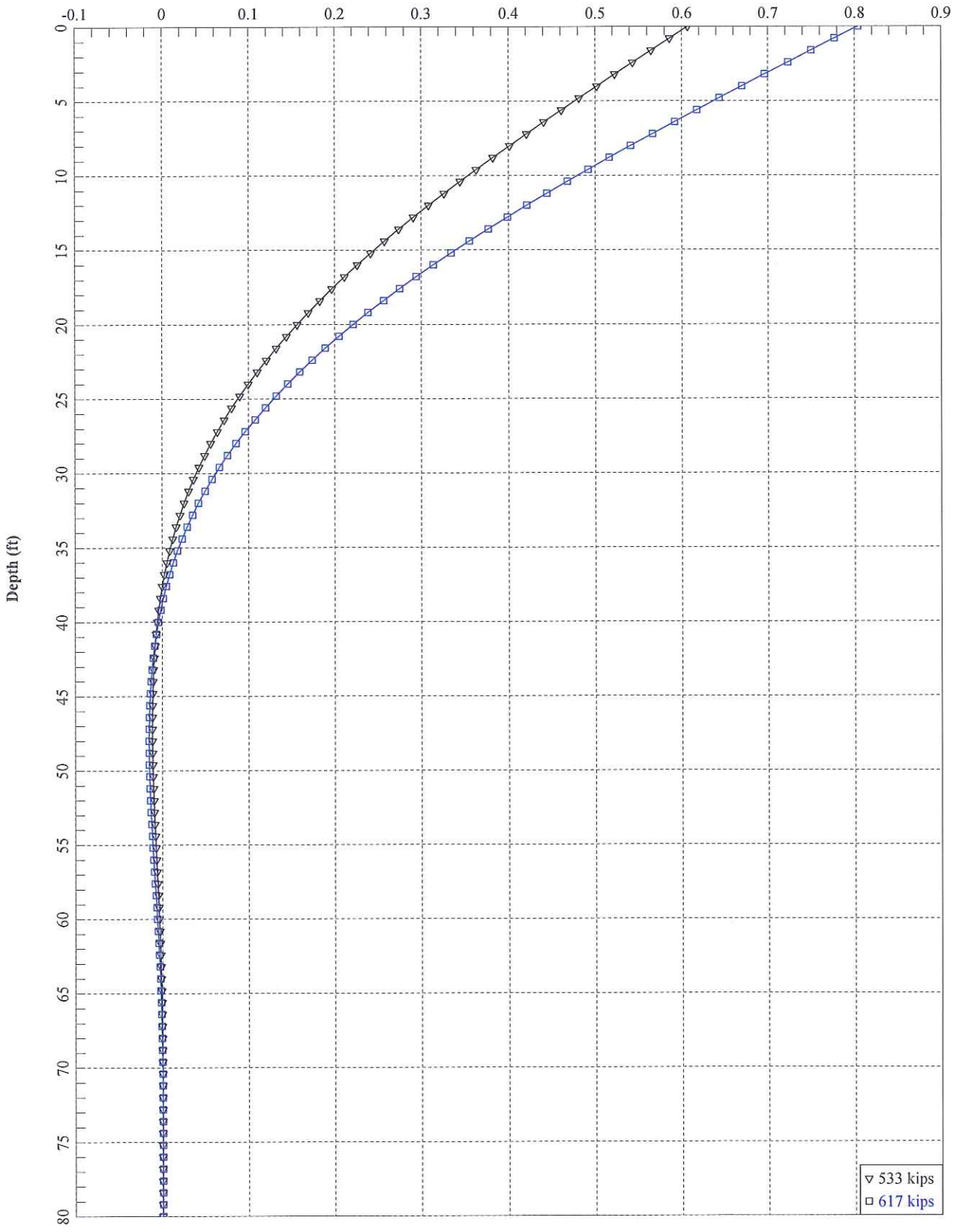

(8-foot Diameter, Shear Only, Model #9)
Lateral Deflection (in)
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
533 kips
617 kips

(8-foot Diameter, Shear Only, Model #9)

Bending Moment (in-kips)

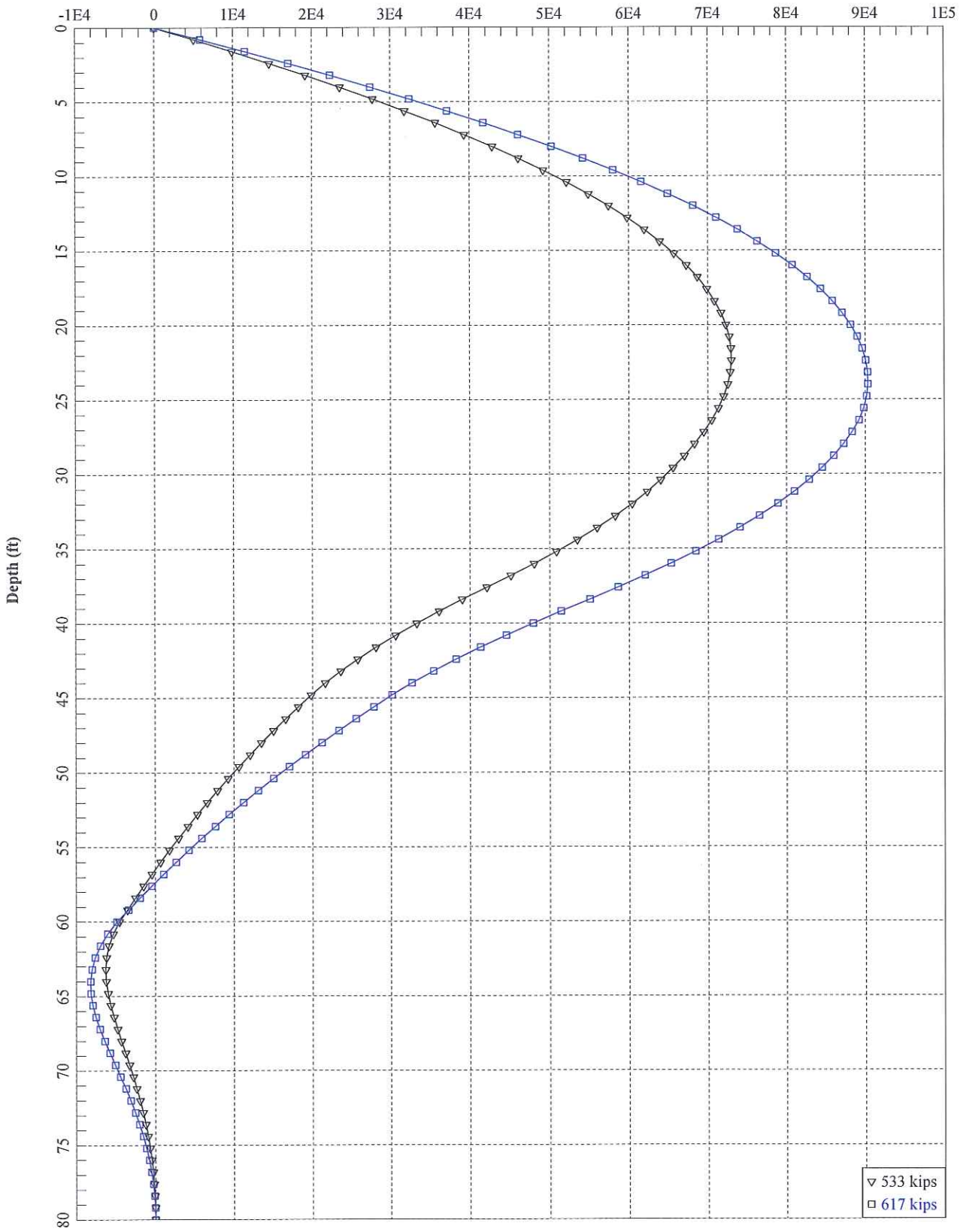


(8-foot Diameter, Shear Only, Model #10)



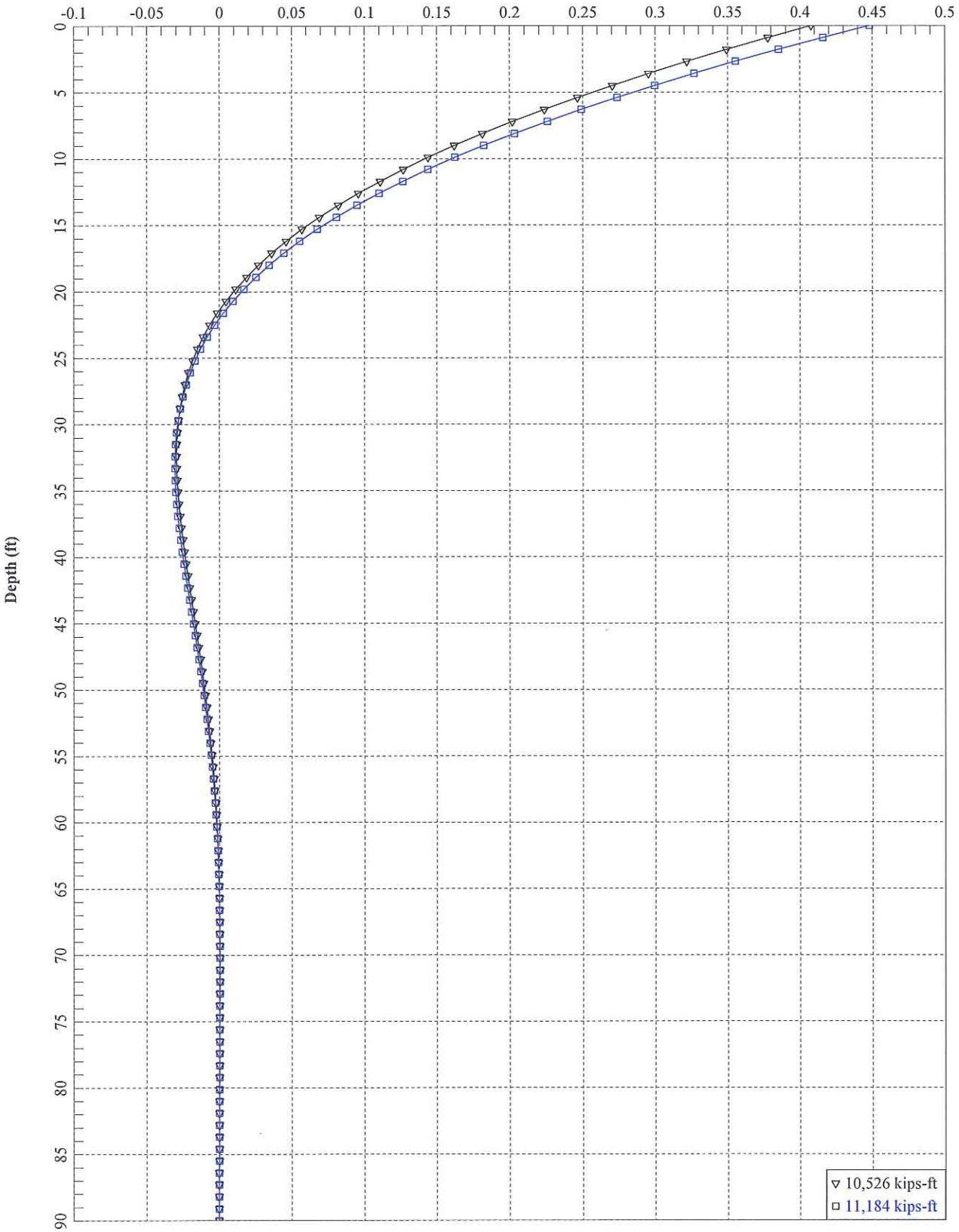
(8-foot Diameter, Shear Only, Model #10)

Bending Moment (in-kips)



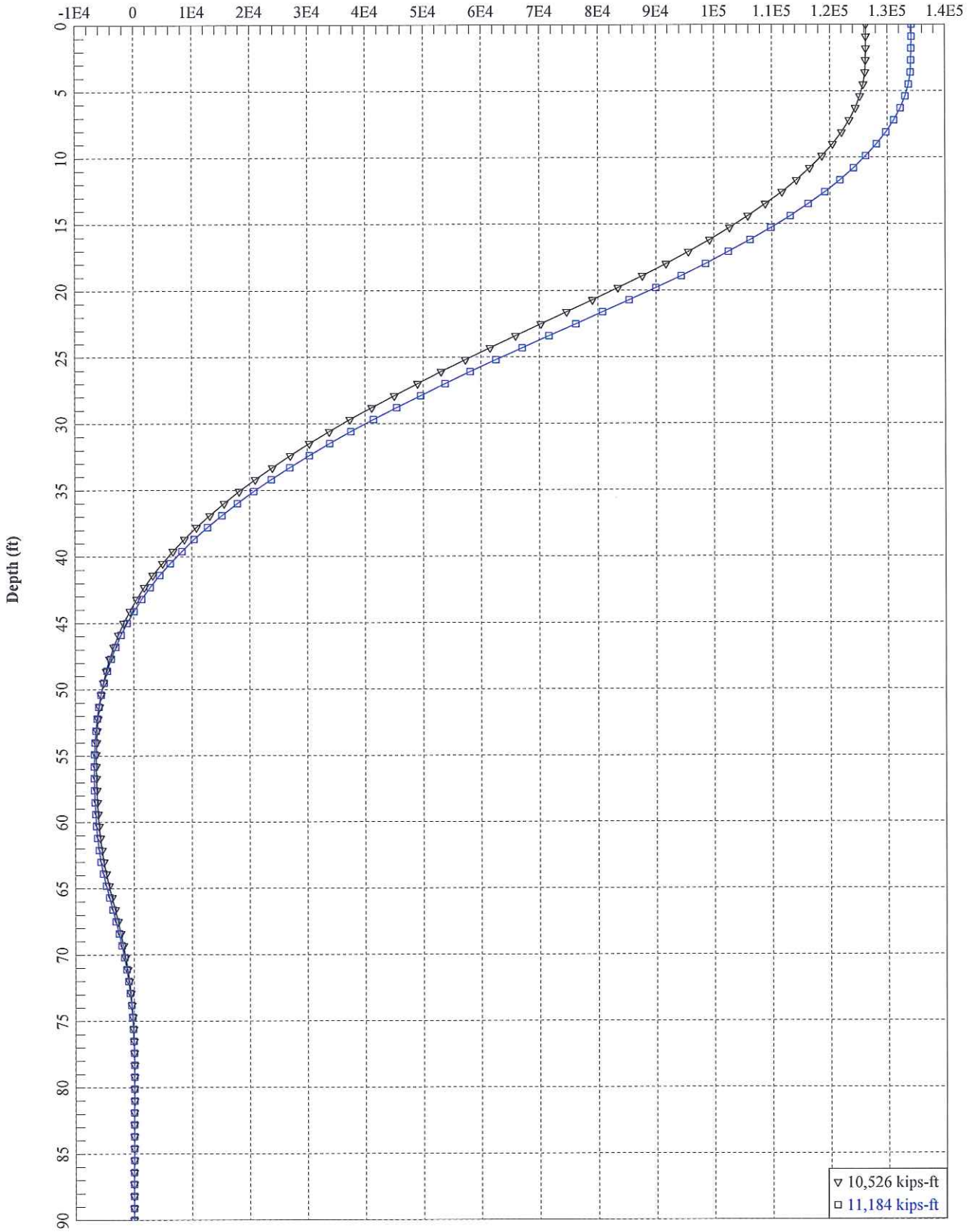
Deflection and Moment Curves for 8 foot Diameter Piles – Moment Applied

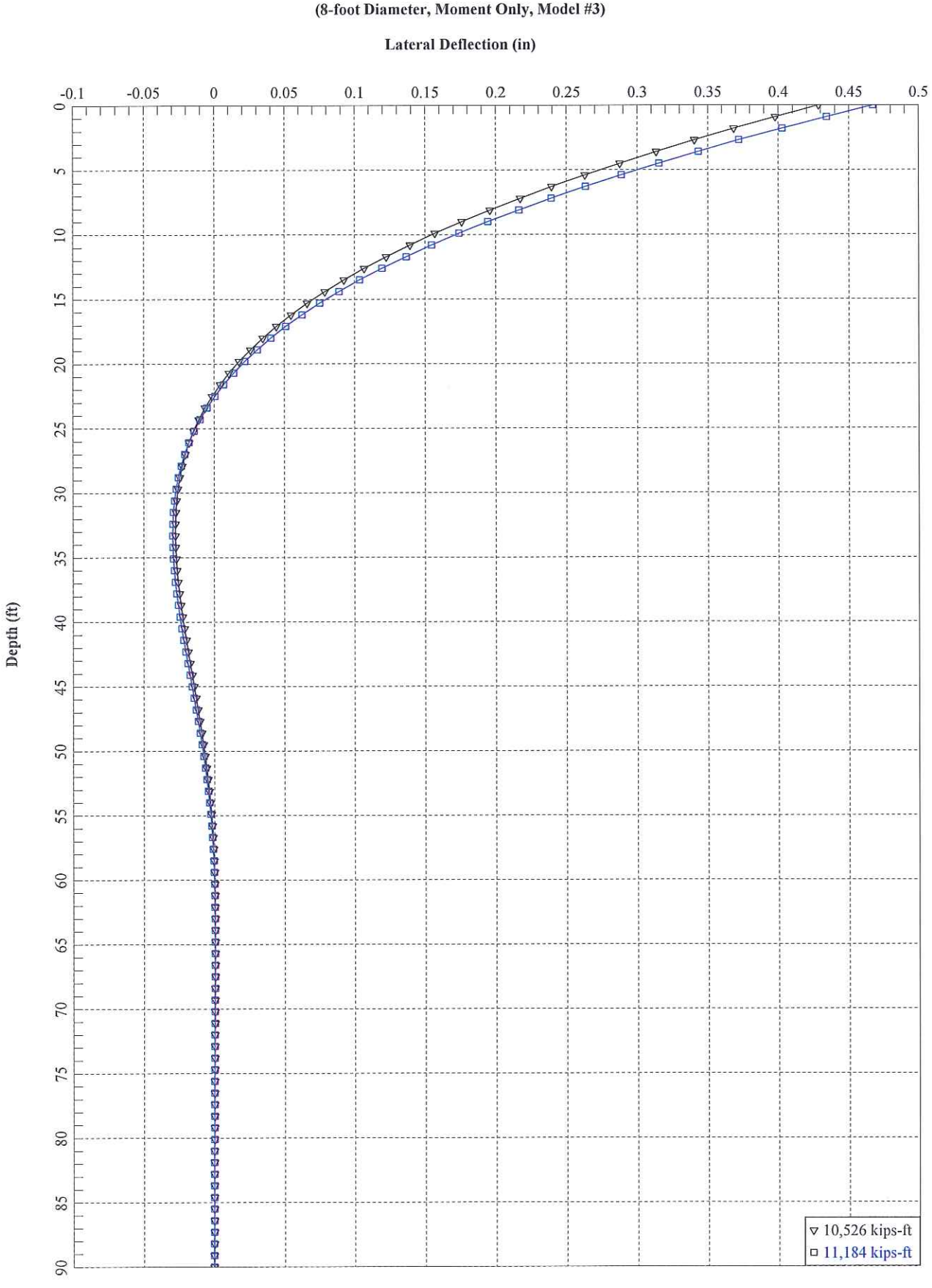
(8-foot Diameter, Moment Only, Model #2)



(8-foot Diameter, Moment Only, Model #2)

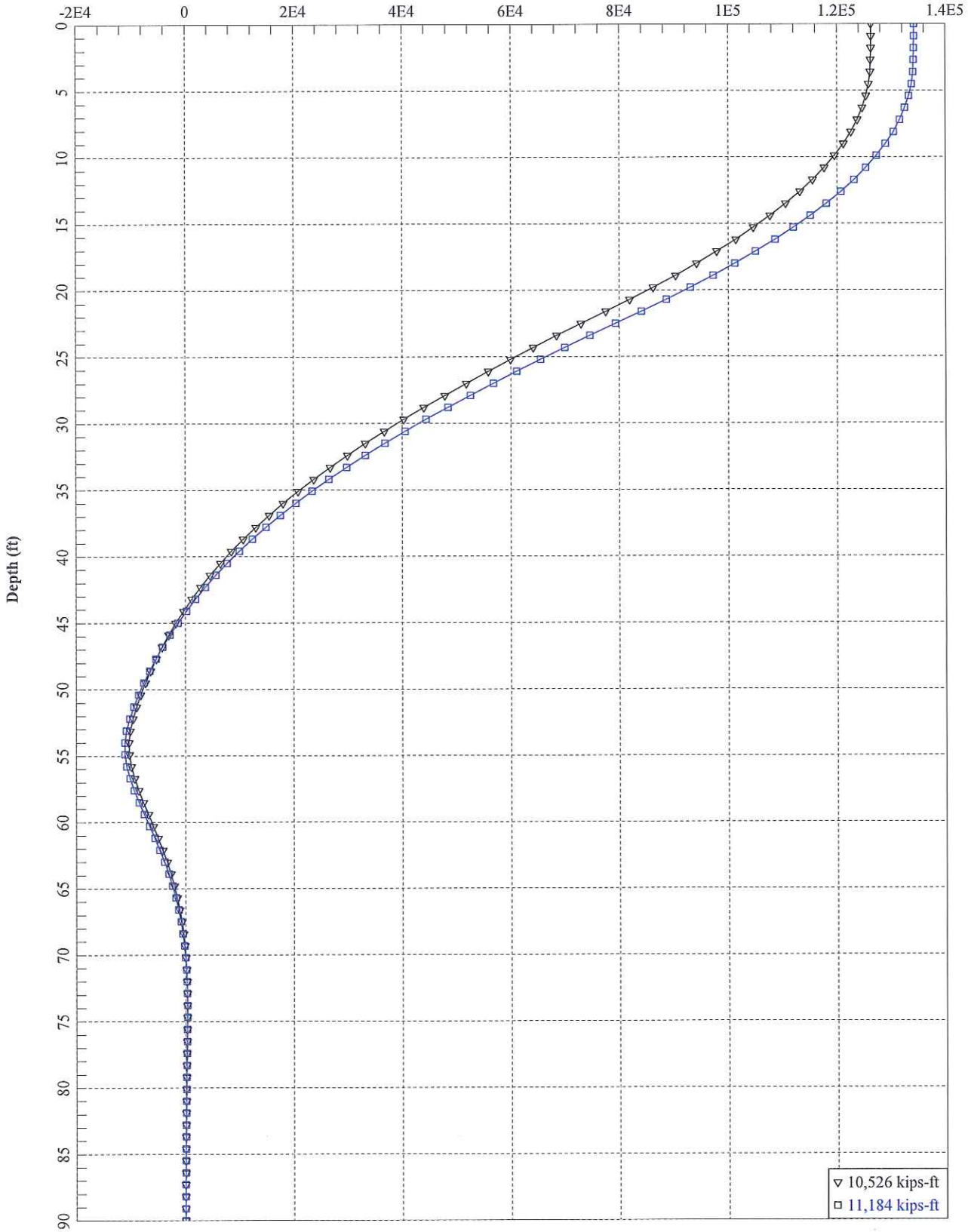
Bending Moment (in-kips)




(8-foot Diameter, Moment Only, Model #3)
Lateral Deflection (in)
-0.1
-0.05
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
0.4
0.45
0.5
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
85
90
10,526 kips-ft
11,184 kips-ft

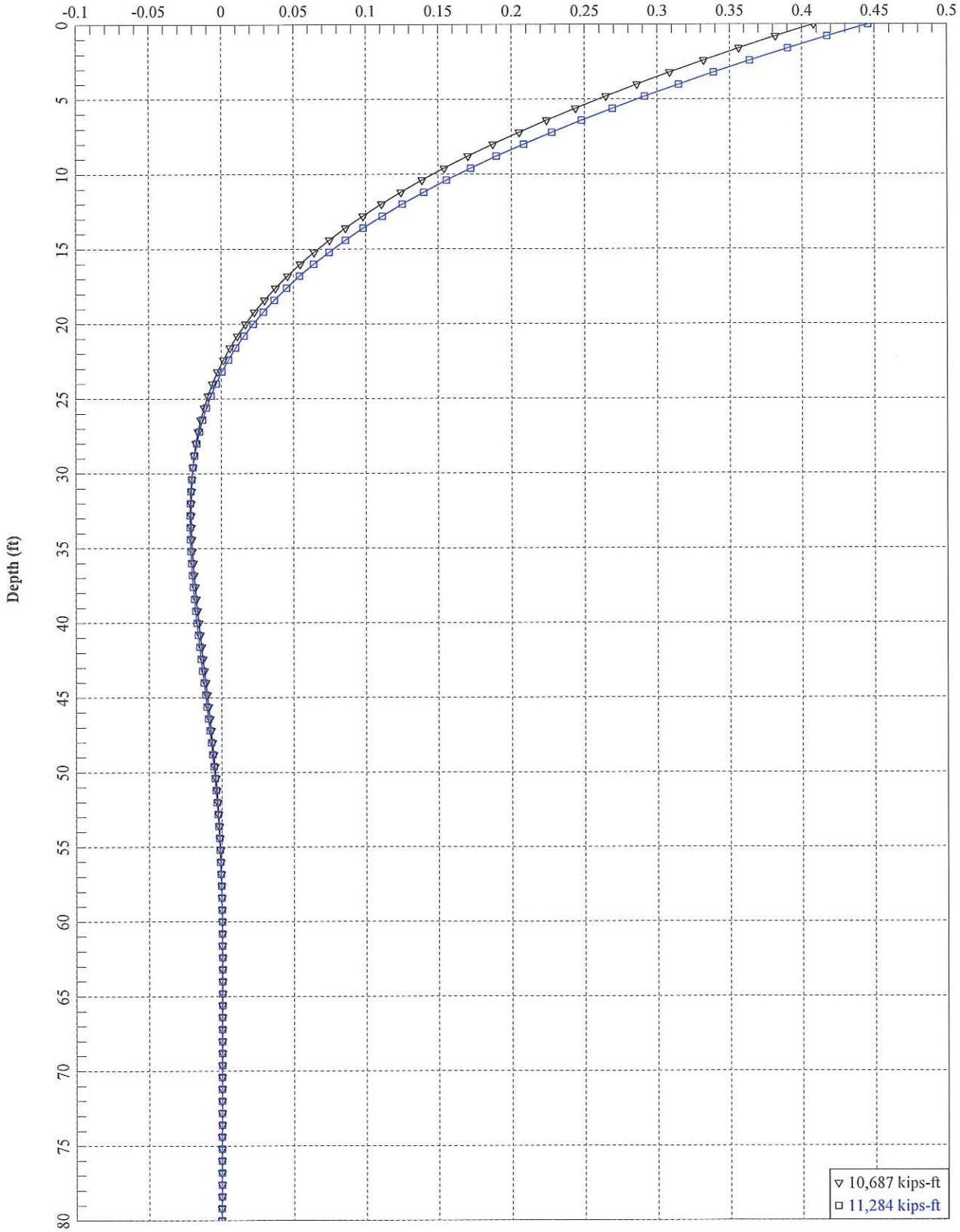
(8-foot Diameter, Moment Only, Model #3)

Bending Moment (in-kips)



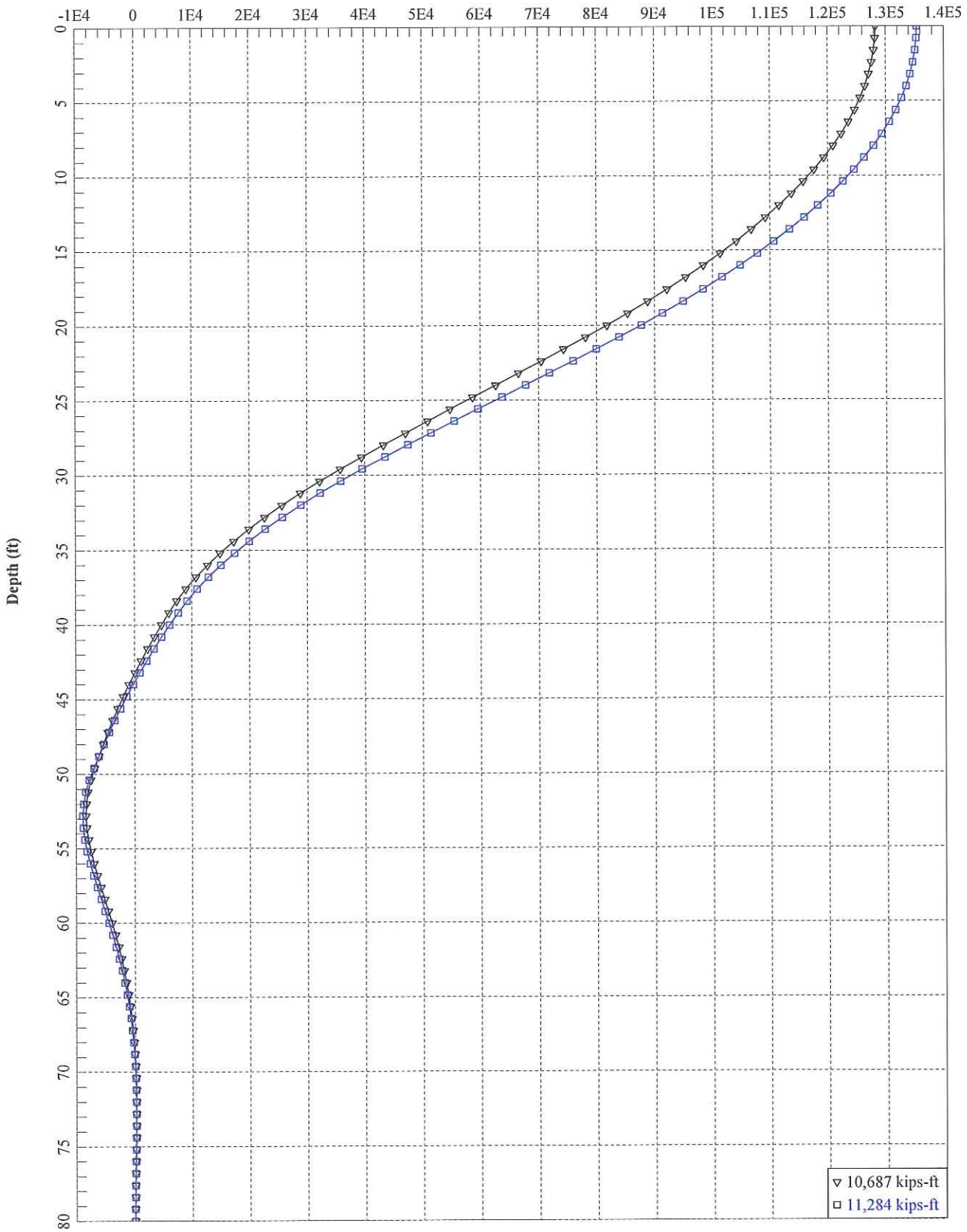
(8-foot Diameter, Moment Only, Model #7)

Lateral Deflection (in)

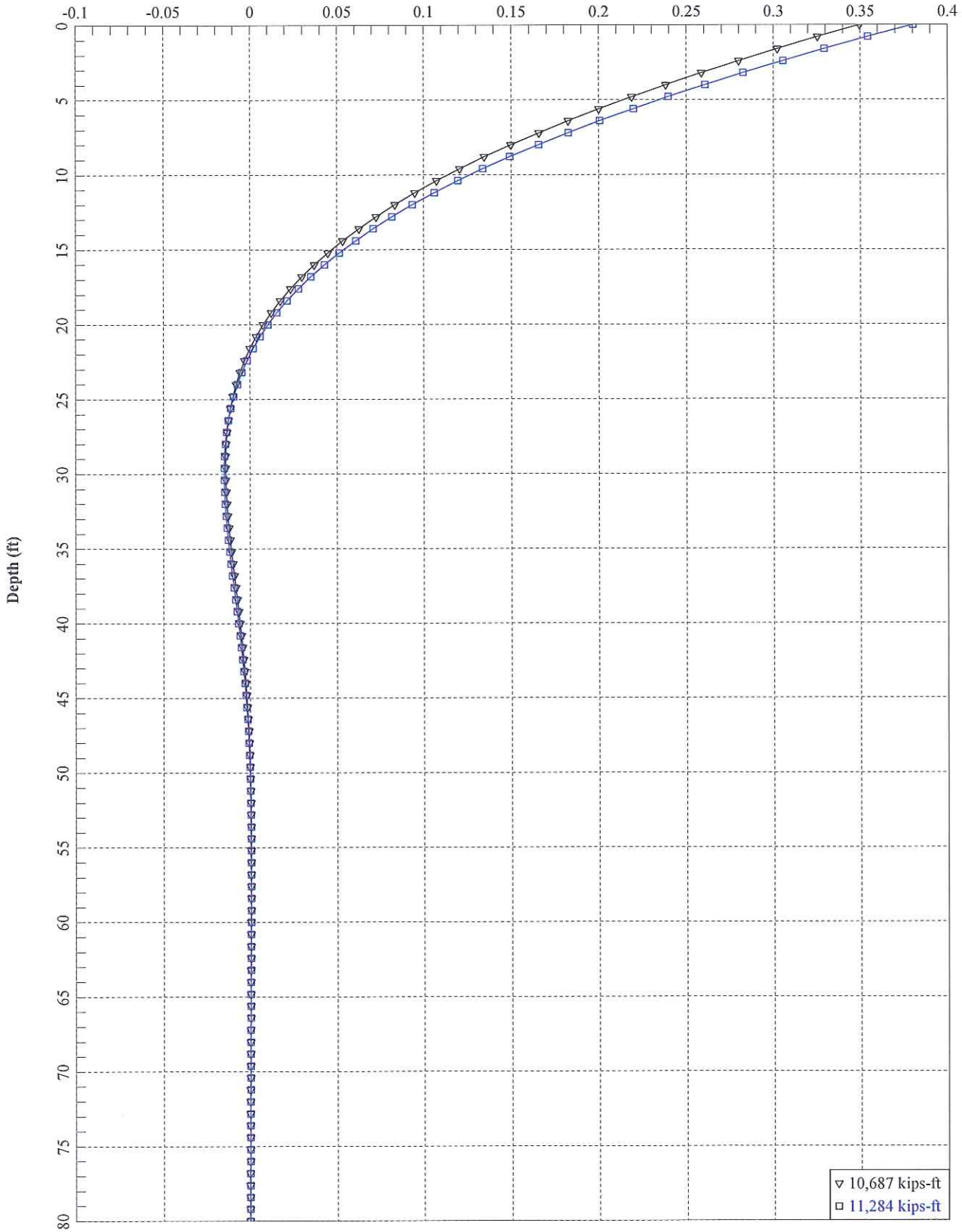


(8-foot Diameter, Moment Only, Model #7)

Bending Moment (in-kips)

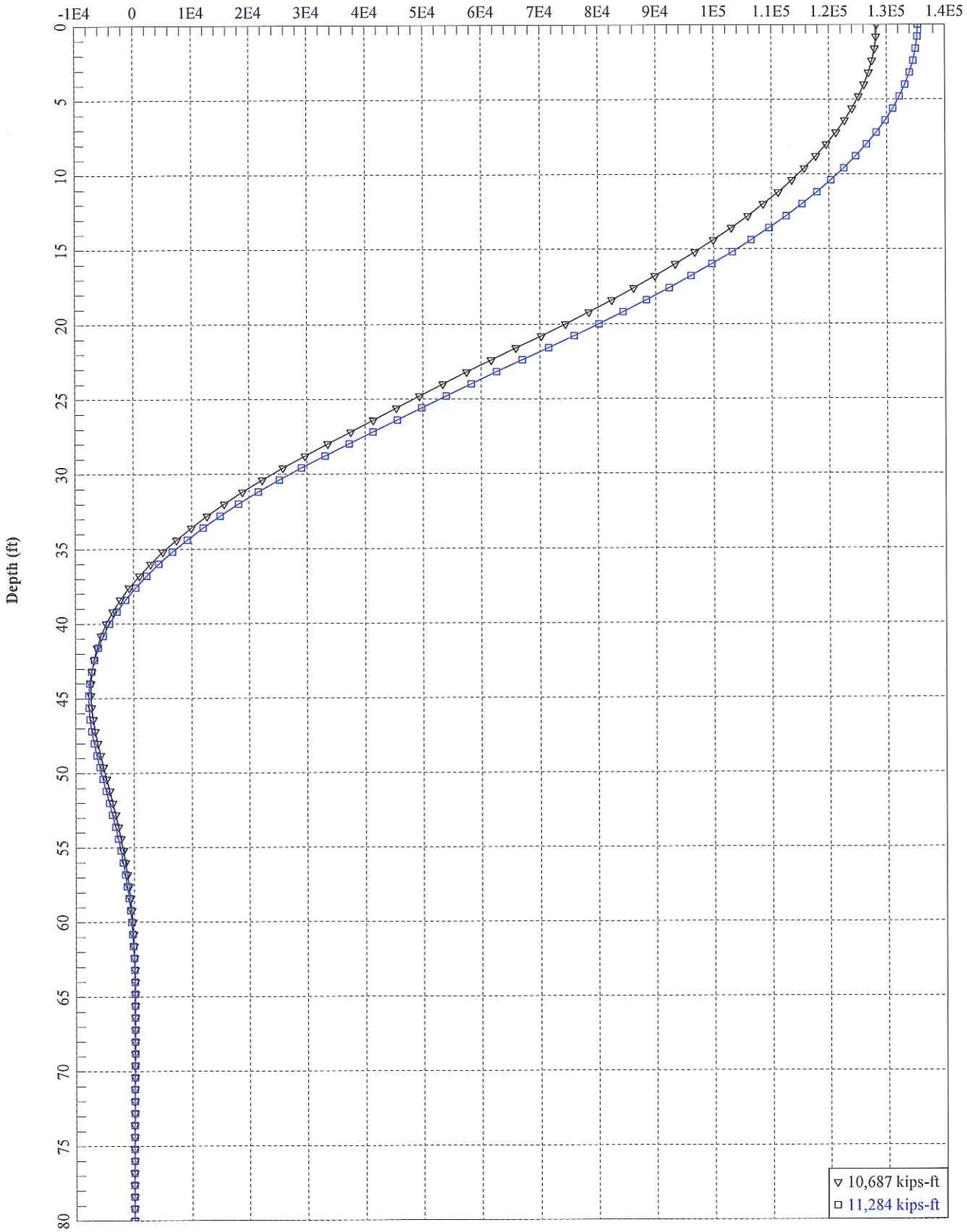


(8-foot Diameter, Moment Only, Model #8)

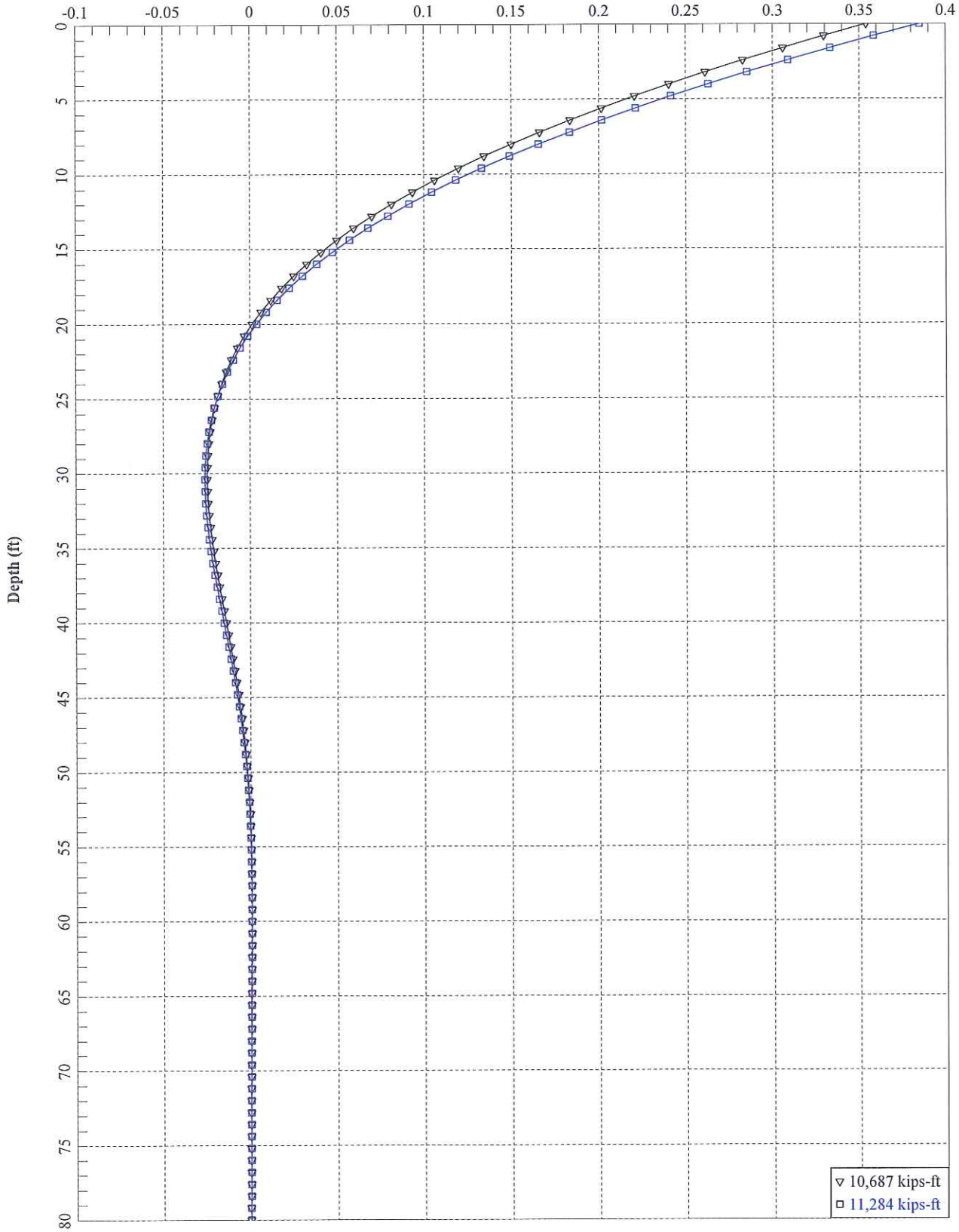


(8-foot Diameter, Moment Only, Model #8)

Bending Moment (in-kips)

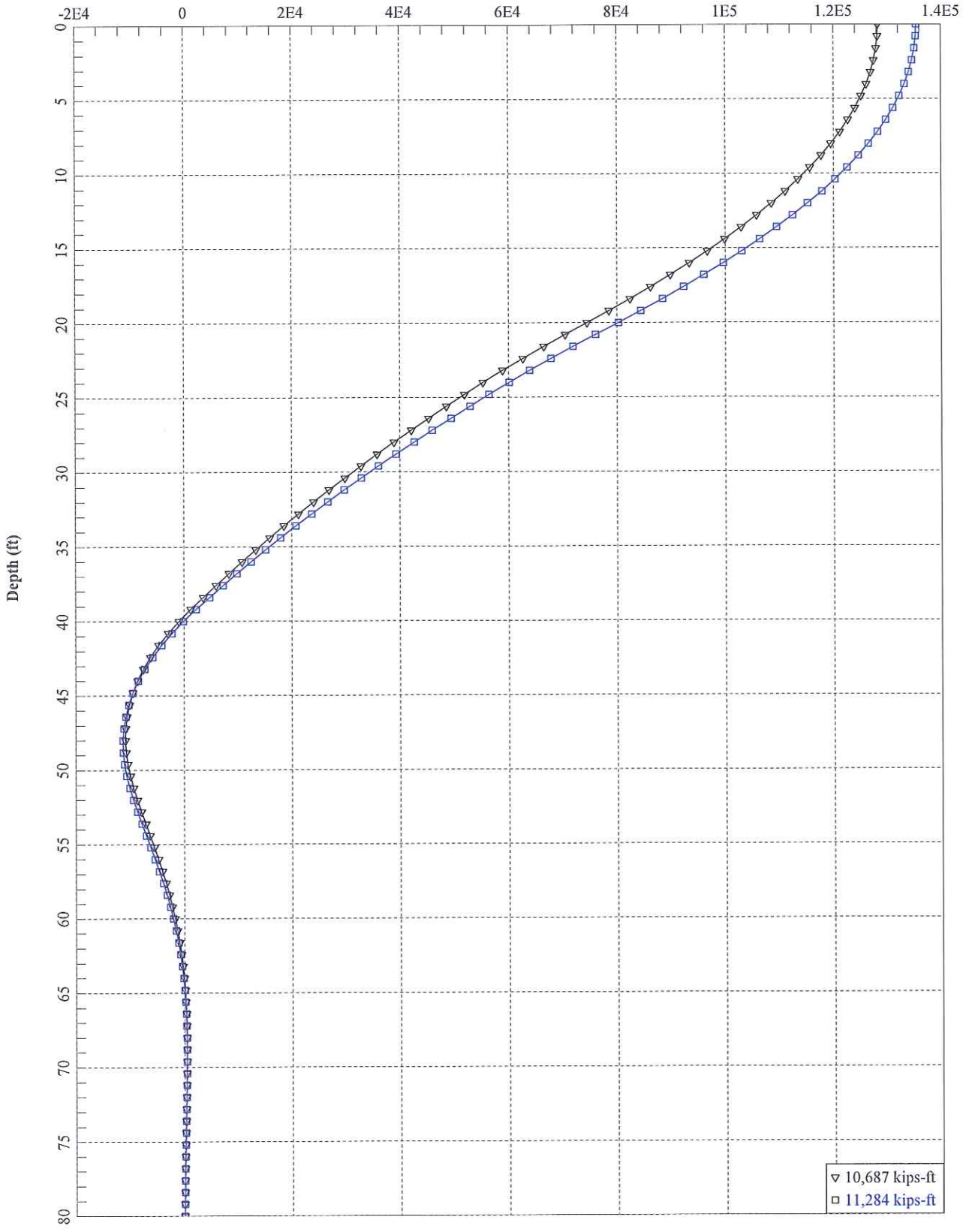


(8-foot Diameter, Moment Only, Model #9)

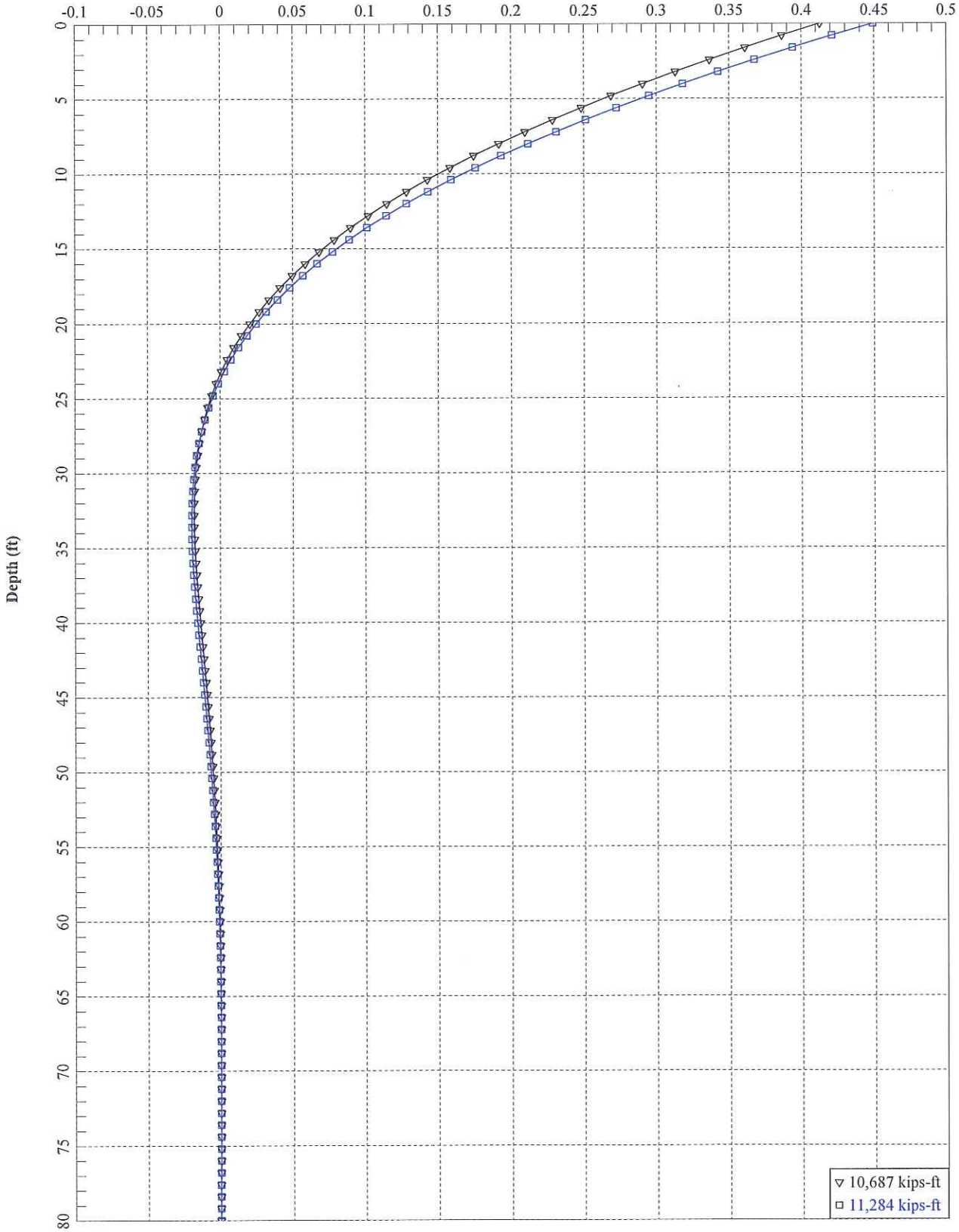


(8-foot Diameter, Moment Only, Model #9)

Bending Moment (in-kips)

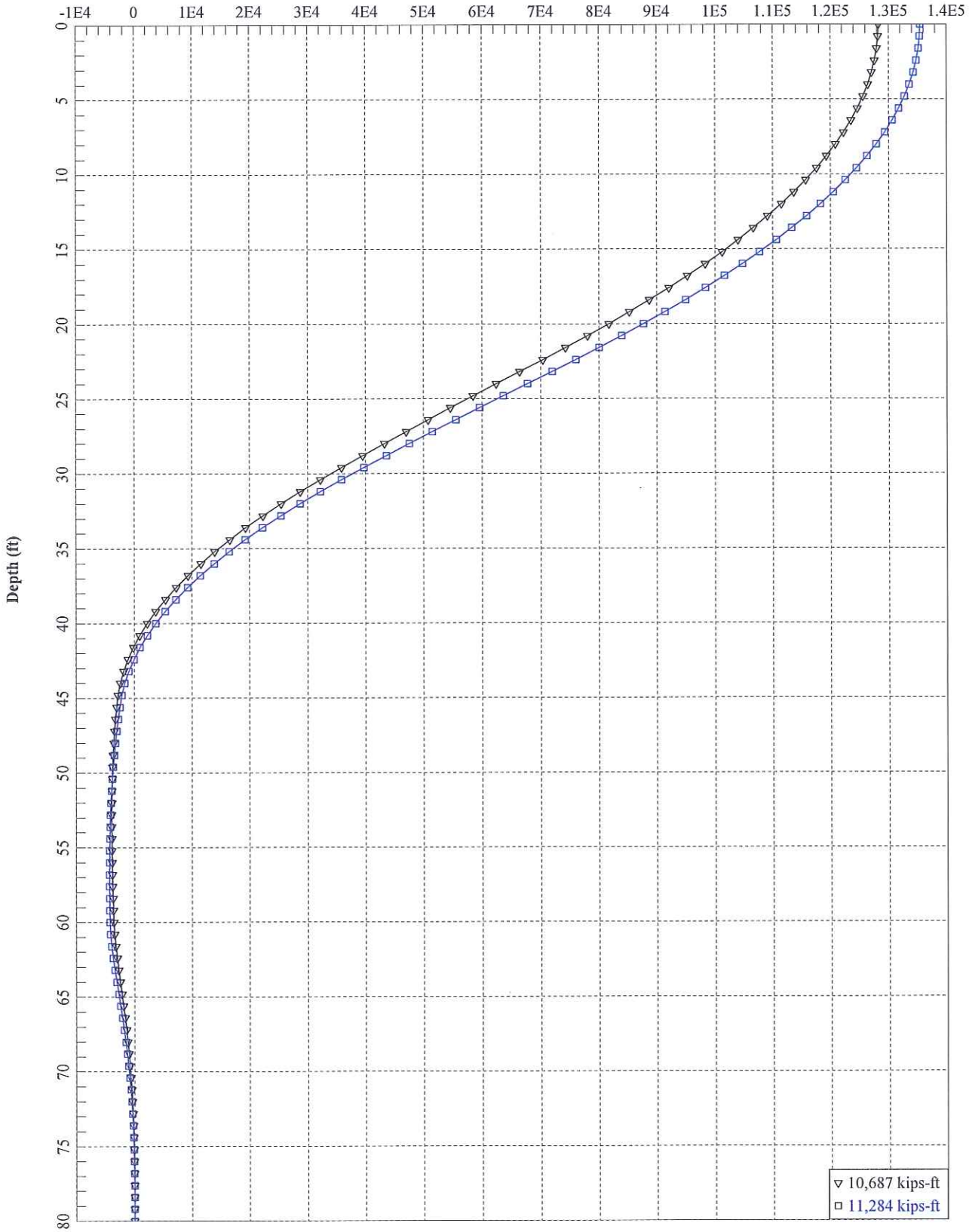


(8-foot Diameter, Moment Only, Model #10)



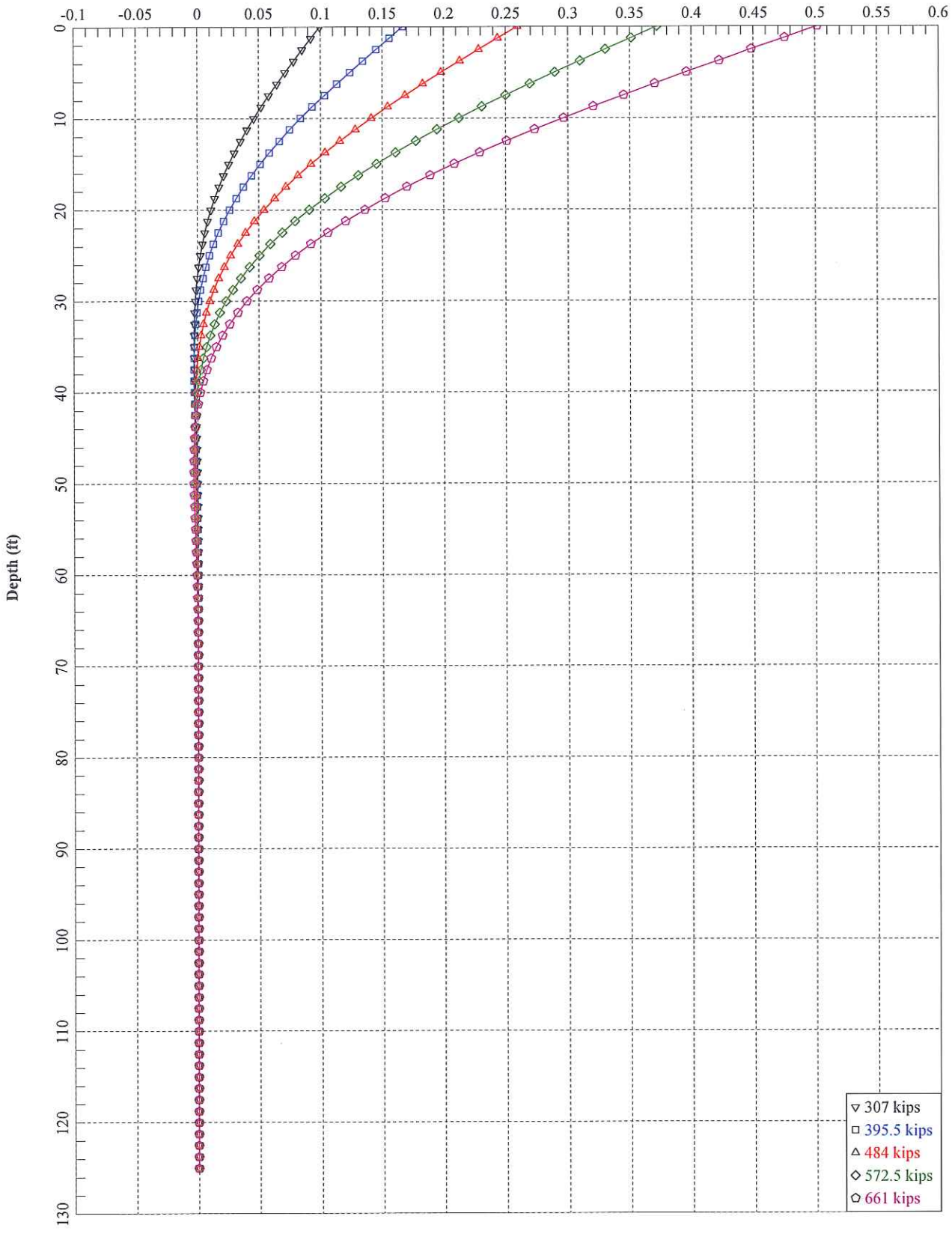
(8-foot Diameter, Moment Only, Model #10)

Bending Moment (in-kips)



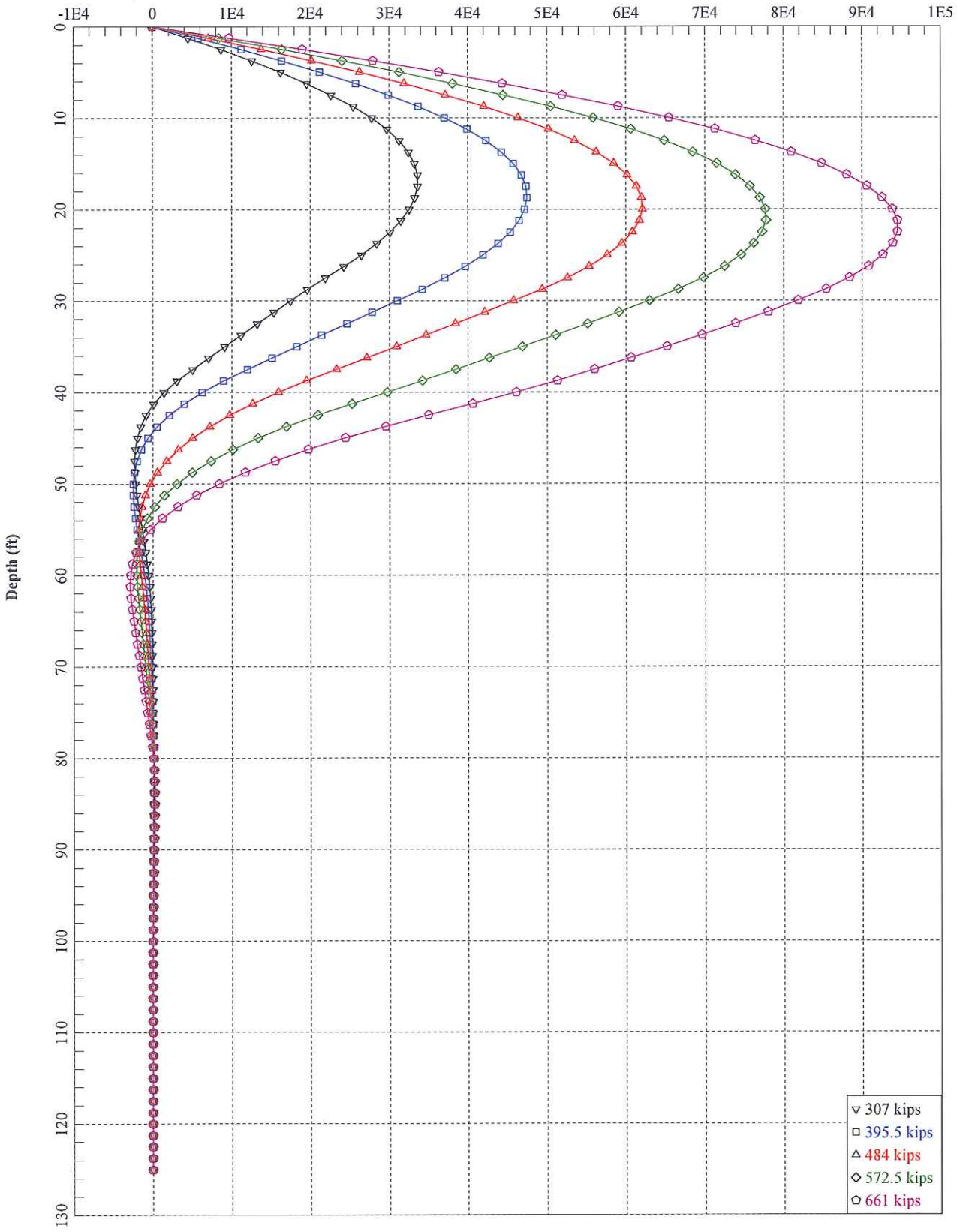
Deflection and Moment Curves for 9 foot Diameter Piles – Shear Force Applied

(9-foot Diameter-Model #4 - Shear Only)

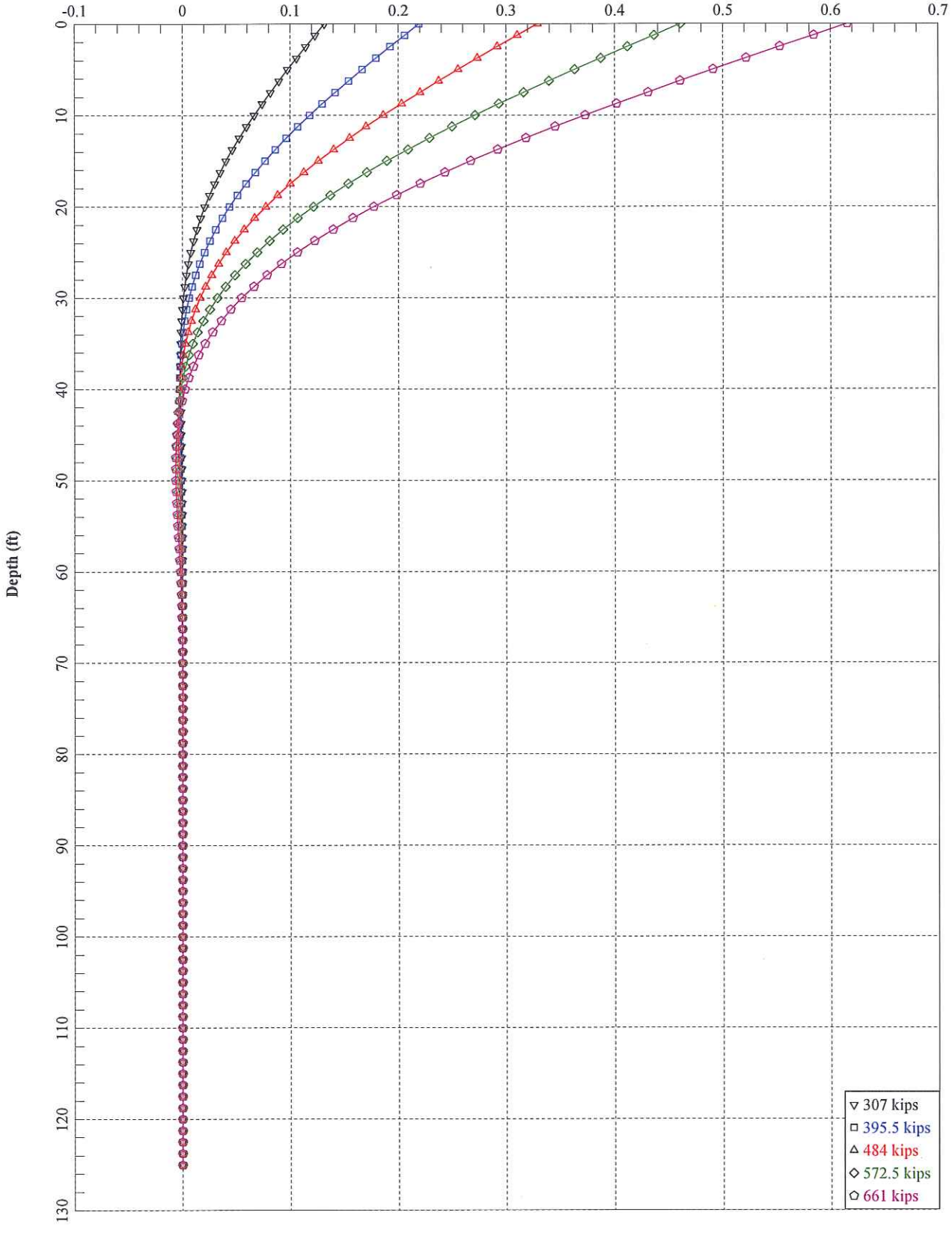


(9-foot Diameter-Model #4 - Shear Only)

Bending Moment (in-kips)

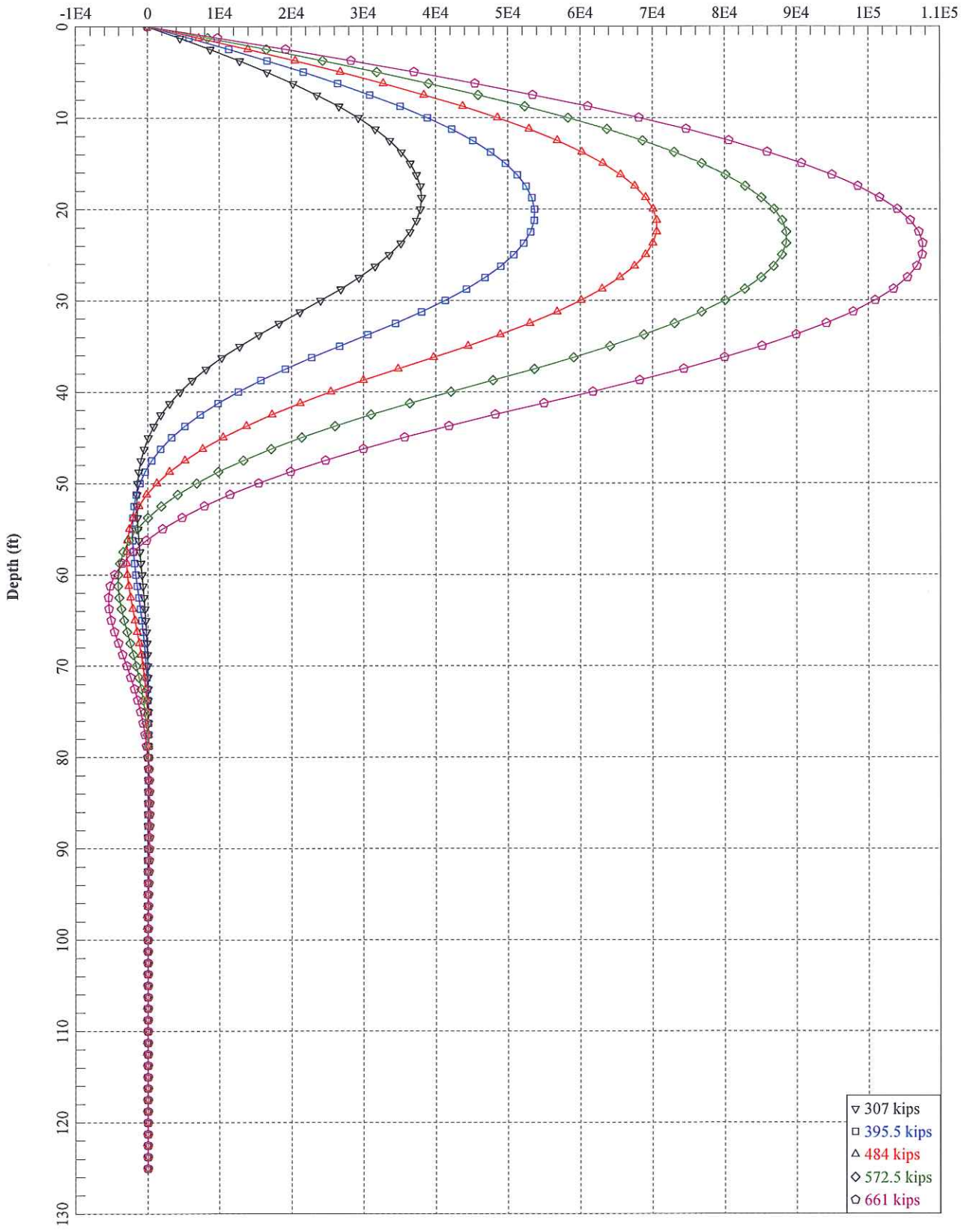


(9-foot Diameter-Model #5 - Shear Only)

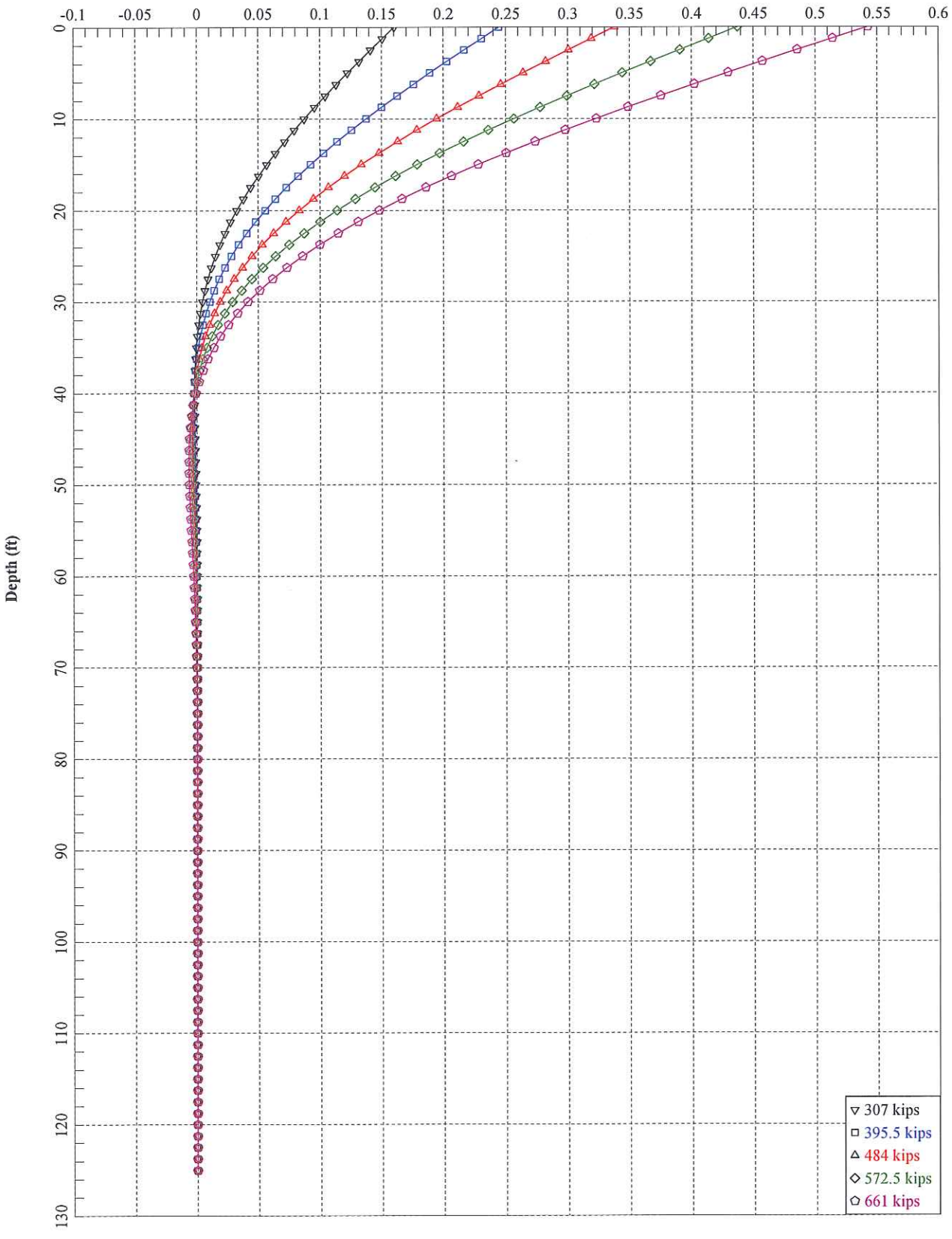


(9-foot Diameter-Model #5- Shear Only)

Bending Moment (in-kips)

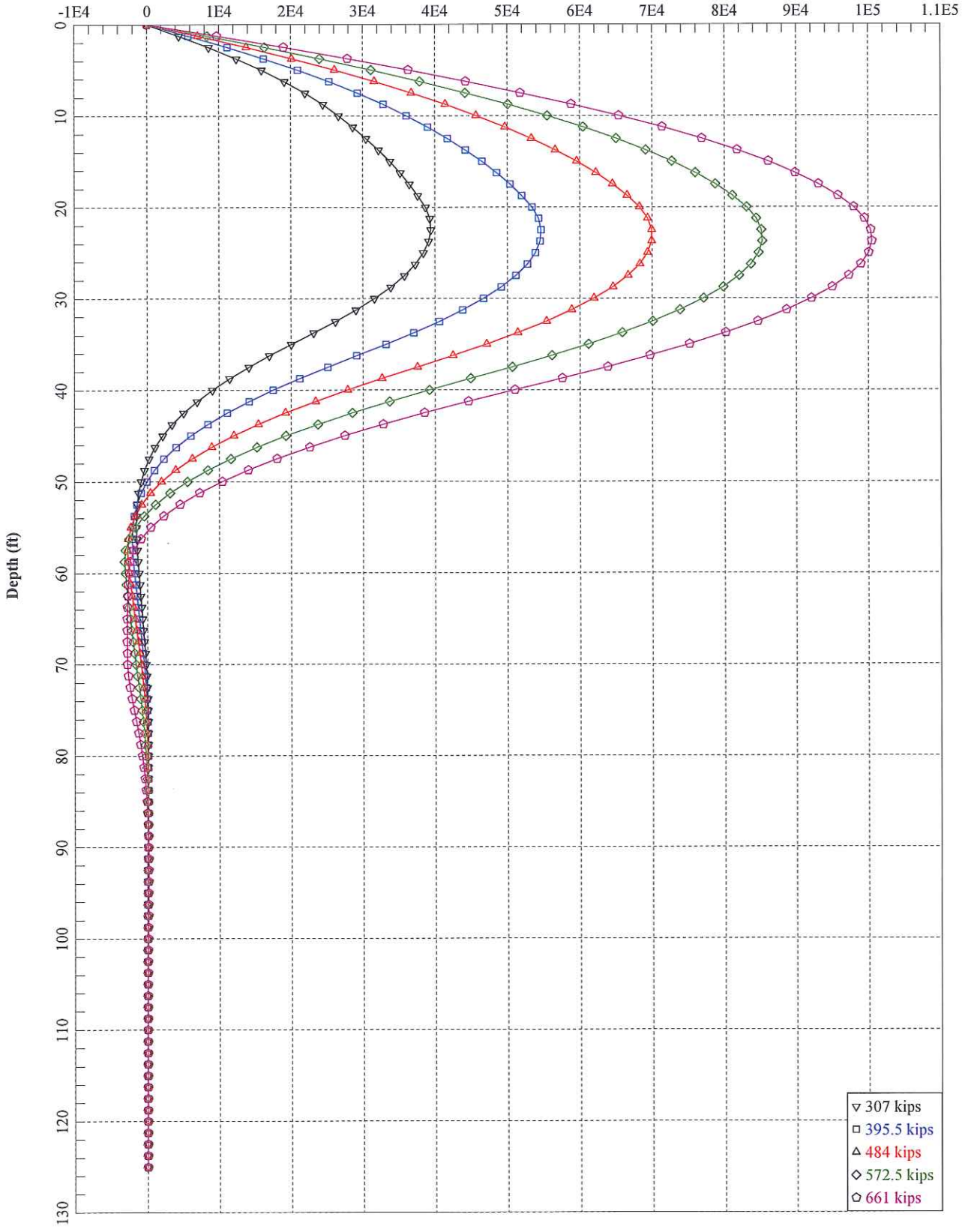


(9-foot Diameter-Model #6 - Shear Only)



(9-foot Diameter-Model #6 - Shear Only)

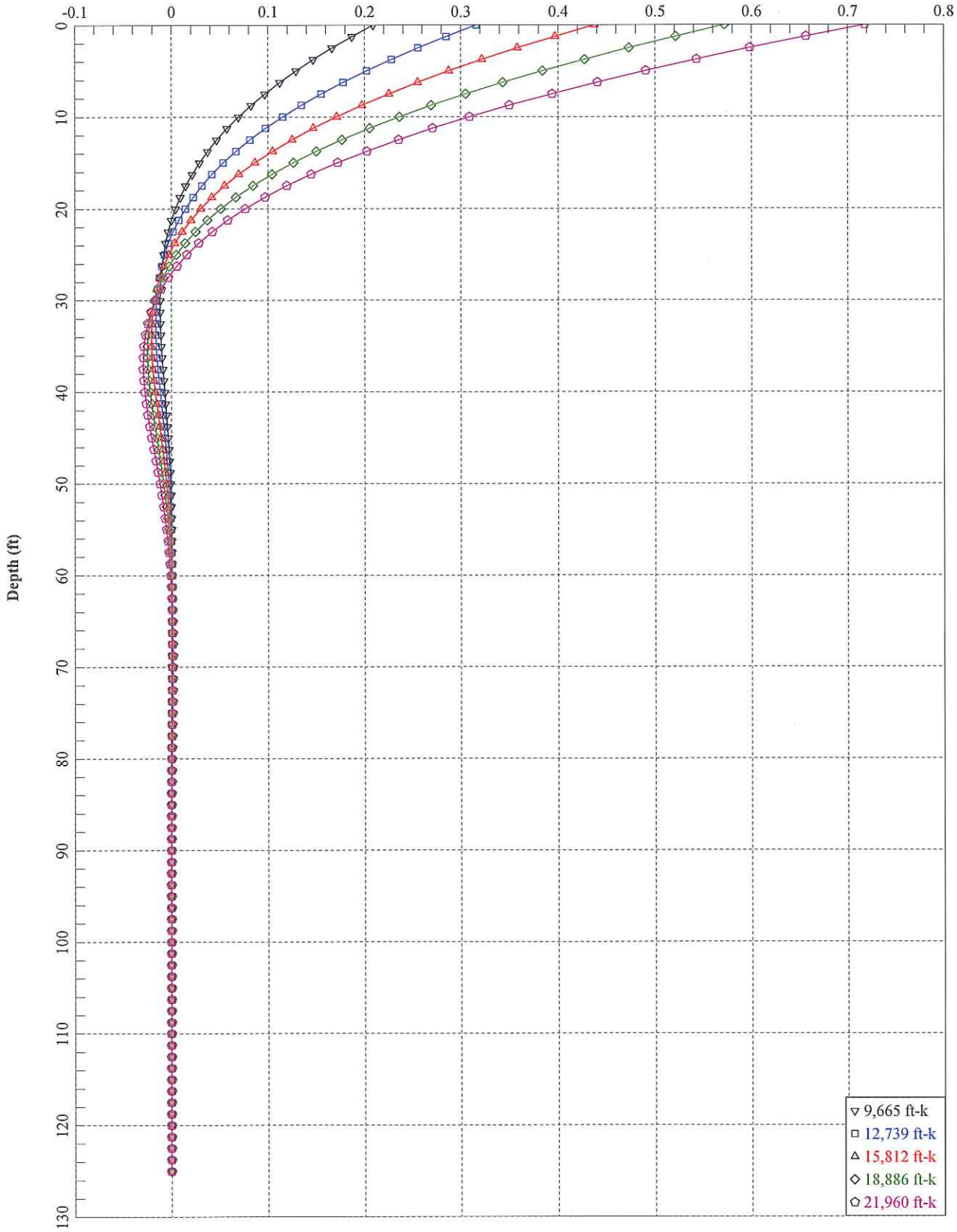
Bending Moment (in-kips)



Deflection and Moment Curves for 9 foot Diameter Piles – Moment Applied

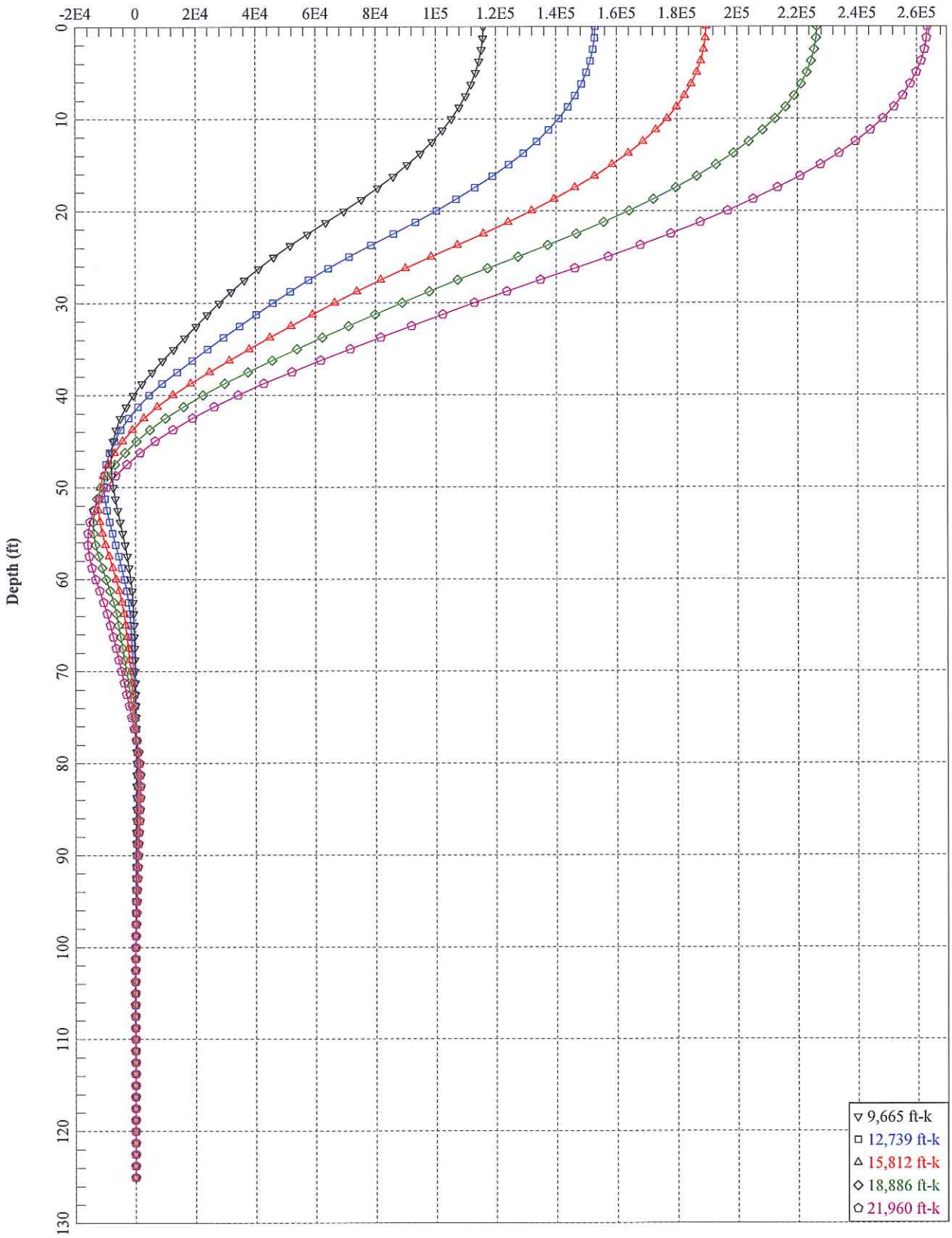
(9-foot Diameter- Model #4 - Moment Only)

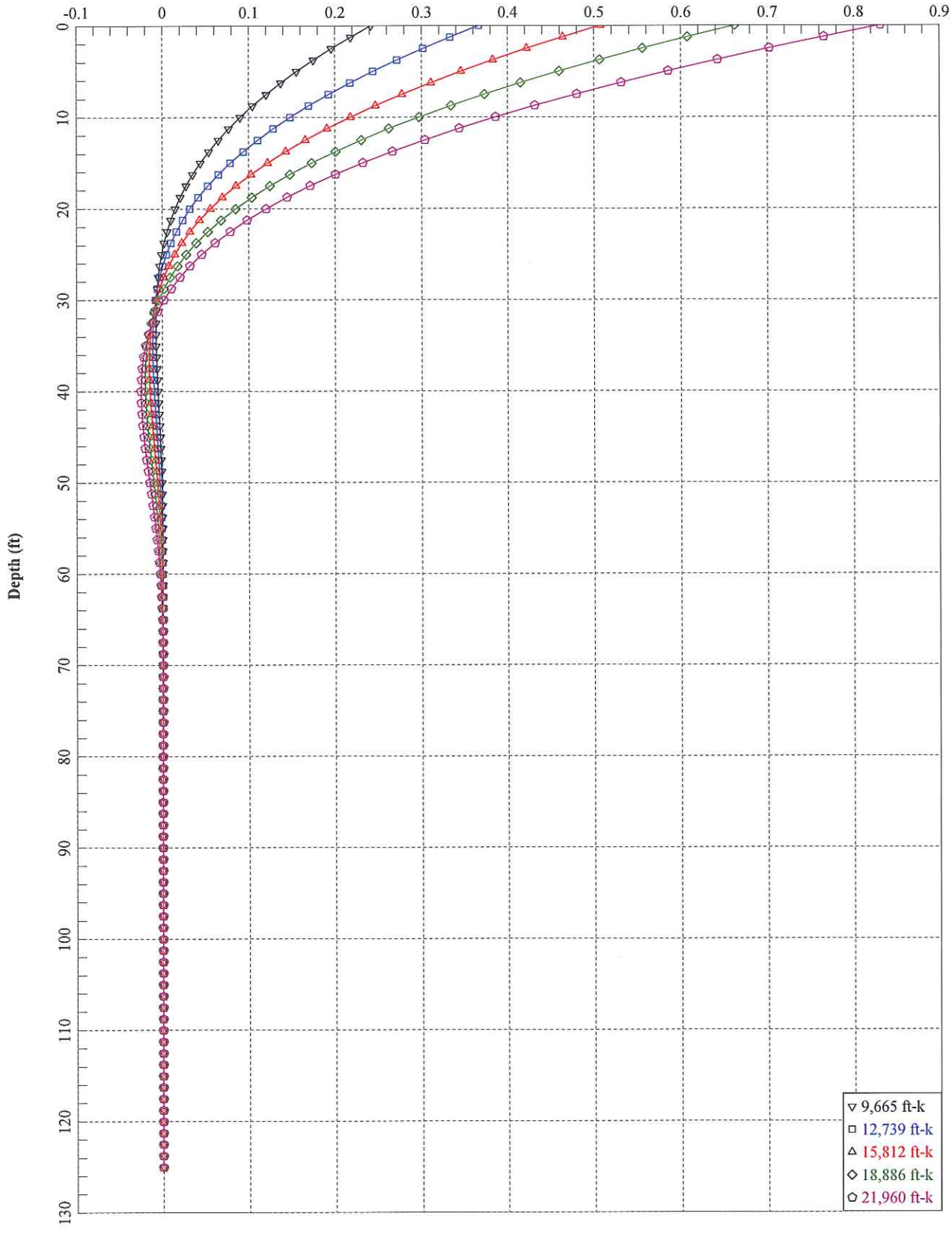
Lateral Deflection (in)



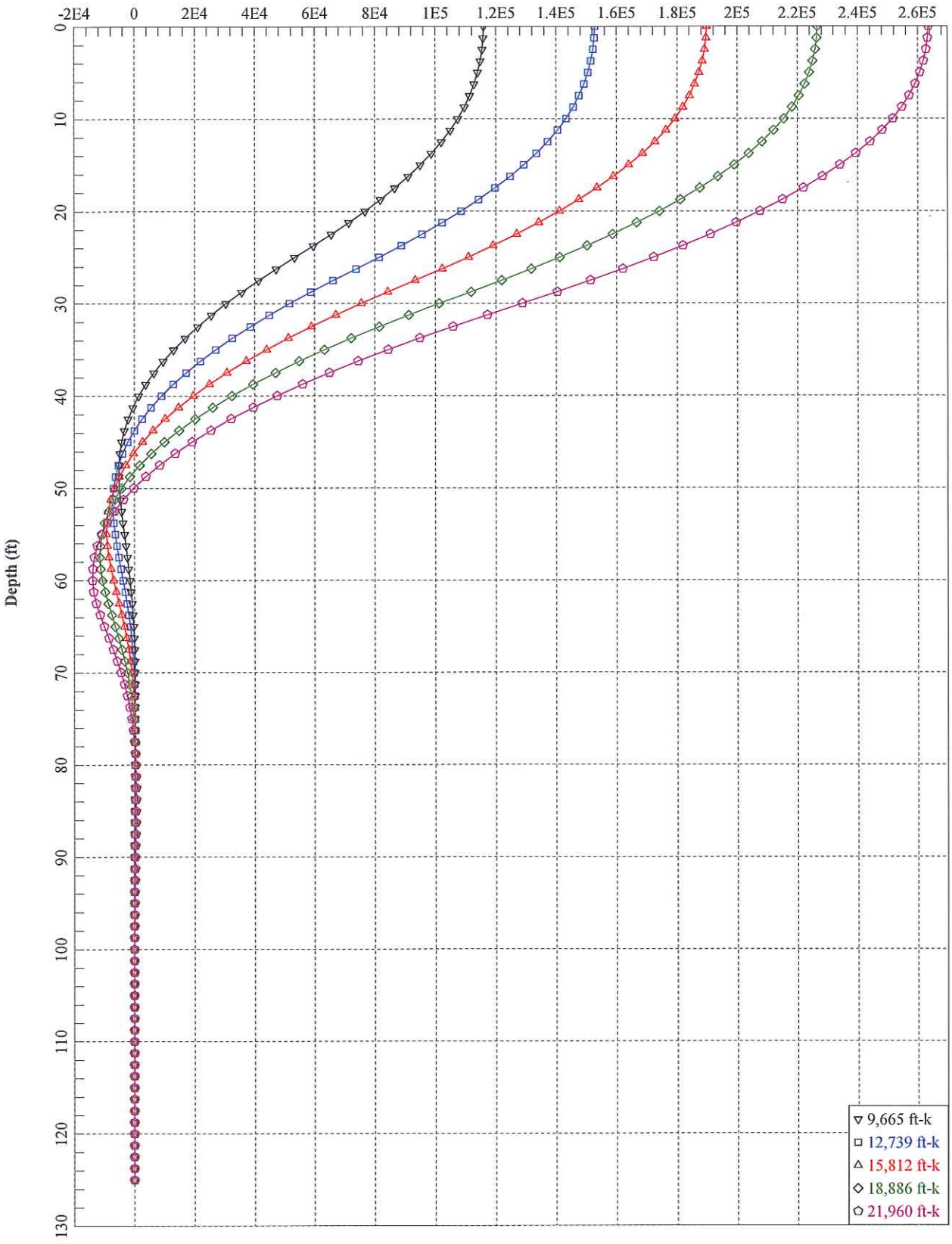
(9-foot Diameter- Model #4 - Moment Only)

Bending Moment (in-kips)

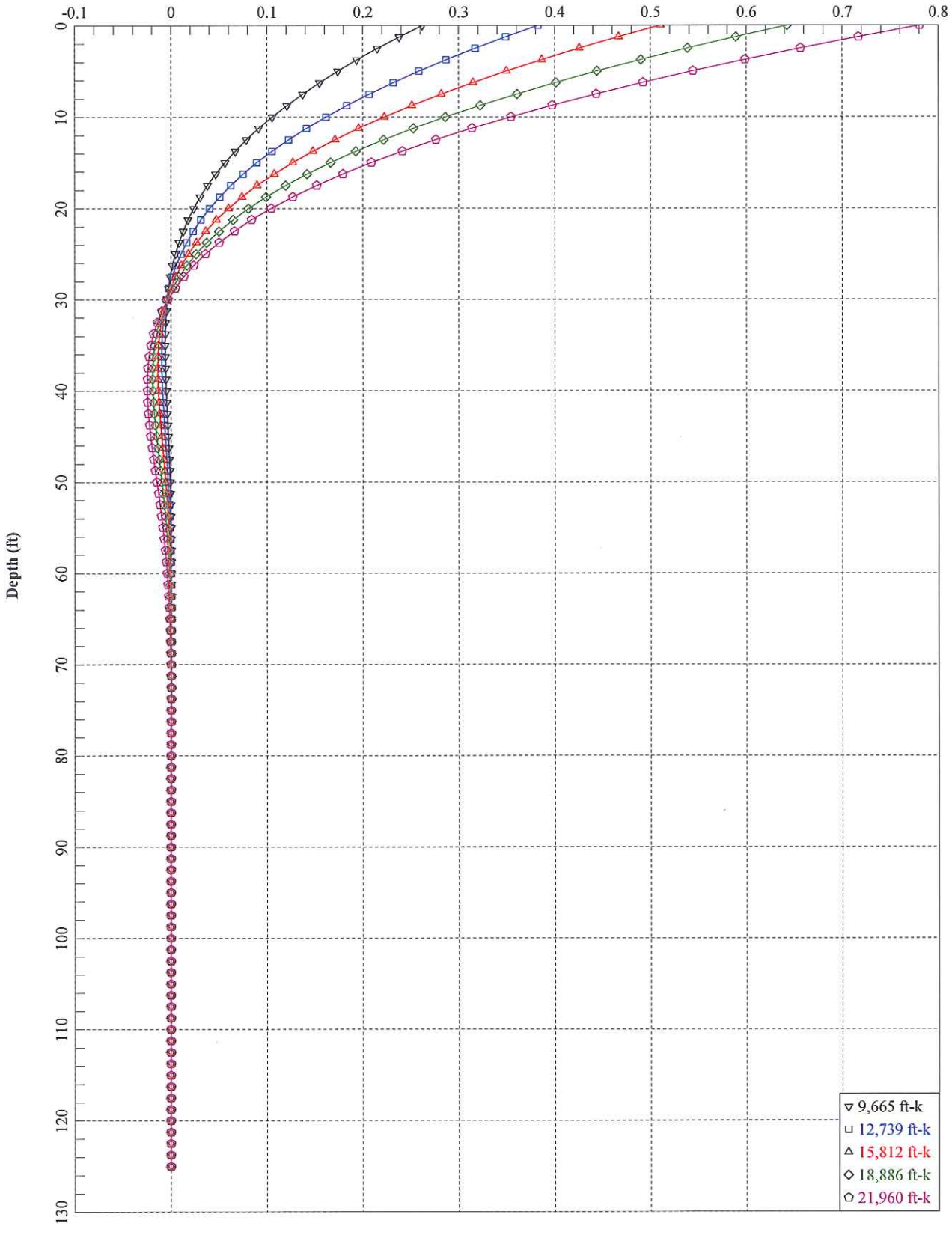



(9-foot Diameter-Model #5 - Moment Only)
Lateral Deflection (in)
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
120
130
9,665 ft-k
12,739 ft-k
15,812 ft-k
18,886 ft-k
21,960 ft-k

(9-foot Diameter-Model #5- Moment Only)

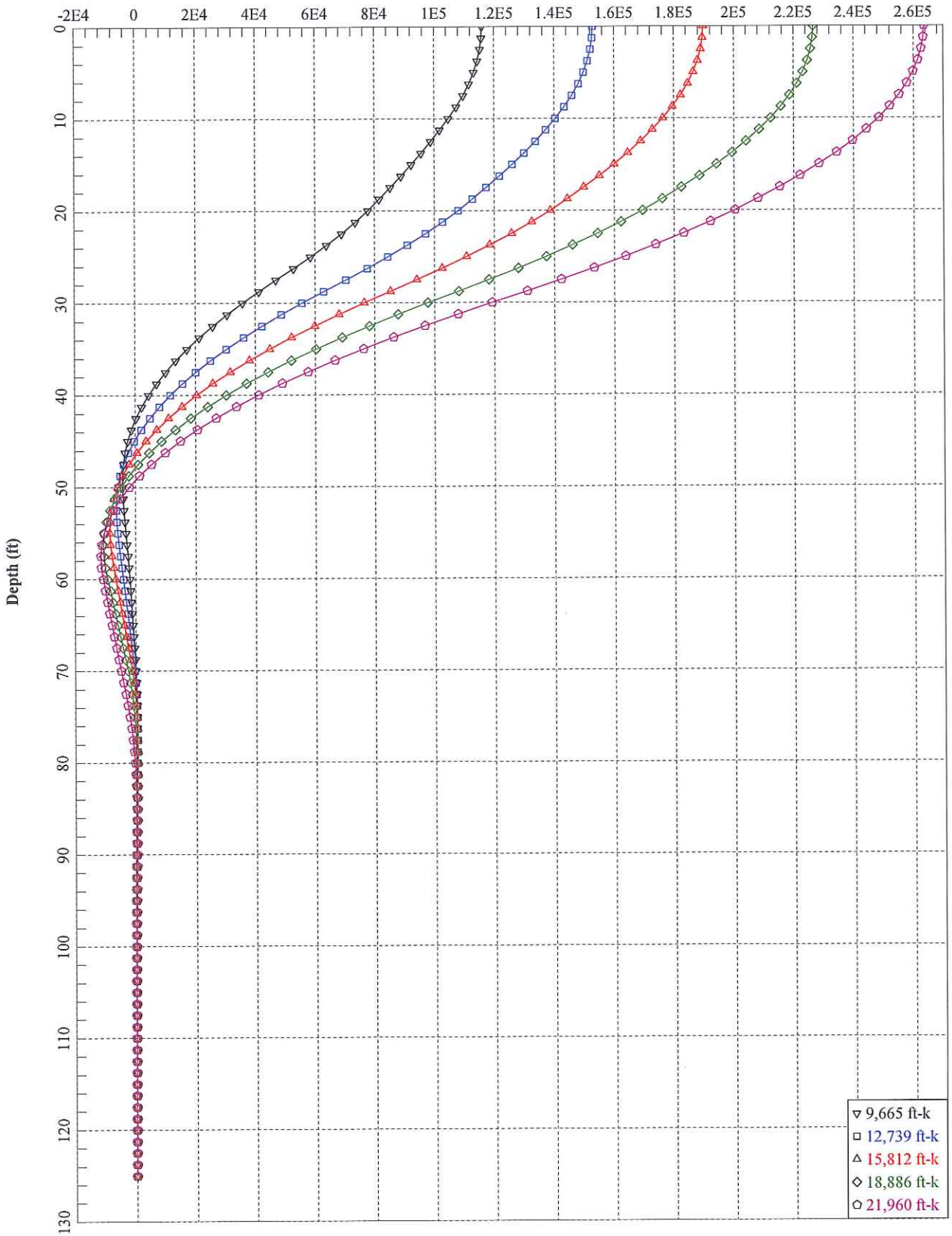


(9-foot Diameter - Model #6 - Moment Only)



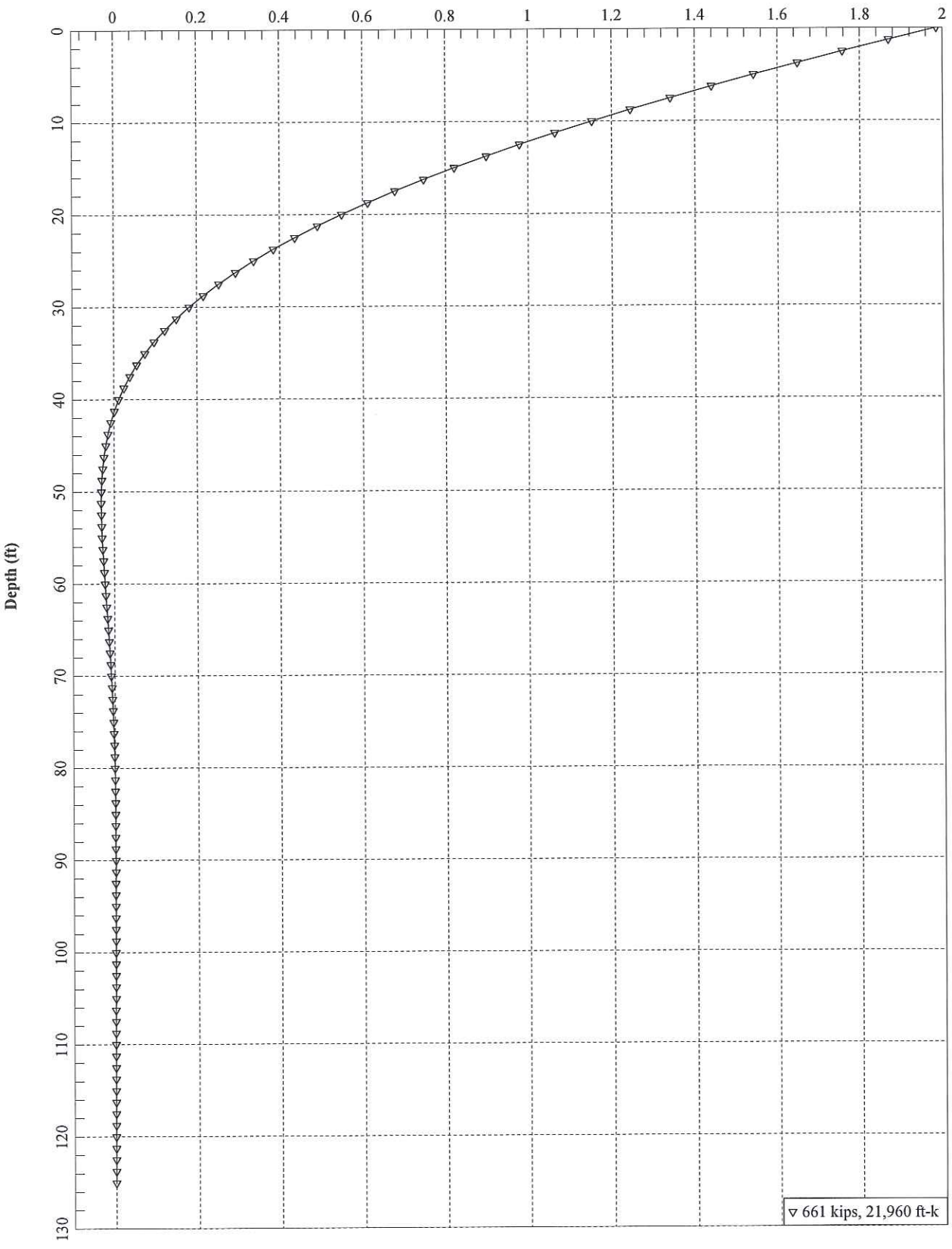
(9-foot Diameter - Model #6 - Moment Only)

Bending Moment (in-kips)

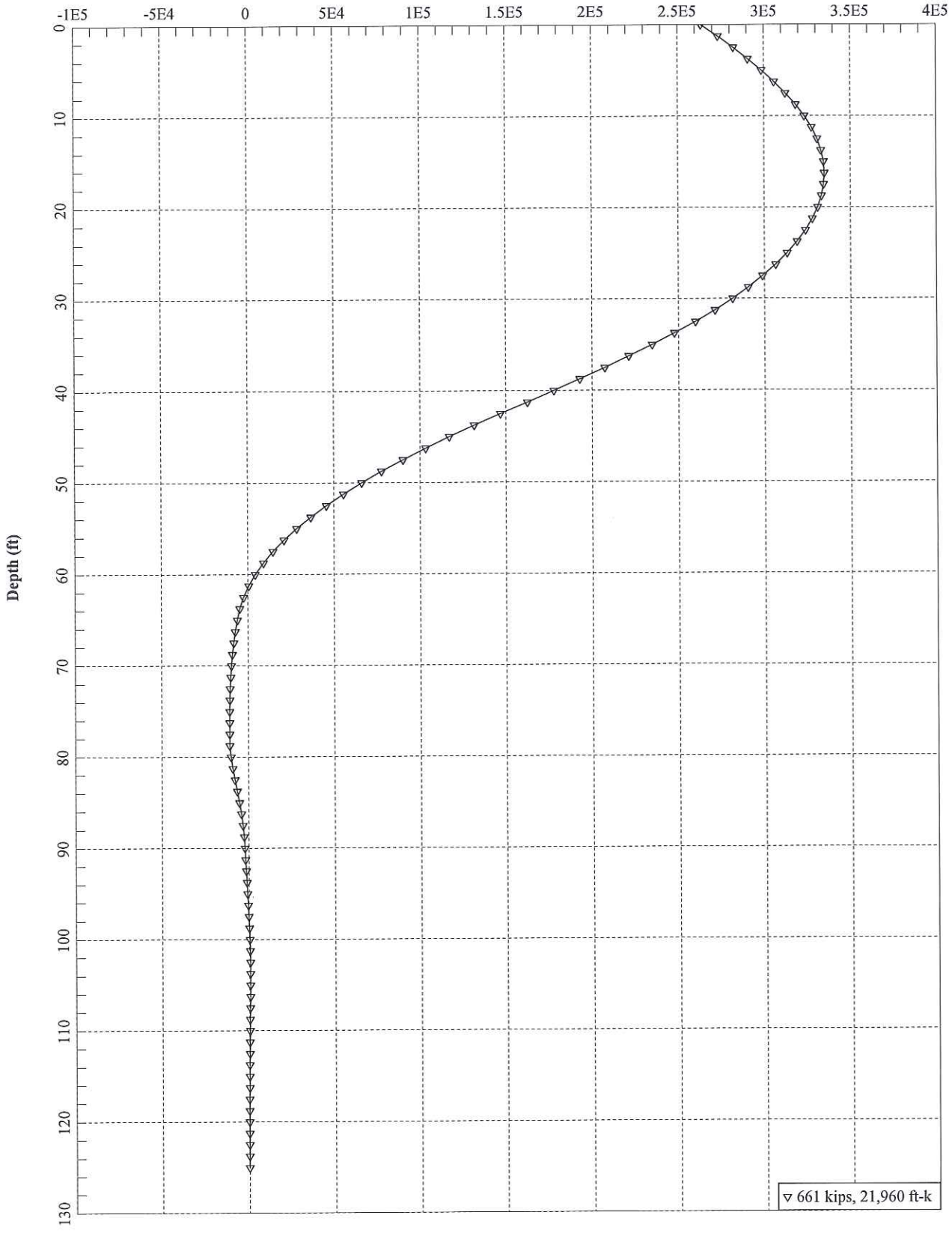


Deflection and Moment Curves for 9 foot Diameter Piles – Shear & Moment Applied

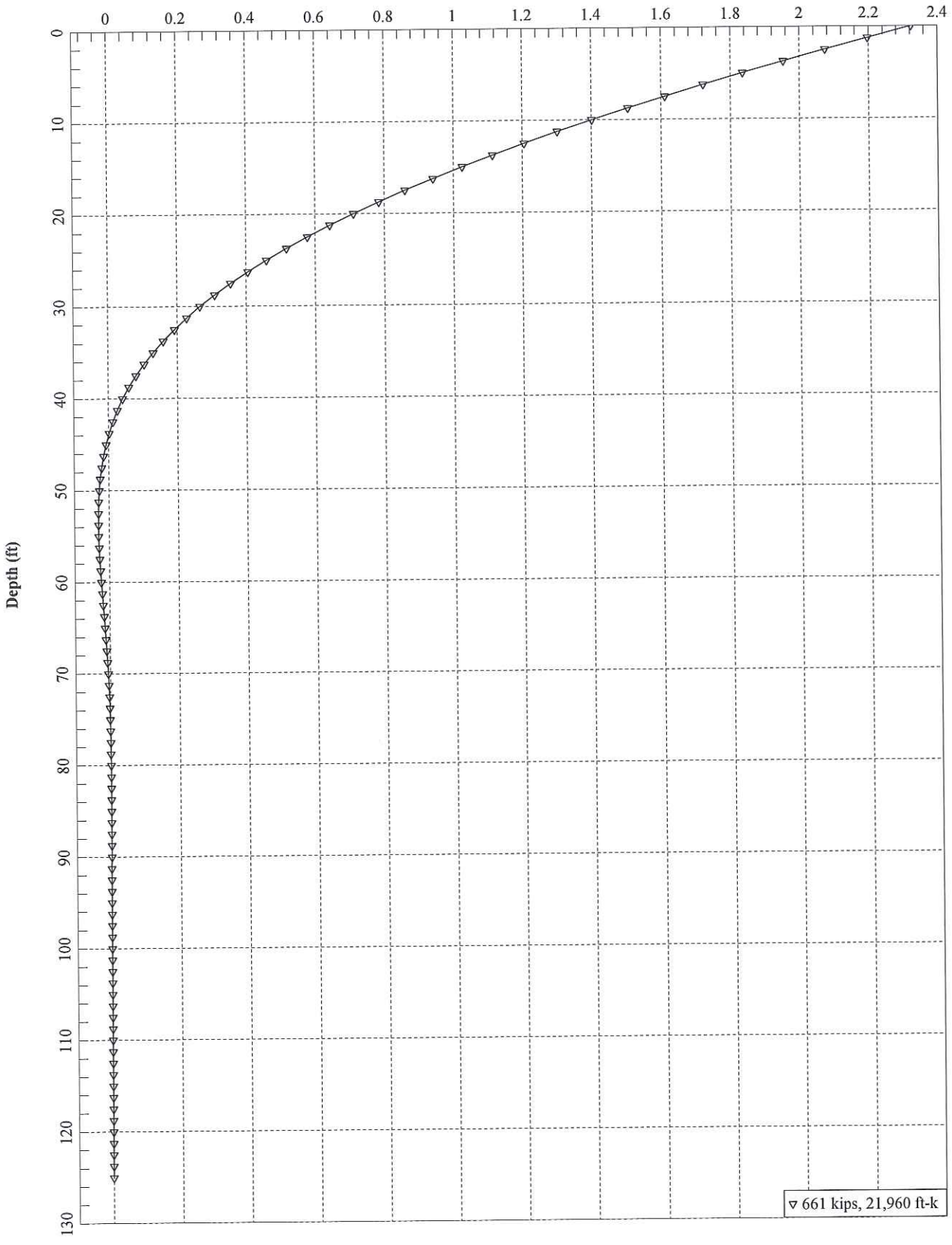
(9-foot Diameter-Model #4- Shear and Moment)



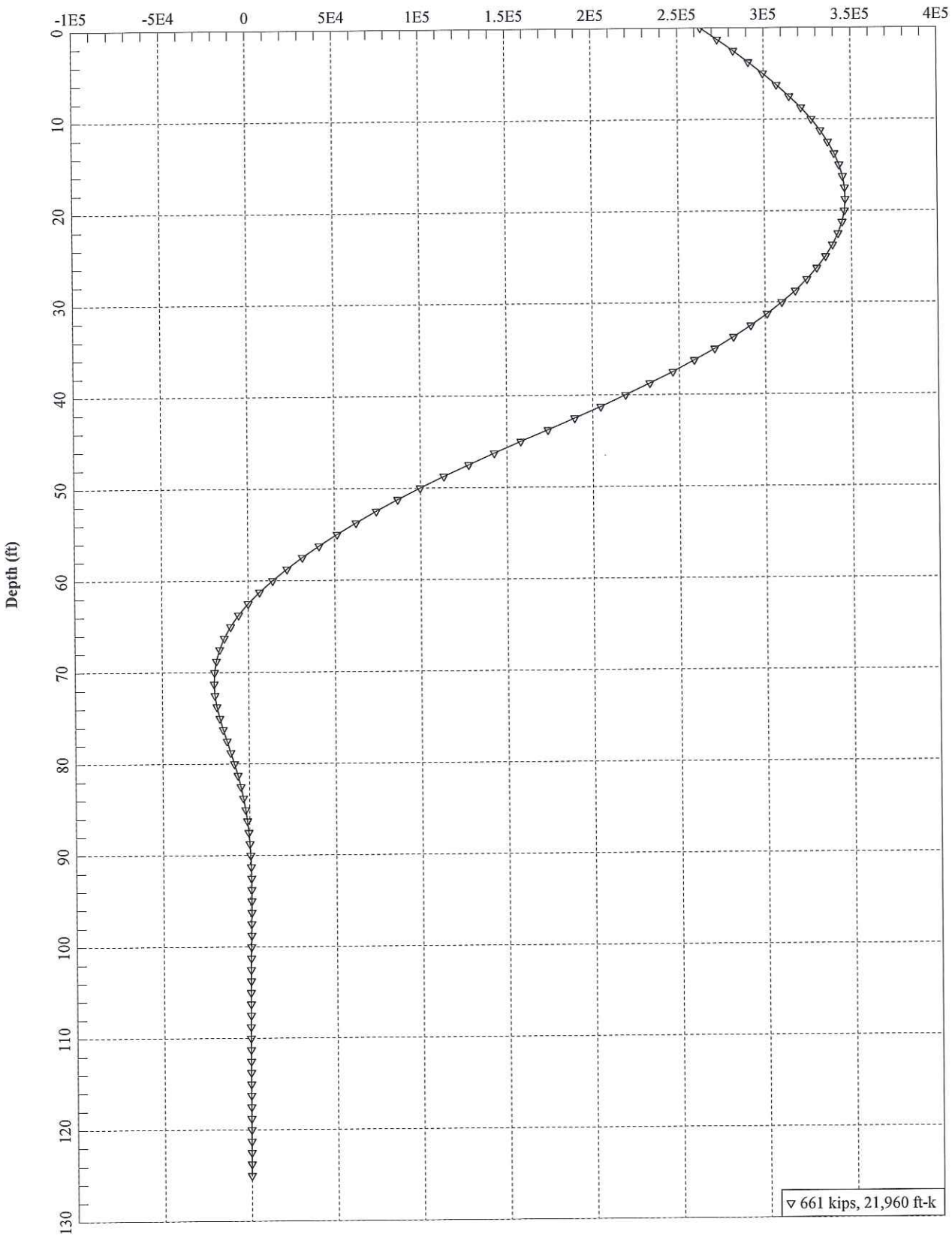
(9-foot Diameter-Model #4- Shear and Moment)



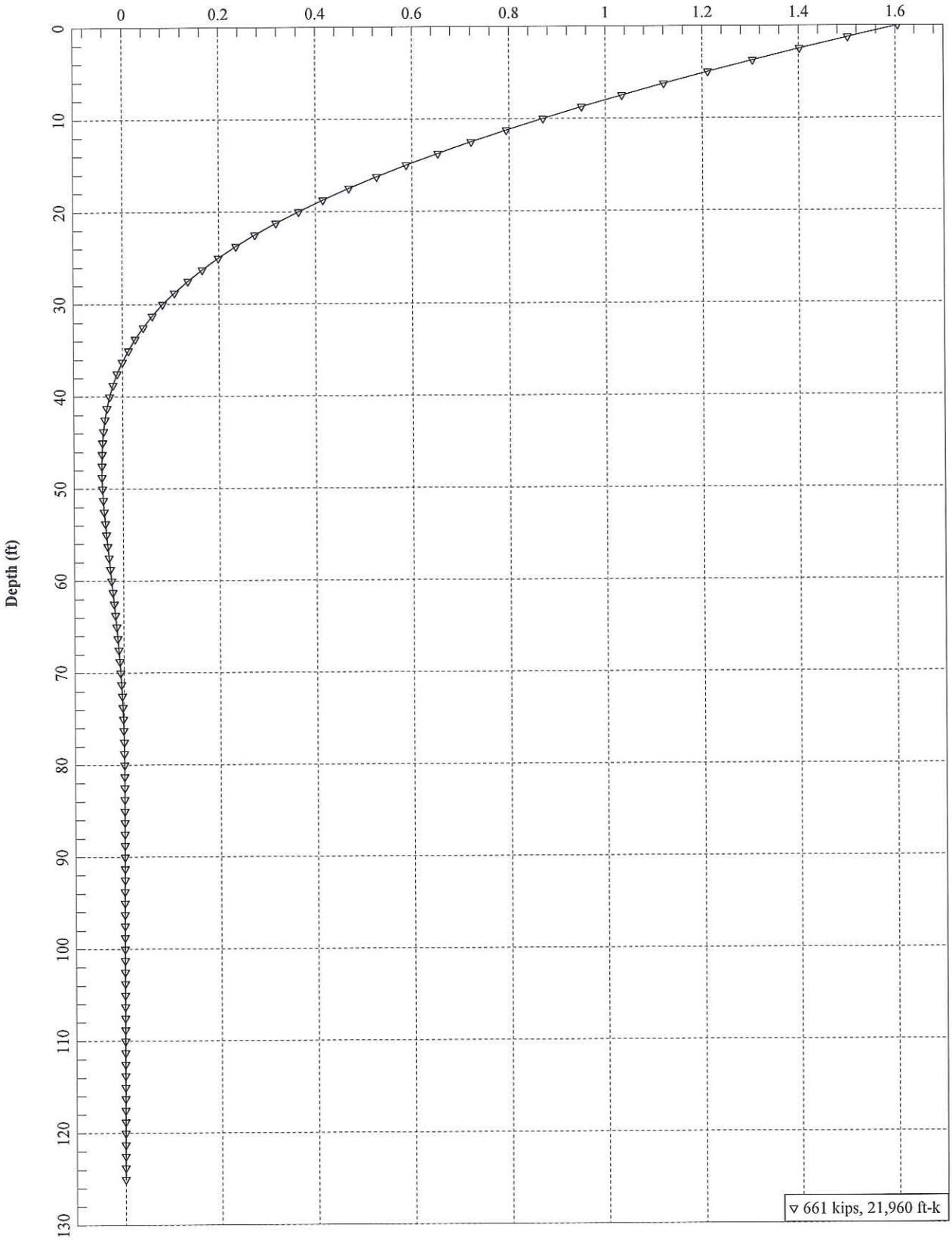
(9-foot Diameter-Model #5 - Shear and Moment)



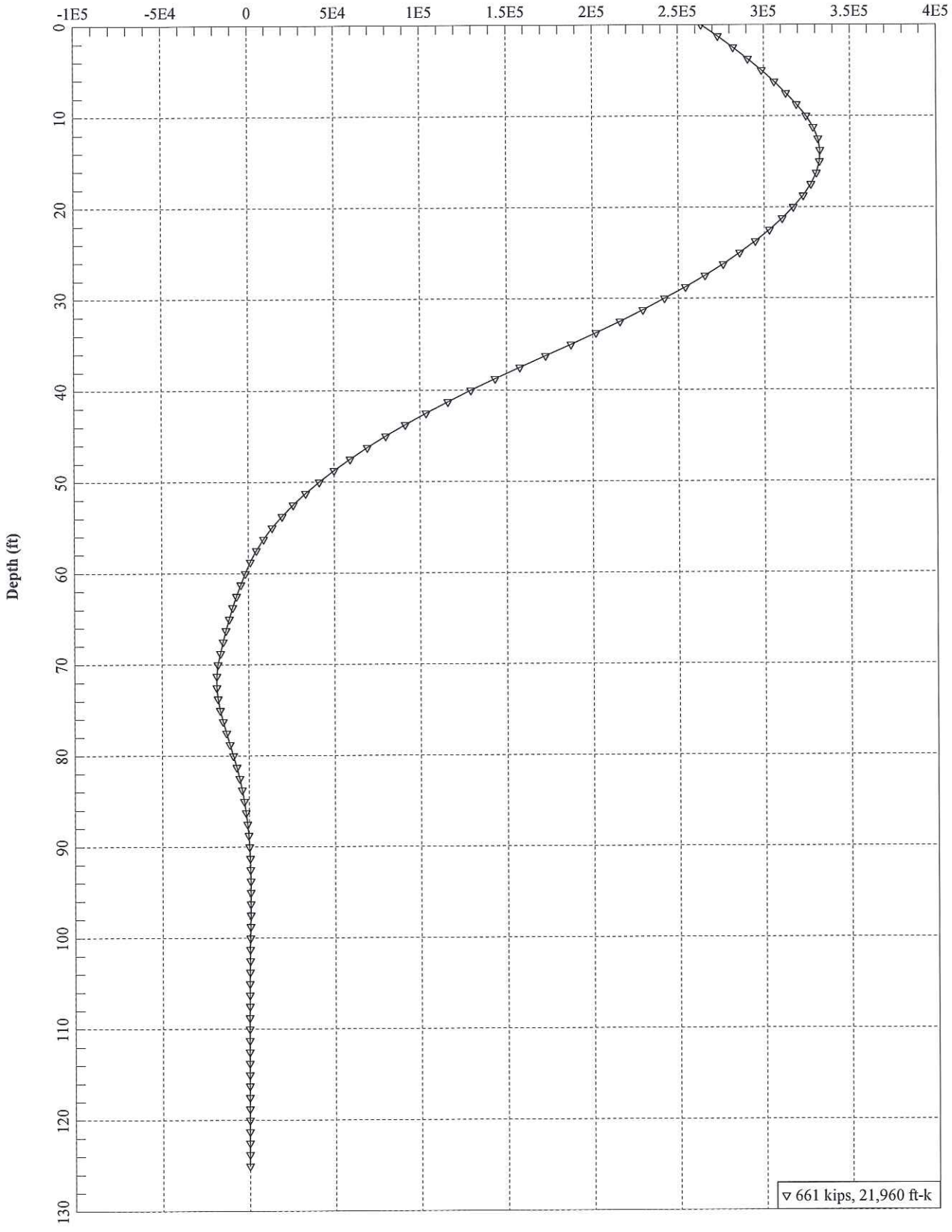
(9-foot Diameter-Model #5- Shear and Moment)



(9-foot Diameter - Model #6 - Shear and Moment)



(9-foot Diameter - Model #6 - Shear and Moment)



APPENDIX J
EMBANKMENT SETTLEMENT ANALYSIS AND GROUND IMPROVEMENT ALTERNATIVES

LIST OF ATTACHMENTS

The following are attached and complete this appendix.

Settlement Analysis and Ground Improvement Alternatives Summary
Subsurface Soil Model Profile and Analysis
Wick Drain Analysis
Ground Improvement Assessment
Settlement Reduction – Distance From Crest of Embankment/Toe of MSE Wall

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Settlement Analysis and Ground Improvement Alternatives Summary

SR-4 Crosstown Freeway Extension Project
Consolidation Settlement and Rate Analysis
and
Ground Improvement Cost Estimates

1. Introduction

The SR-4 project consists of construction of a main viaduct structure and approach embankments at its eastern and western ends. The eastern approach embankments will be constructed with side slopes of 4 (Horizontal):1(vertical). The western approach embankments (maximum height of 42 feet) will be constructed of three MSE walls (Nos. 1, 2, and 3) over existing grade with a total length of approximately 790 feet at the north edge (Wall #2) and 640 feet at the south edge (Walls #1 and 3).

2. Subsurface Soil Conditions

Review of boring logs indicates that the western and eastern approach embankments are generally underlain by stiff to very stiff lean clay/silt/clayey sand, interbedded with medium dense silty sand, to a depth of about 40 to 60 feet below existing grades. Below this layer are layers of very stiff to hard silt/lean clay and/or dense to very dense sandy soils to the maximum depths explored. Groundwater was encountered at a depth of about 7 to 14 feet at the western approach embankment below existing ground surface, and 11 to 36 feet below existing ground surface at the eastern approach embankment. Logs of borings and CPT's are presented in Appendices A and B. Generalized soil profiles for the settlement analysis (eastern approach embankment) are presented at the rear of this appendix.

Kleinfelder and Prikh, peformed consolidation tests on five selected soil samples. Results of consolidation testing are summarized in the table below.

Boring No.	Depth (ft)	Compression Ratio		Preconsolidation Pressure (psf)	Coeff. Of Consolidation C_v (ft2/day)	Approx. OCR
		NC	OC			
R-11-012	20	0.05	0.01	1,700	0.9	1.0
R-11-013	15	0.05	0.015	1,500	0.9	1.0
A-09-001	16	0.05	0.005	1,500	0.73 to 0.94	1.0
A-09-003	16	0.05	0.005	1,800	0.83 to 1.08	1.2
A-09-003	41	0.2	0.03	11,000	0.17 to 0.37	5.0

Notes:
1. Normally consolidated
2. Over consolidated

Based on correlations with moisture content per Lambe & Whitman (1969), soil compression ratios were estimated to be 0.05 to 0.15, which is slightly higher than the lab test results. In our settlement analysis later, we used compression ratios as summarized in the table above.

3. Estimation of Consolidation Settlements without Ground Improvement

We anticipate the settlement criteria for the proposed approach embankments will be established based on the required performance of pavement structure and adjacent bridge structures. On a preliminary basis, the settlement criteria for the approach embankments include a total post-construction settlement of 1 ½ inches over 100 feet.

Settlement of approach embankments typically consist of settlement of embankment itself due to self-weight, immediate settlement of foundations, and primary and secondary consolidation settlements of clayey soils. On a preliminary basis, the following assumptions have been made in the estimation of settlement of approach embankments:

- Embankment settlement due to its self weight is not included. In other words, settlement of embankment itself will complete during the construction phase, and will not contribute to the post-construction settlement of approach embankment;
- Embankment and surcharge loadings are applied instantaneously;
- Immediate settlement of foundations is not considered. We assumed that immediate settlement will occur during the construction phase, and its effect will be mitigated by overbuild the embankment sections during construction;
- Settlement of sandy soils (e.g., sand with silt, silty sand) is not estimated. We assumed the settlement of sandy soils will occur during the construction phase.
- Only primary consolidation of clayey soils will be considered. Secondary settlement of clayey soils is discounted in the estimation.

In the estimation of primary consolidation of clays/silts, the subsurface clay/silts are classified into two layers: The upper zone and the lower zone. We estimated consolidation settlements along the project alignment for soil profiles obtained from Borings R-11-013, R-11-012, and A-09-001; and CPT-11-011 and CPT-11-001. For illustration purposes, the table below presents post-construction settlement at MSE wall abutment #1, which corresponds to an embankment height of about 42 feet. Details of consolidation settlement analysis are presented later in this appendix.

West Approach Embankment.	Total Settlement (Upper Zone + Lower Zone) [1] (inches)	Post-Construction Settlement after 90 percent Consolidation (inches)	Time required for 90 percent consolidation [2] (days)		Remarks
			Upper Zone	Lower Zone	
A-09-001	13.2 (8+5)	1.3	109	109	

Notes:

1. Settlements considered for the Upper Zone and Lower Zone are primary consolidation only
2. Conservative values were used for Borings R-11-012, R-11-013 and A-09-001.

3. A fraction of settlements should occur during the construction of embankments. Therefore waiting periods are estimated by applying a reduction factor of 0.7 to the time required for 90 percent degree of consolidation.

It can be seen from the table above that:

1. Estimated maximum consolidation settlement is about 13 inches, depending on the underlying soil conditions. Considering the difference between actual field conditions and assumed soil conditions in our analysis, a reduction factor of about 0.70 to 0.85 should be adopted for a better estimation of consolidation settlements. Therefore, we anticipated that consolidation settlement will be on the order of about 10 inches for a 42-foot-high embankment.
2. The contribution of the lower clay/silt layer to the total settlement accounts for about 40 percent for the Borings R-11-013/R-11-012/A-09-001 profile.
3. Post-construction settlements after 90 percent consolidation are on the order of about 1.5 inches, which exceeds the tolerable settlements of 1 inch. Surcharge may be considered to deal with issue for high embankments (keep in mind that post-settlements for lower embankments may meet the post-construction settlement criteria).
4. Required time for 90 percent consolidation varies is about 3 to 4 months, depending on the permeability of underlying soils. It is also important to note that the field permeability of underlying soils is usually higher than tested in the laboratory.

4. Ground Improvement Recommendations

Typically, the following options have been considered in the past to control short- and long-term settlement of roadway embankment due to the consolidation of underlying soft to medium stiff silt/clay:

- Lightweight embankment fill (e.g., geoform, shredded tires)
- Wick drain with or without surcharge
- Stone Columns
- Geopiers
- Jetting Columns

The principal involved with the use of lightweight embankment fill (e.g., geofoam) is to reduce the magnitude of consolidation pressure and thus reduce the consolidation settlement. In the construction practice, geofoam has seen wide application and acceptance as a roadway construction material in the past years. The unit weight of geofoam is only about 1 to 2 pcf compared to typical soil unit weight of about 120 to 130 pcf. The use of geofoam as fill material needs to consider potential fire-related issues and

buoancy due to ground water. The geofoam is usually covered by a 3- to 5- foot layer of soils to ensure the appropriate performance of embankment.

The use of wick drains (prefabricated vertical drain) will accelerate the consolidation settlement rate. Depending on the spacing of wick drains, the time needed for primary consolidation to occur can be reduced by up to two thirds. It should be noted that the magnitude of consolidation settlement will not be reduced even with application of wick drains. Most commonly used wick drains are 4 inch wide with an equivalent drain diameter of 0.22 feet.

Stone columns and geopiers are similar in terms of expected performance. Application of stone columns and geopiers not only reduce the magnitude of consolidation of clays due to the increased stiffness of soils but also increase the bearing capacity of soils, which is beneficial especially for the stability of roadway approach embankment.

The approach embankments for the proposed project will be constructed predominately of MSE walls. Therefore, the use of lightweight fill such as geofoam will not be appropriate. Wick drains will be the most cost effective solution if time is not a significant factor in the project construction. Based on the estimation of consolidation settlement and rate, we recommend on a preliminary basis that wick drains and surcharge (if necessary, depending on the height of embankments) be used at the project site to improve the in-situ ground conditions).

Wick Drains

Effects of wick drains depend primarily on the spacing of wick drains. On a preliminary basis, we assumed a wick drain spacing of 6 feet to estimate the time required for 90 percent consolidation, based on the soil profiles obtained from R-11-012, R-11-013 and A-09-001. Results of our calculations are summarized in the table below. Details of consolidation rate analysis are presented later in this appendix.

West Approach Embankment.	Sample Depth (ft)	T_{90} (minutes)	Coeff. Of Consol. (ft2/day)	Time for 90% Consol. Without Wick drains (days)	Time for 90% Consol. With Wick drains (days)
Upper Bound Rate	16	2.25	0.94	85	32
Lower Bound Rate	16	2.89	0.73	109	41

It can be seen from the table above that time required for 90 percent degree of consolidation with wick drains varies from 32 to 41 days. In our analysis, we assumed the horizontal and vertical coefficients of consolidation are equal to each other. In the field condition, however, the horizontal permeability of the soils is usually 2 to 3 times

higher than the vertical permeability, resulting horizontal coefficient of consolidation 2 to 3 times higher than vertical one. Therefore, we anticipate that 90 percent degree of consolidation can be achieved along the most parts of the alignment by the end of the construction of embankments.

Geopiers

We considered 30-inch geopiers installed to a depth of about 30 feet below ground level at a center-to-center spacing of 5 to 7 feet. The settlement of the upper zone will be reduced to be about 2 inches due to the effects of geopier reinforcement. According to the consolidation settlement estimated for the "No Ground Improvement" option, we anticipated that the settlement of geopier-supported embankment would be on the order of about 4 to 7 inches.

5. Cost Estimation

The table below presents approximate cost estimates for the three ground improvement techniques. For the Preloading option, the cost will involve the placement and removal of earthen fill used to preload the foundation soils. The cost for this option varies, depending on the construction schedule, and sizes and sequences of areas selected for preloading.

Option No.	Ground Improvement technique	Approximate Costs ($)	Remarks
1	Preloading	N/A	
2	Wick Drains	305,000	
3	Geopiers	2,160,000	

Detailed cost estimation for Options No. 2 and 3 are presented later in the appendix.

Subsurface Soil Model Profile and Analysis

Prepared By: HP Date: 6/9/2011
Checked By: CH Date: 6/10/2011

Boring No. :	R-11-013		CPT-11-011	R-11-012		CPT-11-001
	Undrained Shear Strength (psf) Based on					
Depth (feet)	Blow Counts/ft	*Pocket Penetrometer data	CPT-11-011 Data	Blow Counts/ft	*Pocket Penetrometer data	CPT-11-001 Data
3	NT	NT	4,000	15	3,500	3,600
5	13	3,000	4,000	24	NT	2,100
10	NT	NT	7,000	9	2,500	4,900
15	37	3,500	3,200	10	3,500	1,700
20	8	3,500	2,800	NT	3,500	2,400
25	13	NT	5,000	30	NT	
30	16	3,500		9	NT	
35				15	4,000	2,200
40				12	NT	2,600
45			1,700	23	4,000	2,600
50			4,500	23	4,000	3,100
55						4,400
60						
65						4,300
70						8,000
75						4,500
80						4,800
85						5,000
90						2,360
95						N/T
100						N/T
110						N/T
120						

* Highest reading on pocket penetrometer for unconfined compression strength is

NT= Not Tested

= Sand

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Sheet 1 of 1

PROJECT CR 4 CROSSTOWN PROJECT NO. 111904.G01

SUBJECT R-11-013 / R-11-012 / A-09-001 BY YLI DATE 9/26/10

IDEALIZED SOIL PROFILE REVIEWED BY CFH DATE 10/07/10

REVISED BY CFH 7/15/11

Elev. 6

0 ▽ G.W (ASSUMED)

Sandy lean clay (CL)
Silt (ML)
Lean clay with sand (CL)
N = 9 ~ 20
LL = 26 - 30
PI = 3 - 20 w = (24 - 32)%

CR = 0.05 ~ 0.09 (Lambe and Whitman, 1969)

CR = 0.05 (KLEINFELDER & PARIKH CONSOLIDATION TEST DATA)

-20

Silty sand (SM) / Sand with Silt (SW-SM)
N = 10 ~ > 50/6"
w = (13 - 15)%

-30

Lean clay (CL) / with sand
N = 11 ~ 16
LL = 36 ~ 47
PI = 14 ~ 23
w = (28 - 33)%

CR = 0.07 ~ 0.1

-55

Non-Compressible
Silty Sand (SM)
N = 48 ~ > 63/6"

-70

SILT (ML), very stiff
N = 24
w = 31%

CR_recompression = 0.01

Elev. -80

Non-Compressible Base

SR-4 Crosstwon Freeway Extension Project

Consolidation Settlement Analysis for Approach Embankment

Prepared By: YLI Date 10/8/2010

Checked By: CFH Date 10/8/2010

REVISED BY: CFH DATE 7/13/2011

Based on Boring A-09-001 /R-11-010 / R-11-013

45 feet, Fill Height

100 feet, Fill Width

130 pcf, Fill density

Soil No.	Depth (ft) Top	Depth (ft) Bottom	Thickness (ft)	Mid Depth (ft)	Effective Unit Weight (pcf)	Weight of the Layer (psf)	Midlayer Effective Overburden pressure (psf)	Applied Pressure (psf)	Compression Ratio	Settlement (inches)
1	0	6	6	3	125	750	375	5680	0.0001	0.0
2	6	26	20	16	62.6	1252	1376	5043	0.05	8.0
3	26	36	10	31	62.6	626	2315	4466	0.0001	0.0
4	36	61	25	48.5	62.6	1565	3410.5	3939	0.05	5.0
5	61	76	15	68.5	62.6	939	4662.5	3472	0.0001	0.0
6	76	86	10	81	62.6	626	5445	3232	0.01	0.2
7	86	110	24	98	62.6	1502.4	6509.2	2955	0.0001	0.0
									Total Settlement	13.3

Fill Height (ft)	Consol. Settlement (inches)	70% of Estimated Consol. Settlement (inches)
5	2.6	1.82
10	4.7	3.29
15	6.4	4.48
20	7.9	5.53
25	9.2	6.44
30	10.4	7.28
35	11.4	7.98
40	12.4	8.68
45	13.3	9.31
50	14.1	9.87

Wick Drain Analysis

CONSOLIDATION-TIME CALCULATIONS ACCORDING TO BARRON-HANSBO EQUATION

ASSUMPTIONS:

1. Radial drainage only
2. Uniform soil deposit
3. No backpressure from horizontal drainage
4. Soil parameters constant in time and position
5. No drain resistance considered
6. No disturbance considered

INPUT

Ch =	0.94	ft^2/day	
U =	90	%	# Days
Spacing 1 =	8	ft	63
Spacing 2 =	6	ft	32
Spacing 3 =	5	ft	20
Spacing 4 =	3	ft	6

Triangular
Square x

Equivalent drain diameter = 0.22 ft
(Recommend = 0.22 ft)

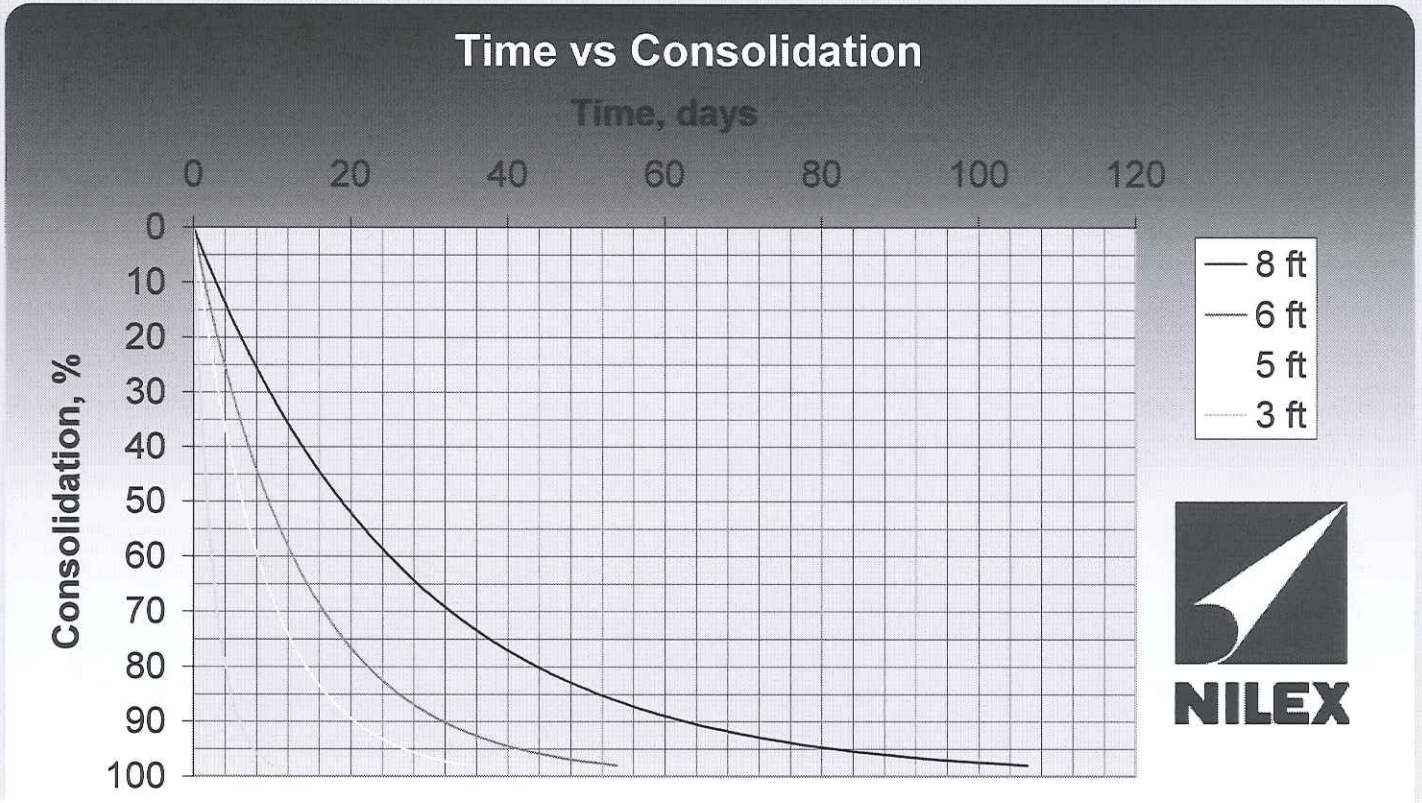

PLATE C-5

CONSOLIDATION-TIME CALCULATIONS ACCORDING TO BARRON-HANSBO EQUATION

ASSUMPTIONS:

1. Radial drainage only
2. Uniform soil deposit
3. No backpressure from horizontal drainage
4. Soil parameters constant in time and position
5. No drain resistance considered
6. No disturbance considered

	INPUT		
Ch =	0.73	ft^2/day	
U =	90	%	# Days
Spacing 1 =	8	ft	81
Spacing 2 =	6	ft	41
Spacing 3 =	5	ft	26
Spacing 4 =	3	ft	8
Triangular	x		
Square			
Equivalent drain diameter =	0.22	ft	
(Recommend = 0.22 ft)			

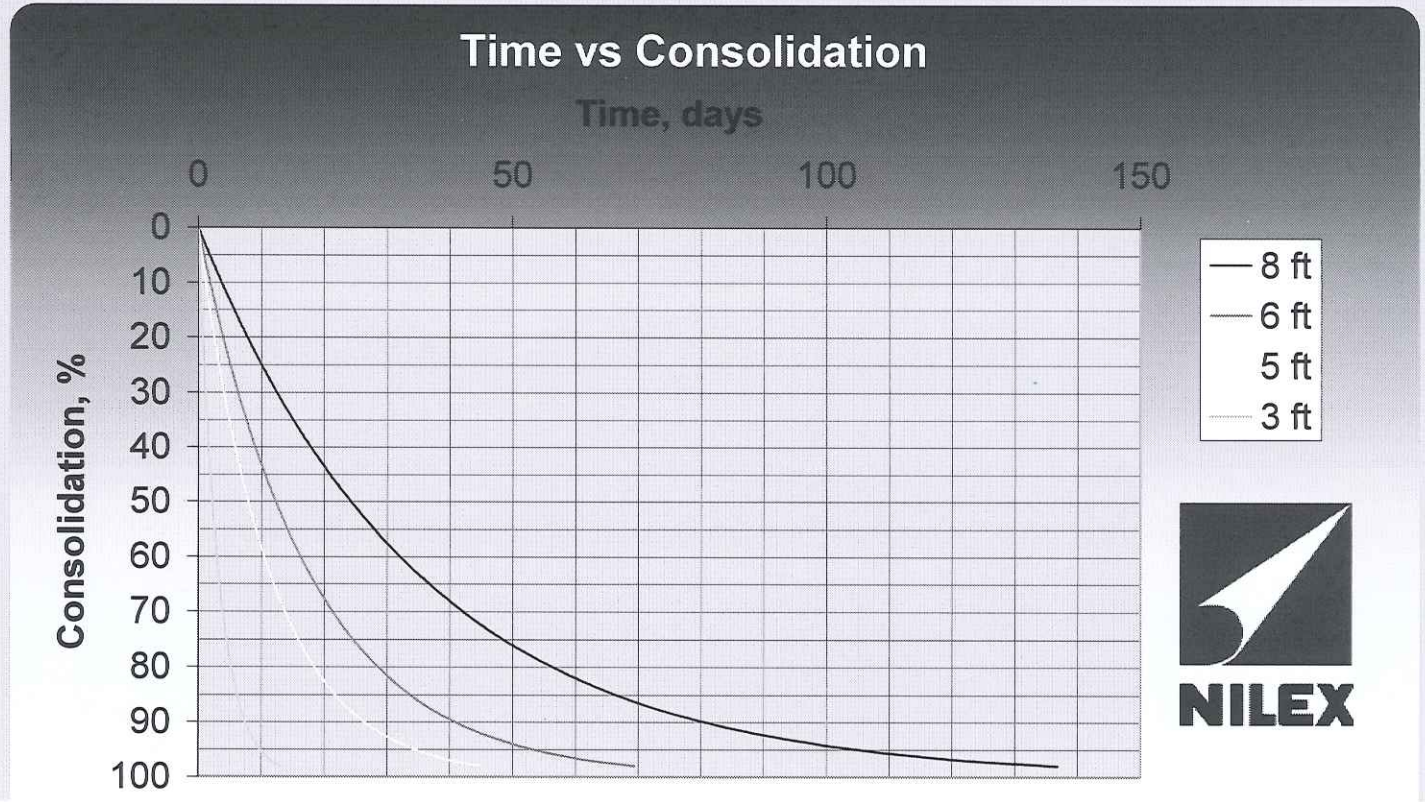

PLATE C-6

Ground Improvement Assessment

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PROJECT SR-4 crosstown PROJECT NO. 111904. G01

SUBJECT Ground improvement BY YLi DATE 10/8/10

REVIEWED BY CFH DATE 10/8/10

Wick Drains

* 60 feet long each
* 5 feet spacing
* Western embankment: about 1000 ft long, wide 110 to 75 ft
  Eastern embankment: about 300 ft long wide 170 ft

Embankment	Row	column (along the alignment)	PV length (ft)
WEST	100/5 = 20	1000/5 = 200	20 (201) (60) = 241,200
EAST	170/5 = 34	300/5 = 60	34 (61) (60) = 124,440
			= 365,640 ft

use strip drains to discharge the water (instead of drain rock or sandy blanet):

Strip drains: 20,000 + 10200 = 30,200

west Embank East embank

Cost Analysis:

ITEM	UNIT PRICE	Quantity	COST
PV DRAIN	0.6/ft	365,640	219,384
STRIP DRAIN	1.3/ft	30,200	39,260
MOB	—	—	20,000
			278,644 (1.1 = 306,508

Amplification factor 1.1

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Sheet 1 of 2

PROJECT SR-4 PROJECT NO. 111904.G01

SUBJECT Ground Improvement BY YC DATE 10/8/10

REVIEWED BY CEH DATE 10/8/10

Geopiers

* 30 feet long each
* 6 feet spacing (average)
* West Embankment: 1000 feet long by 100 feet wide (average)

East Embankment: 300 feet long by 170 feet wide

Embankment	Row	Column	Pier lengths (ft)
WEST	100/6 = 17	1000/6 = 167	17(168)(30) = 85,680
East	170/6 = 28	300/6 = 50	28(50)(30) = 42,840
			Total: 128,520 ft

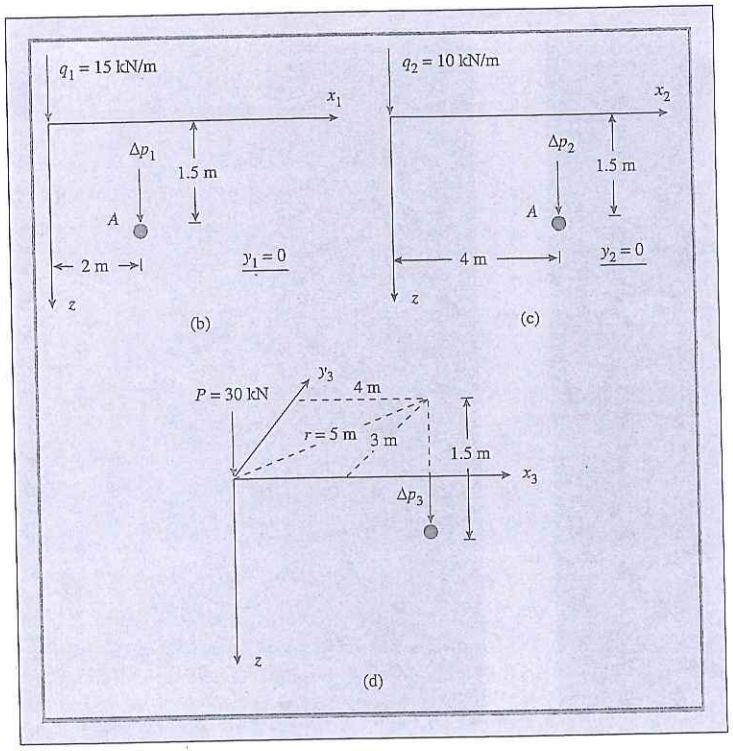
Item	Unit price	Quantity	cost ($)
Geopier	42 / ft	128,520	5,397,840

the calculations above are based on improvement across embankment foot prints. We anticipate foundations for embankment less than 15 ft to 20 high may not require ground improvement.

Embankment 15-20 ft high accounts for about 40 ~~to~~ 50 percent of all the approach embankments therefore,

(5,397,840)(40%) = $2,159,136

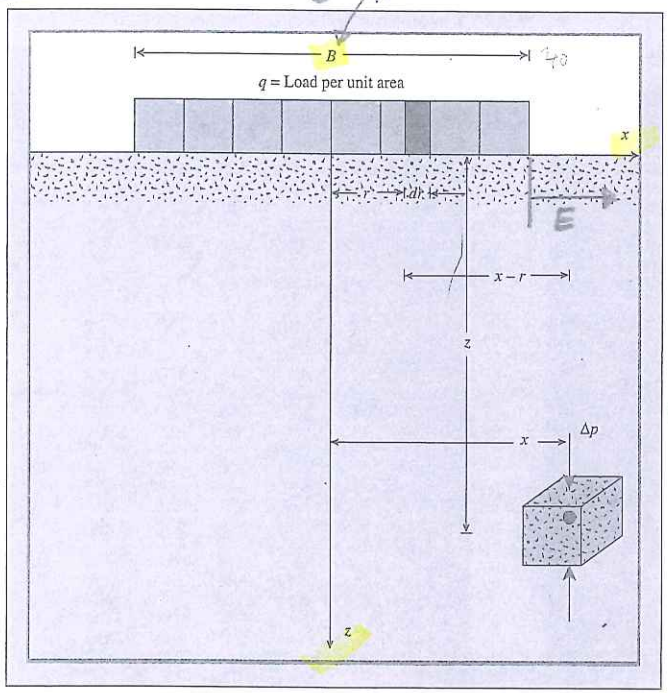
Settlement Reduction – Distance From Crest of Embankment/Toe of MSE Wall



▼ FIGURE 7.12 Continued

7.6 VERTICAL STRESS CAUSED BY A STRIP LOAD (FINITE WIDTH AND INFINITE LENGTH)

The fundamental equation for the vertical stress increase at a point in a soil mass as the result of a line load (Section 7.5) can be used to determine the vertical stress at a point caused by a flexible strip load of width B (Figure 7.13). Let the load per unit area of the strip shown in Figure 7.13 be equal to q. If we consider an elemental strip of width dr, the load per unit length of this strip is equal to $q\,dr$. This elemental strip can be treated as a line load. Equation (7.18) gives the vertical stress increase, dp, at point A inside the soil mass caused by this elemental strip load. To calculate the vertical stress increase, we need to substitute $q\,dr$ for q and $(x - r)$ for x. So,



▼ FIGURE 7.13 Vertical stress caused by a flexible strip load

The total increase in the vertical stress (Δp) at point A caused by the entire strip load of width B can be determined by integration of Eq. (7.20) with limits of r from $-B/2$ to $+B/2$, or

$$\Delta p = \int dp = \int_{-B/2}^{+B/2} \left(\frac{2q}{\pi}\right)\left\{\frac{z^3}{[(x-r)^2+z^2]^2}\right\} dr$$

$$= \frac{q}{\pi}\left\{\tan^{-1}\left[\frac{z}{x-(B/2)}\right] - \tan^{-1}\left[\frac{z}{x+(B/2)}\right] - \frac{Bz[x^2 - z^2 - (B^2/4)]}{[x^2+z^2-(B^2/4)]+B^2z^2}\right\} \quad (7.21)$$

Table 7.3 shows the variation of $\Delta p/q$ with $2z/B$ for $2x/B$. This table can be

BELOW 50FT DEPTH OF 90 FT SOIL SETTLEMENT IS ASSUMED AS NEGLIGIBLE

B=110 FT

E = DISTANCE FROM EDGE OF MSE WALL/ EMBANKMENT (FT)

▼ TABLE 7.3 Variation of $\Delta p/q$ with $2z/B$ and $2x/B$ [Eq. (7.21)]

2z/B	2x/B 0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
0.00	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	0.500
0.10	1.000	1.000	0.999	0.999	0.999	0.998	0.997	0.993	0.980	0.909	0.500
0.20	0.997	0.997	0.996	0.995	0.992	0.988	0.979	0.959	0.909	0.775	0.500
0.30	0.990	0.989	0.987	0.984	0.978	0.967	0.947	0.908	0.833	0.697	0.499
0.40	0.977	0.976	0.973	0.966	0.955	0.937	0.906	0.855	0.773	0.651	0.498
0.50	0.959	0.958	0.953	0.943	0.927	0.902	0.864	0.808	0.727	0.620	0.497
0.60	0.937	0.935	0.928	0.915	0.896	0.866	0.825	0.767	0.691	0.598	0.495
0.70	0.910	0.908	0.899	0.885	0.863	0.831	0.788	0.732	0.662	0.581	0.492
0.80	0.881	0.878	0.869	0.853	0.829	0.797	0.755	0.701	0.638	0.566	0.489
0.90	0.850	0.847	0.837	0.821	0.797	0.765	0.724	0.675	0.617	0.552	0.485
1.00	0.818	0.815	0.805	0.789	0.766	0.735	0.696	0.650	0.598	0.540	0.480
1.10	0.787	0.783	0.774	0.758	0.735	0.706	0.670	0.628	0.580	0.529	0.474
1.20	0.755	0.752	0.743	0.728	0.707	0.679	0.646	0.607	0.564	0.517	0.468
1.30	0.725	0.722	0.714	0.699	0.679	0.654	0.623	0.588	0.548	0.506	0.462
1.40	0.696	0.693	0.685	0.672	0.653	0.630	0.602	0.569	0.534	0.495	0.455
1.50	0.668	0.666	0.658	0.646	0.629	0.607	0.581	0.552	0.519	0.484	0.448
1.60	0.642	0.639	0.633	0.621	0.605	0.586	0.562	0.535	0.506	0.474	0.440
1.70	0.617	0.615	0.608	0.598	0.583	0.565	0.544	0.519	0.492	0.463	0.433
1.80	0.593	0.591	0.585	0.576	0.563	0.546	0.526	0.504	0.479	0.453	0.425
1.90	0.571	0.569	0.564	0.555	0.543	0.528	0.510	0.489	0.467	0.443	0.417
2.00	0.550	0.548	0.543	0.535	0.524	0.510	0.494	0.475	0.455	0.433	0.409
2.10	0.530	0.529	0.524	0.517	0.507	0.494	0.479	0.462	0.443	0.423	0.401
2.20	0.511	0.510	0.506	0.499	0.490	0.479	0.465	0.449	0.432	0.413	0.393
2.30	0.494	0.493	0.489	0.483	0.474	0.464	0.451	0.437	0.421	0.404	0.385
2.40	0.477	0.476	0.473	0.467	0.460	0.450	0.438	0.425	0.410	0.395	0.378
2.50	0.462	0.461	0.458	0.452	0.445	0.436	0.426	0.414	0.400	0.386	0.370
2.60	0.447	0.446	0.443	0.439	0.432	0.424	0.414	0.403	0.390	0.377	0.363
2.70	0.433	0.432	0.430	0.425	0.419	0.412	0.403	0.393	0.381	0.369	0.355
2.80	0.420	0.419	0.417	0.413	0.407	0.400	0.392	0.383	0.372	0.360	0.348
2.90	0.408	0.407	0.405	0.401	0.396	0.389	0.382	0.373	0.363	0.352	0.341
3.00	0.396	0.395	0.393	0.390	0.385	0.379	0.372	0.364	0.355	0.345	0.334
3.10	0.385	0.384	0.382	0.379	0.375	0.369	0.363	0.355	0.347	0.337	0.327
3.20	0.374	0.373	0.372	0.369	0.365	0.360	0.354	0.347	0.339	0.330	0.321
3.30	0.364	0.363	0.362	0.359	0.355	0.351	0.345	0.339	0.331	0.323	0.315
3.40	0.354	0.354	0.352	0.350	0.346	0.342	0.337	0.331	0.324	0.316	0.308
3.50	0.345	0.345	0.343	0.341	0.338	0.334	0.329	0.323	0.317	0.310	0.302
3.60	0.337	0.336	0.335	0.333	0.330	0.326	0.321	0.316	0.310	0.304	0.297
3.70	0.328	0.328	0.327	0.325	0.322	0.318	0.314	0.309	0.304	0.298	0.291
3.80	0.320	0.320	0.319	0.317	0.315	0.311	0.307	0.303	0.297	0.292	0.285
3.90	0.313	0.313	0.312	0.310	0.307	0.304	0.301	0.296	0.291	0.286	0.280
4.00	0.306	0.305	0.304	0.303	0.301	0.298	0.294	0.290	0.285	0.280	0.275
4.10	0.299	0.299	0.298	0.296	0.294	0.291	0.288	0.284	0.280	0.275	0.270
4.20	0.292	0.292	0.291	0.290	0.288	0.285	0.282	0.278	0.274	0.270	0.265
4.30	0.286	0.286	0.285	0.283	0.282	0.279	0.276	0.273	0.269	0.265	0.260
4.40	0.280	0.280	0.279	0.278	0.276	0.274	0.271	0.268	0.264	0.260	0.256
4.50	0.274	0.274	0.273	0.272	0.270	0.268	0.266	0.263	0.259	0.255	0.251
4.60	0.268	0.268	0.268	0.266	0.265	0.263	0.260	0.258	0.254	0.251	0.247
4.70	0.263	0.263	0.262	0.261	0.260	0.258	0.255	0.253	0.250	0.246	0.243
4.80	0.258	0.258	0.257	0.256	0.255	0.253	0.251	0.248	0.245	0.242	0.239
4.90	0.253	0.253	0.252	0.251	0.250	0.248	0.246	0.244	0.241	0.238	0.235
5.00	0.248	0.248	0.247	0.246	0.245	0.244	0.242	0.239	0.237	0.234	0.231

(continued)

▼ TABLE 7.3 Continued

2z/B	2x/B 1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.9	2.0
0.00	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.10	0.091	0.020	0.007	0.003	0.002	0.001	0.001	0.000	0.000	0.000
0.20	0.225	0.091	0.040	0.020	0.011	0.007	0.004	0.003	0.002	0.002
0.30	0.301	0.165	0.090	0.052	0.031	0.020	0.013	0.009	0.007	0.005
0.40	0.346	0.224	0.141	0.090	0.059	0.040	0.027	0.020	0.014	0.011
0.50	0.373	0.267	0.185	0.128	0.089	0.063	0.046	0.034	0.025	0.019
0.60	0.391	0.298	0.222	0.163	0.120	0.088	0.066	0.050	0.038	0.030
0.70	0.403	0.321	0.250	0.193	0.148	0.113	0.087	0.068	0.053	0.042
0.80	0.411	0.338	0.273	0.218	0.173	0.137	0.108	0.086	0.069	0.056
0.90	0.416	0.351	0.291	0.239	0.195	0.158	0.128	0.104	0.085	0.070
1.00	0.419	0.360	0.305	0.256	0.214	0.177	0.147	0.122	0.101	0.084
1.10	0.420	0.366	0.316	0.271	0.230	0.194	0.164	0.138	0.116	0.098
1.20	0.419	0.371	0.325	0.282	0.243	0.209	0.178	0.152	0.130	0.111
1.30	0.417	0.373	0.331	0.291	0.254	0.221	0.191	0.166	0.143	0.123
1.40	0.414	0.374	0.335	0.298	0.263	0.232	0.203	0.177	0.155	0.135
1.50	0.411	0.374	0.338	0.303	0.271	0.240	0.213	0.188	0.165	0.146
1.60	0.407	0.373	0.339	0.307	0.276	0.248	0.221	0.197	0.175	0.155
1.70	0.402	0.370	0.339	0.309	0.281	0.254	0.228	0.205	0.183	0.164
1.80	0.396	0.368	0.339	0.311	0.284	0.258	0.234	0.212	0.191	0.172
1.90	0.391	0.364	0.338	0.312	0.286	0.262	0.239	0.217	0.197	0.179
2.00	0.385	0.360	0.336	0.311	0.288	0.265	0.243	0.222	0.203	0.185
2.10	0.379	0.356	0.333	0.311	0.288	0.267	0.246	0.226	0.208	0.190
2.20	0.373	0.352	0.330	0.309	0.288	0.268	0.248	0.229	0.212	0.195
2.30	0.366	0.347	0.327	0.307	0.288	0.268	0.250	0.232	0.215	0.199
2.40	0.360	0.342	0.323	0.305	0.287	0.268	0.251	0.234	0.217	0.202
2.50	0.354	0.337	0.320	0.302	0.285	0.268	0.251	0.235	0.220	0.205
2.60	0.347	0.332	0.316	0.299	0.283	0.267	0.251	0.236	0.221	0.207
2.70	0.341	0.327	0.312	0.296	0.281	0.266	0.251	0.236	0.222	0.208
2.80	0.335	0.321	0.307	0.293	0.279	0.265	0.250	0.236	0.223	0.210
2.90	0.329	0.316	0.303	0.290	0.276	0.263	0.249	0.236	0.223	0.211
3.00	0.323	0.311	0.299	0.286	0.274	0.261	0.248	0.236	0.223	0.211
3.10	0.317	0.306	0.294	0.283	0.271	0.259	0.247	0.235	0.223	0.212
3.20	0.311	0.301	0.290	0.279	0.268	0.256	0.245	0.234	0.223	0.212
3.30	0.305	0.296	0.286	0.275	0.265	0.254	0.243	0.232	0.222	0.211
3.40	0.300	0.291	0.281	0.271	0.261	0.251	0.241	0.231	0.221	0.211
3.50	0.294	0.286	0.277	0.268	0.258	0.249	0.239	0.229	0.220	0.210
3.60	0.289	0.281	0.273	0.264	0.255	0.246	0.237	0.228	0.218	0.209
3.70	0.284	0.276	0.268	0.260	0.252	0.243	0.235	0.226	0.217	0.208
3.80	0.279	0.272	0.264	0.256	0.249	0.240	0.232	0.224	0.216	0.207
3.90	0.274	0.267	0.260	0.253	0.245	0.238	0.230	0.222	0.214	0.206
4.00	0.269	0.263	0.256	0.249	0.242	0.235	0.227	0.220	0.212	0.205
4.10	0.264	0.258	0.252	0.246	0.239	0.232	0.225	0.218	0.211	0.203
4.20	0.260	0.254	0.248	0.242	0.236	0.229	0.222	0.216	0.209	0.202
4.30	0.255	0.250	0.244	0.239	0.233	0.226	0.220	0.213	0.207	0.200
4.40	0.251	0.246	0.241	0.235	0.229	0.224	0.217	0.211	0.205	0.199
4.50	0.247	0.242	0.237	0.232	0.226	0.221	0.215	0.209	0.203	0.197
4.60	0.243	0.238	0.234	0.229	0.223	0.218	0.212	0.207	0.201	0.195
4.70	0.239	0.235	0.230	0.225	0.220	0.215	0.210	0.205	0.199	0.194
4.80	0.235	0.231	0.227	0.222	0.217	0.213	0.208	0.202	0.197	0.192
4.90	0.231	0.227	0.223	0.219	0.215	0.210	0.205	0.200	0.195	0.190
5.00	0.227	0.224	0.220	0.216	0.212	0.207	0.203	0.198	0.193	0.188

E	x	2x/B
0	55	1
25	80	1.45
50	105	1.91
75	130	2.34
100	155	2.82
125	180	3.27
150	205	3.72
10	60	1.09

Z	27/6
5	0.09
0	0.18
5	0.27
20	0.36
25	0.45
30	0.55
35	0.64
40	0.73
45	0.82
50	0.91
55	1
60	1.1
65	1.18
70	1.27
75	1.34
80	1.45
85	1.55
90	1.64
95	1.73
100	

DEPTH (FT)

*

E (FT)	APPROXIMATE ASSUMED REDUCTION IN SETTLEMENT
10	50%
25	75%
50	90%

APPENDIX K
MSE WALL EXTERNAL AND GLOBAL STABILITY ANALYSIS

LIST OF ATTACHMENTS

The following are attached and complete this appendix.

Conceptual Retaining Wall Profile
Subsurface Soil Model Profile
External Stability Check Calculations
Global Stability Analysis – Static Condition
Global Stability Analysis – Seismic Condition

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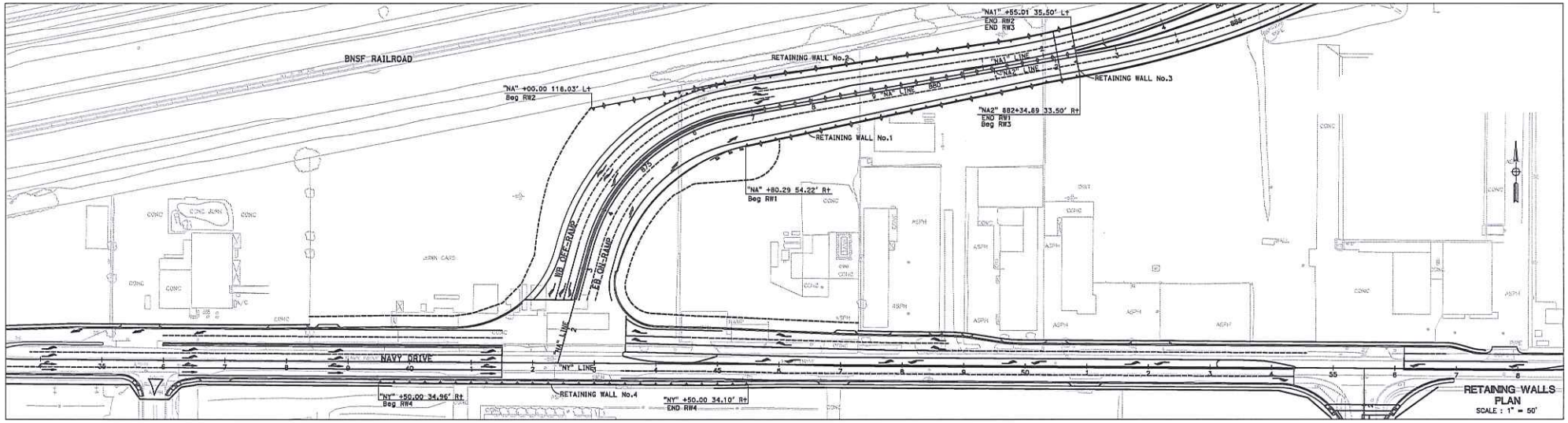
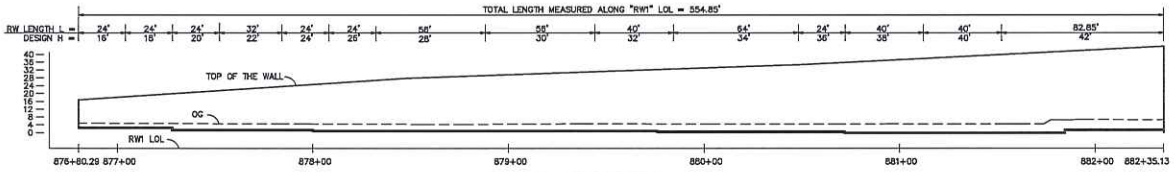
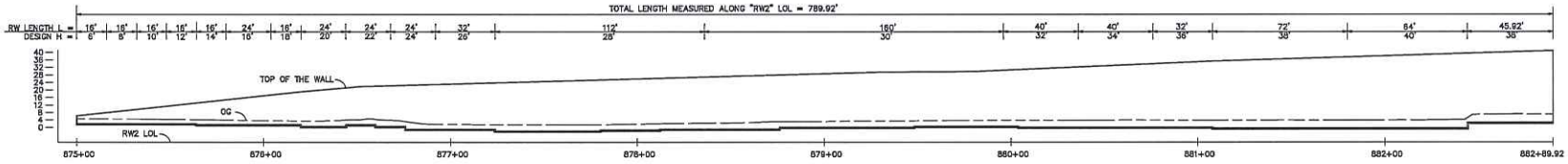
Conceptual Retaining Wall Profile

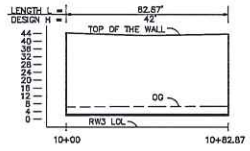




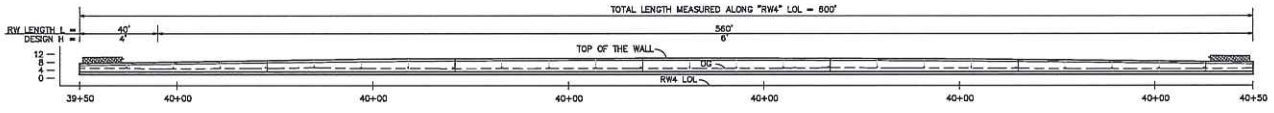
RETAINING WALL 1



RETAINING WALL 2



RETAINING WALL 3



RETAINING WALL 4

RETAINING WALLS PROFILE
SCALE : 1" = 20'

Subsurface Soil Model Profile

Prepared By: HP Date: 6/9/2011
Checked By: CH Date: 6/10/2011

Boring No. :	R-11-013		CPT-11-011	R-11-012		CPT-11-001
	Undrained Shear Strength (psf) Based on					
Depth (feet)	Blow Counts/ft	*Pocket Penetrometer data	CPT-11-011 Data	Blow Counts/ft	*Pocket Penetrometer data	CPT-11-001 Data
3	NT	NT	4,000	15	3,500	3,600
5	13	3,000	4,000	24	NT	2,100
10	NT	NT	7,000	9	2,500	4,900
15	37	3,500	3,200	10	3,500	1,700
20	8	3,500	2,800	NT	3,500	2,400
25	13	NT	5,000	30	NT	
30	16	3,500		9	NT	
35				15	4,000	2,200
40				12	NT	2,600
45			1,700	23	4,000	2,600
50			4,500	23	4,000	3,100
55						4,400
60						
65						4,300
70						8,000
75						4,500
80						4,800
85						5,000
90						2,360
95						N/T
100						N/T
110						N/T
120						

* Highest reading on pocket penetrometer for unconfined compression strength is

NT= Not Tested

= Sand

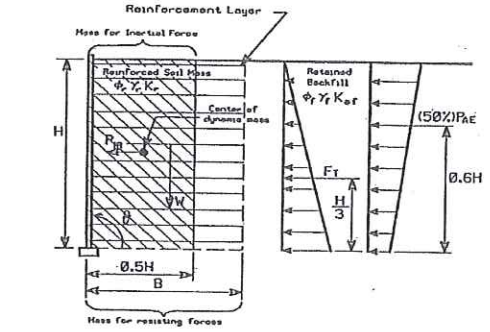
External Stability Check Calculations

MSE WALL EXTERNAL STABILITY CHECK CALCULATIONS

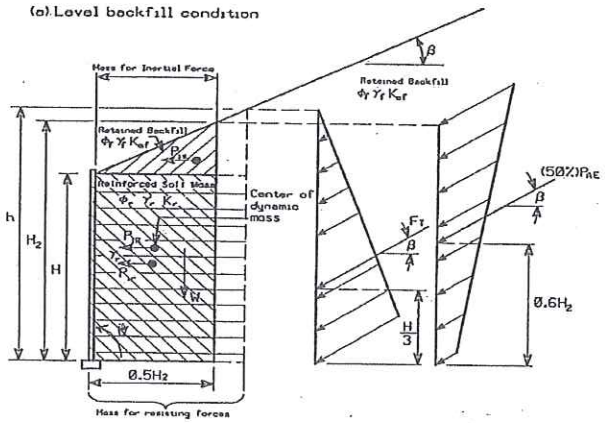
(EZ_MSEWall Version R1.0, 2007)

PROJECT INFORMATION	
Project Name	Crosstown Freeway, Retaining Wall 1, Stn. 877+00
Project No.	111904.G01
Location	Stockton, CA

INPUT PARAMETERS		
WALL DIMENSIONS		
Design Wall Height, H	16.00 ft	Maximum wall height
Wall Face Slope Angle, θ	90.00 degrees	Vertical wall =90 degrees
Wall Friction Angle, δ	0.00 degrees	Enter 0 for vertical walls
Backfill Slope Angle, β	0.00 degrees	
Wall Embedment Depth, D	2.00 ft	Minimum is 2 feet or 0.1H whichever is greater
Reinforcement Length, L	11.20 ft	Minimum is 70% of H or 8, whichever is greater
LOADS		
Traffic Surcharge, q	240.00 psf	Equiv. to 2-ft of overburden soil pressure
Horizontal Seismic Coefficient, k_h	0.16	kh = 1/2 PGA for Mononobe-Okabe Pressure
Vertical Seismic Coefficient, k_v	0.10	
FOUNDATION SOIL		
Friction Angle, ϕ_b	30.00 degrees	For standard design use 30 deg. (BDS 5.9.2)
Cohesion, c_b	0.00 psf	
Total Unit Weight, γ_b	110.00 pcf	
REINFORCED SOIL MASS		
Friction Angle, ϕ_r	30.00 degrees	To be specified as a minimum
Cohesion, c_r	0.00 psf	Imported material
Total Unit Weight, γ_r	120.00 pcf	
RETAINED BACKFILL		
Friction Angle, ϕ_f	34.00 degrees	For standard design use 34 deg. (BDS 5.9.2)
Cohesion, c_f	0.00 psf	Imported structural backfill
Total Unit Weight, γ_f	120.00 pcf	



(a) Level backfill condition



(b) Sloping backfill condition

References:

1. **Federal Highway Administration (FHWA), "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design & Construction Guidelines," Publication No. FHWA-NHI-00-043, March 2001.**
2. **Caltrans Memo to Designers 5-8, "Mechanically Stabilized Embankment," January 1991.**
3. **Caltrans Bridge Design Aids, "Mechanically Stabilized Embankment," pp. 3-8.1 - 3-8.10, April 2002.**

MSE WALL EXTERNAL STABILITY CHECK CALCULATIONS

(EZ_MSEWall Version R1.0, 2007)

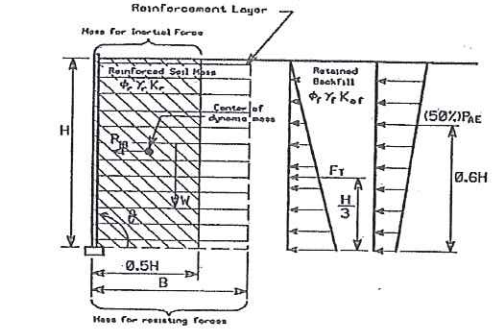
PROJECT INFORMATION		
Project Name	Crosstown Freeway, Retaining Wall 1, Stn. 877+00	
Project No.	111904.G01	
Wall Description	Stockton, CA	
INPUT PARAMETERS		
WALL DIMENSIONS:		
Design Height, H	16.00 ft	Maximum wall height
Wall Face Slope Angle, θ	90.00 degrees	Vertical wall =90 degrees
Wall Friction Angle, δ	0.00	Enter 0 for vertical walls
Backfill Slope Angle, β	0.00 degrees	
Wall Embedment Depth, D	2.00 ft	Minimum is 2 feet or 0.1H whichever is greater
Reinforcement Length, L	11.20 ft	Minimum is 70% of H or 8, whichever is greater
LOADS:		
Traffic Surcharge, q	240.00 psf	Equiv. to 2-ft of overburden soil pressure
Horizontal Seismic Coefficient, k_h	0.16	kh = 1/2 PGA for Mononobe-Okabe Pressure
Vertical Seismic Coefficient, k_v	0.10	
FOUNDATION SOIL:		
Friction Angle, ϕ_b	30.00 degrees	For standard design use 30 deg. (BDS 5.9.2)
Cohesion, c_b	0.00 psf	
Total Unit Weight, γ_b	110.00 pcf	
REINFORCED SOIL MASS:		
Friction Angle, ϕ_r	30.00 degrees	To be specified as a minimum
Cohesion, c_r	0.00 psf	Imported material
Total Unit Weight, γ_r	120.00 pcf	
RETAINED BACKFILL:		
Friction Angle, ϕ_f	34.00 degrees	For standard design use 34 deg. (BDS 5.9.2)
Cohesion, c_f	0.00 psf	Imported structural backfill
Total Unit Weight, γ_f	120.00 pcf	
CALCULATED PARAMETERS		
Static (Active) Earth Pressure Coefficient, K_a	0.283	Based on Rankine Theory (BDS C5.5.5.3)
Dynamic (Active) Earth Pressure Coefficient, K_{ae}	0.394	From Mononobe-Okabe Equation (Kramer p478-479)
Effective Wall Height for Static Active Thrust Force, H_1	16.00 ft	$H_1 = H + L \tan\beta$
Effective Wall Height for Dynamic Active Thrust Force, H_2	16.00 ft	$H_2 = H + (0.5\ H \tan\beta)/(1 - 0.5 \tan\beta)$ (FHWA-NHI-00-043 p.100)
Static Active Thrust Force From Retained Fill, F_T	4.34 kips/ft of wall	$F_T = 1/2\ \gamma_f K_a\ (H_1)^2$
Dynamic Active Thrust Force From Retained Fill, P_{ae}	1.70 kips/ft of wall	$P_{ae} = 1/2\ \gamma_f\ (K_{ae} - K_a)\ (H_2)^2$ (FHWA-NHI-00-043 p.100)
Horizontal Component of Static Active Thrust Force, F_1	4.34 kips/ft of wall	$F_1 = F_T \cos\beta$
Horizontal Force From Traffic Surcharge, F_2	1.09 kips/ft of wall	$F_2 = q\ H\ K_a$
Horizontal Inertial Force From Reinforced Soil Mass, P_{ir}	2.46 kips/ft of wall	$P_{ir} = 0.5\ k_h\ \gamma_f\ H_2\ H$ (FHWA-NHI-00-043 p.100)
Horizontal Inertial Force From Sloping Soil Surcharge, P_{is}	0.00 kips/ft of wall	$P_{is} = 0.125\ k_h\ \gamma_f\ (H_2)^2 \tan\beta$
Weight of Reinforced Soil Mass, V_1	21.50 kips/ft of wall	$V_1 = \gamma_r\ H\ L$
Weight of Sloping Backfill, V_2	0.00 kips/ft of wall	$V_2 = 1/2\ \gamma_f\ L\ (L \tan\beta)$
Sum of Resisting Moments, M_R	120.42 kip-ft/ft of wall	$M_R = V_1\ (L/2) + V_2\ (1/2\ L + 1/6\ L)$
Sum of Overturning Moments (Static), M_{OS}	31.85 kip-ft/ft of wall	$M_{OS} = F_1\ (H1/3) + F_2\ (H1/2)$
Sum of Overturning Moments (Static + Dynamic), M_{OD}	67.86 kip-ft/ft of wall	$M_{OD} = Mos + P_{ir}\ (H/2) + P_{is}\ [H + 1/3\ (H_2 - H)] + P_{ae} \cos\beta\ (0.6\ H_2)$
Sum of Vertical Forces, V_T	21.50 kips/ft of wall	$V_T = V_1 + V_2 + F_T \sin\beta$
Sum of Horizontal Resisting Forces, P_R	12.42 kips/ft of wall	$P_R = V_T \tan[\min(\phi_r, \phi_b)]$
Sum of Horizontal Driving Forces (Static), P_{DS}	5.43 kips/ft of wall	$P_{DS} = F_1 + F_2$
Sum of Horizontal Driving Forces (Static + Dynamic), P_{DD}	9.59 kips/ft of wall	$P_{DD} = P_{DS} + P_{ir} + P_{is} + P_{ae}$
CHECK SLIDING FAILURE:		
Static Sliding Factor of Safety, $FS_{(static\ sliding)} = P_R/P_{DS}$	2.29 ≥ 1.5	**OK**
Dynamic Sliding Factor of Safety, $FS_{(dynamic\ sliding)} = P_R/P_{DD}$	1.29 ≥ 1.125	**OK**
CHECK OVERTURNING FAILURE:		
Static Overturning Factor of Safety, $FS_{(static\ overturning)} = M_R/M_{OS}$	3.78 ≥ 2.0	**OK**
Dynamic Overturning Factor of Safety, $FS_{(dynamic\ overturning)} = M_R/M_{OD}$	1.77 ≥ 1.5	**OK**
Static Eccentricity, $e_s = [L/2 - (M_R - M_{OS})/V_T]$	1.48 ft < L/6	**OK**
Dynamic Eccentricity, $e_d = [L/2 - (M_R - M_{OD})/V_T]$	3.16 ft < L/3	**OK**
CHECK BEARING FAILURE:		
Bearing Capacity Factor, Nc	30.14	Based on Caltrans DBS Table 4.4.7.1A (2003)
Bearing Capacity Factor, Nγ	22.40	Based on Caltrans DBS Table 4.4.7.1A (2003)
Max. Applied Bearing Pressure at the Base of the Wall, σ_v	2.61 ksf	$\sigma_v = V_T / (L - 2\ e_s)$
Ultimate Bearing Capacity, q_{ult}	10.15 ksf	$q_{ult} = c_b\ N_c + 0.5\ \gamma_b\ N_\gamma\ (L - 2\ e_s)$ (Embedment is neglected)
Bearing Capacity Factor of Safety, $FS_{(bearing\ capacity)}$	3.89 ≥ 2.0	**OK**

MSE WALL EXTERNAL STABILITY CHECK CALCULATIONS

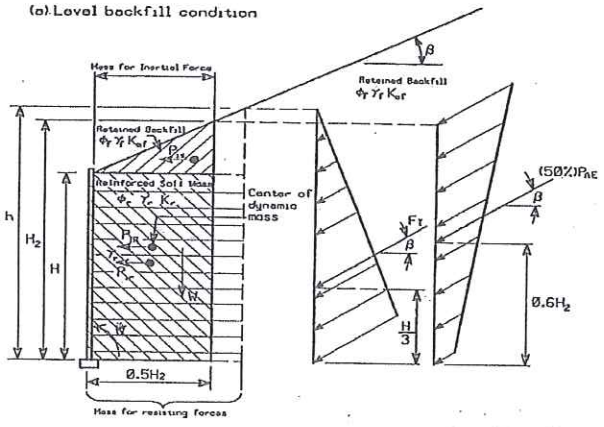
(EZ_MSEWall Version R1.0, 2007)

PROJECT INFORMATION	
Project Name	Crosstown Freeway, Retaining Wall 1, Stn. 878+50
Project No.	111904.G01
Location	Stockton, CA

INPUT PARAMETERS		
WALL DIMENSIONS		
Design Wall Height, H	28.00 ft	Maximum wall height
Wall Face Slope Angle, θ	90.00 degrees	Vertical wall =90 degrees
Wall Friction Angle, δ	0.00 degrees	Enter 0 for vertical walls
Backfill Slope Angle, β	0.00 degrees	
Wall Embedment Depth, D	2.80 ft	Minimum is 2 feet or 0.1H whichever is greater
Reinforcement Length, L	19.60 ft	Minimum is 70% of H or 8, whichever is greater
LOADS		
Traffic Surcharge, q	240.00 psf	Equiv. to 2-ft of overburden soil pressure
Horizontal Seismic Coefficient, k_h	0.16	kh = 1/2 PGA for Mononobe-Okabe Pressure
Vertical Seismic Coefficient, k_v	0.10	
FOUNDATION SOIL		
Friction Angle, ϕ_b	30.00 degrees	For standard design use 30 deg. (BDS 5.9.2)
Cohesion, c_b	0.00 psf	
Total Unit Weight, γ_b	110.00 pcf	
REINFORCED SOIL MASS		
Friction Angle, ϕ_r	30.00 degrees	To be specified as a minimum
Cohesion, c_r	0.00 psf	Imported material
Total Unit Weight, γ_r	120.00 pcf	
RETAINED BACKFILL		
Friction Angle, ϕ_f	34.00 degrees	For standard design use 34 deg. (BDS 5.9.2)
Cohesion, c_f	0.00 psf	Imported structural backfill
Total Unit Weight, γ_f	120.00 pcf	



(a) Level backfill condition



(b) Sloping backfill condition

References:

1. **Federal Highway Administration (FHWA), "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design & Construction Guidelines," Publication No. FHWA-NHI-00-043, March 2001.**
2. **Caltrans Memo to Designers 5-8, "Mechanically Stabilized Embankment," January 1991.**
3. **Caltrans Bridge Design Aids, "Mechanically Stabilized Embankment," pp. 3-8.1 - 3-8.10, April 2002.**

MSE WALL EXTERNAL STABILITY CHECK CALCULATIONS

(EZ_MSEWall Version R1.0, 2007)

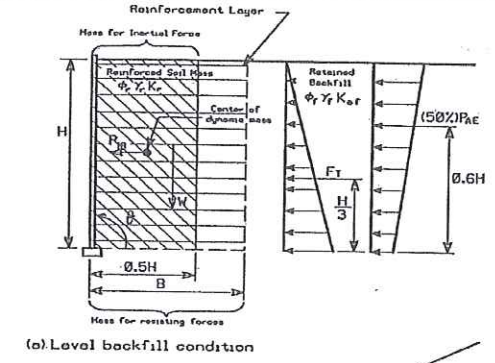
PROJECT INFORMATION		
Project Name	Crosstown Freeway, Retaining Wall 1, Stn. 878+50	
Project No.	111904.G01	
Wall Description	Stockton, CA	
INPUT PARAMETERS		
WALL DIMENSIONS:		
Design Height, H	28.00 ft	Maximum wall height
Wall Face Slope Angle, θ	90.00 degrees	Vertical wall =90 degrees
Wall Friction Angle, δ	0.00	Enter 0 for vertical walls
Backfill Slope Angle, β	0.00 degrees	
Wall Embedment Depth, D	2.80 ft	Minimum is 2 feet or 0.1H whichever is greater
Reinforcement Length, L	19.60 ft	Minimum is 70% of H or 8, whichever is greater
LOADS:		
Traffic Surcharge, q	240.00 psf	Equiv. to 2-ft of overburden soil pressure
Horizontal Seismic Coefficient, k_h	0.16	kh = 1/2 PGA for Mononobe-Okabe Pressure
Vertical Seismic Coefficient, k_v	0.10	
FOUNDATION SOIL:		
Friction Angle, ϕ_b	30.00 degrees	For standard design use 30 deg. (BDS 5.9.2)
Cohesion, c_b	0.00 psf	
Total Unit Weight, γ_b	110.00 pcf	
REINFORCED SOIL MASS:		
Friction Angle, ϕ_r	30.00 degrees	To be specified as a minimum
Cohesion, c_r	0.00 psf	Imported material
Total Unit Weight, γ_r	120.00 pcf	
RETAINED BACKFILL:		
Friction Angle, ϕ_f	34.00 degrees	For standard design use 34 deg. (BDS 5.9.2)
Cohesion, c_f	0.00 psf	Imported structural backfill
Total Unit Weight, γ_f	120.00 pcf	
CALCULATED PARAMETERS		
Static (Active) Earth Pressure Coefficient, K_a	0.283	Based on Rankine Theory (BDS C5.5.5.3)
Dynamic (Active) Earth Pressure Coefficient, K_{ae}	0.394	From Mononobe-Okabe Equation (Kramer p478-479)
Effective Wall Height for Static Active Thrust Force, H_1	28.00 ft	$H_1 = H + L \tan\beta$
Effective Wall Height for Dynamic Active Thrust Force, H_2	28.00 ft	$H_2 = H + (0.5\ H \tan\beta)/(1 - 0.5 \tan\beta)$ (FHWA-NHI-00-043 p.100)
Static Active Thrust Force From Retained Fill, F_T	13.30 kips/ft of wall	$F_T = 1/2\ \gamma_f K_a\ (H_1)^2$
Dynamic Active Thrust Force From Retained Fill, P_{ae}	5.22 kips/ft of wall	$P_{ae} = 1/2\ \gamma_f\ (K_{ae} - K_a)\ (H_2)^2$ (FHWA-NHI-00-043 p.100)
Horizontal Component of Static Active Thrust Force, F_1	13.30 kips/ft of wall	$F_1 = F_T \cos\beta$
Horizontal Force From Traffic Surcharge, F_2	1.90 kips/ft of wall	$F_2 = q\ H\ K_a$
Horizontal Inertial Force From Reinforced Soil Mass, P_{ir}	7.53 kips/ft of wall	$P_{ir} = 0.5\ k_h\ \gamma_f\ H_2\ H$ (FHWA-NHI-00-043 p.100)
Horizontal Inertial Force From Sloping Soil Surcharge, P_{is}	0.00 kips/ft of wall	$P_{is} = 0.125\ k_h\ \gamma_f\ (H_2)^2 \tan\beta$
Weight of Reinforced Soil Mass, V_1	65.86 kips/ft of wall	$V_1 = \gamma_r\ H\ L$
Weight of Sloping Backfill, V_2	0.00 kips/ft of wall	$V_2 = 1/2\ \gamma_f\ L\ (L \tan\beta)$
Sum of Resisting Moments, M_R	645.39 kip-ft/ft of wall	$M_R = V_1\ (L/2) + V_2\ (1/2\ L+1/6\ L)$
Sum of Overturning Moments (Static), M_{OS}	150.72 kip-ft/ft of wall	$M_{OS} = F_1\ (H1/3) + F_2\ (H1/2)$
Sum of Overturning Moments (Static + Dynamic), M_{OD}	343.73 kip-ft/ft of wall	$M_{OD} = Mos + P_{ir}\ (H/2) + P_{is}\ [H+ 1/3\ (H_2-H)] + P_{ae} \cos\beta\ (0.6\ H_2)$
Sum of Vertical Forces, V_T	65.86 kips/ft of wall	$V_T = V_1 + V_2 + F_T \sin\beta$
Sum of Horizontal Resisting Forces, P_R	38.02 kips/ft of wall	$P_R = V_T\ \tan\ [\min\ (\phi_r, \phi_b)]$
Sum of Horizontal Driving Forces (Static), P_{DS}	15.20 kips/ft of wall	$P_{DS} = F_1 + F_2$
Sum of Horizontal Driving Forces (Static + Dynamic), P_{DD}	27.94 kips/ft of wall	$P_{DD} = P_{DS} + P_{ir} + P_{is} + P_{ae}$
CHECK SLIDING FAILURE:		
Static Sliding Factor of Safety, $FS_{(static\ sliding)} = P_R/P_{DS}$	2.50 ≥ 1.5	**OK**
Dynamic Sliding Factor of Safety, $FS_{(dynamic\ sliding)} = P_R/P_{DD}$	1.36 ≥ 1.125	**OK**
CHECK OVERTURNING FAILURE:		
Static Overturning Factor of Safety, $FS_{(static\ overturning)} = M_R/M_{OS}$	4.28 ≥ 2.0	**OK**
Dynamic Overturning Factor of Safety, $FS_{(dynamic\ overturning)} = M_R/M_{OD}$	1.88 ≥ 1.5	**OK**
Static Eccentricity, $e_s = [L/2 - (M_R-M_{OS})/V_T]$	2.29 ft < L/6	**OK**
Dynamic Eccentricity, $e_d = [L/2 - (M_R-M_{OD})/V_T]$	5.22 ft < L/3	**OK**
CHECK BEARING FAILURE:		
Bearing Capacity Factor, Nc	30.14	Based on Caltrans DBS Table 4.4.7.1A (2003)
Bearing Capacity Factor, Nγ	22.40	Based on Caltrans DBS Table 4.4.7.1A (2003)
Max. Applied Bearing Pressure at the Base of the Wall, σ_v	4.38 ksf	$\sigma_v = V_T / (L - 2\ e_s)$
Ultimate Bearing Capacity, q_{ult}	18.51 ksf	$q_{ult} = c_b\ N_c + 0.5\ \gamma_b\ N_\gamma\ (L - 2\ e_s)$ (Embedment is neglected)
Bearing Capacity Factor of Safety, $FS_{(bearing\ capacity)}$	4.22 ≥ 2.0	**OK**

MSE WALL EXTERNAL STABILITY CHECK CALCULATIONS

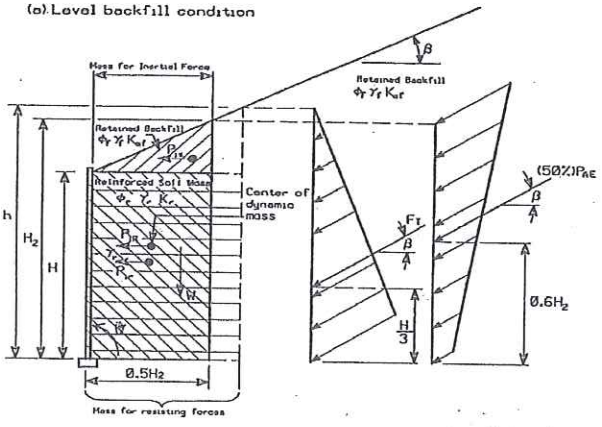
(EZ_MSEWall Version R1.0, 2007)

PROJECT INFORMATION	
Project Name	Crosstown Freeway, Retaining wall 1, Stn. 880+50
Project No.	111904.G01
Location	Stockton, CA

INPUT PARAMETERS		
WALL DIMENSIONS		
Design Wall Height, H	36.00 ft	Maximum wall height
Wall Face Slope Angle, θ	90.00 degrees	Vertical wall =90 degrees
Wall Friction Angle, δ	0.00 degrees	Enter 0 for vertical walls
Backfill Slope Angle, β	0.00 degrees	
Wall Embedment Depth, D	3.60 ft	Minimum is 2 feet or 0.1H whichever is greater
Reinforcement Length, L	25.20 ft	Minimum is 70% of H or 8, whichever is greater
LOADS		
Traffic Surcharge, q	240.00 psf	Equiv. to 2-ft of overburden soil pressure
Horizontal Seismic Coefficient, k_h	0.16	kh = 1/2 PGA for Mononobe-Okabe Pressure
Vertical Seismic Coefficient, k_v	0.10	
FOUNDATION SOIL		
Friction Angle, ϕ_b	30.00 degrees	For standard design use 30 deg. (BDS 5.9.2)
Cohesion, c_b	0.00 psf	
Total Unit Weight, γ_b	110.00 pcf	
REINFORCED SOIL MASS		
Friction Angle, ϕ_r	30.00 degrees	To be specified as a minimum
Cohesion, c_r	0.00 psf	Imported material
Total Unit Weight, γ_r	120.00 pcf	
RETAINED BACKFILL		
Friction Angle, ϕ_f	34.00 degrees	For standard design use 34 deg. (BDS 5.9.2)
Cohesion, c_f	0.00 psf	Imported structural backfill
Total Unit Weight, γ_f	120.00 pcf	



(a) Level backfill condition



(b) Sloping backfill condition

References:

1. **Federal Highway Administration (FHWA), "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design & Construction Guidelines," Publication No. FHWA-NHI-00-043, March 2001.**
2. **Caltrans Memo to Designers 5-8, "Mechanically Stabilized Embankment," January 1991.**
3. **Caltrans Bridge Design Aids, "Mechanically Stabilized Embankment," pp. 3-8.1 - 3-8.10, April 2002.**

MSE WALL EXTERNAL STABILITY CHECK CALCULATIONS

(EZ_MSEWall Version R1.0, 2007)

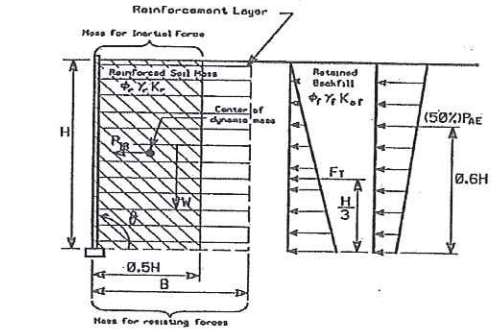
PROJECT INFORMATION		
Project Name	Crosstown Freeway, Retaining wall 1, Stn. 880+50	
Project No.	111904.G01	
Wall Description	Stockton, CA	
INPUT PARAMETERS		
WALL DIMENSIONS:		
Design Height, H	36.00 ft	Maximum wall height
Wall Face Slope Angle, θ	90.00 degrees	Vertical wall =90 degrees
Wall Friction Angle, δ	0.00	Enter 0 for vertical walls
Backfill Slope Angle, β	0.00 degrees	
Wall Embedment Depth, D	3.60 ft	Minimum is 2 feet or 0.1H whichever is greater
Reinforcement Length, L	25.20 ft	Minimum is 70% of H or 8, whichever is greater
LOADS:		
Traffic Surcharge, q	240.00 psf	Equiv. to 2-ft of overburden soil pressure
Horizontal Seismic Coefficient, k_h	0.16	kh = 1/2 PGA for Mononobe-Okabe Pressure
Vertical Seismic Coefficient, k_v	0.10	
FOUNDATION SOIL:		
Friction Angle, ϕ_b	30.00 degrees	For standard design use 30 deg. (BDS 5.9.2)
Cohesion, c_b	0.00 psf	
Total Unit Weight, γ_b	110.00 pcf	
REINFORCED SOIL MASS:		
Friction Angle, ϕ_r	30.00 degrees	To be specified as a minimum
Cohesion, c_r	0.00 psf	Imported material
Total Unit Weight, γ_r	120.00 pcf	
RETAINED BACKFILL:		
Friction Angle, ϕ_f	34.00 degrees	For standard design use 34 deg. (BDS 5.9.2)
Cohesion, c_f	0.00 psf	Imported structural backfill
Total Unit Weight, γ_f	120.00 pcf	
CALCULATED PARAMETERS		
Static (Active) Earth Pressure Coefficient, K_a	0.283	Based on Rankine Theory (BDS C5.5.5.3)
Dynamic (Active) Earth Pressure Coefficient, K_{ae}	0.394	From Mononobe-Okabe Equation (Kramer p478-479)
Effective Wall Height for Static Active Thrust Force, H_1	36.00 ft	$H_1 = H + L \tan \beta$
Effective Wall Height for Dynamic Active Thrust Force, H_2	36.00 ft	$H_2 = H + (0.5\ H \tan \beta)/(1 - 0.5 \tan \beta)$ (FHWA-NHI-00-043 p.100)
Static Active Thrust Force From Retained Fill, F_T	21.98 kips/ft of wall	$F_T = 1/2\ \gamma_f K_a\ (H_1)^2$
Dynamic Active Thrust Force From Retained Fill, P_{ae}	8.62 kips/ft of wall	$P_{ae} = 1/2\ \gamma_f\ (K_{ae} - K_a)\ (H_2)^2$ (FHWA-NHI-00-043 p.100)
Horizontal Component of Static Active Thrust Force, F_1	21.98 kips/ft of wall	$F_1 = F_T \cos \beta$
Horizontal Force From Traffic Surcharge, F_2	2.44 kips/ft of wall	$F_2 = q\ H\ K_a$
Horizontal Inertial Force From Reinforced Soil Mass, P_{ir}	12.44 kips/ft of wall	$P_{ir} = 0.5\ k_h\ \gamma_f\ H_2\ H$ (FHWA-NHI-00-043 p.100)
Horizontal Inertial Force From Sloping Soil Surcharge, P_{is}	0.00 kips/ft of wall	$P_{is} = 0.125\ k_h\ \gamma_f\ (H_2)^2 \tan \beta$
Weight of Reinforced Soil Mass, V_1	108.86 kips/ft of wall	$V_1 = \gamma_r\ H\ L$
Weight of Sloping Backfill, V_2	0.00 kips/ft of wall	$V_2 = 1/2\ \gamma_f\ L\ (L \tan \beta)$
Sum of Resisting Moments, M_R	1,371.69 kip-ft/ft of wall	$M_R = V_1\ (L/2) + V_2\ (1/2\ L+1/6\ L)$
Sum of Overturning Moments (Static), M_{OS}	307.77 kip-ft/ft of wall	$M_{OS} = F_1\ (H1/3) + F_2\ (H1/2)$
Sum of Overturning Moments (Static + Dynamic), M_{OD}	717.99 kip-ft/ft of wall	$M_{OD} = Mos + P_{ir}\ (H/2) + P_{is}\ [H+ 1/3\ (H_2-H)] + P_{ae} \cos \beta\ (0.6\ H_2)$
Sum of Vertical Forces, V_T	108.86 kips/ft of wall	$V_T = V_1 + V_2 + F_T \sin \beta$
Sum of Horizontal Resisting Forces, P_R	62.85 kips/ft of wall	$P_R = V_T\ \tan\ [\min\ (\phi_f, \phi_b)]$
Sum of Horizontal Driving Forces (Static), P_{DS}	24.43 kips/ft of wall	$P_{DS} = F_1 + F_2$
Sum of Horizontal Driving Forces (Static + Dynamic), P_{DD}	45.49 kips/ft of wall	$P_{DD} = P_{DS} + P_{ir} + P_{is} + P_{ae}$
CHECK SLIDING FAILURE:		
Static Sliding Factor of Safety, $FS_{(static\ sliding)} = P_R/P_{DS}$	2.57 ≥ 1.5	**OK**
Dynamic Sliding Factor of Safety, $FS_{(dynamic\ sliding)} = P_R/P_{DD}$	1.38 ≥ 1.125	**OK**
CHECK OVERTURNING FAILURE:		
Static Overturning Factor of Safety, $FS_{(static\ overturning)} = M_R/M_{OS}$	4.46 ≥ 2.0	**OK**
Dynamic Overturning Factor of Safety, $FS_{(dynamic\ overturning)} = M_R/M_{OD}$	1.91 ≥ 1.5	**OK**
Static Eccentricity, $e_s = [L/2 - (M_R-M_{OS})/V_T]$	2.83 ft < L/6	**OK**
Dynamic Eccentricity, $e_d = [L/2 - (M_R-M_{OD})/V_T]$	6.60 ft < L/3	**OK**
CHECK BEARING FAILURE:		
Bearing Capacity Factor, Nc	30.14	Based on Caltrans DBS Table 4.4.7.1A (2003)
Bearing Capacity Factor, Nγ	22.40	Based on Caltrans DBS Table 4.4.7.1A (2003)
Max. Applied Bearing Pressure at the Base of the Wall, σ_v	5.57 ksf	$\sigma_v = V_T / (L - 2\ e_s)$
Ultimate Bearing Capacity, q_{ult}	24.08 ksf	$q_{ult} = c_b\ N_c + 0.5\ \gamma_b\ N_\gamma\ (L - 2\ e_s)$ (Embedment is neglected)
Bearing Capacity Factor of Safety, $FS_{(bearing\ capacity)}$	4.32 ≥ 2.0	**OK**

MSE WALL EXTERNAL STABILITY CHECK CALCULATIONS

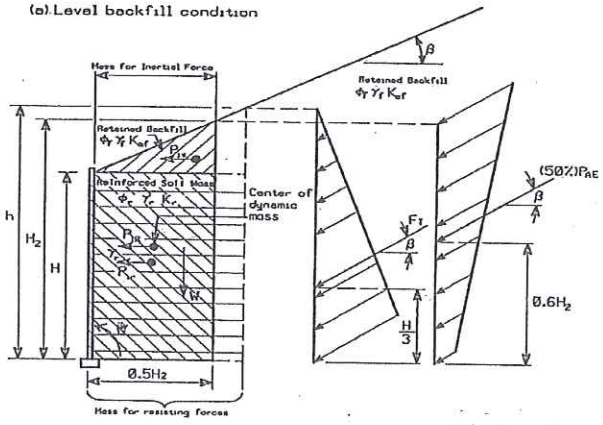
(EZ_MSEWall Version R1.0, 2007)

PROJECT INFORMATION	
Project Name	Crosstown Freeway-Retaining Wall 1, Stn. 882+00
Project No.	111904.G01
Location	Stockton, CA

INPUT PARAMETERS		
WALL DIMENSIONS		
Design Wall Height, H	42.00 ft	Maximum wall height
Wall Face Slope Angle, θ	90.00 degrees	Vertical wall =90 degrees
Wall Friction Angle, δ	0.00 degrees	Enter 0 for vertical walls
Backfill Slope Angle, β	0.00 degrees	
Wall Embedment Depth, D	4.20 ft	Minimum is 2 feet or 0.1H whichever is greater
Reinforcement Length, L	29.40 ft	Minimum is 70% of H or 8, whichever is greater
LOADS		
Traffic Surcharge, q	240.00 psf	Equiv. to 2-ft of overburden soil pressure
Horizontal Seismic Coefficient, k_h	0.16	kh = 1/2 PGA for Mononobe-Okabe Pressure
Vertical Seismic Coefficient, k_v	0.10	
FOUNDATION SOIL		
Friction Angle, ϕ_b	30.00 degrees	For standard design use 30 deg. (BDS 5.9.2)
Cohesion, c_b	0.00 psf	
Total Unit Weight, γ_b	110.00 pcf	
REINFORCED SOIL MASS		
Friction Angle, ϕ_r	30.00 degrees	To be specified as a minimum
Cohesion, c_r	0.00 psf	Imported material
Total Unit Weight, γ_r	120.00 pcf	
RETAINED BACKFILL		
Friction Angle, ϕ_f	34.00 degrees	For standard design use 34 deg. (BDS 5.9.2)
Cohesion, c_f	0.00 psf	Imported structural backfill
Total Unit Weight, γ_f	120.00 pcf	



(a) Level backfill condition



(b) Sloping backfill condition

References:

1. **Federal Highway Administration (FHWA), "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design & Construction Guidelines," Publication No. FHWA-NHI-00-043, March 2001.**
2. **Caltrans Memo to Designers 5-8, "Mechanically Stabilized Embankment," January 1991.**
3. **Caltrans Bridge Design Aids, "Mechanically Stabilized Embankment," pp. 3-8.1 - 3-8.10, April 2002.**

MSE WALL EXTERNAL STABILITY CHECK CALCULATIONS

(EZ_MSEWall Version R1.0, 2007)

PROJECT INFORMATION		
Project Name	Crosstown Freeway-Retaining Wall 1, Stn. 882+00	
Project No.	111904.G01	
Wall Description	Stockton, CA	

INPUT PARAMETERS		
WALL DIMENSIONS:		
Design Height, H	42.00 ft	Maximum wall height
Wall Face Slope Angle, θ	90.00 degrees	Vertical wall =90 degrees
Wall Friction Angle, δ	0.00	Enter 0 for vertical walls
Backfill Slope Angle, β	0.00 degrees	
Wall Embedment Depth, D	4.20 ft	Minimum is 2 feet or 0.1H whichever is greater
Reinforcement Length, L	29.40 ft	Minimum is 70% of H or 8, whichever is greater
LOADS:		
Traffic Surcharge, q	240.00 psf	Equiv. to 2-ft of overburden soil pressure
Horizontal Seismic Coefficient, k_h	0.16	kh = 1/2 PGA for Mononobe-Okabe Pressure
Vertical Seismic Coefficient, k_v	0.10	
FOUNDATION SOIL:		
Friction Angle, ϕ_b	30.00 degrees	For standard design use 30 deg. (BDS 5.9.2)
Cohesion, c_b	0.00 psf	
Total Unit Weight, γ_b	110.00 pcf	
REINFORCED SOIL MASS:		
Friction Angle, ϕ_r	30.00 degrees	To be specified as a minimum
Cohesion, c_r	0.00 psf	Imported material
Total Unit Weight, γ_r	120.00 pcf	
RETAINED BACKFILL:		
Friction Angle, ϕ_f	34.00 degrees	For standard design use 34 deg. (BDS 5.9.2)
Cohesion, c_f	0.00 psf	Imported structural backfill
Total Unit Weight, γ_f	120.00 pcf	

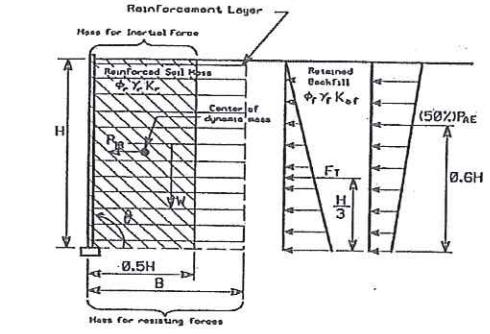
CALCULATED PARAMETERS		
Static (Active) Earth Pressure Coefficient, K_a	0.283	Based on Rankine Theory (BDS C5.5.5.3)
Dynamic (Active) Earth Pressure Coefficient, K_{ae}	0.394	From Mononobe-Okabe Equation (Kramer p478-479)
Effective Wall Height for Static Active Thrust Force, H_1	42.00 ft	$H_1 = H + L \tan \beta$
Effective Wall Height for Dynamic Active Thrust Force, H_2	42.00 ft	$H_2 = H + (0.5\ H \tan \beta)/(1 - 0.5 \tan \beta)$ (FHWA-NHI-00-043 p.100)
Static Active Thrust Force From Retained Fill, F_T	29.92 kips/ft of wall	$F_T = 1/2\ \gamma_f K_a\ (H_1)^2$
Dynamic Active Thrust Force From Retained Fill, P_{ae}	11.74 kips/ft of wall	$P_{ae} = 1/2\ \gamma_f\ (K_{ae} - K_a)\ (H_2)^2$ (FHWA-NHI-00-043 p.100)
Horizontal Component of Static Active Thrust Force, F_1	29.92 kips/ft of wall	$F_1 = F_T \cos \beta$
Horizontal Force From Traffic Surcharge, F_2	2.85 kips/ft of wall	$F_2 = q\ H\ K_a$
Horizontal Inertial Force From Reinforced Soil Mass, P_{ir}	16.93 kips/ft of wall	$P_{ir} = 0.5\ k_h\ \gamma_f\ H_2\ H$ (FHWA-NHI-00-043 p.100)
Horizontal Inertial Force From Sloping Soil Surcharge, P_{is}	0.00 kips/ft of wall	$P_{is} = 0.125\ k_h\ \gamma_f\ (H_2)^2 \tan \beta$
Weight of Reinforced Soil Mass, V_1	148.18 kips/ft of wall	$V_1 = \gamma_r\ H\ L$
Weight of Sloping Backfill, V_2	0.00 kips/ft of wall	$V_2 = 1/2\ \gamma_f\ L\ (L \tan \beta)$
Sum of Resisting Moments, M_R	2,178.19 kip-ft/ft of wall	$M_R = V_1\ (L/2) + V_2\ (1/2\ L+1/6\ L)$
Sum of Overturning Moments (Static), M_{OS}	478.76 kip-ft/ft of wall	$M_{OS} = F_1\ (H1/3) + F_2\ (H1/2)$
Sum of Overturning Moments (Static + Dynamic), M_{OD}	1,130.16 kip-ft/ft of wall	$M_{OD} = Mos + P_{ir}\ (H/2) + P_{is}\ [H+ 1/3\ (H_2-H)] + P_{ae} \cos \beta\ (0.6\ H_2)$
Sum of Vertical Forces, V_T	148.18 kips/ft of wall	$V_T = V_1 + V_2 + F_T \sin \beta$
Sum of Horizontal Resisting Forces, P_R	85.55 kips/ft of wall	$P_R = V_T\ \tan [\min\ (\phi_r, \phi_b)]$
Sum of Horizontal Driving Forces (Static), P_{DS}	32.77 kips/ft of wall	$P_{DS} = F_1 + F_2$
Sum of Horizontal Driving Forces (Static + Dynamic), P_{DD}	61.44 kips/ft of wall	$P_{DD} = P_{DS} + P_{ir} + P_{is} + P_{ae}$
CHECK SLIDING FAILURE:		
Static Sliding Factor of Safety, $FS_{(static\ sliding)} = P_R/P_{DS}$	2.61 ≥ 1.5	**OK**
Dynamic Sliding Factor of Safety, $FS_{(dynamic\ sliding)} = P_R/P_{DD}$	1.39 ≥ 1.125	**OK**
CHECK OVERTURNING FAILURE:		
Static Overturning Factor of Safety, $FS_{(static\ overturning)} = M_R/M_{OS}$	4.55 ≥ 2.0	**OK**
Dynamic Overturning Factor of Safety, $FS_{(dynamic\ overturning)} = M_R/M_{OD}$	1.93 ≥ 1.5	**OK**
Static Eccentricity, $e_s = [L/2 - (M_R-M_{OS})/V_T]$	3.23 ft < L/6	**OK**
Dynamic Eccentricity, $e_d = [L/2 - (M_R-M_{OD})/V_T]$	7.63 ft < L/3	**OK**
CHECK BEARING FAILURE:		
Bearing Capacity Factor, Nc	30.14	Based on Caltrans DBS Table 4.4.7.1A (2003)
Bearing Capacity Factor, Nγ	22.40	Based on Caltrans DBS Table 4.4.7.1A (2003)
Max. Applied Bearing Pressure at the Base of the Wall, σ_v	6.46 ksf	$\sigma_v = V_T / (L - 2\ e_s)$
Ultimate Bearing Capacity, q_{ult}	28.26 ksf	$q_{ult} = c_b\ N_c + 0.5\ \gamma_b\ N_\gamma\ (L - 2\ e_s)$ (Embedment is neglected)
Bearing Capacity Factor of Safety, $FS_{(bearing\ capacity)}$	4.37 ≥ 2.0	**OK**

MSE WALL EXTERNAL STABILITY CHECK CALCULATIONS

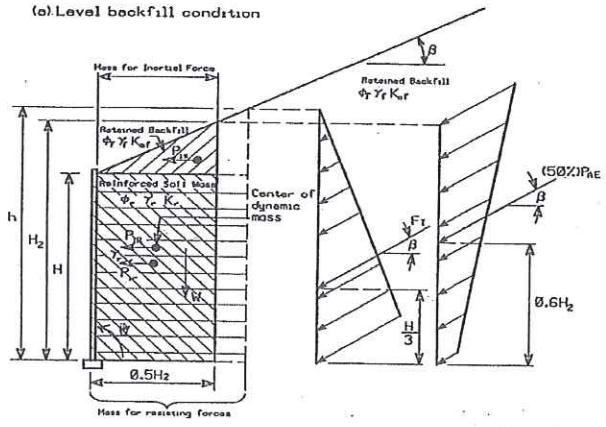
(EZ_MSEWall Version R1.0, 2007)

PROJECT INFORMATION	
Project Name	Crosstown Freeway, Retaining Wall 2, Stn. 876+50
Project No.	111904.G01
Location	Stockton, CA

INPUT PARAMETERS		
WALL DIMENSIONS		
Design Wall Height, H	22.00 ft	Maximum wall height
Wall Face Slope Angle, θ	90.00 degrees	Vertical wall =90 degrees
Wall Friction Angle, δ	0.00 degrees	Enter 0 for vertical walls
Backfill Slope Angle, β	0.00 degrees	
Wall Embedment Depth, D	2.20 ft	Minimum is 2 feet or 0.1H whichever is greater
Reinforcement Length, L	15.40 ft	Minimum is 70% of H or 8, whichever is greater
LOADS		
Traffic Surcharge, q	240.00 psf	Equiv. to 2-ft of overburden soil pressure
Horizontal Seismic Coefficient, k_h	0.16	kh = 1/2 PGA for Mononobe-Okabe Pressure
Vertical Seismic Coefficient, k_v	0.10	
FOUNDATION SOIL		
Friction Angle, ϕ_b	30.00 degrees	For standard design use 30 deg. (BDS 5.9.2)
Cohesion, c_b	0.00 psf	
Total Unit Weight, γ_b	110.00 pcf	
REINFORCED SOIL MASS		
Friction Angle, ϕ_r	30.00 degrees	To be specified as a minimum
Cohesion, c_r	0.00 psf	Imported material
Total Unit Weight, γ_r	120.00 pcf	
RETAINED BACKFILL		
Friction Angle, ϕ_f	34.00 degrees	For standard design use 34 deg. (BDS 5.9.2)
Cohesion, c_f	0.00 psf	Imported structural backfill
Total Unit Weight, γ_f	120.00 pcf	



(a) Level backfill condition



(b) Sloping backfill condition

References:

1. **Federal Highway Administration (FHWA), "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design & Construction Guidelines," Publication No. FHWA-NHI-00-043, March 2001.**
2. **Caltrans Memo to Designers 5-8, "Mechanically Stabilized Embankment," January 1991.**
3. **Caltrans Bridge Design Aids, "Mechanically Stabilized Embankment," pp. 3-8.1 - 3-8.10, April 2002.**

MSE WALL EXTERNAL STABILITY CHECK CALCULATIONS

(EZ_MSEWall Version R1.0, 2007)

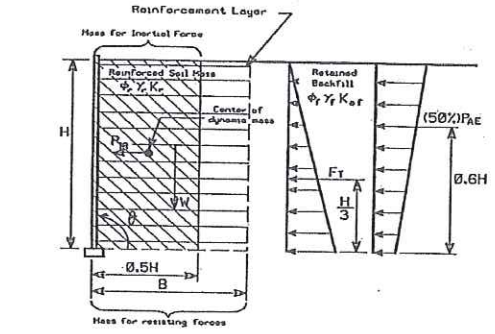
PROJECT INFORMATION		
Project Name	Crosstown Freeway, Retaining Wall 2, Stn. 876+50	
Project No.	111904.G01	
Wall Description	Stockton, CA	
INPUT PARAMETERS		
WALL DIMENSIONS:		
Design Height, H	22.00 ft	Maximum wall height
Wall Face Slope Angle, θ	90.00 degrees	Vertical wall =90 degrees
Wall Friction Angle, δ	0.00	Enter 0 for vertical walls
Backfill Slope Angle, β	0.00 degrees	
Wall Embedment Depth, D	2.20 ft	Minimum is 2 feet or 0.1H whichever is greater
Reinforcement Length, L	15.40 ft	Minimum is 70% of H or 8, whichever is greater
LOADS:		
Traffic Surcharge, q	240.00 psf	Equiv. to 2-ft of overburden soil pressure
Horizontal Seismic Coefficient, k_h	0.16	kh = 1/2 PGA for Mononobe-Okabe Pressure
Vertical Seismic Coefficient, k_v	0.10	
FOUNDATION SOIL:		
Friction Angle, ϕ_b	30.00 degrees	For standard design use 30 deg. (BDS 5.9.2)
Cohesion, c_b	0.00 psf	
Total Unit Weight, γ_b	110.00 pcf	
REINFORCED SOIL MASS:		
Friction Angle, ϕ_r	30.00 degrees	To be specified as a minimum
Cohesion, c_r	0.00 psf	Imported material
Total Unit Weight, γ_r	120.00 pcf	
RETAINED BACKFILL:		
Friction Angle, ϕ_f	34.00 degrees	For standard design use 34 deg. (BDS 5.9.2)
Cohesion, c_f	0.00 psf	Imported structural backfill
Total Unit Weight, γ_f	120.00 pcf	
CALCULATED PARAMETERS		
Static (Active) Earth Pressure Coefficient, K_a	0.283	Based on Rankine Theory (BDS C5.5.5.3)
Dynamic (Active) Earth Pressure Coefficient, K_{ae}	0.394	From Mononobe-Okabe Equation (Kramer p478-479)
Effective Wall Height for Static Active Thrust Force, H_1	22.00 ft	$H_1 = H + L \tan\beta$
Effective Wall Height for Dynamic Active Thrust Force, H_2	22.00 ft	$H_2 = H + (0.5\ H \tan\beta)/(1 - 0.5 \tan\beta)$ (FHWA-NHI-00-043 p.100)
Static Active Thrust Force From Retained Fill, F_T	8.21 kips/ft of wall	$F_T = 1/2\ \gamma_f K_a\ (H_1)^2$
Dynamic Active Thrust Force From Retained Fill, P_{ae}	3.22 kips/ft of wall	$P_{ae} = 1/2\ \gamma_f\ (K_{ae} - K_a)\ (H_2)^2$ (FHWA-NHI-00-043 p.100)
Horizontal Component of Static Active Thrust Force, F_1	8.21 kips/ft of wall	$F_1 = F_T \cos\beta$
Horizontal Force From Traffic Surcharge, F_2	1.49 kips/ft of wall	$F_2 = q\ H\ K_a$
Horizontal Inertial Force From Reinforced Soil Mass, P_{ir}	4.65 kips/ft of wall	$P_{ir} = 0.5\ k_h\ \gamma_f\ H_2\ H$ (FHWA-NHI-00-043 p.100)
Horizontal Inertial Force From Sloping Soil Surcharge, P_{is}	0.00 kips/ft of wall	$P_{is} = 0.125\ k_h\ \gamma_f\ (H_2)^2 \tan\beta$
Weight of Reinforced Soil Mass, V_1	40.66 kips/ft of wall	$V_1 = \gamma_r\ H\ L$
Weight of Sloping Backfill, V_2	0.00 kips/ft of wall	$V_2 = 1/2\ \gamma_f\ L\ (L \tan\beta)$
Sum of Resisting Moments, M_R	313.05 kip-ft/ft of wall	$M_R = V_1\ (L/2) + V_2\ (1/2\ L + 1/6\ L)$
Sum of Overturning Moments (Static), M_{OS}	76.63 kip-ft/ft of wall	$M_{OS} = F_1\ (H1/3) + F_2\ (H1/2)$
Sum of Overturning Moments (Static + Dynamic), M_{OD}	170.25 kip-ft/ft of wall	$M_{OD} = Mos + P_{ir}\ (H/2) + P_{is}\ [H + 1/3\ (H_2 - H)] + P_{ae} \cos\beta\ (0.6\ H_2)$
Sum of Vertical Forces, V_T	40.66 kips/ft of wall	$V_T = V_1 + V_2 + F_T \sin\beta$
Sum of Horizontal Resisting Forces, P_R	23.47 kips/ft of wall	$P_R = V_T \tan[\min(\phi_r, \phi_b)]$
Sum of Horizontal Driving Forces (Static), P_{DS}	9.70 kips/ft of wall	$P_{DS} = F_1 + F_2$
Sum of Horizontal Driving Forces (Static + Dynamic), P_{DD}	17.57 kips/ft of wall	$P_{DD} = P_{DS} + P_{ir} + P_{is} + P_{ae}$
CHECK SLIDING FAILURE:		
Static Sliding Factor of Safety, $FS_{(static\ sliding)} = P_R/P_{DS}$	2.42 ≥ 1.5	**OK**
Dynamic Sliding Factor of Safety, $FS_{(dynamic\ sliding)} = P_R/P_{DD}$	1.34 ≥ 1.125	**OK**
CHECK OVERTURNING FAILURE:		
Static Overturning Factor of Safety, $FS_{(static\ overturning)} = M_R/M_{OS}$	4.09 ≥ 2.0	**OK**
Dynamic Overturning Factor of Safety, $FS_{(dynamic\ overturning)} = M_R/M_{OD}$	1.84 ≥ 1.5	**OK**
Static Eccentricity, $e_s = [L/2 - (M_R - M_{OS})/V_T]$	1.88 ft < L/6	**OK**
Dynamic Eccentricity, $e_d = [L/2 - (M_R - M_{OD})/V_T]$	4.19 ft < L/3	**OK**
CHECK BEARING FAILURE:		
Bearing Capacity Factor, Nc	30.14	Based on Caltrans DBS Table 4.4.7.1A (2003)
Bearing Capacity Factor, Nγ	22.40	Based on Caltrans DBS Table 4.4.7.1A (2003)
Max. Applied Bearing Pressure at the Base of the Wall, σ_v	3.50 ksf	$\sigma_v = V_T / (L - 2\ e_s)$
Ultimate Bearing Capacity, q_{ult}	14.33 ksf	$q_{ult} = c_b\ N_c + 0.5\ \gamma_b\ N_\gamma\ (L - 2\ e_s)$ (Embedment is neglected)
Bearing Capacity Factor of Safety, $FS_{(bearing\ capacity)}$	4.10 ≥ 2.0	**OK**

MSE WALL EXTERNAL STABILITY CHECK CALCULATIONS

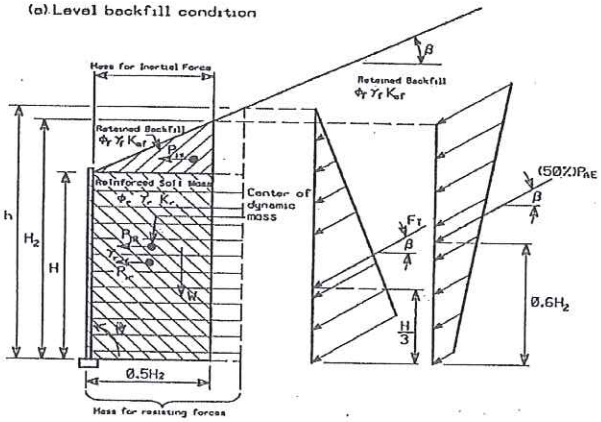
(EZ_MSEWall Version R1.0, 2007)

PROJECT INFORMATION	
Project Name	Crosstown Freeway, Retaining Wall 2, Stn. 878+50
Project No.	111904.G01
Location	Stockton, CA

INPUT PARAMETERS		
WALL DIMENSIONS		
Design Wall Height, H	30.00 ft	Maximum wall height
Wall Face Slope Angle, θ	90.00 degrees	Vertical wall =90 degrees
Wall Friction Angle, δ	0.00 degrees	Enter 0 for vertical walls
Backfill Slope Angle, β	0.00 degrees	
Wall Embedment Depth, D	3.00 ft	Minimum is 2 feet or 0.1H whichever is greater
Reinforcement Length, L	21.00 ft	Minimum is 70% of H or 8, whichever is greater
LOADS		
Traffic Surcharge, q	240.00 psf	Equiv. to 2-ft of overburden soil pressure
Horizontal Seismic Coefficient, k_h	0.16	kh = 1/2 PGA for Mononobe-Okabe Pressure
Vertical Seismic Coefficient, k_v	0.10	
FOUNDATION SOIL		
Friction Angle, ϕ_b	30.00 degrees	For standard design use 30 deg. (BDS 5.9.2)
Cohesion, c_b	0.00 psf	
Total Unit Weight, γ_b	110.00 pcf	
REINFORCED SOIL MASS		
Friction Angle, ϕ_r	30.00 degrees	To be specified as a minimum
Cohesion, c_r	0.00 psf	Imported material
Total Unit Weight, γ_r	120.00 pcf	
RETAINED BACKFILL		
Friction Angle, ϕ_f	34.00 degrees	For standard design use 34 deg. (BDS 5.9.2)
Cohesion, c_f	0.00 psf	Imported structural backfill
Total Unit Weight, γ_f	120.00 pcf	



(a) Level backfill condition



(b) Sloping backfill condition

References:

1. **Federal Highway Administration (FHWA), "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design & Construction Guidelines," Publication No. FHWA-NHI-00-043, March 2001.**
2. **Caltrans Memo to Designers 5-8, "Mechanically Stabilized Embankment," January 1991.**
3. **Caltrans Bridge Design Aids, "Mechanically Stabilized Embankment," pp. 3-8.1 - 3-8.10, April 2002.**

MSE WALL EXTERNAL STABILITY CHECK CALCULATIONS

(EZ_MSEWall Version R1.0, 2007)

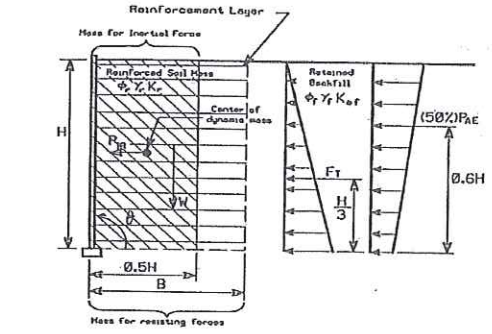
PROJECT INFORMATION		
Project Name	Crosstown Freeway, Retaining Wall 2, Stn. 878+50	
Project No.	111904.G01	
Wall Description	Stockton, CA	
INPUT PARAMETERS		
WALL DIMENSIONS:		
Design Height, H	30.00 ft	Maximum wall height
Wall Face Slope Angle, θ	90.00 degrees	Vertical wall =90 degrees
Wall Friction Angle, δ	0.00	Enter 0 for vertical walls
Backfill Slope Angle, β	0.00 degrees	
Wall Embedment Depth, D	3.00 ft	Minimum is 2 feet or 0.1H whichever is greater
Reinforcement Length, L	21.00 ft	Minimum is 70% of H or 8, whichever is greater
LOADS:		
Traffic Surcharge, q	240.00 psf	Equiv. to 2-ft of overburden soil pressure
Horizontal Seismic Coefficient, k_h	0.16	kh = 1/2 PGA for Mononobe-Okabe Pressure
Vertical Seismic Coefficient, k_v	0.10	
FOUNDATION SOIL:		
Friction Angle, ϕ_b	30.00 degrees	For standard design use 30 deg. (BDS 5.9.2)
Cohesion, c_b	0.00 psf	
Total Unit Weight, γ_b	110.00 pcf	
REINFORCED SOIL MASS:		
Friction Angle, ϕ_r	30.00 degrees	To be specified as a minimum
Cohesion, c_r	0.00 psf	Imported material
Total Unit Weight, γ_r	120.00 pcf	
RETAINED BACKFILL:		
Friction Angle, ϕ_f	34.00 degrees	For standard design use 34 deg. (BDS 5.9.2)
Cohesion, c_f	0.00 psf	Imported structural backfill
Total Unit Weight, γ_f	120.00 pcf	
CALCULATED PARAMETERS		
Static (Active) Earth Pressure Coefficient, K_a	0.283	Based on Rankine Theory (BDS C5.5.5.3)
Dynamic (Active) Earth Pressure Coefficient, K_{ae}	0.394	From Mononobe-Okabe Equation (Kramer p478-479)
Effective Wall Height for Static Active Thrust Force, H_1	30.00 ft	$H_1 = H + L \tan\beta$
Effective Wall Height for Dynamic Active Thrust Force, H_2	30.00 ft	$H_2 = H + (0.5\ H \tan\beta)/(1 - 0.5 \tan\beta)$ (FHWA-NHI-00-043 p.100)
Static Active Thrust Force From Retained Fill, F_T	15.27 kips/ft of wall	$F_T = 1/2\ \gamma_f K_a\ (H_1)^2$
Dynamic Active Thrust Force From Retained Fill, P_{ae}	5.99 kips/ft of wall	$P_{ae} = 1/2\ \gamma_f\ (K_{ae} - K_a)\ (H_2)^2$ (FHWA-NHI-00-043 p.100)
Horizontal Component of Static Active Thrust Force, F_1	15.27 kips/ft of wall	$F_1 = F_T \cos\beta$
Horizontal Force From Traffic Surcharge, F_2	2.04 kips/ft of wall	$F_2 = q\ H\ K_a$
Horizontal Inertial Force From Reinforced Soil Mass, P_{ir}	8.64 kips/ft of wall	$P_{ir} = 0.5\ k_h\ \gamma_f\ H_2\ H$ (FHWA-NHI-00-043 p.100)
Horizontal Inertial Force From Sloping Soil Surcharge, P_{is}	0.00 kips/ft of wall	$P_{is} = 0.125\ k_h\ \gamma_f\ (H_2)^2 \tan\beta$
Weight of Reinforced Soil Mass, V_1	75.60 kips/ft of wall	$V_1 = \gamma_r\ H\ L$
Weight of Sloping Backfill, V_2	0.00 kips/ft of wall	$V_2 = 1/2\ \gamma_f\ L\ (L \tan\beta)$
Sum of Resisting Moments, M_R	793.80 kip-ft/ft of wall	$M_R = V_1\ (L/2) + V_2\ (1/2\ L + 1/6\ L)$
Sum of Overturning Moments (Static), M_{OS}	183.20 kip-ft/ft of wall	$M_{OS} = F_1\ (H1/3) + F_2\ (H1/2)$
Sum of Overturning Moments (Static + Dynamic), M_{OD}	420.59 kip-ft/ft of wall	$M_{OD} = Mos + P_{ir}\ (H/2) + P_{is}\ [H + 1/3\ (H_2 - H)] + P_{ae} \cos\beta\ (0.6\ H_2)$
Sum of Vertical Forces, V_T	75.60 kips/ft of wall	$V_T = V_1 + V_2 + F_T \sin\beta$
Sum of Horizontal Resisting Forces, P_R	43.65 kips/ft of wall	$P_R = V_T \tan[\min(\phi_r, \phi_b)]$
Sum of Horizontal Driving Forces (Static), P_{DS}	17.30 kips/ft of wall	$P_{DS} = F_1 + F_2$
Sum of Horizontal Driving Forces (Static + Dynamic), P_{DD}	31.93 kips/ft of wall	$P_{DD} = P_{DS} + P_{ir} + P_{is} + P_{ae}$
CHECK SLIDING FAILURE:		
Static Sliding Factor of Safety, $FS_{(static\ sliding)} = P_R/P_{DS}$	2.52 ≥ 1.5	**OK**
Dynamic Sliding Factor of Safety, $FS_{(dynamic\ sliding)} = P_R/P_{DD}$	1.37 ≥ 1.125	**OK**
CHECK OVERTURNING FAILURE:		
Static Overturning Factor of Safety, $FS_{(static\ overturning)} = M_R/M_{OS}$	4.33 ≥ 2.0	**OK**
Dynamic Overturning Factor of Safety, $FS_{(dynamic\ overturning)} = M_R/M_{OD}$	1.89 ≥ 1.5	**OK**
Static Eccentricity, $e_s = [L/2 - (M_R - M_{OS})/V_T]$	2.42 ft < L/6	**OK**
Dynamic Eccentricity, $e_d = [L/2 - (M_R - M_{OD})/V_T]$	5.56 ft < L/3	**OK**
CHECK BEARING FAILURE:		
Bearing Capacity Factor, Nc	30.14	Based on Caltrans DBS Table 4.4.7.1A (2003)
Bearing Capacity Factor, Nγ	22.40	Based on Caltrans DBS Table 4.4.7.1A (2003)
Max. Applied Bearing Pressure at the Base of the Wall, σ_v	4.68 ksf	$\sigma_v = V_T / (L - 2\ e_s)$
Ultimate Bearing Capacity, q_{ult}	19.90 ksf	$q_{ult} = c_b\ N_c + 0.5\ \gamma_b\ N_\gamma\ (L - 2\ e_s)$ (Embedment is neglected)
Bearing Capacity Factor of Safety, $FS_{(bearing\ capacity)}$	4.25 ≥ 2.0	**OK**

MSE WALL EXTERNAL STABILITY CHECK CALCULATIONS

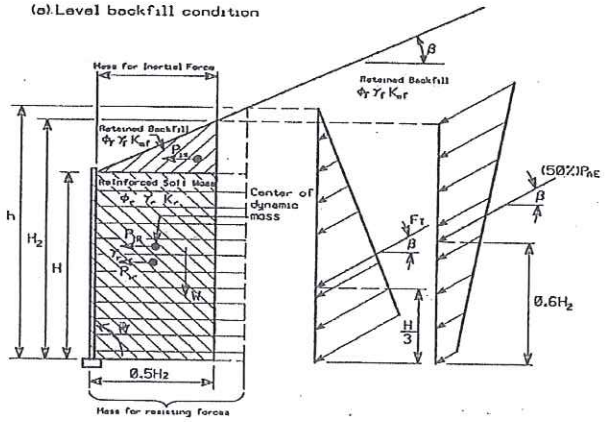
(EZ_MSEWall Version R1.0, 2007)

PROJECT INFORMATION	
Project Name	Crosstown Freeway, Retaining Wall 2, Stn. 880+50
Project No.	111904.G01
Location	Stockton, CA

INPUT PARAMETERS		
WALL DIMENSIONS		
Design Wall Height, H	34.00 ft	Maximum wall height
Wall Face Slope Angle, θ	90.00 degrees	Vertical wall =90 degrees
Wall Friction Angle, δ	0.00 degrees	Enter 0 for vertical walls
Backfill Slope Angle, β	0.00 degrees	
Wall Embedment Depth, D	3.40 ft	Minimum is 2 feet or 0.1H whichever is greater
Reinforcement Length, L	23.80 ft	Minimum is 70% of H or 8, whichever is greater
LOADS		
Traffic Surcharge, q	240.00 psf	Equiv. to 2-ft of overburden soil pressure
Horizontal Seismic Coefficient, k_h	0.16	kh = 1/2 PGA for Mononobe-Okabe Pressure
Vertical Seismic Coefficient, k_v	0.10	
FOUNDATION SOIL		
Friction Angle, ϕ_b	30.00 degrees	For standard design use 30 deg. (BDS 5.9.2)
Cohesion, c_b	0.00 psf	
Total Unit Weight, γ_b	110.00 pcf	
REINFORCED SOIL MASS		
Friction Angle, ϕ_r	30.00 degrees	To be specified as a minimum
Cohesion, c_r	0.00 psf	Imported material
Total Unit Weight, γ_r	120.00 pcf	
RETAINED BACKFILL		
Friction Angle, ϕ_f	34.00 degrees	For standard design use 34 deg. (BDS 5.9.2)
Cohesion, c_f	0.00 psf	Imported structural backfill
Total Unit Weight, γ_f	120.00 pcf	



(a) Level backfill condition



(b) Sloping backfill condition

References:

1. **Federal Highway Administration (FHWA), "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design & Construction Guidelines," Publication No. FHWA-NHI-00-043, March 2001.**
2. **Caltrans Memo to Designers 5-8, "Mechanically Stabilized Embankment," January 1991.**
3. **Caltrans Bridge Design Aids, "Mechanically Stabilized Embankment," pp. 3-8.1 - 3-8.10, April 2002.**

MSE WALL EXTERNAL STABILITY CHECK CALCULATIONS

(EZ_MSEWall Version R1.0, 2007)

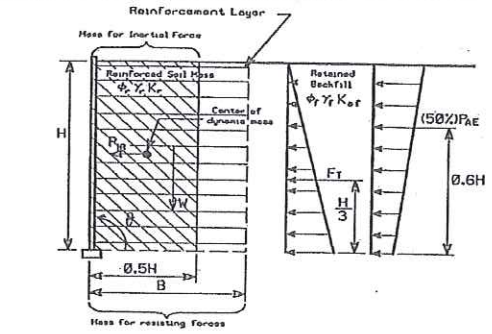
PROJECT INFORMATION		
Project Name	Crosstown Freeway, Retaining Wall 2, Stn. 880+50	
Project No.	111904.G01	
Wall Description	Stockton, CA	
INPUT PARAMETERS		
WALL DIMENSIONS:		
Design Height, H	34.00 ft	Maximum wall height
Wall Face Slope Angle, θ	90.00 degrees	Vertical wall =90 degrees
Wall Friction Angle, δ	0.00	Enter 0 for vertical walls
Backfill Slope Angle, β	0.00 degrees	
Wall Embedment Depth, D	3.40 ft	Minimum is 2 feet or 0.1H whichever is greater
Reinforcement Length, L	23.80 ft	Minimum is 70% of H or 8, whichever is greater
LOADS:		
Traffic Surcharge, q	240.00 psf	Equiv. to 2-ft of overburden soil pressure
Horizontal Seismic Coefficient, k_h	0.16	kh = 1/2 PGA for Mononobe-Okabe Pressure
Vertical Seismic Coefficient, k_v	0.10	
FOUNDATION SOIL:		
Friction Angle, ϕ_b	30.00 degrees	For standard design use 30 deg. (BDS 5.9.2)
Cohesion, c_b	0.00 psf	
Total Unit Weight, γ_b	110.00 pcf	
REINFORCED SOIL MASS:		
Friction Angle, ϕ_r	30.00 degrees	To be specified as a minimum
Cohesion, c_r	0.00 psf	Imported material
Total Unit Weight, γ_r	120.00 pcf	
RETAINED BACKFILL:		
Friction Angle, ϕ_f	34.00 degrees	For standard design use 34 deg. (BDS 5.9.2)
Cohesion, c_f	0.00 psf	Imported structural backfill
Total Unit Weight, γ_f	120.00 pcf	
CALCULATED PARAMETERS		
Static (Active) Earth Pressure Coefficient, K_a	0.283	Based on Rankine Theory (BDS C5.5.5.3)
Dynamic (Active) Earth Pressure Coefficient, K_{ae}	0.394	From Mononobe-Okabe Equation (Kramer p478-479)
Effective Wall Height for Static Active Thrust Force, H_1	34.00 ft	$H_1 = H + L \tan\beta$
Effective Wall Height for Dynamic Active Thrust Force, H_2	34.00 ft	$H_2 = H + (0.5\ H \tan\beta)/(1 - 0.5 \tan\beta)$ (FHWA-NHI-00-043 p.100)
Static Active Thrust Force From Retained Fill, F_T	19.61 kips/ft of wall	$F_T = 1/2\ \gamma_f K_a\ (H_1)^2$
Dynamic Active Thrust Force From Retained Fill, P_{ae}	7.69 kips/ft of wall	$P_{ae} = 1/2\ \gamma_f\ (K_{ae} - K_a)\ (H_2)^2$ (FHWA-NHI-00-043 p.100)
Horizontal Component of Static Active Thrust Force, F_1	19.61 kips/ft of wall	$F_1 = F_T \cos\beta$
Horizontal Force From Traffic Surcharge, F_2	2.31 kips/ft of wall	$F_2 = q\ H\ K_a$
Horizontal Inertial Force From Reinforced Soil Mass, P_{ir}	11.10 kips/ft of wall	$P_{ir} = 0.5\ k_h\ \gamma_f\ H_2\ H$ (FHWA-NHI-00-043 p.100)
Horizontal Inertial Force From Sloping Soil Surcharge, P_{is}	0.00 kips/ft of wall	$P_{is} = 0.125\ k_h\ \gamma_f\ (H_2)^2 \tan\beta$
Weight of Reinforced Soil Mass, V_1	97.10 kips/ft of wall	$V_1 = \gamma_r\ H\ L$
Weight of Sloping Backfill, V_2	0.00 kips/ft of wall	$V_2 = 1/2\ \gamma_f\ L\ (L \tan\beta)$
Sum of Resisting Moments, M_R	1,155.54 kip-ft/ft of wall	$M_R = V_1\ (L/2) + V_2\ (1/2\ L+1/6\ L)$
Sum of Overturning Moments (Static), M_{OS}	261.45 kip-ft/ft of wall	$M_{OS} = F_1\ (H1/3) + F_2\ (H1/2)$
Sum of Overturning Moments (Static + Dynamic), M_{OD}	607.02 kip-ft/ft of wall	$M_{OD} = Mos + P_{ir}\ (H/2) + P_{is}\ [H+ 1/3\ (H_2-H)] + P_{ae} \cos\beta\ (0.6\ H_2)$
Sum of Vertical Forces, V_T	97.10 kips/ft of wall	$V_T = V_1 + V_2 + F_T \sin\beta$
Sum of Horizontal Resisting Forces, P_R	56.06 kips/ft of wall	$P_R = V_T\ \tan\ [\min\ (\phi_r, \phi_b)]$
Sum of Horizontal Driving Forces (Static), P_{DS}	21.92 kips/ft of wall	$P_{DS} = F_1 + F_2$
Sum of Horizontal Driving Forces (Static + Dynamic), P_{DD}	40.71 kips/ft of wall	$P_{DD} = P_{DS} + P_{ir} + P_{is} + P_{ae}$
CHECK SLIDING FAILURE:		
Static Sliding Factor of Safety, $FS_{(static\ sliding)} = P_R/P_{DS}$	2.56 ≥ 1.5	**OK**
Dynamic Sliding Factor of Safety, $FS_{(dynamic\ sliding)} = P_R/P_{DD}$	1.38 ≥ 1.125	**OK**
CHECK OVERTURNING FAILURE:		
Static Overturning Factor of Safety, $FS_{(static\ overturning)} = M_R/M_{OS}$	4.42 ≥ 2.0	**OK**
Dynamic Overturning Factor of Safety, $FS_{(dynamic\ overturning)} = M_R/M_{OD}$	1.90 ≥ 1.5	**OK**
Static Eccentricity, $e_s = [L/2 - (M_R-M_{OS})/V_T]$	2.69 ft < L/6	**OK**
Dynamic Eccentricity, $e_d = [L/2 - (M_R-M_{OD})/V_T]$	6.25 ft < L/3	**OK**
CHECK BEARING FAILURE:		
Bearing Capacity Factor, Nc	30.14	Based on Caltrans DBS Table 4.4.7.1A (2003)
Bearing Capacity Factor, Nγ	22.40	Based on Caltrans DBS Table 4.4.7.1A (2003)
Max. Applied Bearing Pressure at the Base of the Wall, σ_v	5.27 ksf	$\sigma_v = V_T / (L - 2\ e_s)$
Ultimate Bearing Capacity, q_{ult}	22.69 ksf	$q_{ult} = c_b\ N_c + 0.5\ \gamma_b\ N_\gamma\ (L - 2\ e_s)$ (Embedment is neglected)
Bearing Capacity Factor of Safety, $FS_{(bearing\ capacity)}$	4.30 ≥ 2.0	**OK**

MSE WALL EXTERNAL STABILITY CHECK CALCULATIONS

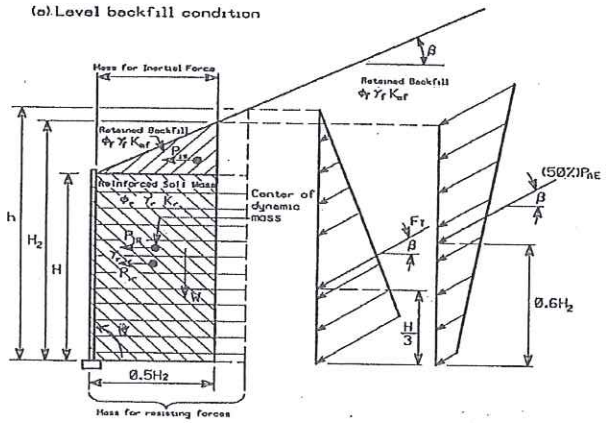
(EZ_MSEWall Version R1.0, 2007)

PROJECT INFORMATION	
Project Name	Crosstown Freeway, Retaining Wall 2, Stn. 882+50
Project No.	111904.G01
Location	Stockton, CA

INPUT PARAMETERS		
WALL DIMENSIONS		
Design Wall Height, H	38.00 ft	Maximum wall height
Wall Face Slope Angle, θ	90.00 degrees	Vertical wall =90 degrees
Wall Friction Angle, δ	0.00 degrees	Enter 0 for vertical walls
Backfill Slope Angle, β	0.00 degrees	
Wall Embedment Depth, D	3.80 ft	Minimum is 2 feet or 0.1H whichever is greater
Reinforcement Length, L	26.60 ft	Minimum is 70% of H or 8, whichever is greater
LOADS		
Traffic Surcharge, q	240.00 psf	Equiv. to 2-ft of overburden soil pressure
Horizontal Seismic Coefficient, k_h	0.16	kh = 1/2 PGA for Mononobe-Okabe Pressure
Vertical Seismic Coefficient, k_v	0.10	
FOUNDATION SOIL		
Friction Angle, ϕ_b	30.00 degrees	For standard design use 30 deg. (BDS 5.9.2)
Cohesion, c_b	0.00 psf	
Total Unit Weight, γ_b	110.00 pcf	
REINFORCED SOIL MASS		
Friction Angle, ϕ_r	30.00 degrees	To be specified as a minimum
Cohesion, c_r	0.00 psf	Imported material
Total Unit Weight, γ_r	120.00 pcf	
RETAINED BACKFILL		
Friction Angle, ϕ_f	34.00 degrees	For standard design use 34 deg. (BDS 5.9.2)
Cohesion, c_f	0.00 psf	Imported structural backfill
Total Unit Weight, γ_f	120.00 pcf	



(a) Level backfill condition



(b) Sloping backfill condition

References:

1. **Federal Highway Administration (FHWA), "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design & Construction Guidelines," Publication No. FHWA-NHI-00-043, March 2001.**
2. **Caltrans Memo to Designers 5-8, "Mechanically Stabilized Embankment," January 1991.**
3. **Caltrans Bridge Design Aids, "Mechanically Stabilized Embankment," pp. 3-8.1 - 3-8.10, April 2002.**

MSE WALL EXTERNAL STABILITY CHECK CALCULATIONS

(EZ_MSEWall Version R1.0, 2007)

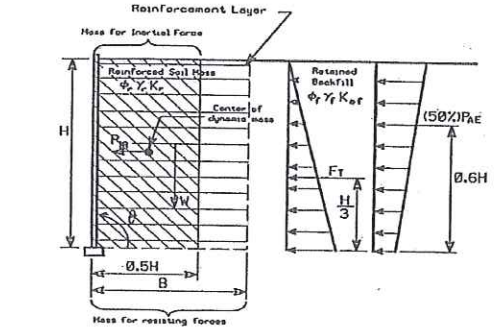
PROJECT INFORMATION		
Project Name	Crosstown Freeway, Retaining Wall 2, Stn. 882+50	
Project No.	111904.G01	
Wall Description	Stockton, CA	
INPUT PARAMETERS		
WALL DIMENSIONS:		
Design Height, H	38.00 ft	Maximum wall height
Wall Face Slope Angle, θ	90.00 degrees	Vertical wall =90 degrees
Wall Friction Angle, δ	0.00	Enter 0 for vertical walls
Backfill Slope Angle, β	0.00 degrees	
Wall Embedment Depth, D	3.80 ft	Minimum is 2 feet or 0.1H whichever is greater
Reinforcement Length, L	26.60 ft	Minimum is 70% of H or 8, whichever is greater
LOADS:		
Traffic Surcharge, q	240.00 psf	Equiv. to 2-ft of overburden soil pressure
Horizontal Seismic Coefficient, k_h	0.16	kh = 1/2 PGA for Mononobe-Okabe Pressure
Vertical Seismic Coefficient, k_v	0.10	
FOUNDATION SOIL:		
Friction Angle, ϕ_b	30.00 degrees	For standard design use 30 deg. (BDS 5.9.2)
Cohesion, c_b	0.00 psf	
Total Unit Weight, γ_b	110.00 pcf	
REINFORCED SOIL MASS:		
Friction Angle, ϕ_r	30.00 degrees	To be specified as a minimum
Cohesion, c_r	0.00 psf	Imported material
Total Unit Weight, γ_r	120.00 pcf	
RETAINED BACKFILL:		
Friction Angle, ϕ_f	34.00 degrees	For standard design use 34 deg. (BDS 5.9.2)
Cohesion, c_f	0.00 psf	Imported structural backfill
Total Unit Weight, γ_f	120.00 pcf	
CALCULATED PARAMETERS		
Static (Active) Earth Pressure Coefficient, K_a	0.283	Based on Rankine Theory (BDS C5.5.5.3)
Dynamic (Active) Earth Pressure Coefficient, K_{ae}	0.394	From Mononobe-Okabe Equation (Kramer p478-479)
Effective Wall Height for Static Active Thrust Force, H_1	38.00 ft	$H_1 = H + L \tan\beta$
Effective Wall Height for Dynamic Active Thrust Force, H_2	38.00 ft	$H_2 = H + (0.5\ H \tan\beta)/(1 - 0.5 \tan\beta)$ (FHWA-NHI-00-043 p.100)
Static Active Thrust Force From Retained Fill, F_T	24.49 kips/ft of wall	$F_T = 1/2\ \gamma_f K_a\ (H_1)^2$
Dynamic Active Thrust Force From Retained Fill, P_{ae}	9.61 kips/ft of wall	$P_{ae} = 1/2\ \gamma_f\ (K_{ae} - K_a)\ (H_2)^2$ (FHWA-NHI-00-043 p.100)
Horizontal Component of Static Active Thrust Force, F_1	24.49 kips/ft of wall	$F_1 = F_T \cos\beta$
Horizontal Force From Traffic Surcharge, F_2	2.58 kips/ft of wall	$F_2 = q\ H\ K_a$
Horizontal Inertial Force From Reinforced Soil Mass, P_{ir}	13.86 kips/ft of wall	$P_{ir} = 0.5\ k_h\ \gamma_f\ H_2\ H$ (FHWA-NHI-00-043 p.100)
Horizontal Inertial Force From Sloping Soil Surcharge, P_{is}	0.00 kips/ft of wall	$P_{is} = 0.125\ k_h\ \gamma_f\ (H_2)^2 \tan\beta$
Weight of Reinforced Soil Mass, V_1	121.30 kips/ft of wall	$V_1 = \gamma_r\ H\ L$
Weight of Sloping Backfill, V_2	0.00 kips/ft of wall	$V_2 = 1/2\ \gamma_f\ L\ (L \tan\beta)$
Sum of Resisting Moments, M_R	1,613.24 kip-ft/ft of wall	$M_R = V_1\ (L/2) + V_2\ (1/2\ L+1/6\ L)$
Sum of Overturning Moments (Static), M_{OS}	359.25 kip-ft/ft of wall	$M_{OS} = F_1\ (H1/3) + F_2\ (H1/2)$
Sum of Overturning Moments (Static + Dynamic), M_{OD}	841.70 kip-ft/ft of wall	$M_{OD} = Mos + P_{ir}\ (H/2) + P_{is}\ [H+ 1/3\ (H_2-H)] + P_{ae} \cos\beta\ (0.6\ H_2)$
Sum of Vertical Forces, V_T	121.30 kips/ft of wall	$V_T = V_1 + V_2 + F_T \sin\beta$
Sum of Horizontal Resisting Forces, P_R	70.03 kips/ft of wall	$P_R = V_T \tan[\min(\phi_r, \phi_b)]$
Sum of Horizontal Driving Forces (Static), P_{DS}	27.07 kips/ft of wall	$P_{DS} = F_1 + F_2$
Sum of Horizontal Driving Forces (Static + Dynamic), P_{DD}	50.54 kips/ft of wall	$P_{DD} = P_{DS} + P_{ir} + P_{is} + P_{ae}$
CHECK SLIDING FAILURE:		
Static Sliding Factor of Safety, $FS_{(static\ sliding)} = P_R/P_{DS}$	2.59 ≥ 1.5	**OK**
Dynamic Sliding Factor of Safety, $FS_{(dynamic\ sliding)} = P_R/P_{DD}$	1.39 ≥ 1.125	**OK**
CHECK OVERTURNING FAILURE:		
Static Overturning Factor of Safety, $FS_{(static\ overturning)} = M_R/M_{OS}$	4.49 ≥ 2.0	**OK**
Dynamic Overturning Factor of Safety, $FS_{(dynamic\ overturning)} = M_R/M_{OD}$	1.92 ≥ 1.5	**OK**
Static Eccentricity, $e_s = [L/2 - (M_R-M_{OS})/V_T]$	2.96 ft < L/6	**OK**
Dynamic Eccentricity, $e_d = [L/2 - (M_R-M_{OD})/V_T]$	6.94 ft < L/3	**OK**
CHECK BEARING FAILURE:		
Bearing Capacity Factor, Nc	30.14	Based on Caltrans DBS Table 4.4.7.1A (2003)
Bearing Capacity Factor, Nγ	22.40	Based on Caltrans DBS Table 4.4.7.1A (2003)
Max. Applied Bearing Pressure at the Base of the Wall, σ_v	5.87 ksf	$\sigma_v = V_T / (L - 2\ e_s)$
Ultimate Bearing Capacity, q_{ult}	25.47 ksf	$q_{ult} = c_b\ N_c + 0.5\ \gamma_b\ N_\gamma\ (L - 2\ e_s)$ (Embedment is neglected)
Bearing Capacity Factor of Safety, $FS_{(bearing\ capacity)}$	4.34 ≥ 2.0	**OK**

MSE WALL EXTERNAL STABILITY CHECK CALCULATIONS

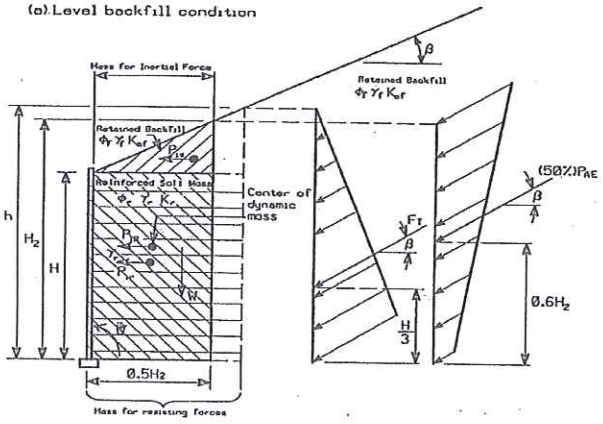
(EZ_MSEWall Version R1.0, 2007)

PROJECT INFORMATION	
Project Name	Crosstown Freeway, Retaining Wall 3, Stn. 10+50
Project No.	111904.G01
Location	Stockton, CA

INPUT PARAMETERS		
WALL DIMENSIONS		
Design Wall Height, H	42.00 ft	Maximum wall height
Wall Face Slope Angle, θ	90.00 degrees	Vertical wall =90 degrees
Wall Friction Angle, δ	0.00 degrees	Enter 0 for vertical walls
Backfill Slope Angle, β	0.00 degrees	
Wall Embedment Depth, D	4.20 ft	Minimum is 2 feet or 0.1H whichever is greater
Reinforcement Length, L	29.40 ft	Minimum is 70% of H or 8, whichever is greater
LOADS		
Traffic Surcharge, q	240.00 psf	Equiv. to 2-ft of overburden soil pressure
Horizontal Seismic Coefficient, k_h	0.16	kh = 1/2 PGA for Mononobe-Okabe Pressure
Vertical Seismic Coefficient, k_v	0.10	
FOUNDATION SOIL		
Friction Angle, ϕ_b	30.00 degrees	For standard design use 30 deg. (BDS 5.9.2)
Cohesion, c_b	0.00 psf	
Total Unit Weight, γ_b	110.00 pcf	
REINFORCED SOIL MASS		
Friction Angle, ϕ_r	30.00 degrees	To be specified as a minimum
Cohesion, c_r	0.00 psf	Imported material
Total Unit Weight, γ_r	120.00 pcf	
RETAINED BACKFILL		
Friction Angle, ϕ_f	34.00 degrees	For standard design use 34 deg. (BDS 5.9.2)
Cohesion, c_f	0.00 psf	Imported structural backfill
Total Unit Weight, γ_f	120.00 pcf	



(a) Level backfill condition



(b) Sloping backfill condition

References:

1. **Federal Highway Administration (FHWA), "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design & Construction Guidelines," Publication No. FHWA-NHI-00-043, March 2001.**
2. **Caltrans Memo to Designers 5-8, "Mechanically Stabilized Embankment," January 1991.**
3. **Caltrans Bridge Design Aids, "Mechanically Stabilized Embankment," pp. 3-8.1 - 3-8.10, April 2002.**

MSE WALL EXTERNAL STABILITY CHECK CALCULATIONS

(EZ_MSEWall Version R1.0, 2007)

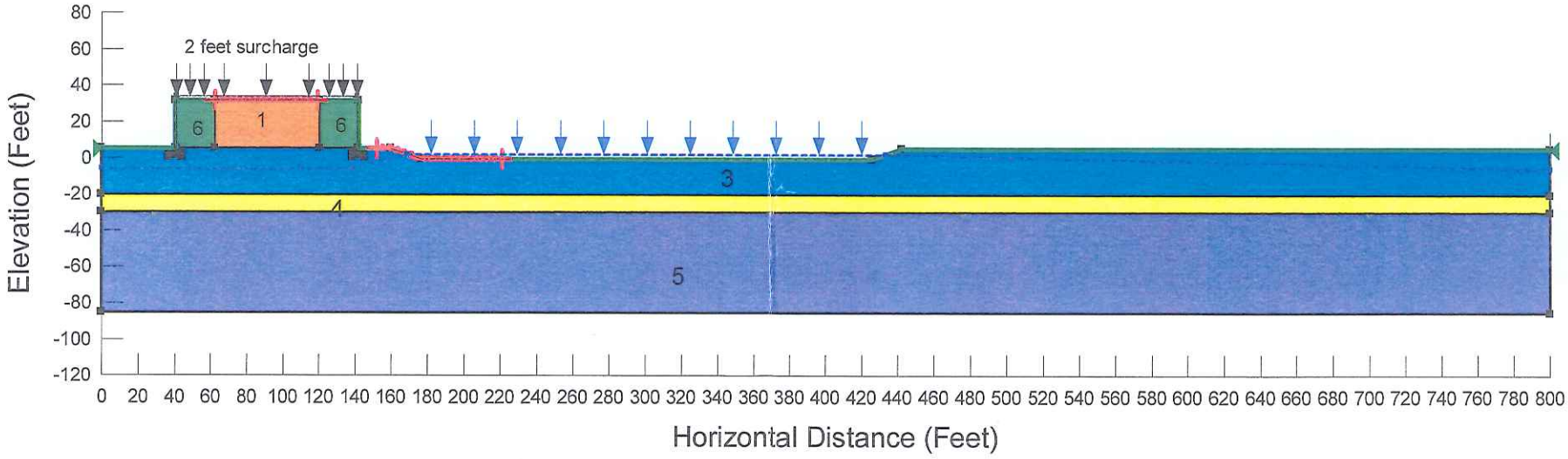
PROJECT INFORMATION		
Project Name	Crosstown Freeway, Retaining Wall 3, Stn. 10+50	
Project No.	111904.G01	
Wall Description	Stockton, CA	
INPUT PARAMETERS		
WALL DIMENSIONS:		
Design Height, H	42.00 ft	Maximum wall height
Wall Face Slope Angle, θ	90.00 degrees	Vertical wall =90 degrees
Wall Friction Angle, δ	0.00	Enter 0 for vertical walls
Backfill Slope Angle, β	0.00 degrees	
Wall Embedment Depth, D	4.20 ft	Minimum is 2 feet or 0.1H whichever is greater
Reinforcement Length, L	29.40 ft	Minimum is 70% of H or 8, whichever is greater
LOADS:		
Traffic Surcharge, q	240.00 psf	Equiv. to 2-ft of overburden soil pressure
Horizontal Seismic Coefficient, k_h	0.16	kh = 1/2 PGA for Mononobe-Okabe Pressure
Vertical Seismic Coefficient, k_v	0.10	
FOUNDATION SOIL:		
Friction Angle, ϕ_b	30.00 degrees	For standard design use 30 deg. (BDS 5.9.2)
Cohesion, c_b	0.00 psf	
Total Unit Weight, γ_b	110.00 pcf	
REINFORCED SOIL MASS:		
Friction Angle, ϕ_r	30.00 degrees	To be specified as a minimum
Cohesion, c_r	0.00 psf	Imported material
Total Unit Weight, γ_r	120.00 pcf	
RETAINED BACKFILL:		
Friction Angle, ϕ_f	34.00 degrees	For standard design use 34 deg. (BDS 5.9.2)
Cohesion, c_f	0.00 psf	Imported structural backfill
Total Unit Weight, γ_f	120.00 pcf	
CALCULATED PARAMETERS		
Static (Active) Earth Pressure Coefficient, K_a	0.283	Based on Rankine Theory (BDS C5.5.5.3)
Dynamic (Active) Earth Pressure Coefficient, K_{ae}	0.394	From Mononobe-Okabe Equation (Kramer p478-479)
Effective Wall Height for Static Active Thrust Force, H_1	42.00 ft	$H_1 = H + L \tan \beta$
Effective Wall Height for Dynamic Active Thrust Force, H_2	42.00 ft	$H_2 = H + (0.5\ H \tan \beta)/(1 - 0.5 \tan \beta)$ (FHWA-NHI-00-043 p.100)
Static Active Thrust Force From Retained Fill, F_T	29.92 kips/ft of wall	$F_T = 1/2\ \gamma_f K_a\ (H_1)^2$
Dynamic Active Thrust Force From Retained Fill, P_{ae}	11.74 kips/ft of wall	$P_{ae} = 1/2\ \gamma_f\ (K_{ae} - K_a)\ (H_2)^2$ (FHWA-NHI-00-043 p.100)
Horizontal Component of Static Active Thrust Force, F_1	29.92 kips/ft of wall	$F_1 = F_T \cos \beta$
Horizontal Force From Traffic Surcharge, F_2	2.85 kips/ft of wall	$F_2 = q\ H\ K_a$
Horizontal Inertial Force From Reinforced Soil Mass, P_{ir}	16.93 kips/ft of wall	$P_{ir} = 0.5\ k_h\ \gamma_f\ H_2\ H$ (FHWA-NHI-00-043 p.100)
Horizontal Inertial Force From Sloping Soil Surcharge, P_{is}	0.00 kips/ft of wall	$P_{is} = 0.125\ k_h\ \gamma_f\ (H_2)^2 \tan \beta$
Weight of Reinforced Soil Mass, V_1	148.18 kips/ft of wall	$V_1 = \gamma_r\ H\ L$
Weight of Sloping Backfill, V_2	0.00 kips/ft of wall	$V_2 = 1/2\ \gamma_f\ L\ (L \tan \beta)$
Sum of Resisting Moments, M_R	2,178.19 kip-ft/ft of wall	$M_R = V_1\ (L/2) + V_2\ (1/2\ L+1/6\ L)$
Sum of Overturning Moments (Static), M_{OS}	478.76 kip-ft/ft of wall	$M_{OS} = F_1\ (H1/3) + F_2\ (H1/2)$
Sum of Overturning Moments (Static + Dynamic), M_{OD}	1,130.16 kip-ft/ft of wall	$M_{OD} = Mos + P_{ir}\ (H/2) + P_{is}\ [H+ 1/3\ (H_2-H)] + P_{ae} \cos \beta\ (0.6\ H_2)$
Sum of Vertical Forces, V_T	148.18 kips/ft of wall	$V_T = V_1 + V_2 + F_T \sin \beta$
Sum of Horizontal Resisting Forces, P_R	85.55 kips/ft of wall	$P_R = V_T \tan [\min (\phi_r, \phi_b)]$
Sum of Horizontal Driving Forces (Static), P_{DS}	32.77 kips/ft of wall	$P_{DS} = F_1 + F_2$
Sum of Horizontal Driving Forces (Static + Dynamic), P_{DD}	61.44 kips/ft of wall	$P_{DD} = P_{DS} + P_{ir} + P_{is} + P_{ae}$
CHECK SLIDING FAILURE:		
Static Sliding Factor of Safety, $FS_{(static\ sliding)} = P_R/P_{DS}$	2.61 $\geq$ 1.5	**OK**
Dynamic Sliding Factor of Safety, $FS_{(dynamic\ sliding)} = P_R/P_{DD}$	1.39 $\geq$ 1.125	**OK**
CHECK OVERTURNING FAILURE:		
Static Overturning Factor of Safety, $FS_{(static\ overturning)} = M_R/M_{OS}$	4.55 $\geq$ 2.0	**OK**
Dynamic Overturning Factor of Safety, $FS_{(dynamic\ overturning)} = M_R/M_{OD}$	1.93 $\geq$ 1.5	**OK**
Static Eccentricity, $e_s = [L/2 - (M_R-M_{OS})/V_T]$	3.23 ft < L/6	**OK**
Dynamic Eccentricity, $e_d = [L/2 - (M_R-M_{OD})/V_T]$	7.63 ft < L/3	**OK**
CHECK BEARING FAILURE:		
Bearing Capacity Factor, Nc	30.14	Based on Caltrans DBS Table 4.4.7.1A (2003)
Bearing Capacity Factor, Nγ	22.40	Based on Caltrans DBS Table 4.4.7.1A (2003)
Max. Applied Bearing Pressure at the Base of the Wall, σ_v	6.46 ksf	$\sigma_v = V_T / (L - 2\ e_s)$
Ultimate Bearing Capacity, q_{ult}	28.26 ksf	$q_{ult} = c_b\ N_c + 0.5\ \gamma_b\ N_\gamma\ (L - 2\ e_s)$ (Embedment is neglected)
Bearing Capacity Factor of Safety, $FS_{(bearing\ capacity)}$	4.37 $\geq$ 2.0	**OK**

Global Stability Analysis – Static Condition

Slope stability Analysis for MSE Wall RW-1
File Name: Slope Stability-MSE Wall-RW1-Water in Basin-Static.gsz
Date: 7/14/2011
Original grouns Surface Elevation: 5.08 ft
Bottom of Basin elevation: -0.5 ft
Height of Retaining Wall: 30 ft
Distance between retaining wall and the Basin: 15 ft
Seismic Load = 0

Material Number, Description, Unit Weight, Cohesion, Friction Angle

Material # 1: Retained Backfill Unit Weight: 120 pcf; Cohesion: 0 psf; Friction Angle: Value: 34 degrees
Material # 2: MSE Wall Unit Weight: 150 pcf; Cohesion: 2000 psf; Friction Angle: Value: 40 degrees
Material # 3: Clay Unit Weight: 120 pcf; Cohesion: 100 psf; Friction Angle: Value: 25 degrees
Material # 4: Sand Unit Weight: 120 pcf; Cohesion: 0 psf; Friction Angle: Value: 33 degrees
Material # 5: Very Stiff Clay Unit Weight: 125 pcf; Cohesion: 250 psf; Friction Angle: Value: 28 degrees
Material # 6: Reinforced Material Unit Weight: 120 pcf; Cohesion: 2000 psf; Friction Angle: Value: 42 degrees
Parameters selected to force failure circles outside of reinforced area



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PROJECT NO.	111904.G0
DATE:	7/14/11
DRAWN BY:	H. Parsa
CHECKED BY:	C. Henderson

Station 878+50
Near the Proposed Basin

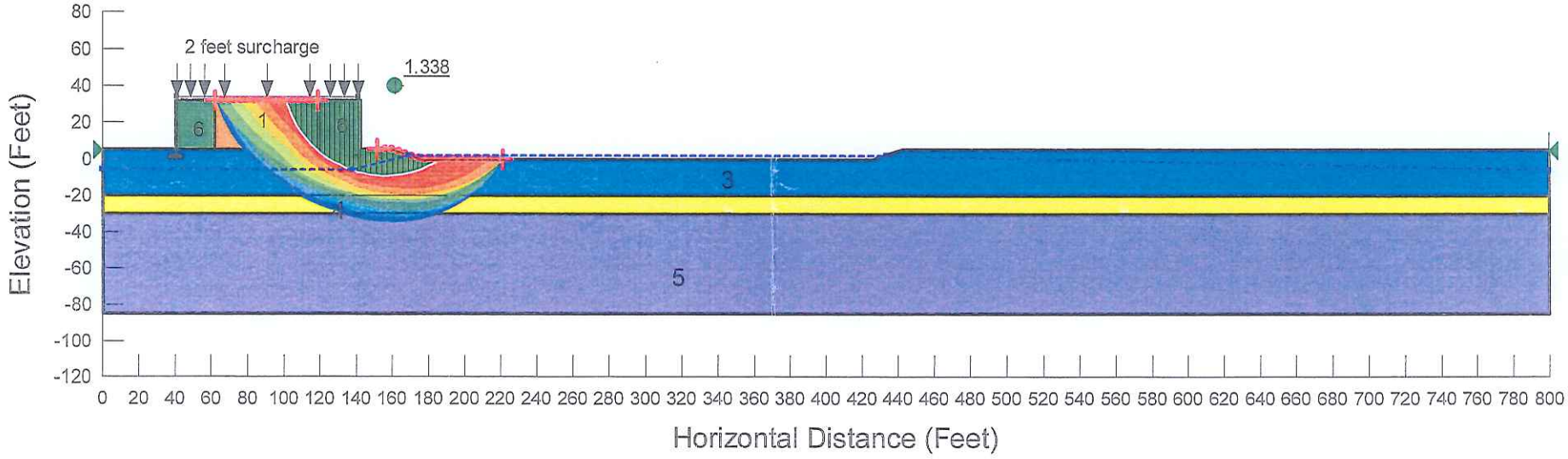
MSE Wall Global Stability Evaluation
Crosstown Freeway DR 4
Stockton, CALIFORNIA

PLATE

Slope stability Analysis for MSE Wall RW-1
File Name: Slope Stability-MSE Wall-RW1-Water in Basin-Static.gsz
Date: 7/14/2011
Original grouns Surface Elevation: 5.08 ft
Bottom of Basin elevation: -0.5 ft
Height of Retaining Wall: 30 ft
Distance between retaining wall and the Basin: 15 ft
Seismic Load = 0

Material Number, Description, Unit Weight, Cohesion, Friction Angle

Material # 1: Embankment Unit Weight: 120 pcf; Cohesion: 0 psf; Friction Angle: Value: 34 degrees
Material # 2: MSE Wall Unit Weight: 150 pcf; Cohesion: 2000 psf; Friction Angle: Value: 40 degrees
Material # 3: Clay Unit Weight: 120 pcf; Cohesion: 100 psf; Friction Angle: Value: 25 degrees
Material # 4: Sand Unit Weight: 120 pcf; Cohesion: 0 psf; Friction Angle: Value: 33 degrees
Material # 5: Very Stiff Clay Unit Weight: 125 pcf; Cohesion: 250 psf; Friction Angle: Value: 28 degrees
Material # 6: Reinforced Material Unit Weight: 120 pcf; Cohesion: 2000 psf; Friction Angle: Value: 42 degrees
Parameters selected to force failure circles outside of reinforced area



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Station 878+50
Near the Proposed Basin

MSE Wall Global Stability Evaluation
Crosstown Freeway DR 4
Stockton, CALIFORNIA

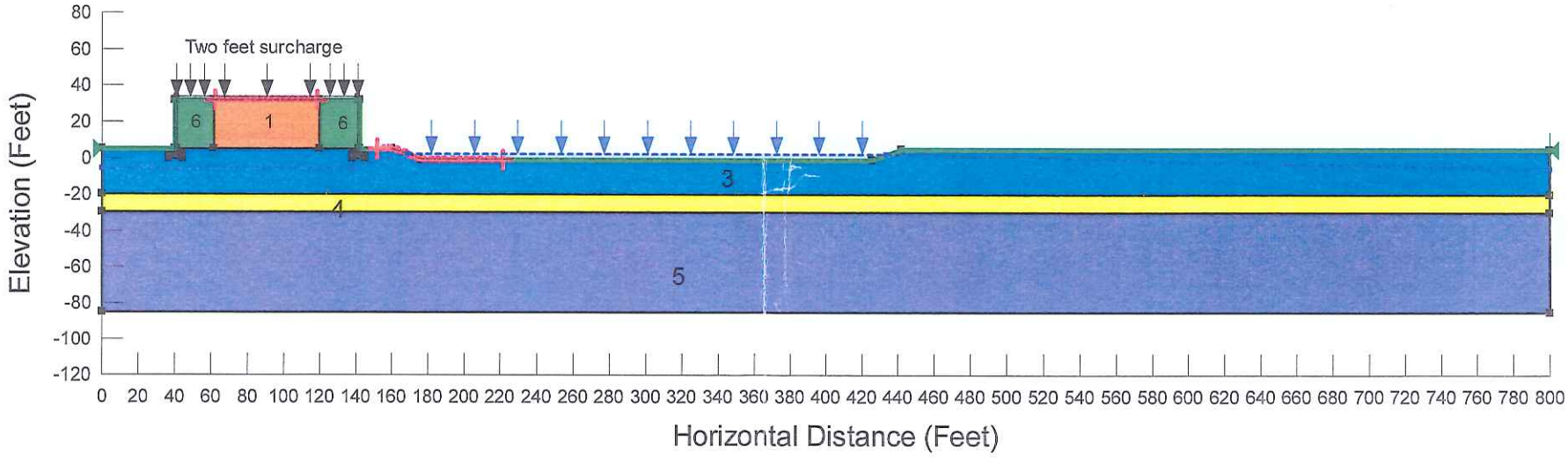
PLATE

Global Stability Analysis – Seismic Condition

Slope stability Analysis with Seismic Load for MSE Wall RW-1
File Name: Slope Stability-MSE Wall-RW1-Water in Basin-Seismic.gsz
Date: 7/14/2011
Original grouns Surface Elevation: 5.08 ft
Bottom of Basin elevation: -0.5 ft
Height of Retaining Wall: 30 ft
Distance between retaining wall and the Basin: 15 ft
Seismic Load = 0.16

Material Number, Description, Unit Weight, Cohesion, Friction Angle

Material # 1: Retained Backfill Unit Weight: 120 pcf; Cohesion: 0 psf; Friction Angle: Value: 34 degrees
Material # 2: MSE Wall Unit Weight: 150 pcf; Cohesion: 2000 psf; Friction Angle: Value: 40 degrees
Material # 3: Clay Unit Weight: 120 pcf; Cohesion: 100 psf; Friction Angle: Value: 25 degrees
Material # 4: Sand Unit Weight: 120 pcf; Cohesion: 0 psf; Friction Angle: Value: 33 degrees
Material # 5: Very Stiff Clay Unit Weight: 125 pcf; Cohesion: 250 psf; Friction Angle: Value: 28 degrees
Material # 6: Reinforced Material Unit Weight: 120 pcf; Cohesion: 2000 psf; Friction Angle: Value: 42 degrees
Parameters selected to force failure circles outside of reinforced area



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PROJECT NO.	111904.G0
DATE:	7/14/11
DRAWN BY:	H. Parsa
CHECKED BY:	C. Henderson

Station 878+50
Near the Proposed Basin

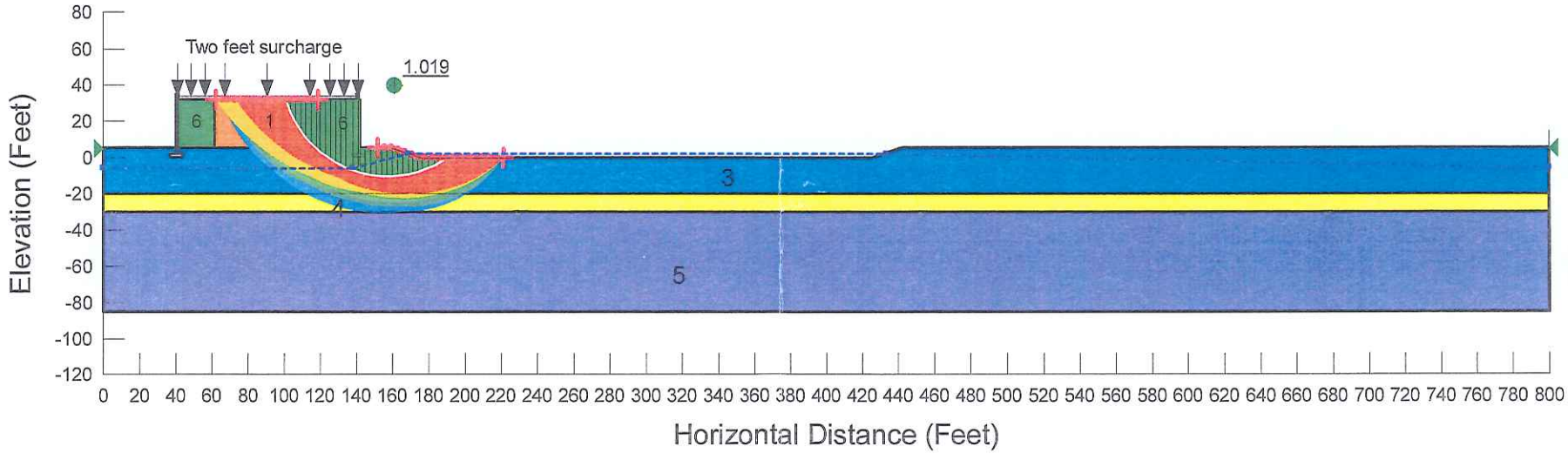
MSE Wall Global Stability Evaluation
Crosstown Freeway DR 4
Stockton, CALIFORNIA

PLATE

Slope stability Analysis with Seismic Load for MSE Wall RW-1
File Name: Slope Stability-MSE Wall-RW1-Water in Basin-Seismic.gsz
Date: 7/14/2011
Original grouns Surface Elevation: 5.08 ft
Bottom of Basin elevation: -0.5 ft
Height of Retaining Wall: 30 ft
Distance between retaining wall and the Basin: 15 ft
Seismic Load = 0.16

Material Number, Description, Unit Weight, Cohesion, Friction Angle

Material # 1: Embankment Unit Weight: 120 pcf; Cohesion: 0 psf; Friction Angle: Value: 34 degrees
Material # 2: MSE Wall Unit Weight: 150 pcf; Cohesion: 2000 psf; Friction Angle: Value: 40 degrees
Material # 3: Clay Unit Weight: 120 pcf; Cohesion: 100 psf; Friction Angle: Value: 25 degrees
Material # 4: Sand Unit Weight: 120 pcf; Cohesion: 0 psf; Friction Angle: Value: 33 degrees
Material # 5: Very Stiff Clay Unit Weight: 125 pcf; Cohesion: 250 psf; Friction Angle: Value: 28 degrees
Material # 6: Reinforced Material Unit Weight: 120 pcf; Cohesion: 2000 psf; Friction Angle: Value: 42 degrees
Parameters selected to force failure circles outside of reinforced area



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PROJECT NO.	111904.G0
DATE:	7/14/11
DRAWN BY:	H. Parsa
CHECKED BY:	C. Henderson

Station 878+50
Near the Proposed Basin

MSE Wall Global Stability Evaluation
Crosstown Freeway DR 4
Stockton, CALIFORNIA

PLATE

APPENDIX L
GABION BASKET WALL ANALYSIS

LIST OF ATTACHMENTS

The following are attached and complete this appendix.

Conceptual Plan – Partial Filling of LKQ Property Basin

Terra Aqua Gabions Gravity Retaining Wall Calculations

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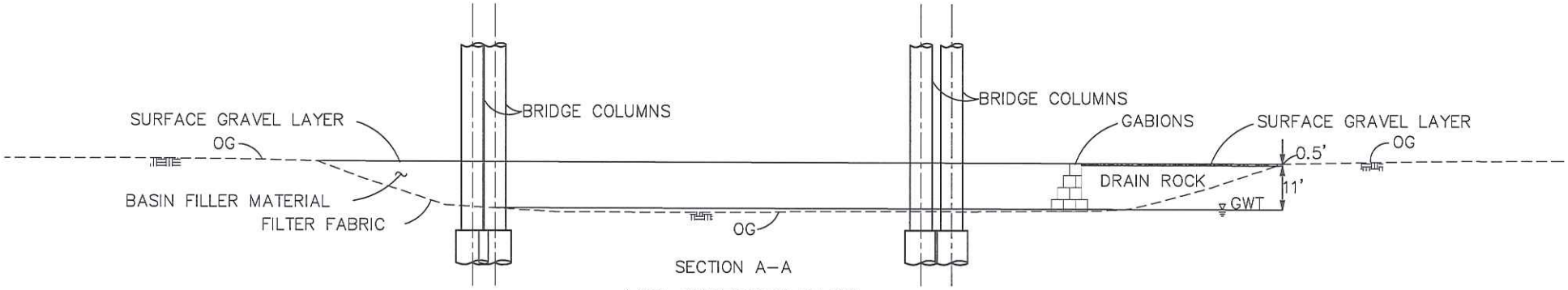
Conceptual Plan – Partial Filling of LKQ Property Basin



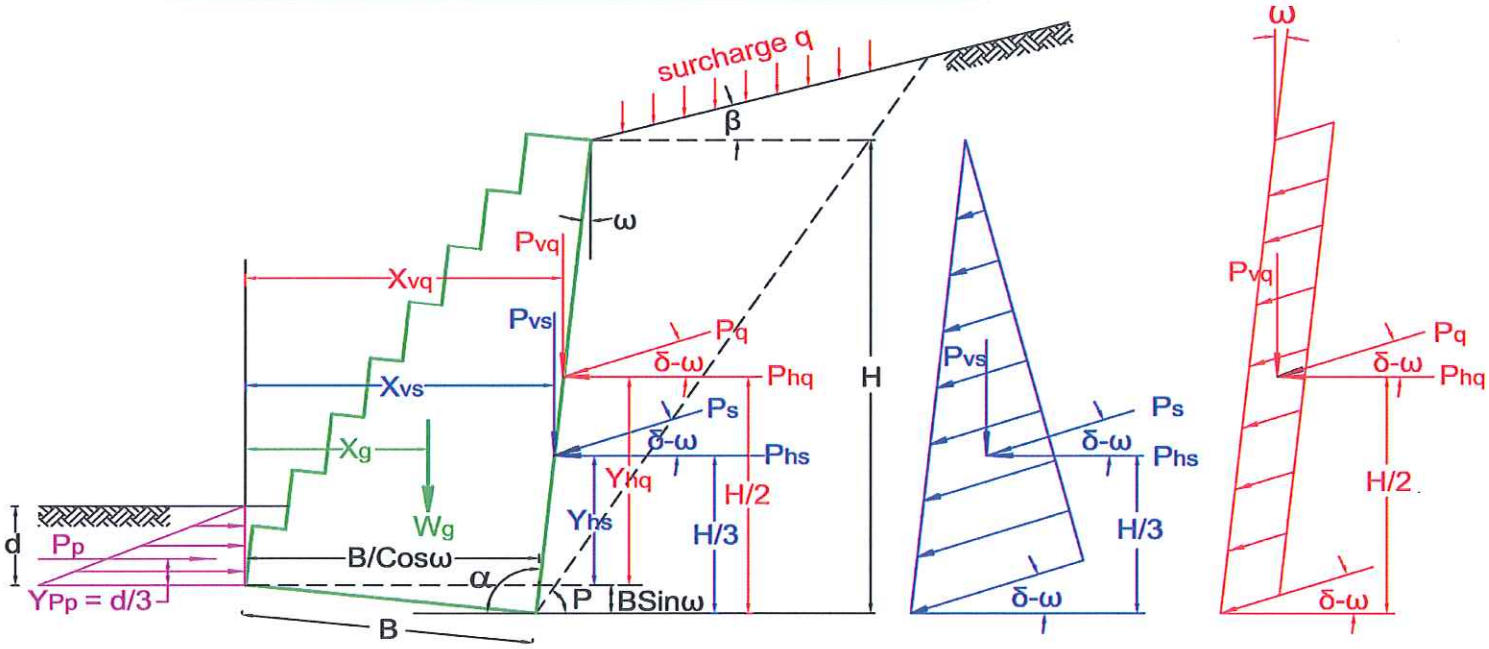
SECTION A–A

LKQ PROPERTY BASIN

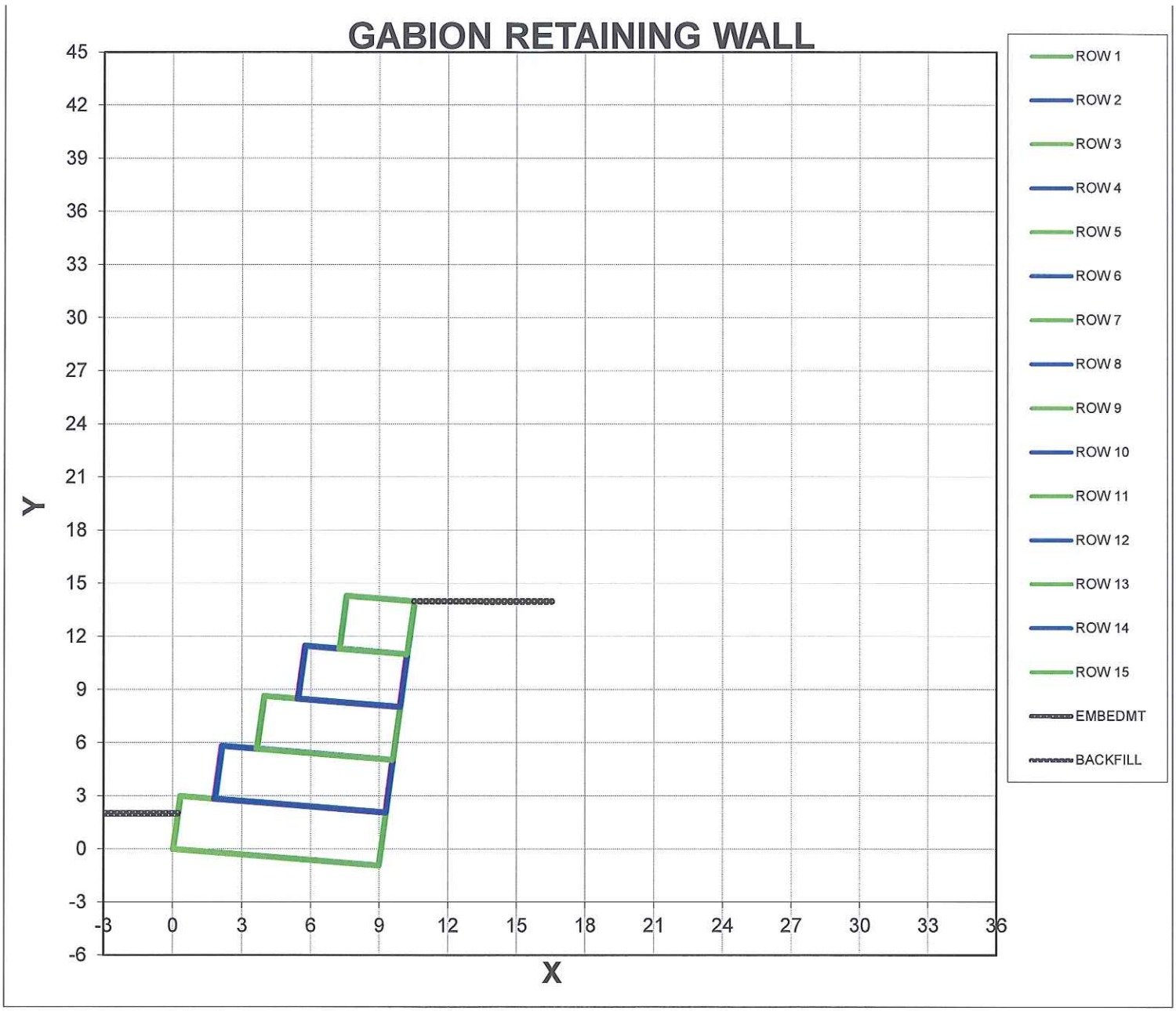
Terra Aqua Gabions Gravity Retaining Wall Calculations

TERRA AQUA GABIONS GRAVITY RETAINING WALL CALCULATIONS (STRAIGHT LINE BACK)

PROJECT NAME:	RT 4 Crosstown Freeway Extension	**PROJECT #:**	111904.G01
LOCATION:	San Joaquin County, California	**SECTION:**	
GEOTECHNICAL ENGINEER:	Carl Henderson	**REPORT #:**	
NOTES:	Proposed Gabion Wall at LKQ Basin	**DRAWING #:**	
		DATE:	10-May-12



Descriptions	symbols	Input Values	Units	Notes
Backfill slope angle above wall	β	0.000	°	< Φ
Angle of internal friction	Φ	34.000	°	
Wall friction reduction by geotextile	fr	20.000	%	Back of wall
Angle of wall friction	δ	27.200	°	Φ(100-fr)/100
Inclination angle to vertical plane	ω	6.000	°	for wall with straight back (no offsets)
Back of wall angle to horizontal	α	96.000	°	90+ω
Cohesion	c	0	psf	Ignore cohesion
Surcharge	q	240.000	psf	
Soil density	γs	120.000	pcf	
Rock density	γr	140.000	pcf	
Void in gabion	v	30.000	%	
Gabion density	γg	98.000	pcf	γr(100-v)/100
Actual height of wall	H	14.918	ft	(hCosω) Corrected for inclination
Embedment	d	2.000	ft	0 ft to ignore passive thrust
Width of base	B	9	ft	
Allowable soil bearing capacity	qa	2000.000	psf	determined by Geotechnical Engineer



Row #	Width (ft)	Height (ft)	offset (from toe) (ft)	Area (ft²)	X (ft)	Moment (ft³)	Y (ft)	Moment (ft³)
15				0.000	0.000	0.000	0.000	0.000
14				0.000	0.000	0.000	0.000	0.000
13				0.000	0.000	0.000	0.000	0.000
12				0.000	0.000	0.000	0.000	0.000
11				0.000	0.000	0.000	0.000	0.000
10				0.000	0.000	0.000	0.000	0.000
9				0.000	0.000	0.000	0.000	0.000
8				0.000	0.000	0.000	0.000	0.000
7				0.000	0.000	0.000	0.000	0.000
6				0.000	0.000	0.000	0.000	0.000
5	3.0	3.0	6.00	9.000	7.500	67.500	13.500	121.500
4	4.5	3.0	4.50	13.500	6.750	91.125	10.500	141.750
3	6.0	3.0	3.00	18.000	6.000	108.000	7.500	135.000
2	7.5	3.0	1.50	22.500	5.250	118.125	4.500	101.250
1	9.0	3.0	0.00	27.000	4.500	121.500	1.500	40.500
	h=	15.0		90.000	5.625	506.250	6.000	540.000

1 **COULOMB'S THEORY**

BACK

Active earth pressure coefficient

$$K_a = \frac{\sin^2(\alpha+\Phi)}{\sin^2\alpha \cdot \sin(\alpha-\delta)\left[1+\sqrt{\frac{\sin(\Phi+\delta)\sin(\Phi-\beta)}{\sin(\alpha-\delta)\sin(\alpha+\beta)}}\right]^2}$$

$$= \frac{0.587}{0.989 \cdot 0.932\left[1+\sqrt{\frac{0.876 \cdot 0.559}{0.932 \cdot 0.995}}\right]^2}$$

$$= \frac{0.587}{2.750}$$

$$= 0.213$$

Active soil thrust $P_s = 0.5K_a\gamma_s H^2$

$= 2849.074$ lb/ft

Active surcharge thrust $P_q = \frac{\sin\alpha}{\sin(\alpha+\beta)} K_{aq} H$

$= \frac{0.995}{0.995} 763.938$

$= 763.938$ lb/ft

Horizontal active soil thrust $P_{hs} = P_s\cos(\delta-\omega)$

$= 2656.260$ lb/ft

Horizontal active surcharge thrust $P_{hq} = P_q\cos(\delta-\omega)$

$= 712.238$ lb/ft

Vertical active soil thrust $P_{vs} = P_s\sin(\delta-\omega)$

$= 1030.295$ lb/ft

Vertical active surcharge thrust $P_{vq} = P_q\sin(\delta-\omega)$

$= 276.259$ lb/ft

FRONT

Inclination angle to vertical $\omega_p = 0.000$

Front face angle to horizontal $\alpha_p = 90-\omega_p$

$= 90.000$

Backfill slope $\beta_p = 0.000$

Angle of wall friction $\delta_p = 0.000$

Passive earth pressure coefficient

$$K_p = \frac{\sin^2(\alpha-\Phi)}{\sin^2\alpha \cdot \sin(\alpha+\delta)\left[1-\sqrt{\frac{\sin(\Phi+\delta)\sin(\Phi+\beta)}{\sin(\alpha+\delta)\sin(\alpha+\beta)}}\right]^2}$$

$$= \frac{0.687}{1.000 \cdot 1.000\left[1-\sqrt{0.559 \cdot 0.559}\right]^2}$$

| | √ | 1.000 | 1.000 |

$$\frac{0.687}{0.194}$$

3.537

Passive soil thrust $P_p = 0.5K_p\gamma_s d^2$

$= 848.912$ lb/ft

2 Check Overturning:

Vertical distance to P_{hs} $Y_{hs} = H/3-B\sin\omega$

$= 4.032$ ft

Vertical distance to P_{hq} $Y_{hq} = H/2-B\sin\omega$

$= 6.518$ ft

Overturning moment $\sum M_o = Y_{hs}P_{hs} + Y_{hq}P_{hq}$

$= 15352.128$ ft-lb / ft

Weight of Gabion $W_g = \sum A\,\gamma_g$

$= 8820.000$ lb / ft

Horizontal distance to W_g $X_g = Y\sin\omega + X\cos\omega$

$= 6.221$ ft

Horizontal distance to P_{vs} $X_{vs} = B/\cos\omega + (H/3-B\sin\omega)\tan\omega$

9.473

Horizontal distance to P_{vq} $X_{vq} = B/\cos\omega + (H/2-B\sin\omega)\tan\omega$

9.735

Vertical distance to P_p $Y_{Pp} = d/3$

$= 0.667$

Resisting moment $\sum M_r = W_g X_g + P_{vs} X_{vs} + P_{vq} X_{vq} + P_p Y_{Pp}$

$= 67887.928$ ft-lb / ft

Overturning factor of safety $SF_o = \frac{\sum M_r}{\sum M_o}$

4.422 ≥ 2.000 **O.K**

3 Check Sliding

Total Normal forces $\sum W = W_g\cos\omega + P_s\sin\delta + P_q\sin\delta - P_p\sin\omega$

		=	10334.448	lb/ft				
Frictional force	Ff	=	∑W	TanΦ				
		=	10334.448	0.675				
		=	6970.673					
Total Resisting Forces	∑Fr	=	Ff + CosωPp					
		=	7814.935					
Total Driving Forces at base	∑Fd	=	PsCosδ +	PqCosδ -	WgSinω			
			2291.531					
Siding factor of safety	SFs	=	∑Fr / ∑Fd					
			3.410	≥	1.500	**O.K**		

4 Check the Eccentricity of Resultant Force
(Resultant is in middle one third)

Eccentricity	e	=	0.5 B	-	(∑Mr / ∑W	-	Mo)	
		=	4.500	-	5.084			
		=	-B/6	≤	e	≤	+B/6	
			-1.500	≤ **O.K**	-0.584	≤ **O.K**	1.500	

5 Check Bearing

Applied bearing pressure	P	=	∑W / B	(1±	6e/B)			
		=	1148.272	(1±	0.389	)		
	Right	=	1594.997	psf	≤	2000.000	**O.K**	
	Left	=	701.547	psf	≤	2000.000	**O.K**	

Additional calculations may have to be performed including and not limiting to global stability analysis (by (Geotechnical Engineer), seismic forces (by Seismic Engineer) and hydraulic forces (by Hydraulic Engineer). Please check local, State and Federal requirements. The calculator assumes drained uniformed retained and foundation soil properties. The calculations should be reviewed, checked and certified by a Professional Engineer. To the best of our knowledge, the calculator and information was prepared accurately. Terra Aqua is not responsible for the reliability and validity of the geotechnical parameters assumed in the calculations. The calculator is intended to provided design design assistance to the engineer for the purpose of designing with Terra Aqua products.

GEOTECHNICAL DESIGN AND MATERIALS REPORT ROUTE 4 CROSSTOWN FREEWAY EXTENSION SAN JOAQUIN COUNTY, CALIFORNIA

May 4, 2012

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Bright People. Right Solutions.

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San Jose, CA 95126

GEOTECHNICAL DESIGN AND MATERIALS REPORT
ROUTE 4 CROSSTOWN FREEWAY EXTENSION
SAN JOAQUIN COUNTY, CALIFORNIA

Kleinfelder Job No. 111904.G01
May 4, 2012

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M ALTPIPE ANALYSIS

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GEOTECHNICAL DESIGN AND MATERIALS REPORT
ROUTE 4 CROSSTOWN FREEWAY EXTENSION
SAN JOAQUIN COUNTY, CALIFORNIA

1.0 INTRODUCTION

This report presents the results of our geotechnical services performed for the proposed Route 4 Crosstown Freeway Extension project, herein referred to as "project," in Stockton, California. Kleinfelder's work was performed in general accordance with the scope of work outlined in our proposal to Rajappan & Meyer Consulting Engineers, Inc. (Designer) and includes our assessment of proposed construction to extend State Route 4 (SR-4) West, also known as the "Crosstown Freeway," southwesterly from its current terminus at Fresno Avenue to Navy Drive in the City of Stockton (see Plate 1) in order to improve traffic access to the Port of Stockton and alleviate truck traffic through the Boggs Tract neighborhood. The improvements proposed for the project also include the widening of Navy Drive just south of the Burlington Northern Santa Fe (BNSF) Underpass to Fresno Avenue, stripping improvements on West Charter Way (SR-4) from Army Court to Tillie Lewis Drive, and installation of traffic signals at the Navy Drive/Tillie Lewis and West Charter Way/Tillie Lewis Drive intersections.

The purpose of this investigation was to explore the site subsurface conditions and provide geotechnical recommendations and opinions regarding the following:

- Characteristics of the subsurface materials along the proposed project alignment
- The geologic and seismic setting of the project area
- Site preparation and earthwork
- Cut and fill slopes
- Embankment fills
- Proposed retaining walls

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- Corrosion potential
- Pavement design criteria

Our scope of services consisted of review of readily available soils and geologic conditions to include as-built "Log of Test Borings, field exploration program, laboratory testing, design analysis, and preparation of this written report as generally described in our agreement with Rajappan & Meyer. This report is intended for use by Rajappan & Meyer, the San Joaquin Council of Governments (SJCOG), and the California Department of Transportation (Caltrans) for information and reference purposes and should not be construed as project specifications.

This report also addresses comments provided by Caltrans District 10, dated September 29, 2012 on our initial Geotechnical Design and Materials Report dated July 25, 2011. Our responses to these comments are provided in Appendix A of this report. The following bridge foundation report was prepared and submitted separately by Kleinfelder .

> Foundation Report, , Route 4 Crosstown Freeway Extension, San Joaquin County, California, Kleinfelder File No. 111904.G01/STO11R158R1, dated May 4 , 2012

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2.0 PROPOSED IMPROVEMENTS

The San Joaquin Council of Governments, in cooperation with Caltrans, proposes to extend State Route 4 (SR-4) West southwesterly from its current terminus at Fresno Avenue to Navy Drive in the City of Stockton (see Plate 1) in order to improve traffic access to the Port of Stockton and alleviate truck traffic through the Boggs Tract neighborhood. The improvements proposed for the project also include the widening of Navy Drive just south of the Burlington Northern Santa Fe (BNSF) Underpass to Fresno Avenue, stripping improvements on West Charter Way (SR-4) from Army Court to Tillie Lewis Drive, and installation of traffic signals at the Navy Drive/Tillie Lewis and West Charter Way/Tillie Lewis Drive intersections. Major construction elements of the project include a new elevated structure, retaining walls, and embankments which are described in more detail below.

2.1 ELEVATED STRUCTURE

The proposed extension will include approximately ½ mile of elevated structure spanning from just east of Fresno Avenue to about 1,100 feet west of South Ventura Avenue. The proposed 21-span, elevated structure will be twin viaducts supported by embankments at both ends and concrete columns varying in height from approximately 30 to 55 feet above existing ground surface.

The proposed elevated structure will have 12-foot wide lanes (2 lanes in each direction). A third lane providing through movement for eastbound Crosstown Freeway traffic will be reconstructed to begin east of Fresno Avenue and conform to the existing lane at the Garfield Street Overhead. Two viaducts were proposed from just west of South Ventura Avenue to just east of Fresno Avenue. The twin viaducts are each 42 feet wide and separated by about 33 feet. The proposed eastbound and westbound mainline will each have 5-foot wide inside shoulders and 10-foot wide outside shoulders. Concrete barriers and railings will be constructed and placed on both the inside and outside shoulders at the edge of pavement for the entire length of the elevated structure.

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2.2 RETAINING WALLS AND EMBANKMENT

Embankments are proposed at both ends of the elevated structure. The western embankment, containing the eastbound on-ramp and westbound off-ramp, will extend from Navy Drive to the viaduct structure just south of the BNSF Railway. This embankment will vary in width from 75 to 110 feet and be retained by three mechanically stabilized embankment (MSE) walls. The MSE walls are approximately 555 feet (Retaining Wall 1), 790 feet (Retaining Wall 2), and 83 feet (Retaining Wall 3) in length and range in height from about 0 feet at Navy Drive (west) to 42 feet at the viaduct interface (east). A Standard Type 1 wall (Retaining Wall 4) with a maximum height of 8 feet and approximately 750 feet long will be located on the south side of Navy Drive.

The eastern embankment for the eastbound and westbound mainline from just east of Fresno Avenue to the Garfield Street Overhead will include sloped embankments. It will measure about 170 feet wide and vary in height from about 0 to 30 feet above the existing ground level, with its highest point just east of Fresno Avenue. A Standard Type 1 Wall (Retaining Wall 5) with a maximum height of 14 feet and approximately 300 feet long will adjoin the east bound viaduct, eastern abutment.

2.3 BASINS

Retention basins (two total) are proposed to be constructed adjacent the eastbound on-ramp and westbound off-ramp of the western approach embankment. Detention Basins (two total) are proposed to be constructed at the east end of the alignment, located adjacent (southwest and southeast) of Fresno Avenue. The existing "LKQ" property basin located within the approximate footprint of proposed Bents 4 and 5 of the eastbound viaduct structure will be partially filled to be used as a temporary staging area for bridge construction. An 11 feet high (exposed height) gabion basket gravity wall is proposed to be placed to retain the crushed drain rock which will be used to partially fill the drainage basin.

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3.0 EXISTING FACILITIES

The project site is bounded by the Stockton Deep Water Channel on the North, San Joaquin River and Rough and Ready Island on the west, West Charter Way on the South, and I-5 on the East. The project alignment is generally located in a semi-developed area consisting of residential, industrial, and commercial buildings. The terrain along the alignment west of Fresno Avenue is generally flat, with an elevation of about 6 feet, increasing to an elevation of about 30 feet at the eastern end of the proposed project. The BNSF and CCTC rail lines also bisect the project site. Existing drainage includes roadside ditches, underground pipes, detention ponds, pump stations, and outfalls to major waterways.

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4.0 PERTINENT REPORTS AND INVESTIGATIONS

Parikh Consultant, Inc. performed a preliminary geotechnical investigation at the project site. Their findings and preliminary foundation recommendations for the project were summarized in the report Preliminary Foundation Report, Route 4 Crosstown Connector, San Joaquin County, California, dated March 2010. As part of their investigation, three exploratory borings and two cone penetration tests (CPTs) were performed along the project alignment to a maximum depth of about 121 feet below existing ground surface. Logs of exploratory borings and CPTs are presented in Appendix C.

Parikh also performed laboratory tests in accordance with ASTM Standards on selected soil samples collected during field exploration to evaluate their physical characteristics and engineering properties. The laboratory testing program was formulated with emphasis on the evaluation of natural moisture content, in-place density, plasticity, consolidation, and grain size distribution of the soil samples. Results of laboratory tests are presented in Appendix E.

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5.0 PHYSICAL SETTING

5.1 CLIMATE

The project site is located near the central portion of the northern San Joaquin Valley near the southwest portion of Stockton. The climate of the project area is characterized by hot, dry summers and cool, moist winters. At Stockton, the average maximum temperature in summer ranges from about 88 degrees in June to about 93 degrees in July. The average minimum temperature in winter ranges from about 37 degrees in January to about 42 degrees in March. Average precipitation in the project area ranges from about 13 to 13¾ inches, with the highest recorded precipitation of 25½ inches in 1982 and the lowest of 5½ inches in 1976. About 90 percent of the total annual rainfall falls between November and April. Of this, about 55 percent falls between December and February. About 1 percent of the average annual rainfall falls between June and August.

5.2 TOPOGRAPHY

The project alignment is generally located in a semi-developed area consisting of residential, industrial, and commercial buildings. The terrain along the alignment west of Fresno Avenue is generally flat, with an elevation of about 6 feet, increasing to an elevation of about 30 feet at the eastern end of the proposed project.

5.3 SURFACE DRAINAGE

Existing drainage includes roadside ditches, underground pipes, detention ponds, pump stations, and outfalls to major waterways. The northern half of the Boggs Tract drains to an interim drainage pond and emergency storm discharge line which conveys water to the Stockton Deep Water Channel to the north. The southern half of the Boggs Tract drains to the Mormon Slough Outfall to the east via a pump station located at the corner of Fresno and West Scotts Avenues.

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5.4 MAN-MADE AND NATURAL FEATURES OF ENGINEERING AND CONSTRUCTION SIGNIFICANCE

Significant man-made features along the project alignment include various industrial facilities and residences.

5.5 REGIONAL GEOLOGY

The project site lies within the central portion of the Great Valley geomorphic province of California. The province is bordered to the north by the Cascade and Klamath ranges; to the west by the structurally complex sedimentary and volcanic rock units of the Coast Ranges, to the east by the granitic and metamorphic basement rocks which form the gently sloping western foothills of the Sierra Nevada range, and to the south by the east-west trending Transverse ranges. About 400 miles (645 km) long and 50 miles (80 km) wide, the Great Valley is an asymmetrical, synclinal trough formed by tilting of the Sierran block during the late Tertiary and Quaternary periods with the western side dropping to form the valley and the eastern side uplifting to form the Sierra Nevada mountains. Within the project area, erosion of the adjacent Sierra Nevada and Coast Ranges has in-filled the valley with a thick sequence of unconsolidated to semi-consolidated Quaternary (Pleistocene and Holocene) age alluvial, basin, and delta plain sediments deposited by the Sacramento and San Joaquin rivers and their tributaries. The thickness of the valley sediments varies from a thin veneer at the edges of the valley to thousands of meters in the western portion. The bedrock complex is composed of metamorphosed marine sediments similar to those found in the foothills of the western Sierra Nevada and the core of the Coast Ranges.

5.6 FAULTING AND SEISMICITY

Based on the information provided in Hart and Bryant (2007), the site is not located within a State-designated Alquist-Priolo Earthquake Fault Zone where site-specific studies addressing the potential for surface fault rupture are required, and no known active faults traverse the site. Stockton is in a region where there are very few active faults.

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An active fault is a fault that has experienced seismic activity during historic time (since roughly 1800) or exhibits evidence of surface displacement during Holocene time (Hart and Bryant, 2007). The definition of "potentially active" varies. A generally accepted definition of "potentially active" is a fault showing evidence of displacement that is older than 11,000 years (Holocene age) and younger than 1.7 million years (Pleistocene age). However, "potentially active" is no longer used as criteria for zoning by the California Geologic Survey (CGS), formerly known as the State of California Division of Mines and Geology. The terms "sufficiently active" and "well-defined" are now used by the CGS as criteria for zoning faults under the Alquist-Priolo Earthquake Fault Act. A "sufficiently active fault" is a fault that shows evidence of Holocene surface displacement along one or more of its segments and branches, while a "well-defined fault" is a fault whose trace is clearly detectable by a trained geologist as a physical feature at or just below the ground surface. The definition "inactive" generally implies that a fault has not been active since the beginning of the Pleistocene Epoch (older than 1.7 million years old).

The San Andreas fault (SAF), located about 70 miles (113 km) to the west, is considered to dominate the structure and seismicity of the region. This right-lateral strike-slip fault, which extends from the Gulf of California in Mexico to Cape Mendocino off the Coast of Humboldt County in Northern California, forms a portion of the boundary between two independent tectonic plates on the surface of the earth. To the west of the SAF is the Pacific Plate, which moves north relative to the North American Plate, located east of the fault. In the San Francisco Bay area, movement across this plate boundary is concentrated on the SAF; however, it is also distributed, to a lesser extent, across a number of faults which include the Calaveras, Concord-Green Valley, Clayton-Marsh Creek-Greenville, and Hayward-Rodgers Creek faults among others. Together, these faults are referred to as the SAF system.

Several secondary or potentially active fault traces are located in the general vicinity of the project site. The closest of these are the Great Valley fault 7, the Marsh Creek-Greenville section of the Greenville fault zone, and the Las Positas fault, located to the west of the site.

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The Great Valley fault is characterized by a zone of concealed or "blind" thrust faulting, reverse faults, and folds that extend for several hundred kilometers from the southern San Joaquin Valley in Kern County northward to the northern San Joaquin Valley in Tehama County. Although not exposed at the surface, regional studies have suggested the Great Valley fault system may be comprised of between 18 and 25 segments ranging from about 8 to 35 miles (12 to 57 km) in length, with most segments between 12 and 18 miles (20 and 30 km), on the basis of the seismo-tectonic environment of the area. The characteristic earthquake for the average-length segment is believed capable of yielding earthquakes of magnitude 6.3 to 6.4, or slightly smaller than the 1983 (M6.5) Coalinga earthquake, and the longest segments are believed capable of yielding earthquakes as large as magnitude 6.8 to 7.0, or equal to the Vacaville-Winters earthquake.

Located in the Diablo Range of the Coast Ranges Geomorphic Province, the Greenville fault zone is identified as a right lateral strike slip fault with a dip of 90 degrees. The Marsh Creek-Greenville section of the fault zone experienced minor surface fault rupture in January of 1980.

The table below presents a list of significant active or potentially active faults within approximately 50 kilometers of the project site, their assigned moment magnitudes and the distance from the fault trace at the surface to the site (Merriam and Shantz, 2007). A fault map for the general project area is presented on Plate 4.

Table 5.1
Regional Faults and Characteristics

Fault	Closest Distance to Site (km)	Maximum Magnitude (M_{Max})
Great Valley fault 7	28 (W)	6.7
Greenville fault zone (Marsh Creek-Greenville section)	42 (W)	6.6
Las Positas Fault	43 (W)	6.5

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6.0 FIELD EXPLORATIONS

6.1 FIELD INVESTIGATION – TEST BORINGS,CPTS, AND GEOPROBES

For Kleinfelder's initial roadway design investigation, thirteen test borings and eleven CPTs were advanced to depths ranging from about 31½ to 151½ feet below existing grade. Subsurface information obtained from five geoprobe explorations advanced to depths ranging from about 16 to 20 feet, from a Detailed Site Investigation completed by Kleinfelder for the Debco Autowrecking , Inc. facility (Kleinfelder File No. 113961.4/STO11R302, dated December 16, 2011) was also used for our roadway design investigation (Specifically for Retaining Wall 4). Detailed descriptions of site surface conditions encountered in our test borings,CPTs, and geoprobe explorations are presented in Section 8.4 of this report. The approximate boring,CPT, an geoprobe locations are presented on Plate 2.

The borings were drilled using a CME-75 or 85 truck-mounted drill rig equipped with 8-inch O.D. hollow-stem auger and using rotary wash method below groundwater. During the drilling operations, penetration tests were performed in accordance with ASTMD-1586 at regular intervals using a modified California sampler to evaluate the relative density of coarse-grained (cohesionless) soil and to retain soil samples for laboratory testing. A pocket penetrometer was used to evaluate the consistency of fine-grained (cohesive) soil samples retained. The penetration tests were performed by initially driving the sampler 6 inches into the bottom of the bore hole using a cathead system and a 140-pound trip-hammer, falling 30 inches to penetrate loose soil cuttings and "seat" the sampler. Thereafter, the sampler was progressively driven an additional 12 inches, with the results recorded as the corresponding number of blows required to advance the sampler 12 inches, or any part thereof. A representative with our firm maintained a log of the borings and visually classified the soils encountered according to the unified soil classification system. Soil samples obtained from the borings were packaged and sealed in the field to reduce moisture loss and disturbance and brought to our Stockton laboratory for future testing. Additionally, 3 inch diameter Shelby Tubes were used to obtain relatively undisturbed soil samples at several locations.

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The CPTs were performed by hydraulically pushing a 1.4-inch diameter Dutch Cone Penetrometer into the subsurface soils using an enclosed truck-mounted 20-ton ram system. During penetration, the cone or tip resistance and sleeve friction resistance is recorded on a nearly continuous basis to the depth of exploration. Based on published correlations, the data obtained is used to estimate stratigraphy, soil type, groundwater depth, and in situ soil parameters such as undrained shear strength and standard penetration (N) values. The soils classifications and blow counts encountered in the test borings are consistent with the data correlations obtained from the CPTs.

Geoprobes/soil probes (probes) were continuously advanced in 4- to 5-foot intervals. During advancement of the probes, Kleinfelder collected soil samples for logging purposes, field screening, and potential chemical analysis. The probes were pneumatically and/or hydraulically advanced using a core barrel. Soil samples were continuously collected by inserting a new acetate liner into the core barrel prior to each sampling interval. While probing, an experienced environmental geologist/engineer classifies the soil, logs the stratigraphy of the borings, and collected soil samples.

Descriptions of soil materials encountered in the explorations are presented on the Logs of Test Borings in Appendix B. The borings,CPTs, and geoprobes were located in the field by visual sighting and/or pacing from existing site features; therefore, the locations shown on Plate 2 should be considered approximate. The penetration resistance (blows/foot) shown on the logs of borings represents field penetration that has not been corrected for overburden pressure, sampler size, hammer type, borehole diameter, rod length, sampling method or any other correction factor. The preparation of boring logs was according to the Caltrans "Soil & Rock Logging, Classifications, and Presentation Manual."

6.2 FIELD INVESTIGATION – PAVEMENT CORING

In an effort to assess the thickness of pavement sections within the corridor of Fresno Avenue underneath the proposed elevated structure, pavement coring was performed at two locations designated by the Designer in February 2011. Additional pavement cores were obtained at the intersection of Navy Drive and Tillie Lewis Drive, and on Tillie Lewis Drive as directed by the Designer in October 2011. Approximate locations

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of pavement cores and the resulting estimated pavement sections determined from the coring operations are presented in Appendix L.

6.3 GEOLOGIC MAPPING

Geologic mapping was not performed during this investigation other than to observe and confirm existing literature (see Section 5.5). The alignment is wholly contained within alluvial sediments of Pleistocene age. Alluvial sediments are anticipated to be several hundred meters thick in this area. No fault structures traverse or are near the project site. No surficial evidence of active or past landslides was noted. Because of the flat natural topography, the potential for land sliding within the native soils is remote.

6.4 GEOPHYSICAL STUDIES

Geophysical studies were not performed during the investigation due to a lack of need for such studies at this site.

6.5 INSTRUMENTATION

No instrumentation was installed or deemed necessary during the investigation.

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7.0 GEOTECHNICAL TESTING

7.1 IN SITU TESTING

In situ testing on the project was limited to penetration testing during drilling operations with the drill rig and pocket penetrometer tests on the soil samples retrieved. Blow counts were recorded on the drilling logs to document the apparent relative density of the coarse-grained (cohesionless) soils encountered. The testing was performed in general accordance with ASTM standard test method D-1586 using a trip hammer. The pocket penetrometer test results were used to evaluate the consistency of fine-grained (cohesive) soils.

7.2 LABORATORY TESTING

Laboratory tests were performed in accordance with current ASTM standards on selected soil samples to evaluate their physical characteristics and engineering properties. The laboratory testing program was formulated with emphasis on the evaluation of natural moisture content, in-place density, grain-size distribution, percent soil passing the #200 sieve, plasticity, consolidation, and undrained shear strength of the materials encountered. In addition, pH, minimum resistivity, sulfate concentration and chloride concentration tests were performed on near-surface soil samples to evaluate the general corrosivity of the soils to buried concrete and ferrous metals. For design of the proposed pavement sections within the project site, five resistance value (R-Value) tests were performed on selected near-surface soil samples.

The results of Kleinfelder laboratory tests are summarized in Appendix D. This information, along with the field observations, was used to prepare the final test boring logs.

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8.0 GEOTECHNICAL CONDITIONS

8.1 LOCAL GEOLOGY

Various authors have mapped the local geology of the project area (e.g., Atwater, 1982). These maps differ in scale and detail but agree that the project site is underlain at the surface by Late Pleistocene age (1.7 million years) clay, silt, sand, and gravel of the Modesto Formation (Qm). These soils were probably eroded from the nearby Sierra Nevada Mountain Ranges and deposited as fan, mudflow, terrace, and floodplain deposits from the nearby streams and rivers. A geological map of the general project area is shown on Plate 3.

8.2 STRUCTURE

The soil units in the vicinity of Stockton typically have flat bedding structure. Local variances are likely to be present with a slight dip generally toward the west. No confirmed earthquake faults have been mapped in the vicinity of the proposed alignment.

8.3 NATURAL SLOPE STABILITY

Due to the generally flat to slightly sloping nature of the existing ground, natural slope stability is not an issue on this project.

8.4 SUBSURFACE SOIL CONDITIONS

The subsurface conditions encountered during our field investigations were consistent with the previous findings throughout the project site. The following table presents depths to groundwater and the subsurface soils conditions encountered in our field explorations. Subsurface soils can be generally characterized as fine grained and cohesive (CL, ML, SC) with some interbedded layers of silty and "clean" sand. Soils generally become stiffer with depth and can be characterized as an "intermediate geo material" at depths of approximately 90 feet and deeper below existing grade.

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Table 8.1
Summary of Subsurface Conditions

Exploration ID	Depth (Feet)	Depth to GW (Feet)	Subsurface Soil Conditions
R-11-001	91½	8	Firm to hard fat clay to sandy lean clay to about 30 to 34 feet, followed by medium dense to dense silty to relatively clean sand and very hard fat clay
R-11-002	91½	8	
R-11-003	151½	8	Firm to hard fat clay to sandy lean clay to about 19 feet, followed by medium dense to dense silty to relatively clean sand to about 39 feet, hard lean clay to about 54 feet underlain by interbedded layers of silt, lean clay, and poorly graded sand
R-11-004	91½	18	Firm to hard sandy lean clay and clayey sand to about 24 feet, followed by interbedded layers of clean sand and sandy clay to about 50 feet, underlain by sandy lean clay
R-11-005	91½	18	Firm to hard Sandy lean clay and clayey sand to about 70 feet, underlain by dense to very dense silty to relatively clean sand
R-11-006	91½	18	Firm to hard lean clay to sandy lean clay to the maximum depth with interbedded layers of silty sand between 26 to 49 feet below grade
R-11-007	91½	11	Firm to hard lean clay with interbedded layers of silty to relatively clean sand to 45 feet underlain by hard to very hard sandy lean clay
R-11-008	91½	15	Firm to hard lean clay with interbedded layers of silty to relatively clean sand to about 78 feet underlain by hard to very hard fat clay
R-11-009	121½	14	Firm to hard lean clay to fat clay to about 34 feet underlain by interbedded layers of sandy silt and silty to relatively clean sand to about 80 feet followed be very hard sandy lean clay to 118 feet underlain by clean sand
R-11-010	51½	36	Medium dense silty sand to about 19 feet, underlain by firm to hard sandy lean clay to 44 feet followed by medium dense silty sand
R-11-011	151½	14	Firm to hard fat clay to 10 feet followed by medium dense silty to relatively clean sand to 20 feet, sandy lean clay to 60 feet, medium dense to dense silty to relatively clean sand to 70 feet, and hard to very hard sandy lean clay to fat clay.
R-11-012	51½	14	Firm to hard sandy lean clay to 20 feet, followed by medium dense clean sand to 29 feet, and firm to hard sandy lean clay, clayey sand, and fat clay.
R-11-013	31½	14	Firm to hard sandy lean clay
CPT-11-001	90	11	Lean Clay to sandy lean clay and silt to 60 feet underlain by silty to relatively clean sand to 65 feet, lean clay 86 feet and sand
CPT-11-002	90	10	Lean Clay to sandy lean clay and silt with occasional interbedded layers of silty to relatively clean sand
CPT-11-003	90	10	Lean Clay to 25 feet followed by relatively clean sand to 30 feet, sandy silt and clay to 42 feet, clayey to silty sand and clean sand to 47 feet , underlain by sandy silt and sandy lean clay to 84 feet, and silty to relatively clean sand
CPT-11-004	90	11	Lean Clay with interbedded seams of sandy silt to about 22 feet followed by silty to relatively clean sand to about 25 feet, lean clay with occasional interbedded layers of sandy silt to 67 feet, silty sand to clean sand to 76 feet, sandy lean clay to 82 feet, and silty to clean sand to 87 feet followed by sandy silt and clay
CPT-11-005	90	14	Sandy lean clay and silt with seams of silty sand
CPT-11-006	90	13	Sandy lean clay and silt with a 2-foot thick interbedded layer of silty to relatively clean sand at 84 feet below site grade

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Exploration ID	Depth (Feet)	Depth to GW (Feet)	Subsurface Soil Conditions
CPT-11-007	90	10¾	Sandy lean clay and silt to 20 feet followed by silty sand and clean sand with interbedded seams of clay and silt to 42 feet followed by sandy lean clay and silt
CPT-11-008	90	11	Sandy lean clay and silt to 36 feet followed by silty sand and clean sand with interbedded seams of clay and silt to 49 feet followed by sandy lean clay and silt to 78 feet, underlain by sandy lean clay
CPT-11-009	50	24¾	Silty sand to 5 feet followed by sandy lean clay and silt to 38 feet, underlain by silty to relatively clean sand
CPT-11-010	105 (refusal)	9	Sandy lean clay and silt to 37 feet followed by silty sand and clean sand to 42 feet, sandy lean clay and silt to 48 feet followed by sandy lean clay and silt to 62 feet, underlain by sandy lean clay to 76 feet, silty sand to clean sand to 84 feet, and sandy lean clay to the end
CPT-11-011	50	7½	Sandy lean clay and silt to 27 feet followed by silty to clean sand to 43 feet and sandy silt and clay
O-11-1 (Geoprobe)	16	13	Sandy gravel to 2 feet followed by clay with sand to 16 feet. An interbedded layer of sand with clay was encountered between 13 and 14 feet.
O-11-2 (Geoprobe)	20	16	Clay with sand to a depth of 20 feet, interbedded with a layer of sand with clay between 6.5 and 11 feet.
O-11-3 (Geoprobe)	16	14	Clay with little sand to 11 feet, underlain by sand with clay to depth of 16 feet.
O-11-4 (Geoprobe)	16	11	Sandy gravel to 2 ½ feet followed by clay with sand to 16 feet. An interbedded layer of sand with clay was encountered between 10 ½ and 13 ½ feet.
O-11-5 (Geoprobe)	16	12	Gravelly sand to 2 ½ feet followed by clay with sand to 11 ½ feet. A layer of sand with clay was the next layer to 14 ½ feet followed by gravel with trace sand to 16 feet.

8.5 GROUNDWATER DEPTHS

The test borings and CPTs were checked for the presence of groundwater during and immediately following drilling operations. Groundwater was encountered at a depths ranging from of about 7½ (elev. ± -3½) to 36 (elev. ± -18) feet, from west to east, below existing grade. Records maintained by the California Department of Water Resources (DWR)[1] indicate that historical high groundwater level in the vicinity of the project site is about 10 to 20 feet below the ground surface. Because of its close proximity to the Port of Stockton, we anticipated that the groundwater at the project site should be located at a relatively shallow depth on the order of about 10 to 15 feet. Groundwater elevations and soil moisture conditions within the project area will vary depending on seasonal rainfall, irrigation practices, land use, and/or runoff conditions not apparent at the time of our field investigation. The evaluation of such factors is beyond the scope of this investigation.

[1] *http://wdl.water.ca.gov/gw/*

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8.6 ENGINEERING SEISMOLOGY

The governing deterministic seismic event for the project site is a magnitude 6.7 earthquake on the Great Valley 7 fault (Mw=6.7, style of faulting:reverse). This fault is located about 17.5 miles from the project site.

Based on the available boring and CPT information, the subsurface soil conditions at the project site consist of alternative layers of stiff to hard lean clay and silt, interbedded with loose to medium-dense silty sand and poorly-graded sand with silt to the maximum depth explored. These types of soils generally match the criteria for Soil Type D, as per Caltrans Seismic Design Criteria (version 1.6, 2010). The average small strain shear wave velocity (V_{S30}) for the upper 100 feet (30 meters) was estimated to be 820 feet per second (ft/s)/249 meter per second (m/s), using the SPT blow count correlations developed by Sykora (1987) and Ohta and Goto (1978).

The acceleration response spectrum (ARS) curve was developed with the procedures newly established by Caltrans (2009a, 2009b). Both the deterministic and probabilistic procedures were adopted to calculate the acceleration response spectrum, using the Deterministic Response Spectrum Spreadsheet and Probabilistic Response Spectrum Spreadsheet, respectively. The probabilistic ARS data developed using ARS Online was compared with the data from the USGS Interactive Deaggregation Tool. Since the maximum difference between the ARS Online data and the USGS data was greater than 10 percent, the USGS data was selected for our probabilistic design spectrum. Furthermore, a comparison made between the probabilistic and deterministic response spectra indicated that the probabilistic response spectrum governs the site. The design ARS Curve for the proposed project is presented on Plate 5. The deterministic ARS data are also presented on Plate 5 for comparison purposes. Details of the ARS curve development are presented in Appendix F.

Recommended seismic design parameters are presented in the table below.

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Table 8.2
Seismic Design Parameters

Seismic Parameters	Design Recommendations
Controlling Fault	Great Valley 7
Type of Fault	Reverse
Earthquake Magnitude	6.7 Mw
Peak Ground Acceleration	0.32g
Small Strain Shear Wave Velocity	249 m/s
Design ARS Curve	See Plate 5

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9.0 GEOTECHNICAL ANALYSIS AND DESIGN

9.1 DYNAMIC ANALYSIS

9.1.1 Liquefaction and Earthquake-Induced Settlement

In order for liquefaction and possible associated settlement of soils due to ground shaking to occur, it is generally accepted that four conditions will exist:

- The subsurface soils are in a relatively loose state,
- The soils are saturated,
- The soils are fine, granular, and uniform, and
- Ground shaking of sufficient intensity should occur to act as a triggering mechanism.

A common secondary hazard of strong ground shaking is the potential for soil liquefaction. Soil liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary but essentially total loss of shear strength due to build-up of excess pore water pressure under the reversing cyclic shear strength associated with earthquake shaking. Dissipation of the excess pore pressures will produce volume changes within the liquefied soil layer, which can manifest at the ground surface as settlement of structures, floating of buried structures, failure of retaining walls, lateral migration (lateral spreading), and extensional ground cracking of liquefied material. Factors known to influence liquefaction include soil type, structure, grain size, relative density, confining pressure, depth to groundwater, and the intensity and duration of ground shaking. Soils most susceptible to liquefaction are saturated, loose, sandy soils. . Clays are generally not susceptible to liquefaction.

Liquefaction analysis was performed based on CPT data assuming a peak ground acceleration of 0.32 g, an earthquake magnitude of 6.7, and empirical methods developed by Robertson and Wride (1998) as summarized by the NCEER Liquefaction Workshop expert panel (Youd, et al., 2001) with modifications proposed by Seed

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(2003). Parikh also evaluated the liquefaction potential evaluation at the project site in accordance with the methods proposed by Youd, et al. (2001) using CPT procedures. The Parikh analysis identified potentially liquefiable alluvial soil layers (i.e., silty sand and poorly-graded sand with silt) ranging between 2.5 to 4 feet thick, varying at depths of 11.5 to 43 feet below ground surface, with a post-liquefaction settlement on the order of 3 to 4 inches. However, the Parikh analysis was performed based on a peak ground acceleration of 0.40g. Kleinfelder has reevaluated liquefaction assessments from CPT-09-001 and CPT-09-002 data , using a peak ground acceleration of 0.32g.

The results of our liquefaction analysis are summarized in the following table. The data is presented in order of the most "western" CPT location (CPT-11-011) to the most "eastern" CPT location.

Table 9.1
Summary of Liquefaction Analysis

CPT No.	Total Depth (Feet)	Depths of Continuous Liquefied Layers (Feet)	Total settlement (Inches)	Approximate Depth of ½-inch Relative Soil Movement (Feet)
CPT-11-011	50	27-28, 34-38	0.92	35
CPT-11-001	90	40 – 41, 88 - 89	0.58	24
CPT-11-002	90	48 – 49, 73 - 75	0.73	60
CPT-11-010	105	33-34	0.26	NA
CPT-09-001	120	11-13, 18-20, 31-32, 43	1.90	21
CPT-09-002	120	19-23	1.34	22
CPT-11-003	90	17 - 19	0.25	NA
CPT-11-004	90	10 - 11, 20 – 26, 69 - 75	1.89	70
CPT-11-005	90	45 – 46	0.28	NA
CPT-11-006	90	54 - 55	0.13	NA
CPT-11-007	90	20 - 40	3.90	37
CPT-11-008	90	34–47, 64-65, 67-68, 77-78	1.85	55
CPT-11-009	50	--	0	NA

The results of our liquefaction analysis are also presented on graphs in Appendix G. Results of our liquefaction analysis indicate the presence of discontinuous layers of potentially liquefiable soils throughout the site. However, a clearly defined continuous

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liquefiable soil layer does not appear to present. Therefore, it is our opinion that the potential for liquefaction at the project site is low.

9.1.2 Seismic Design Criteria

Using magnitude as the criteria, an earthquake with a magnitude of 6.7 should be used as the governing design event. The estimated shear wave velocity of the soils underlying the project site is about 249 m/s and the depth to bedrock is estimated to be greater than 305 meters.

9.1.3 Dynamic Loading on Walls

The existing soil should have a shear wave velocity of at least 249 m/second (820 feet/second). An initial wall soil stiffness of 50 kips/in/foot is recommended. The ultimate lateral resistance that may be applied against retaining walls to resist seismic loading will be dependent on the deflection that occurs (which mobilizes shear resistance in the soil). The figure below presents the ultimate equivalent uniform lateral soil resistance as a function of horizontal strain (deflection/height) for the retaining walls. The maximum resistance for strain in excess of 1.0 percent is 5.0 ksf, when the height of the wall that is buried below the ground surface is greater than or equal to 5½ feet. When the wall height is less than 5½ feet, the maximum equivalent uniform lateral soil resistance shall be reduced proportionately using the formula in accordance with Section 7.8.1 of the Caltrans Seismic Design Criteria (Caltrans, Version 1.6, November 2010).

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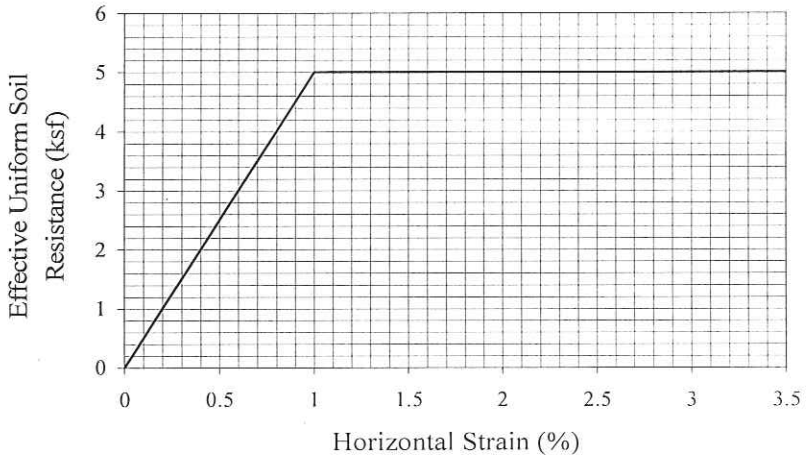
Allowable Lateral Bearing for Seismic Loading
At Abutment Endwalls



9.2 CUT AND EXCAVATIONS

9.2.1 Temporary Excavations

Safe inclinations of temporary cuts and excavations for constructing foundations, drainage improvements, or underground utilities should conform to regulatory requirements and are the contractor's responsibility. Kleinfelder can assist with site specific recommendations based on the specific site conditions. Vertical cuts will require worker protection and/or shoring. Construction equipment and excavated materials should be set back at least 5 feet from the top of temporary slopes or shoring. Problems relating to shallow groundwater, springs, unfavorable bedding planes, or faults are not anticipated to impact temporary cuts. The majority of the cohesive soils, i.e., clays, encountered would classify as Type A per OSHA criteria. The cohesionless soils encountered, i.e., sands, would classify as Type B. Actual site specific classification should be made by the project Geotechnical Engineer during construction operations.

9.2.2 Permanent Cuts

Based on our experience in the project area with fine-grained soil (stiff to very stiff consistency) and coarse-grained soil (medium dense to dense relative density), the following cut slope recommendations have typically been provided in the project area.

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Table 9.2
Cut Slope Recommendations

Cut Slope Depth	Slope Recommendation
<6.5 feet	1:1
6.5 to 13 feet	1.5:1
>13 feet	2:1

Given the inclinations presented above, there would be a modest risk that displacement, and/or movement could occur in the event of a strong seismic ground shaking. If steep cut slopes are planned for the site, proper erosion controls should be designed by the project Landscape Architect.

9.2.3 Rippability

The soils present should be rippable with conventional, well-maintained construction equipment. Bedrock will not be present within the design depth of excavation and blasting will not be required.

9.3 EMBANKMENTS

Embankments are proposed at both ends of the elevated structure. The western embankment, containing the eastbound on-ramp and westbound off-ramp, will extend from Navy Drive to the viaduct structure just south of the BNSF railway. This embankment will vary in width from 75 to 110 feet and be retained by three MSE walls. The MSE walls are approximately 555 feet (retaining wall 1), 790 feet (retaining wall 2) and 83 feet (retaining wall 3) in length and range in height from about 0 feet at navy drive (west) to 42 feet at the viaduct interface (east). A standard type 1 wall (retaining wall 4) ranging in height from 4 to 6 feet and approximately 560 feet long will be located on the south side of Navy Drive.

The eastern embankment for the eastbound and westbound mainline from just east of Fresno Avenue to the Garfield Street Overhead will include sloped embankments. It will measure about 170 feet wide and vary in height from about 0 to 30 feet above the existing ground level, with its highest point just east of Fresno Avenue.

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Where applicable, embankment construction should conform to Sections 19-1, 19-2.02, 19-5, and 19-6 of the latest revision of the Caltrans Standard Specifications.

9.3.1 Embankment Settlement

Review of boring logs indicates that the project site is generally underlain by stiff to very stiff lean clay/silt, interbedded with loose to medium-dense silty sand to a depth of about 40 to 60 feet below existing grade. Below this layer are layers of very stiff to hard silt/lean clay and/or dense to very-dense sandy soils to the maximum depths explored. Groundwater was encountered at a depth of about 5 to 10 feet at the western embankment location below existing ground surface. A settlement analysis for the western approach embankment is presented in Appendix H. The required time for 90 percent consolidation was estimated in this analysis to be between 3 and 4 months. However, based on our experience in the project area, we anticipate the actual time for 90 percent consolidation is between 1 and 2 months. Settlement magnitudes and rates for the eastern approach embankment are anticipated to be significantly less than the western approach embankment as the water table in this area is greater.

Bridge abutment areas are settlement-sensitive. Caltrans practices are such that the remaining total settlement of the abutment embankment should not exceed 0.5 inches prior to the installation of abutments, wingwalls, and pile foundations. We recommend that the actual rate and magnitude of settlement within bridge abutment areas be monitored in the field. Prior to construction, vibrating wire settlement plates and piezometers should be considered for installation. Surface monuments, constructed in accordance with Caltrans Standard Plan A74 or equivalent, should be installed in a timely manner upon completion of the fill placement. Surface monuments should be placed at the south abutment. The actual locations of surface monuments should be determined during grading under the direction of the Geotechnical Engineer.

9.3.2 Ground Improvement Alternatives

Various ground improvement techniques have been used to address long-term consolidation settlement issues for roadway embankment over soft clay. For the proposed project, we considered three alternatives as below:

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- Preloading/One or Two-Stage construction
- Prefabricated Vertical (PV) drains
- Geopier-supported embankments

For the PV drains alternative, we considered 4-inch wide PV drains installed to depths of about 50 to 60 feet below existing grade at center-to-center spacing of 5 to 6 feet. For the geopier-supported embankment alternative, we considered 30-inch diameter geopiers installed to a depth of about 30 feet below the ground surface at a center-to-center spacing of 5 to 7 feet. The three ground improvement alternatives are compared in terms of the magnitude and rate of consolidation settlements and approximate costs, as summarized in the table below. Calculations for these alternatives are presented in Appendix H.

Table 9.3
Ground Improvement Alternatives

Alternative No.	Ground Improvement Technique	Consolidation Settlement [1] (inches)	Time Required for 90 Percent Degree of Consolidation [2] (months)	Approximate Ground Improvement Costs [3] ($)
1	Preloading/One- or Two-stage construction	10-16	3-4	N/A
2	PV drains	10-16	<1	305,000
3	Geopier-supported embankment	4-7	1-2	2,160,000

Notes: 1. Estimated for a 45-foot-high approach embankment (MSE walls), 2. After the end of construction, 3. Current cost estimation as of this writing

Depending on the amount of differential settlements expected, two typical construction techniques are usually used in the construction of MSE walls – one-stage construction and two-stage construction. One-stage construction is usually used in areas where the anticipated differential settlements are within the tolerable limit for the precast panel system (i.e., less than 1 percent or 1 foot of differential settlement along a 100-foot wall length). The two-stage MSE wall system involves initial construction of a reinforced earth wire-faced wall with steel soil reinforcements to the desired height of the proposed embankment. The flexible wire facings are able to tolerate significant amounts of settlement without any adverse impact on the structural integrity and performance of the final wall. Following the completion of the consolidation of the foundation soils, precast concrete facing panels are connected to the wire facings/soil

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reinforcements with an adjustable connection system. After completion of the connection for each course of precast concrete facing panels, the space between the wire facings and concrete panels is then backfilled with free-flowing gravel to complete the structure.

As indicated in the table above, we anticipate that consolidation settlement at the project site will be on the order of 10 to 16 inches, corresponding to a maximum differential settlement of less than 1 percent. Therefore, it is our professional opinion that the one-stage construction technique should be appropriate for the proposed approach embankments (MSE walls). If the two-stage construction technique is considered, on a preliminary basis we anticipate that its cost will be approximately 40 to 50 percent higher than for one-stage construction, and in this case, ground improvement with geopiers might become more cost-effective because it allows one-stage construction of MSE walls.

Construction of western and eastern approach embankments should be appropriately staged for foundation soils to achieve sufficient strength to support the overlying embankment loadings. Time required for 90 percent degree of consolidation for Alternative No. 1 – Preloading is estimated to be about 3 to 4 months. However, based on our experience in the project area, we anticipate the actual time for 90 percent consolidation is between 1 and 2 months. Parikh estimated in their preliminary foundation report dated March 8, 2010 that about 18 months will be required for the total consolidation settlement to occur. Our experience within the project area, however, indicates that the consolidation rate of the underlying soils may be much faster than anticipated by Parikh.

In July 2009, we worked for US Gypsum Company on a project in which a test pad was constructed at the Port of Stockton which is about 2 miles west of the project site. For the US Gypsum project, the test pad was constructed in two stages with a total height of about 21 feet above existing site grade. Foundations soils at the site included soft organic clay and silt. Groundwater was encountered less than 5 feet below the ground surface. Settlements and pore water pressures were monitored for about 3 months after the completion of fill construction. Results of our field monitoring indicate that the total settlement was about 6 to 8 inches, and settlement leveled off within about 1 month.

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A more recent project in which settlement magnitudes and rates were measured was during construction of north and south approach embankments for the Port of Stockton Expressway/BNSF Grade Separation project at the Port of Stockton, located approximately 2 miles southwest of the subject project site. Construction of approach embankments for the BNSF grade separation project was performed from August to November 2011 to a maximum embankment height of 42 feet. Because near surface native soils were identified as being organic, soft and compressible, the settlement time for primary consolidation to occur was a concern along with the magnitude of total settlement at each embankment location. Also, because of the soft nature of the native soils and shallow depth to groundwater (approximately 5 feet), the rate at which fill was placed and its effect on subsurface pore pressures were a concern. However, instrumentation of the embankments and native foundation soil using vibrating wire piezometers, vibrating wire settlement cells, and settlement cells indicated the predicted soil behavior by others (magnitude and settlement rate) was very conservative. Piezometer data indicated excess pore pressures dissipated quickly after fill placement, in most cases over night, allowing fill to be placed nearly continuously without causing significant decreases in foundation soil strength. Additionally, maximum total settlement of approach embankments during and after construction ranged between approximately 5 to 10 inches.

Settlement of embankments sometimes causes damage to nearby properties and utilities. A conservative assessment of embankment and MSE wall loading, modeled as a finite width (110 feet wide) and infinite length strip load, indicates that assuming a maximum total potential settlement of 10 to 16 inches inches, the potential for settlement induced by the embankment reduces the further away from the crest of the embankment and/or toe of MSE Wall as follows:

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Table 9.4
Estimated Induced Settlement Reduction

Distance from Crest of Embankment/ Toe of MSE Walls	Induced Settlement (Based on 10 inches maximum settlement)	Induced Settlement (Based on 16 inches maximum settlement)
10 feet	5 inches	8 inches
25 feet	2 ½ inches	4 inches
50 feet	1 inch	1 ½ inches
75 feet	½ inch	¾ inch
100 feet	--	¼ inch

We recommend that mitigation measures for embankment/MSE wall load induced settlement of nearby structures and utilities be based on a maximum settlement of 10 inches. A review of project layout plans indicate that existing structures, left after embankment and MSE Wall construction along the proposed project alignment, will be at least 100 feet away from the crest of embankments/toe of MSE walls. Therefore embankment/MSE wall loading induced settlement for these structures are not a significant concern. Where utilities are located within the distance from the crest of proposed embankments and or toe of MSE walls indicated in Table 9.4, consideration should be given to providing flexible connections that compensate for the anticipated induced settlement. Once settlement is complete, these flexible connections may be replaced with fixed/rigid connections.

It can be concluded that the preloading/one-stage construction alternative is the most cost-effective technique for the proposed project. We understand that the construction schedule may allow for significant time for the construction of approach embankments and application of ground improvement techniques. Therefore, we recommend that preloading/one-stage construction be considered for the proposed project to mitigate the potential consolidation settlements under approach embankment loadings.

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9.3.3 Embankment Slope Stability

Per a review of preliminary plans, it appears that planned eastern approach embankment slope gradients are generally 2:1. Given the height of the planned embankments and that the fills will be constructed of engineered fill, it is our opinion that stability of these slopes should not be a concern. Also, since the fills will be constructed on fairly competent foundation soils, the planned embankment slopes are considered stable with regard to deep-seated failure. The general cohesive nature of the anticipated embankment soil may lead to surface erosion of slope faces if the slopes are not planted with vegetation and properly maintained. If non-cohesive soil is used for the embankment fills, additional erosion control measures may be required. No surface water should be allowed to pond near the tops of slope or discharge over the slope face.

9.4 MSE WALLS

As discussed previously, MSE walls are planned for the western approach embankment. The proposed MSE walls were analyzed for external and global stability in accordance with Caltrans guidelines (Caltrans Bridge Design Specification, Section 5, Retaining Walls 2004). The results of our conservative analysis, presented in Appendix I, indicates the walls will be satisfactory with regard to external and global stability provided the following general design parameters summarized below in Table 9.5 are used. Traffic loads and temporary construction loads should be considered in the design of MSE reinforced walls.

Table 9.5
Design Parameters for MSE Walls

Zones	Moist Unit Weight (pcf)	Friction Angle (degrees)	Cohesion (psf)
Reinforced Soil	125	35	0
Retained Soil	125	34	0
Foundation Soil	120	30	0

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Based on an minimum wall embedment depth of 2 feet, and a reinforcement length equal to 70 percent of the Design Wall Height, the following allowable bearing capacities are provided in Table 9.6 for MSE wall design. Factors of safety for bearing capacity all were greater than 3.5 (factor of safety of 2.0 required) indicating that wall reinforcement lengths equal to the minimum requirement of 70 percent of Design Wall Height will be satisfactory.

Table 9.6
Allowable Bearing Capacity for MSE Walls

MSE Design Wall Height	Allowable Bearing Capacity
16 feet	2.5 tsf
22 feet	3.5 tsf
28 feet	4.6 tsf
30 feet	5.0 tsf
34 feet	5.6 tsf
36 feet	6.0 tsf
38 feet	6.3 tsf
42 feet	7.0 tsf

Backfill for the reinforced soil mass should be free from organic material or other unsuitable material, and should conform to the gradation and property requirements in Table 9.7.

Table 9.7
Requirements for Reinforced Soils

Gradation Requirements		Property Requirements
Sieve Size	Percentage Passing	
2 inches	100	pH of 4.5 to 9, sand equivalent of 30 or more, and plasticity index of 10 or less
#4	50 - 80	
#40	0 - 30	
#200	0 - 15	

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9.5 GABION BASKET GRAVITY WALL

As mentioned previously, the existing "LKQ" property basin located within the approximate footprint of proposed Bents 4 and 5 of the eastbound viaduct structure will be partially filled to be used as a temporary staging area for bridge construction. An 11 feet high (exposed height) gabion basket gravity wall is proposed to be placed to retain the crushed drain rock which will be used to partially fill the drainage basin. A stability analysis was performed for the proposed gabion basket gravity wall using a spreadsheet developed by a leading gabion basket manufacturer (Terra Aqua Inc.), and is provided in Appendix J. The following recommendations are provided for gabion basket gravity wall design.

1.) Standard Details for Gabion Baskets should be prepared in accordance with Caltrans 20.0 Standard Plan Sheets D100A and D100B

2.) At a depth of 11 feet, foundation soil will likely be in a wet, pumping condition. To allow for a firm foundation for wall construction, place a 12 inch leveling layer of granular stone fill wrapped in a Mirafi 600x geotextile fabric or equivalent at the base of the gabion wall.

3.) To provide additional factor of safety against sliding, place the gabion gravity retaining wall on a 6 degree batter toward the retained slope.

4.) The depth of "scour" for the basin is unknown. For initial design, assume a minimum 2 feet foundation embedment depth to accommodate "scour."

5.) Retaining wall backfill should be free draining (drain rock), and filter fabric should be placed between the gabion wall and the backfill soil interface. This will result in the drain rock being thoroughly encapsulated within the filter fabric. Drainrock should consist of clean, durable stone having 100 percent passing the 1-inch sieve and zero percent passing the No. 4 sieve. Synthetic filter fabric should conform to the requirement in Section 88 "Engineering Fabrics" of the Caltrans Standard Specifications. The upper 12 inches of engineered backfill above the drainrock backfill should consist of native soil to reduce surface drainage into the drainrock backfill.

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6.) Recommended Stone Fill Gradation for Gabion Baskets is as follows:

Table 9.8
Stone fill Gradation for Gabion Basket Wall

Galvanized/Galfan Gabion 8 x 10 cm	4 inches to 8 inches

Specific gravity for stone fill should be no less than 2.5.

7.) To provide the maximum resistance to soil forces, the gabion baskets should be placed with the length dimension of the gabion basket unit running from the back of the gabion wall to the front face of the gabion wall.

8.) The wall configuration may be designed as shown on the gabion basket gravity wall calculation spreadsheet presented in Appendix J. A conceptual wall layout is also provided on the spreadsheet where gabion width and heights are indicated. In summary, the following wall dimensions are recommended:

Table 9.9
Recommended Gabion Basket Wall Dimensions

Wall Row	Width (ft)	Height (ft)	Offset from Toe (ft)
1	9.0	3.0	0.0
2	7.5	3.0	0.0
3	6.0	3.0	1.5
4	4.5	3.0	4.5
5	3.0	3.0	4.5

9.6 TYPE 1 RETAINING WALLS

Type 1 retaining walls (RW 1 and RW 5) may be designed per criteria presented on sheets B3-1 and B3-8 of the Caltrans standard plans published in 2010. Allowable bearing pressure for the proposed retaining walls will be at least equivalent to the design toe pressures presented on Caltrans Standard Plan Sheet B3-1.

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9.7 DRAINAGE

Roadway and subsurface drainage design should meet general specifications presented in Chapter 830 and 840 of the Caltrans Highway Design Manual. Depth and spacing of pipe underdrians should be designed based on a soil class of "Silty Clay" as presented on Table 842.4 of the Caltrans Highway Design Manual. If alternative pipe culverts (APC) are considered for inlet and outlet design, they should be designed to accommodate potential corrosion to buried metals as indicated in Section 9.10 of this report. Corrosion test results presented in Section 9.10 may be used as input into the AltPipe computer program (http://dap1.dot.ca.gov/design/altpipe/). For steel bars used in concrete, a minimum 3-inch concrete over all steel bars will provide adequate protection against corrosion for the bars. No culverts shall be allowed for construction beneath MSE walls.

The general nature of the anticipated embankment soil may lead to surface erosion of slope faces if not planted with vegetation and properly maintained. No surface water should be allowed to pond near the tops of slope or discharge over the slope face.

Retention basins (two total) are proposed to be constructed adjacent the eastbound on-ramp and westbound off-ramp of the western approach embankment. Detention Basins (two total) are proposed to be constructed at the east end of the alignment, located adjacent (southwest and southeast) of Fresno Avenue.

Over time, very little percolation should be assumed through the future drainage basin bottoms due to silting and organic build-up unless a vigorous program is followed to maintain the basins by frequent cleaning and discing to reduce silt build-up. Subsurface drain trenches and/or drywells should be equipped with filtered intakes. Consideration should be given to performing percolation tests in order to ascertain the percolation characteristics of basin/ditch soils.

Based on our experience in the project area, (see Appendix K, Excerpts from Boggs Tract Drainage Study) it has been our experience that percolation rates for near surface soils are very low in the project area. Presented below are estimates of percolation for soils encountered in our field explorations.

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Table 9.10
Estimated Percolation Rates

Soil Type	Percolation (gal/ft^2/day)
Lean Clay to Fat Clay	1
Sandy and Clayey Silt	3 to 5
Silty Sand	5 to 25
Clean Sand	25 to more than 100

9.8 CULVERTS

Culverts can be designed and constructed using standard plans and specifications, and no specific geotechnical investigation is required per Caltrans guidelines. Recommendations for alternate pipe culverts using the AltPipe software is presented in Appendix M.

9.9 PAVEMENTS

9.9.1 Pavement Subgrade

Kleinfelder performed R-value testing on samples taken from 5 locations on the site. The approximate locations where the R-value test samples were obtained are presented in Appendix L. Results of these tests are summarized below.

Table 9.11
R-Value Test Results

Sample Location	Lab Sample Description	R-Value
2345 Navy Drive, South side	Brown Clay with Sand	6
Navy Drive & Tillie Lewis	Brown Clay with Sand	9
Scotts Avenue, Northwest side	Brown Clayey Sand	25
Hazelton Avenue	Brown Clayey Sand	26
Del Norte	Brown Silty Sand	34

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We understand that the fill material will be obtained from local sources in the project vicinity. Based on our experience with materials from local aggregate plants, R-values typically range from 15 to 30. If the fill material is obtained from areas such as Roberts Island, the R-value may be as low as less than 5. Laboratory tests will need to be performed prior to and during construction to determine the R-Value of the roadway subgrade soil.

9.9.2 Asphalt Concrete Pavement Sections

Based on anticipated Traffic Index (TI) provided by Fehr & Peers, the following asphalt concrete (AC) pavement sections are presented for consideration.

Table 9.12
Recommended Asphalt Concrete Pavement Sections

TI	R-Value	AC (inches)	AB (Inches)	Comments
9	25*	5½	14½	Cul-de-sacs within project limits – Hazelton, Ventura and Del Norte
13	5*	8	30½	
13.5	5*	8 ½	32	Tillie Lewis
14	5*	8 ½	33½	
15	5*	Charter Way	36 ½	Charter Way

*R-Value results from laboratory testing. It should be noted that if during the construction the near-surface soils encountered are different from what was tested in our laboratory, additional lab testing should be performed to estimate the proper R-Values for the pavement design.

For geogrid reinforced pavement section alternatives, please see the attached design sheet (Appendix L) prepared by Tensar Group.

Recommendations for temporary full depth AC pavements during construction on Route 4 and Navy Drive are presented in Table 9.12. Calculations for the estimated AC pavement thicknesses are provided in Appendix L.

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Table 9.13
Recommended Temporary Full Depth Asphalt Concrete Pavement Sections

TI (20 Year Life)	Temporary Pavement Section for 2 Year Life		Temporary Pavement Section for 3 Year Life	
	TI (2 Year Life)	Temporary AC (inches)	TI (3 Year Life)	Temporary AC (inches)
11	8.5	14	9	14 ½
11.5	9	14 ½	9.5	15
12	9.5	15	10	16
12.5	9.5	15 ½	10	16 ½
13	10	16	10.5	17
13.5	10.5	17	11	17 ½
14	11	17 ½	11.5	19
14.5	11	18	12	19 ½
15	11.5	19	12	20
15.5	12	19 ½	12.5	20 ½
16	12.5	20 ½	13	21
16.5	12.5	21	13.5	22 ½

9.9.3 Portland Cement Concrete Pavement Sections

The following table presents the recommended Portland Cement Concrete (PCC) pavement sections for the project. PCC pavements are designed based on the assumption of that subgrade soils have a R-value between 10 and 40. Calculations are presented in Appendix L.

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Table 9.14
Recommended Portland Cement Concrete Pavement Sections

Pavement Sections	With Lateral Support (feet)			Without Lateral Support (feet)			TI	Comments
JPCP	0.95	0.95	---	1.05	1.05		13.5 to 14	For the subgrade soils with an R-Value less than 10, a Tensar TX geogrid or equivalent should be placed under AS over the subgrade. For roadway placed over new embankment, the top 3 feet of embankment fill (pavement soil subgrade) should be constructed on imported material with a minimum R-value of 10.
CRCP	---		0.85	---	---	0.95		
LCB	0.5	---	---	0.5	---	---		
HMA-A	---	0.5	0.5	---	0.5	0.5		
AS	0.7	0.7	0.7	0.7	0.7	0.7		
JPCP	1.05	1.05	---	1.20	1.20		15.5 to 16	
CRCP	---	---	0.95	---	---	1.10		
LCB	0.5	---	---	0.5	---	---		
HMA-A	---	0.5	0.5	---	0.5	0.5		
AS	0.7	0.7	0.7	0.7	0.7	0.7		
JPCP	1.10	1.10	---	1.25	1.25	---	16.5 to 17	
CRCP	---	---	0.95	---	---	1.10		
LCB	0.5	---	---	0.5	---	---		
HMA-A	---	0.5	0.5	---	0.5	0.5		
AS	0.7	0.7	0.7	0.7	0.7	0.7		

JPCP: Jointed Plain Concrete Pavement; HMA-A: Hot Mix Asphalt (Type A);
CRCP: Continuously Reinforced Concrete Pavement, AS: Class 2 Aggregate Base;
LCB: Lean Concrete Base

Per Caltrans Highway Design Manual (Section 613.3), all TI's should be rounded to the nearest 0.5. Caltrans recommends rigid pavement for design of roadways with TI's greater than 15 (Highway Design Manual-Section 633.1).

9.10 CORROSION POTENTIAL

Caltrans Corrosion Guidelines state that the Department considers a site to be corrosive to foundation elements if one or more of the following conditions exist for the soil and/or water samples taken at the site (Caltrans):

- Chloride concentration is greater than or equal to 500 ppm
- Sulfate concentration is greater than or equal to 2000 ppm
- pH is 5.5 or less

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The corrosivity of soils towards buried ferrous metals is commonly evaluated based on the correlations provided in Table 9.15:

Table 9.15
Corrosivity vs. Soil Resistivity

Soil Resistivity	Corrosivity
0 to 1,000 ohm-cm	Severely corrosive
1,000 to 2,000 ohm-cm	Corrosive
2,000 to 10,000 ohm-cm	Moderately corrosive
Over 10,000 ohm-cm	Mildly corrosive

Corrosion tests were performed on composite soil samples in accordance with Caltrans and EPA procedures to evaluate the corrosivity of the soils to buried concrete and ferrous metals. The corrosion test results of the soil samples are summarized in Table 9.16. The subsurface soils at the project site are potentially severely to moderately corrosive to buried ferrous metals.

Table 9.16
Results of Soil Corrosion Tests

Sample Locations	Sample Depth (feet)	USCS Soil Type	pH	Sulfate Content (ppm)	Chloride Content (ppm)	Minimum Resistivity (ohm-cm)
Near R-11-003	1-4	Sandy Lean Clay (CL)	8.1	460	230	570
Near R-11-004	1–4	Sandy Lean Clay (CL)	7.5	240	340	470
Near R-11-006	1-4	Sandy Lean Clay (CL)	7.8	400	140	530
Near R-11-008	1-4	Sandy Lean Clay (CL)	7.4	89	17	2,100
Near R-11-009	1-4	Sandy Lean Clay (CL)	7.5	59	18	2,000

The above corrosion test results indicate that the soils near the western portion of the project site are not corrosive to buried concrete. As we indicated above, the soils may be corrosive to buried metals. For steel bars used in concrete, a minimum 3-inch concrete over all steel bars will provide adequate protection against corrosion for the bars.

Kleinfelder has performed these soil corrosivity tests as requested by the client. These tests are only an indicator of soil corrosivity. You may wish to retain a

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competent corrosion engineer to design corrosion protection systems appropriate for the project.

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10.0 MATERIAL SOURCES

Embankments are anticipated to be constructed from materials imported to the site or native subgrade materials. The import fill materials should conform to the requirements for engineered fill in accordance with the appropriate section of Section 19 of the Caltrans standard specifications, latest edition. Embankment or structural fill should be placed and compacted in accordance with the appropriate portions of Section 19 of the Caltrans standard specifications, latest edition. Potential sources of import soil are listed below.

Knife River Corporation
655 W. Clay Street
Stockton, CA 95206
(209) 948-0302

Teichert Construction
265 Val Dervin Parkway
Stockton, CA 95206
(209) 983-2300

Granite Construction Company
10500 S. Harlan Road
French Camp, CA 95201
(209) 982-4750

Commercial suppliers of asphalt, concrete, and aggregate products in the vicinity also include the following:

Source	Location
Quality Rockmaster Mobile Mix, Inc.	345 N. San Joaquin Street, Stockton, CA
A Plus Ready Mix	820 Flint Avenue, Stockton, CA
Pacific Mobile Ready Mix	4047 Oak Shores Drive, Stockton, CA
7-11 Materials	2714 Stagecoach Road, Stockton, CA
A & A Concrete Supply	4035 E Mariposa Road, Stockton, CA

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11.0 MATERIAL DISPOSAL

Disposal of aerially deposited lead (ADL) and other contaminated material is beyond the scope of this report.

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12.0 CONSTRUCTION CONSIDERATIONS

12.1 CONSTRUCTION ADVISORIES

Groundwater at the site is at a depth that should not affect construction; however, during or after extended periods of rainfall, perched water or zones of elevated moisture may be present in localized areas within the areas of construction. Such conditions may produce excessive moisture content or areas with poor vehicle/equipment mobility. Temporary dewatering is not anticipated to be necessary.

12.2 CONSTRUCTION CONSIDERATIONS THAT INFLUENCE DESIGN

Generally, there are no construction-related factors that influenced the analysis or design comments provided in Section 9 of this report. Considerations that could possibly influence construction are basically those listed in Section 12.1.

12.3 CONSTRUCTION MONITORING AND INSTRUMENTATION

Construction monitoring or instrumentation programs may be required as discussed in Section 9.3.1.

12.4 HAZARDOUS WASTE CONSIDERATIONS

If suspected hazardous substances are encountered during construction, the signatories of the reports should be contacted immediately. Detailed Site Investigation (DSI) have been completed by Kleinfelder for various parcels within the project area and the results are presented under separate cover (reports).

12.5 DIFFERING SITE CONDITIONS

Differing site conditions are those not encountered during exploration, latent physical conditions that may arise, or unknown conditions that exist that were not observed and included in the Geotechnical Design Report. If such differing conditions are observed during site operations, it is imperative that the designers, resident engineer, contractor, and/or geotechnical engineer be notified as soon as possible.

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13.0 RECOMMENDATIONS AND SPECIFICATIONS

If designers have questions or concerns with any of these recommendations, or if conditions are found to be different during construction, contact the Stockton office of Kleinfelder. Additional fieldwork, analysis, or changes in recommendations may be required.

The following specific recommendations are presented to assist in design and construction of proposed roadway improvements.

- Embankment foundation soil is adequate to support the proposed embankment without adverse consequences.
- Final unpaved slopes should be 2:1 or flatter.
- Final cut or embankment slopes should be protected from erosion by proper management of drainage, planting drought resistant vegetation, and necessary maintenance.
- Grading and earthwork should be performed in general accordance with the latest edition of the Standard Specifications for the State of California Department of Transportation (Caltrans Standard Specifications or CSS).
- Soil characteristics are compatible with use of Caltrans Standard Plans or Caltrans Bridge Standard Details Sheets for retaining structures.

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14.0 LIMITATIONS

Please be advised that we are performing a professional service and that our conclusions and recommendations are professional opinions only. All work done and all recommendations made are in accordance with generally accepted Geotechnical engineering principles and practices. No warranty, expressed or implied, of merchantability or fitness, is made or intended in connection with our work.

Foundation recommendations contained in this report were developed based on limited information regarding the subsurface soil conditions obtained from our field exploratory borings, CPTs and geoprobes, previous investigations at the project site, and our experience in the vicinity of the project site. It is possible that soil or groundwater conditions could vary between or beyond the points explored. If soil or groundwater conditions are encountered during construction that differ from those described herein, the client is responsible for ensuring that Kleinfelder is notified immediately so that we may reevaluate the recommendations of this report. If the scope of the proposed construction, including the estimated structure loads and the design depths or locations of the foundations, changes from that described in this report, the conclusions and recommendations contained in this report are not considered valid unless the changes are reviewed and the conclusions of this report are modified or approved in writing by Kleinfelder.

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15.0 REFERENCES

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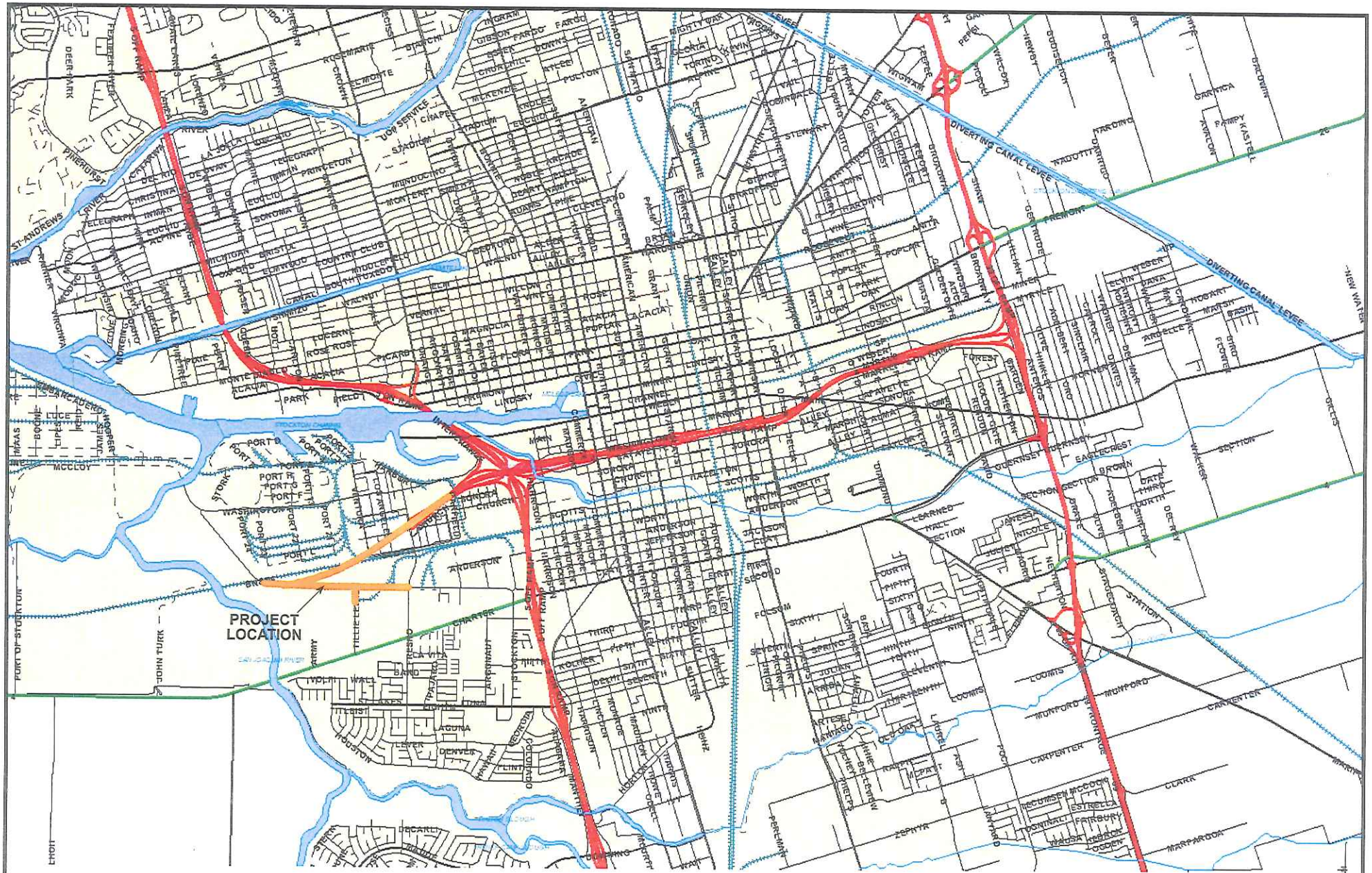

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SITE VICINITY MAP
ROUTE 4 CROSSTOWN FREEWAY EXTENSION
SAN JOAQUIN COUNTY, CALIFORNIA

Plate

1

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CPT-09-001
CPT-09-002
A-09-001
CPT-11-002
R-11-003
A-09-002
CPT-11-010
R-11-004
CPT-11-003
R-11-005
CPT-11-004
R-11-006
CPT-11-005
A-09-003
R-11-007
CPT-11-006
CPT-11-007
CPT-11-008
R-11-009
CPT-11-009
R-11-010
O-11-01
O-11-02
O-11-03
O-11-04
O-11-05

R-11-013 DENOTES NUMBER AND APPROXIMATE LOCATIONS OF BORINGS PERFORMED FOR THIS INVESTIGATION

O-11-01 DENOTES NUMBER AND APPROXIMATE LOCATIONS OF GEOPROPES ADVANCED FOR DEBCO DSI INVESTIGATION

A-09-001 DENOTES NUMBER AND APPROXIMATE LOCATION OF FIELD EXPLORATIONS PERFORMED BY PARIKH

CPT-11-010 DENOTES NUMBERS AND APPROXIMATE LOCATIONS OF CPT'S ADVANCED FOR THIS INVESTIGATION

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DATE PRODUCED:	5/4/12
PROJ. No.:	111904.G01
DRAWN BY:	G. GOMEZ
CHECKED BY:	C. HENDERSON
FILE NAME:	STO10D135.CAD

EXPLORATION LOCATION MAP
ROUTE 4 CROSSTOWN FREEWAY EXTENSION
SAN JOAQUIN COUNTY, CALIFORNIA

PLATE
2

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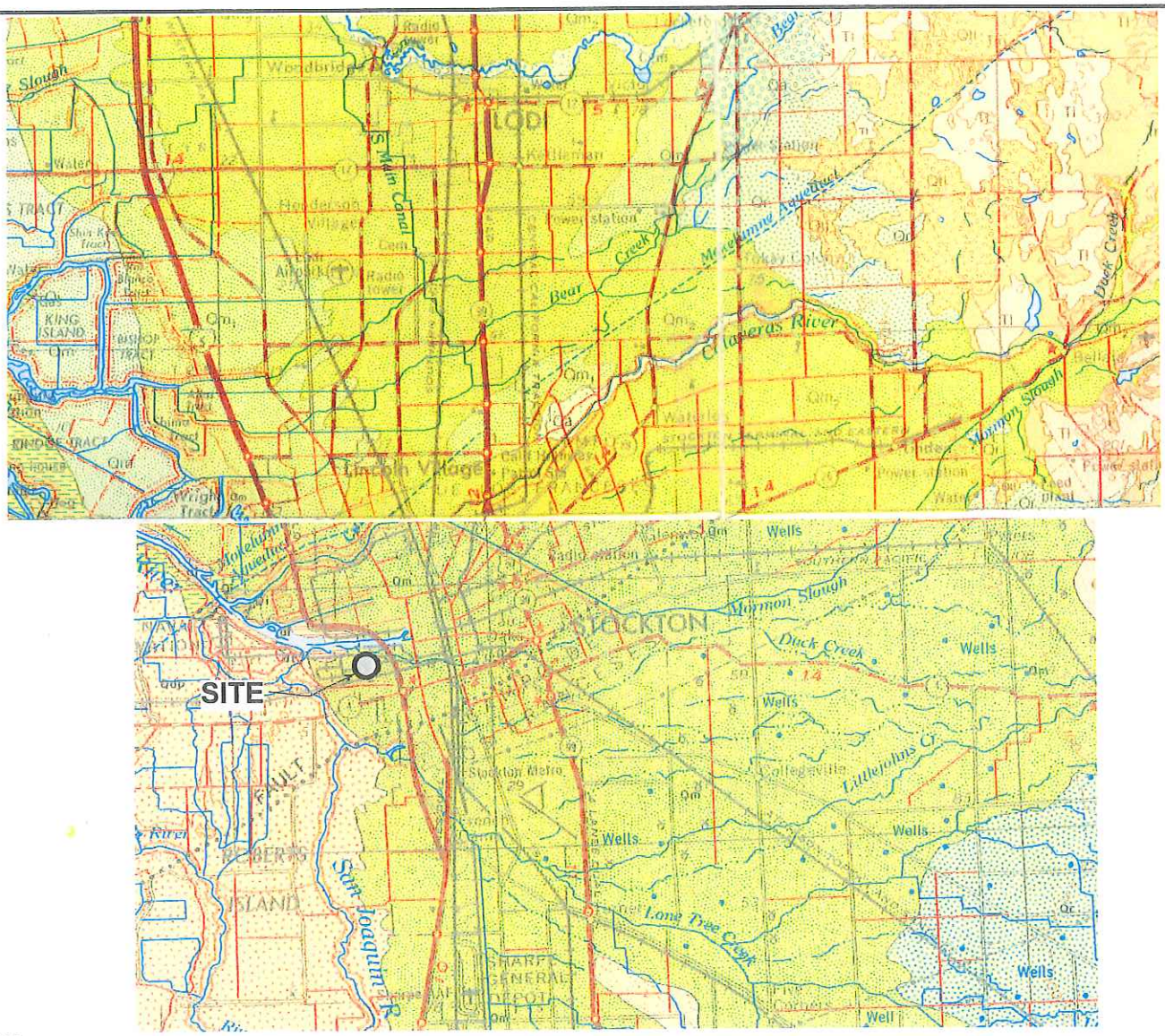

KEY

O	Alluvium
t	Mine and dredge tailings
Qa	Levee and channel deposits
Qt	Intertidal deposits *(Peaty mud)*
Qm	Modesto Formation *(Alluvium)*
Qr	Riverbank Formation *(Alluvium)*
Qmr	Modesto-Riverbank Formations *(Arkosic alluvium)*
Qtl	Turlock Lake Formation *(Sand, silt, and gravel)*
Qmg	North Merced Gravel *(Thin pediment veneer)*
Tl	Laguna Formation *(Consolidated alluvial deposits)*
Tm	Mehrten Formation *(Andesitic conglomerate, sandstone, and breccia)*
Qdp	Dos Palos Alluvium
Qm1 / Qm2	Upper and lower members

SCALE: 1:125,000

NOTES: THE FAULT SHOWN IS IDENTIFIED AS THE STOCKTON FAULT. PUBLISHED REFERENCES DO NOT IDENTIFY THIS FAULT AS ACTIVE OR POTENTIALLY ACTIVE.

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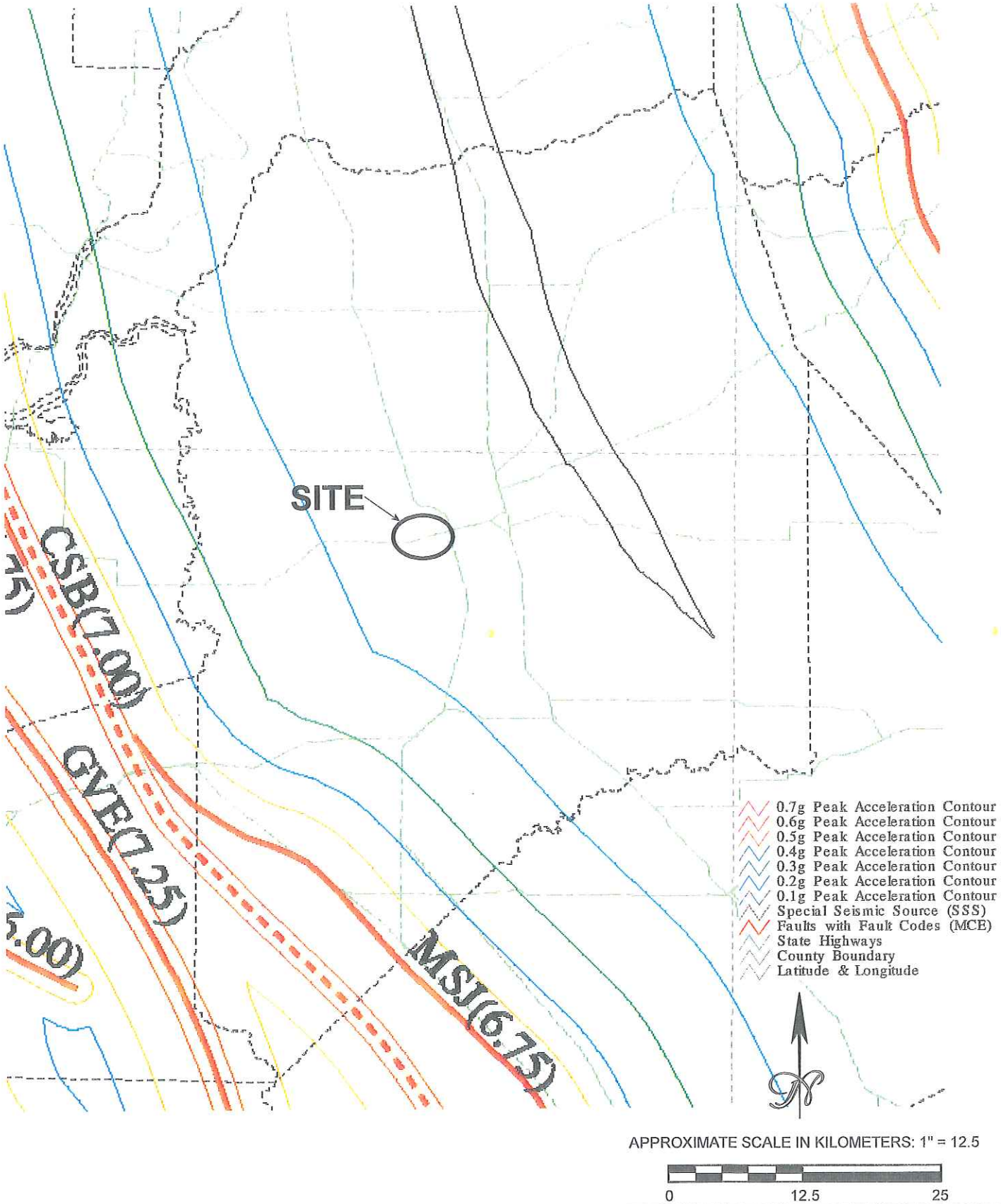
Project Number:	111904.G01
Graphic Date:	7/29/10
Graphic By:	G. GOMEZ
Checked By:	Y.L.
File Name:	GEO.fh11

GEOLOGIC MAP
ROUTE 4 CROSSTOWN FREEWAY EXTENSION
SAN JOAQUIN COUNTY, CALIFORNIA

Plate
3



APPROXIMATE SCALE IN KILOMETERS: 1" = 12.5

0 12.5 25

KLEINFELDER

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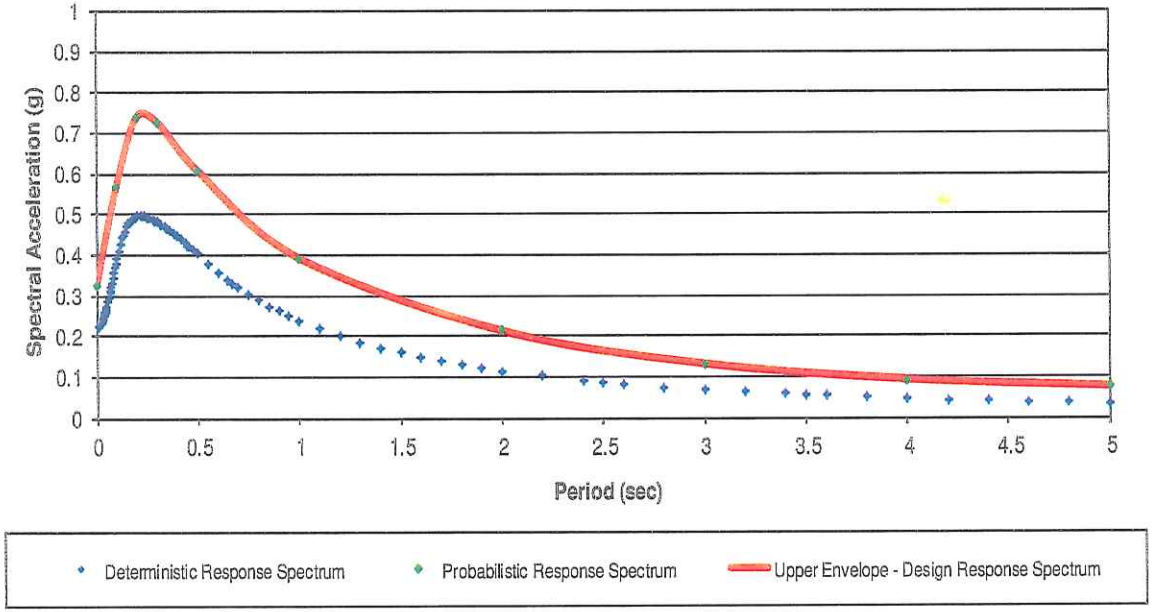
Project Number: 111904	
Graphic Date: 7/11/11	
Graphic By: G. GOMEZ	
Checked By: J.W.	
File Name: SEISMIC.fh11	

FAULT MAP
ROUTE 4 CROSSTOWN FREEWAY EXTENSION
SAN JOAQUIN COUNTY, CALIFORNIA

Plate

4



Period (sec)	Spectral Acceleration (g)	
	Probabilistic	Deterministic
0	0.3244	0.225
0.1	0.5696	0.391
0.2	0.7382	0.497
0.3	0.7281	0.483
0.5	0.6083	0.404
1	0.3919	0.238
2	0.2124	0.114
3	0.13101	0.068
4	0.09244	0.046
5	0.07534	0.034

- Caltrans Geotechnical Services Design Manual (Version 1.0, August 2009)
- ARS Online (V1.0.4, April 2011)
- Governing Deterministic Fault: Great Valley fault 7—Mw 6.7
- V_{S30} : 249 m/s

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Bright People. Right Solutions.
www.kleinfelder.com

PROJECT NO.	111904.G01
DATE:	April 29, 2011
DRAWN BY:	J. Wetenkamp
CHECKED BY:	C. Henderson

Preliminary Acceleration Response Spectrum (ARS) Spectra Comparison

Route 4 Crosstown Freeway Extension
San Joaquin, California

PLATE
5

APPENDIX A
RESPONSE TO REVIEW COMMENTS

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Bright People. Right Solutions.

File No. 111904.G01
May 4, 2012

Mr. Keith Meyer
Rajappan & Meyer Consulting Engineers, Inc.
1030 Leigh Avenue, Suite 100
San Jose, CA 95216

Subject: **Response to Caltrans District 10 Constructability Review Dated September 29, 2011**

Reference: "Geotechnical Design and Materials Report, 65% Design Submittal, Route 4 Crosstown Freeway Extension, San Joaquin County, California," Kleinfelder File No. 111904.G01/STO11R162

Dear Mr. Meyer:

As requested, this letter presents our responses to constructability review comments provided by Caltrans District 10 for the referenced report.

Comment No. 1 *"Geotechnical Services only review the Geotechnical Design portion of the report. The Materials report needs to be reviewed by District Materials."*
Response – Noted.

Comment No. 2 *"Caltrans no longer use the "Deterministic PBA Map" anymore for seismic design. Please revise or delete sentence.*
Response – The sentence has been removed from our revised report dated May 4, 2012.

Comment No. 3 *"On the 5th line, "Caltrans Seismic Design Criteria (Version 1.4, 2006)" is no longer used. Please use Version 1.6, Nov 2010, instead."*
Response – The current version (Version 1.6, Nov. 2010) was used in our analysis. The language was corrected in our revised report dated May 4, 2012.

Comment No. 4 *"Soil Profile Type is no longer used for seismic design for Caltrans. Please revise or delete."*
Response – The soil type has been removed from the table in our revised report dated May 4, 2012.

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2001 Arch-Airport Road, Suite 100, Stockton, CA 95206 **p** | 209.948.1345 **f** | 209.234.4700

Comment No. 5 *"For the last sentence, liquefaction analysis should be performed on soil as deep as it needs to be even if it's deeper than 50 ft. This general statement does not comply with Caltrans general practice."*
Response – Our liquefaction analysis was performed for depths greater than 50 feet. The sentence was corrected in our revised report dated May 4, 2012.

Comment No. 6 *"At the end of Section 9.1.1, please clearly state what the liquefaction potential of the site is (.e. none, low, moderate, or high)"*
Response – Based on our analysis, the liquefaction potential at the site is low. This is now clearly stated in our revised report dated May 4, 2012.

Comment No. 7 *"For the last sentence, Caltrans now uses the Seismic Design Criteria Version 1.6, 2010. Please revise."*
Response – The latest version (Version 1.6, November 2010) for seismic design criteria was used in our analysis. This language has been included in our revised report, dated May 2012.

Comment No. 8 *"For steep cut slopes, appropriate erosion control method shall be implemented. It should be designed by the Landscape Architect, but please briefly mention whether or not such measures are taken."*
Response – Noted. The language has been included in our revised report dated May 4, 2012.

Comment No. 9 *"Please address what level of impact the ground settlements will be on the nearby structures. Is there any remediation needed for the potential impacts on the neighboring residents and businesses?"*
Response – Potential impacts to neighboring residents and business are discussed in Section 9.3.2 of our revised report, dated May 4, 2012. A review of project layout plans indicate that existing structures, left after embankment and MSE Wall construction along the proposed project alignment, will be at least 100 feet away from the crest of embankments/toe of MSE walls. Therefore embankment/MSE wall loading induced settlement for these structures are not a significant concern. Where utilities are located within the distance from the crest of proposed embankments and or toe of MSE walls indicated in Table 9.4, consideration should be given to providing flexible connections that compensate for the anticipated induced settlement. Once settlement is complete, these flexible connections may be replaced with fixed/rigid connections

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Comment No. 10 *"On the 3rd line, please state which Caltrans guidelines is being referred to. (CT Bridge Design Aids?)"*
Response – The proposed MSE walls were analyzed for external and global stability in accordance with Caltrans guidelines (Caltrans Bridge Design Specification, Section 5, Retaining Walls 2004). This language has been included in our revised report dated May 4, 2012.

Comment No. 11 *"Allowable bearing capacities need to be provided on the table."*
Response – Allowable bearing capacities have been provided in a revised table which related wall height to bearing capacity. This language has been included in our revised report dated May 4, 2012.

Comment No. 12 *"No culvert should be running through the MSE walls"*
Response – Language prohibiting the use of culverts through MSE walls has been included in our revised report dated May 4, 2012.

Comment No. 13 *"When drainage designs are finalized, please specify the internal drainage design as well as the external design to ensure proper drainage has been implemented."*
Response – Language which describes the proposed drainage design for the project has been included in our revised report dated May 4, 2012.

We trust this information meets your needs at this time. If you have any questions or if we can be of further assistance, please contact our office.

Sincerely,

KLEINFELDER WEST, INC.

REGISTERED PROFESSIONAL ENGINEER
HAMID REZA GHARIB PARSA
No. 74120
CIVIL
STATE OF CALIFORNIA

Hamid "Saeed" Parsa, G.E. No. 74120
Project Engineer

REGISTERED PROFESSIONAL ENGINEER
CARL FRANCISCO HENDERSON
No. 2886
GEOTECHNICAL
STATE OF CALIFORNIA

Carl Henderson, Ph.D., G.E. No. 2886
Project Manager

SGP:lr

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DISTRICT 10
PROJECT DEVELOPMENT REVIEW RECORD
FOR CONSTRUCTIBILITY REVIEWS (30%, 60% & 95% STAGES)

CO-RTE-PM SJ-4 14.4/14.8, T14.6/R15.7
EA / PROJECT ID# 0S1101 / 10 0000 0229
PROGRAM Prop 1B & Measure K
DESCRIPTION SR-4/Crosstown Freeway Ramp Ext. Proj.
DATE ROUTED 8/18/2011
PS&E STAGE 30% ☐ 60% ■ 95% ☐

PROJECT ENGINEER Rajappan&Meyer / Jes Padda (O. Sight)
DESIGN SENIOR Michael Hutchison
PROJECT MANAGER Scott Guidi
RETURN BY 9/20/2011 (4 week duration)
RETURN TO Jes Padda
DATE OF C.R. MEETING 9/20/11, 1:15 PM Room 53/66

REVIEWER Carolyn Zhen-Ru DATE RECEIVED 9/26/2011 DATE RETURNED 9/29/2011

Plans	Specs	Estimate	Sheet/ Page No.	Station/ Section		Comments* (Reviewer makes comments on white line. PE responds on shaded line.) If this form is used electronically, and the comment/response is lenghty, the row height may need to be increased to show the entire comment/response.
☐	☐	☐	GDR		1.	Geotechnical Services only review the Geotehnical Design portion of the report. The Materials report needs to be reviewed by District Materials.
☐	☐	☐	GDR Page 18 Sec 8.6		2.	Caltrans no longer use the "Deterministic PBA Map" anymore for seismic design. Please revise or delete sentence.
☐	☐	☐	GDR Page 19, 1st paragr		3.	On the 5th line, "Caltrans Seismic Design Criteria (Version 1.4 2006)" is no longer used. Please used Version 1.6, Nov 2010, instead.
☐	☐	☐	GDR Page 19 Table 8.2		4.	Soil Profile Type is no longer used for seismic design for Caltrans. Please revise or delete.
☐	☐	☐	GDR Page 20, 2nd Parag		5.	For the last sentence, liquefaction analyses should performed on soil as deep as it needs to be even if it's deeper than 50 ft. This general statement does not comply with Caltrans general practice.
☐	☐	☐	GDR Page 22, 1st paragr		6.	At the end of Section 9.1.1, please clearly state what the liquefaction potential of the site is.(i.e. none,low, moderate, or high)
☐	☐	☐	GDR Page 22, Sec 9.1.3		7.	For the last sentence, Caltrans now uses the Seismic Design Criteria Version 1.6, 2010. Please revise.
☐	☐	☐	GDR Page 24, Sec 9.2.2		8.	For steep cut slopes, appropriate erosion control method shall be implemented. It should be designed by Lanscape Architect, but please briefly mention whether or not such measures are taken.
☐	☐	☐	GDR Page 25, Sec 9.3.1		9.	Please address what level of impact the ground settlements will be on the nearby structures. Is there any remediation needed for the potential impacts on the neighboring residents and businesses?
☐	☐	☐	GDR Page 28, Sec 9.4		10.	On the 3rd line, please state which Caltrans guidelines is being referred to. (CT Bridge Design Aids?)
☐	☐	☐	GDR Page 29, Table 9.4		11.	Allowable bearing capacities need to be provided on the table.

* Comments NOT requiring follow-up (grammatical, spelling and drafting comments) may be addressed directly on PS & E documents.

PROJECT DEVELOPMENT REVIEW RECORD
FOR CONSTRUCTIBILITY REVIEWS (30%, 60% & 95% STAGES)

CO-RTE-PM SJ-4 14.4/14.8, T14.6/R15.7

EA (6 digits) 0S1101 / 10 0000 0229

Plans	Specs	Estimate	Sheet/ Page No.	Station/ Section	Comments* (Reviewer makes comments on white line. PE responds on shaded line.) If this form is used electronically, and the comment/response is lenghty, the row height may need to be increased to show the entire comment/response.
☐	☐	☐	GDR Page 29, Sec 9.5		12. No Culvert should be running through the MSE walls.
☐	☐	☐	GDR Page 29, Sec 9.5		13. When drainage designs are finalized, please specify the internal drainage design as well as the external design to ensure proper drainage has been implemented.
☐	☐	☐			
☐	☐	☐			
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☐	☐	☐			

* Comments NOT requiring follow-up (grammatical, spelling and drafting comments) may be addressed directly on PS & E documents.

APPENDIX B
LOGS OF BORINGS AND CONE PENETRATION TESTS PROFILES
(KLEINFELDER INVESTIGATION)

LIST OF ATTACHMENTS

The following are attached and complete this appendix.

Log of Test Borings Sheets
Cone Penetrometer Test Profiles

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Log of Test Boring Sheets

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (2010)

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	4	14.4/14.8 T14.6/R15.7		

5/23/12
GEOTECHNICAL PROFESSIONAL DATE

REGISTERED PROFESSIONAL ENGINEER
Carl Henderson
No. 2886
Exp. 6/30/2013
GEOTECHNICAL
STATE OF CALIFORNIA

PLANS APPROVAL DATE

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of electronic copies of this plan sheet.

San Joaquin Council of Governments
555 E. Weber Avenue
Stockton, CA. 95202

Kleinfelder, Inc.
2001 Arch-Airport Rd., Suite 100
Stockton, CA. 95206

CEMENTATION

Description	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

BOREHOLE IDENTIFICATION

Symbol	Hole Type	Description
Size	A	Auger Boring
Size	R P	Rotary drilled boring Rotary percussion boring (air)
Size	R	Rotary drilled diamond core
Size	HD HA	Hand driven (1–inch soil tube) Hand Auger
	D	Dynamic Cone Penetration Boring
	CPT	Cone Penetration Test (ASTM D 5778–95)
	O	Other (Geoprobe)

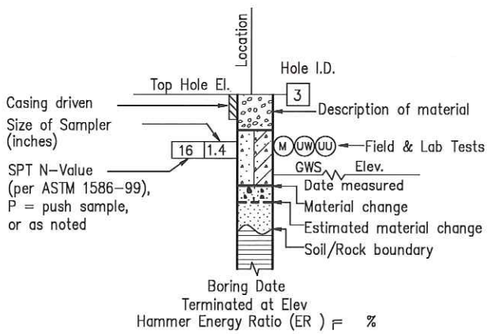
Note: Size in inches.

CONSISTENCY OF COHESIVE SOILS

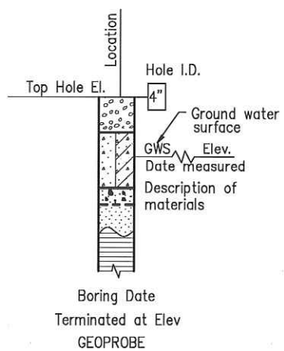
Description	Unconfined Compressive Strength (tsf)	Pocket Penetrometer Measurement (tsf)	Torvane Measurement (tsf)	Field Approximation
Very Soft	< 0.25	< 0.25	< 0.12	Easily penetrated several inches by fist
Soft	0.25 to 0.50	0.25 to 0.50	0.12 to 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 to 1.0	0.50 to 1.0	0.25 to 0.50	Penetrated several inches by thumb with moderate effort
Stiff	1 to 2	1 to 2	0.50 to 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2 to 4	2 to 4	1.0 to 2.0	Readily indented by thumbnail
Hard	> 4.0	> 4.0	> 2.0	Indented by thumbnail with difficulty

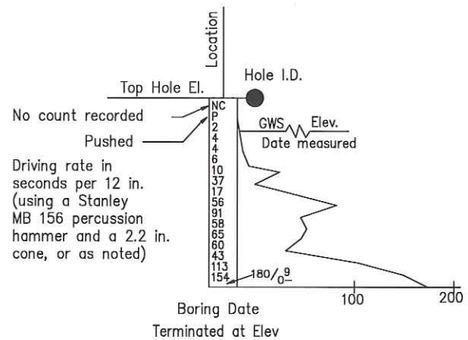
PLASTICITY OF FINE-GRAINED SOILS

Description	Criteria
Nonplastic	A 1/8–inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.

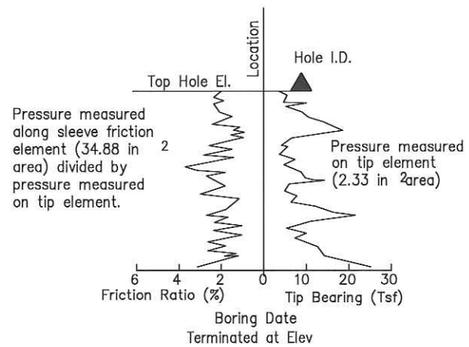


ROTARY BORING





DYNAMIC CONE PENETRATION BORING



CONE PENETRATION TEST (CPT) BORING

X DESIGN OVERSIGHT X SIGN OFF DATE	DRAWN BY CHECKED BY	G. GOMEZ Y. LI	H. PARSA FIELD INVESTIGATION BY: DATE: FEBUARY 2011		PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	H. PARSA PROJECT ENGINEER	BRIDGE NO. POST MILES

CROSSTOWN FREEWAY SR4 IMPROVEMENTS

SOIL LEGEND No. 1

GS LOTB SOIL LEGEND SHEET 1 (ENGLISH) (REV. 07/16/10)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: 1454
PROJECT NUMBER & PHASE: 1000020414 CONTRACT NO.: X

DISREGARD PRINTS BEARING EARLIER REVISION DATES →

REVISION DATES			SHEET	OF
3/2/11	5/4/12	5/23/12		

FILE => $REQUEST

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (2010)

GROUP SYMBOLS AND NAMES

Graphic/Symbol	Group Names	Graphic/Symbol	Group Names
GW	Well-graded GRAVEL Well-graded GRAVEL with SAND	CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
GP	Poorly-graded GRAVEL Poorly-graded GRAVEL with SAND		
GW-GM	Well-graded GRAVEL with SILT Well-graded GRAVEL with SILT and SAND	CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
GW-GC	Well-graded GRAVEL with CLAY (or SILTY CLAY) Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)		
GP-GM	Poorly-graded GRAVEL with SILT Poorly-graded GRAVEL with SILT and SAND	ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
GP-GC	Poorly-graded GRAVEL with CLAY (or SILTY CLAY) Poorly-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)		
GM	SILTY GRAVEL SILTY GRAVEL with SAND	OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND		
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND	OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
SW	Well-graded SAND Well-graded SAND with GRAVEL		
SP	Poorly-graded SAND Poorly-graded SAND with GRAVEL	CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
SW-SM	Well-graded SAND with SILT Well-graded SAND with SILT and GRAVEL		
SW-SC	Well-graded SAND with CLAY (or SILTY CLAY) Well-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)	MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
SP-SM	Poorly-graded SAND with SILT Poorly-graded SAND with SILT and GRAVEL		
SP-SC	Poorly-graded SAND with CLAY (or SILTY CLAY) Poorly-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)	OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
SM	SILTY SAND SILTY SAND with GRAVEL		
SC	CLAYEY SAND CLAYEY SAND with GRAVEL	OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY ORGANIC elastic SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL		
PT	PEAT	OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND
	COBBLES COBBLES and BOULDERS BOULDERS		

FIELD AND LABORATORY TESTING

- (C) Consolidation (ASTM D 2435)
- (CL) Collapse Potential (ASTM D 5333)
- (CP) Compaction Curve (CTM 216)
- (CR) Corrosivity Testing (CTM 643, CTM 422, CTM 417)
- (CU) Consolidated Undrained Triaxial (ASTM D 4767)
- (DS) Direct Shear (ASTM D 3080)
- (EI) Expansion Index (ASTM D 4829)
- (M) Moisture Content (ASTM D 2216)
- (OC) Organic Content–% (ASTM D 2974)
- (P) Permeability (CTM 220)
- (PA) Particle Size Analysis (ASTM D 422)
- (PI) Plasticity Index (AASHTO T 90)
Liquid Limit (AASHTO T 89)
- (PL) Point Load Index (ASTM D 5731)
- (PP) Pocket Penetrometer
- (R) R-Value (CTM 301)
- (SE) Sand Equivalent (CTM 217)
- (SG) Specific Gravity (AASHTO T 100)
- (SL) Shrinkage Limit (ASTM D 427)
- (SW) Swell Potential (ASTM D 4546)
- (UC) Unconfined Compression–Soil (ASTM D 2166)
Unconfined Compression–Rock (ASTM D 2938)
- (UU) Unconsolidated Undrained Triaxial (ASTM D 2850)
- (UW) Unit Weight (ASTM D 4767)

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	4	14.4/14.8 T14.6/R15.7		

5/4/12
GEOTECHNICAL PROFESSIONAL DATE

REGISTERED PROFESSIONAL ENGINEER
Carl Henderson
No. 2886
Exp. 6/30/2013 GEOTECHNICAL
STATE OF CALIFORNIA

PLANS APPROVAL DATE

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of electronic copies of this plan sheet.

San Joaquin Council of Governments
555 E. Weber Avenue
Stockton, CA. 95202

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2001 Arch-Airport Rd., Suite 100
Stockton, CA. 95206

APPARENT DENSITY OF COHESIONLESS SOILS

Description	SPT N_{60} (Blows / 12 in.)
Very Loose	0 – 5
Loose	5 – 10
Medium Dense	10 – 30
Dense	30 – 50
Very Dense	Greater than 50

MOISTURE

Description	Criteria
Dry	No discernable moisture
Moist	Moisture present, but no free water
Wet	Visible free water

PERCENT OR PROPORTION OF SOILS

Description	Criteria
Trace	Particles are present but estimated to be less than 5%
Few	5% – 10%
Little	15% – 25%
Some	30% – 45%
Mostly	50% – 100%

PARTICLE SIZE

Description		Size (in.)
Boulder		Greater than 12
Cobble		3 – 12
Gravel	Coarse	3/4 – 3
	Fine	1/5 – 3/4
Sand	Coarse	1/16 – 1/5
	Medium	1/64 – 1/16
	Fine	1/300 – 1/64
Silt and Clay		Less than 1/300

X
DESIGN OVERSIGHT

X
SIGN OFF DATE

DRAWN BY: G. GOMEZ
CHECKED BY: Y. LI

H. PARSA
FIELD INVESTIGATION BY:

DATE: FEBUARY 2011

PREPARED FOR THE
STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

H. PARSA
PROJECT ENGINEER

BRIDGE NO.

POST MILES

CROSSTOWN FREEWAY SR4 IMPROVEMENTS

SOIL LEGEND No. 2

GS LOTB SOIL LEGEND SHEET 2 (ENGLISH) (REV. 07/16/10)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: 1454
PROJECT NUMBER & PHASE: 1000020414 CONTRACT NO.: X

FILE => $REQUEST

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES			SHEET	OF
3/2/11	5/4/12	5/23/12		

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5	14.4/14.8 T14.6/R15.7		

BENCHMARK:
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GEOTECHNICAL PROFESSIONAL — 5/23/12 DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER — Carl Henderson No. 2886 Exp. 6/30/2013 GEOTECHNICAL — STATE OF CALIFORNIA

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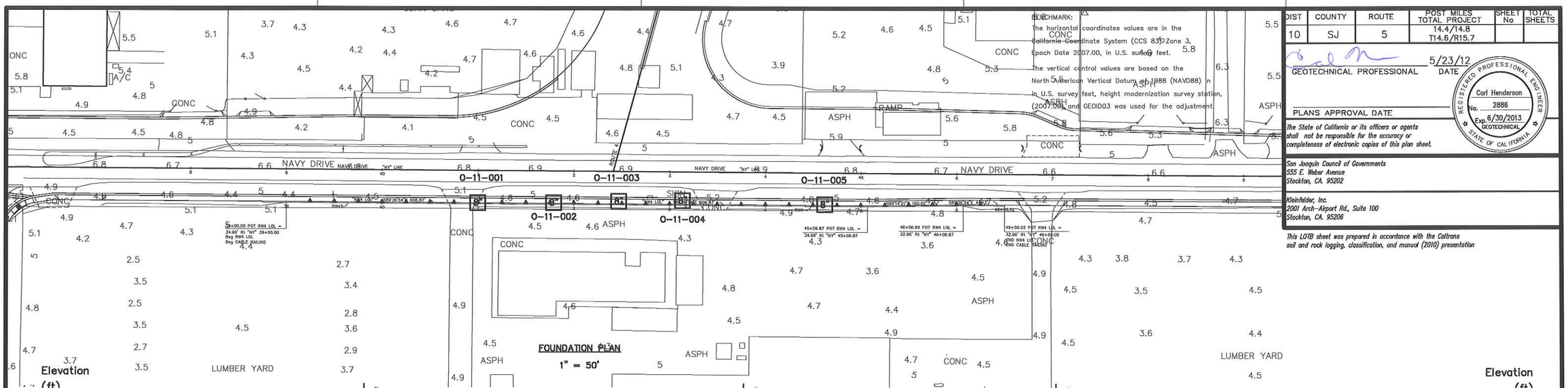

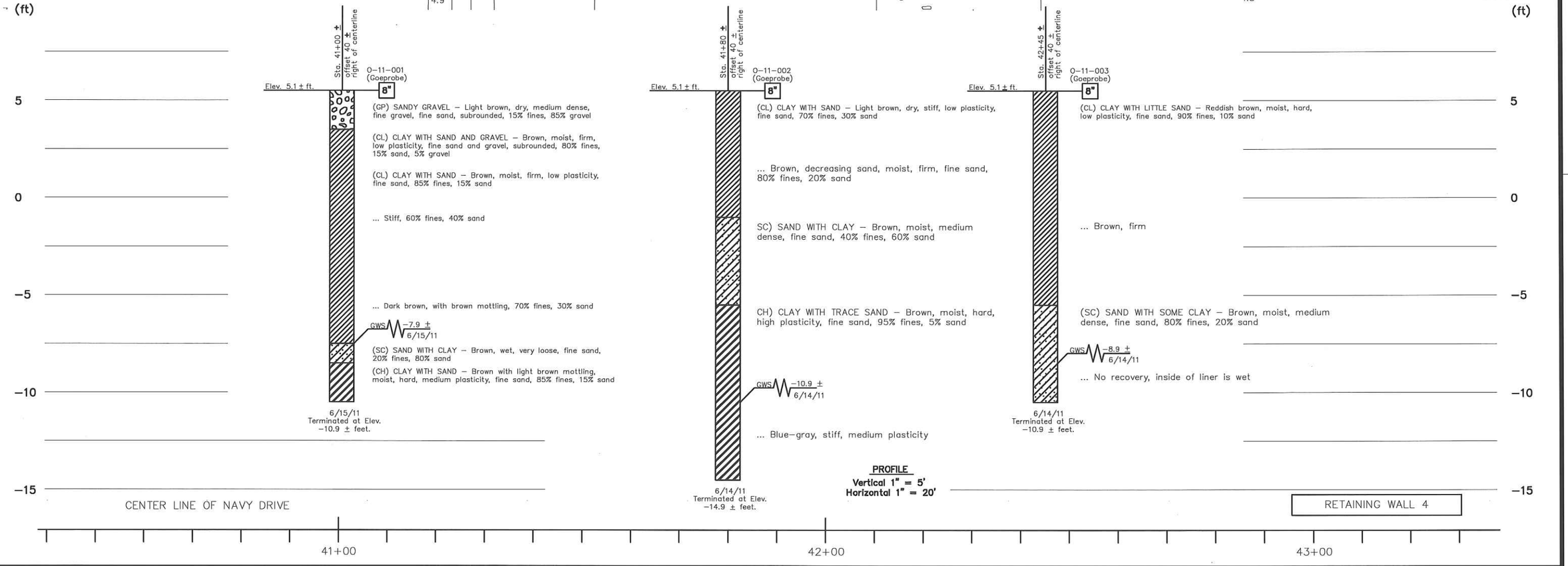

X DESIGN OVERSIGHT	DRAWN BY	G. GOMEZ	N. BERNER FIELD INVESTIGATION BY:		PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	H. PARSA PROJECT ENGINEER	BRIDGE NO.
X SIGN OFF DATE	CHECKED BY	C. HENDERSON	DATE: JUNE 2011				POST MILES

CROSSTOWN FREEWAY SR4 IMPROVEMENT

LOG OF TEST BORINGS 1 of 10

GS LOTB SOIL LEGEND SHEET 1 (ENGLISH) (REV. 07/16/10)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: 1454
PROJECT NUMBER & PHASE: 1000020414 CONTRACT NO.: X

DISREGARD PRINTS BEARING EARLIER REVISION DATES →

REVISION DATES				SHEET	OF
3/2/11	5/4/12	5/23/12			

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5	14.4/14.8 T14.6/R15.7		

GEOTECHNICAL PROFESSIONAL — DATE 5/23/12

PLANS APPROVAL DATE

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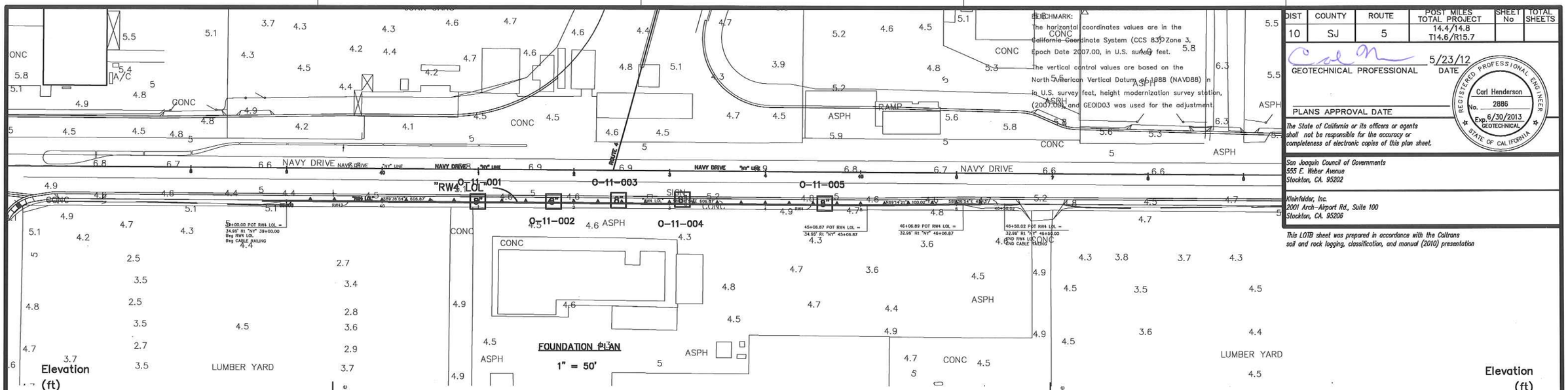


Elevation (ft)

Sta. 43+10 ± offset 40 ± right of centerline

Elev. 5.1 ± ft.

0-11-004 (Geoprobe)

8"

(GP) SANDY GRAVEL – Brown, dry, medium dense, subrounded, fine grained, asphalt pieces, 20% sand, 80% gravel

(CL) CLAY WITH SAND – Brown, moist, firm, low plasticity, fine sand, 80% fines, 20% sand

... With little gray mottling, hard, 95% fines, 5% sand

GWS −5.9 ± 6/14/11

(SC) SAND WITH CLAY – Brown, wet, loose, subrounded, fine sand, minor fine gravel, 20% fines, 75% sand, 5% gravel

(CH) CLAY WITH SAND – Brown with dark brown mottling, moist, hard, medium plasticity, fine sand, 85% fines, 15% sand

6/14/11
Terminated at Elev. −10.9 ± feet.

Sta. 44+60 ± offset 40 ± right of centerline

Elev. 5.1 ± ft.

0-11-005 (Geoprobe)

8"

(SP) GRAVELLY SAND – Light brown, dry, medium dense, subrounded, fine sand, coarse gravel, cobble size concrete, 85% sand, 15% gravel

(CL) CLAY WITH SAND – Brown, moist, firm, low plasticity, fine sand, 70% fines, 30% sand

... Dark brown

GWS −6.9 ± 6/14/11

(SC) SAND WITH CLAY – Brown, moist, medium dense, subrounded, minor gravel, fine sand, fine gravel, 20% fines, 75% sand, 5% gravel

(GP) GRAVEL WITH TRACE SAND – Gray, wet, medium dense, subrounded, fine gravel, fine sand, 10% fines, 90% gravel

6/14/11
Terminated at Elev. −10.9 ± feet.

Elevation (ft)

5
0
−5
−10
−15

PROFILE
Vertical 1" = 5'
Horizontal 1" = 20'

CENTER LINE OF NAVY DRIVE

43+00 44+00 45+00

RETAINING WALL 4

X DESIGN OVERSIGHT	DRAWN BY	G. GOMEZ	N. BERNER FIELD INVESTIGATION BY:		PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	H. PARSA PROJECT ENGINEER	BRIDGE NO.
X SIGN OFF DATE	CHECKED BY	C. HENDERSON	DATE: JUNE 2011				POST MILES

CROSSTOWN FREEWAY SR4 IMPROVEMENT

LOG OF TEST BORINGS 2 of 10

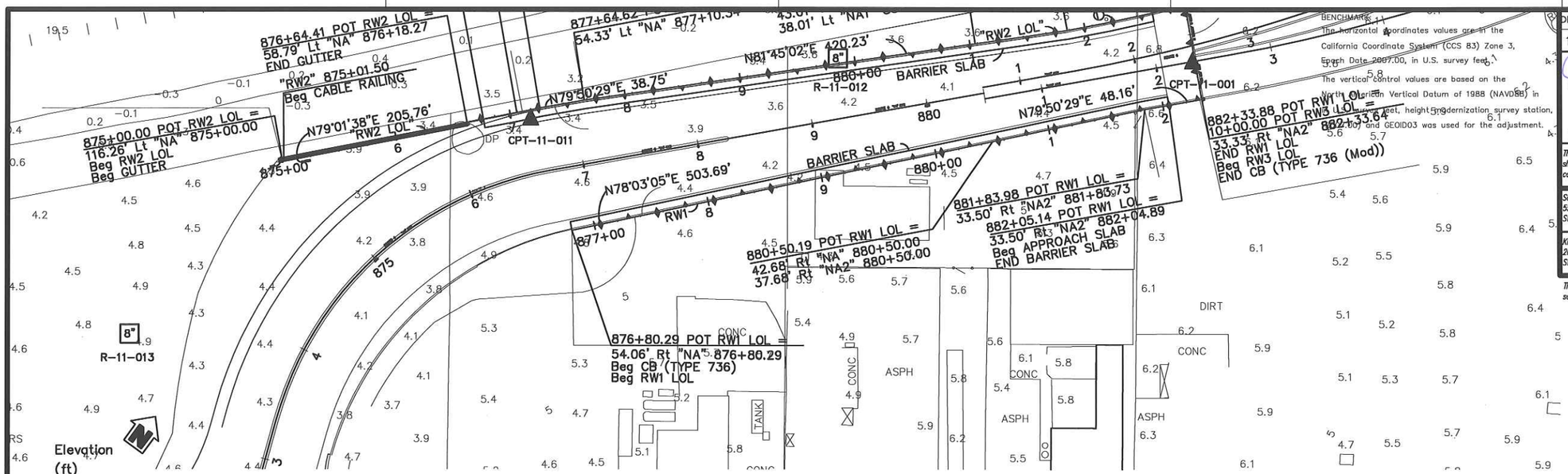
GS LOTB SOIL LEGEND SHEET 1 (ENGLISH) (REV. 07/16/10)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: 1454
PROJECT NUMBER & PHASE: 1000020414 CONTRACT NO.: X

DISREGARD PRINTS BEARING EARLIER REVISION DATES →

REVISION DATES				SHEET	OF
3/2/11	5/4/12	5/23/12			



FOUNDATION PLAN

1" = 50'

BENCHMARKS (+)
The horizontal coordinates values are in the California Coordinate System (CCS 83) Zone 3, Epoch Date 2007.00, in U.S. survey feet.
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DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5	14.4/14.8 T14.6/R15.7		

5/23/12
GEOTECHNICAL PROFESSIONAL DATE

REGISTERED PROFESSIONAL ENGINEER
Carl Henderson
No. 2886
Exp. 6/30/2013
GEOTECHNICAL
STATE OF CALIFORNIA

PLANS APPROVAL DATE

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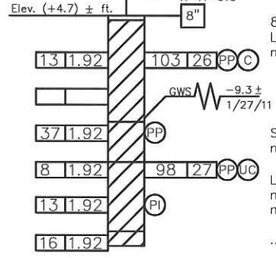


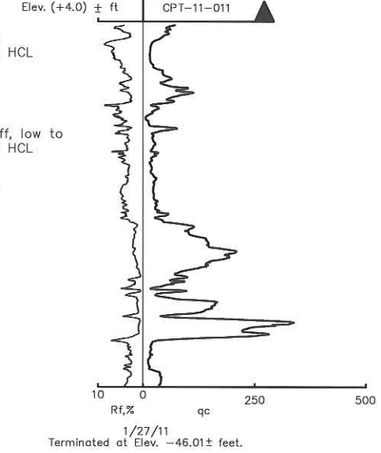
8" Concrete
LEAN CLAY (CL) – Light brown, moist, very stiff, medium plasticity, and toughness, no reaction to HCL

SANDY LEAN CLAY (CL) – Brown, moist, very stiff, low to medium plasticity, and toughness, no reaction to HCL

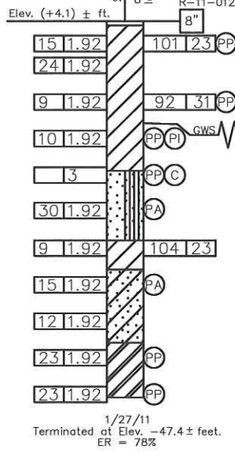
LEAN CLAY (CL) – Light brown, moist, very stiff, medium plasticity, medium to high toughness, no reaction to HCL

... Green, medium toughness, no dilatancy

1/27/11
Terminated at Elev. −26.8 ± feet.
ER = 78%



1/27/11
Terminated at Elev. −46.01± feet.



At surface, 3" gravel and sand road base
SANDY LEAN CLAY (CL) – Light brown, moist, firm to hard, medium plasticity, medium toughness, slow dilatancy, no reaction to HCL

Greenish brown

POORLY GRADED SAND WITH SILT (SP-SM) – Green, moist, fine grained, no cementation
... Dense, fine to coarse grained, 13% fines

SANDY LEAN CLAY (CL) – Green, moist, low plasticity, low toughness, rapid dilatancy, no reaction to HCL
CLAYEY SAND (SC) – Green, moist, medium dense, fine to medium grained, weak cementation, 17% fines

FAT CLAY (CH) – Green, moist, hard, high plasticity, high toughness, no dilatancy, no reaction to HCL

1/27/11
Terminated at Elev. −47.4 ± feet.
ER = 78%

PROFILE
Vertical 1" = 10'
Horizontal 1" = 50'

CENTER LINE OF RT 4

RETAINING WALLS (MSE) 1, 2 & 3

871+00 872+00 873+00 874+00 875+00 876+00 877+00 878+00 879+00 880+00 881+00 882+00 883+00 884+00 885+00 886+00

X DESIGN OVERSIGHT	DRAWN BY	G. GOMEZ	M, Shubert FIELD INVESTIGATION BY:		PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	H. PARSA PROJECT ENGINEER
X SIGN OFF DATE	CHECKED BY	C. HENDERSON	DATE: January, 2011			

BRIDGE NO.
POST MILES

CROSSTOWN FREEWAY SR4 IMPROVEMENT

LOG OF TEST BORINGS 3 of 10

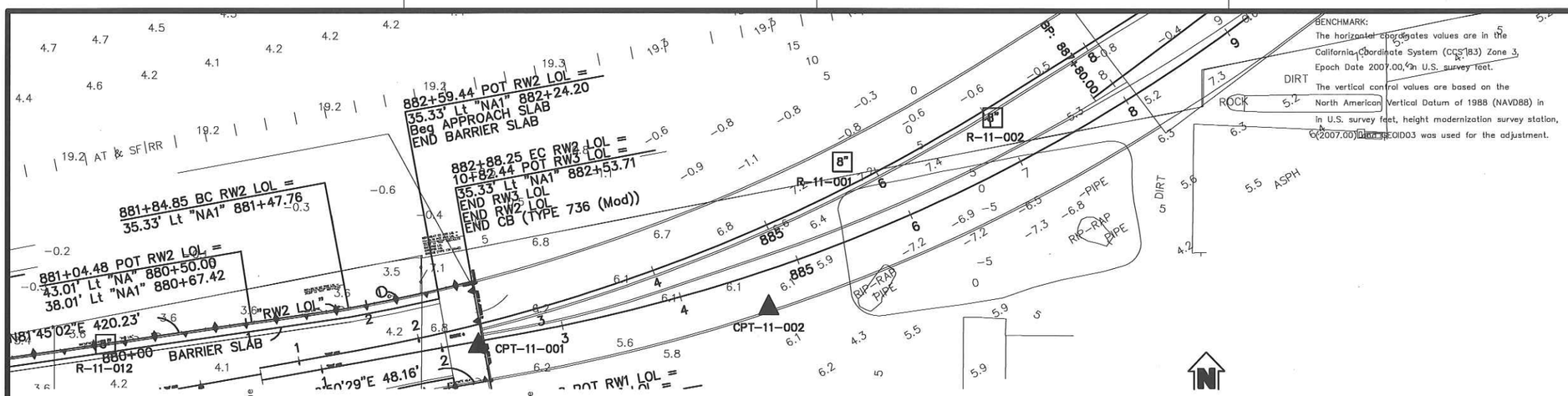
GS LOTB SOIL LEGEND SHEET 1 (ENGLISH) (REV. 07/16/10)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: 1454
PROJECT NUMBER & PHASE: 1000020414 CONTRACT NO.: X

DISREGARD PRINTS BEARING EARLIER REVISION DATES →

REVISION DATES			SHEET	OF
3/2/11	5/4/12	5/23/12		



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DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5	14.4/14.8 T14.6/R15.7		

GEOTECHNICAL PROFESSIONAL — 5/23/12 DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER — Carl Henderson — No. 2886 — Exp. 6/30/2013 GEOTECHNICAL — STATE OF CALIFORNIA

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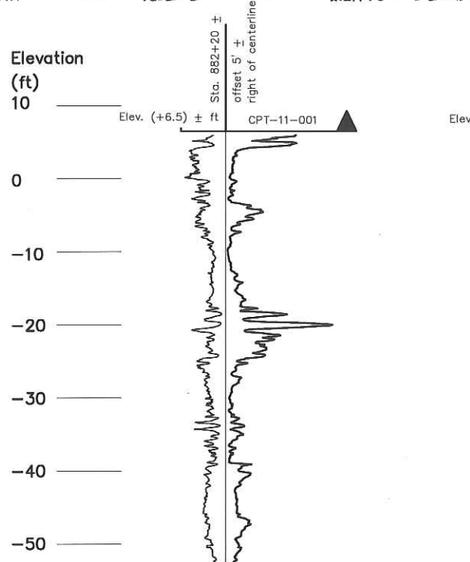
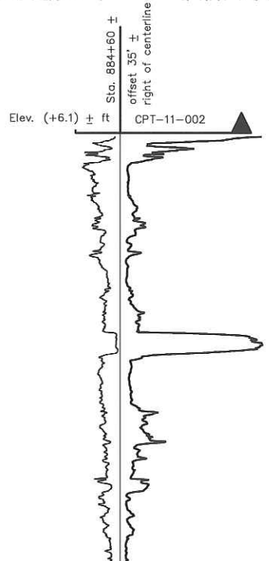
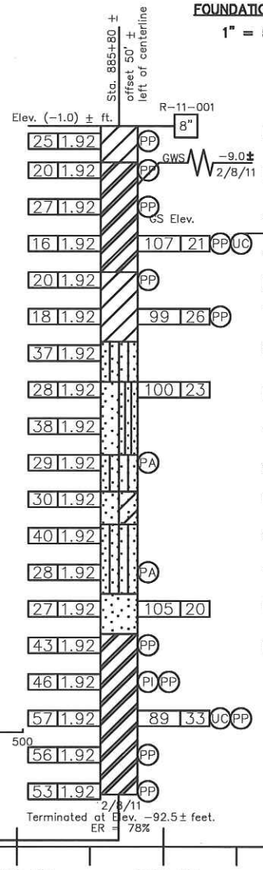




FOUNDATION PLAN
1" = 50'



LEAN CLAY (CL) – Light brown, moist, very stiff, medium to high plasticity, high toughness, no dilatancy

FAT CLAY (CH) – Brown, moist, hard, high plasticity, high toughness, no dilatancy

... With fine sand

... No sand

LEAN CLAY (CL) – Brown, moist, hard, medium to high plasticity, medium toughness, no dilatancy

... Green, high plasticity

SILTY SAND (SM) – Green, moist, medium dense, fine to medium grained, low cementation, 20% fines

POORLY GRADED SAND WITH SILT (SP–SM) – Green, moist, medium dense, fine to medium grained, weak cementation, 10% fines

... About 15% Fines, dense

SILTY SAND (SM) – Greenish gray, moist, medium dense, fine to medium grained, 18% fines

POORLY GRADED SAND WITH CLAY (SP–SC) – Green, moist, dense, fine to medium grained, weak cementation, 10% fines

SILTY SAND (SM) – Green, moist, dense, fine to medium grained, weak to medium cementation, 30% fines, no plasticity

... 45% Fines

POORLY GRADED SAND (SM) – Green, moist, medium dense, fine to medium grained, no cementation, 5% fines

FAT CLAY (CH) – Green, moist, hard, high plasticity, medium toughness, no dilatancy

PROFILE
Vertical 1" = 10'
Horizontal 1" = 50'

RETAINING WALLS (MSE) 1,2 & 3

CENTER LINE OF RT 4

881+00 882+00 883+00 884+00 885+00 886+00 887+00 888+00 889+00 890+00 891+00 892+00 893+00 894+00 895+00 896+00

X DESIGN OVERSIGHT	DRAWN BY	G. GOMEZ	M. Shubert FIELD INVESTIGATION BY:		PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	H. PARSA PROJECT ENGINEER
X SIGN OFF DATE	CHECKED BY	C. HENDERSON	DATE: February, 2011			

BRIDGE NO. | POST MILES

CROSSTOWN FREEWAY SR4 IMPROVEMENT

LOG OF TEST BORINGS 4 of 10

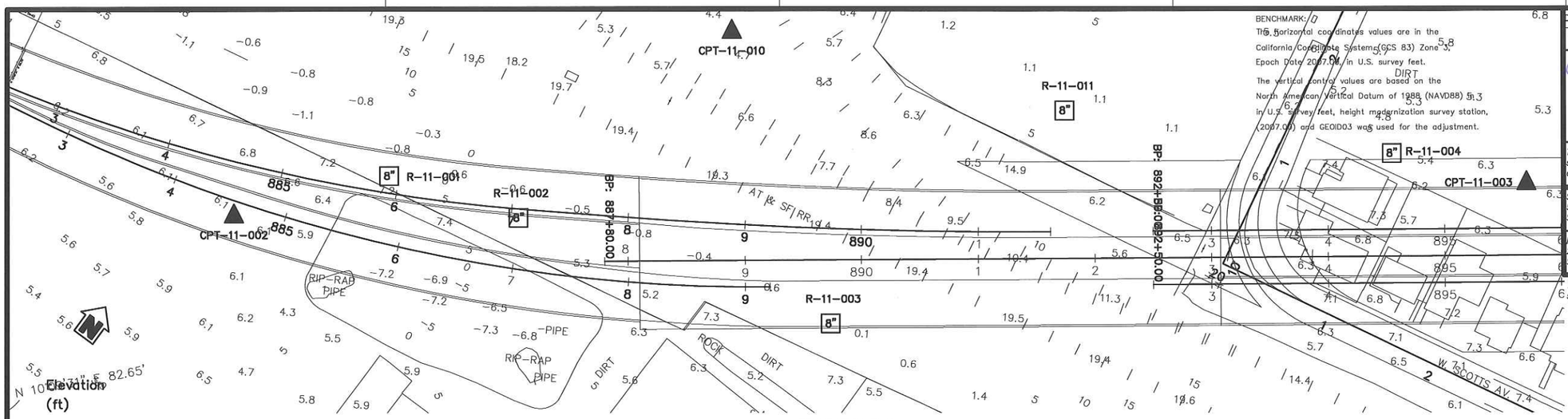
GS LOTB SOIL LEGEND SHEET 1 (ENGLISH) (REV. 07/16/10)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: 1454
PROJECT NUMBER & PHASE: 1000020414 CONTRACT NO.: X

DISREGARD PRINTS BEARING EARLIER REVISION DATES →

REVISION DATES			SHEET	OF
3/2/11	5/4/12	5/23/12		



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10	SJ	5	14.4/14.8 T14.6/R15.7		

GEOTECHNICAL PROFESSIONAL — 5/23/12 DATE

REGISTERED PROFESSIONAL ENGINEER — Carl Henderson No. 2886 Exp. 6/30/2013 GEOTECHNICAL — STATE OF CALIFORNIA

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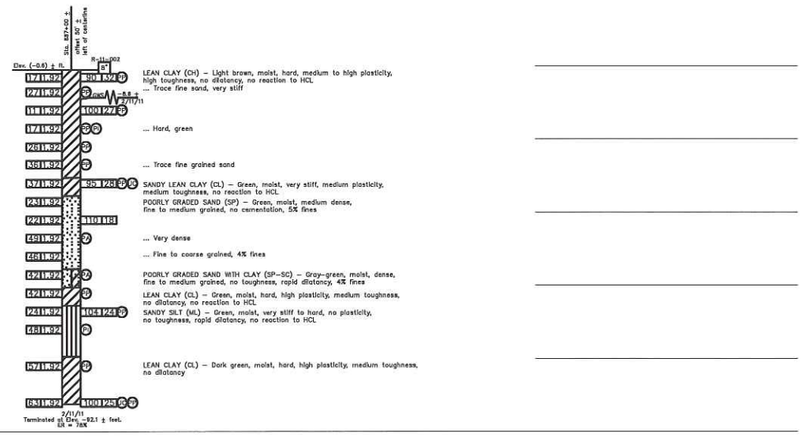
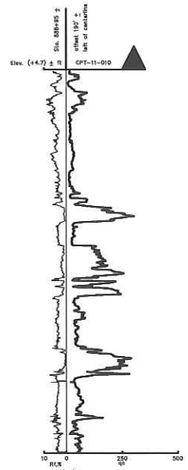
FOUNDATION PLAN
1" = 50'

PROFILE
Vertical 1" = 10'
Horizontal 1" = 20'

Elevation (ft): 20, 0, −20, −40, −60, −80, −100, −120, −140, −160, −180

CENTER LINE OF RT 4
887+00 888+00 889+00 890+00 891+00





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X SIGN OFF DATE	CHECKED BY	C. HENDERSON	DATE: February 2011				POST MILES

CROSSTOWN FREEWAY SR4 IMPROVEMENT

LOG OF TEST BORINGS 5 of 10

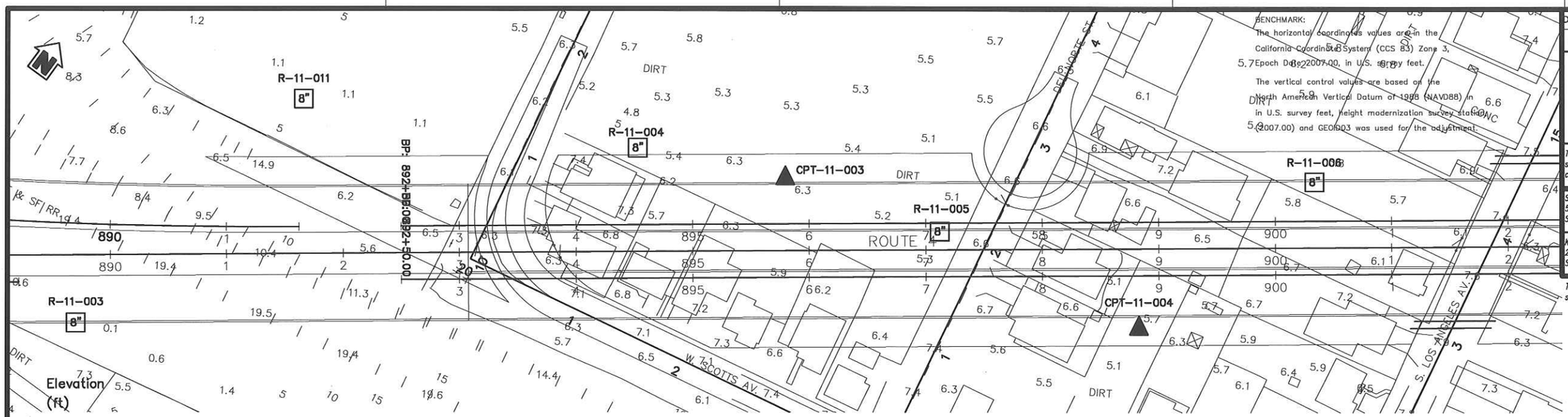
GS LOTB SOIL LEGEND SHEET 1 (ENGLISH) (REV. 07/18/10)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: 1454
PROJECT NUMBER & PHASE: 1000020414 CONTRACT NO.: X

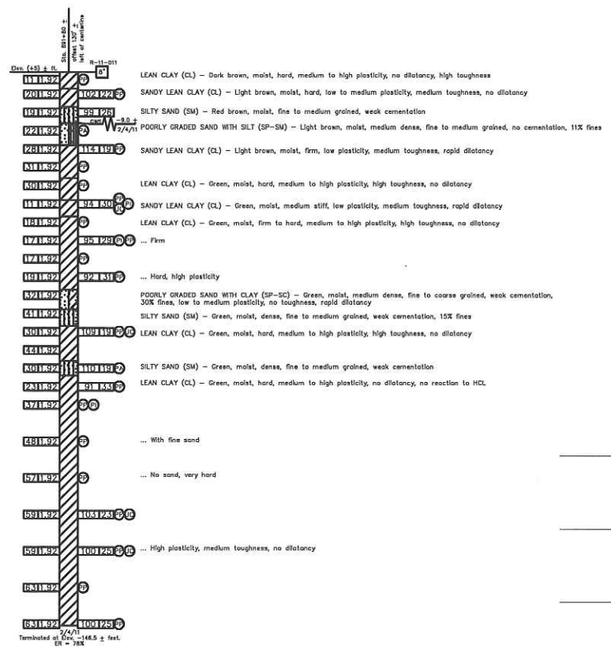
DISREGARD PRINTS BEARING EARLIER REVISION DATES

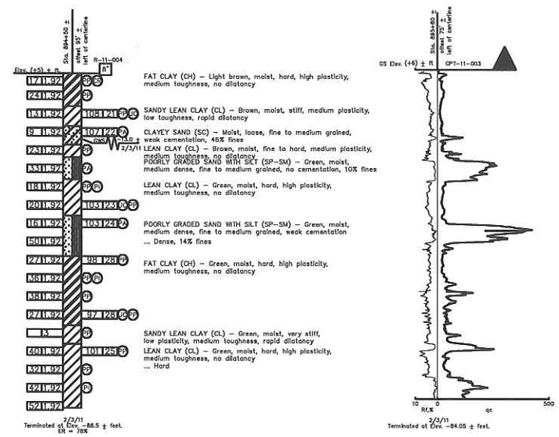
REVISION DATES				SHEET	OF
3/2/11	5/4/12	5/23/12			



FOUNDATION PLAN

1" = 50'





PROFILE
Vertical 1" = 10'
Horizontal 1" = 25'

CENTER LINE OF RT 4

891+00 892+00 893+00 894+00 895+00 896+00 897+00 898+00

Elevation (ft): 20, 0, −20, −40, −60, −80, −100, −120, −140, −160, −180

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GEOTECHNICAL PROFESSIONAL — DATE 5/23/12

PLANS APPROVAL DATE

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X DESIGN OVERSIGHT	DRAWN BY	G. GOMEZ	M. Shubert FIELD INVESTIGATION BY:		PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	H. PARSA PROJECT ENGINEER	BRIDGE NO.
X SIGN OFF DATE	CHECKED BY	C. HENDERSON	DATE: February 2011				POST MILES

CROSSTOWN FREEWAY SR4 IMPROVEMENT

LOG OF TEST BORINGS 6 of 10

GS LOTB SOIL LEGEND SHEET 1 (ENGLISH) (REV. 07/16/10)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: 1454
PROJECT NUMBER & PHASE: 1000020414 CONTRACT NO.: X

DISREGARD PRINTS BEARING EARLIER REVISION DATES →

REVISON DATES				SHEET	OF
3/2/11	5/4/12	5/23/12			

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5	14.4/14.8 T14.6/R15.7		

GEOTECHNICAL PROFESSIONAL — 5/23/12 DATE

REGISTERED PROFESSIONAL ENGINEER — Carl Henderson No. 2886 Exp. 6/30/2013 GEOTECHNICAL — STATE OF CALIFORNIA

PLANS APPROVAL DATE

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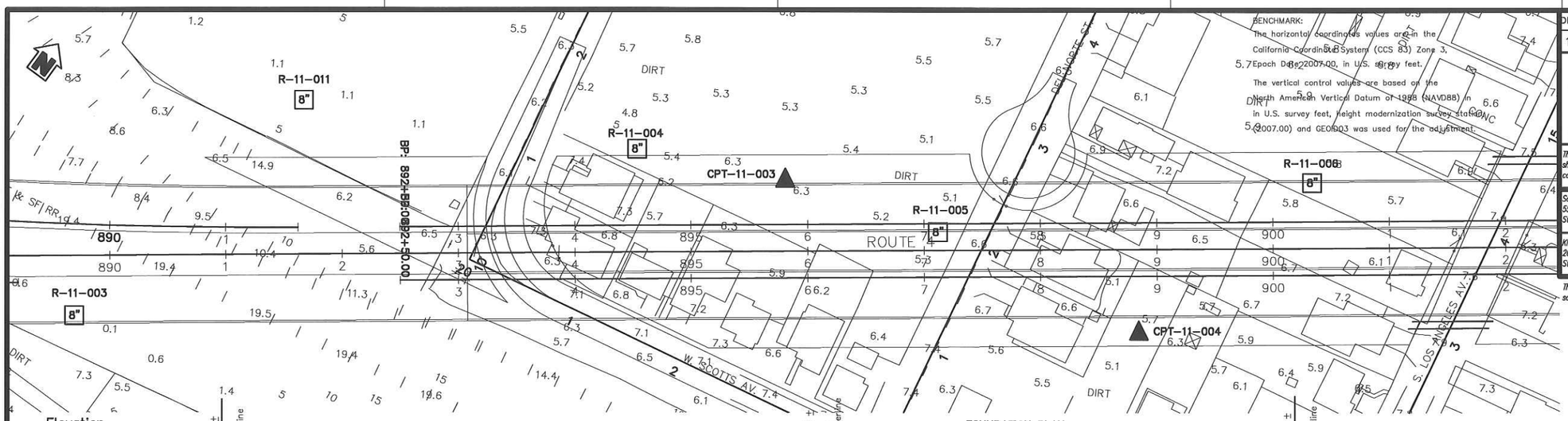
555 E. Weber Avenue
Stockton, CA. 95202

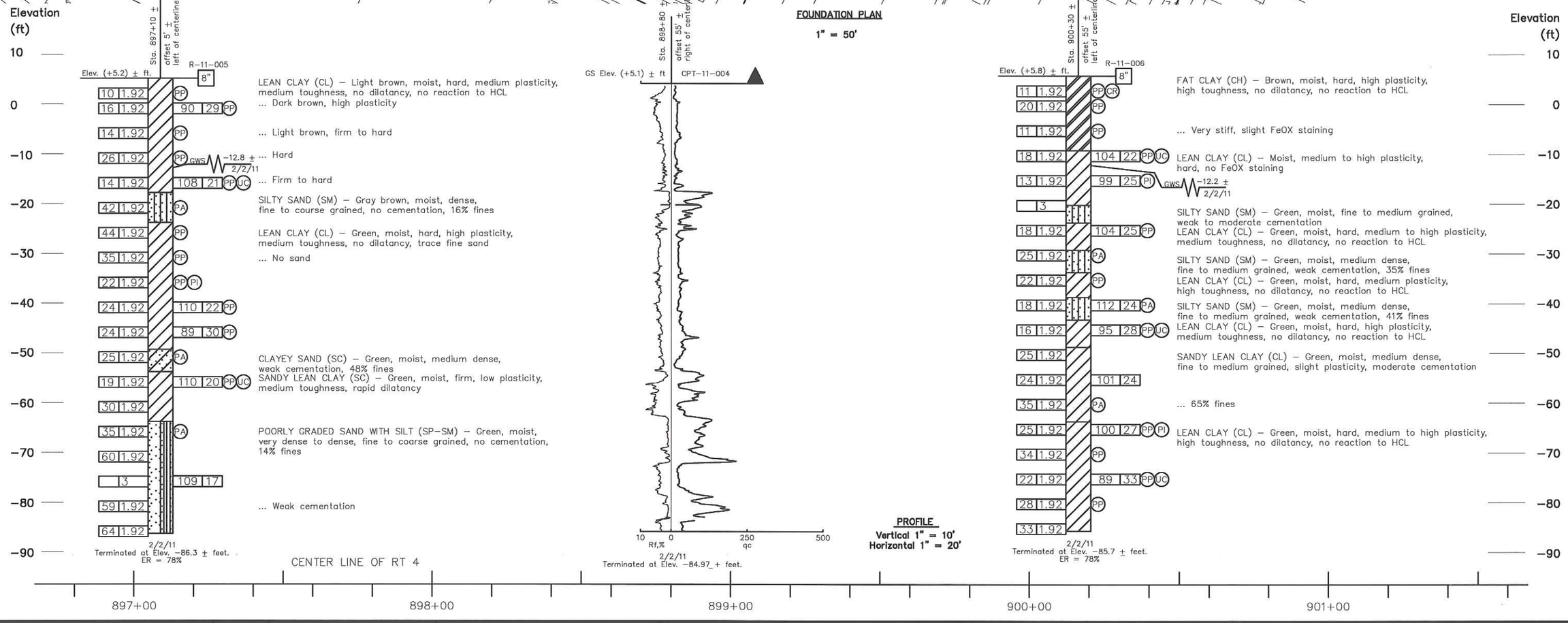
Kleinfelder, Inc.
2001 Arch-Airport Rd., Suite 100
Stockton, CA. 95206

This LOTB sheet was prepared in accordance with the Caltrans soil and rock logging, classification, and manual (2010) presentation

BENCHMARK:
The horizontal coordinates values are in the California Coordinate System (CCS 83) Zone 3, Epoch Date 2007.00, in U.S. survey feet.
The vertical control values are based on the North American Vertical Datum of 1988 (NAVD88) in in U.S. survey feet, Height modernization survey station (2007.00) and GEOID03 was used for the adjustment.



FOUNDATION PLAN
1" = 50'



PROFILE
Vertical 1" = 10'
Horizontal 1" = 20'

X DESIGN OVERSIGHT	DRAWN BY	G. GOMEZ	M. Shubert FIELD INVESTIGATION BY:		PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	H. PARSA PROJECT ENGINEER	BRIDGE NO.
X SIGN OFF DATE	CHECKED BY	C. HENDERSON	DATE: February 2011				POST MILES

CROSSTOWN FREEWAY SR4 IMPROVEMENT

LOG OF TEST BORINGS 7 of 10

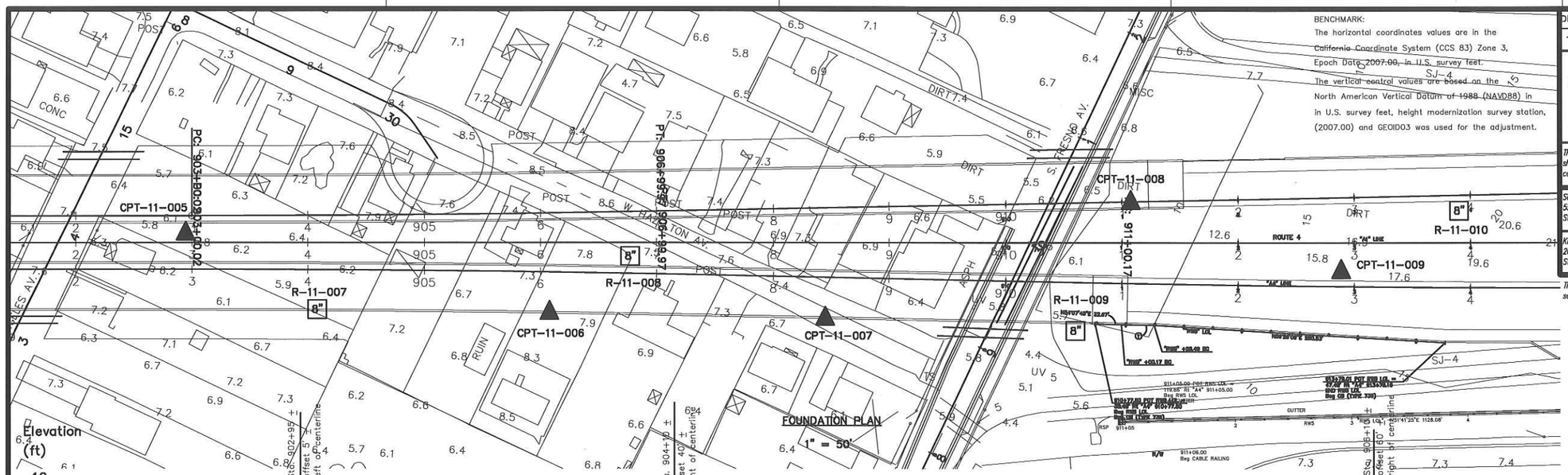
GS LOTB SOIL LEGEND SHEET 1 (ENGLISH) (REV. 07/16/10)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: 1454
PROJECT NUMBER & PHASE: 1000020414 CONTRACT NO.: X

DISREGARD PRINTS BEARING EARLIER REVISION DATES →

REVISION DATES			SHEET	OF
3/2/11	5/4/12	5/23/12		



BENCHMARK:
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DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5	14.4/14.8 T14.6/R15.7		

GEOTECHNICAL PROFESSIONAL — 5/23/12 DATE

REGISTERED PROFESSIONAL ENGINEER — Cari Henderson No. 2886 Exp. 6/30/2013 GEOTECHNICAL — STATE OF CALIFORNIA

PLANS APPROVAL DATE

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Stockton, CA. 95206

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Elevation (ft)

Sta. 902+95 offset 5' left of centerline

Elev. (+6.1) ± ft — CPT–11–005

Rf,% 10 0 250 500 qc

1/31/11
Terminated at Elev. –83.91 ± feet.

Sta. 904+10 ± offset 40' ± right of centerline

Elev. (+6.0) ± ft — R–11–007 8"

11	1.92			PP
36	1.92			PP
35	1.92	GWS –5.0 ± 1/3/11	101	18
3			94	29 PP UC
51	1.92			PP
19	1.92		109	21 PP PI
87	1.92			PA
38	1.92			PP
34	1.92			PA
37	1.92		106	22 UC
29	1.92		108	22 PP
34	1.92			PP PI
30	1.92		98	28 PP
35	1.92			PP
54	1.92		102	25 PP UC
36	1.92		91	32 UC
53	1.92			PP
44	1.92			PP
55	1.92			PP

1/31/11
Terminated at Elev. –85.5 ± feet.
ER = 78%

LEAN CLAY (CL) – Brown, moist, hard, medium plasticity, medium toughness, No dilatancy, no reaction to HCL

SANDY SILT (ML) – Light brown, moist, hard, low plasticity

POORLY GRADED SAND (SP) – Light brown, moist, mediun dense, fine to medium grained, no cementation

LEAN CLAY (CL) – Brown, moist, hard, medium to high plasticity, high toughness, no dilatancy

... Slight increase in fine sand

SILTY SAND (SM) – Green, moist, very dense, fine to medium grained, strong cementation, 30% fines

SANDY LEAN CLAY (CL) – Green, moist, hard, medium to high plasticity, medium toughness, no to slow dilatancy, no reaction to HCL

POORLY GRADED SAND WITH SILT (SP–SM) – Green, moist, medium dense, fine to medium grained, no reaction to HCL, 14% fines

LEAN CLAY (CL) – Green, moist, hard, high plasticity and toughness, no dilatancy, no reaction to HCL

FAT CLAY (CH) – Green, moist, hard

PROFILE
Vertical 1" = 10'
Horizontal 1" = 20'

CENTER LINE OF RT 4

Sta. 906+10 ± offset 80' ± right of centerline

Elev. (+7.0) ± ft — CPT–11–006

Rf,% 10 0 250 500 qc

1/31/11
Terminated at Elev. –83.03 ± feet.

Elevation (ft)

10 0 –10 –20 –30 –40 –50 –60 –70 –80 –90

903+00 904+00 905+00 906+00 907+00

X DESIGN OVERSIGHT	DRAWN BY	G. GOMEZ	M. Shubert FIELD INVESTIGATION BY:		PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	H. PARSA PROJECT ENGINEER
X SIGN OFF DATE	CHECKED BY	C. HENDERSON	DATE: January 2011			

BRIDGE NO.
POST MILES

CROSSTOWN FREEWAY SR4 IMPROVEMENT

LOG OF TEST BORINGS 8 of 10

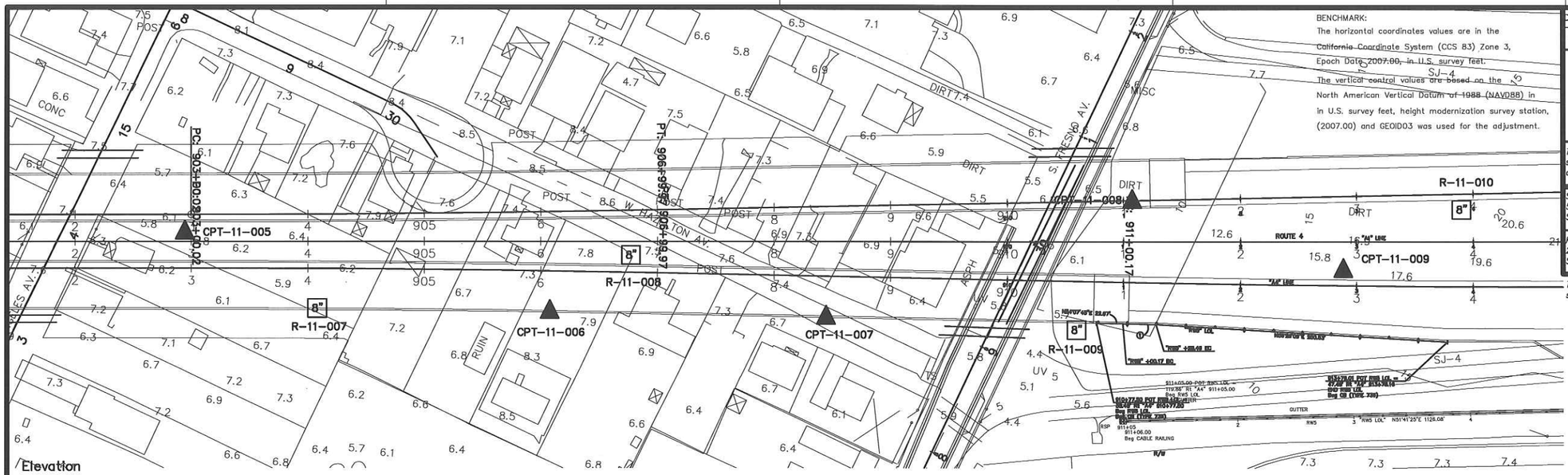
GS LOTB SOIL LEGEND SHEET 1 (ENGLISH) (REV. 07/16/10)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: 1454
PROJECT NUMBER & PHASE: 1000020414 CONTRACT NO.: X

DISREGARD PRINTS BEARING EARLIER REVISION DATES →

REVISION DATES			SHEET	OF
3/2/11	5/4/12	5/23/12		



BENCHMARK:
The horizontal coordinates values are in the California Coordinate System (CCS 83) Zone 3, Epoch Date: 2007.00, in U.S. survey feet. The vertical control values are based on the North American Vertical Datum of 1988 (NAVD88) in in U.S. survey feet, height modernization survey station, (2007.00) and GEOID03 was used for the adjustment.

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5	14.4/14.8 T14.6/R15.7		

GEOTECHNICAL PROFESSIONAL — 5/23/12 DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER — Carl Henderson No. 2886 Exp. 6/30/2013 GEOTECHNICAL — STATE OF CALIFORNIA

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Stockton, CA. 95206

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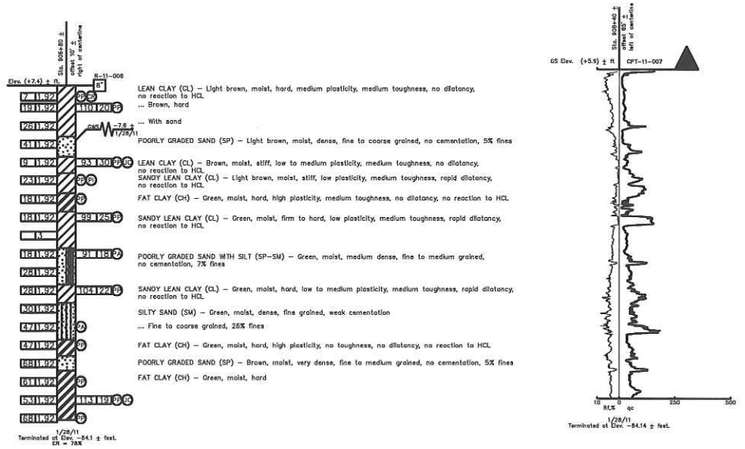
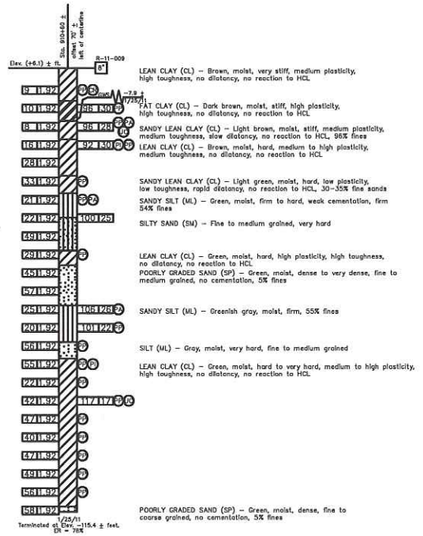
FOUNDATION PLAN
1" = 50'

PROFILE
Vertical 1" = 10'
Horizontal 1" = 25'

Elevation (ft): 20, 0, −20, −40, −60, −80, −100, −120, −140, −160





RETAINING WALL 5

CENTER LINE OF RT 4

906+00 907+00 908+00 909+00 910+00 911+00 912+00 913+00

X DESIGN OVERSIGHT	DRAWN BY	G. GOMEZ	M. Shubert FIELD INVESTIGATION BY:		PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	H. PARSA PROJECT ENGINEER	BRIDGE NO.
X SIGN OFF DATE	CHECKED BY	C. HENDERSON	DATE: January 2011				POST MILES

CROSSTOWN FREEWAY SR4 IMPROVEMENT

LOG OF TEST BORINGS 9 of 10

GS LOTB SOIL LEGEND SHEET 1 (ENGLISH) (REV. 07/16/10)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: 1454
PROJECT NUMBER & PHASE: 1000020414 CONTRACT NO.: X

DISREGARD PRINTS BEARING EARLIER REVISION DATES →

REVISION DATES			SHEET	OF
3/2/11	5/4/12	5/23/12		

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5	14.4/14.8 T14.6/R15.7		

GEOTECHNICAL PROFESSIONAL — 5/23/12 DATE

PLANS APPROVAL DATE

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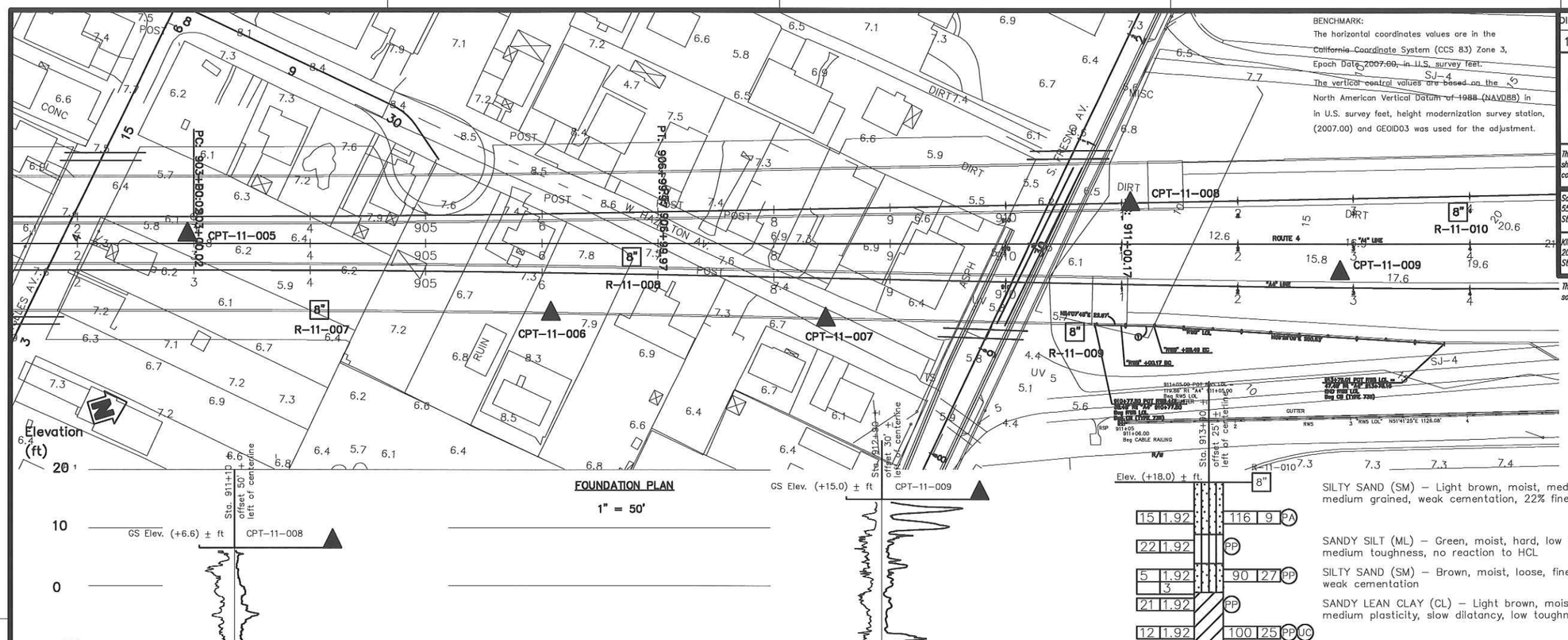
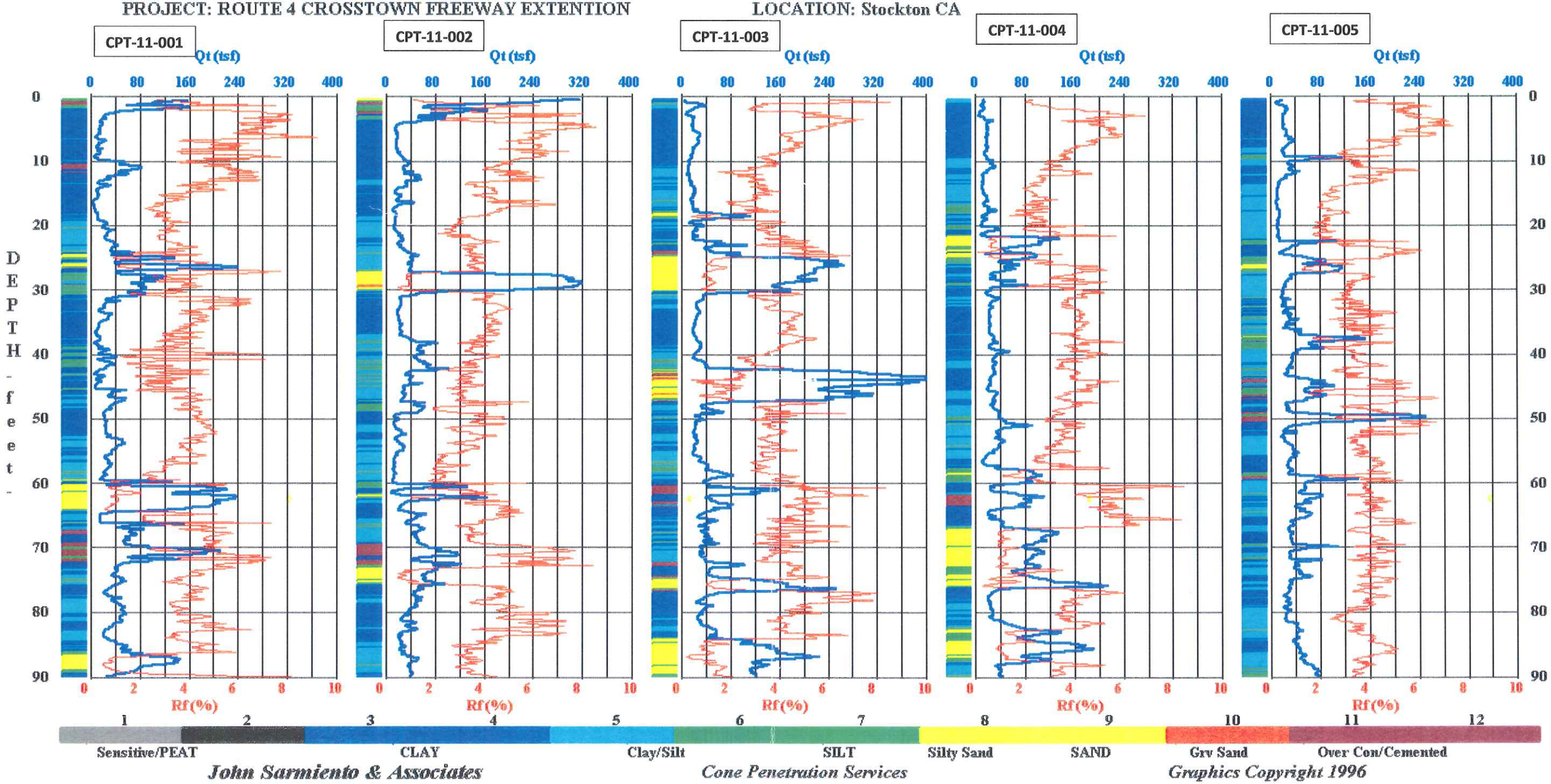

CPT–11–005
CPT–11–006
CPT–11–007
CPT–11–008
CPT–11–009
R–11–007
R–11–008
R–11–009
R–11–010
PC 903+80.00
PT 906+82.97
W. HAZELTON AV.
S. PRESNO AV.
DIRT
ASPH
RUIN
CONC
MISC
GUTTER
ROUTE 4
SJ–4
911+00.17 BK
911+05.00 POT
911+06.00 Beg CABLE RAILING
911+05.00 POT
911+27.61 POT
911+27.61 POT
911+27.61

Cone Penetrometer Test Profiles

KLEINFELDER
Bright People. Right Solutions.

PROJECT: ROUTE 4 CROSSTOWN FREEWAY EXTENTION

LOCATION: Stockton CA

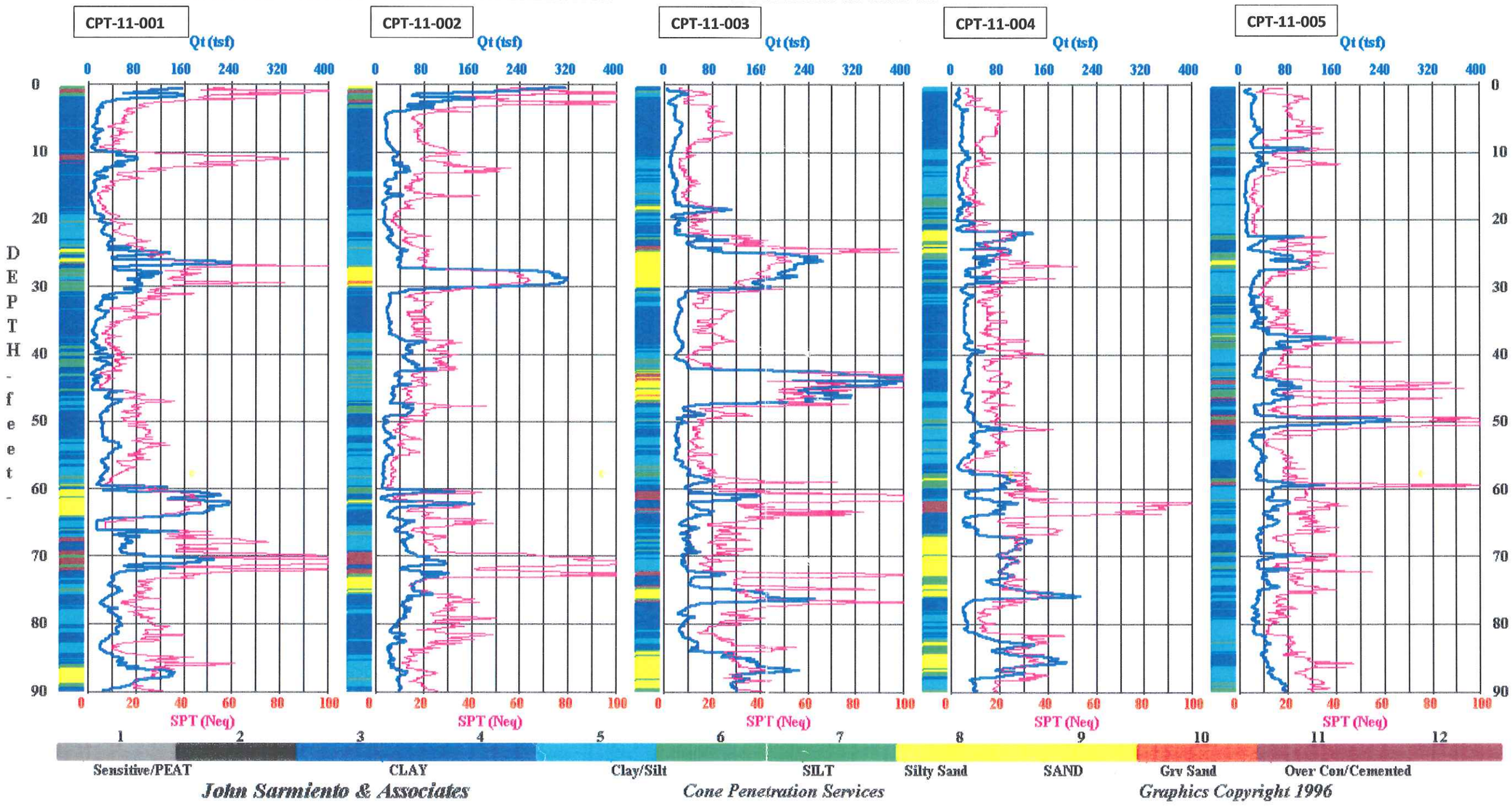


John Sarmiento & Associates *Cone Penetration Services*

Graphics Copyright 1996

PROJECT: ROUTE 4 CROSSTOWN FREEWAY EXTENTION

LOCATION: Stockton CA



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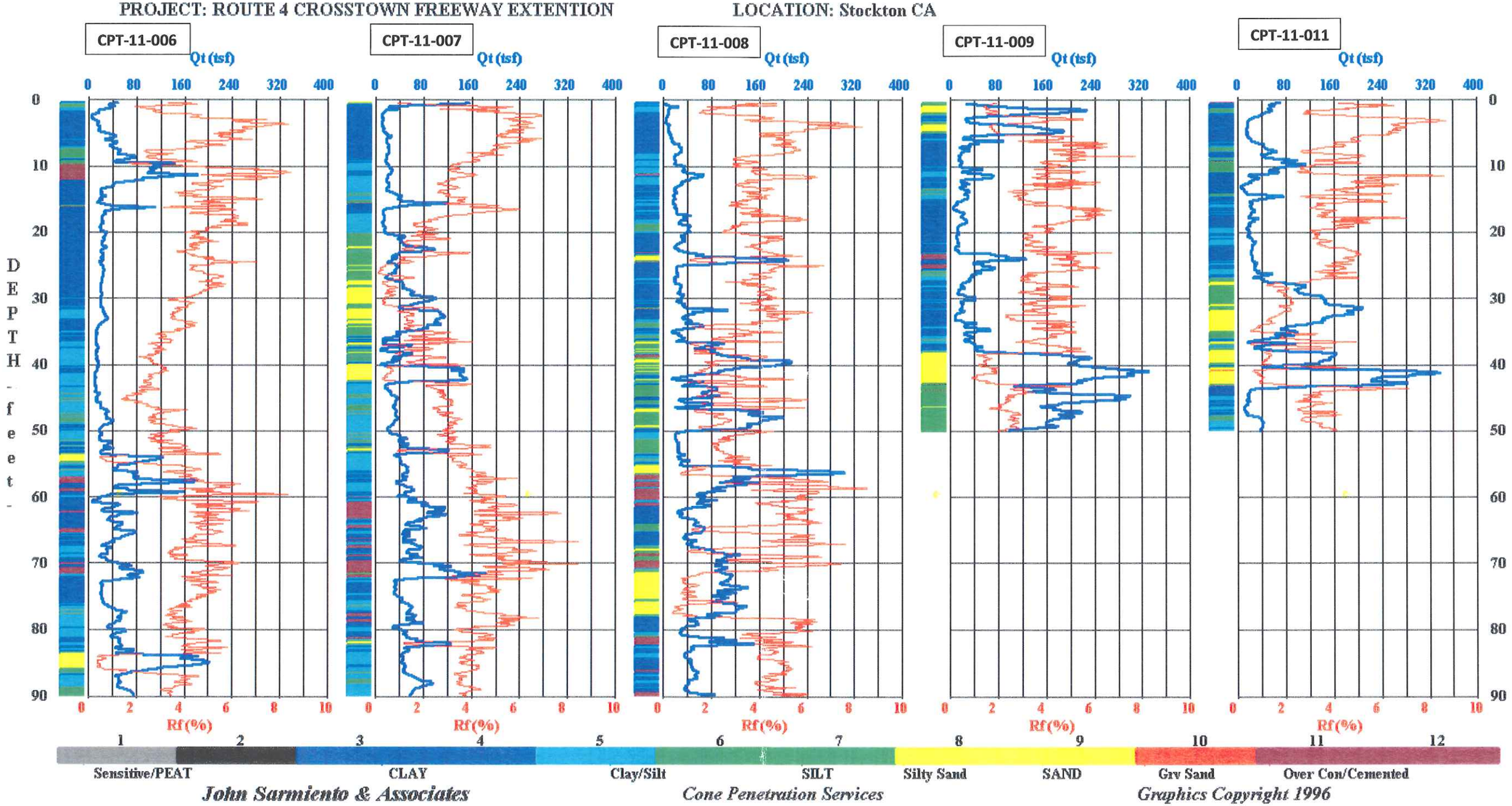
KLEINFELDER
Bright People. Right Solutions.

PROJECT: ROUTE 4 CROSSTOWN FREEWAY EXTENTION

LOCATION: Stockton CA



John Sarmiento & Associates *Cone Penetration Services*

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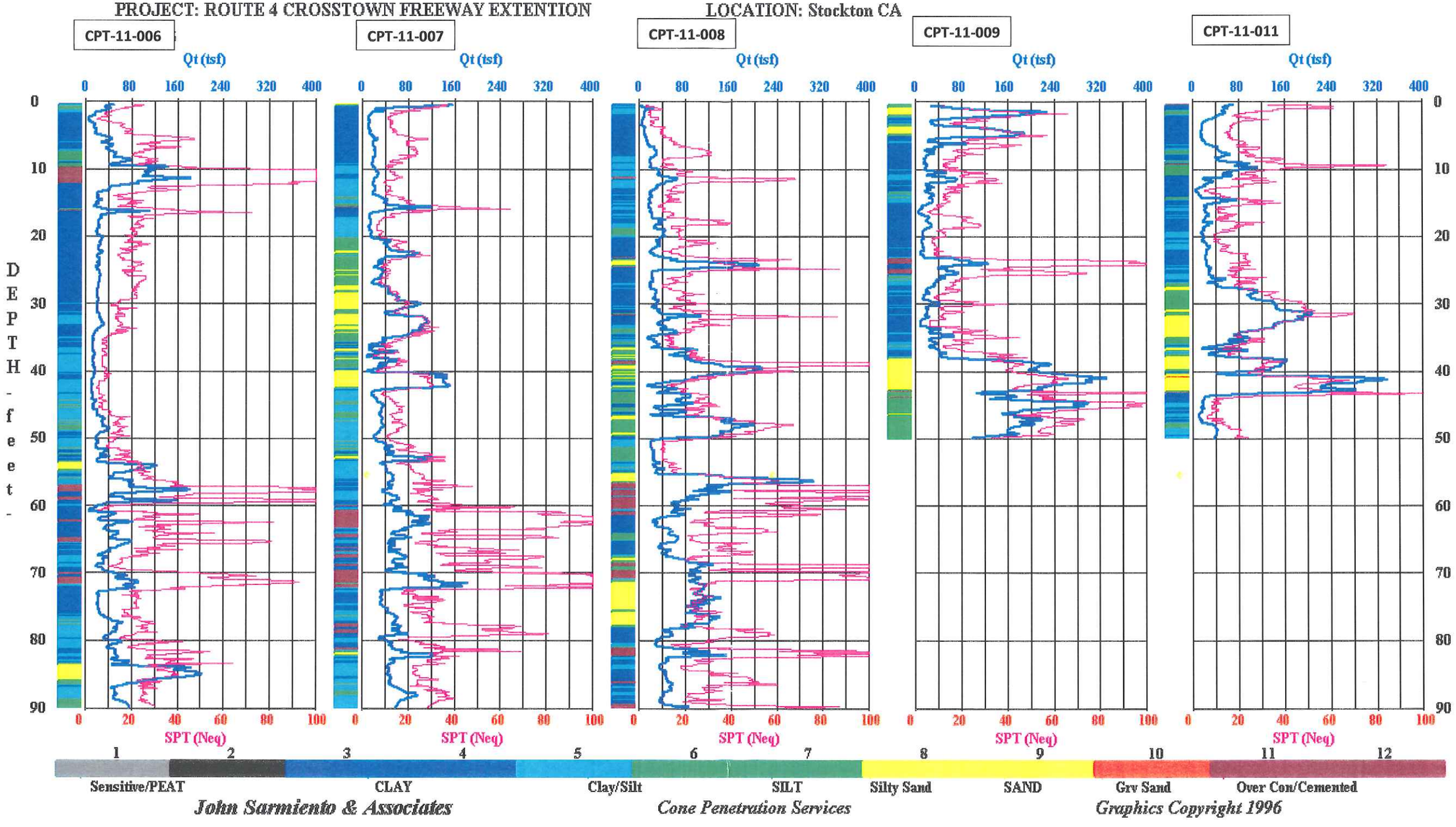
PROJECT: ROUTE 4 CROSSTOWN FREEWAY EXTENTION

LOCATION: Stockton CA

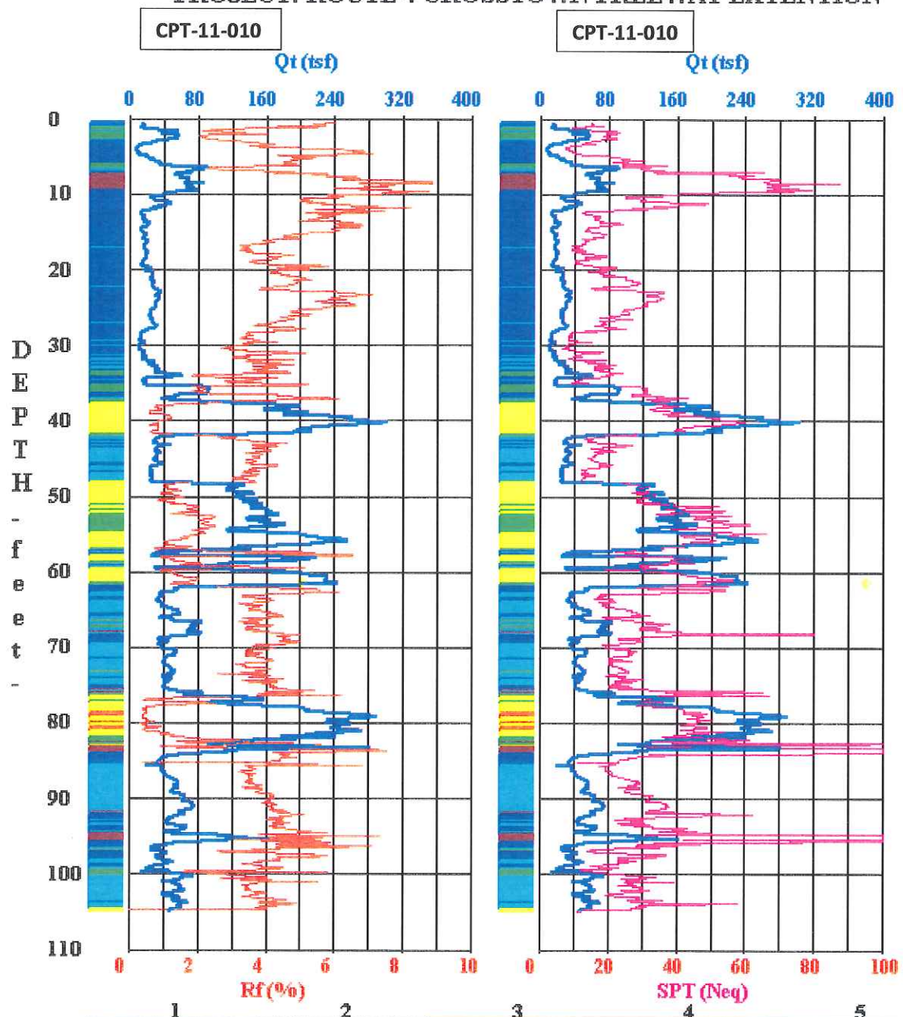


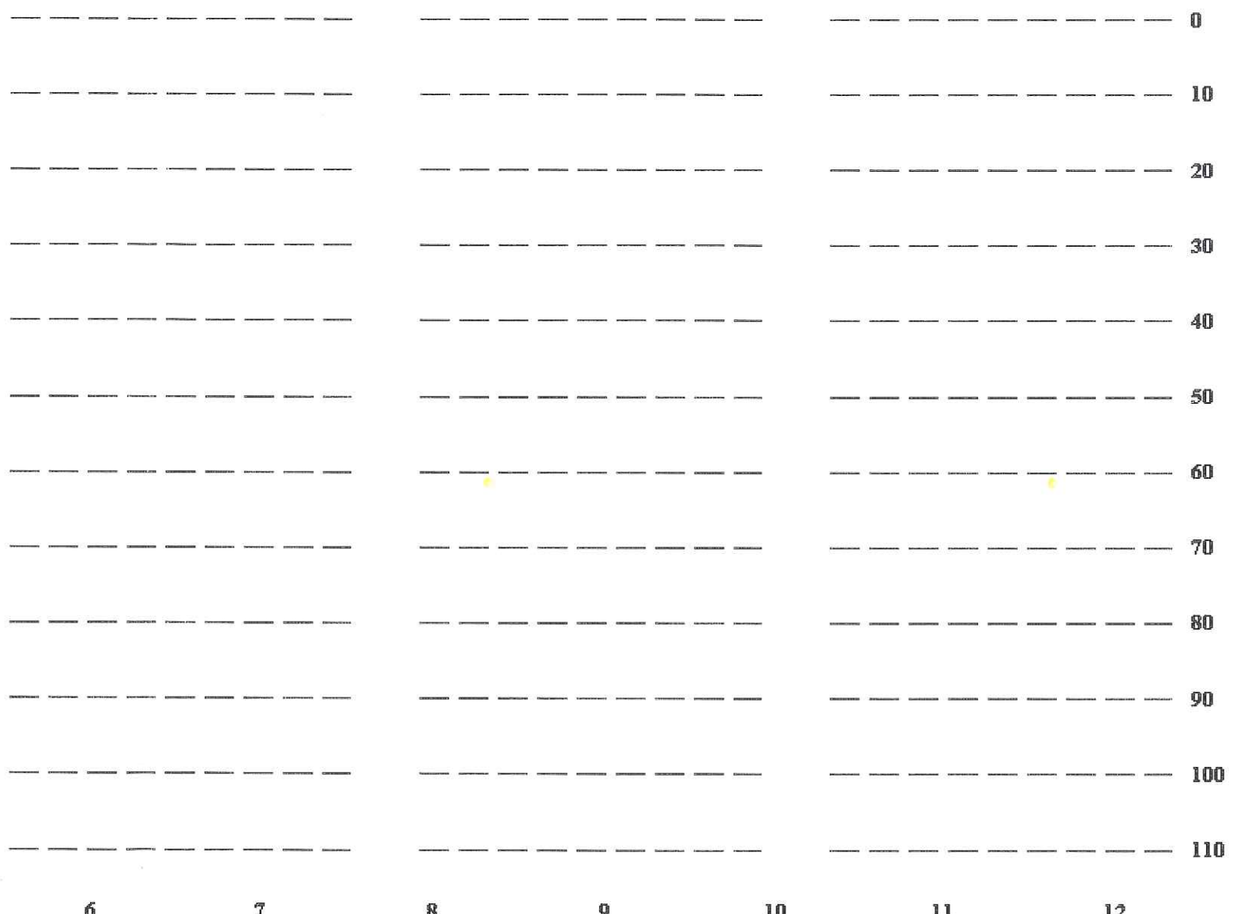
John Sarmiento & Associates *Cone Penetration Services*

Graphics Copyright 1996

PROJECT: ROUTE 4 CROSSTOWN FREEWAY EXTENTION

LOCATION: Stockton CA





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APPENDIX C
LOGS OF BORINGS AND CONE PENETRATION TESTS PROFILES
(PARIKH CONSULTANT, INC. INVESTIGATION, MARCH 2010)

LIST OF ATTACHMENTS

The following are attached and complete this appendix.

Log of Test Borings Sheets
Cone Penetrometer Test Profiles

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Log of Test Borings Sheets

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (JUNE 2007)

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	4	R12.6/R15.7		

GEOTECHNICAL PROFESSIONAL DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER — GARY PARIKH — No. G.E. 666 — Exp. 12/31/11 — GEOTECHNICAL — STATE OF CALIFORNIA

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131

SAN JOAQUIN COUNCIL OF GOVERNMENTS
555 EAST WEBER AVENUE
STOCKTON, CALIFORNIA 95202

BKF ENGINEERS
4670 WILLOW ROAD, SUITE 250
PLEASANTON CALIFORNIA 94588

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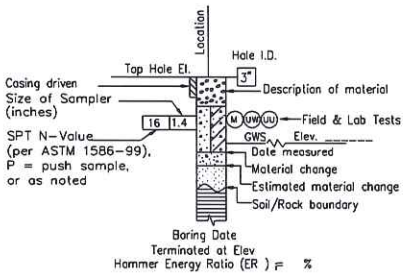
CEMENTATION	
Description	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

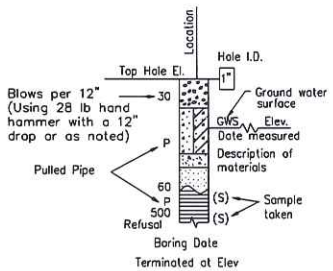
CONSISTENCY OF COHESIVE SOILS				
Description	Unconfined Compressive Strength (tsf)	Pocket Penetrometer Measurement (tsf)	Torvane Measurement (tsf)	Field Approximation
Very Soft	< 0.25	< 0.25	< 0.12	Easily penetrated several inches by fist
Soft	0.25 to 0.50	0.25 to 0.50	0.12 to 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 to 1.0	0.50 to 1.0	0.25 to 0.50	Penetrated several inches by thumb with moderate effort
Stiff	1 to 2	1 to 2	0.50 to 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2 to 4	2 to 4	1.0 to 2.0	Readily indented by thumbnail
Hard	> 4.0	> 4.0	> 2.0	Indented by thumbnail with difficulty

BOREHOLE IDENTIFICATION		
Symbol	Hole Type	Description
Size	A	Auger Boring
Size	R P	Rotary drilled boring Rotary percussion boring (air)
Size	R	Rotary drilled diamond core
Size	HD HA	Hand driven (1-inch soil tube) Hand Auger
●	D	Dynamic Cone Penetration Boring
▲	CPT	Cone Penetration Test (ASTM D 5778-95)
□	O	Other
Note: Size in inches.		

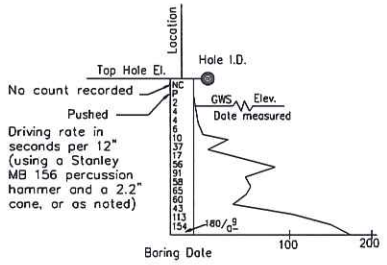
PLASTICITY OF FINE-GRAINED SOILS	
Description	Criteria
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.



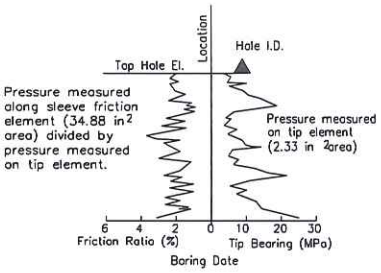
ROTARY BORING



HAND BORING



DYNAMIC CONE PENETRATION BORING



CONE PENETRATION TEST (CPT) SOUNDING

DESIGN OVERSIGHT	DRAWN BY	Amit Bharadwaj	L. S. Bhangoo	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	Gary Parikh	BRIDGE NO. –
SIGN OFF DATE	CHECKED BY	David Wang	FIELD INVESTIGATION BY: DATE: November 2009		PROJECT ENGINEER	POST MILES –

ROUTE 4 CROSSTOWN CONNECTOR

LOG OF TEST BORINGS

DOS CIVIL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 08-01-09)	ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3	CU – EA –	DISREGARD PRINTS BEARING EARLIER REVISION DATES → REVISION DATES

FILE => $REQUEST

USERNAME => $USER DATE PLOTTED => $DATE TIME PLOTTED => $TIME

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (JUNE 2007)

GROUP SYMBOLS AND NAMES

Graphic/Symbol	Group Names	Graphic/Symbol	Group Names
GW	Well-graded GRAVEL Well-graded GRAVEL with SAND	CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
GP	Poorly graded GRAVEL Poorly graded GRAVEL with SAND		
GW-GM	Well-graded GRAVEL with SILT Well-graded GRAVEL with SILT and SAND	CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
GW-GC	Well-graded GRAVEL with CLAY (or SILTY CLAY) Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)		
GP-GM	Poorly graded GRAVEL with SILT Poorly graded GRAVEL with SILT and SAND	ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
GP-GC	Poorly graded GRAVEL with CLAY (or SILTY CLAY) Poorly graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)		
GM	SILTY GRAVEL SILTY GRAVEL with SAND	OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND		
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND	OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
SW	Well-graded SAND Well-graded SAND with GRAVEL		
SP	Poorly graded SAND Poorly graded SAND with GRAVEL	CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
SW-SM	Well-graded SAND with SILT Well-graded SAND with SILT and GRAVEL		
SW-SC	Well-graded SAND with CLAY (or SILTY CLAY) Well-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)	MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
SP-SM	Poorly graded SAND with SILT Poorly graded SAND with SILT and GRAVEL		
SP-SC	Poorly graded SAND with CLAY (or SILTY CLAY) Poorly graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)	OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
SM	SILTY SAND SILTY SAND with GRAVEL		
SC	CLAYEY SAND CLAYEY SAND with GRAVEL	OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY ORGANIC elastic SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL		
PT	PEAT	OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND
	COBBLES COBBLES and BOULDERS BOULDERS		

FIELD AND LABORATORY TESTING

- (C) Consolidation (ASTM D 2435)
- (CL) Collapse Potential (ASTM D 5333)
- (CP) Compaction Curve (CTM 216)
- (CR) Corrosivity Testing (CTM 643, CTM 422, CTM 417)
- (CU) Consolidated Undrained Triaxial (ASTM D 4767)
- (DS) Direct Shear (ASTM D 3080)
- (EI) Expansion Index (ASTM D 4829)
- (M) Moisture Content (ASTM D 2216)
- (OC) Organic Content-% (ASTM D 2974)
- (P) Permeability (CTM 220)
- (PA) Particle Size Analysis (ASTM D 422)
- (PI) Plasticity Index (AASHTO T 90) Liquid Limit (AASHTO T 89)
- (PL) Point Load Index (ASTM D 5731)
- (PM) Pressure Meter
- (PP) Pocket Penetrometer
- (R) R-Value (CTM 301)
- (SE) Sand Equivalent (CTM 217)
- (SG) Specific Gravity (AASHTO T 100)
- (SL) Shrinkage Limit (ASTM D 427)
- (SW) Swell Potential (ASTM D 4546)
- (TV) Pocket Torvane
- (UC) Unconfined Compression-Soil (ASTM D 2166) Unconfined Compression-Rock (ASTM D 2938)
- (UU) Unconsolidated Undrained Triaxial (ASTM D 2850)
- (UW) Unit Weight (ASTM D 4767)
- (VS) Vane Shear (AASHTO T 223)

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	4	R12.6/R15.7		

GEOTECHNICAL PROFESSIONAL DATE

REGISTERED PROFESSIONAL ENGINEER — GARY PARIKH — No. G.E. 666 — Exp. 12/31/11 — GEOTECHNICAL — STATE OF CALIFORNIA

PLANS APPROVAL DATE

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131

SAN JOAQUIN COUNCIL OF GOVERNMENTS
555 EAST WEBER AVENUE
STOCKTON, CALIFORNIA 95202

BKF ENGINEERS
4670 WILLOW ROAD, SUITE 250
PLEASANTON CALIFORNIA 94588

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APPARENT DENSITY OF COHESIONLESS SOILS

Description	SPT N_{60} (Blows / 12 inches)
Very loose	0 – 4
Loose	5 – 10
Medium Dense	11 – 30
Dense	31 – 50
Very Dense	> 50

MOISTURE

Description	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

PERCENT OR PROPORTION OF SOILS

Description	Criteria
Trace	Particles are present but estimated to be less than 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

PARTICLE SIZE

Description		Size
Boulder		> 12"
Cobble		3" to 12"
Gravel	Coarse	3/4" to 3"
	Fine	No. 4 to 3/4"
Sand	Coarse	No. 10 to No. 4
	Medium	No. 40 to No. 10
	Fine	No. 200 to No. 40

DESIGN OVERSIGHT
SIGN OFF DATE
DRAWN BY: Amit Bharadwaj
CHECKED BY: David Wang
L. S. Bhangoo
FIELD INVESTIGATION BY:
DATE: November 2009

PREPARED FOR THE STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

Gary Parikh
PROJECT ENGINEER

BRIDGE NO. –
POST MILES –

ROUTE 4 CROSSTOWN CONNECTOR
LOG OF TEST BORINGS

CU –
EA –

DGS CIVIL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 08-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES | SHEET | OF

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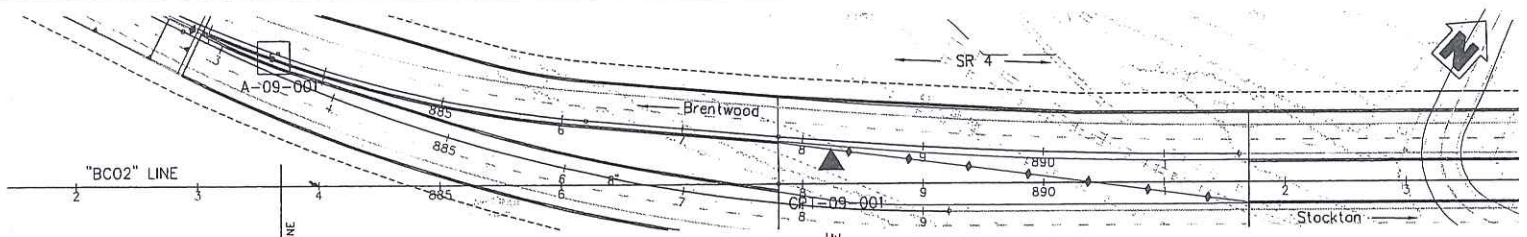
USERNAME => $USER DATE PLOTTED => $DATE TIME PLOTTED => $TIME

PLATE A-3



DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	4	R12.6/R15.7		

GEOTECHNICAL PROFESSIONAL DATE

REGISTERED PROFESSIONAL ENGINEER — GARY PARIKH — No. G.E. 666 — Exp. 12/31/11 — GEOTECHNICAL — STATE OF CALIFORNIA

PLANS APPROVAL DATE

PARIKH CONSULTANTS, INC.
2360 OUNE DRIVE, SUITE A
SAN JOSE, CA 95131

SAN JOAQUIN COUNCIL OF GOVERNMENTS
555 EAST WEBER AVENUE
STOCKTON, CALIFORNIA 95202

BKF ENGINEERS
4670 WILLOW ROAD, SUITE 250
PLEASANTON CALIFORNIA 94588

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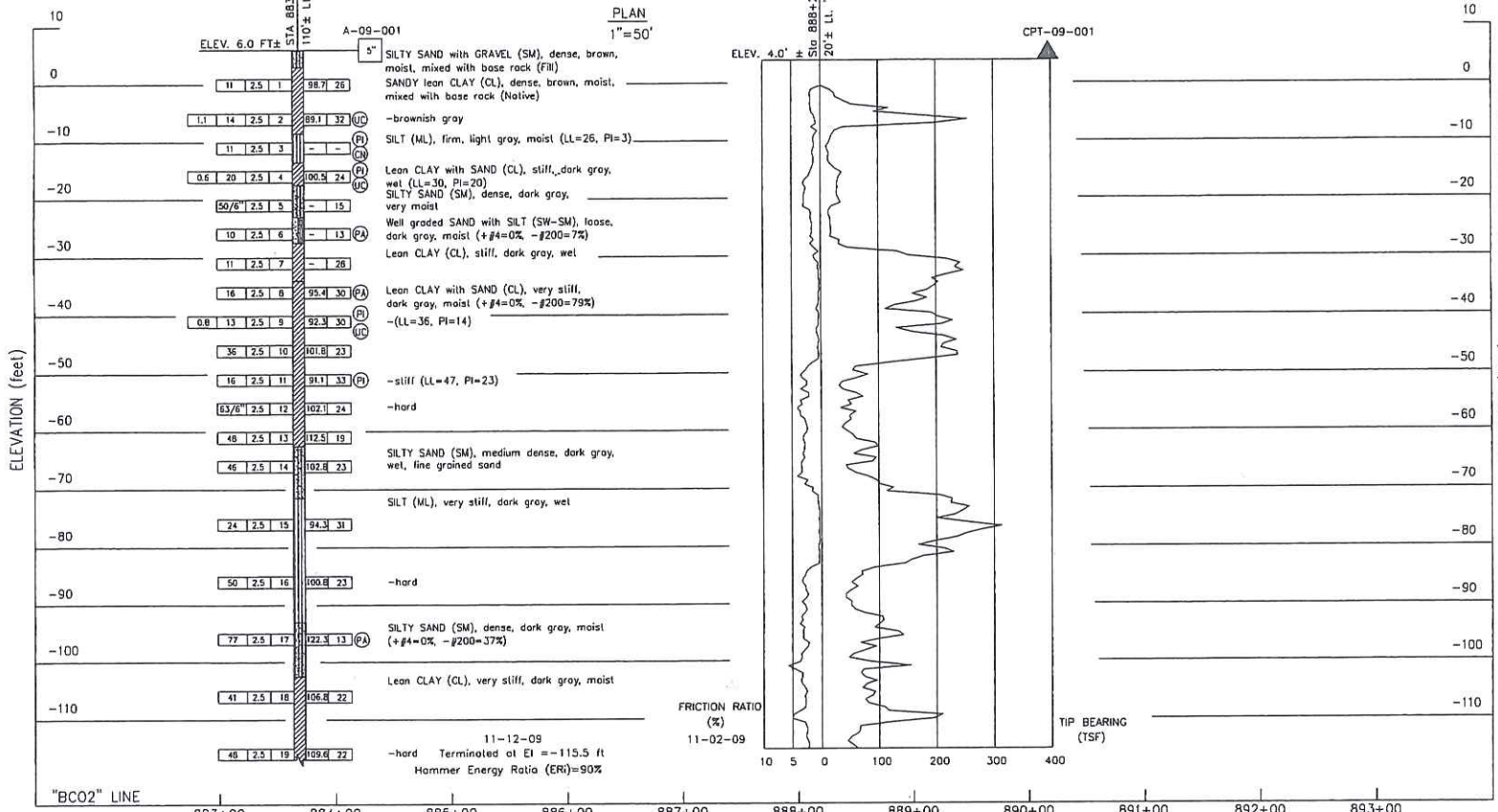
DRAFT



Notes:
Modified California Sampler: I.D. = 2.5"; O.D. = 3", Hammer Assembly: A 140 lb hammer with a 30" drop, (Automatic Hammer)

This LOTB sheet was prepared in accordance with the Caltrans Soil & Rock, Logging, Classification, and Presentation Manual (June 2007)

All dimensions are in feet unless otherwise shown

PROFILE
Vert. : 1" = 10'
Hor. : 1" = 50'

DESIGN OVERSIGHT	DRAWN BY	Amit Bharadwaj	L. S. Bhangoo	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	Gary Parikh	BRIDGE NO. –
SIGN OFF DATE	CHECKED BY	David Wong	FIELD INVESTIGATION BY: DATE: November 2009		PROJECT ENGINEER	POST MILES –

ROUTE 4 CROSSTOWN CONNECTOR
LOG OF TEST BORINGS

CU –
EA –
FILE => $REQUEST

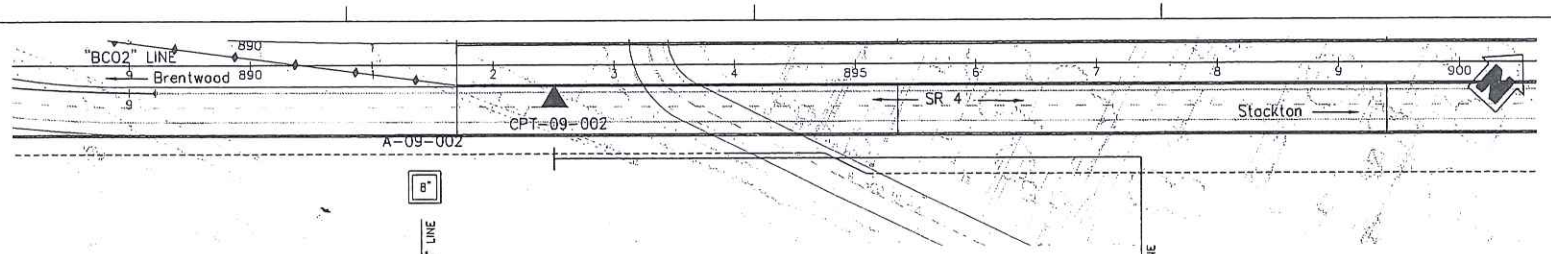
DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES | SHEET | OF

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

005 GEOTECHNICAL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 08-01-09)

PLATE A-4



PLAN
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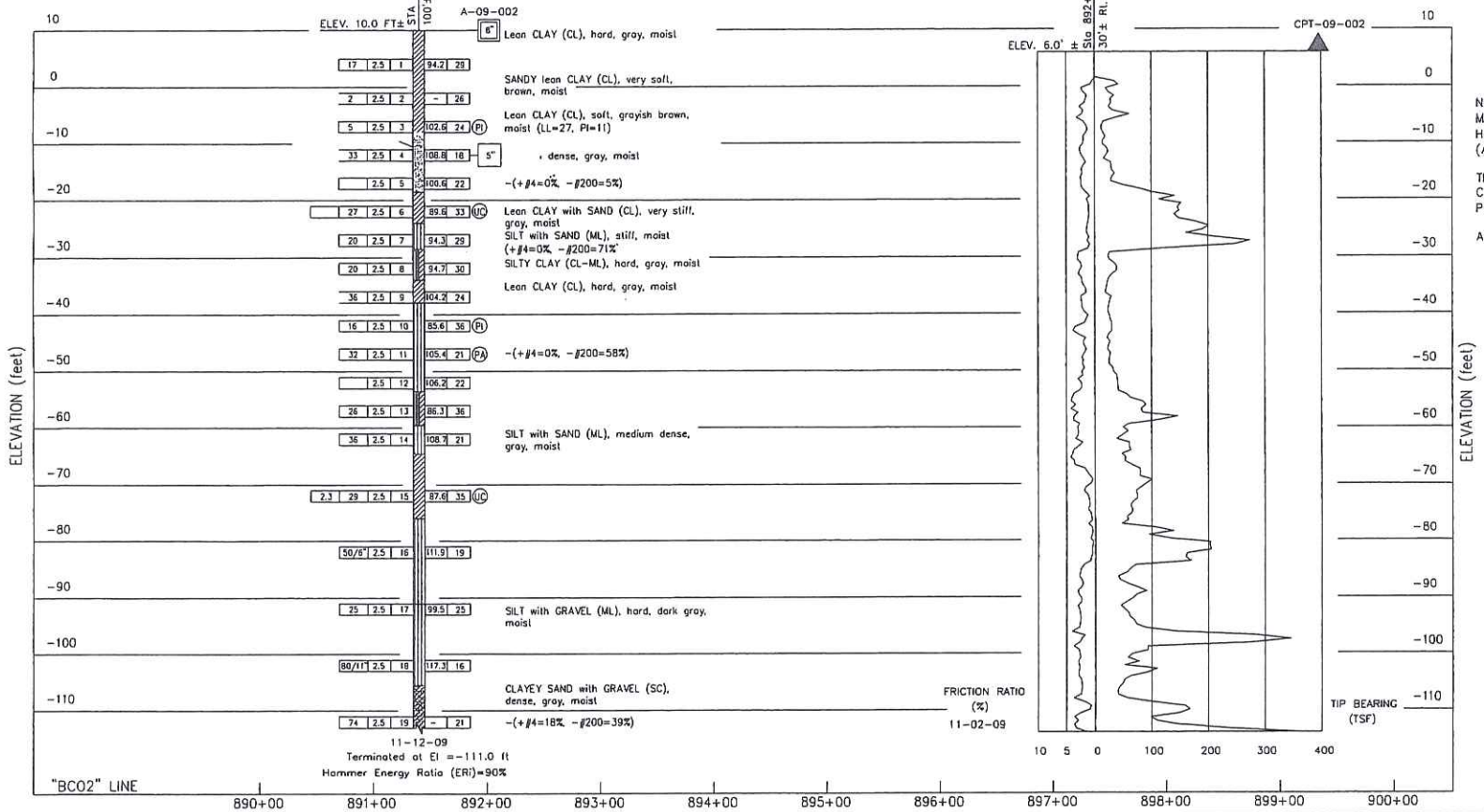

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	4	R12.6/R15.7		

GEOTECHNICAL PROFESSIONAL DATE

REGISTERED PROFESSIONAL ENGINEER
GARY PARIKH
No. G.E. 666
Exp. 12/31/11
GEOTECHNICAL
STATE OF CALIFORNIA

PLANS APPROVAL DATE

PARIKH CONSULTANTS, INC.
2360 OAKE DRIVE, SUITE A
SAN JOSE, CA 95131

SAN JOAQUIN COUNCIL OF GOVERNMENTS
555 EAST WEBER AVENUE
STOCKTON, CALIFORNIA 95202

BKF ENGINEERS
4670 WILLOW ROAD, SUITE 250
PLEASANTON CALIFORNIA 94588

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DRAFT

Notes:
Modified California Sampler: I.D. = 2.5"; O.D. = 3".
Hammer Assembly: A 140 lb hammer with a 30" drop, (Automatic Hammer)

This LOTB sheet was prepared in accordance with the Caltrans Soil & Rock, Logging, Classification, and Presentation Manual (June 2007)

All dimensions are in feet unless otherwise shown

PROFILE
Vert. : 1" = 10'
Hor. : 1" = 50'

DESIGN OVERSIGHT	DRAWN BY	Amit Bharadwaj	L. S. Bhangoo	Gary Parikh	BRIDGE NO. –
SIGN OFF DATE	CHECKED BY	David Wang	FIELD INVESTIGATION BY: DATE: November 2009	PROJECT ENGINEER	POST MILES –

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

CU –
EA –

ROUTE 4 CROSSTOWN CONNECTOR
LOG OF TEST BORINGS

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES | SHEET | OF

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

FILE => $REQUEST

DGS GEOTECHNICAL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 08-01-09)

USERNAME => $USER DATE PLOTTED => $DATE TIME PLOTTED => $TIME

PLATE A-5

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	4	R12.6/R15.7		

GEOTECHNICAL PROFESSIONAL DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER
GARY PARIKH
No. G.E. 666
Exp. 12/31/11
GEOTECHNICAL
STATE OF CALIFORNIA

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131

SAN JOAQUIN COUNCIL OF GOVERNMENTS
555 EAST WEBER AVENUE
STOCKTON, CALIFORNIA 95202

BKF ENGINEERS
4670 WILLOW ROAD, SUITE 250
PLEASANTON CALIFORNIA 94588

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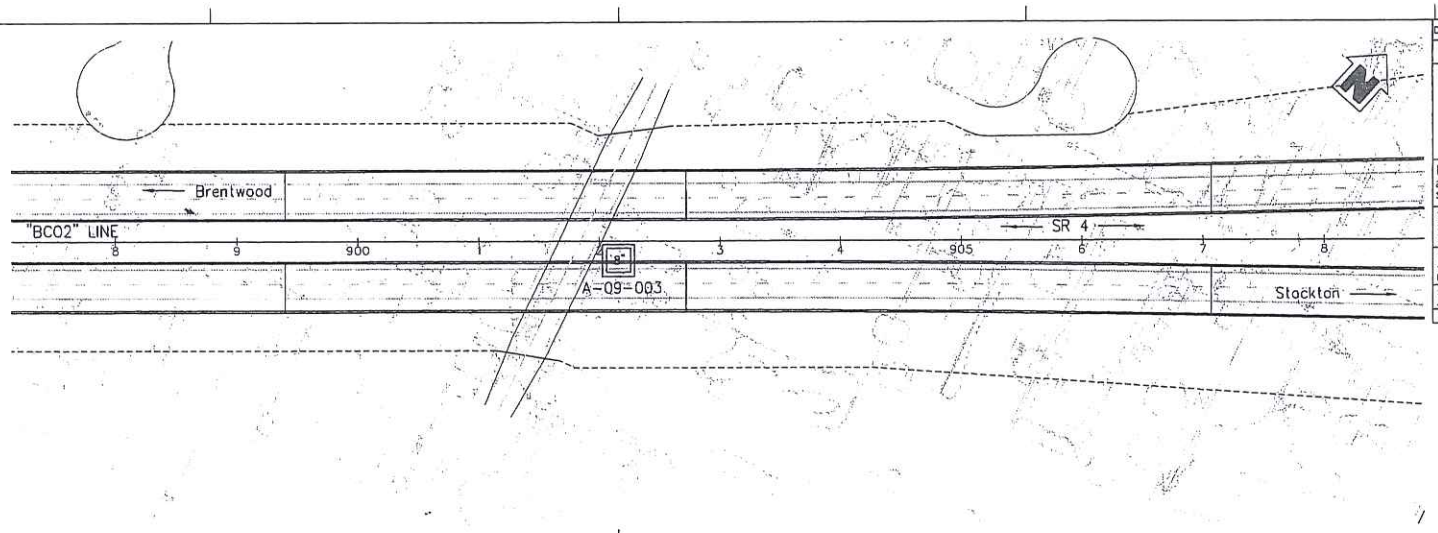
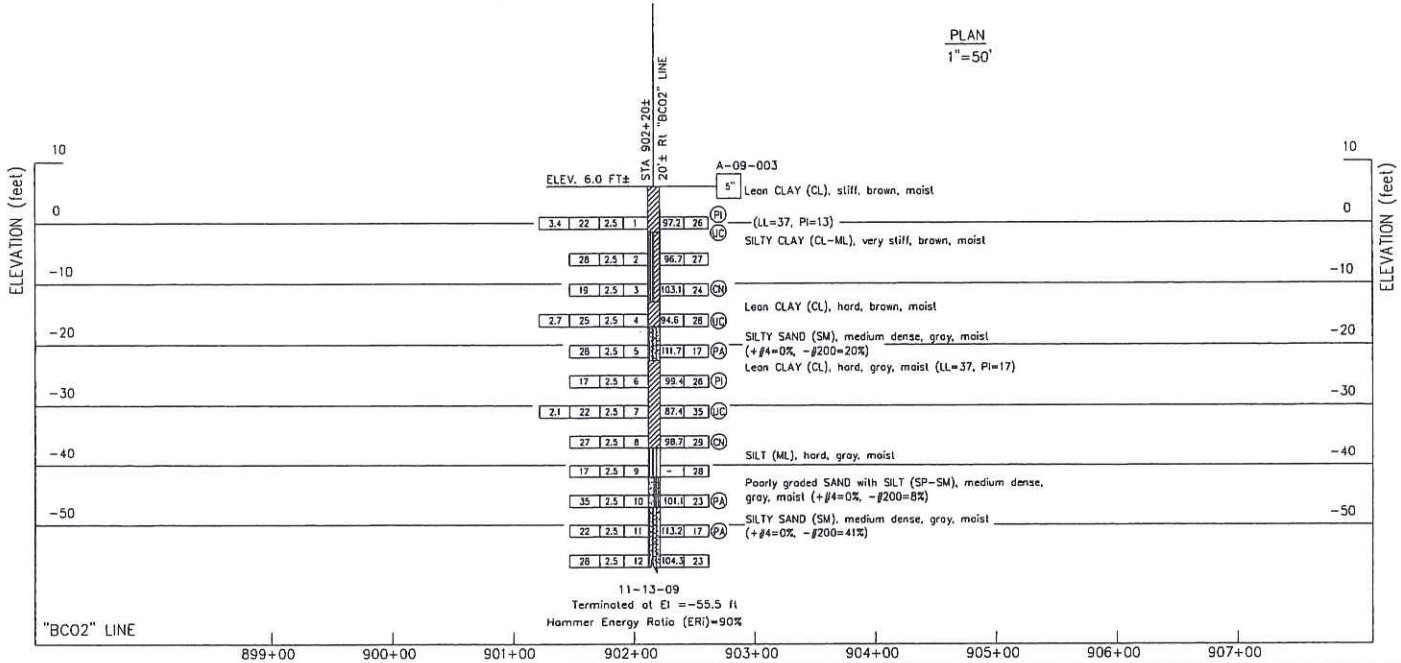
DRAFT



PLAN
1"=50'



Notes:
Modified California Sampler: I.D. = 2.5"; O.D. = 3", Hammer Assembly: A 140 lb hammer with a 30" drop, (Automatic Hammer)

This LOTB sheet was prepared in accordance with the Caltrans Soil & Rock, Logging, Classification, and Presentation Manual (June 2007)

All dimensions are in feet unless otherwise shown

PROFILE
Vert. : 1" = 10'
Hor. : 1" = 50'

DESIGN OVERSIGHT	DRAWN BY	Amit Bharadwaj	L. S. Bhangoo	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	Gary Parikh	BRIDGE NO. -
SIGN OFF DATE	CHECKED BY	David Wang	FIELD INVESTIGATION BY: DATE: November 2009		PROJECT ENGINEER	POST MILES -

ROUTE 4 CROSSTOWN CONNECTOR
LOG OF TEST BORINGS

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES | SHEET | OF

CU -
EA -

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

DGS GEOTECHNICAL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 08-01-09)

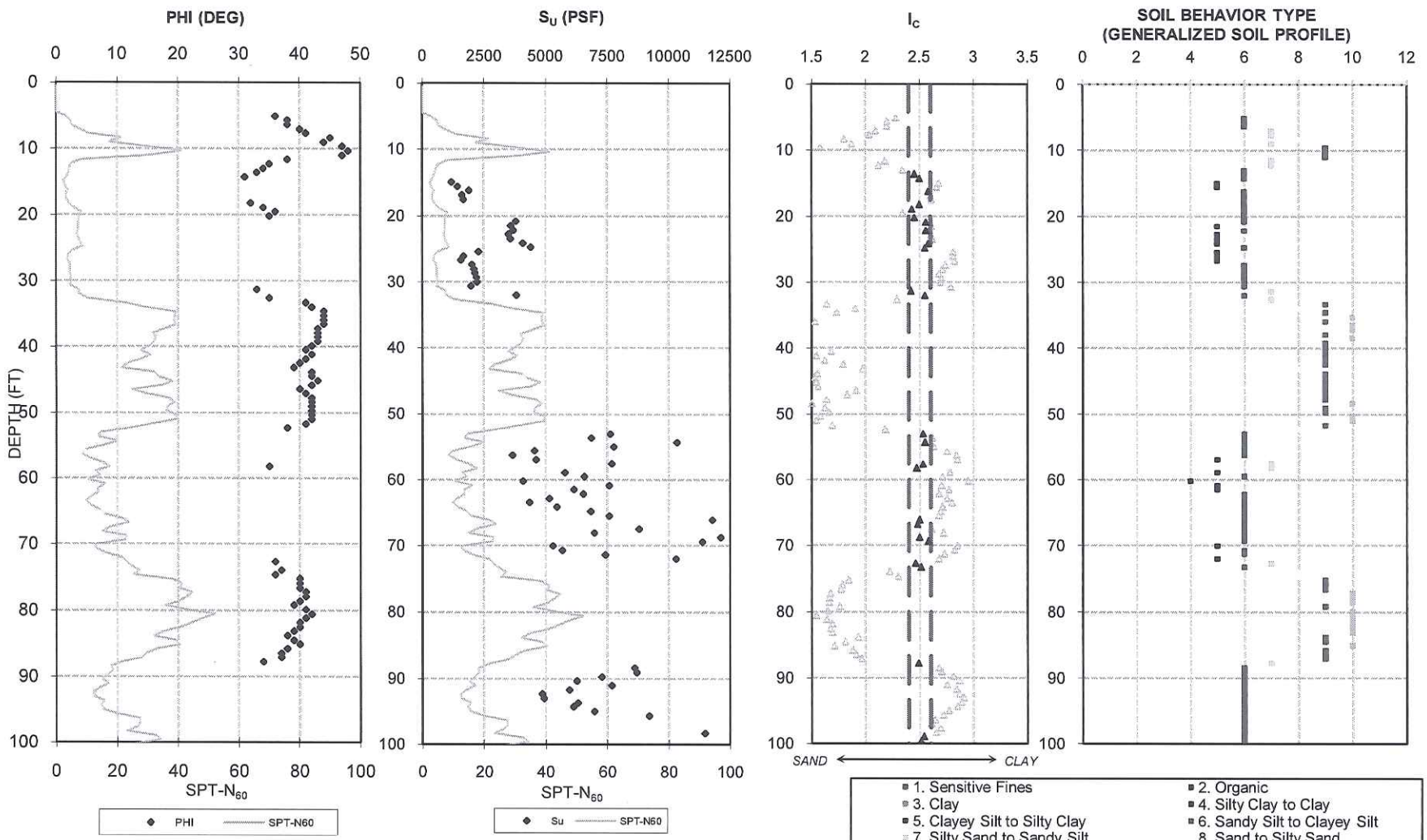
FILE => $REQUEST

USERNAME => $USER DATE PLOTTED => $DATE TIME PLOTTED => $TIME

Cone Penetrometer Test Profiles

CPT DATA CORRELATION

CPT-09-001



Reference: CPT data and correlation provided by Gregg Drilling and Testing, Inc.

PARIKH

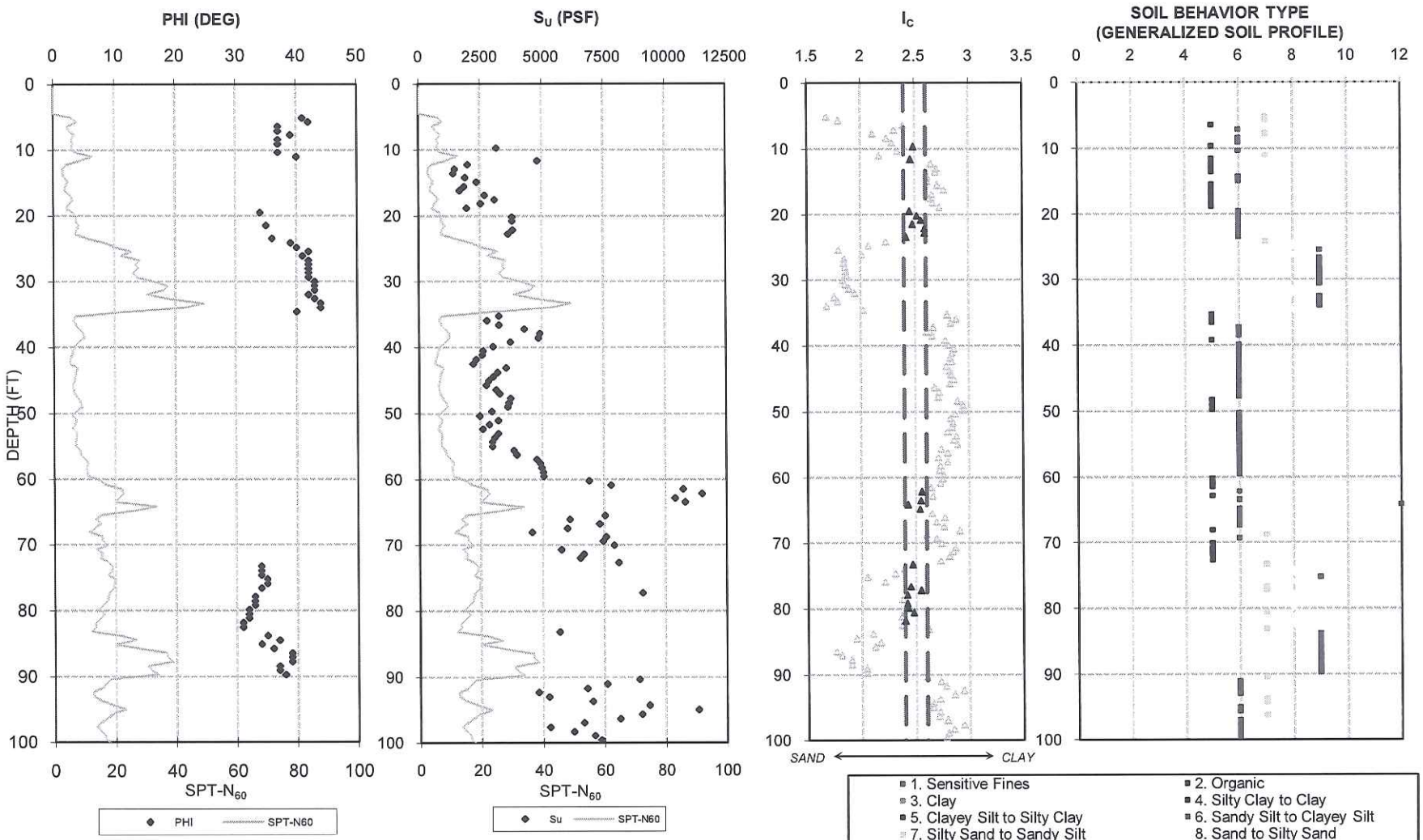
Practicing in the Geosciences

ROUTE 4 CROSSTOWN CONNECTOR
SAN JOAQUIN COUNTY, CALIFORNIA

JOB NO.:	PLATE NO.:

CPT DATA CORRELATION

CPT-09-002



Reference: CPT data and correlation provided by Gregg Drilling and Testing, Inc.

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Practicing in the Geosciences

ROUTE 4 CROSSTOWN CONNECTOR
SAN JOAQUIN COUNTY, CALIFORNIA

JOB NO.:	PLATE NO.:

APPENDIX D
RESULTS OF LABORATORY TESTS
(KLEINFELDER INVESTIGATION)

LIST OF ATTACHMENTS

The following are attached and complete this appendix.

Unconfined Compression Tests
Plasticity Charts (Atterberg Limits)
Consolidation Test Results
R-Value Test Results
Sulfate and Chloride Contents by California Laboratory Service

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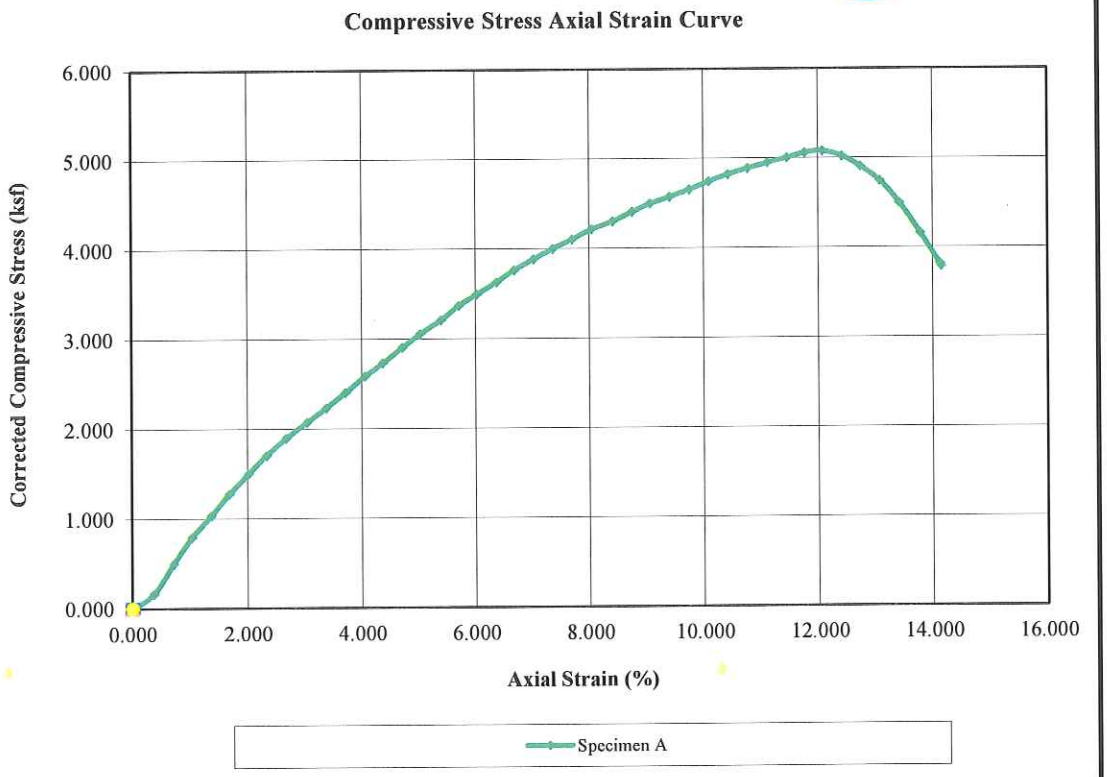
Unconfined Compression Tests

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

KLEINFELDER

Tested By: M. Baishiki — Date: 3/14/2011
Computed By: M. Baishiki — Date: 3/15/2011
Checked By: Christine Erickson — Date: 3/15/2011



Before Test	A
Water Content (%)	21.43
Dry Density (pcf)	106.80
Saturation (%)	98.92
Void Ratio	0.59
Diameter (in)	1.91
Height (in)	3.99
Test Data	**A**
Unconfined Strength (ksf)	5.08
Undrained Shear Strength (tsf)	1.27
Undrained Shear Strength (ksf)	2.54
Rate of Strain (in/min)	0.08
Strain at Failure (%)	12.07
Sample Location	B1-15-1

Specimen

Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	2/8/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Light Brown Lean Clay

Test Variables	
Specific Gravity	2.72

Remarks

Kleinfelder

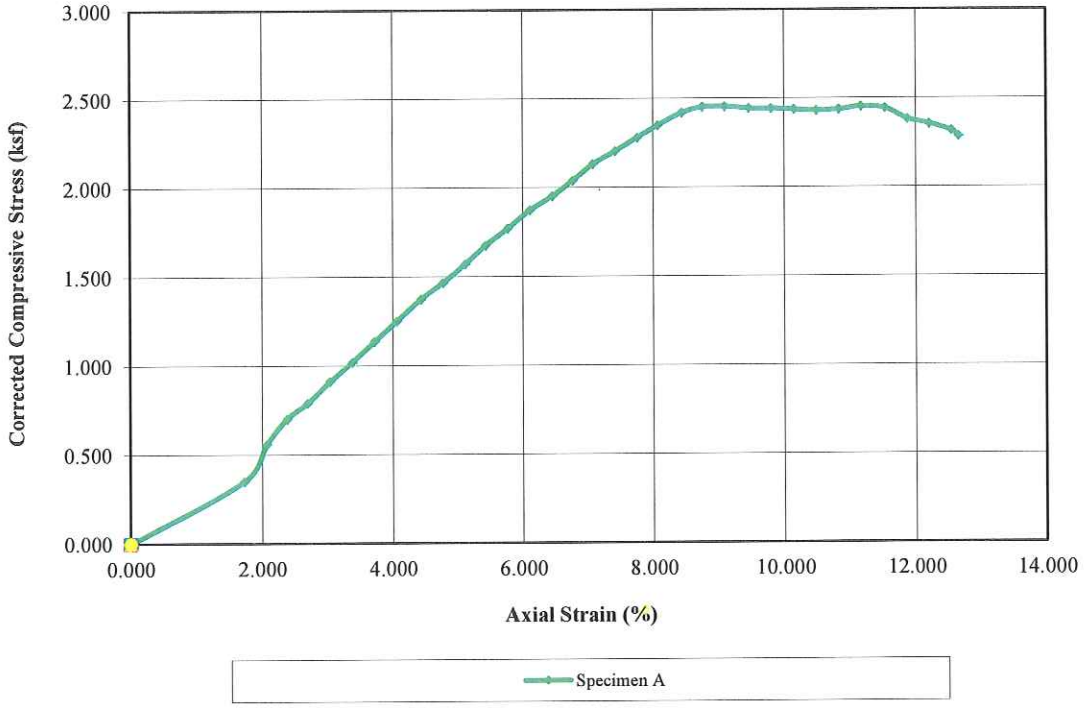
Unconfined Compression Test Report (ASTM D2166)

Tested By: M. Baishiki Date: 3/14/2011

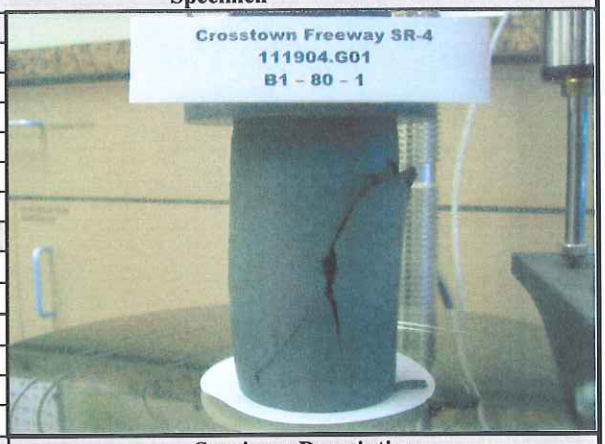
Computed By: M. Baishiki Date: 3/15/2011

Checked By: Date:

Compressive Stress Axial Strain Curve



Before Test	Specimen A
Water Content (%)	33.25
Dry Density (pcf)	88.90
Saturation (%)	99.44
Void Ratio	0.91
Diameter (in)	1.91
Height (in)	3.99
Test Data	**A**
Unconfined Strength (ksf)	2.45
Undrained Shear Strength (tsf)	0.61
Undrained Shear Strength (ksf)	1.23
Rate of Strain (in/min)	0.08
Strain at Failure (%)	9.09
Sample Location	B1-80-1



Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	2/8/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Gray Lean Clay

Test Variables	
Specific Gravity	2.72

Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

Tested By: M. Baishiki — Date: 3/14/2011

Computed By: M. Baishiki — Date: 3/15/2011

Checked By — Date

Before Test	Specimen A
Water Content (%)	27.69
Dry Density (pcf)	95.30
Saturation (%)	98.27
Void Ratio	0.76
Diameter (in)	1.91
Height (in)	3.93
Test Data	**A**
Unconfined Strength (ksf)	1.43
Undrained Shear Strength (tsf)	0.36
Undrained Shear Strength (ksf)	0.72
Rate of Strain (in/min)	0.08
Strain at Failure (%)	4.10
Sample Location	B2-30-1

Specimen

Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	2/9/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Greenish Gray Clayey Sand

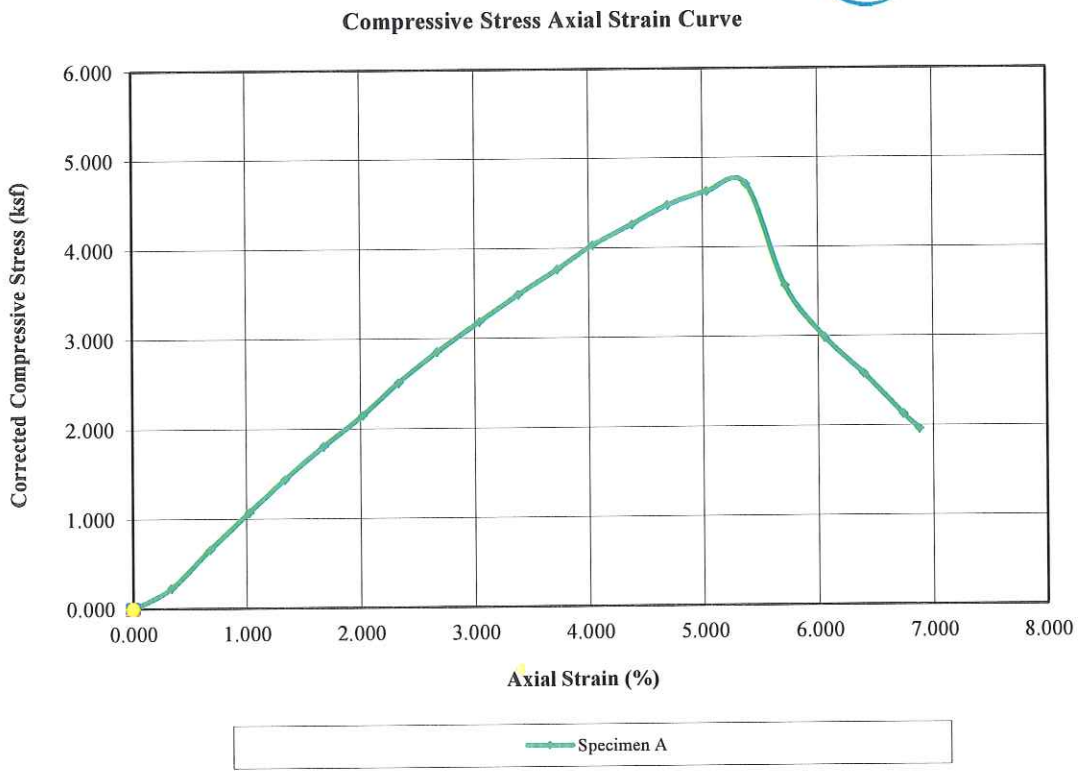
Test Variables	
Specific Gravity	2.68

Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

Tested By: M. Baishiki Date: 3/14/2011
Computed By: M. Baishiki Date: 3/15/2011
Checked By: Date:



Before Test	Specimen A
Water Content (%)	25.01
Dry Density (pcf)	100.40
Saturation (%)	98.46
Void Ratio	0.69
Diameter (in)	1.90
Height (in)	3.99
Test Data	**A**
Unconfined Strength (ksf)	4.71
Undrained Shear Strength (tsf)	1.18
Undrained Shear Strength (ksf)	2.35
Rate of Strain (in/min)	0.08
Strain at Failure (%)	5.37
Sample Location	B2-90-1

Specimen

Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	2/9/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Brownish Gray Lean Clay

Test Variables	
Specific Gravity	2.72

Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

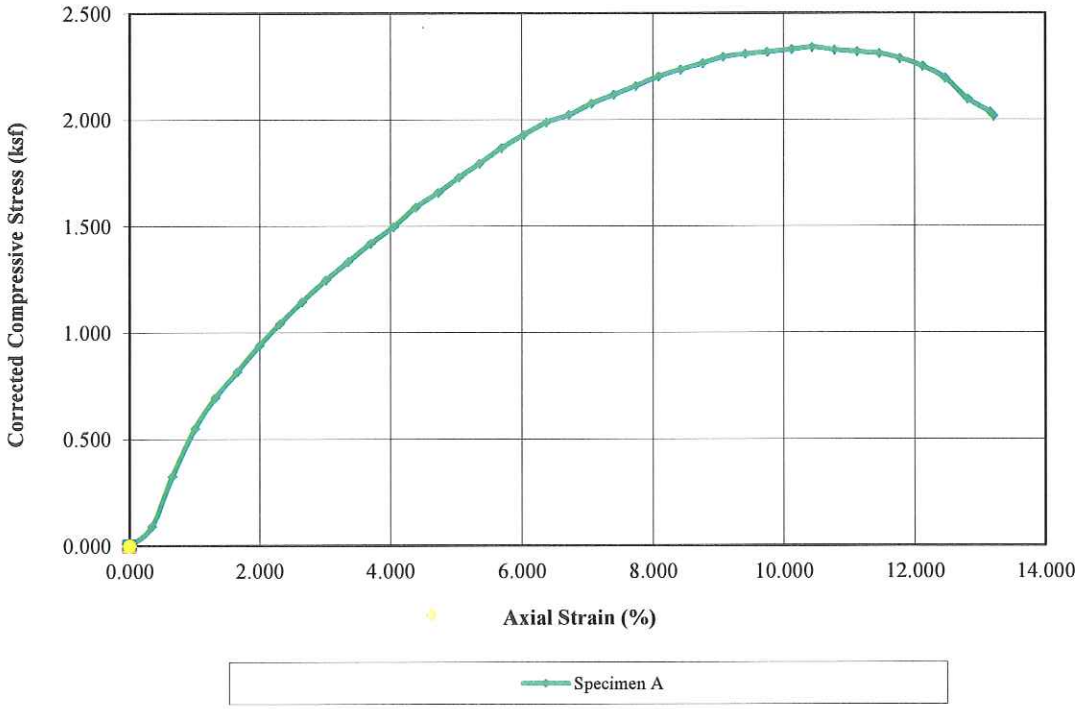
KLEINFELDER

Tested By: M. Baishiki — Date: 3/17/2011

Computed By: M. Baishiki — Date: 3/18/2011

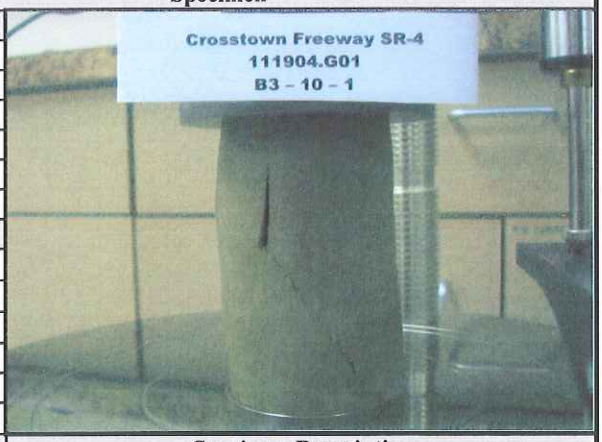
Checked By — Date

Compressive Stress Axial Strain Curve



Before Test	A
Water Content (%)	24.16
Dry Density (pcf)	102.22
Saturation (%)	99.38
Void Ratio	0.66
Diameter (in)	1.92
Height (in)	3.99
Test Data	**A**
Unconfined Strength (ksf)	2.34
Undrained Shear Strength (tsf)	0.59
Undrained Shear Strength (ksf)	1.17
Rate of Strain (in/min)	0.08
Strain at Failure (%)	10.44
Sample Location	B3-10-1

Specimen



Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	2/10/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Light Brown Lean Clay
Test Variables	
Specific Gravity	2.72

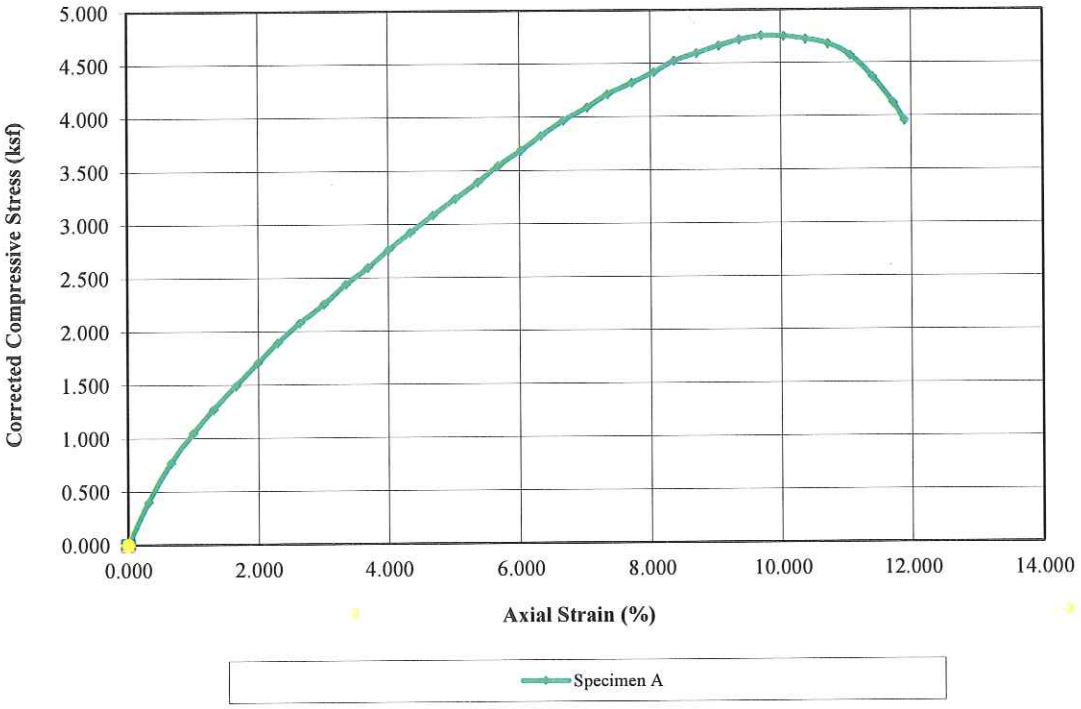
Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

Tested By: M. Baishiki Date: 3/17/2011
Computed By: M. Baishiki Date: 3/18/2011
Checked By: Date:

Compressive Stress Axial Strain Curve



Before Test	Specimen A
Water Content (%)	27.60
Dry Density (pcf)	96.60
Saturation (%)	99.16
Void Ratio	0.76
Diameter (in)	1.90
Height (in)	4.00
Test Data	**A**
Unconfined Strength (ksf)	4.76
Undrained Shear Strength (tsf)	1.19
Undrained Shear Strength (ksf)	2.38
Rate of Strain (in/min)	0.08
Strain at Failure (%)	9.71
Sample Location	B3-50-1

Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	2/10/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Greenish Gray Lean Clay

Test Variables	
Specific Gravity	2.72

Remarks

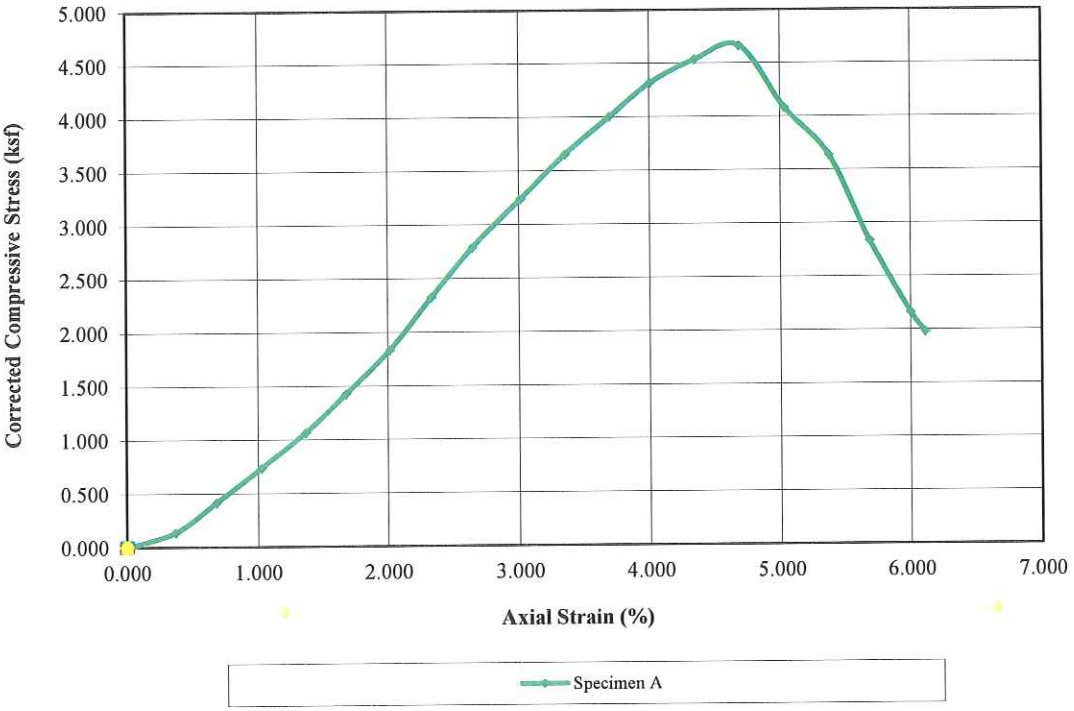
Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

KLEINFELDER

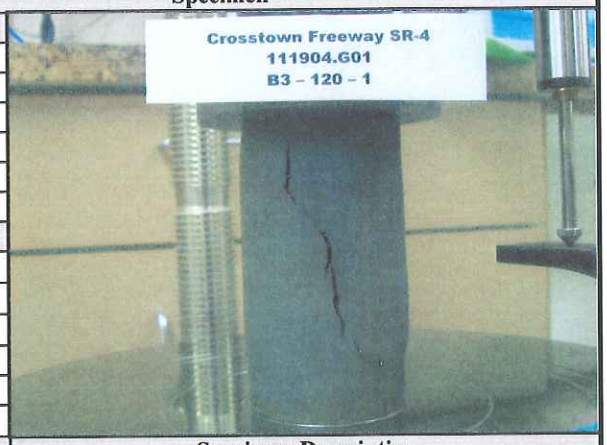
Tested By: M. Baishiki — Date: 3/17/2010
Computed By: M. Baishiki — Date: 3/18/2011
Checked By — Date

Compressive Stress Axial Strain Curve



Before Test	Specimen A
Water Content (%)	27.04
Dry Density (pcf)	97.48
Saturation (%)	99.12
Void Ratio	0.74
Diameter (in)	1.91
Height (in)	3.99
Test Data	**A**
Unconfined Strength (ksf)	4.67
Undrained Shear Strength (tsf)	1.17
Undrained Shear Strength (ksf)	2.33
Rate of Strain (in/min)	0.08
Strain at Failure (%)	4.69
Sample Location	B3-120-1

Specimen



Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	2/11/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Greenish Gray Lean Clay

Test Variables	
Specific Gravity	2.72

Remarks

Kleinfelder

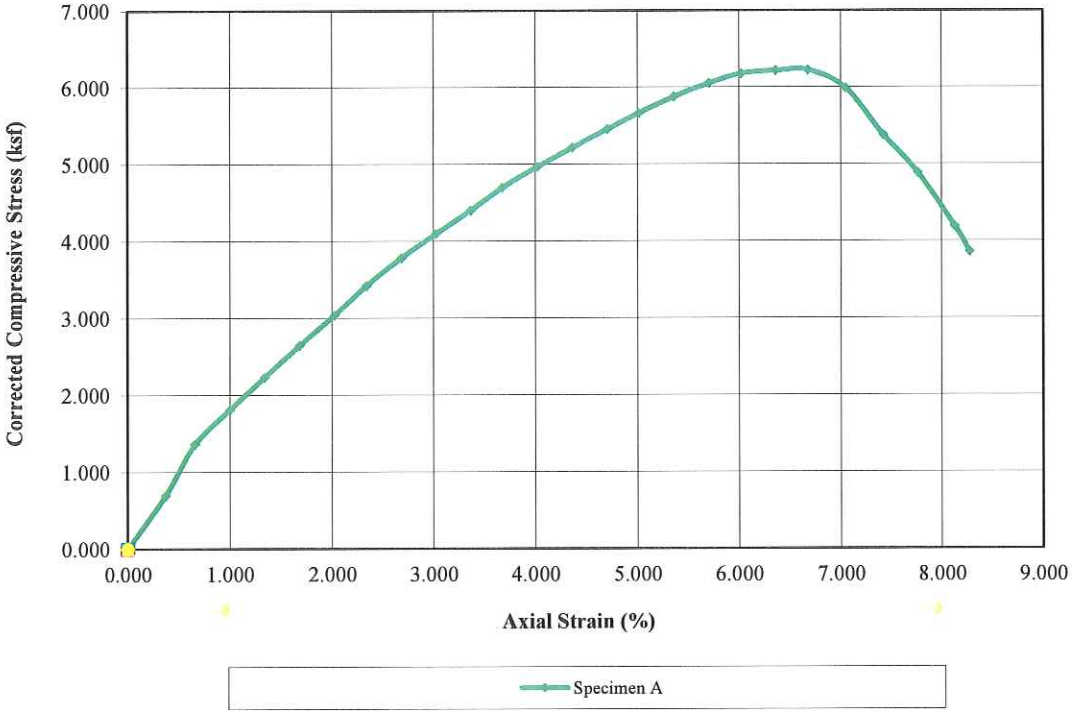
Unconfined Compression Test Report (ASTM D2166)

Date: 3/28/2011 — Tested By: M. Baishiki

Date: 3/29/2011 — Computed By: M. Baishiki

Date: — Checked By:

Compressive Stress Axial Strain Curve



Before Test	A
Water Content (%)	24.88
Dry Density (pcf)	101.30
Saturation (%)	100.01
Void Ratio	0.68
Diameter (in)	1.92
Height (in)	3.98
Test Data	**A**
Unconfined Strength (ksf)	6.22
Undrained Shear Strength (tsf)	1.56
Undrained Shear Strength (ksf)	3.11
Rate of Strain (in/min)	0.08
Strain at Failure (%)	6.68
Sample Location	B3-140-1

Specimen

Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	2/11/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Gray Lean Clay

Test Variables	
Specific Gravity	2.72

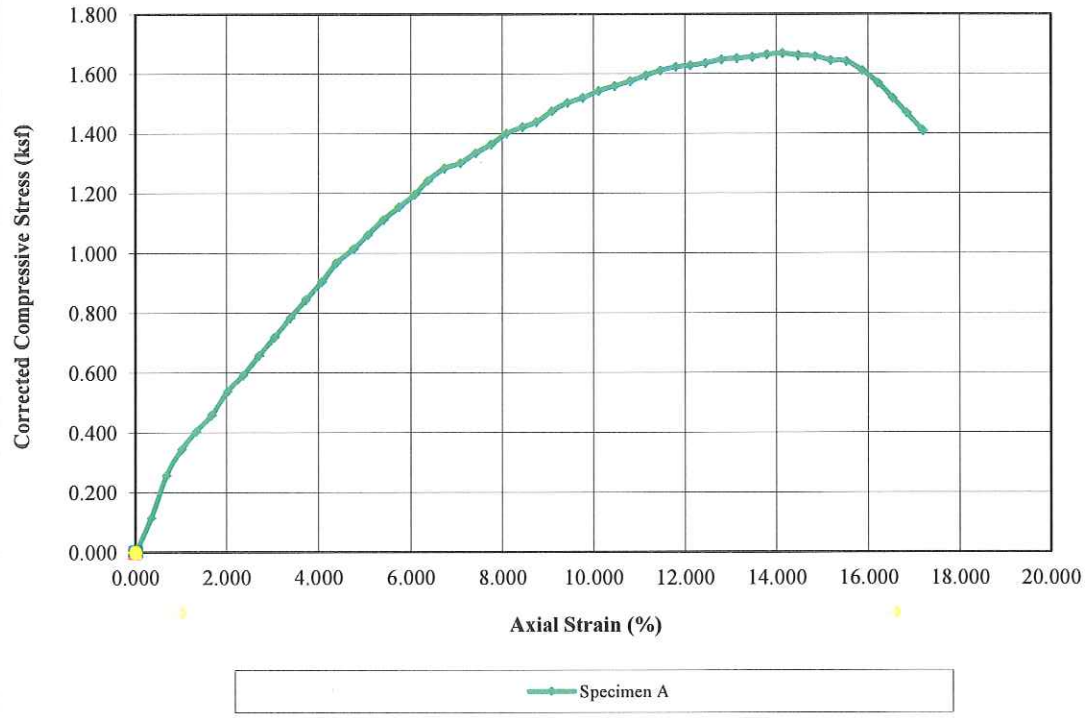
Remarks

Kleinfelder

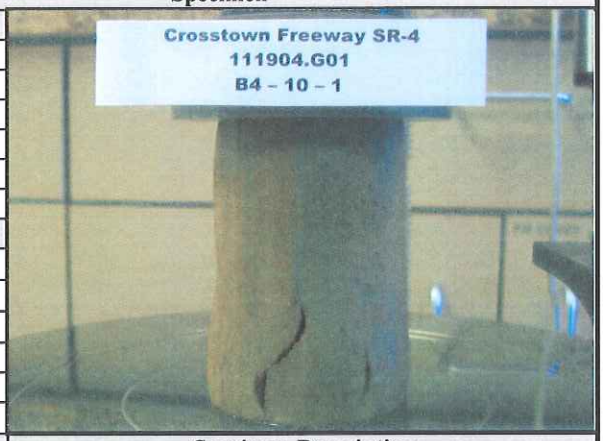
Unconfined Compression Test Report (ASTM D2166)

Tested By: M. Baishiki — Date: 3/30/2011
Computed By: M. Baishiki — Date: 3/31/2011
Checked By — Date

Compressive Stress Axial Strain Curve



Before Test	Specimen A
Water Content (%)	20.87
Dry Density (pcf)	107.50
Saturation (%)	99.22
Void Ratio	0.57
Diameter (in)	1.92
Height (in)	3.99
Test Data	**A**
Unconfined Strength (ksf)	1.67
Undrained Shear Strength (tsf)	0.42
Undrained Shear Strength (ksf)	0.83
Rate of Strain (in/min)	0.08
Strain at Failure (%)	14.13
Sample Location	B4-10-1



Project Information		Specimen Description	
Project Num	111904.G01	Specimen A	Brown Sandy Lean Clay
Project	Crosstown Freeway SR-4		
Sampling Date	2/3/2011		
Sample #	1103017		
Client		**Test Variables**	
		Specific Gravity	2.70

Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

KLEINFELDER

Date

Checked By

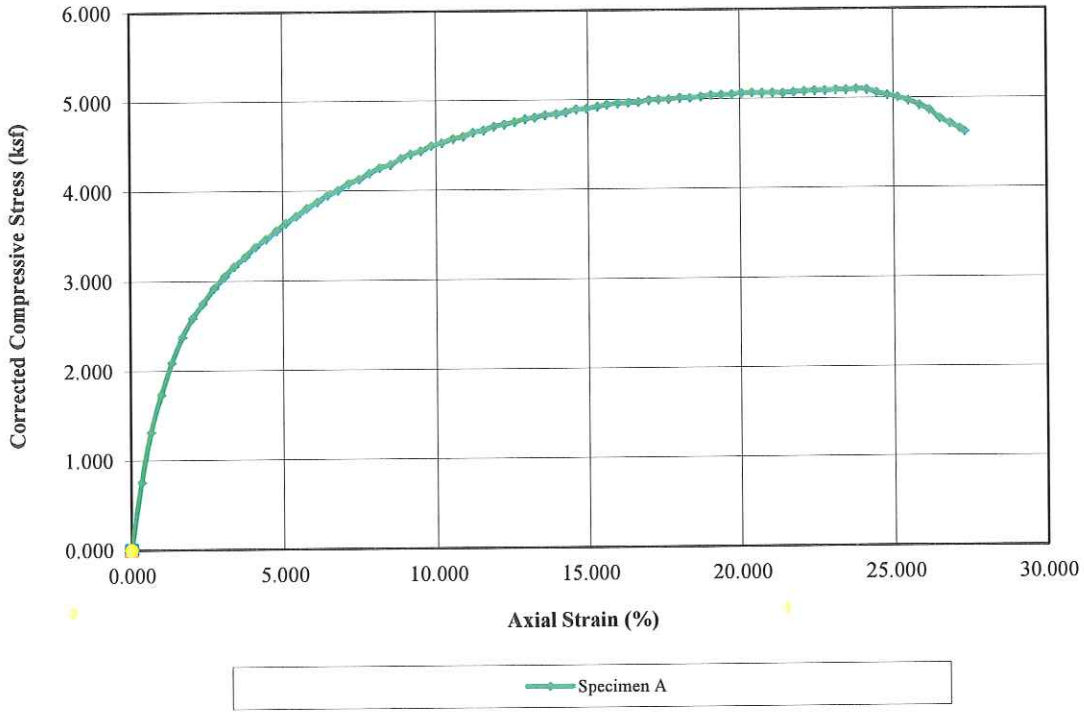
3/31/2011 Date

M. Baishiki Computed By

3/30/2011 Date

M. Baishiki Tested By

Compressive Stress Axial Strain Curve



Before Test	Specimen A
Water Content (%)	23.11
Dry Density (pcf)	103.05
Saturation (%)	98.16
Void Ratio	0.64
Diameter (in)	1.92
Height (in)	3.98
Test Data	**A**
Unconfined Strength (ksf)	5.10
Undrained Shear Strength (tsf)	1.28
Undrained Shear Strength (ksf)	2.55
Rate of Strain (in/min)	0.08
Strain at Failure (%)	23.81
Sample Location	B4-35-1

Specimen

Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	2/3/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Brownish Gray Lean Clay w/ Sand

Test Variables	
Specific Gravity	2.70

Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

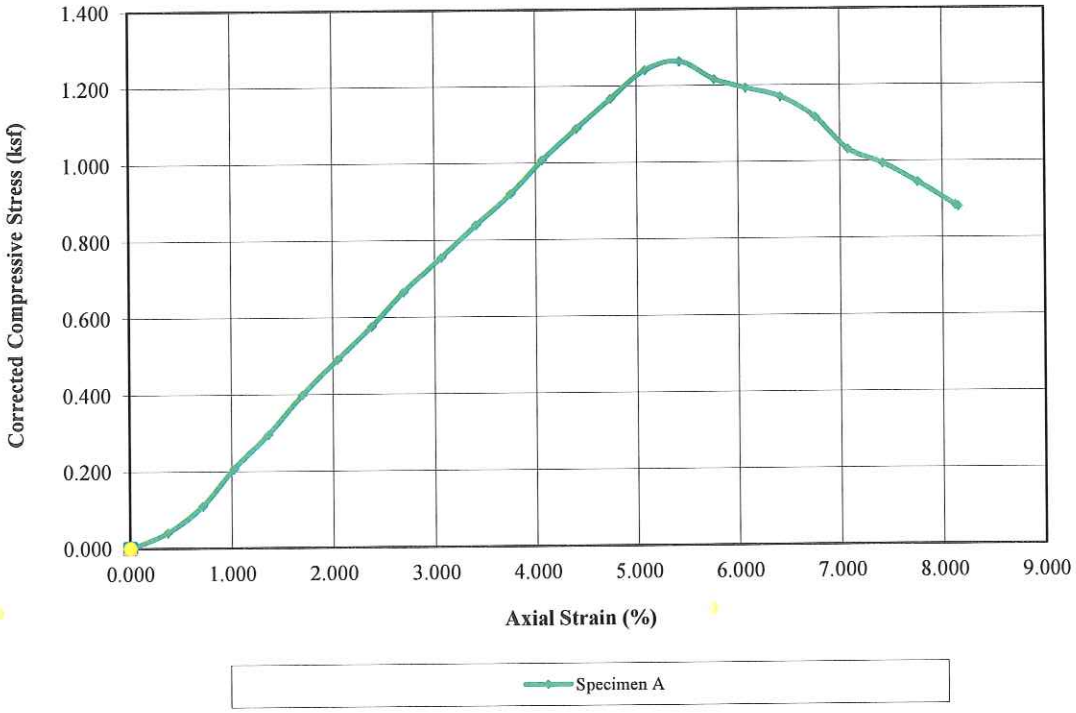
KLEINFELDER

Tested By: M. Baishiki — Date: 3/30/2011

Computed By: M. Baishiki — Date: 3/31/2011

Checked By: — Date:

Compressive Stress Axial Strain Curve



Before Test	Specimen A
Water Content (%)	27.78
Dry Density (pcf)	96.70
Saturation (%)	99.90
Void Ratio	0.76
Diameter (in)	1.92
Height (in)	3.99
Test Data	**A**
Unconfined Strength (ksf)	1.26
Undrained Shear Strength (tsf)	0.32
Undrained Shear Strength (ksf)	0.63
Rate of Strain (in/min)	0.08
Strain at Failure (%)	5.43
Sample Location	B4-65-1

Specimen

Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	2/3/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Brownish Gray Silt w/ Sand

Test Variables	
Specific Gravity	2.72

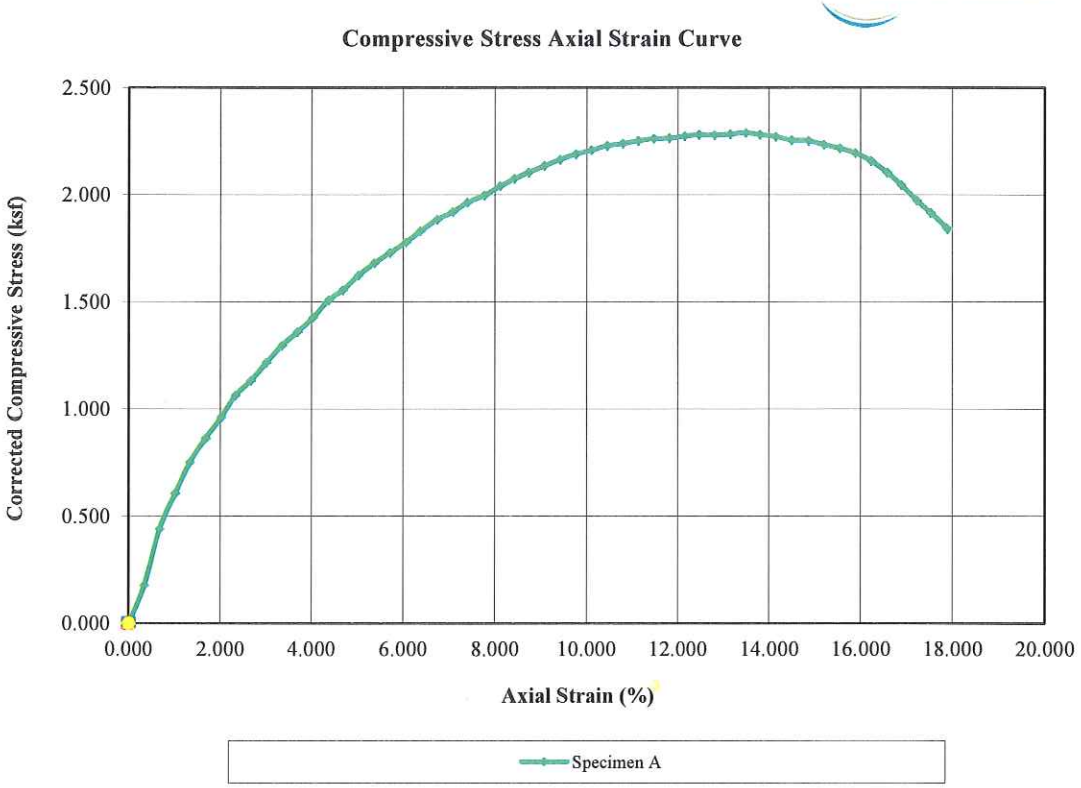
Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

KLEINFELDER

Tested By: M. Baishiki — Date: 3/17/2011
Computed By: M. Baishiki — Date: 3/18/2011
Checked By: — Date:



Before Test	A
Water Content (%)	20.94
Dry Density (pcf)	108.10
Saturation (%)	99.70
Void Ratio	0.57
Diameter (in)	1.91
Height (in)	3.99
Test Data	**A**
Unconfined Strength (ksf)	2.29
Undrained Shear Strength (tsf)	0.57
Undrained Shear Strength (ksf)	1.14
Rate of Strain (in/min)	0.08
Strain at Failure (%)	13.49
Sample Location	B5-20-1

Specimen

Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	2/2/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Brown Lean Clay

Test Variables	
Specific Gravity	2.72

Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

KLEINFELDER

Date

Checked By

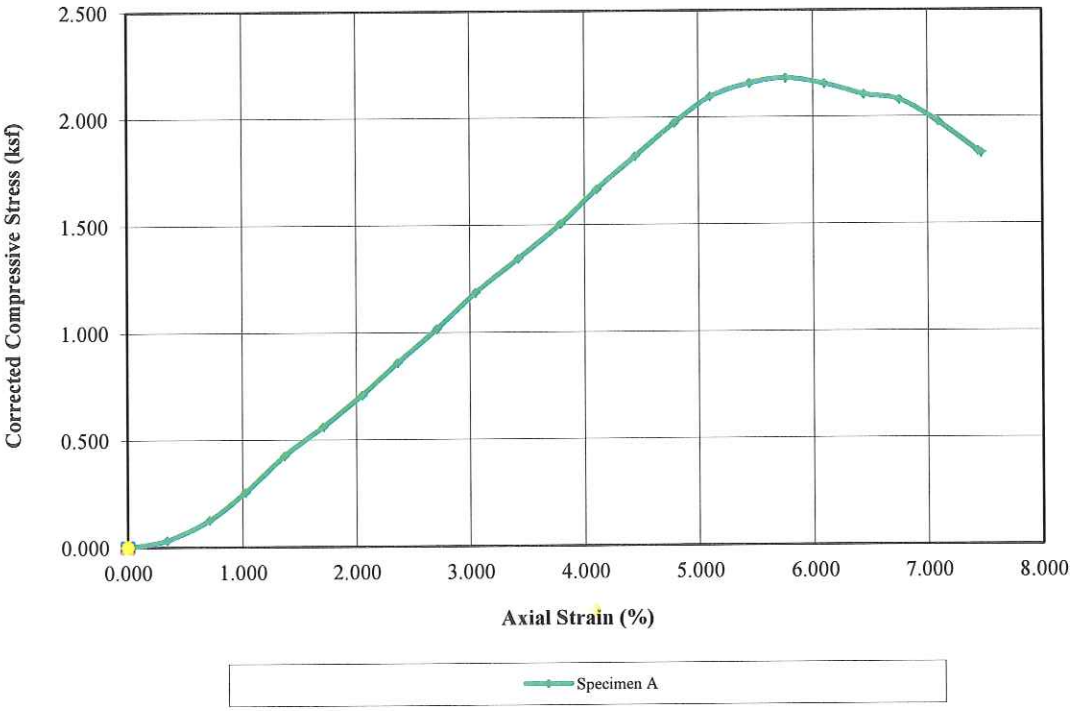
3/18/2011 Date

M. Baishiki Computed By

3/17/2011 Date

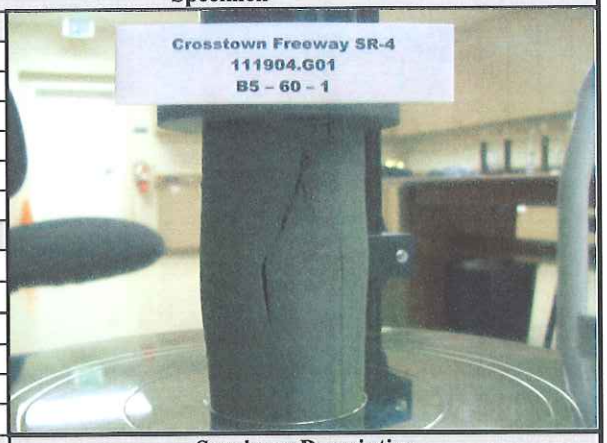
M. Baishiki Tested By

Compressive Stress Axial Strain Curve



Before Test	Specimen A
Water Content (%)	20.02
Dry Density (pcf)	110.10
Saturation (%)	99.84
Void Ratio	0.55
Diameter (in)	1.92
Height (in)	3.98
Test Data	**A**
Unconfined Strength (ksf)	2.18
Undrained Shear Strength (tsf)	0.55
Undrained Shear Strength (ksf)	1.09
Rate of Strain (in/min)	0.08
Strain at Failure (%)	5.76
Sample Location	B5-60-1

Specimen



Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	2/2/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Gray Sandy Lean Clay

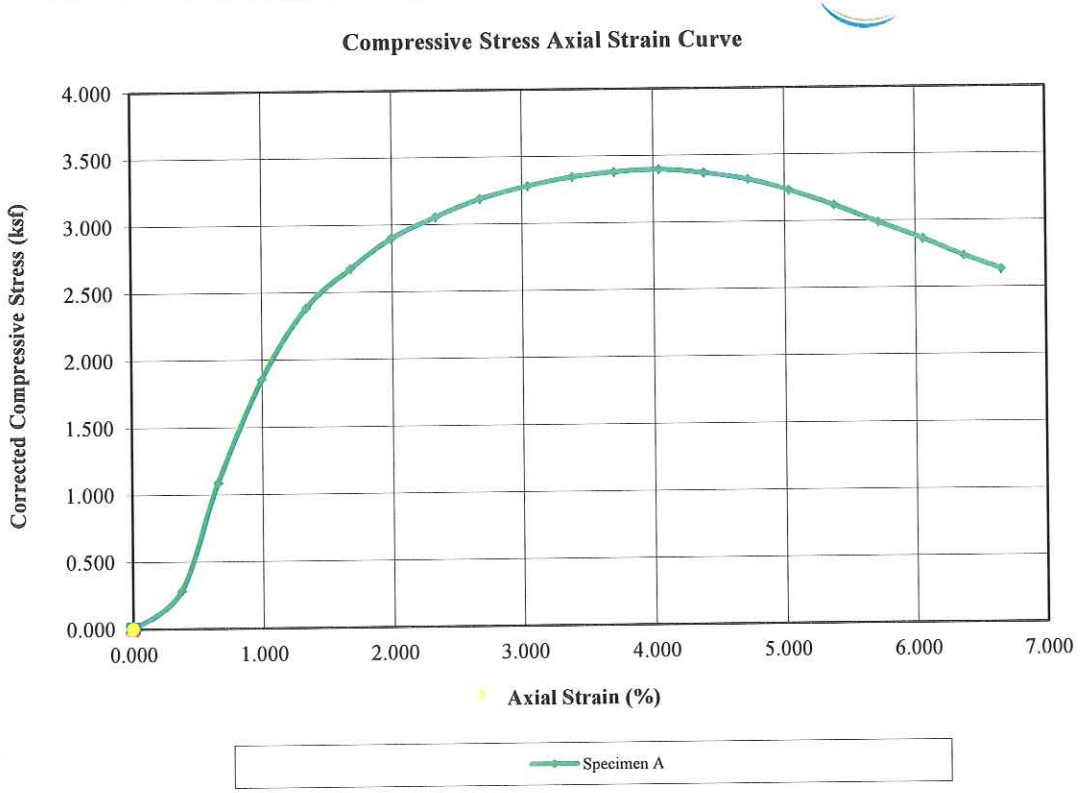
Test Variables	
Specific Gravity	2.73

Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

Tested By: M. Baishiki Date: 3/17/2011
Computed By: M. Baishiki Date: 3/18/2011
Checked By: Date:



Before Test	A
Water Content (%)	22.44
Dry Density (pcf)	103.70
Saturation (%)	95.70
Void Ratio	0.64
Diameter (in)	1.90
Height (in)	3.99
Test Data	**A**
Unconfined Strength (ksf)	3.40
Undrained Shear Strength (tsf)	0.85
Undrained Shear Strength (ksf)	1.70
Rate of Strain (in/min)	0.08
Strain at Failure (%)	4.04
Sample Location	B6-15-1

Specimen

Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	2/10/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Brown Lean Clay w/ Sand

Test Variables	
Specific Gravity	2.72

Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

KLEINFELDER

Date

Checked By

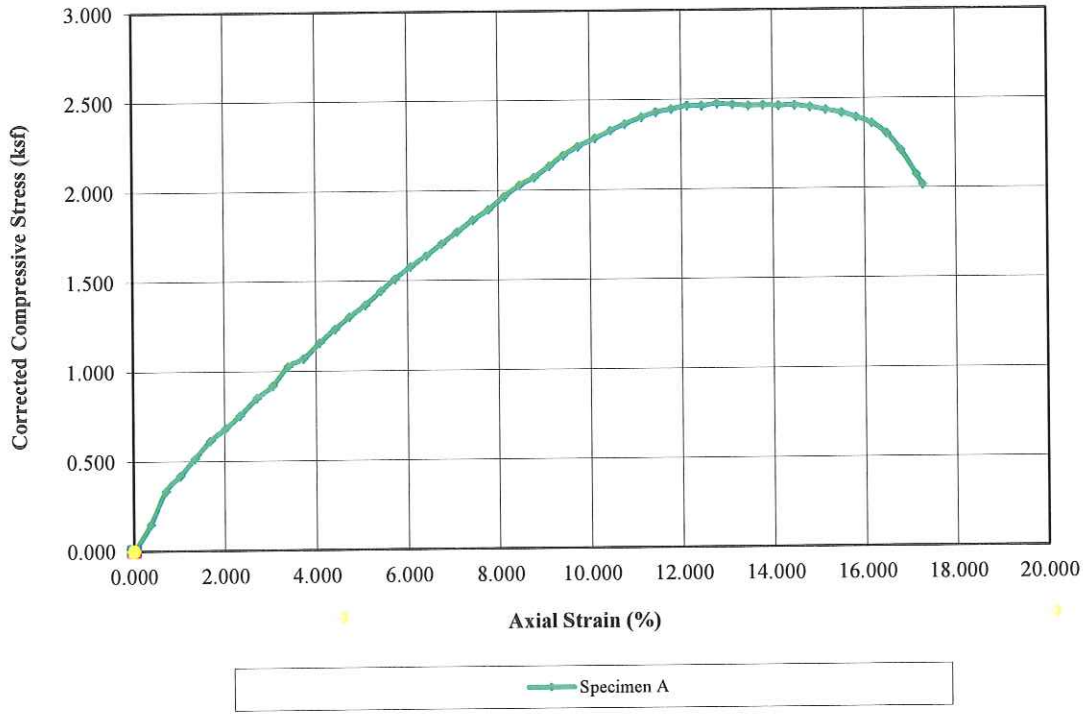
Date 3/31/2011

Computed By M. Baishiki

Date 3/30/2011

Tested By M. Baishiki

Compressive Stress Axial Strain Curve



Before Test	Specimen A
Water Content (%)	27.66
Dry Density (pcf)	94.60
Saturation (%)	95.63
Void Ratio	0.78
Diameter (in)	1.92
Height (in)	3.98
Test Data	**A**
Unconfined Strength (ksf)	2.48
Undrained Shear Strength (tsf)	0.62
Undrained Shear Strength (ksf)	1.24
Rate of Strain (in/min)	0.08
Strain at Failure (%)	12.80
Sample Location	B6-50-1

Specimen

Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	2/1/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Brownish Gray Lean Clay w/ Sand

Test Variables	
Specific Gravity	2.70

Remarks

Kleinfelder

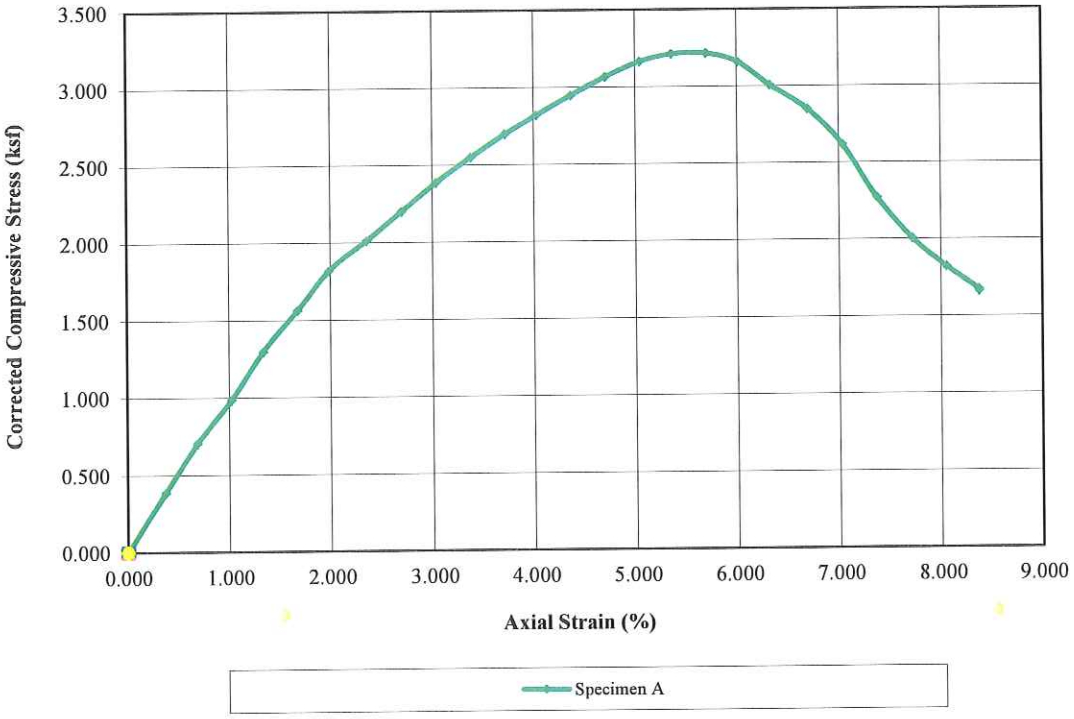
Unconfined Compression Test Report (ASTM D2166)

Date: 3/30/2011 — Tested By: M. Baishiki

Date: 3/31/2011 — Computed By: M. Baishiki

Checked By:

Compressive Stress Axial Strain Curve



Before Test	Specimen A
Water Content (%)	33.01
Dry Density (pcf)	89.49
Saturation (%)	99.64
Void Ratio	0.90
Diameter (in)	1.92
Height (in)	4.00
Test Data	**A**
Unconfined Strength (ksf)	3.22
Undrained Shear Strength (tsf)	0.80
Undrained Shear Strength (ksf)	1.61
Rate of Strain (in/min)	0.08
Strain at Failure (%)	5.70
Sample Location	B6-80-1

Specimen

Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	2/1/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Gray Lean Clay

Test Variables	
Specific Gravity	2.73

Remarks

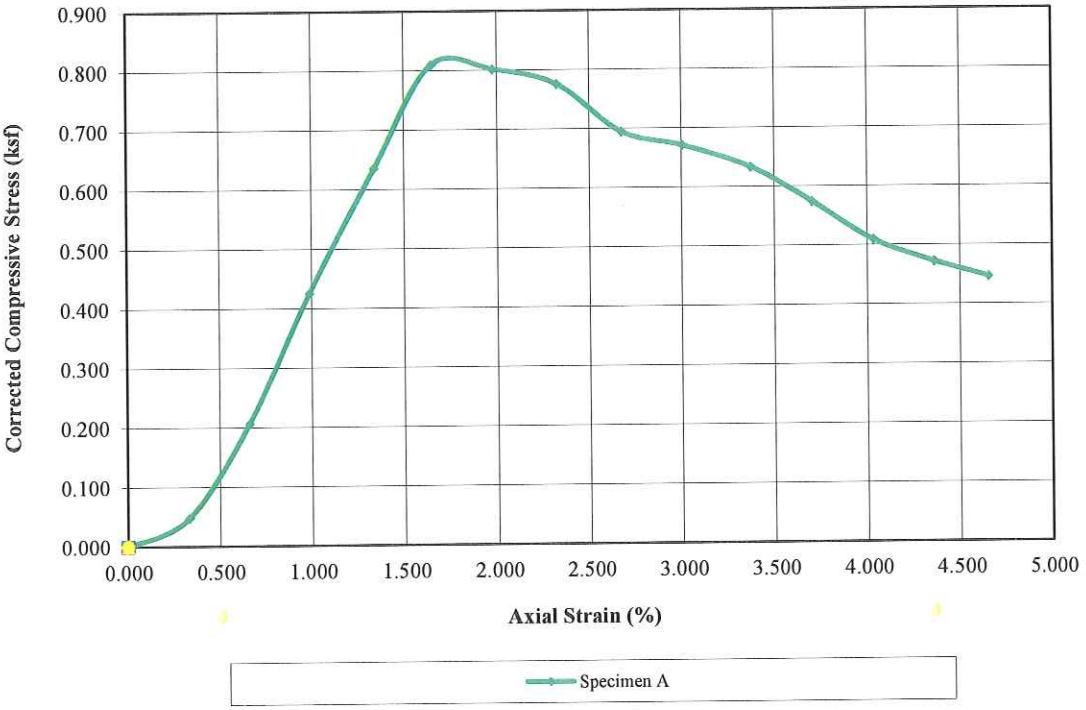
Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

KLEINFELDER

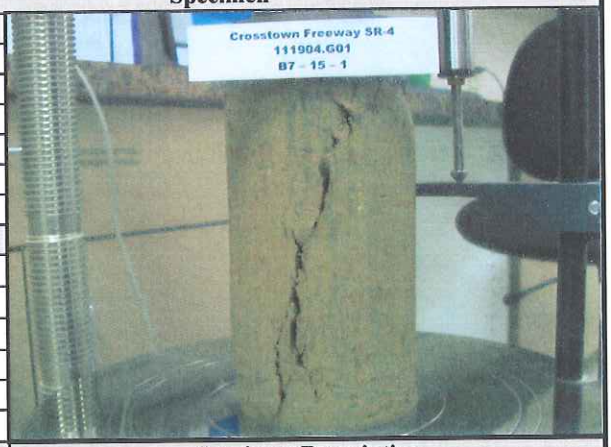
Tested By: M. Baishiki — Date: 3/31/2011
Computed By: M. Baishiki — Date: 4/1/2011
Checked By: — Date:

Compressive Stress Axial Strain Curve



Before Test	Specimen A
Water Content (%)	28.59
Dry Density (pcf)	94.30
Saturation (%)	98.08
Void Ratio	0.79
Diameter (in)	2.87
Height (in)	5.85
Test Data	**A**
Unconfined Strength (ksf)	0.81
Undrained Shear Strength (tsf)	0.20
Undrained Shear Strength (ksf)	0.41
Rate of Strain (in/min)	0.12
Strain at Failure (%)	1.65
Sample Location	B7-15-1

Specimen



Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	1/31/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Light Brown Lean Clay

Test Variables	
Specific Gravity	2.70

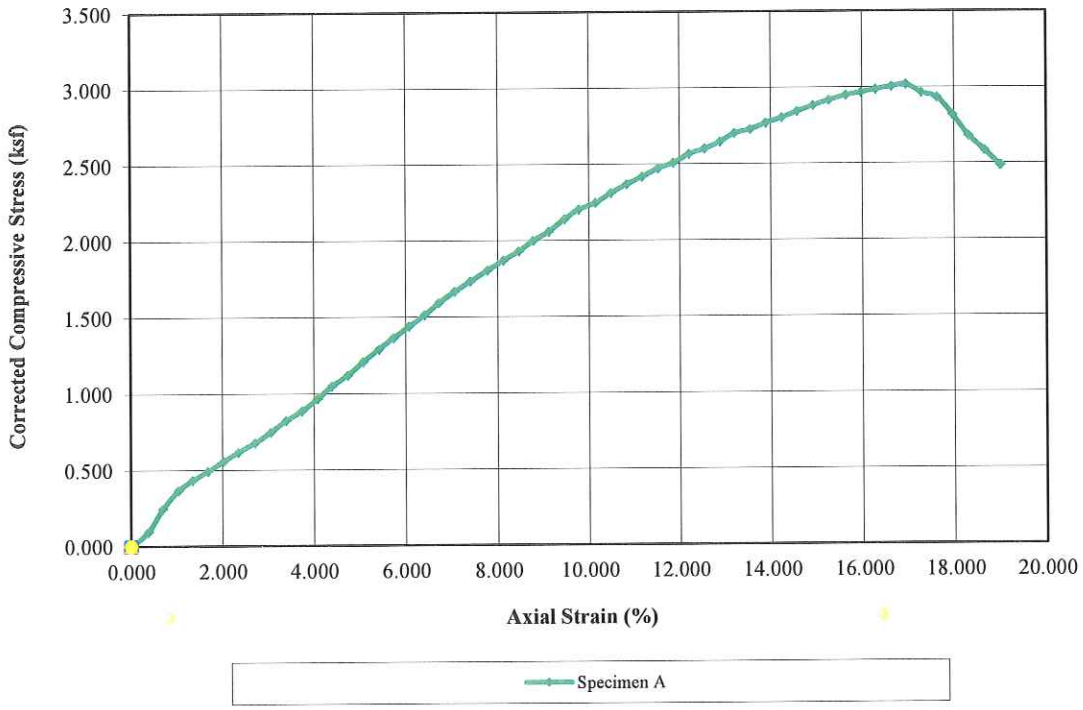
Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

Tested By: M. Baishiki | Date: 3/30/2011
Computed By: M. Baishiki | Date: 3/31/2011
Checked By | Date

Compressive Stress Axial Strain Curve



Before Test	Specimen A
Water Content (%)	21.90
Dry Density (pcf)	105.70
Saturation (%)	99.52
Void Ratio	0.60
Diameter (in)	1.92
Height (in)	3.98
Test Data	**A**
Unconfined Strength (ksf)	3.02
Undrained Shear Strength (tsf)	0.75
Undrained Shear Strength (ksf)	1.51
Rate of Strain (in/min)	0.08
Strain at Failure (%)	16.94
Sample Location	B7-45-1

Specimen

Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	1/31/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Greenish Gray Lean Clay

Test Variables	
Specific Gravity	2.70

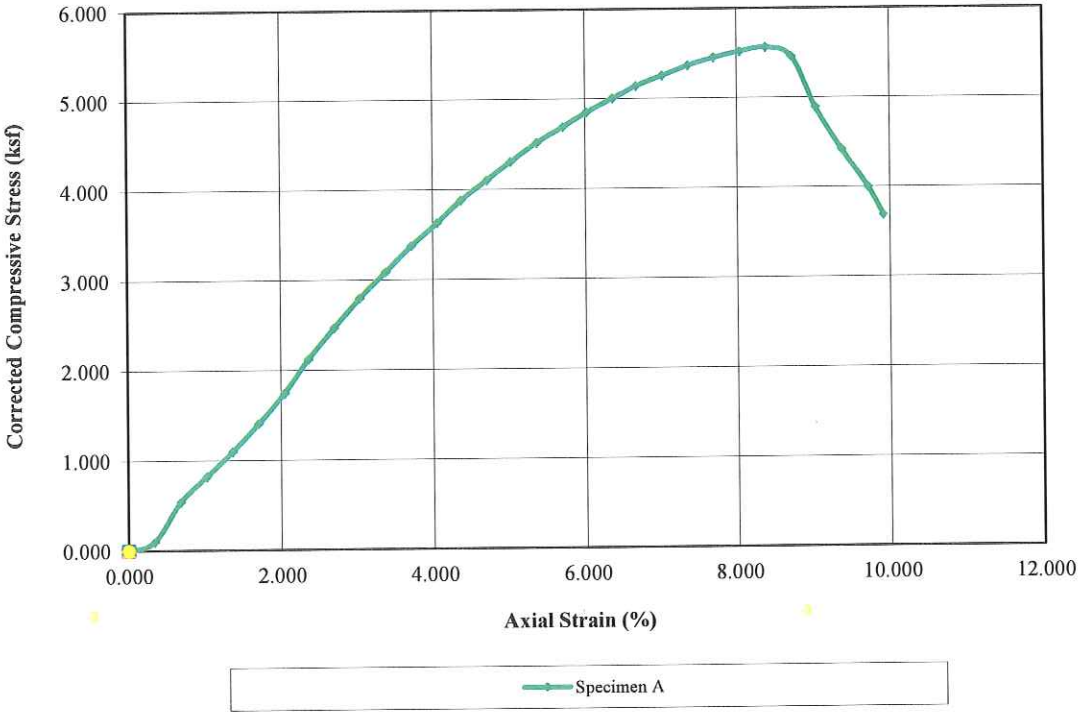
Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

Tested By: M. Baishiki — Date: 3/28/2011
Computed By: M. Baishiki — Date: 3/29/2011
Checked By — Date

Compressive Stress Axial Strain Curve



Before Test	Specimen A
Water Content (%)	24.57
Dry Density (pcf)	101.70
Saturation (%)	99.78
Void Ratio	0.67
Diameter (in)	1.92
Height (in)	3.99
Test Data	**A**
Unconfined Strength (ksf)	5.56
Undrained Shear Strength (tsf)	1.39
Undrained Shear Strength (ksf)	2.78
Rate of Strain (in/min)	0.08
Strain at Failure (%)	8.38
Sample Location	B7-70-1

Specimen

Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	1/31/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Brownish Gray Lean Clay

Test Variables	
Specific Gravity	2.72

Remarks

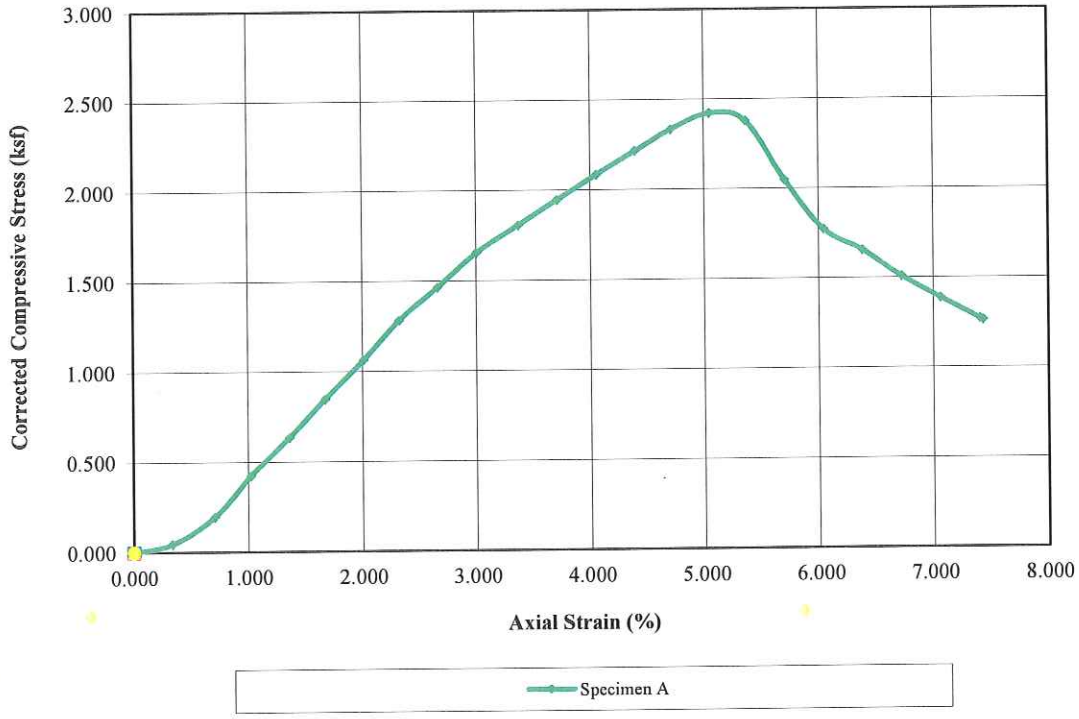
Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

KLEINFELDER

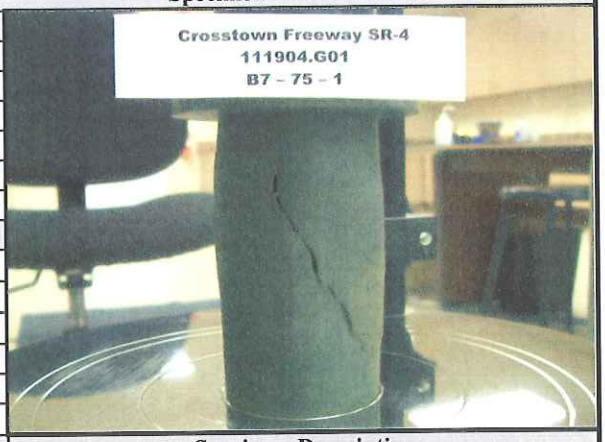
Tested By: M. Baishiki — Date: 3/30/2011
Computed By: M. Baishiki — Date: 3/31/2011
Checked By — Date

Compressive Stress Axial Strain Curve



Before Test	Specimen A
Water Content (%)	31.92
Dry Density (pcf)	91.00
Saturation (%)	99.86
Void Ratio	0.87
Diameter (in)	1.92
Height (in)	4.00
Test Data	**A**
Unconfined Strength (ksf)	2.42
Undrained Shear Strength (tsf)	0.61
Undrained Shear Strength (ksf)	1.21
Rate of Strain (in/min)	0.08
Strain at Failure (%)	5.05
Sample Location	B7-75-1

Specimen



Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	1/31/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Brownish Gray Lean Clay

Test Variables	
Specific Gravity	2.73

Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

Date

Checked By

4/4/20111

Date

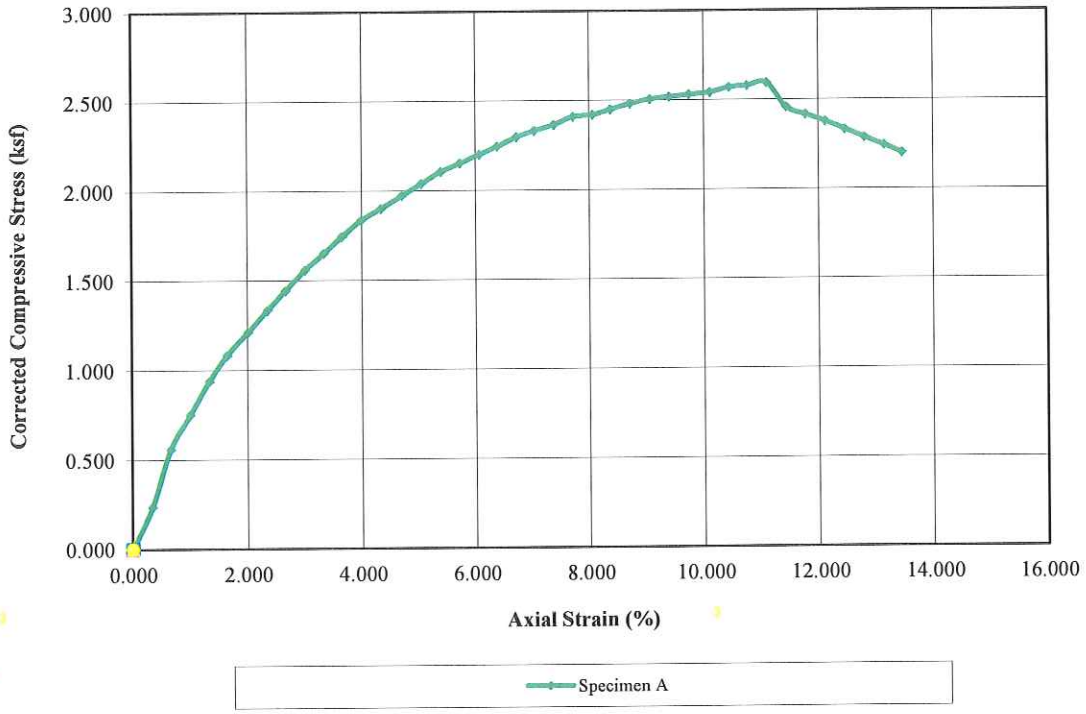
Computed By M. Baishiki

4/1/2011

Date

Tested By M. Baishiki

Compressive Stress Axial Strain Curve



Before Test	A
Water Content (%)	30.09
Dry Density (pcf)	93.40
Saturation (%)	99.94
Void Ratio	0.82
Diameter (in)	1.92
Height (in)	4.00
Test Data	**A**
Unconfined Strength (ksf)	2.59
Undrained Shear Strength (tsf)	0.65
Undrained Shear Strength (ksf)	1.30
Rate of Strain (in/min)	0.08
Strain at Failure (%)	11.10
Sample Location	B8-20-1

Specimen

Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	1/28/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Light Brown Lean Clay

Test Variables	
Specific Gravity	2.72

Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

Date

Checked By

4/4/2011 Date

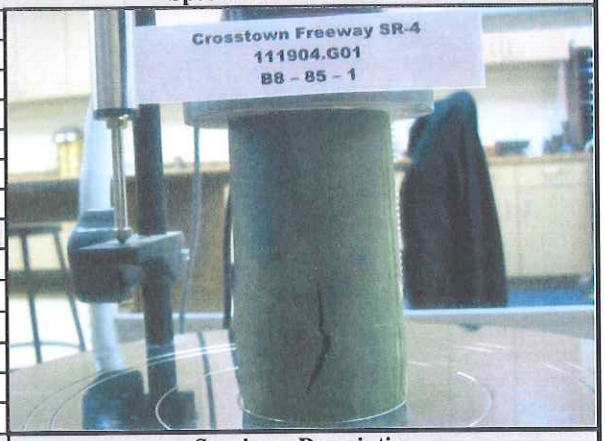
Computed By M. Baishiki

4/1/2011 Date

Tested By M. Baishiki

Before Test	Specimen A
Water Content (%)	18.68
Dry Density (pcf)	112.79
Saturation (%)	99.80
Void Ratio	0.51
Diameter (in)	1.92
Height (in)	3.99
Test Data	**A**
Unconfined Strength (ksf)	12.49
Undrained Shear Strength (tsf)	3.12
Undrained Shear Strength (ksf)	6.25
Rate of Strain (in/min)	0.08
Strain at Failure (%)	6.68
Sample Location	B8-85-1

Specimen



Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	1/28/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Greenish Gray Lean Clay

Test Variables	
Specific Gravity	2.73

Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

KLEINFELDER

Date

Checked By

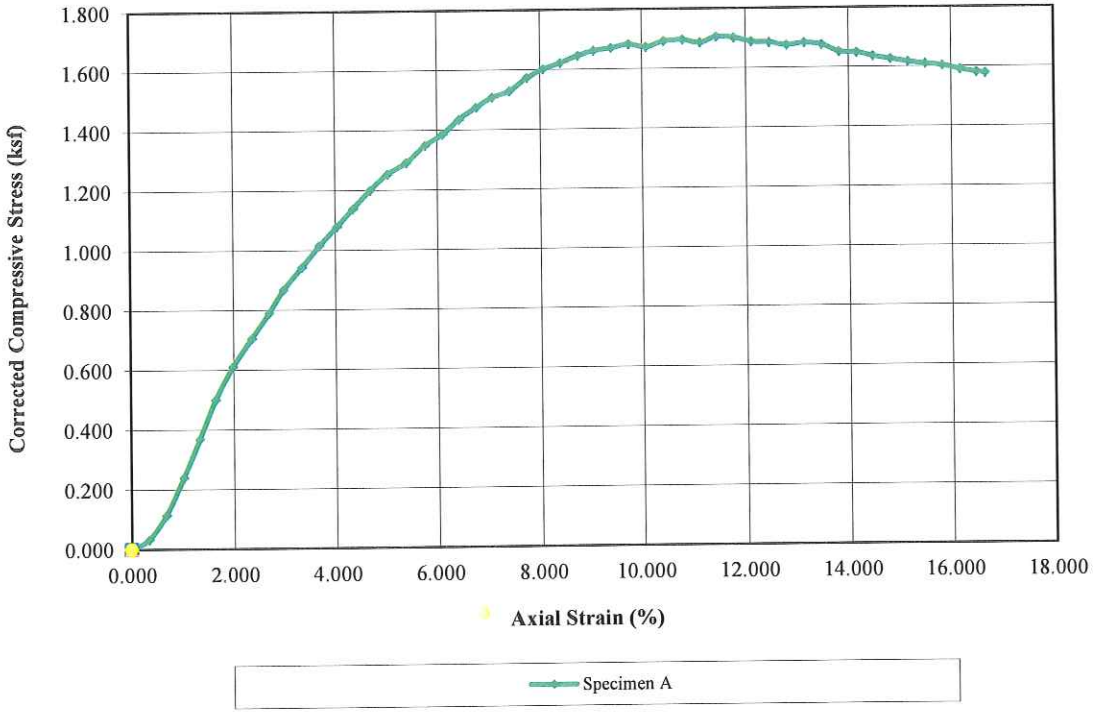
Date 4/4/2011

Computed By M. Baishiki

Date 4/1/2011

Tested By M. Baishiki

Compressive Stress Axial Strain Curve



Before Test	A
Water Content (%)	28.34
Dry Density (pcf)	96.00
Saturation (%)	99.91
Void Ratio	0.77
Diameter (in)	1.92
Height (in)	4.00
Test Data	**A**
Unconfined Strength (ksf)	1.71
Undrained Shear Strength (tsf)	0.43
Undrained Shear Strength (ksf)	0.85
Rate of Strain (in/min)	0.08
Strain at Failure (%)	11.44
Sample Location	B9-15-1

Specimen

Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	1/25/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Light Brown Lean Clay

Test Variables	
Specific Gravity	2.73

Remarks

Kleinfelder

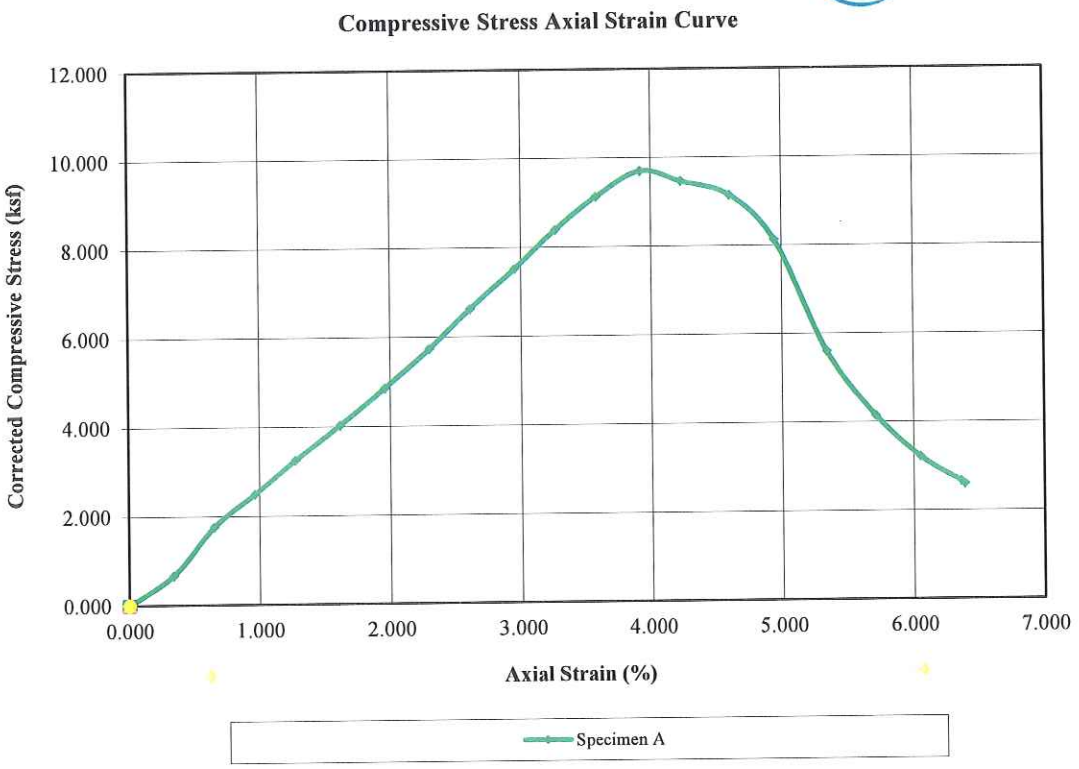
Unconfined Compression Test Report (ASTM D2166)

KLEINFELDER

Tested By: M. Baishiki — Date: 4/1/2011

Computed By: M. Baishiki — Date: 4/4/2011

Checked By: — Date:



Before Test	A
Water Content (%)	16.46
Dry Density (pcf)	117.47
Saturation (%)	99.67
Void Ratio	0.45
Diameter (in)	1.92
Height (in)	3.99
Test Data	**A**
Unconfined Strength (ksf)	9.70
Undrained Shear Strength (tsf)	2.42
Undrained Shear Strength (ksf)	4.85
Rate of Strain (in/min)	0.08
Strain at Failure (%)	3.92
Sample Location	B9-90-1

Specimen

Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	1/25/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Greenish Gray Lean Clay w/ Sand

Test Variables	
Specific Gravity	2.73

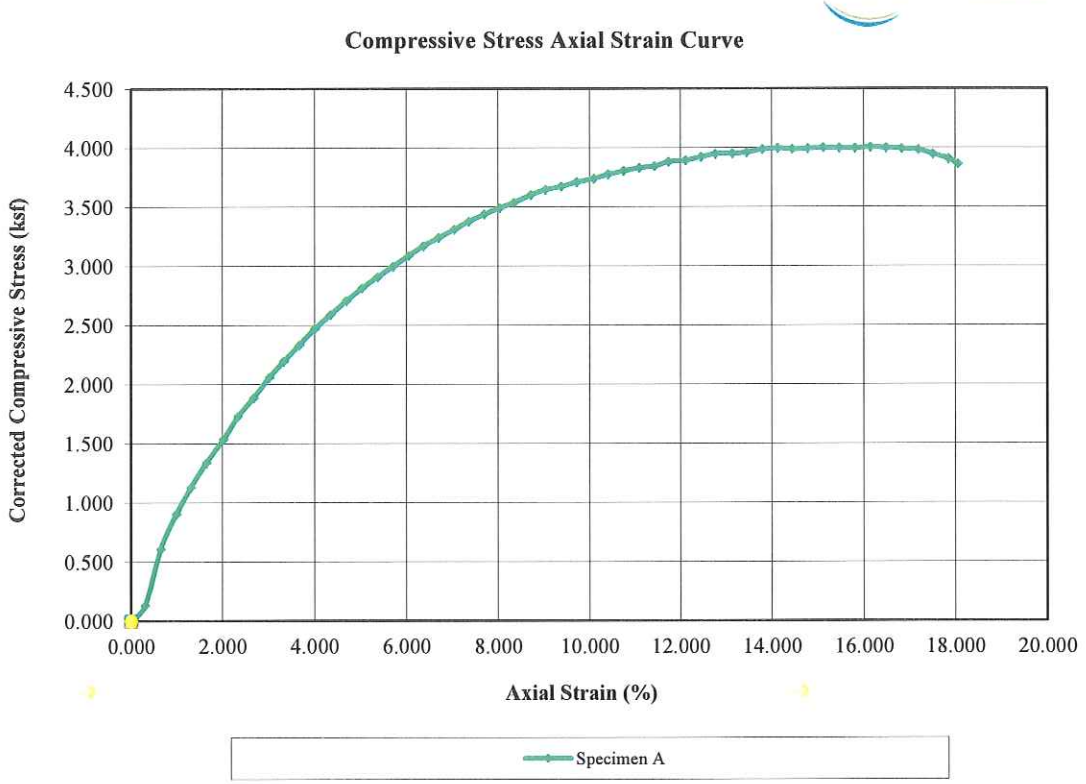
Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

KLEINFELDER

Tested By M. Baishiki Date 4/6/2011 | Computed By M. Baishiki Date 4/7/2011 | Checked By L.C.E Date ######



Before Test	Specimen A
Water Content (%)	24.87
Dry Density (pcf)	99.60
Saturation (%)	96.03
Void Ratio	0.70
Diameter (in)	1.92
Height (in)	3.99
Test Data	**A**
Unconfined Strength (ksf)	4.00
Undrained Shear Strength (tsf)	1.00
Undrained Shear Strength (ksf)	2.00
Rate of Strain (in/min)	0.08
Strain at Failure (%)	16.15
Sample Location	B10-25-1

Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	1/26/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Brownish Gray Lean Clay
Test Variables	
Specific Gravity	2.72

Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

Date: #####

Checked By: L.C.E.

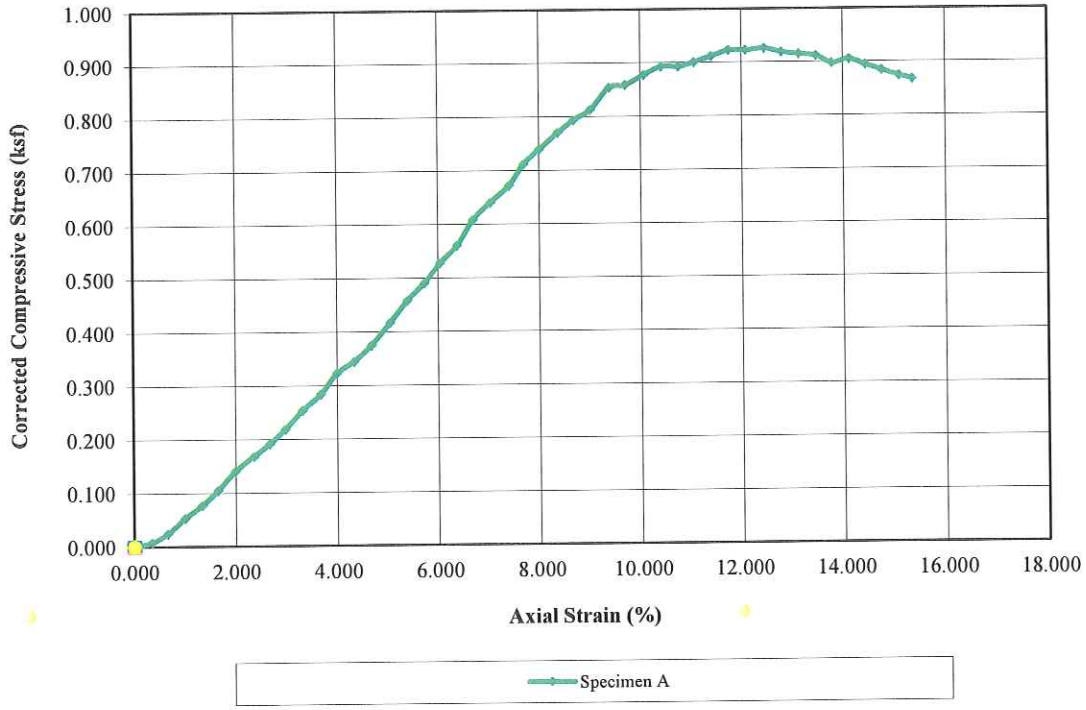
Date: 4/7/2011

Computed By: M. Baishiki

Date: 4/6/2011

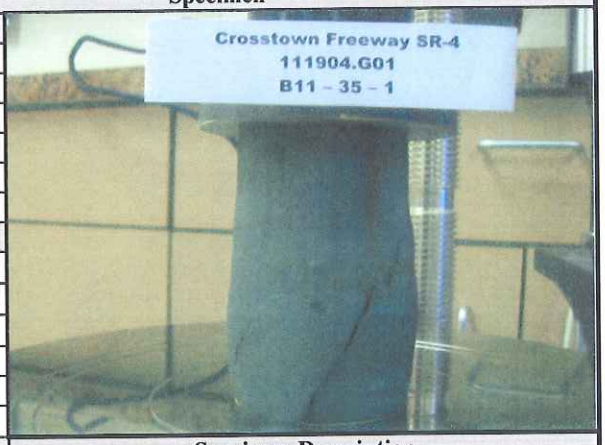
Tested By: M. Baishiki

Compressive Stress Axial Strain Curve



Before Test	A
Water Content (%)	29.89
Dry Density (pcf)	93.50
Saturation (%)	99.71
Void Ratio	0.82
Diameter (in)	1.92
Height (in)	4.00
Test Data	**A**
Unconfined Strength (ksf)	0.93
Undrained Shear Strength (tsf)	0.23
Undrained Shear Strength (ksf)	0.46
Rate of Strain (in/min)	0.08
Strain at Failure (%)	12.45
Sample Location	B11-35-1

Specimen



Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	2/4/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Brownish Gray Sandy Lean Clay

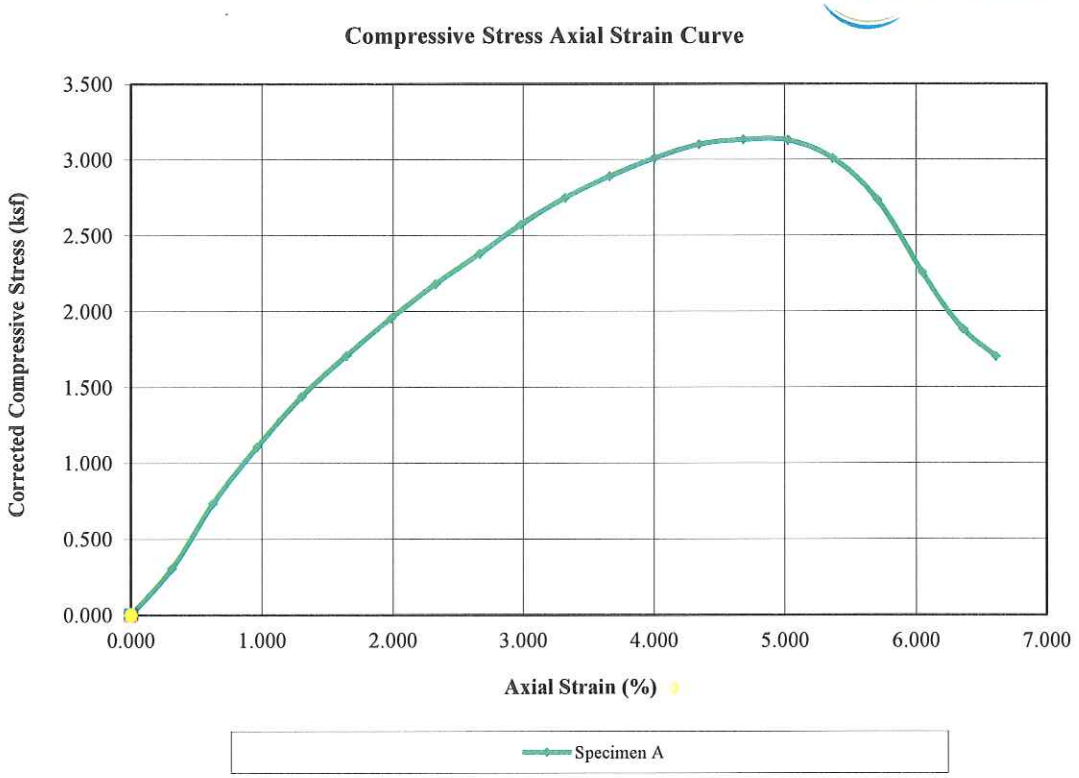
Test Variables	
Specific Gravity	2.72

Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

Tested By: M. Baishiki | Date: 4/6/2011
Computed By: M. Baishiki | Date: 4/7/2011
Checked By: L.C.E | Date: #####



Before Test	A
Water Content (%)	19.49
Dry Density (pcf)	108.70
Saturation (%)	94.38
Void Ratio	0.56
Diameter (in)	1.92
Height (in)	4.00
Test Data	**A**
Unconfined Strength (ksf)	3.13
Undrained Shear Strength (tsf)	0.78
Undrained Shear Strength (ksf)	1.57
Rate of Strain (in/min)	0.08
Strain at Failure (%)	4.68
Sample Location	B11-70-1

Specimen

Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	2/4/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Brownish Gray Lean Clay w/ Sand

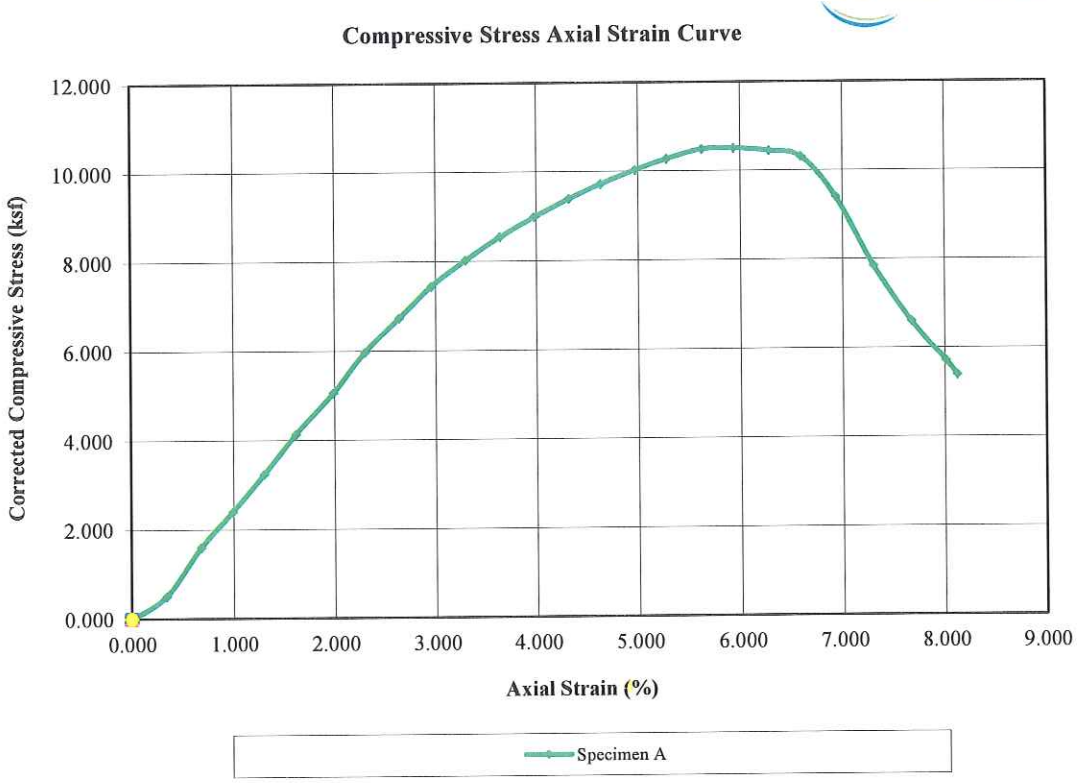
Test Variables	
Specific Gravity	2.72

Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

Tested By: M. Baishiki — Date: 3/28/2011
Computed By: M. Baishiki — Date: 3/29/2011
Checked By: — Date:



Before Test	Specimen A
Water Content (%)	23.27
Dry Density (pcf)	103.40
Saturation (%)	98.52
Void Ratio	0.64
Diameter (in)	1.92
Height (in)	3.99
Test Data	**A**
Unconfined Strength (ksf)	10.51
Undrained Shear Strength (tsf)	2.63
Undrained Shear Strength (ksf)	5.25
Rate of Strain (in/min)	0.08
Strain at Failure (%)	5.94
Sample Location	B11-120-1

Specimen

Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	2/7/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Bluish Gray Lean Clay

Test Variables	
Specific Gravity	2.72

Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

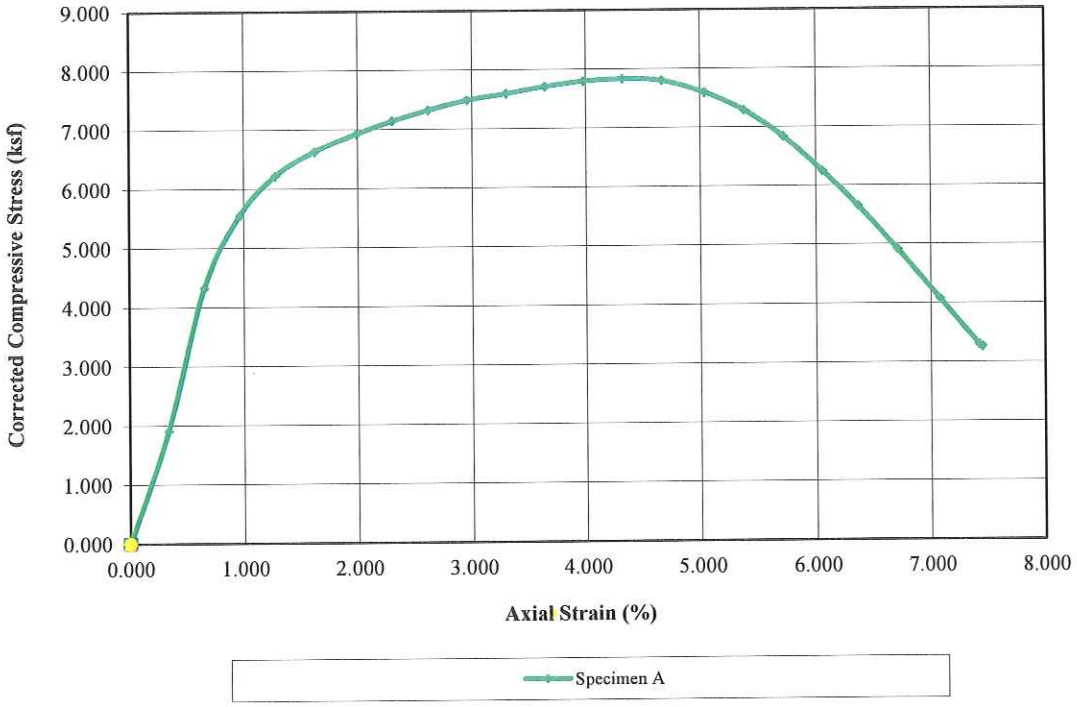
KLEINFELDER

Tested By: M. Baishiki Date: 4/7/2011

Computed By: M. Baishiki Date: 4/8/2011

Checked By: L.C.E Date: #####

Compressive Stress Axial Strain Curve



Before Test	Specimen A
Water Content (%)	24.72
Dry Density (pcf)	100.00
Saturation (%)	96.41
Void Ratio	0.70
Diameter (in)	1.92
Height (in)	3.99
Test Data	**A**
Unconfined Strength (ksf)	7.83
Undrained Shear Strength (tsf)	1.96
Undrained Shear Strength (ksf)	3.92
Rate of Strain (in/min)	0.08
Strain at Failure (%)	4.32
Sample Location	B11-130-1

Specimen

Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	2/7/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Brownish Gray Lean Clay

Test Variables	
Specific Gravity	2.72

Remarks

Kleinfelder

Unconfined Compression Test Report (ASTM D2166)

Date #####

Checked By L.C.E.

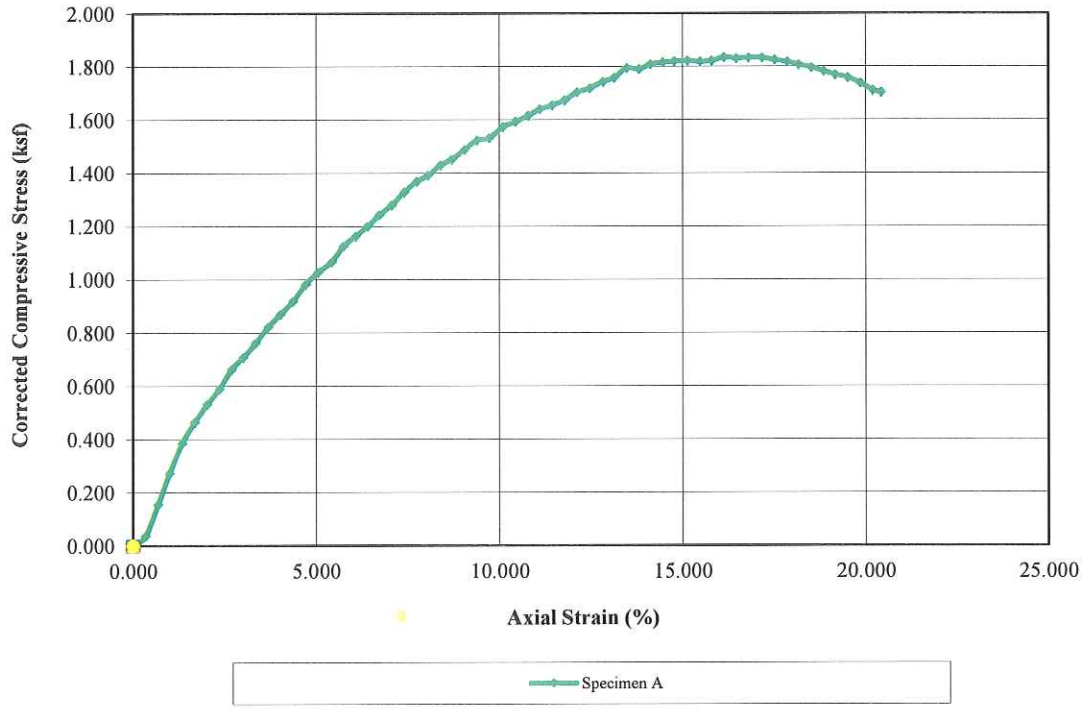
Date 4/8/2011

Computed By M. Baishiki

Date 4/7/2011

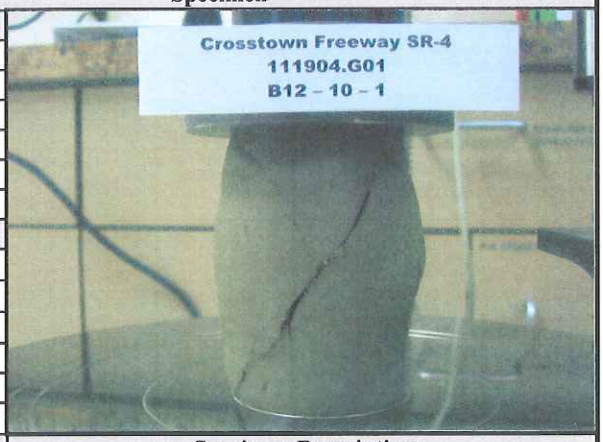
Tested By M. Baishiki

Compressive Stress Axial Strain Curve



Before Test	Specimen A
Water Content (%)	30.75
Dry Density (pcf)	92.40
Saturation (%)	99.95
Void Ratio	0.84
Diameter (in)	1.92
Height (in)	4.00
Test Data	**A**
Unconfined Strength (ksf)	1.84
Undrained Shear Strength (tsf)	0.46
Undrained Shear Strength (ksf)	0.92
Rate of Strain (in/min)	0.08
Strain at Failure (%)	16.14
Sample Location	B12-10-1

Specimen



Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	1/27/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Light Brown Lean Clay

Test Variables	
Specific Gravity	2.72

Remarks

Kleinfelder

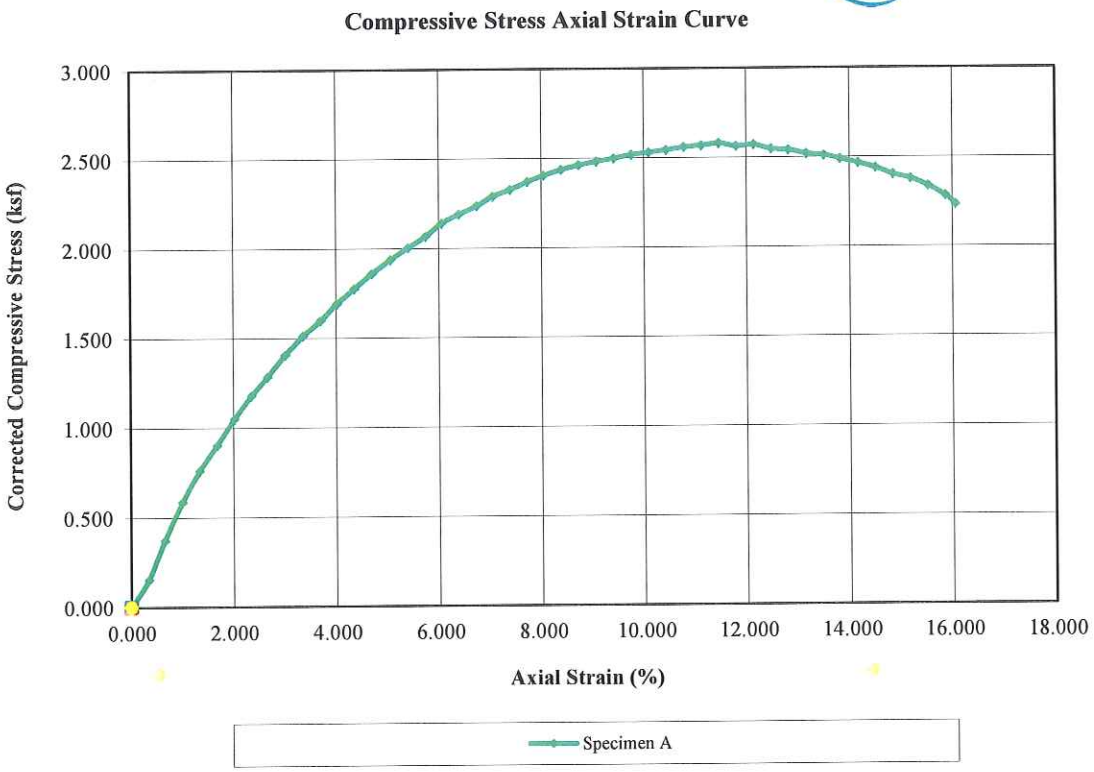
Unconfined Compression Test Report (ASTM D2166)

KLEINFELDER

Tested By: M. Baishiki — Date: 4/1/2011

Computed By: M. Baishiki — Date: 4/4/2011

Checked By: — Date:



Before Test	Specimen A
Water Content (%)	26.63
Dry Density (pcf)	98.45
Saturation (%)	99.95
Void Ratio	0.72
Diameter (in)	1.92
Height (in)	3.99
Test Data	**A**
Unconfined Strength (ksf)	2.58
Undrained Shear Strength (tsf)	0.65
Undrained Shear Strength (ksf)	1.29
Rate of Strain (in/min)	0.08
Strain at Failure (%)	11.46
Sample Location	B13-20-1

Specimen

Project Information	
Project Num	111904.G01
Project	Crosstown Freeway SR-4
Sampling Date	1/27/2011
Sample #	1103017
Client	

Specimen Description	
Specimen A	Brown Lean Clay

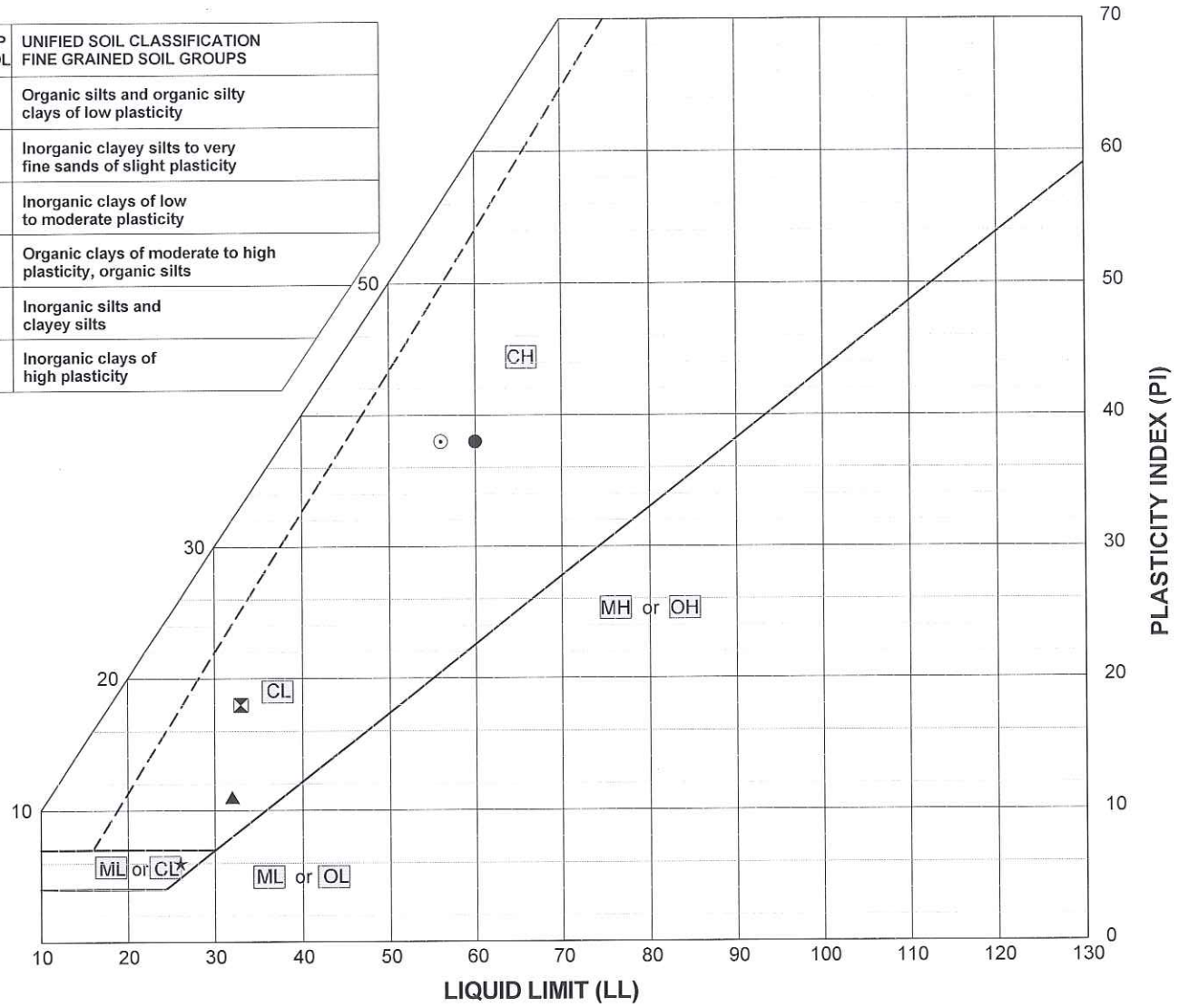
Test Variables	
Specific Gravity	2.72

Remarks

Plasticity Charts (Atterberg Limits)

GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE GRAINED SOIL GROUPS
OL	Organic silts and organic silty clays of low plasticity
ML	Inorganic clayey silts to very fine sands of slight plasticity
CL	Inorganic clays of low to moderate plasticity
OH	Organic clays of moderate to high plasticity, organic silts
MH	Inorganic silts and clayey silts
CH	Inorganic clays of high plasticity

EXPLANATION



LEGEND:	SOURCE	DEPTH	LL	PL	PI	DESCRIPTION
●	R-11-001	75.0	60	22	38	
⊠	R-11-002	15.0	33	15	18	
▲	R-11-002	70.0	32	21	11	
★	R-11-003	55.0	26	20	6	
⊙	R-11-003	110.0	56	18	38	

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PLASTICITY CHART
ROUTE 4 CROSSTOWN FREEWAY EXTENSION
SAN JOAQUIN COUNTY, CALIFORNIA

PLATE
1 of 5

Drafted By: G. GOMEZ
Date: 7/14/2011
Project No.: 111904
File Number: STO11G015

ALT ATTERBERG STO11G015.GPJ 7/14/11

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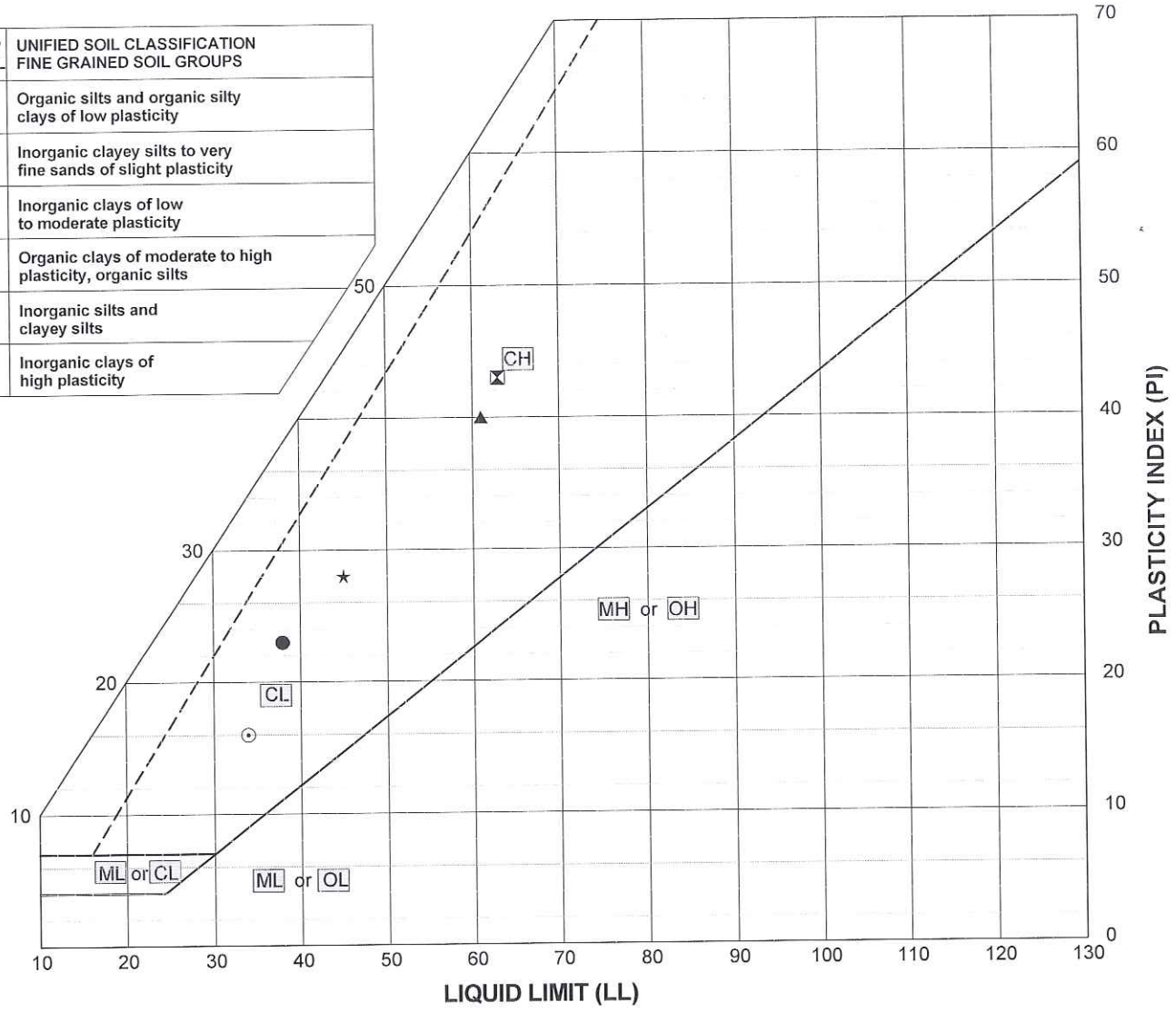
GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE GRAINED SOIL GROUPS
OL	Organic silts and organic silty clays of low plasticity
ML	Inorganic clayey silts to very fine sands of slight plasticity
CL	Inorganic clays of low to moderate plasticity
OH	Organic clays of moderate to high plasticity, organic silts
MH	Inorganic silts and clayey silts
CH	Inorganic clays of high plasticity

EXPLANATION



LEGEND:	SOURCE	DEPTH	LL	PL	PI	DESCRIPTION
●	R-11-004	30.0	38	15	23	
⊠	R-11-004	55.0	63	20	43	
▲	R-11-004	85.0	61	21	40	
★	R-11-005	40.0	45	17	28	
⊙	R-11-006	20.0	34	18	16	

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Drafted By: G. GOMEZ
Date: 7/14/2011

Project No.: 111904
File Number: STO11G015

PLASTICITY CHART
ROUTE 4 CROSSTOWN FREEWAY EXTENSION
SAN JOAQUIN COUNTY, CALIFORNIA

PLATE
2 of 5

ALT ATTERBERG STO11G015.GPJ 7/14/11

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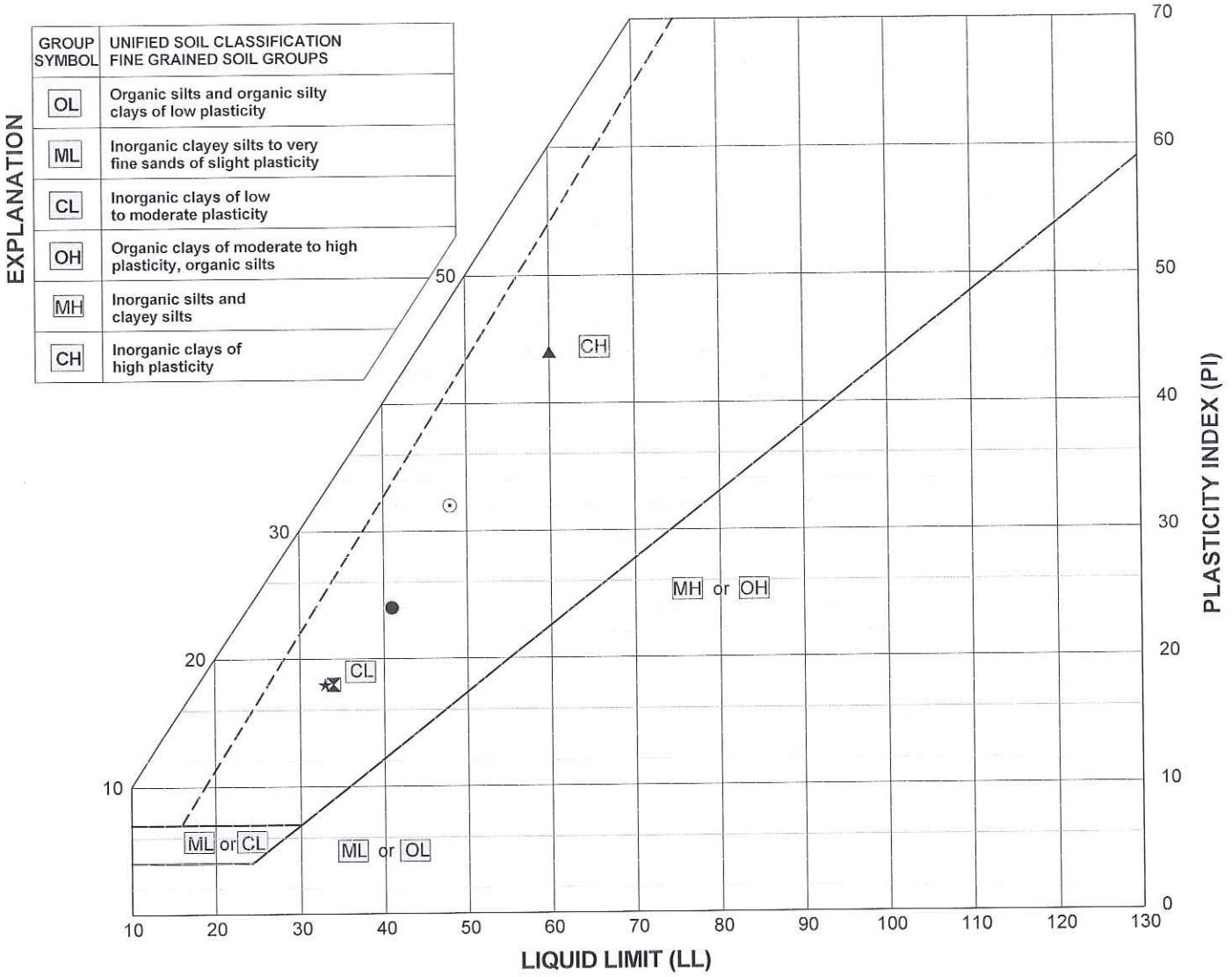

LEGEND:	SOURCE	DEPTH	LL	PL	PI	DESCRIPTION
●	R-11-006	70.0	41	17	24	
⊠	R-11-007	25.0	34	16	18	
▲	R-11-007	55.0	60	16	44	
★	R-11-008	25.0	33	15	18	
⊙	R-11-009	20.0	48	16	32	

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Drafted By: G. GOMEZ
Date: 7/14/2011

Project No.: 111904
File Number: STO11G015

PLASTICITY CHART
ROUTE 4 CROSSTOWN FREEWAY EXTENSION
SAN JOAQUIN COUNTY, CALIFORNIA

PLATE
3 of 5

ALT ATTERBERG STO11G015.GPJ 7/14/11

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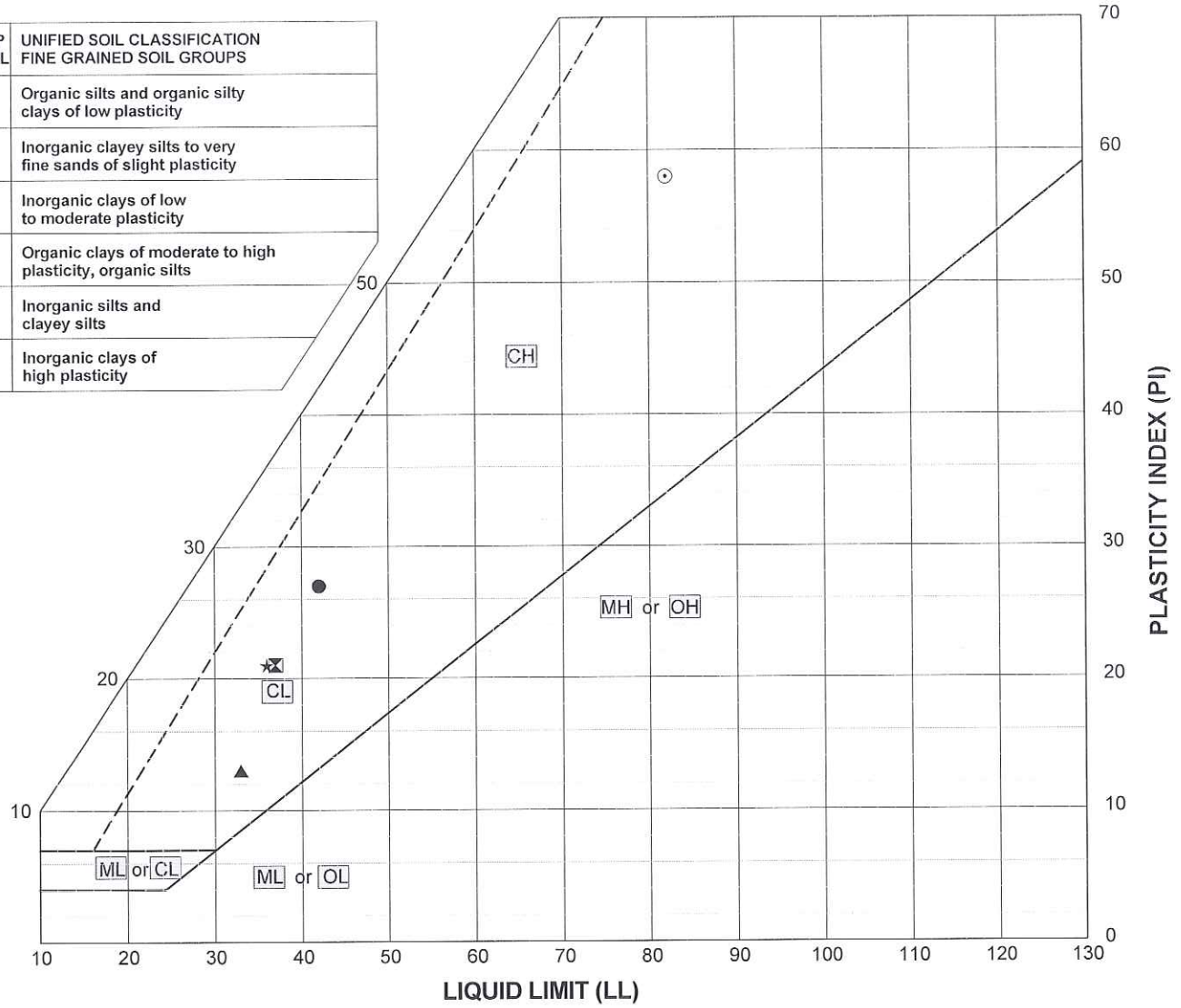
GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE GRAINED SOIL GROUPS
OL	Organic silts and organic silty clays of low plasticity
ML	Inorganic clayey silts to very fine sands of slight plasticity
CL	Inorganic clays of low to moderate plasticity
OH	Organic clays of moderate to high plasticity, organic silts
MH	Inorganic silts and clayey silts
CH	Inorganic clays of high plasticity

EXPLANATION



LEGEND:	SOURCE	DEPTH	LL	PL	PI	DESCRIPTION
●	R-11-009	80.0	42	15	27	
⊠	R-11-010	35.0	37	16	21	
▲	R-11-011	35.0	33	20	13	
★	R-11-011	45.0	36	15	21	
⊙	R-11-011	90.0	82	24	58	

KLEINFELDER
Bright People. Right Solutions.

Drafted By: G. GOMEZ
Date: 7/14/2011

Project No.: 111904
File Number: STO11G015

PLASTICITY CHART
ROUTE 4 CROSSTOWN FREEWAY EXTENSION
SAN JOAQUIN COUNTY, CALIFORNIA

PLATE
4 of 5

ALT ATTERBERG STO11G015.GPJ 7/14/11

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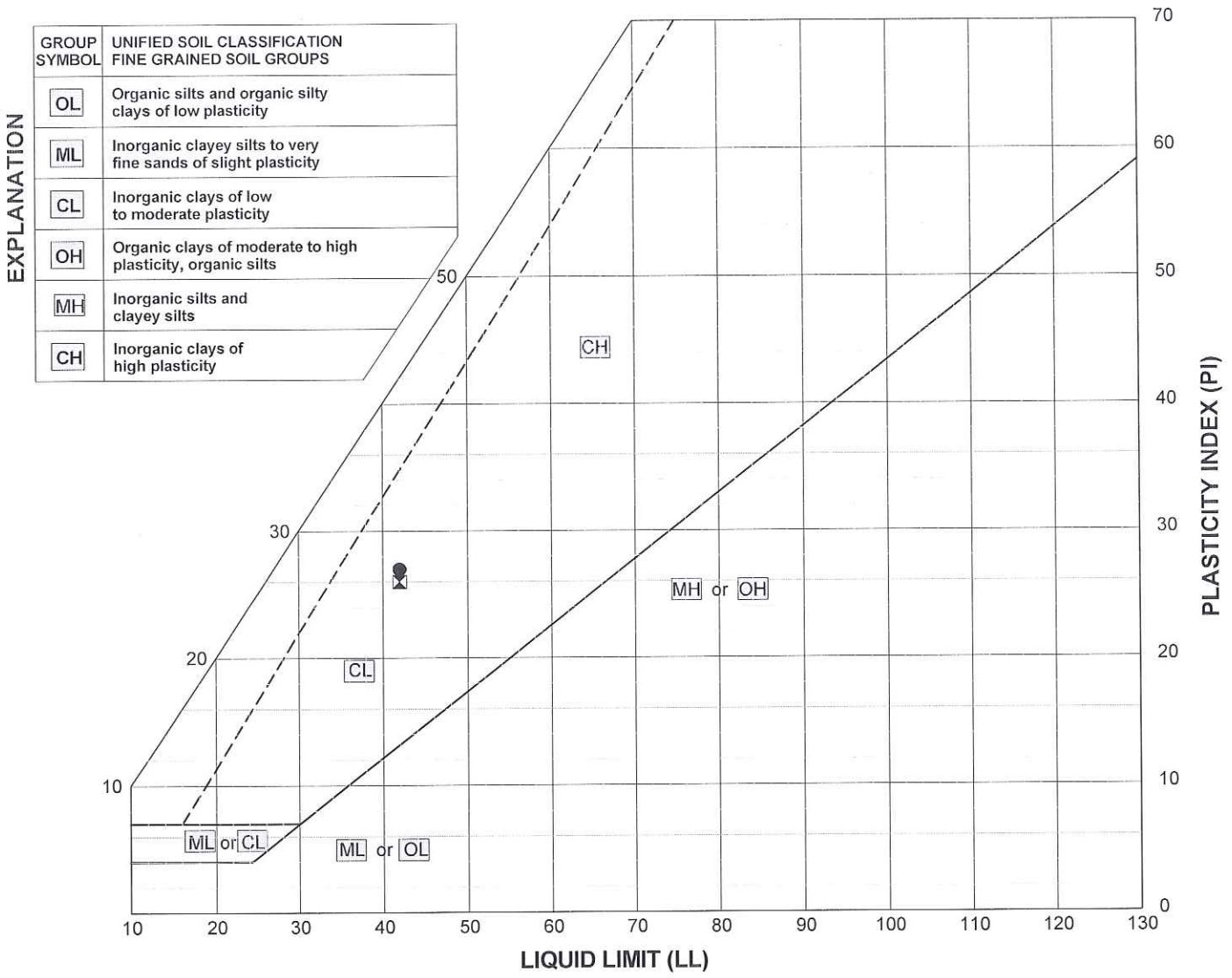

LEGEND:	SOURCE	DEPTH	LL	PL	PI	DESCRIPTION
●	R-11-012	15.0	42	15	27	
⊠	R-11-013	25.0	42	16	26	

ALT ATTERBERG STO11G015.GPJ 7/14/11

KLEINFELDER
Bright People. Right Solutions.

Drafted By: G. GOMEZ
Date: 7/14/2011

Project No.: 111904
File Number: STO11G015

PLASTICITY CHART
ROUTE 4 CROSSTOWN FREEWAY EXTENSION
SAN JOAQUIN COUNTY, CALIFORNIA

PLATE
5 of 5

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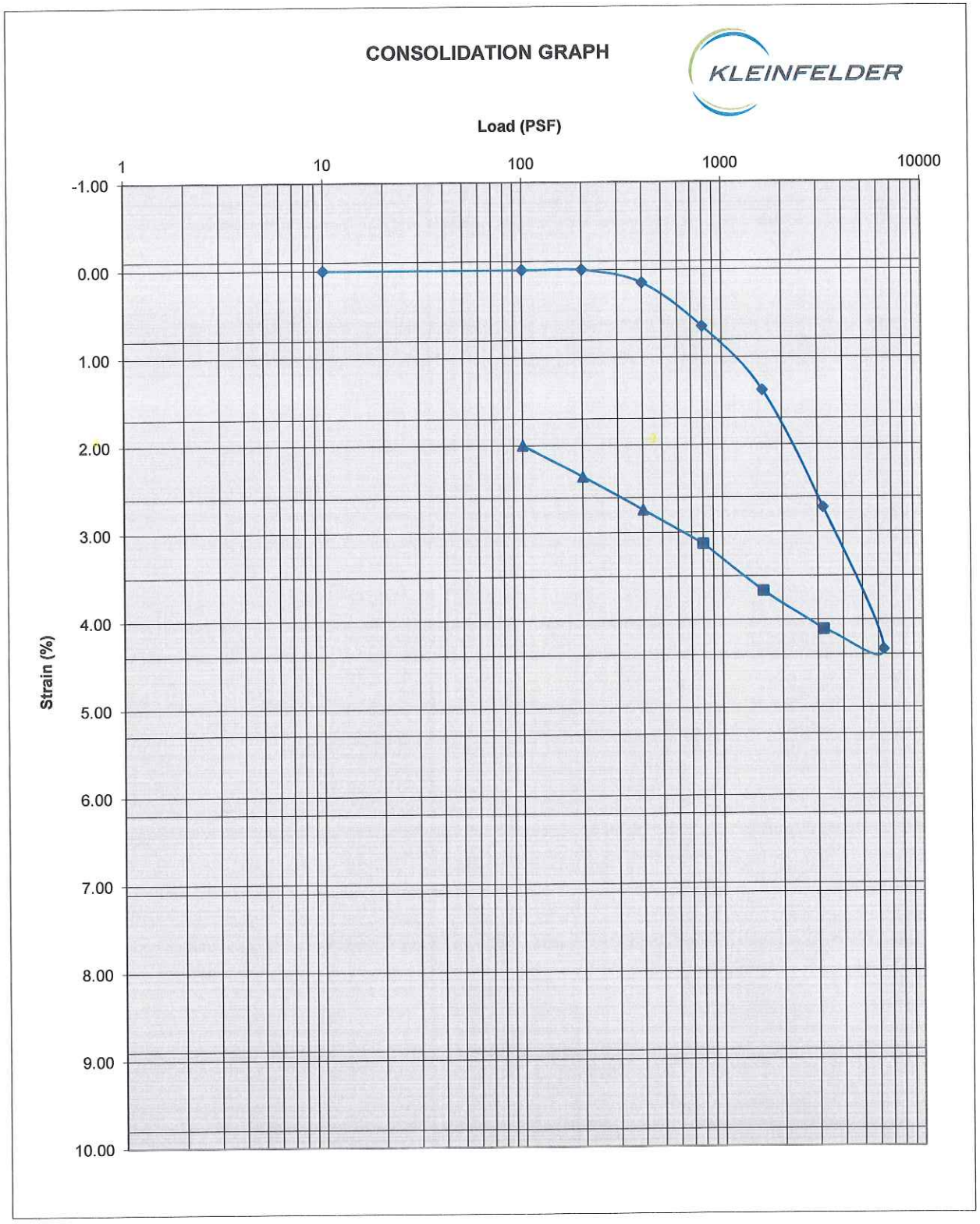
Consolidation Test Results

Laboratory Test Data

Project Name: ***Crosstown Freeway SR-4***
Project No: **111904.G01**
Report Date: **4/15/2011**
Boring No: **B12-20-1**
Sample Description: **Greenish Gray Lean Clay**

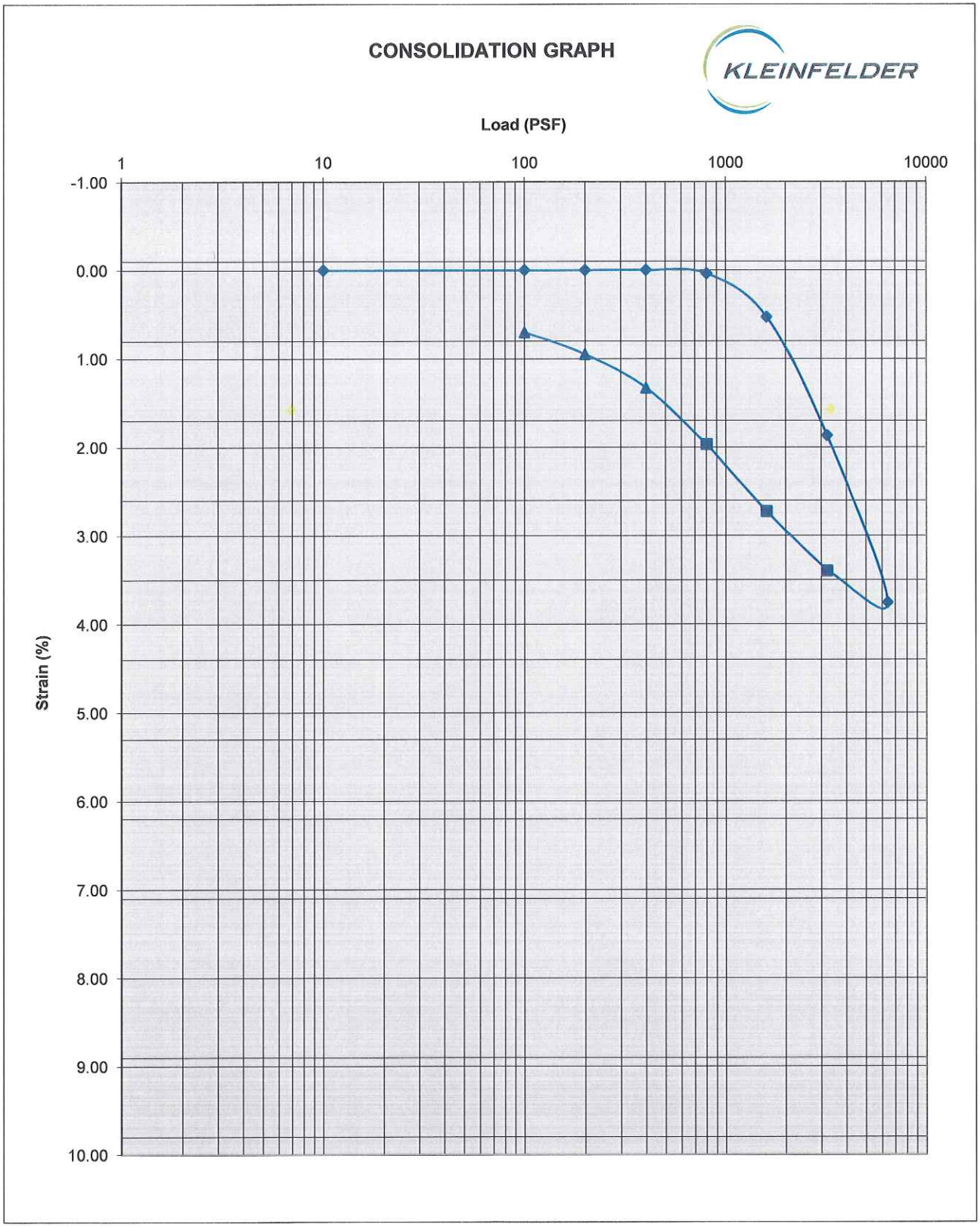
One-Dimensional Consolidation of Soil (ASTM D 2435)



Laboratory Test Data

Project Name: ***Crosstown Freeway SR-4***
Project No: **111904.G01**
Report Date: **4/19/2011**
Boring No: **B13-5-1**
Sample Description: **Greenish Gray Lean Clay**

One-Dimensional Consolidation of Soil (ASTM D 2435)



R-Value Test Results

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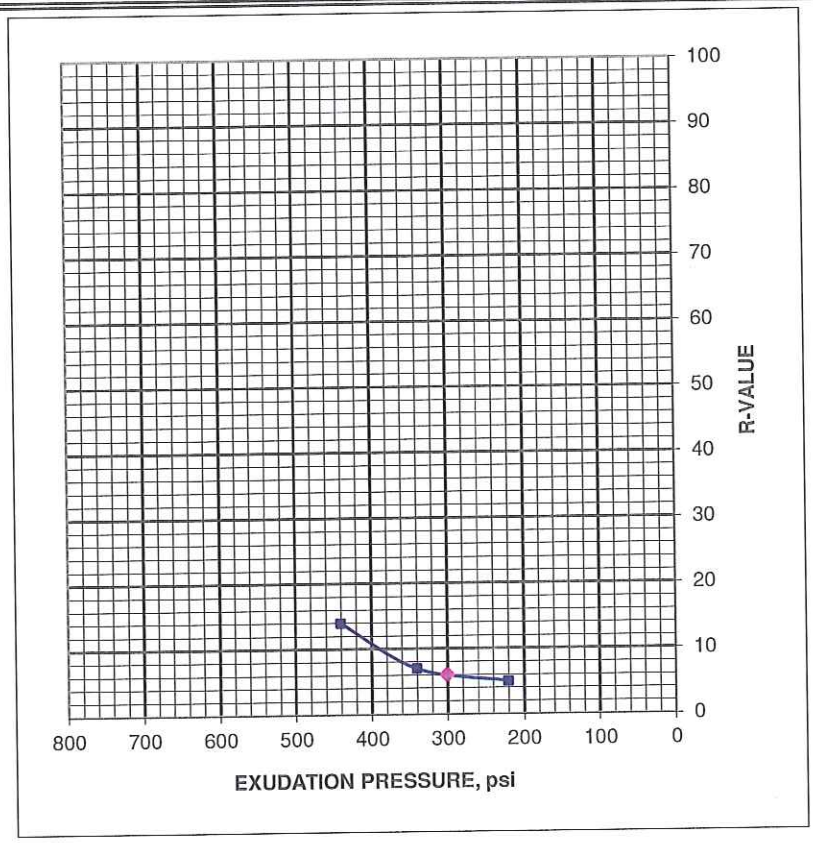
Project Name: Hwy 4 Extension
Project Number: 111904.G01
Date Sampled: February 4, 2011
Laboratory Test Number: 1102008
Sample Description: Brown Clay w/ Sand
Sample Location: 2345 Navy Dr., South Side

CTM 301, Resistance "R" Value of Treated and Untreated Bases



LABORATORY DATA

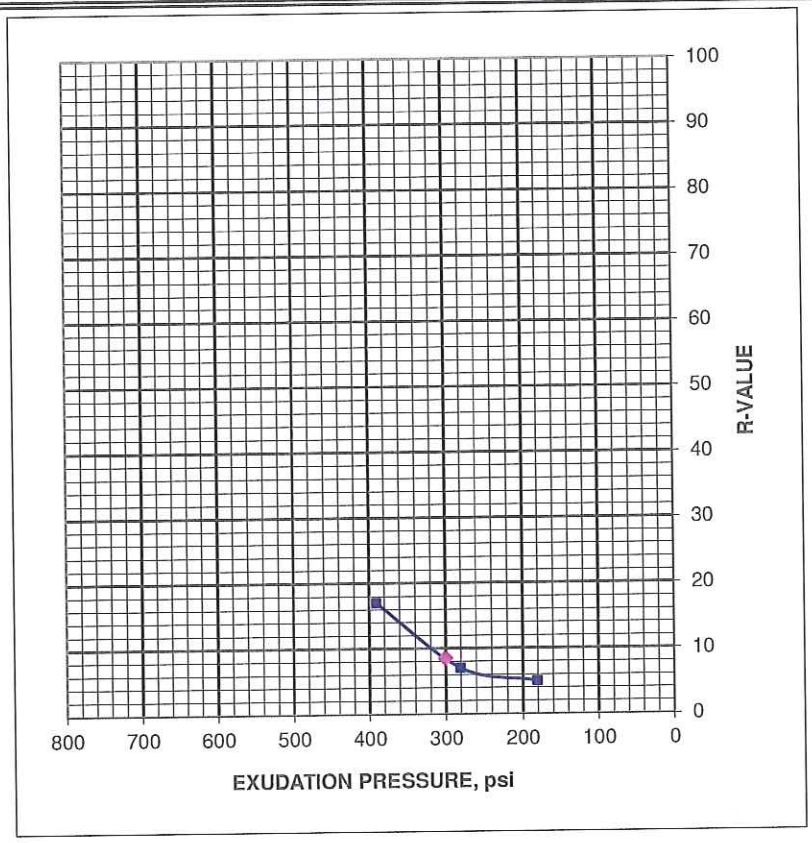
Specimen	A	B	C
Exudation Pressure, psi	220	340	440
Expansion Pressure, psf	35	143	320
Resistance Value, R	5	7	14
Moisture at Test, %	46.6	45.0	42.5
Dry Density at Test, pcf	82.7	85.4	88.4
R-Value at 300 psi Exudation Pressure			**6**

Reviewed By: _____
L. Christine Erickson
Lab Manager

KLEINFELDER
Bright People. Right Solutions.

Project Name: Hwy 4 Extension
Project Number: 111904.G01
Date Sampled: February 4, 2011
Laboratory Test Number: 1102009
Sample Description: Brown Clay w/ Sand
Sample Location: Navy Drive & Tellie Lewis

CTM 301, Resistance "R" Value of Treated and Untreated Bases



LABORATORY DATA

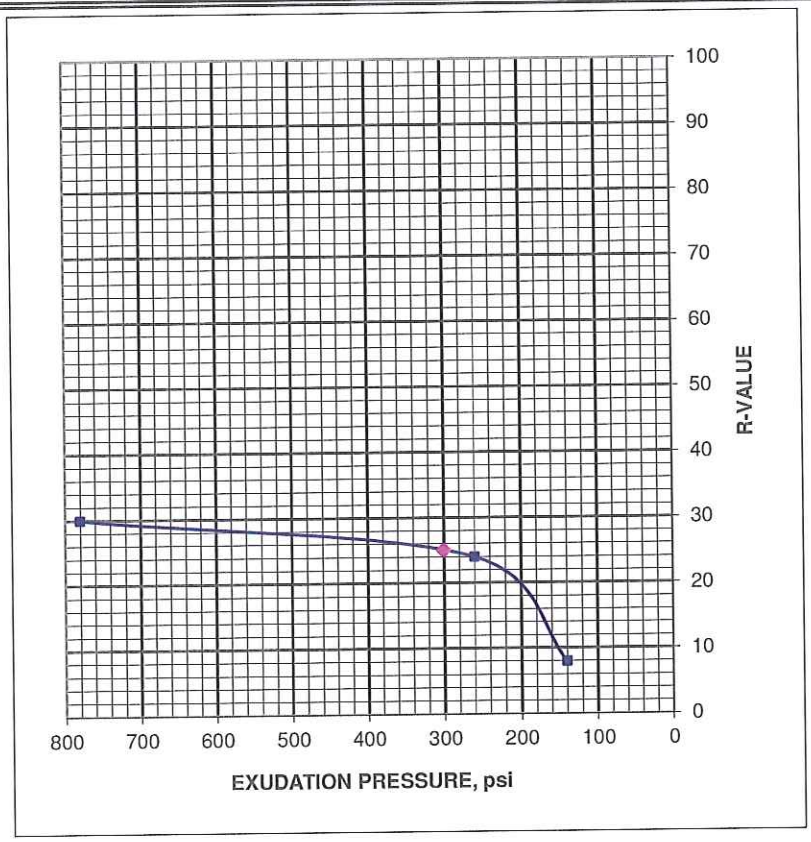
Specimen	A	B	C
Exudation Pressure, psi	180	280	390
Expansion Pressure, psf	0	61	320
Resistance Value, R	5	7	17
Moisture at Test, %	41.2	38.7	36.2
Dry Density at Test, pcf	88.0	91.7	95.2
R-Value at 300 psi Exudation Pressure			**9**

Reviewed By: _____
L. Christine Erickson
Lab Manager

KLEINFELDER
Bright People. Right Solutions.

Project Name: Hwy 4 Extension
Project Number: 111904.G01
Date Sampled: February 4, 2011
Laboratory Test Number: 1102007
Sample Description: Brown Clayey Sand
Sample Location: Scotts Ave.; NW Side

CTM 301, Resistance "R" Value of Treated and Untreated Bases



LABORATORY DATA

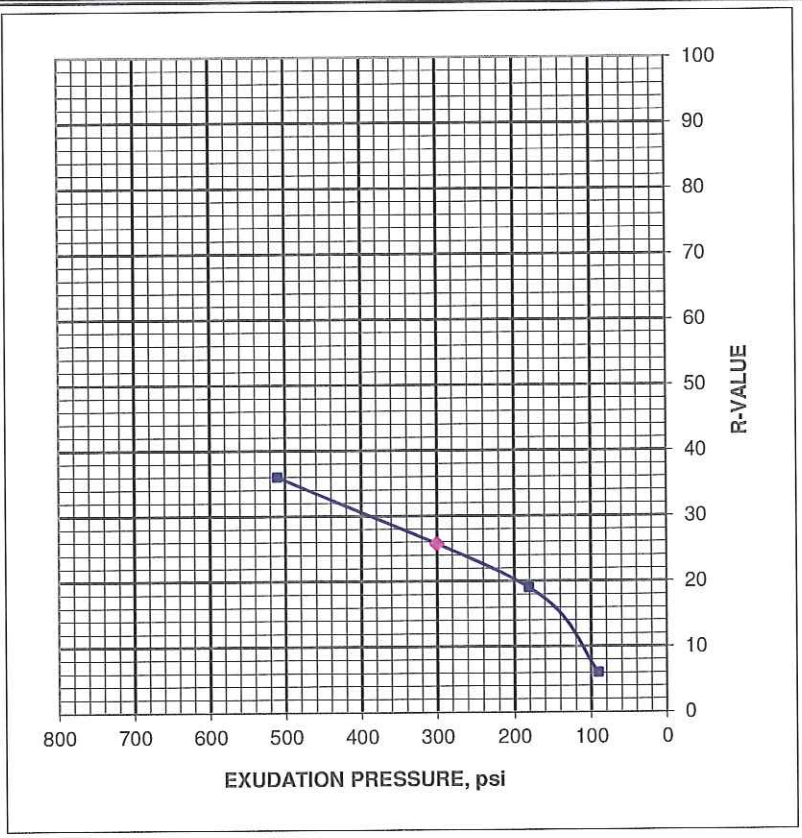
Specimen	A	B	C
Exudation Pressure, psi	140	260	780
Expansion Pressure, psf	4	69	108
Resistance Value, R	8	24	30
Moisture at Test, %	65.9	58.8	54.1
Dry Density at Test, pcf	73.8	77.6	82.4
R-Value at 300 psi Exudation Pressure			**25**

Reviewed By: _____
L. Christine Erickson
Lab Manager

KLEINFELDER
Bright People. Right Solutions.

Project Name: Hwy 4 Extension
Project Number: 111904.G01
Date Sampled: February 4, 2011
Laboratory Test Number: 1102005
Sample Description: Brown Clayey Sand
Sample Location: Hazelton Avenue

CTM 301, Resistance "R" Value of Treated and Untreated Bases



LABORATORY DATA

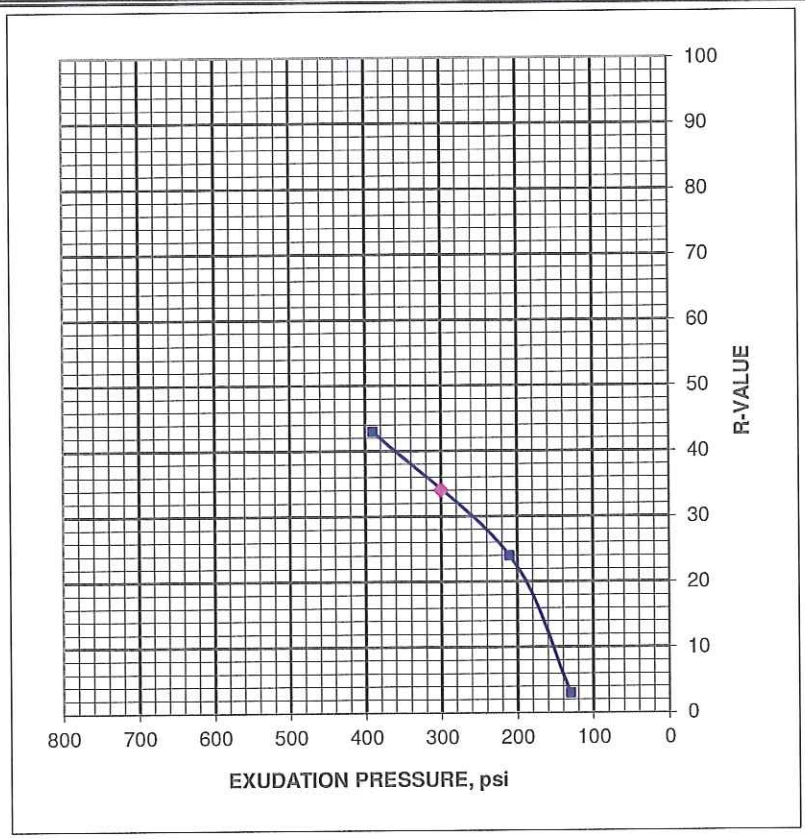
Specimen	A	B	C
Exudation Pressure, psi	90	180	510
Expansion Pressure, psf	0	39	217
Resistance Value, R	6	19	36
Moisture at Test, %	70.4	62.6	57.0
Dry Density at Test, pcf	71.6	75.5	80.0
R-Value at 300 psi Exudation Pressure			**26**

Reviewed By: _____
L. Christine Erickson
Lab Manager

KLEINFELDER
Bright People. Right Solutions.

Project Name: Hwy 4 Extension
Project Number: 111904.G01
Date Sampled: February 4, 2011
Laboratory Test Number: 1102006
Sample Description: Brown Silty Sand
Sample Location: Del Norte

CTM 301, Resistance "R" Value of Treated and Untreated Bases



LABORATORY DATA

Specimen	A	B	C
Exudation Pressure, psi	130	210	390
Expansion Pressure, psf	0	0	39
Resistance Value, R	3	24	43
Moisture at Test, %	79.8	73.3	67.5
Dry Density at Test, pcf	67.7	70.0	74.2
R-Value at 300 psi Exudation Pressure			**34**

Reviewed By: _____
L. Christine Erickson
Lab Manager

KLEINFELDER INC., 2001 Arch-Airport Rd. Suite 100, Stockton, CA 95206 (209) 948-1345 Fax (209) 234-4700

Sulfate and Chloride Contents by California Laboratory Service

CALIFORNIA LABORATORY SERVICES

3249 Fitzgerald Road Rancho Cordova, CA 95742

March 25, 2011

CLS Work Order #: CUC0760
COC #: 19319

Mackenzie Baishiki
Kleinfelder (Stockton)
2001 Arch-Airport Road, suite 100
Stockton, CA 95206

Project Name: Crosstown Freeway SR-4

Enclosed are the results of analyses for samples received by the laboratory on 03/18/11 14:10. Samples were analyzed pursuant to client request utilizing EPA or other ELAP approved methodologies. I certify that the results are in compliance both technically and for completeness.

Analytical results are attached to this letter. Please call if we can provide additional assistance.

Sincerely,

James Liang, Ph.D.
Laboratory Director

CA DOHS ELAP Accreditation/Registration number 1233

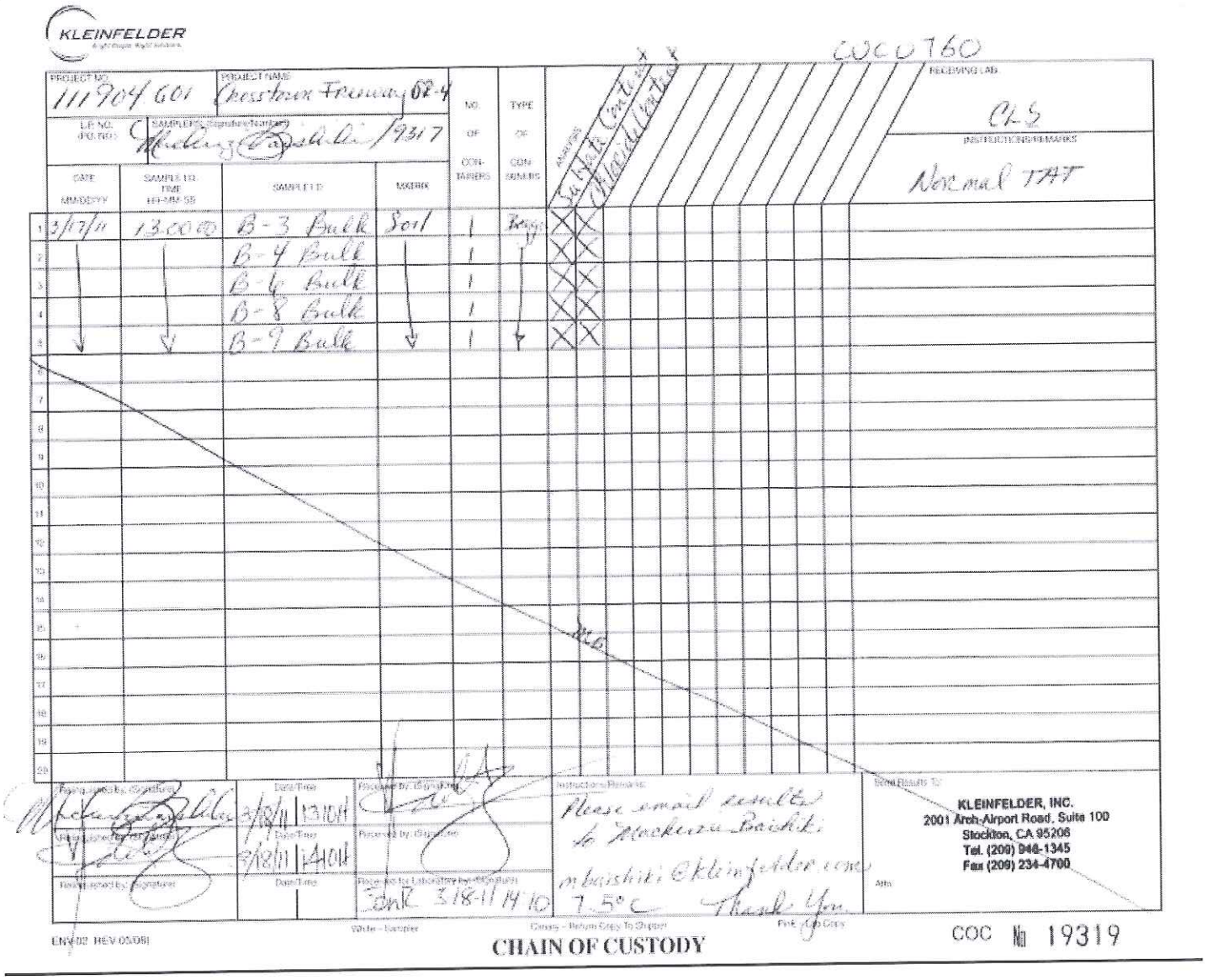
CALIFORNIA LABORATORY SERVICES

03/25/11 09:36

Kleinfelder (Stockton)
2001 Arch-Airport Road, suite 100
Stockton, CA 95206

Project: Crosstown Freeway SR-4
Project Number: 111904.G01
Project Manager: Mackenzie Baishiki

CLS Work Order #: CUC0760
COC #: 19319



Kleinfelder (Stockton)
2001 Arch-Airport Road, suite 100
Stockton, CA 95206

Project: Crosstown Freeway SR-4
Project Number: 111904.G01
Project Manager: Mackenzie Baishiki

CLS Work Order #: CUC0760
COC #: 19319

Conventional Chemistry Parameters by APHA/EPA Methods

Analyte	Result	Reporting Limit	Units	Dilution	Batch	Prepared	Analyzed	Method	Notes
B-3 Bulk (CUC0760-01) Soil	**Sampled: 03/17/11 13:00**	**Received: 03/18/11 14:10**							
Chloride	**230**	25	mg/kg	5	CU01902	03/21/11	03/23/11	EPA 300.0	
Sulfate as SO4	**460**	25	"	"	"	"	"	"	
B-4 Bulk (CUC0760-02) Soil	**Sampled: 03/17/11 13:00**	**Received: 03/18/11 14:10**							
Chloride	**340**	5.0	mg/kg	1	CU01902	03/21/11	03/21/11	EPA 300.0	
Sulfate as SO4	**240**	5.0	"	"	"	"	"	"	
B-6 Bulk (CUC0760-03) Soil	**Sampled: 03/17/11 13:00**	**Received: 03/18/11 14:10**							
Chloride	**140**	5.0	mg/kg	1	CU01902	03/21/11	03/21/11	EPA 300.0	
Sulfate as SO4	**400**	5.0	"	"	"	"	"	"	
B-8 Bulk (CUC0760-04) Soil	**Sampled: 03/17/11 13:00**	**Received: 03/18/11 14:10**							
Chloride	**17**	5.0	mg/kg	1	CU01902	03/21/11	03/21/11	EPA 300.0	
Sulfate as SO4	**89**	5.0	"	"	"	"	"	"	
B-9 Bulk (CUC0760-05) Soil	**Sampled: 03/17/11 13:00**	**Received: 03/18/11 14:10**							
Chloride	**18**	5.0	mg/kg	1	CU01902	03/21/11	03/21/11	EPA 300.0	
Sulfate as SO4	**59**	5.0	"	"	"	"	"	"	

Kleinfelder (Stockton)
2001 Arch-Airport Road, suite 100
Stockton, CA 95206

Project: Crosstown Freeway SR-4
Project Number: 111904.G01
Project Manager: Mackenzie Baishiki

CLS Work Order #: CUC0760
COC #: 19319

Conventional Chemistry Parameters by APHA/EPA Methods - Quality Control

Analyte	Result	Reporting Limit	Units	Spike Level	Source Result	%REC	%REC Limits	RPD	RPD Limit	Notes
Batch CU01902 - General Prep										
Blank (CU01902-BLK1)				Prepared & Analyzed: 03/21/11						
Sulfate as SO4	ND	5.0	mg/kg							
Chloride	ND	5.0	"							
LCS (CU01902-BS1)				Prepared & Analyzed: 03/21/11						
Sulfate as SO4	51.9	5.0	mg/kg	50.0		104	75-125			
Chloride	20.3	5.0	"	20.0		101	75-125			
LCS Dup (CU01902-BSD1)				Prepared & Analyzed: 03/21/11						
Sulfate as SO4	52.6	5.0	mg/kg	50.0		105	75-125	1	25	
Chloride	20.7	5.0	"	20.0		104	75-125	2	25	
Matrix Spike (CU01902-MS1)	**Source: CUC0760-04**			Prepared & Analyzed: 03/21/11						
Sulfate as SO4	133	5.0	mg/kg	50.0	88.7	89	75-125			
Chloride	32.8	5.0	"	20.0	17.0	79	75-125			
Matrix Spike Dup (CU01902-MSD1)	**Source: CUC0760-04**			Prepared & Analyzed: 03/21/11						
Sulfate as SO4	137	5.0	mg/kg	50.0	88.7	96	75-125	3	30	
Chloride	35.7	5.0	"	20.0	17.0	94	75-125	9	30	

CALIFORNIA LABORATORY SERVICES

03/25/11 09:36

Kleinfelder (Stockton)	Project: Crosstown Freeway SR-4	
2001 Arch-Airport Road, suite 100	Project Number: 111904.G01	**CLS Work Order #: CUC0760**
Stockton, CA 95206	Project Manager: Mackenzie Baishiki	COC #: 19319

Notes and Definitions

DET	Analyte DETECTED
ND	Analyte NOT DETECTED at or above the reporting limit
NR	Not Reported
dry	Sample results reported on a dry weight basis
RPD	Relative Percent Difference

APPENDIX E
RESULTS OF LABORATORY TESTS
(PARIKH CONSULTANT, INC. INVESTIGATION, MARCH 2010)

LIST OF ATTACHMENTS

The following are attached and complete this appendix.

General Notes
Plasticity Charts (Atterberg Limits)
Grain Size Distribution Curves
Consolidation Test Results

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General Notes

APPENDIX B

LABORATORY TESTS

Classification Tests
The field classification of the samples was visually verified in the laboratory according to the Unified Soil Classification System and is presented in Appendix A.

Moisture-Density
The natural moisture contents and in-situ dry density were determined for selected samples. The results are presented in Appendix A together with material descriptions.

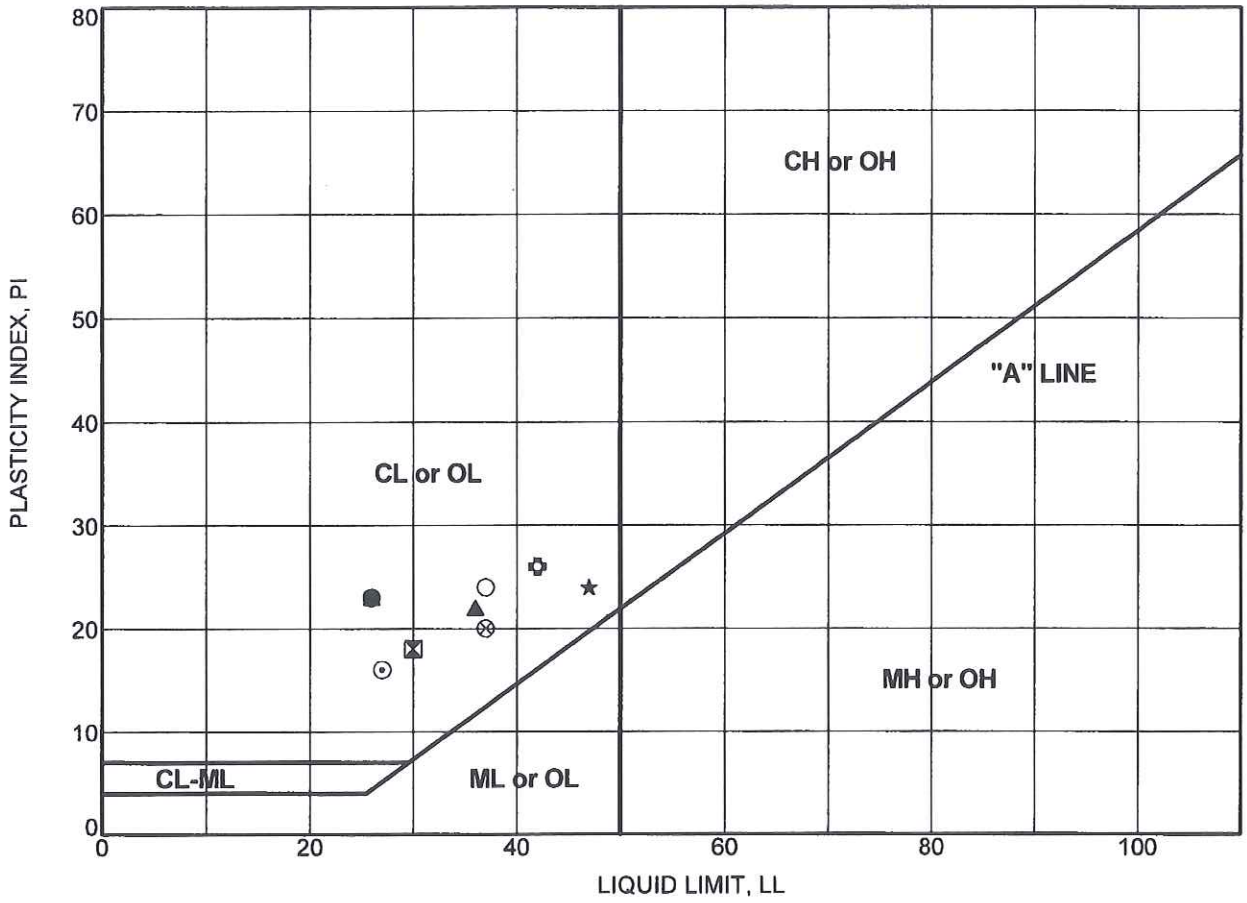
Unconfined Compression Tests
Strength tests were performed on selected undisturbed sample using unconfined compression machine. Unconfined compression test was performed in general accordance with ASTM Test Method D2166. The results are presented on the "Log of Test Borings."

Atterberg Limits
The Atterberg Limits were determined for selected samples of the fine-grained materials. These results were used to classify the soils, as well as to obtain an indication of the expansion potential with variations in moisture content. The Atterberg Limits were determined in general accordance with ASTM Test Method D 4318-93. The results of these tests are presented on Plate B-2, "Plasticity Chart".

Grain Size Classification
Grain size classification tests (ASTM Test Method D 422-63) were performed on selected samples of granular soil to aid in the classification. The results are presented on Plate B-3A to B-3C, "Grain Size Distribution Curves".

Consolidation Tests
Consolidation tests were performed on selected samples in accordance with ASTM Test Method D 2435. The test resulrs are presented on the Plates B-4A to B-4I.

Plasticity Charts (Atterberg Limits)



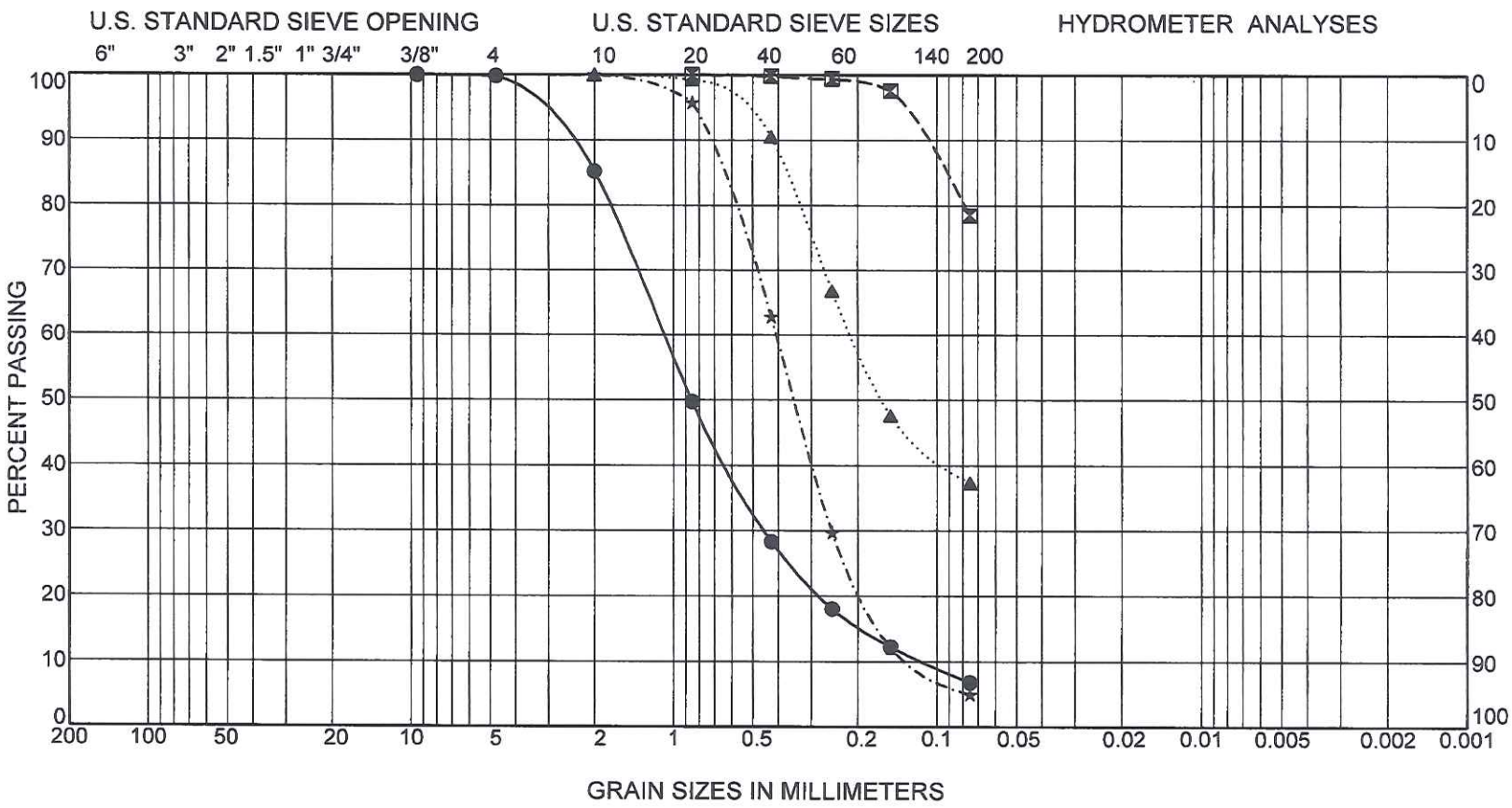
PLASTICITY CHART

Boring Number	Sample Number	Depth (feet)	Test Symbol	Moisture Content (%)	LL	PL	PI	Description
A-09-001	MC-3	16.0	●	25	26	3	23	SILT (ML)
A-09-001	MC-4	21.0	⊠	24	30	12	18	LEAN CLAY (CL)
A-09-001	MC-9	46.0	▲	30	36	14	22	LEAN CLAY (CL)
A-09-001	MC-11	56.0	★	33	47	23	24	LEAN CLAY (CL)
A-09-002	MC-3	16.0	⊙	24	27	11	16	LEAN CLAY (CL)
A-09-002	MC-7	36.0	✿	29	42	16	26	SILT WITH SAND (SM)
A-09-003	MC-1	5.0	○	26	37	13	24	LEAN CLAY (CL)
A-09-003	MC-3	16.0	△	24	26	3	23	SILT (ML)
A-09-003	MC-6	31.0	⊗	26	37	17	20	LEAN CLAY (CL)

Grain Size Distribution Curves

GRAIN SIZE DISTRIBUTION CURVES

COBBLES	GRAVEL		SAND			SILT AND CLAY
	coarse	fine	coarse	medium	fine	



Boring Number	Sample Number	Depth (feet)	Symbol	LL	PI	Description
A-09-001	SPT-6	31.0	●			WELL GRADED SAND WITH SILT (SW-SM)
A-09-001	MC-8	41.0	⊠			LEAN CLAY WITH SAND (CL)
A-09-001	MC-17	101.0	▲			SILTY SAND (SM)
A-09-002	MC-5	26.0	★			POORLY GRADED SAND WITH SILT (SP-SM)

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

RTE 4 CROSSTOWN CONNECTOR
STOCKTON, CALIFORNIA

JOB NO: 207145.10

PLATE NO: B-3A

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

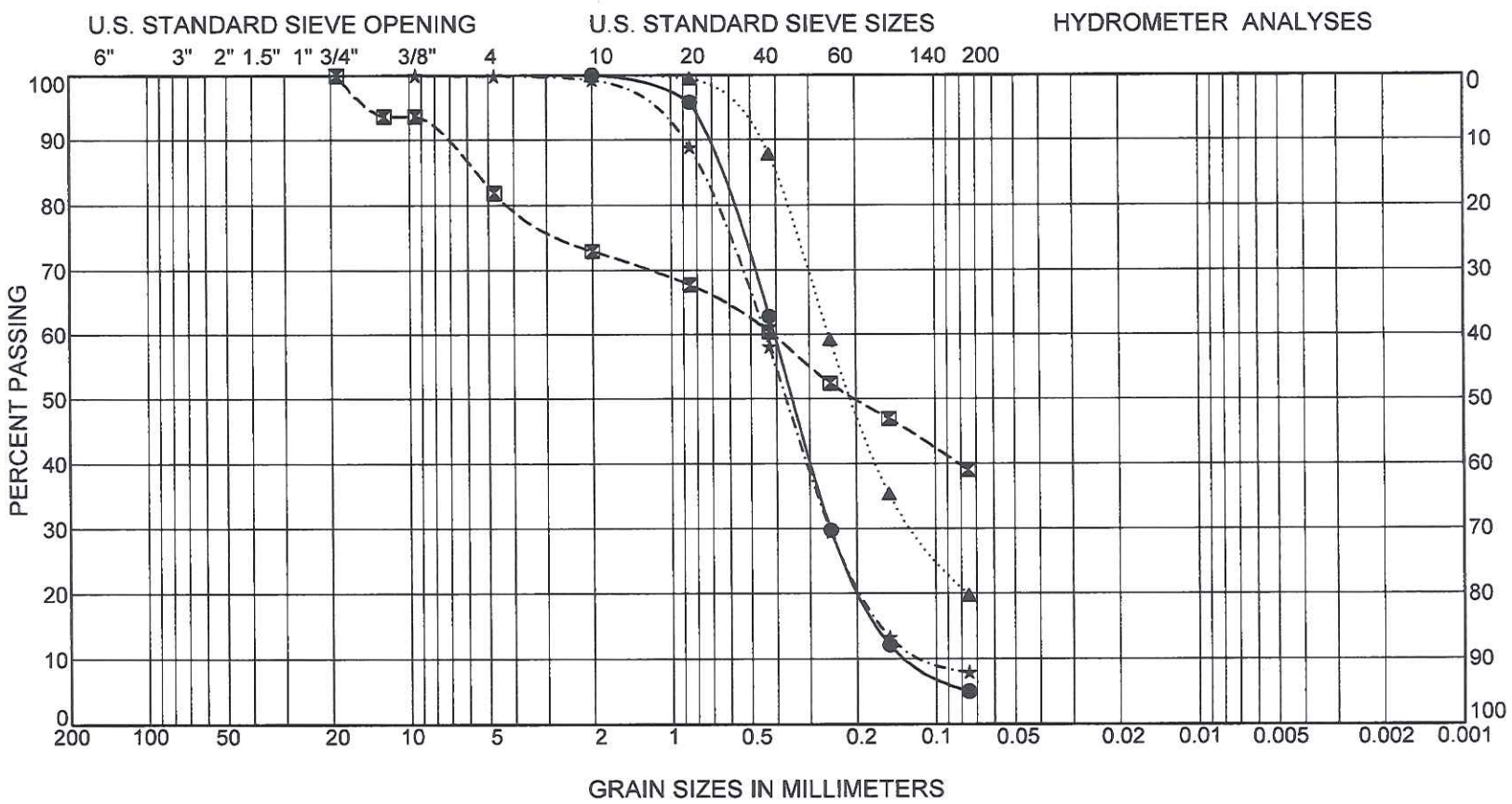
JOB NO: 207145.10

RTE 4 CROSSTOWN CONNECTOR
STOCKTON, CALIFORNIA

PLATE NO: B-3B

GRAIN SIZE DISTRIBUTION CURVES

COBBLES	GRAVEL		SAND			SILT AND CLAY
	coarse	fine	coarse	medium	fine	



Boring Number	Sample Number	Depth (feet)	Symbol	LL	PI	Description
A-09-002	MC-7	36.0	●	42	26	SILT WITH SAND (SM)
A-09-002	MC-11	56.0	⊠			SANDY SILT (ML)
A-09-003	MC-5	26.0	▲			SILTY SAND (SM)
A-09-003	MC-10	51.0	★			POORLY GRADED SAND WITH SILT (SP-SM)

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GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

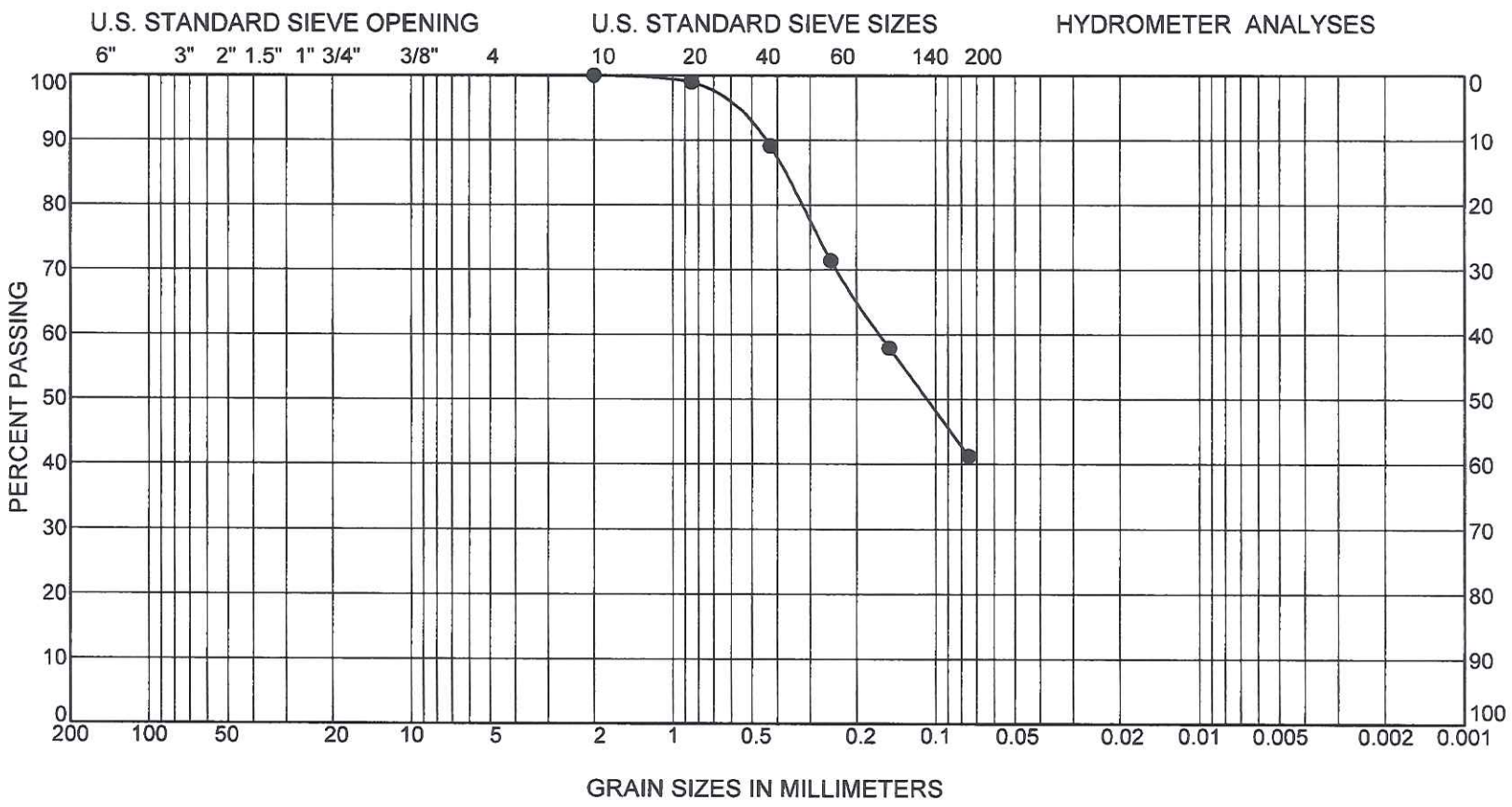
JOB NO: 207145.10

RTE 4 CROSSTOWN CONNECTOR
STOCKTON, CALIFORNIA

PLATE NO: B-3C

GRAIN SIZE DISTRIBUTION CURVES

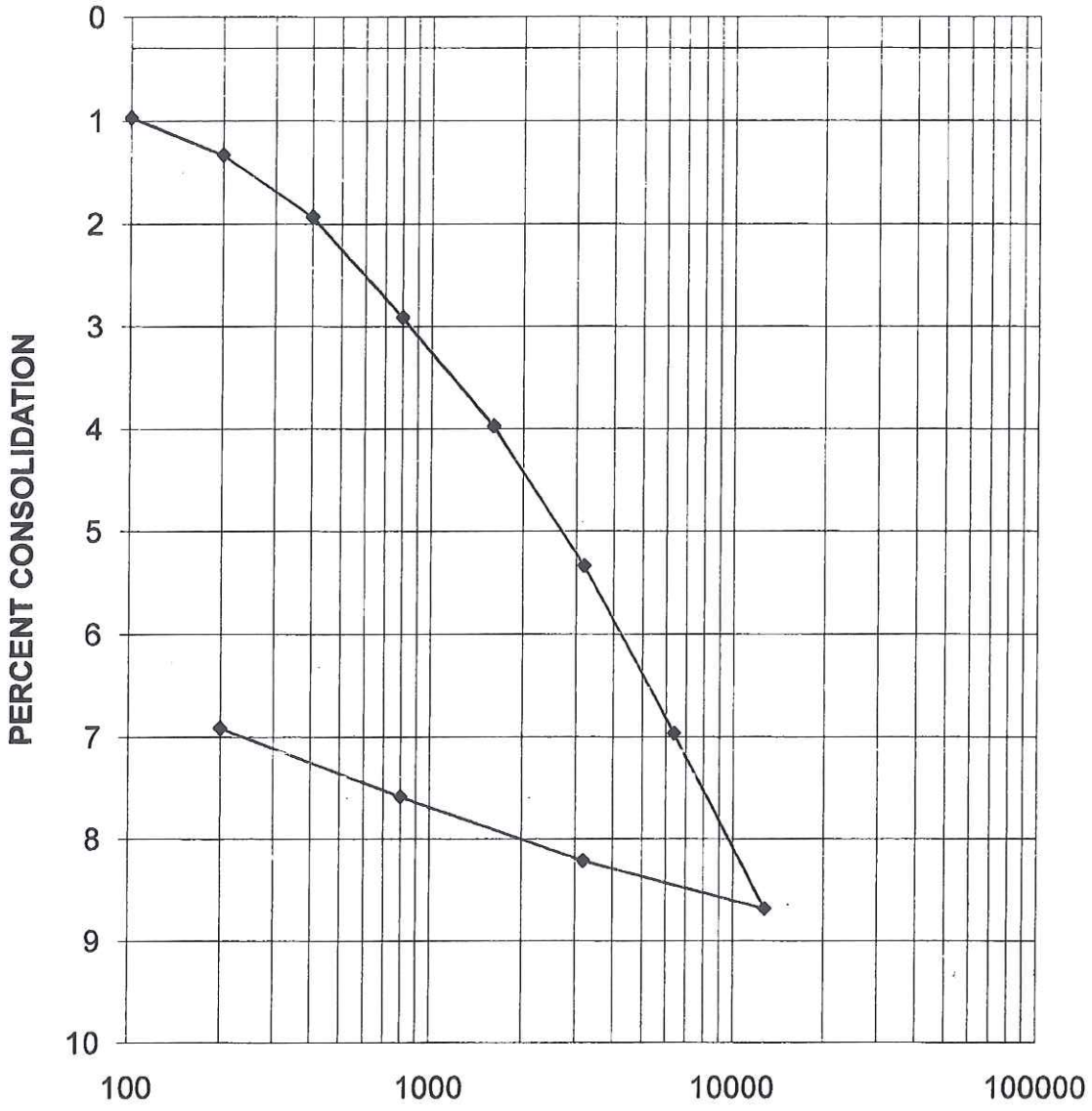
COBBLES	GRAVEL		SAND			SILT AND CLAY
	coarse	fine	coarse	medium	fine	



Boring Number	Sample Number	Depth (feet)	Symbol	LL	PI	Description
A-09-003	MC-11	56.0	●			SILTY SAND (SM)

Consolidation Test Results

CONSOLIDATION TEST RESULTS



	MOISTURE CONTENT %	DRY DENSITY PCF	HEIGHT (INCHES)	DIAMETER (INCHES)
INITIAL	25.0	101.9	1.0000	2.416
FINAL	21.8	109.4	0.9309	2.416

BORING NO.	B1	SAMPLE NO.	3	ELEV. OR DEPTH	16'
DESCRIPTION	Silt, light gray				

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

Rte 4 Crosstown Connector
BKF ENGINEERING

DATE	JOB NO:
12/07/09	207145.PFR

Reported by: Prav Dayah

PLATE NO.: B-4A

CONSOLIDATION TEST - TIME CURVE

JOB #: 207145.PFR

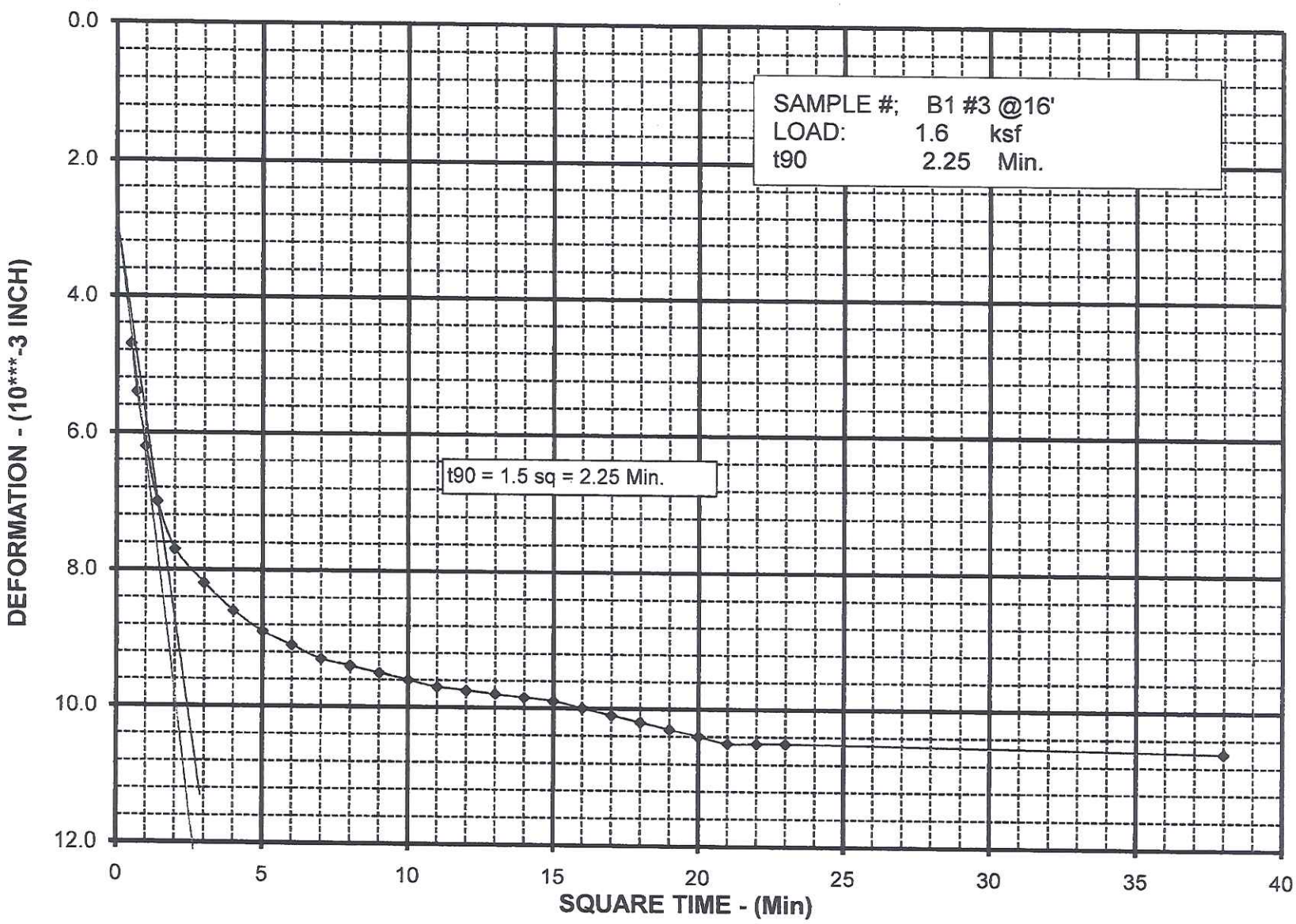


PLATE NO.: B-4B

CONSOLIDATION TEST - TIME CURVE

JOB #: 207145.PFR

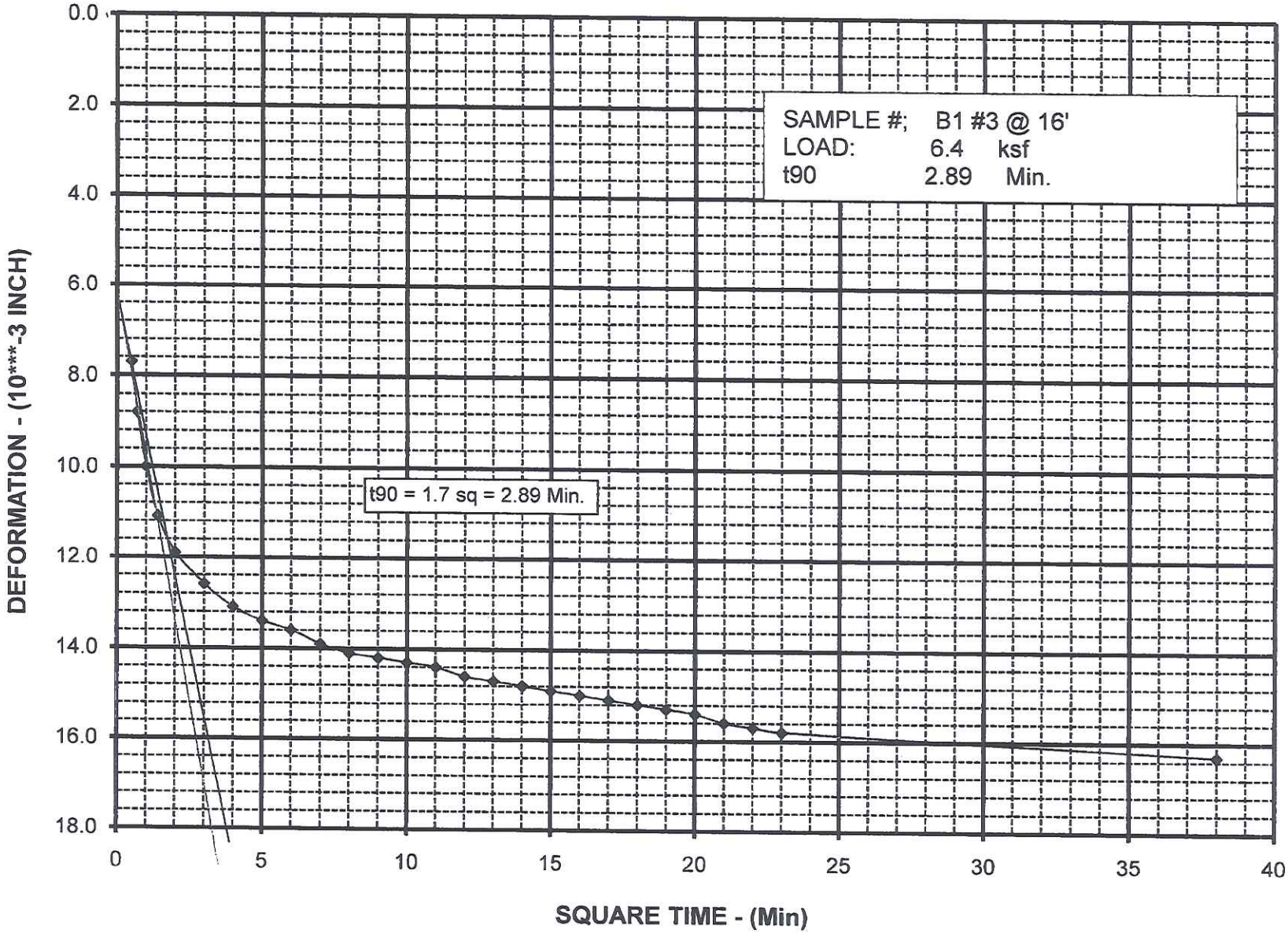
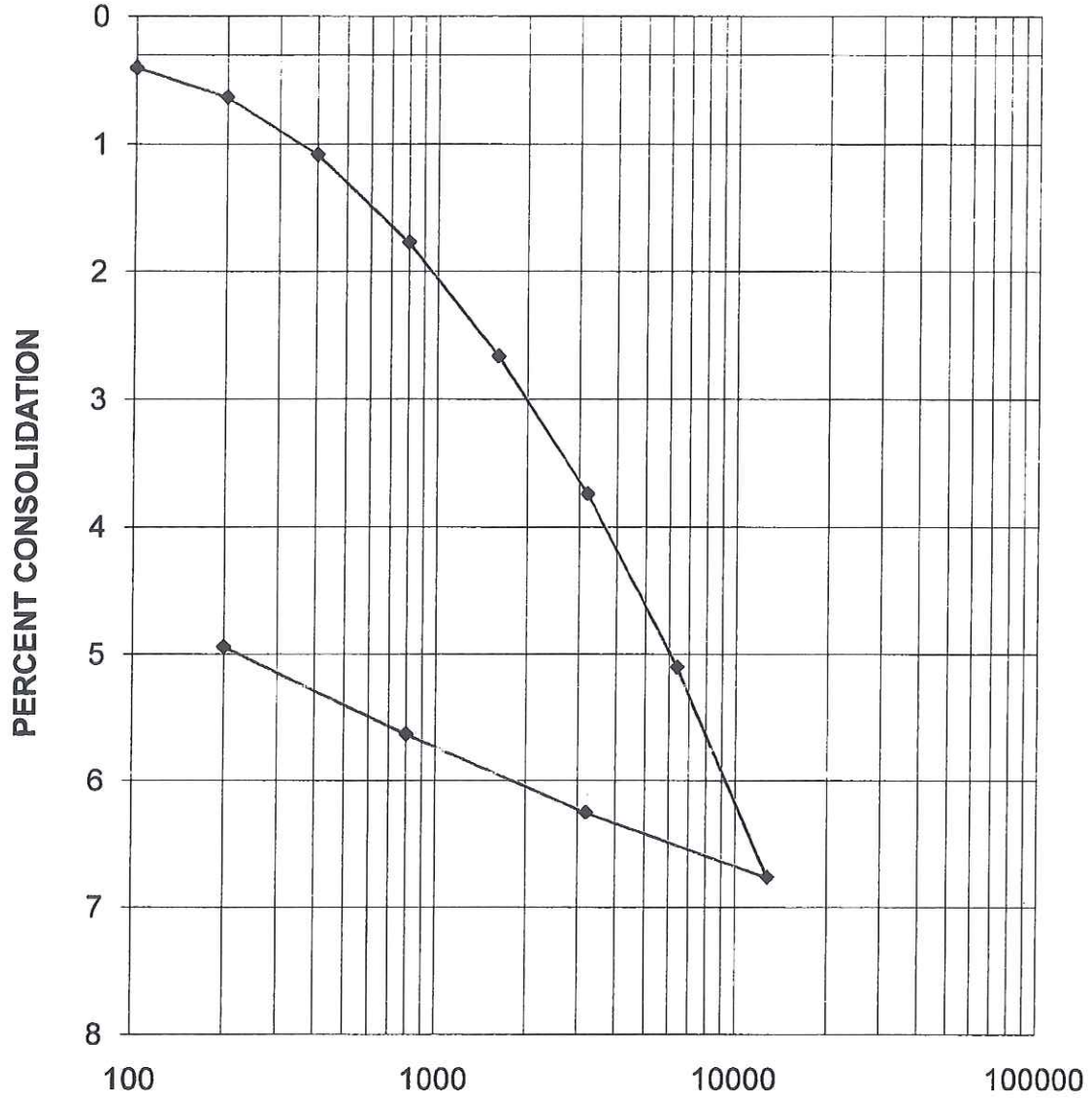


PLATE NO.: B-4C

CONSOLIDATION TEST RESULTS



	MOISTURE CONTENT %	DRY DENSITY PCF	HEIGHT (INCHES)	DIAMETER (INCHES)
INITIAL	24.2	103.1	1.0000	2.416
FINAL	21.6	108.5	0.9506	2.416

BORING NO.	B3	SAMPLE NO.	3	ELEV. OR DEPTH	16'
DESCRIPTION	Lean clay, brown				

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

Rte 4 Crosstown Connector
BKF ENGINEERING

DATE	JOB NO:
12/07/09	207145.PFR

Reported by: Prav Dayah

PLATE NO.: B-4D

CONSOLIDATION TEST - TIME CURVE

JOB #: 207145.PFR

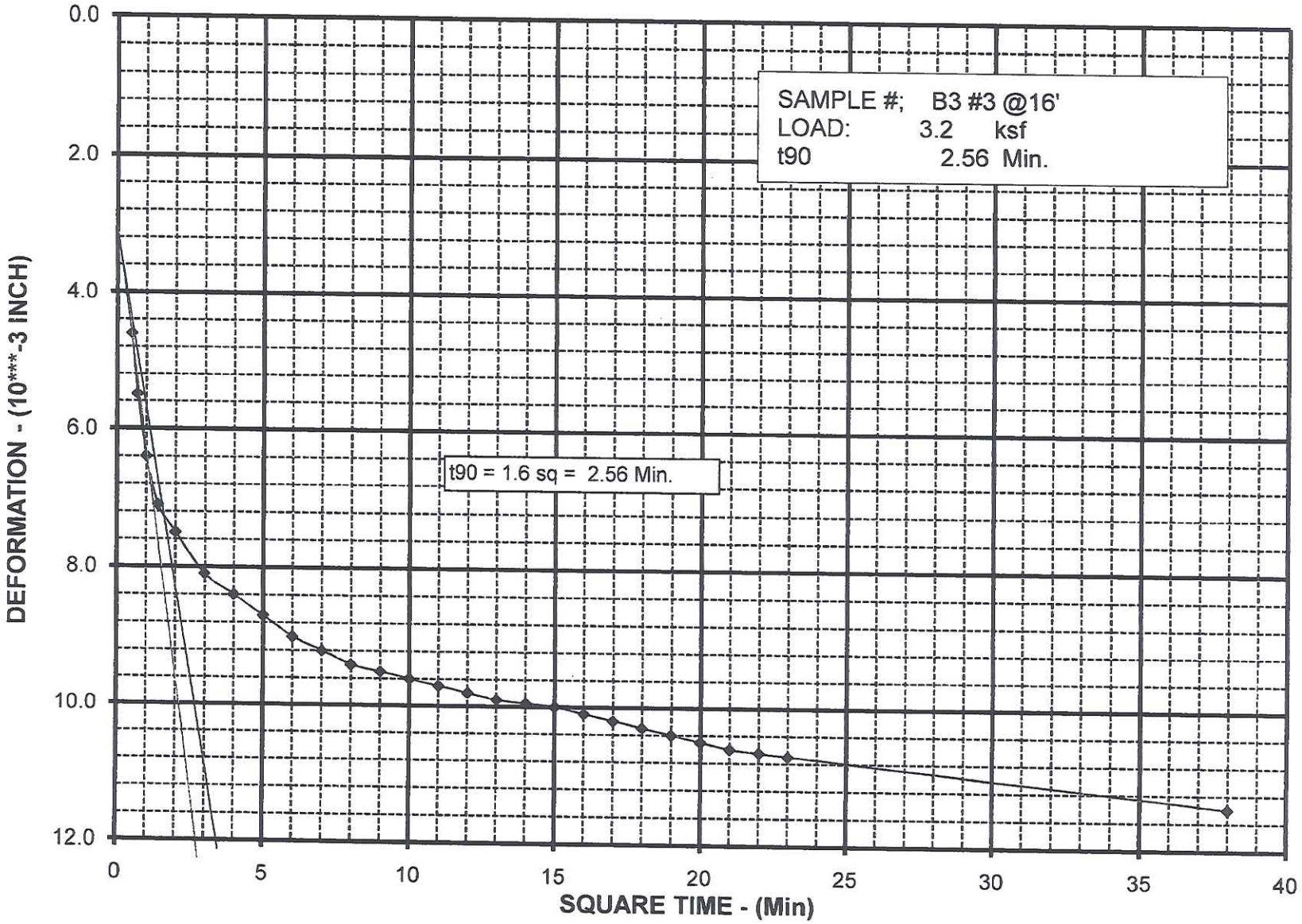


PLATE NO.: B-4E

CONSOLIDATION TEST - TIME CURVE

JOB #: 207145.PFR

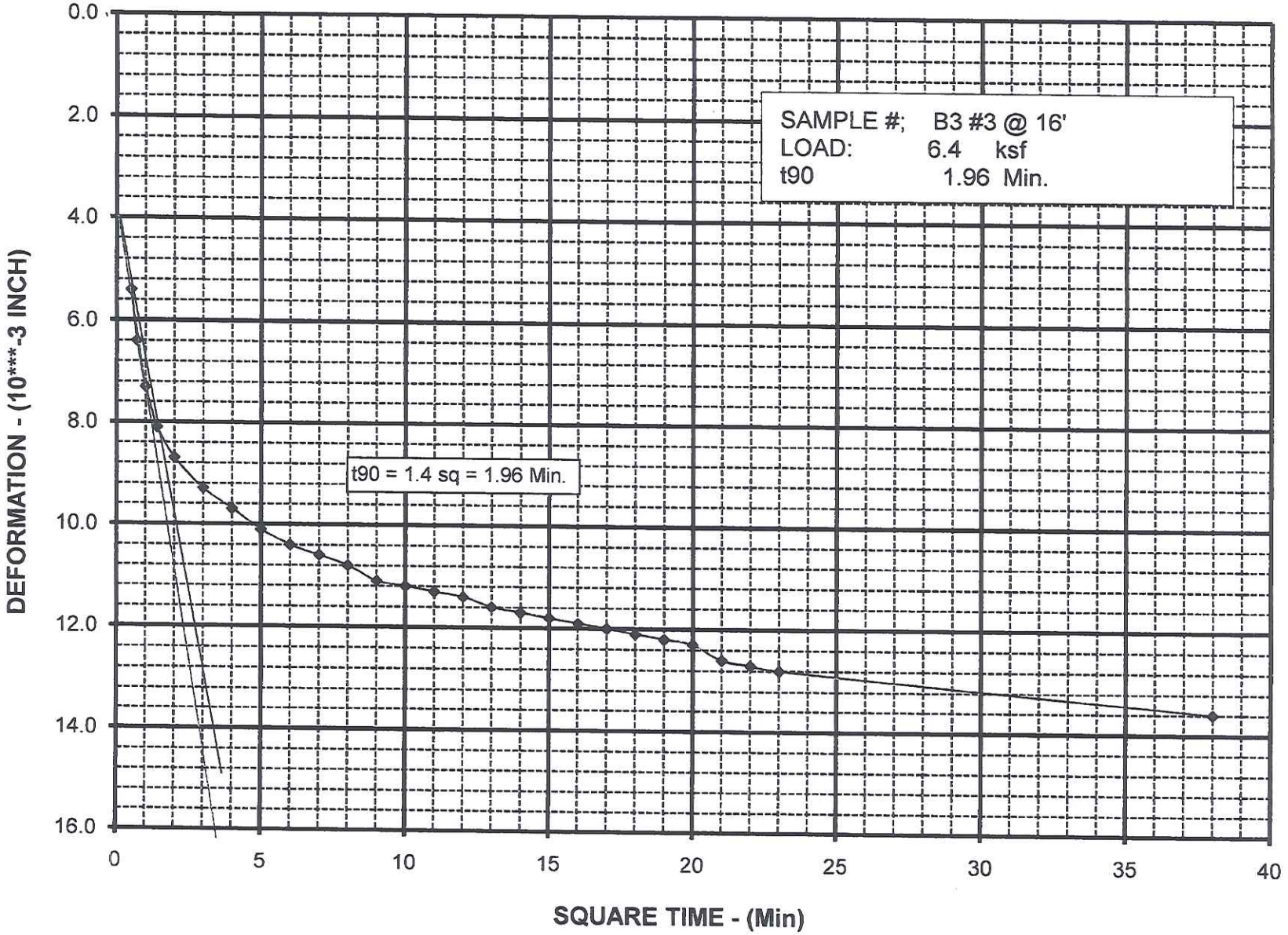
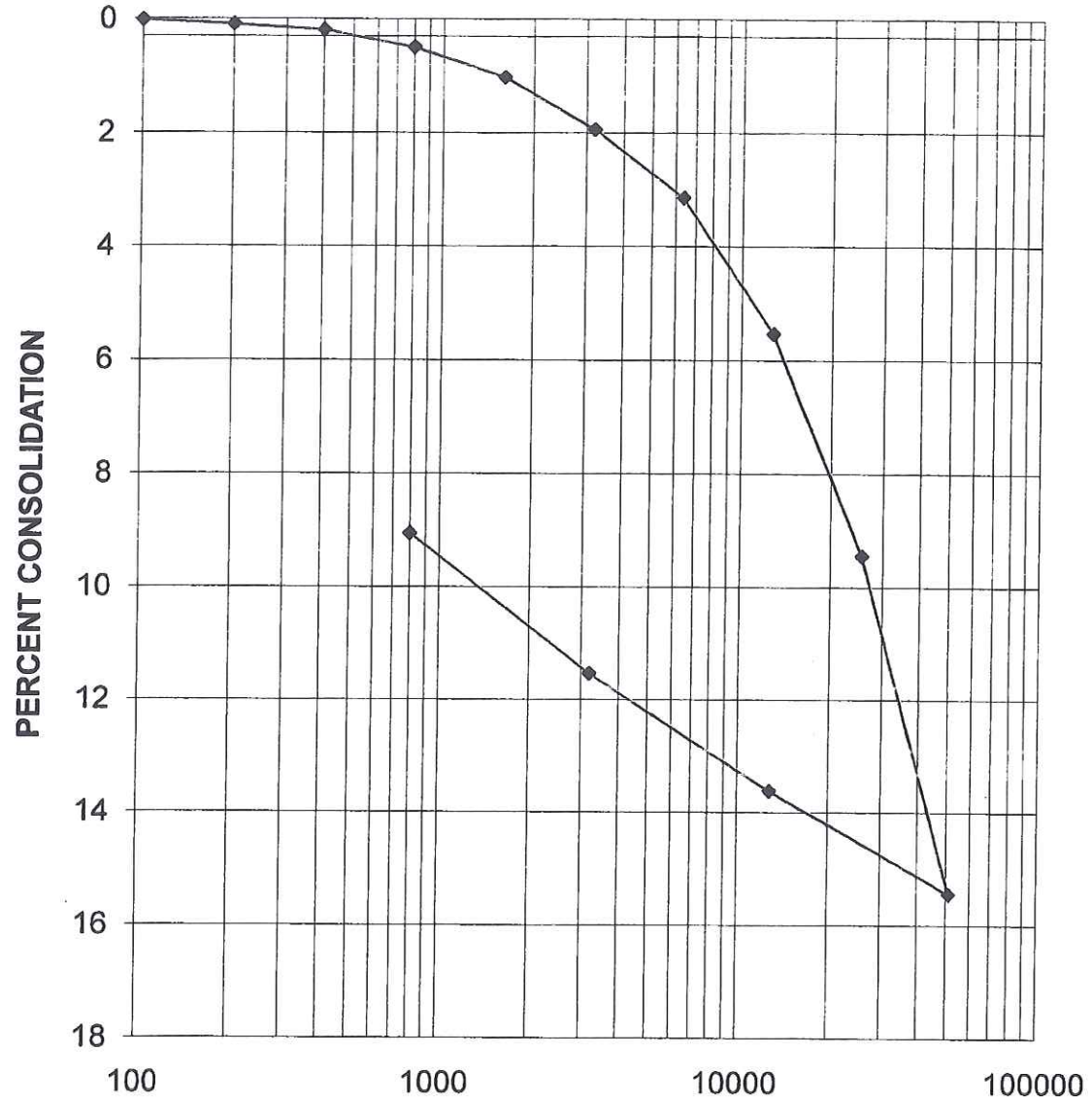


PLATE NO.: B-4F

CONSOLIDATION TEST RESULTS



	MOISTURE CONTENT %	DRY DENSITY PCF	HEIGHT (INCHES)	DIAMETER (INCHES)
INITIAL	29.2	98.7	1.0000	2.416
FINAL	23.8	108.5	0.9095	2.416

BORING NO.	B3	SAMPLE NO.	8	ELEV. OR DEPTH	41'
DESCRIPTION	Lean clay, bluish gray				

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GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

Rte 4 Crosstown Connector
BKF ENGINEERING

DATE	JOB NO:
12/7/2009	207143.PFR

Reported by: Prav Dayah

PLATE NO.: B-4G

CONSOLIDATION TEST - TIME CURVE

JOB #: 207143.PFR

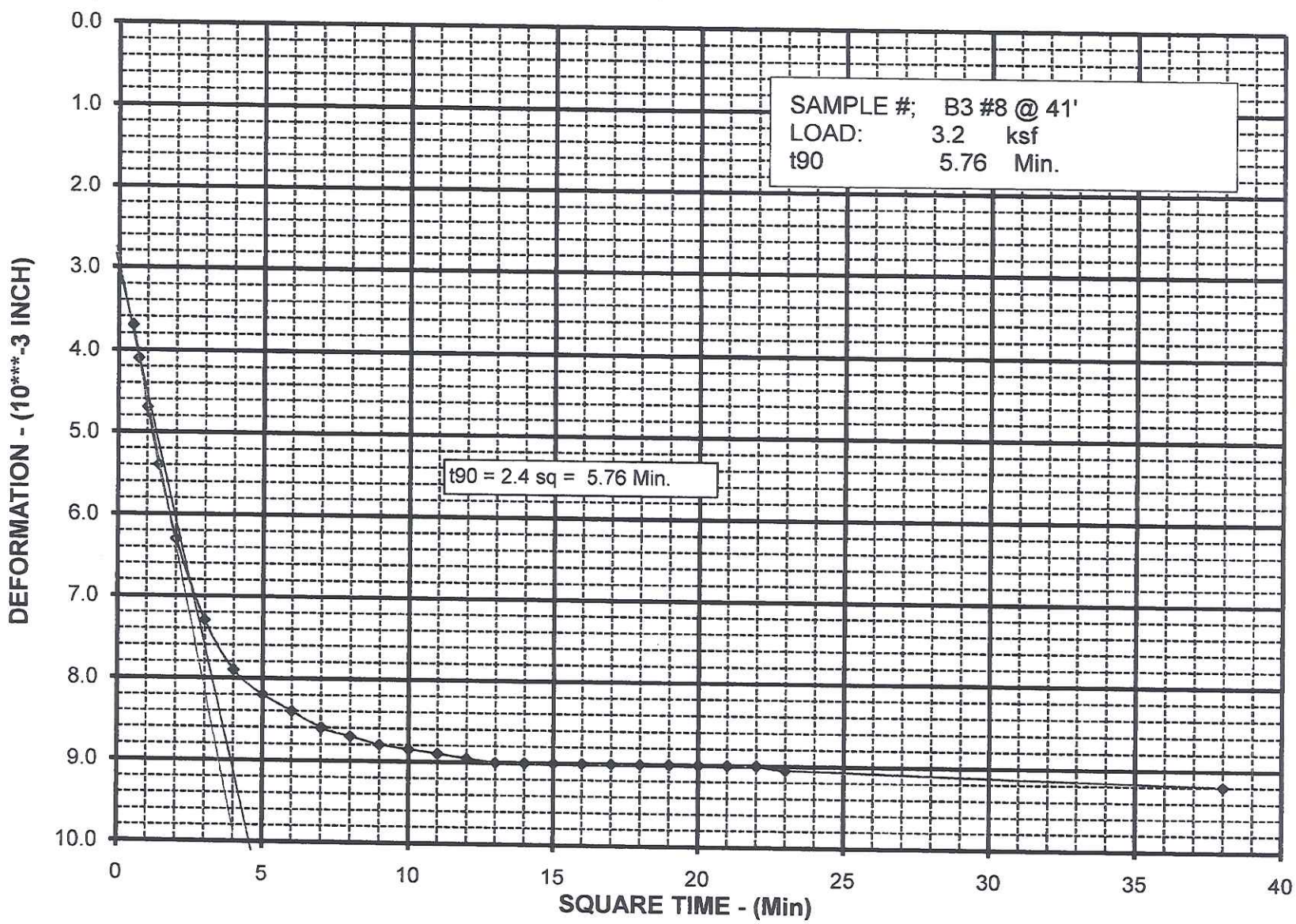

PLATE NO.: B-4H

CONSOLIDATION TEST - TIME CURVE

JOB #: 207143.PFR

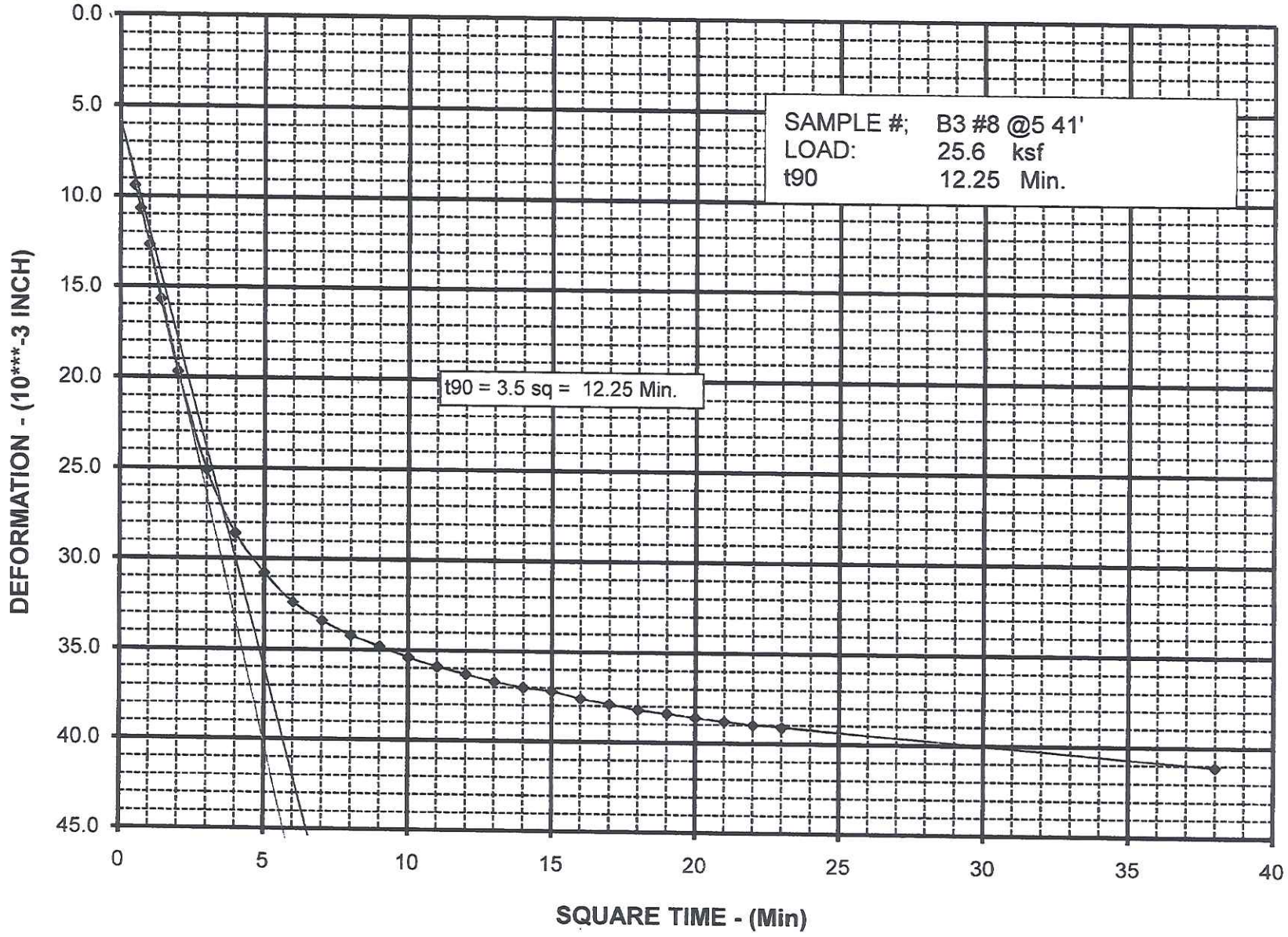

PLATE NO.: B-4I

APPENDIX F
ARS CURVE DEVELOPMENT

LIST OF ATTACHMENTS

The following are attached and complete this appendix.

Development of Shear Wave Velocity Parameter V_{s30}
Development of Probabilistic and Deterministic Response Spectrums

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Development of Shear Wave Velocity Parameter V_{s30}

Project Name	SR-4 Expansion	Calc by:	Jim Wetenkamp
Project Number	111904.G01	Checked by:	Carl Henderson
Date	April 28, 2011		
CPT Number	CPT-11-001	Average small strain sheer wave velocity in meters per second	*Vs30= 221*
CPT Max Depth (ft)	90.05		
Total d_i/V_i	0.42249		

1. For cohesive soils, Mayne and Rix (1995) is used to calculate V_i. $V_S = 1.75(q_t)^{0.627}$
For cohesionless soils, Mayne (2007) is used to calculate V_i. $V_S=277(q_t)^{0.13}(s'_{vo})^{0.27}$

2. If CPT depth is less than 100 feet, Boore (2004) is used to extrapolate to 100 feet.
$V_{S30} = [1.45-(0.015d)]V_{s(d)}$, where d = CPT depth (m), $V_{s(d)}$ = time averaged velocity (m/sec)

Depth (ft)	Layer Thickness (ft)	qt (tsf)	Soil Type	s'_{vo} (ksf)	Shear Wave Velocity (V_i) (m/sec)	d_i/V_i
0.57	0.57	157.15	7	0.07	84.60	0.00674
0.64	0.07	153.53	6	0.08	717.77	0.00010
0.71	0.07	153.86	7	0.09	90.29	0.00078
0.79	0.08	133.07	6	0.1	656.20	0.00012
0.87	0.08	117.89	6	0.11	608.21	0.00013
0.94	0.07	103.47	11	0.12	92.68	0.00076
1.02	0.08	114.62	11	0.13	95.97	0.00083
1.1	0.08	98.92	11	0.14	96.05	0.00083
1.18	0.08	86.12	11	0.15	96.11	0.00083
1.27	0.09	74.78	11	0.16	96.02	0.00094
1.36	0.09	58.66	11	0.17	94.58	0.00095
1.43	0.07	90.12	11	0.18	101.56	0.00069
1.52	0.09	149.55	12	0.2	111.60	0.00081
1.59	0.07	155.55	12	0.21	113.66	0.00062
1.68	0.09	158.18	7	0.22	115.35	0.00078
1.75	0.07	147.01	7	0.23	115.64	0.00061
1.84	0.09	147.6	7	0.24	117.03	0.00077
1.91	0.07	145	7	0.25	118.06	0.00059
1.97	0.06	137.16	7	0.26	118.46	0.00051
2.05	0.08	117.48	6	0.27	606.89	0.00013
2.13	0.08	96.67	5	0.28	537.05	0.00015
2.21	0.08	69.74	11	0.29	111.73	0.00072
2.3	0.09	51.86	3	0.3	363.45	0.00025
2.38	0.08	45.39	3	0.31	334.31	0.00024
2.46	0.08	38.35	3	0.32	300.79	0.00027
2.54	0.08	34.07	3	0.33	279.28	0.00029
2.62	0.08	29.12	3	0.34	253.10	0.00032
2.7	0.08	25.23	3	0.36	231.34	0.00035
2.78	0.08	25.38	3	0.37	232.20	0.00034
2.87	0.09	24.24	3	0.38	225.60	0.00040
2.95	0.08	21.75	3	0.39	210.78	0.00038
3.03	0.08	20.94	3	0.4	205.82	0.00039
3.11	0.08	21.11	3	0.41	206.87	0.00039
3.2	0.09	21.89	3	0.42	211.63	0.00043
3.28	0.08	24.56	3	0.43	227.46	0.00035

Project Name	SR-4 Expansion	Calc by:	Jim Wetenkamp
Project Number	111904.G01	Checked by:	Carl Henderson
Date	April 28, 2011		
CPT Number	CPT-11-002	Average small strain sheer wave velocity in meters per second	*Vs30= 243*
CPT Max Depth (ft)	90.07		
Total d_i/V_i	0.38454		

1. For cohesive soils, Mayne and Rix (1995) is used to calculate V_i. $V_S = 1.75(q_t)^{0.627}$
For cohesionless soils, Mayne (2007) is used to calculate V_i. $V_S=277(q_t)^{0.13}(s'_{vo})^{0.27}$

2. If CPT depth is less than 100 feet, Boore (2004) is used to extrapolate to 100 feet.
$V_{S30} = [1.45-(0.015d)]V_{s(d)}$, where d = CPT depth (m), $V_{s(d)}$ = time averaged velocity (m/sec)

Depth (ft)	Layer Thickness (ft)	qt (tsf)	Soil Type	s'_{vo} (ksf)	Shear Wave Velocity (V_i) (m/sec)	d_i/V_i
0.52	0.52	315.08	9	0.06	88.83	0.00585
0.56	0.04	309.3	9	0.07	92.38	0.00043
0.62	0.06	286.76	9	0.07	91.48	0.00066
0.68	0.06	273.29	9	0.08	94.25	0.00064
0.71	0.03	267.52	9	0.09	97.02	0.00031
0.77	0.06	251.12	9	0.09	96.23	0.00062
0.81	0.04	250.68	9	0.1	98.98	0.00040
0.86	0.05	251.39	9	0.11	101.60	0.00049
0.91	0.05	238.56	7	0.11	100.91	0.00050
0.96	0.05	240.02	7	0.12	103.39	0.00048
1.01	0.05	228.5	7	0.13	104.98	0.00048
1.06	0.05	230.17	7	0.13	105.08	0.00048
1.11	0.05	197.34	12	0.14	105.08	0.00048
1.17	0.06	169.69	12	0.15	104.97	0.00057
1.22	0.05	157.26	12	0.16	105.77	0.00047
1.29	0.07	128.53	11	0.17	104.73	0.00067
1.35	0.06	101.49	11	0.17	101.56	0.00059
1.41	0.06	72.82	11	0.18	98.79	0.00061
1.48	0.07	67.94	11	0.19	99.34	0.00070
1.54	0.06	60.98	11	0.2	99.32	0.00060
1.6	0.06	61.65	11	0.21	100.78	0.00060
1.66	0.06	65.98	11	0.21	101.67	0.00059
1.72	0.06	65.92	5	0.22	422.44	0.00014
1.77	0.05	59.51	5	0.23	396.19	0.00013
1.83	0.06	72.4	6	0.24	448.02	0.00013
1.89	0.06	78.76	6	0.25	472.31	0.00013
1.96	0.07	109.82	7	0.26	115.08	0.00061
2.03	0.07	140.72	7	0.26	118.85	0.00059
2.07	0.04	144.9	7	0.27	120.53	0.00033
2.12	0.05	151.53	8	0.28	122.43	0.00041
2.19	0.07	163.95	7	0.29	124.86	0.00056
2.25	0.06	150.08	7	0.3	124.57	0.00048
2.31	0.06	133.72	6	0.3	658.21	0.00009
2.37	0.06	125.93	6	0.31	633.90	0.00009
2.43	0.06	127.49	6	0.32	638.81	0.00009

Project Name	SR-4 Expansion	Calc by:	Jim Wetenkamp
Project Number	111904.G01	Checked by:	Carl Henderson
Date	April 28, 2011		
CPT Number	CPT-11-003	Average small strain sheer wave velocity in meters per second	*Vs30= 250*
CPT Max Depth (ft)	90.07		
Total d_i/V_i	0.37470		

1. For cohesive soils, Mayne and Rix (1995) is used to calculate V_i. $V_S = 1.75(q_t)^{0.627}$
For cohesionless soils, Mayne (2007) is used to calculate V_i. $V_S=277(q_t)^{0.13}(s'_{vo})^{0.27}$

2. If CPT depth is less than 100 feet, Boore (2004) is used to extrapolate to 100 feet.
$V_{S30} = [1.45-(0.015d)]V_{s(d)}$, where d = CPT depth (m), $V_{s(d)}$ = time averaged velocity (m/sec)

Depth (ft)	Layer Thickness (ft)	qt (tsf)	Soil Type	s'_{vo} (ksf)	Shear Wave Velocity (V_i) (m/sec)	d_i/V_i
0.52	0.52	6.76	3	0.06	101.30	0.00513
0.61	0.09	6.15	3	0.07	95.47	0.00094
0.7	0.09	5.83	3	0.08	92.32	0.00097
0.78	0.08	5.93	3	0.09	93.31	0.00086
0.86	0.08	5.94	3	0.1	93.41	0.00086
0.94	0.08	6.9	3	0.11	102.61	0.00078
1.02	0.08	15.83	3	0.12	172.71	0.00046
1.1	0.08	26.49	5	0.13	238.51	0.00034
1.19	0.09	31.37	5	0.14	265.19	0.00034
1.27	0.08	34.1	5	0.15	279.43	0.00029
1.36	0.09	40.87	6	0.16	313.03	0.00029
1.44	0.08	38.82	5	0.17	303.09	0.00026
1.53	0.09	33.97	5	0.19	278.76	0.00032
1.61	0.08	30.43	5	0.2	260.18	0.00031
1.7	0.09	27.22	5	0.21	242.61	0.00037
1.79	0.09	24.51	5	0.22	227.17	0.00040
1.87	0.08	20.55	5	0.23	203.41	0.00039
1.95	0.08	20.08	5	0.24	200.48	0.00040
2.03	0.08	18.99	5	0.25	193.59	0.00041
2.12	0.09	17.88	5	0.26	186.41	0.00048
2.21	0.09	16.27	5	0.27	175.70	0.00051
2.3	0.09	14.56	4	0.28	163.89	0.00055
2.39	0.09	12.39	3	0.3	148.11	0.00061
2.46	0.07	7.89	3	0.3	111.61	0.00063
2.5	0.04	8.86	3	0.31	120.03	0.00033
2.58	0.08	7.98	3	0.32	112.41	0.00071
2.67	0.09	7.92	3	0.33	111.88	0.00080
2.75	0.08	7.83	3	0.34	111.08	0.00072
2.84	0.09	10.25	3	0.35	131.51	0.00068
2.92	0.08	10.9	3	0.36	136.68	0.00059
3.01	0.09	14.97	3	0.37	166.76	0.00054
3.09	0.08	21.74	3	0.38	210.72	0.00038
3.18	0.09	21.9	3	0.39	211.69	0.00043
3.27	0.09	19.88	3	0.4	199.23	0.00045
3.36	0.09	19.87	3	0.42	199.16	0.00045

Project Name	SR-4 Expansion	Calc by:	Jim Wetenkamp
Project Number	111904.G01	Checked by:	Carl Henderson
Date	April 28, 2011		
CPT Number	CPT-11-004	Average small strain sheer wave velocity in meters per second	*Vs30= 237*
CPT Max Depth (ft)	90.07		
Total d_i/V_i	0.39526		

1. For cohesive soils, Mayne and Rix (1995) is used to calculate V_i. $V_S = 1.75(q_t)^{0.627}$
For cohesionless soils, Mayne (2007) is used to calculate V_i. $V_S=277(q_t)^{0.13}(s'_{vo})^{0.27}$

2. If CPT depth is less than 100 feet, Boore (2004) is used to extrapolate to 100 feet.
$V_{S30} = [1.45-(0.015d)]V_{s(d)}$, where d = CPT depth (m), $V_{s(d)}$ = time averaged velocity (m/sec)

Depth (ft)	Layer Thickness (ft)	qt (tsf)	Soil Type	s'_{vo} (ksf)	Shear Wave Velocity (V_i) (m/sec)	d_i/V_i
0.52	0.52	11.77	5	0.06	143.42	0.00363
0.62	0.10	13.3	5	0.07	154.84	0.00065
0.72	0.10	13.94	5	0.08	159.48	0.00063
0.81	0.09	14.83	5	0.09	165.79	0.00054
0.91	0.10	14.33	5	0.1	162.26	0.00062
1	0.09	13.46	5	0.11	156.01	0.00058
1.08	0.08	12.57	5	0.12	149.46	0.00054
1.17	0.09	11.45	5	0.13	140.97	0.00064
1.27	0.10	10.08	5	0.15	130.14	0.00077
1.36	0.09	9.76	4	0.16	127.53	0.00071
1.45	0.09	10.33	4	0.17	132.15	0.00068
1.53	0.08	11.2	4	0.18	139.03	0.00058
1.63	0.10	12.97	4	0.19	152.42	0.00066
1.72	0.09	14.19	4	0.2	161.26	0.00056
1.81	0.09	14.82	5	0.21	165.72	0.00054
1.9	0.09	14.42	4	0.22	162.90	0.00055
2	0.10	12.73	3	0.23	150.65	0.00066
2.1	0.10	11.92	3	0.24	144.57	0.00069
2.17	0.07	12.32	3	0.25	147.59	0.00047
2.25	0.08	11.6	3	0.26	142.12	0.00056
2.33	0.08	9.84	3	0.27	128.19	0.00062
2.42	0.09	8.39	3	0.28	115.99	0.00078
2.49	0.07	7.92	3	0.29	111.88	0.00063
2.57	0.08	7.61	3	0.3	109.11	0.00073
2.65	0.08	7.28	3	0.31	106.12	0.00075
2.73	0.08	6.06	3	0.32	94.59	0.00085
2.81	0.08	6.31	3	0.33	97.02	0.00082
2.89	0.08	5.98	3	0.34	93.81	0.00085
2.97	0.08	6.55	3	0.35	99.32	0.00081
3.05	0.08	7.83	3	0.36	111.08	0.00072
3.13	0.08	9.75	3	0.37	127.45	0.00063
3.22	0.09	12.65	3	0.38	150.06	0.00060
3.29	0.07	15.48	3	0.39	170.30	0.00041
3.38	0.09	15.7	3	0.4	171.82	0.00052
3.46	0.08	15.15	3	0.41	168.02	0.00048

Project Name	SR-4 Expansion	Calc by:	Jim Wetenkamp
Project Number	111904.G01	Checked by:	Carl Henderson
Date	April 28, 2011		
CPT Number	CPT-11-005	Average small strain sheer wave velocity in meters per second	*Vs30= 255*
CPT Max Depth (ft)	90.01		
Total d_i/V_i	0.36593		

1. For cohesive soils, Mayne and Rix (1995) is used to calculate V_i. $V_S = 1.75(q_t)^{0.627}$
For cohesionless soils, Mayne (2007) is used to calculate V_i. $V_S=277(q_t)^{0.13}(s'_{vo})^{0.27}$

2. If CPT depth is less than 100 feet, Boore (2004) is used to extrapolate to 100 feet.
V_{S30} = [1.45-(0.015d)]$V_{s(d)}$, where d = CPT depth (m), $V_{s(d)}$ = time averaged velocity (m/sec)

Depth (ft)	Layer Thickness (ft)	qt (tsf)	Soil Type	s'_{vo} (ksf)	Shear Wave Velocity (V_i) (m/sec)	d_i/V_i
0.51	0.51	19.29	4	0.06	195.50	0.00261
0.56	0.05	18.4	3	0.07	189.79	0.00026
0.62	0.06	18.24	4	0.07	188.76	0.00032
0.67	0.05	17.66	4	0.08	184.97	0.00027
0.72	0.05	17.2	4	0.09	181.94	0.00027
0.77	0.05	15.66	4	0.09	171.54	0.00029
0.83	0.06	13.24	4	0.1	154.41	0.00039
0.89	0.06	11.06	3	0.11	137.94	0.00043
0.94	0.05	9.36	3	0.11	124.23	0.00040
0.99	0.05	8.41	3	0.12	116.17	0.00043
1.04	0.05	7.2	3	0.12	105.39	0.00047
1.09	0.05	8.01	3	0.13	112.67	0.00044
1.15	0.06	8.57	3	0.14	117.55	0.00051
1.2	0.05	10.51	3	0.14	133.59	0.00037
1.26	0.06	12.85	3	0.15	151.54	0.00040
1.31	0.05	13.8	3	0.16	158.47	0.00032
1.38	0.07	17.1	3	0.17	181.27	0.00039
1.43	0.05	19.6	3	0.17	197.46	0.00025
1.48	0.05	21.2	3	0.18	207.42	0.00024
1.53	0.05	23.39	3	0.19	220.61	0.00023
1.58	0.05	23.39	3	0.19	220.61	0.00023
1.64	0.06	25.28	3	0.2	231.62	0.00026
1.69	0.05	25.67	3	0.21	233.86	0.00021
1.75	0.06	23	3	0.21	218.30	0.00027
1.79	0.04	24.86	3	0.22	229.20	0.00017
1.84	0.05	26.16	3	0.23	236.65	0.00021
1.9	0.06	26.07	3	0.24	236.14	0.00025
1.95	0.05	26.54	3	0.24	238.80	0.00021
2.01	0.06	28	3	0.25	246.95	0.00024
2.08	0.07	29.21	3	0.26	253.59	0.00028
2.15	0.07	26.72	3	0.27	239.81	0.00029
2.22	0.07	25.9	3	0.28	235.17	0.00030
2.28	0.06	23.16	3	0.29	219.25	0.00027
2.36	0.08	24.3	3	0.3	225.95	0.00035
2.43	0.07	24.53	3	0.31	227.29	0.00031

Project Name	SR-4 Expansion	Calc by:	Jim Wetenkamp
Project Number	111904.G01	Checked by:	Carl Henderson
Date	April 28, 2011		
CPT Number	CPT-11-006	Average small strain sheer wave velocity in meters per second	*Vs30= 244*
CPT Max Depth (ft)	90.03		
Total d_i/V_i	0.38325		

1. For cohesive soils, Mayne and Rix (1995) is used to calculate V_i. $V_S = 1.75(q_t)^{0.627}$
For cohesionless soils, Mayne (2007) is used to calculate V_i. $V_S=277(q_t)^{0.13}(s'_{vo})^{0.27}$

2. If CPT depth is less than 100 feet, Boore (2004) is used to extrapolate to 100 feet.
$V_{S30} = [1.45-(0.015d)]V_{s(d)}$, where d = CPT depth (m), $V_{s(d)}$ = time averaged velocity (m/sec)

Depth (ft)	Layer Thickness (ft)	qt (tsf)	Soil Type	s'_{vo} (ksf)	Shear Wave Velocity (V_i) (m/sec)	d_i/V_i
0.54	0.54	49.97	5	0.06	355.08	0.00152
0.62	0.08	42.44	5	0.07	320.52	0.00025
0.7	0.08	36.76	4	0.08	292.91	0.00027
0.78	0.08	34.61	4	0.09	282.05	0.00028
0.85	0.07	33.92	5	0.11	278.51	0.00025
0.93	0.08	36.66	5	0.12	292.41	0.00027
1.01	0.08	42.53	6	0.13	320.95	0.00025
1.09	0.08	44.07	6	0.14	328.18	0.00024
1.16	0.07	40.51	6	0.15	311.30	0.00022
1.24	0.08	35.62	6	0.16	287.18	0.00028
1.31	0.07	36.14	6	0.17	289.80	0.00024
1.39	0.08	30.24	6	0.18	259.16	0.00031
1.46	0.07	27.36	6	0.19	243.40	0.00029
1.55	0.09	22.18	5	0.2	213.38	0.00042
1.62	0.07	20.05	5	0.21	200.29	0.00035
1.7	0.08	17.36	5	0.22	182.99	0.00044
1.77	0.07	16.96	5	0.23	180.34	0.00039
1.84	0.07	17.03	5	0.23	180.81	0.00039
1.94	0.10	16.79	5	0.25	179.20	0.00056
2.01	0.07	16.8	5	0.26	179.27	0.00039
2.08	0.07	15.27	4	0.26	168.85	0.00041
2.16	0.08	13.9	4	0.27	159.19	0.00050
2.23	0.07	11.15	3	0.28	138.64	0.00050
2.3	0.07	9.06	3	0.29	121.72	0.00058
2.38	0.08	7.13	3	0.3	104.74	0.00076
2.46	0.08	5.85	3	0.31	92.52	0.00086
2.53	0.07	4.82	3	0.32	81.94	0.00085
2.62	0.09	4.58	3	0.32	79.36	0.00113
2.69	0.07	5.72	3	0.33	91.23	0.00077
2.77	0.08	6.61	3	0.34	99.89	0.00080
2.86	0.09	8.15	3	0.35	113.90	0.00079
2.93	0.07	8.52	3	0.36	117.12	0.00060
3.02	0.09	9.3	3	0.37	123.73	0.00073
3.09	0.07	12.62	3	0.38	149.83	0.00047
3.18	0.09	14.65	3	0.39	164.52	0.00055

Project Name	SR-4 Expansion	Calc by:	Jim Wetenkamp
Project Number	111904.G01	Checked by:	Carl Henderson
Date	April 28, 2011		
CPT Number	CPT-11-007	Average small strain sheer wave velocity in meters per second	*Vs30= 250*
CPT Max Depth (ft)	90.04		
Total d_i/V_i	0.37341		

1. For cohesive soils, Mayne and Rix (1995) is used to calculate V_i. $V_S = 1.75(q_t)^{0.627}$
For cohesionless soils, Mayne (2007) is used to calculate V_i. $V_S=277(q_t)^{0.13}(s'_{vo})^{0.27}$

2. If CPT depth is less than 100 feet, Boore (2004) is used to extrapolate to 100 feet.
$V_{S30} = [1.45-(0.015d)]V_{s(d)}$, where d = CPT depth (m), $V_{s(d)}$ = time averaged velocity (m/sec)

Depth (ft)	Layer Thickness (ft)	qt (tsf)	Soil Type	s'_{vo} (ksf)	Shear Wave Velocity (V_i) (m/sec)	d_i/V_i
0.54	0.54	156.23	9	0.06	81.09	0.00666
0.62	0.08	149	8	0.07	84.02	0.00095
0.7	0.08	125.39	8	0.08	85.17	0.00094
0.77	0.07	107.28	7	0.09	86.16	0.00081
0.84	0.07	76.54	6	0.1	463.92	0.00015
0.91	0.07	53.52	5	0.11	370.70	0.00019
0.99	0.08	38.54	4	0.12	301.72	0.00027
1.06	0.07	30.17	3	0.13	258.78	0.00027
1.13	0.07	23.23	3	0.14	219.66	0.00032
1.21	0.08	21.24	3	0.15	207.67	0.00039
1.29	0.08	20.64	3	0.16	203.97	0.00039
1.36	0.07	25.58	3	0.17	233.34	0.00030
1.42	0.06	39.48	6	0.18	306.32	0.00020
1.5	0.08	35.66	6	0.19	287.38	0.00028
1.58	0.08	19.54	3	0.2	197.08	0.00041
1.65	0.07	17.16	3	0.21	181.67	0.00039
1.71	0.06	16.75	3	0.22	178.94	0.00034
1.79	0.08	17.9	4	0.23	186.54	0.00043
1.86	0.07	16.12	3	0.24	174.69	0.00040
1.93	0.07	14.58	3	0.25	164.03	0.00043
2.01	0.08	13.92	3	0.26	159.33	0.00050
2.09	0.08	12.79	3	0.27	151.09	0.00053
2.16	0.07	12.23	3	0.28	146.91	0.00048
2.23	0.07	12.33	3	0.28	147.66	0.00047
2.31	0.08	11.77	3	0.29	143.42	0.00056
2.38	0.07	11.28	3	0.3	139.65	0.00050
2.45	0.07	11.03	3	0.31	137.70	0.00051
2.53	0.08	11.44	3	0.32	140.89	0.00057
2.6	0.07	11.84	3	0.33	143.96	0.00049
2.67	0.07	11.75	3	0.34	143.27	0.00049
2.74	0.07	12.41	3	0.35	148.26	0.00047
2.82	0.08	12.98	3	0.36	152.50	0.00052
2.9	0.08	12.4	3	0.37	148.19	0.00054
2.98	0.08	12.71	3	0.38	150.50	0.00053
3.06	0.08	13.21	3	0.39	154.19	0.00052

Project Name	SR-4 Expansion	Calc by:	Jim Wetenkamp
Project Number	111904.G01	Checked by:	Carl Henderson
Date	April 28, 2011		
CPT Number	CPT-11-008	Average small strain sheer wave velocity in meters per second	*Vs30= 248*
CPT Max Depth (ft)	90.04		
Total d_i/V_i	0.37652		

1. For cohesive soils, Mayne and Rix (1995) is used to calculate V_i. $V_S = 1.75(q_t)^{0.627}$
For cohesionless soils, Mayne (2007) is used to calculate V_i. $V_S=277(q_t)^{0.13}(s'_{vo})^{0.27}$

2. If CPT depth is less than 100 feet, Boore (2004) is used to extrapolate to 100 feet.
$V_{S30} = [1.45-(0.015d)]V_{s(d)}$, where d = CPT depth (m), $V_{s(d)}$ = time averaged velocity (m/sec)

Depth (ft)	Layer Thickness (ft)	qt (tsf)	Soil Type	s'_{vo} (ksf)	Shear Wave Velocity (V_i) (m/sec)	d_i/V_i
0.56	0.56	3.58	3	0.06	68.00	0.00823
0.61	0.05	3.26	3	0.07	64.13	0.00078
0.67	0.06	3.42	3	0.07	66.08	0.00091
0.73	0.06	3.57	3	0.08	67.88	0.00088
0.79	0.06	5.67	3	0.08	90.73	0.00066
0.84	0.05	5.43	3	0.09	88.30	0.00057
0.9	0.06	7.2	3	0.1	105.39	0.00057
0.96	0.06	10.18	4	0.1	130.95	0.00046
1.01	0.05	16.16	5	0.11	174.96	0.00029
1.07	0.06	21.46	6	0.12	209.01	0.00029
1.13	0.06	24.28	6	0.12	225.84	0.00027
1.19	0.06	13.26	4	0.13	154.55	0.00039
1.26	0.07	12.2	5	0.14	146.69	0.00048
1.32	0.06	10.83	5	0.15	136.13	0.00044
1.38	0.06	10.91	5	0.15	136.76	0.00044
1.44	0.06	9.46	5	0.16	125.06	0.00048
1.51	0.07	9.38	5	0.17	124.40	0.00056
1.57	0.06	8.58	5	0.17	117.64	0.00051
1.63	0.06	8.49	5	0.18	116.86	0.00051
1.7	0.07	9.13	5	0.19	122.31	0.00057
1.76	0.06	9.13	5	0.19	122.31	0.00049
1.83	0.07	9.46	5	0.2	125.06	0.00056
1.89	0.06	9.86	5	0.21	128.35	0.00047
1.95	0.06	9.46	5	0.21	125.06	0.00048
2.03	0.08	8.73	5	0.22	118.92	0.00067
2.09	0.06	9.62	5	0.23	126.38	0.00047
2.16	0.07	9.62	5	0.24	126.38	0.00055
2.23	0.07	9.22	5	0.24	123.06	0.00057
2.29	0.06	8.25	4	0.25	114.78	0.00052
2.36	0.07	8.09	3	0.26	113.38	0.00062
2.42	0.06	7.93	3	0.26	111.97	0.00054
2.49	0.07	7.77	3	0.27	110.54	0.00063
2.55	0.06	7.85	3	0.28	111.26	0.00054
2.63	0.08	8.02	3	0.29	112.76	0.00071
2.69	0.06	6.56	3	0.29	99.41	0.00060

Project Name	SR-4 Expansion	Calc by:	Jim Wetenkamp
Project Number	111904.G01	Checked by:	Carl Henderson
Date	April 28, 2011		
CPT Number	CPT-11-009	Average small strain sheer wave velocity in meters per second	*Vs30= 258*
CPT Max Depth (ft)	50.05		
Total d_i/V_i	0.23723		

1. For cohesive soils, Mayne and Rix (1995) is used to calculate V_i. $V_S = 1.75(q_t)^{0.627}$
For cohesionless soils, Mayne (2007) is used to calculate V_i. $V_S=277(q_t)^{0.13}(s'_{vo})^{0.27}$

2. If CPT depth is less than 100 feet, Boore (2004) is used to extrapolate to 100 feet.
$V_{S30} = [1.45-(0.015d)]V_{s(d)}$, where d = CPT depth (m), $V_{s(d)}$ = time averaged velocity (m/sec)

Depth (ft)	Layer Thickness (ft)	qt (tsf)	Soil Type	s'_{vo} (ksf)	Shear Wave Velocity (V_i) (m/sec)	d_i/V_i
0.55	0.55	26.56	6	0.06	238.91	0.00230
0.64	0.09	36.01	6	0.07	289.15	0.00031
0.73	0.09	43.39	7	0.09	76.59	0.00118
0.81	0.08	49.99	7	0.1	80.27	0.00100
0.9	0.09	62.08	7	0.11	84.71	0.00106
0.99	0.09	74.99	7	0.12	88.88	0.00101
1.07	0.08	85.36	8	0.13	92.36	0.00087
1.16	0.09	94.14	8	0.14	95.44	0.00094
1.23	0.07	106.03	8	0.15	98.75	0.00071
1.32	0.09	128.73	8	0.16	103.05	0.00087
1.39	0.07	174.2	9	0.17	108.95	0.00064
1.48	0.09	215.91	9	0.18	113.78	0.00079
1.56	0.08	227.8	9	0.19	116.26	0.00069
1.64	0.08	225.64	9	0.21	119.29	0.00067
1.71	0.07	216.94	8	0.22	120.19	0.00058
1.79	0.08	206.72	8	0.23	120.88	0.00066
1.87	0.08	197.53	7	0.24	121.55	0.00066
1.94	0.07	188.97	9	0.25	122.19	0.00057
2.02	0.08	181.26	9	0.26	122.83	0.00065
2.1	0.08	171.73	8	0.27	123.22	0.00065
2.17	0.07	158.96	8	0.28	123.19	0.00057
2.23	0.06	131.49	7	0.28	120.19	0.00050
2.3	0.07	118.54	7	0.29	119.71	0.00058
2.37	0.07	99	7	0.3	118.01	0.00059
2.44	0.07	79.66	6	0.31	475.68	0.00015
2.51	0.07	60.63	6	0.32	400.85	0.00017
2.59	0.08	44	5	0.33	327.86	0.00024
2.67	0.08	37.69	4	0.34	297.53	0.00027
2.74	0.07	31.3	3	0.35	264.82	0.00026
2.82	0.08	26.99	3	0.36	241.33	0.00033
2.89	0.07	25.13	3	0.38	230.76	0.00030
2.96	0.07	24.47	3	0.38	226.94	0.00031
3.04	0.08	27.28	4	0.39	242.95	0.00033
3.11	0.07	29.92	5	0.4	257.44	0.00027
3.19	0.08	29.21	4	0.41	253.59	0.00032

Project Name	SR-4 Expansion	Calc by:	Jim Wetenkamp
Project Number	111904.G01	Checked by:	Carl Henderson
Date	April 28, 2011		
CPT Number	CPT-11-010	Average small strain sheer wave velocity in meters per second	*Vs30= 253*
CPT Max Depth (ft)	100		
Total d_i/V_i	0.39581		

1. For cohesive soils, Mayne and Rix (1995) is used to calculate V_i. $V_S = 1.75(q_t)^{0.627}$

For cohesionless soils, Mayne (2007) is used to calculate V_i. $V_S=277(q_t)^{0.13}(s'_{vo})^{0.27}$

2. If CPT depth is less than 100 feet, Boore (2004) is used to extrapolate to 100 feet.
$V_{S30} = [1.45-(0.015d)]V_{s(d)}$, where d = CPT depth (m), $V_{s(d)}$ = time averaged velocity (m/sec)

Depth (ft)	Layer Thickness (ft)	qt (tsf)	Soil Type	s'_{vo} (ksf)	Shear Wave Velocity (V_i) (m/sec)	d_i/V_i
0.54	0.54	15.05	3	0.06	167.32	0.00323
0.59	0.05	15.14	3	0.07	167.95	0.00030
0.63	0.04	15.06	3	0.07	167.39	0.00024
0.68	0.05	14.97	3	0.08	166.76	0.00030
0.72	0.04	15.14	3	0.09	167.95	0.00024
0.77	0.05	15.63	3	0.09	171.34	0.00029
0.82	0.05	16.91	3	0.1	180.01	0.00028
0.87	0.05	18.12	3	0.1	187.98	0.00027
0.92	0.05	13.35	3	0.11	155.21	0.00032
0.96	0.04	13.2	3	0.12	154.11	0.00026
1.01	0.05	13.12	3	0.12	153.53	0.00033
1.06	0.05	13.04	3	0.13	152.94	0.00033
1.11	0.05	13.03	3	0.13	152.87	0.00033
1.15	0.04	13.2	3	0.14	154.11	0.00026
1.2	0.05	14.9	4	0.15	166.28	0.00030
1.25	0.05	16.91	5	0.15	180.01	0.00028
1.3	0.05	20.06	5	0.16	200.36	0.00025
1.35	0.05	27.23	6	0.16	242.67	0.00021
1.4	0.05	32.15	6	0.17	269.30	0.00019
1.43	0.03	38.78	6	0.17	302.90	0.00010
1.47	0.04	44.42	6	0.18	329.82	0.00012
1.51	0.04	49.03	6	0.19	350.88	0.00011
1.55	0.04	51.53	6	0.19	361.99	0.00011
1.59	0.04	54.09	6	0.2	373.17	0.00011
1.63	0.04	55.56	6	0.2	379.50	0.00011
1.66	0.03	54.5	6	0.21	374.94	0.00008
1.69	0.03	54.32	6	0.21	374.16	0.00008
1.74	0.05	51.44	6	0.22	361.60	0.00014
1.79	0.05	48.72	6	0.22	349.49	0.00014
1.83	0.04	46.21	5	0.23	338.09	0.00012
1.88	0.05	43.7	5	0.24	326.45	0.00015
1.93	0.05	42.5	5	0.24	320.80	0.00016
1.97	0.04	37.08	5	0.25	294.50	0.00014
2.02	0.05	46.35	6	0.25	338.73	0.00015
2.08	0.06	52.41	6	0.26	365.86	0.00016

Project Name	SR-4 Expansion	Calc by:	Jim Wetenkamp
Project Number	111904.G01	Checked by:	Carl Henderson
Date	April 28, 2011		
CPT Number	CPT-11-011	Average small strain sheer wave velocity in meters per second	*Vs30= 277*
CPT Max Depth (ft)	50.01		
Total d_i/V_i	0.22026		

1. For cohesive soils, Mayne and Rix (1995) is used to calculate V_i. $V_S = 1.75(q_t)^{0.627}$
For cohesionless soils, Mayne (2007) is used to calculate V_i. $V_S=277(q_t)^{0.13}(s'_{vo})^{0.27}$

2. If CPT depth is less than 100 feet, Boore (2004) is used to extrapolate to 100 feet.
$V_{S30} = [1.45-(0.015d)]V_{s(d)}$, where d = CPT depth (m), $V_{s(d)}$ = time averaged velocity (m/sec)

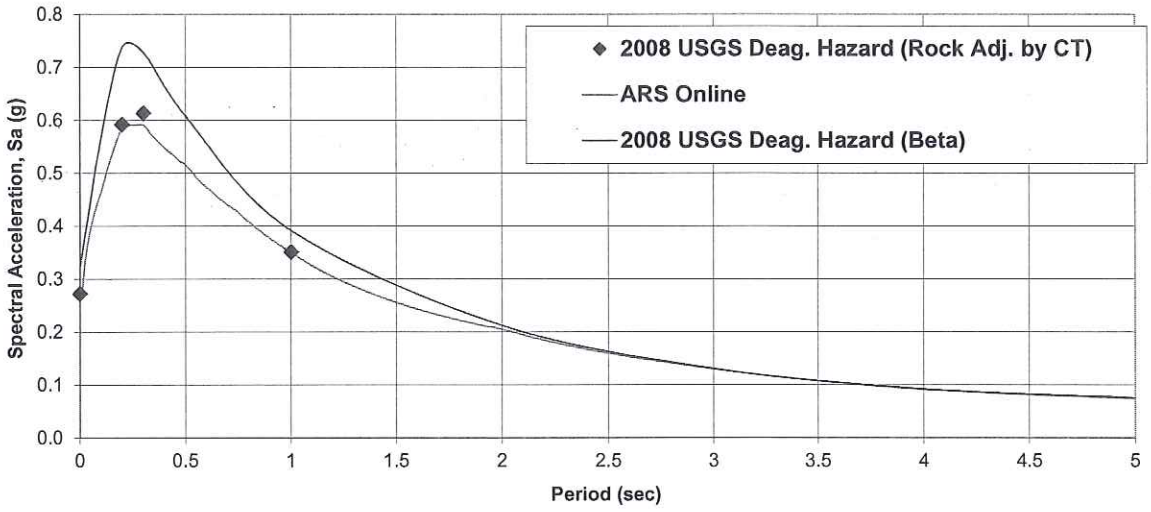
Depth (ft)	Layer Thickness (ft)	qt (tsf)	Soil Type	s'_{vo} (ksf)	Shear Wave Velocity (V_i) (m/sec)	d_i/V_i
0.52	0.52	69.44	5	0.06	436.45	0.00119
0.55	0.03	71.02	5	0.06	442.65	0.00007
0.59	0.04	66.07	5	0.07	423.04	0.00009
0.62	0.03	63.04	4	0.07	410.77	0.00007
0.67	0.05	60.65	11	0.08	77.50	0.00065
0.72	0.05	57.89	11	0.09	79.52	0.00063
0.76	0.04	55.41	3	0.09	378.85	0.00011
0.81	0.05	53.98	3	0.1	372.69	0.00013
0.87	0.06	52.37	3	0.11	365.68	0.00016
0.91	0.04	51.31	3	0.11	361.02	0.00011
0.97	0.06	59.26	11	0.12	86.20	0.00070
1.01	0.04	55.4	3	0.13	378.81	0.00011
1.05	0.04	57.12	3	0.13	386.14	0.00010
1.11	0.06	61.06	11	0.14	90.21	0.00067
1.16	0.05	56.28	4	0.15	382.57	0.00013
1.22	0.06	53.01	4	0.16	368.48	0.00016
1.28	0.06	55.75	4	0.16	380.31	0.00016
1.34	0.06	53.82	4	0.17	372.00	0.00016
1.39	0.05	50.99	4	0.18	359.61	0.00014
1.45	0.06	49.24	4	0.19	351.82	0.00017
1.5	0.05	53.79	5	0.19	371.87	0.00013
1.56	0.06	53.75	5	0.2	371.70	0.00016
1.61	0.05	55.73	5	0.21	380.22	0.00013
1.67	0.06	57.7	6	0.22	388.60	0.00015
1.73	0.06	57.06	6	0.22	385.89	0.00016
1.78	0.05	54.45	6	0.23	374.72	0.00013
1.84	0.06	52.58	6	0.24	366.60	0.00016
1.89	0.05	50.81	6	0.25	358.82	0.00014
1.93	0.04	46.95	6	0.25	341.47	0.00012
1.99	0.06	46.55	6	0.26	339.65	0.00018
2.04	0.05	45.07	6	0.27	332.83	0.00015
2.1	0.06	44.55	6	0.27	330.42	0.00018
2.16	0.06	42.19	6	0.28	319.34	0.00019
2.22	0.06	42.67	5	0.29	321.61	0.00019
2.27	0.05	40.81	5	0.3	312.75	0.00016

Development of Probabilistic and Deterministic Response Spectrums

Comparison spreadsheet of the 2008 USGS Probabilistic Seismic Hazard Data and ARS Online Probabilistic Data (unlock spreadsheet "shmi")
Spectral Accelerations Points from USGS Website at http://earthquake.usgs.gov/research/hazmaps/products_data/2008/data/

*** Note: This spreadsheet uses the given latitude and longitude data provided by the user to estimate spectral acceleration values with a probability of exceedence 5% in 50 yrs (or 975 yr return period). The four spectral acceleration data points plotted on the graph are from the USGS website and are based on a 0.05 degree grid. Basic interpolation is used to estimate intermediate values inside each grid. Raw Data points are provided in the tabs of this spreadsheet. Corner grid spectral acceleration data are shown in the "calculation" tab.**

Input Site Information	
Latitude	Longitude
37.9400	-121.3173
V_{S30} (m/s) =	249
Near Fault Factor, Derived from USGS Deagg. Dist (km) =	29.4
$Z_{1.0}$ (m) =	0
$Z_{2.5}$ (km) =	0



Place ARS Online Probabilistic Data Here "Paste"

T (sec)	Base Spectrum S(a)	Basin Factor	Near Fault Factor	Final Adj. Spectrum S(a)
0.01	0.273	1	1	0.273
0.02	0.321	1	1	0.321
0.022	0.328	1	1	0.328
0.025	0.338	1	1	0.338
0.029	0.349	1	1	0.349
0.03	0.352	1	1	0.352
0.032	0.357	1	1	0.357
0.035	0.365	1	1	0.365
0.036	0.367	1	1	0.367
0.04	0.376	1	1	0.376
0.042	0.381	1	1	0.381
0.044	0.385	1	1	0.385
0.045	0.387	1	1	0.387
0.046	0.389	1	1	0.389
0.048	0.392	1	1	0.392
0.05	0.396	1	1	0.396
0.055	0.405	1	1	0.405
0.06	0.413	1	1	0.413
0.065	0.421	1	1	0.421
0.067	0.424	1	1	0.424
0.07	0.428	1	1	0.428
0.075	0.435	1	1	0.435
0.08	0.442	1	1	0.442
0.085	0.448	1	1	0.448
0.09	0.454	1	1	0.454
0.095	0.46	1	1	0.46
0.1	0.465	1	1	0.465
0.11	0.48	1	1	0.48
0.12	0.495	1	1	0.495
0.13	0.509	1	1	0.509
0.133	0.513	1	1	0.513
0.14	0.522	1	1	0.522
0.15	0.534	1	1	0.534
0.16	0.546	1	1	0.546
0.17	0.558	1	1	0.558
0.18	0.569	1	1	0.569
0.19	0.58	1	1	0.58
0.2	0.59	1	1	0.59
0.22	0.59	1	1	0.59
0.24	0.591	1	1	0.591
0.25	0.591	1	1	0.591

Analysis of ARS Online Results vs USGS Deaggregation Hazard (Adj. By CT)

Period (sec)	USGS Interpolated Spectral Accel.	Adj. for Near Fault Effect	Adj. for Soil Amplification	Adj. For Basin Effect	Final Adj. USGS Spec Accel	ARS Online Final Adj. Spect. Accel.	% Difference (bet. USGS & ARS Online)
0	0.243	1.000	1.122	1.000	0.272	0.273	-0.3%
0.2	0.586	1.000	1.011	1.000	0.592	0.59	0.4%
0.3	0.485	1.000	1.265	1.000	0.614	0.592	3.5%
1	0.180	1.000	1.945	1.000	0.351	0.35	0.3%

Max % Difference = 3.5%

USGS Deaggregation Hazard (Beta) with Near Field and Basin Factors

Period (sec)	INPUT USGS Deagg. Spec Accel	Adj. for Near Fault Effect	Adj. For Basin Effect	Final Adj. USGS Deagg Spec Accel	ARS Online Final Adj. Spect. Accel.	% Difference (bet. USGS & ARS Online)
0	0.3244	1.000	1.000	0.324	0.273	15.8%
0.1	0.5696	1.000	1.000	0.570	0.465	17.4%
0.2	0.7382	1.000	1.000	0.738	0.59	20.1%
0.3	0.7281	1.000	1.000	0.728	0.592	18.7%
0.5	0.6083	1.000	1.000	0.608	0.516	15.2%
1	0.3919	1.000	1.000	0.392	0.35	10.7%
2	0.2124	1.000	1.000	0.212	0.205	3.5%
3	0.13101	1.000	1.000	0.131	0.13	0.8%
4	0.09244	1.000	1.000	0.092	0.091	1.6%
5	0.07534	1.000	1.000	0.075	0.074	1.8%

Max % Difference = 15.2%

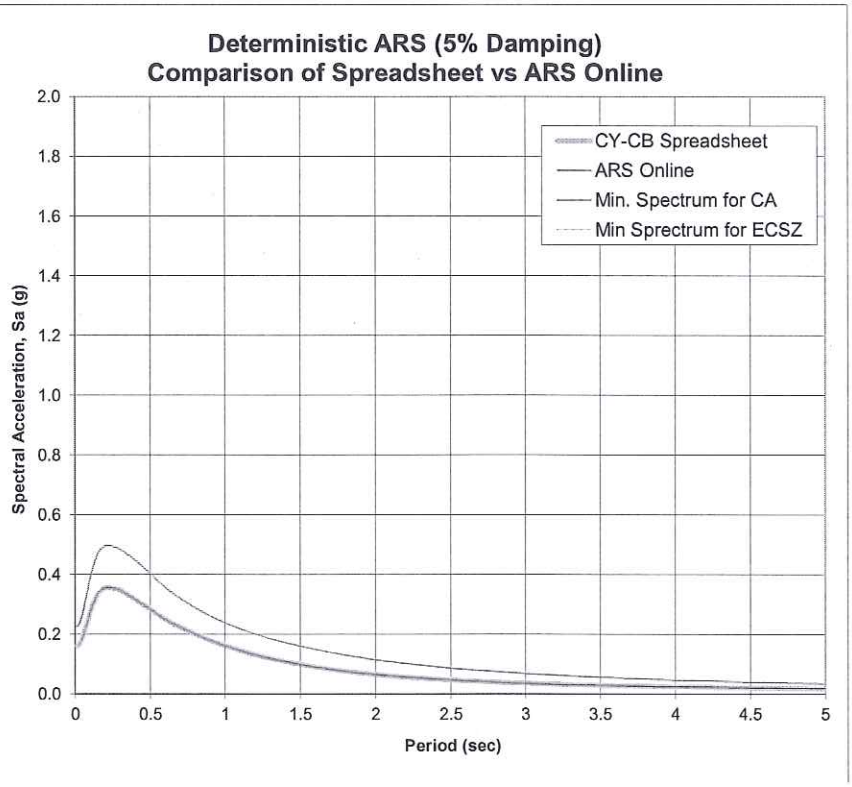
Comparison of ARS Curves
(unlock sheet with "shmi")

Model Inputs

Fault		
Magnitude	6.7	(5 to 8.5)
F_{RV}	1	(input 1 = Rev)
F_{NM}	0	(input 1 = Normal)
Dip (degree)	15	(0 to 90)
Z_{TOR} (km)	7	
Distance		
R_{RUP} (km)	30.1	
R_{JB} (km)	29.3	
R_x (km)	29.1	
Hanging Wall?	☐ Yes?	
Near-Field Factor?	☐ Yes?	
Site		
V_{S30} (m/sec)	249	(270 to 1500 m/s)
$Z_{1.0}$ (m)	0	(0 - No Basin)
$Z_{2.5}$ (km)	0	(0 - No Basin)
No. Cal. Basin?	☐ Yes ?	(Check only for sites located within a Basin)
So. Cal. Basin?	☐ Yes ?	

Analysis

ARS Online vs CY-CB Spreadsheet Results
MAX. % Diff. = 2%



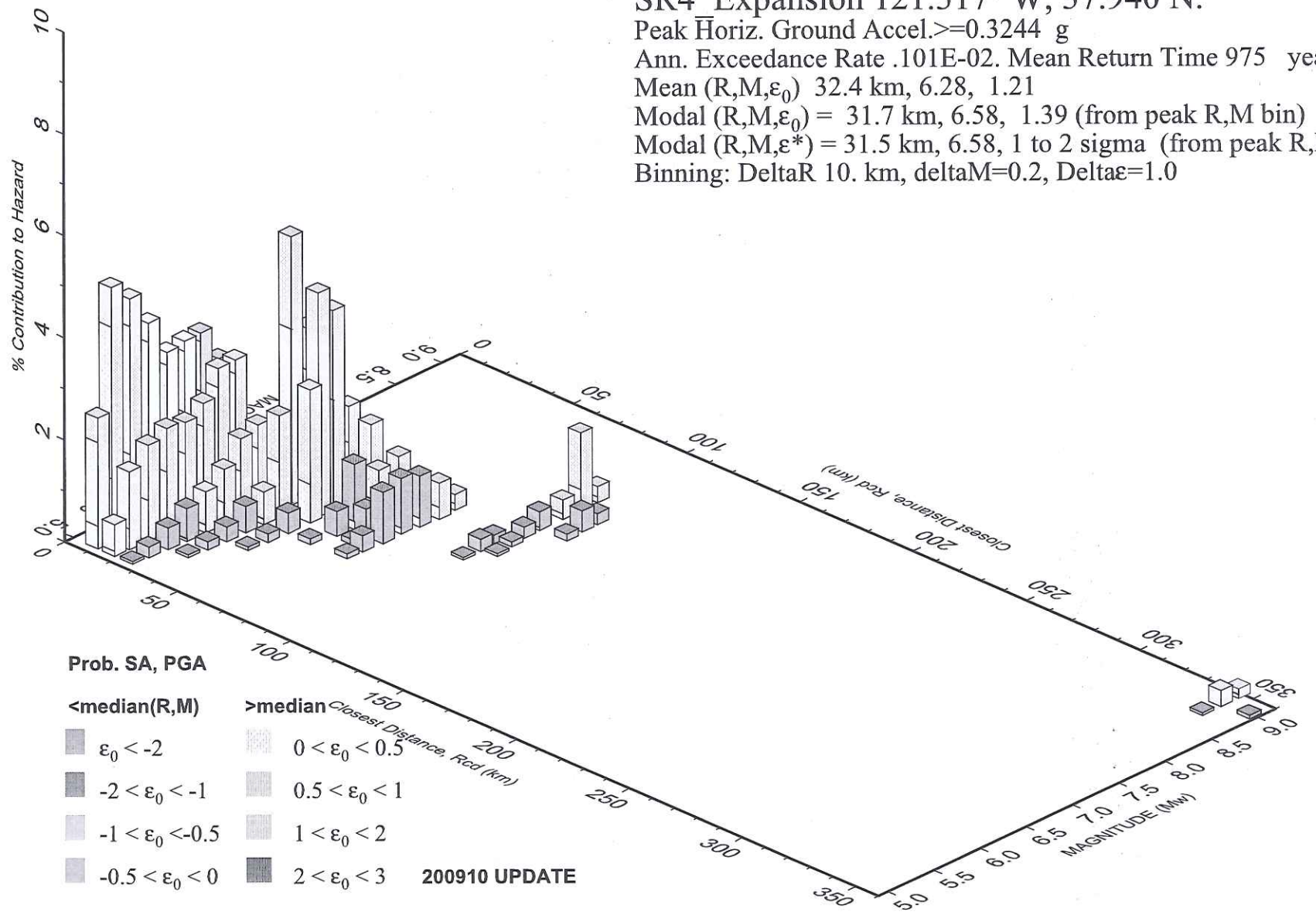
CY-CB Spreadsheet Results	
T (sec)	CB-CY S(a)
0.010	0.15919
0.020	0.16067
0.022	0.16203
0.025	0.16389
0.029	0.16616
0.030	0.16684
0.032	0.16895
0.035	0.17199
0.036	0.17301
0.040	0.17705
0.042	0.17922
0.044	0.18144
0.045	0.18261
0.046	0.18374
0.048	0.18606
0.050	0.18834
0.055	0.19643
0.060	0.20462
0.065	0.21285
0.067	0.21615
0.070	0.22109
0.075	0.22927
0.080	0.23822
0.085	0.24701
0.090	0.25574
0.095	0.26432
0.100	0.27266
0.110	0.28741
0.120	0.30105
0.130	0.31338
0.133	0.31676
0.140	0.32424
0.150	0.33401
0.160	0.33974
0.170	0.34455
0.180	0.34862

Place ARS Online Deterministic Data Here "Paste"					
T (sec)	Base S(a)	Basin Factor	Near Fault Factor	Final Adj. S(a)	Diff. (%)
0.01	0.159	1	1	0.159	0%
0.02	0.161	1	1	0.161	0%
0.022	0.162	1	1	0.162	0%
0.025	0.164	1	1	0.164	0%
0.029	0.166	1	1	0.166	0%
0.03	0.167	1	1	0.167	0%
0.032	0.169	1	1	0.169	0%
0.035	0.172	1	1	0.172	0%
0.036	0.173	1	1	0.173	0%
0.04	0.177	1	1	0.177	0%
0.042	0.179	1	1	0.179	0%
0.044	0.181	1	1	0.181	0%
0.045	0.183	1	1	0.183	0%
0.046	0.184	1	1	0.184	0%
0.048	0.186	1	1	0.186	0%
0.05	0.188	1	1	0.188	0%
0.055	0.196	1	1	0.196	0%
0.06	0.205	1	1	0.205	0%
0.065	0.213	1	1	0.213	0%
0.067	0.216	1	1	0.216	0%
0.07	0.221	1	1	0.221	0%
0.075	0.229	1	1	0.229	0%
0.08	0.238	1	1	0.238	0%
0.085	0.247	1	1	0.247	0%
0.09	0.256	1	1	0.256	0%
0.095	0.264	1	1	0.264	0%
0.1	0.273	1	1	0.273	0%
0.11	0.287	1	1	0.287	0%
0.12	0.301	1	1	0.301	0%
0.13	0.313	1	1	0.313	0%
0.133	0.317	1	1	0.317	0%
0.14	0.324	1	1	0.324	0%
0.15	0.334	1	1	0.334	0%
0.16	0.34	1	1	0.34	0%
0.17	0.345	1	1	0.345	0%
0.18	0.349	1	1	0.349	0%

For Comparsion Plots of Min. Sprectra, Paste Special into Cells			
Min. Spectrum for CA		Min Sprectrum for ECSZ	
T (sec)	S (a)	T (sec)	S (a)
0.01	0.225		
0.02	0.228		
0.022	0.23		
0.025	0.233		
0.029	0.237		
0.03	0.238		
0.032	0.241		
0.035	0.246		
0.036	0.248		
0.04	0.254		
0.042	0.257		
0.044	0.26		
0.045	0.262		
0.046	0.264		
0.048	0.267		
0.05	0.271		
0.055	0.283		
0.06	0.295		
0.065	0.307		
0.067	0.311		
0.07	0.319		
0.075	0.331		
0.08	0.343		
0.085	0.355		
0.09	0.368		
0.095	0.379		
0.1	0.391		
0.11	0.41		
0.12	0.428		
0.13	0.444		
0.133	0.448		
0.14	0.458		
0.15	0.471		
0.16	0.478		
0.17	0.484		
0.18	0.489		

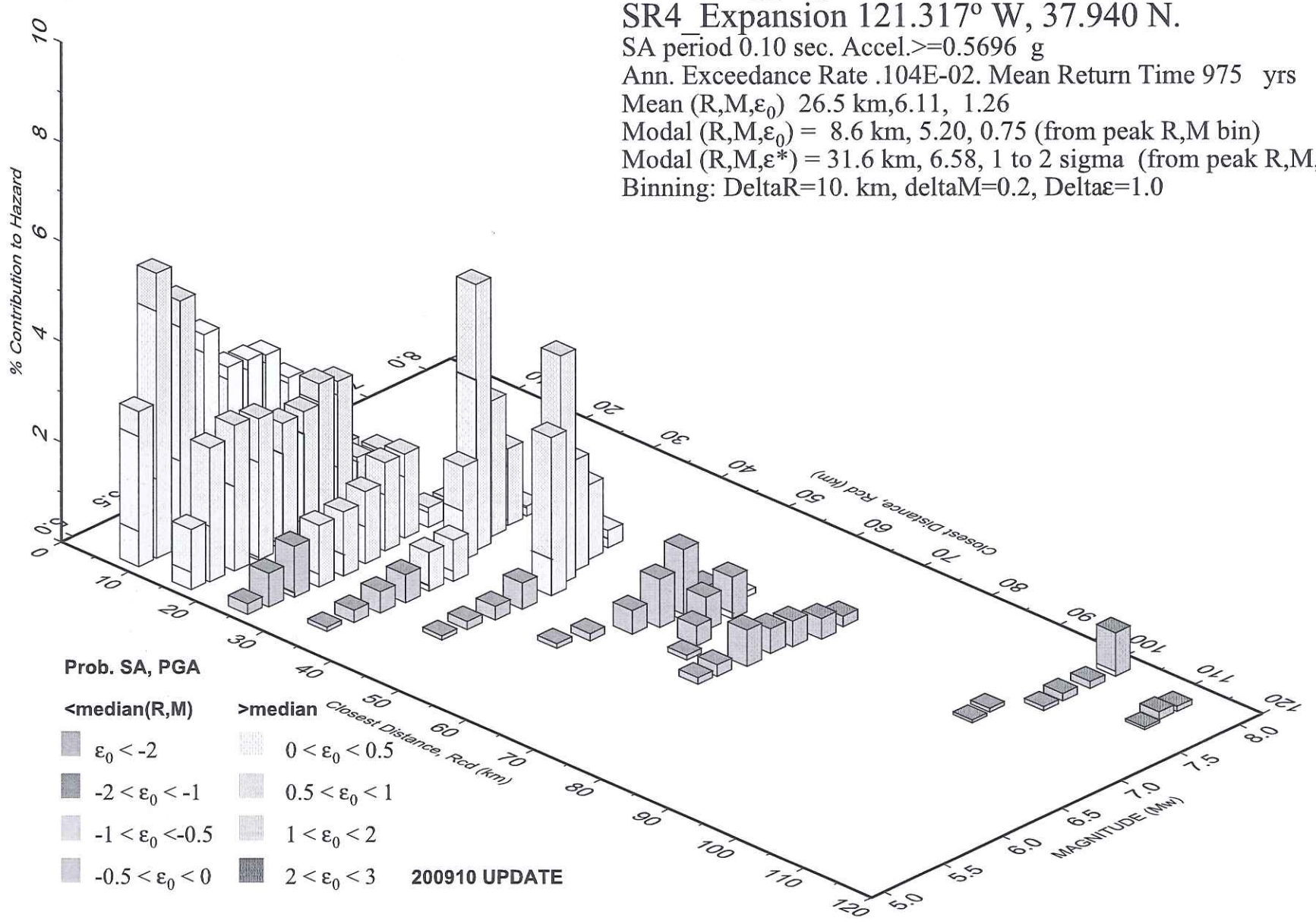
PSH Deaggregation on NEHRP D soil
SR4 Expansion 121.317° W, 37.940 N.

Peak Horiz. Ground Accel.>=0.3244 g
Ann. Exceedance Rate .101E-02. Mean Return Time 975 years
Mean (R,M,ε_0) 32.4 km, 6.28, 1.21
Modal (R,M,ε_0) = 31.7 km, 6.58, 1.39 (from peak R,M bin)
Modal (R,M,ε*) = 31.5 km, 6.58, 1 to 2 sigma (from peak R,M,ε bin)
Binning: DeltaR 10. km, deltaM=0.2, Deltaε=1.0



PSH Deaggregation on NEHRP D soil
SR4_Expansion 121.317° W, 37.940 N.

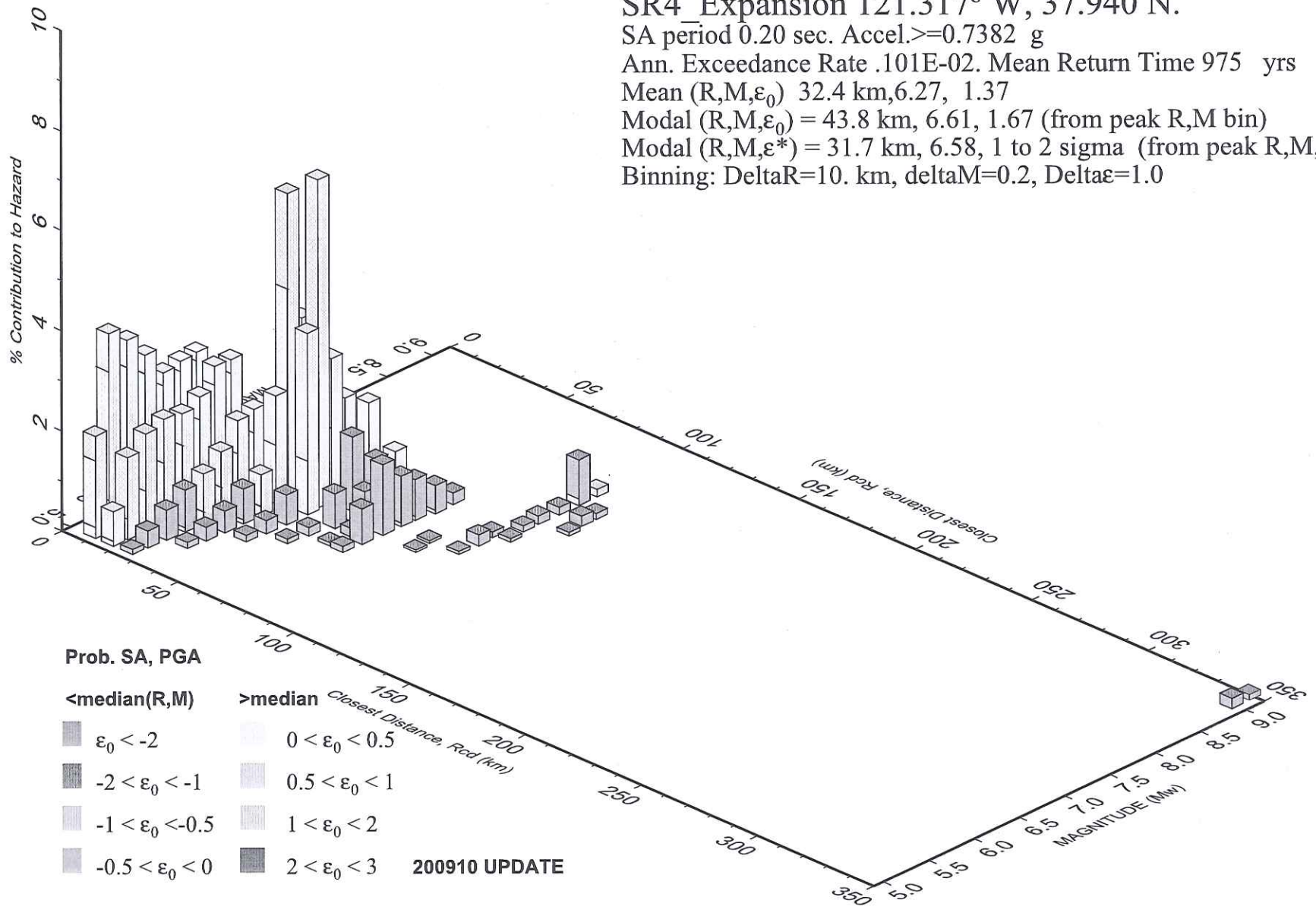
SA period 0.10 sec. Accel.>=0.5696 g
Ann. Exceedance Rate .104E-02. Mean Return Time 975 yrs
Mean (R,M,ε_0) 26.5 km,6.11, 1.26
Modal (R,M,ε_0) = 8.6 km, 5.20, 0.75 (from peak R,M bin)
Modal (R,M,ε^*) = 31.6 km, 6.58, 1 to 2 sigma (from peak R,M,ε bin)
Binning: DeltaR=10. km, deltaM=0.2, Deltaε=1.0



GMT 2011 Apr 29 20:37:27 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 249. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

PSH Deaggregation on NEHRP D soil
SR4_Expansion 121.317° W, 37.940 N.

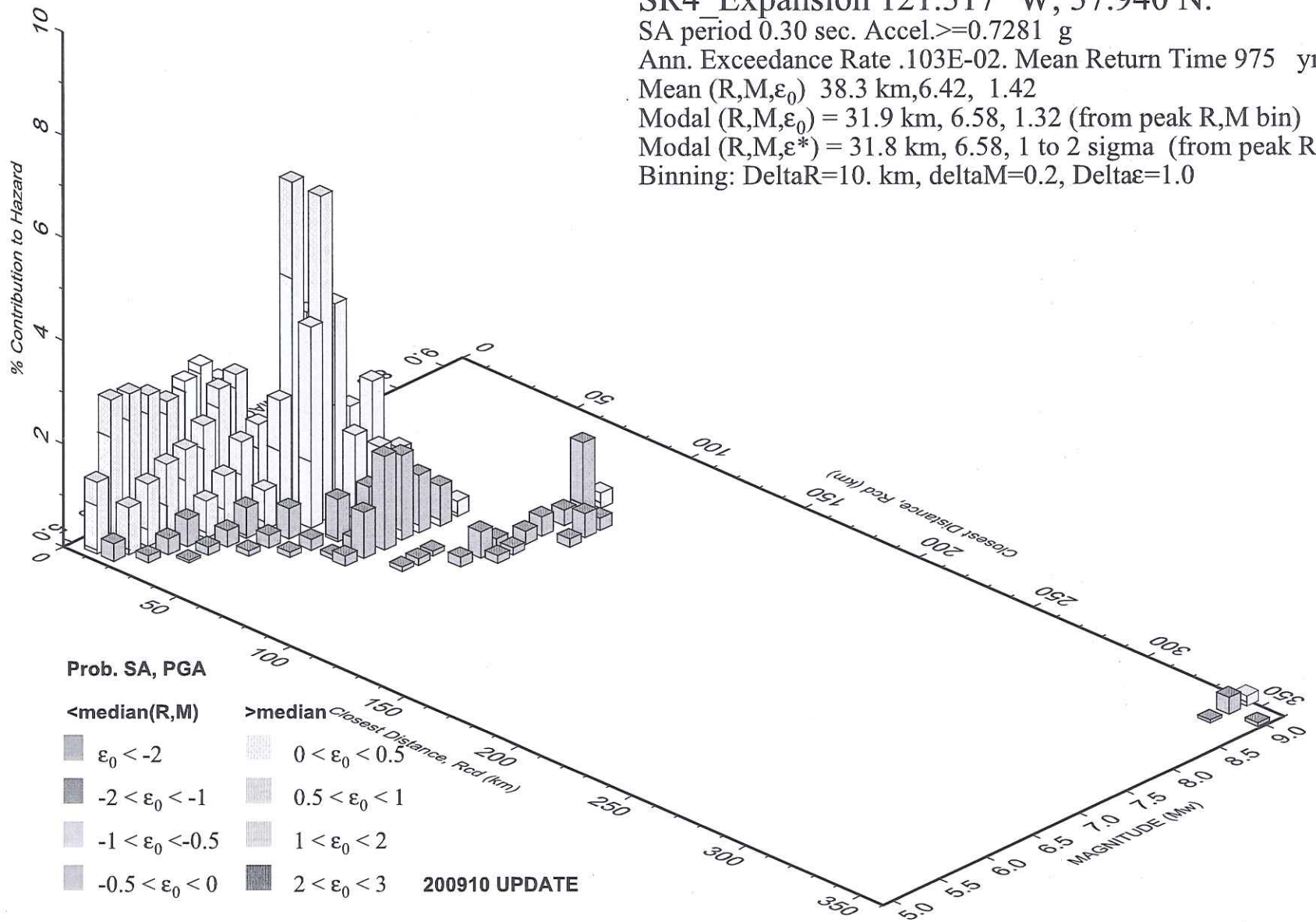
SA period 0.20 sec. Accel.>=0.7382 g
Ann. Exceedance Rate .101E-02. Mean Return Time 975 yrs
Mean (R,M,ε_0) 32.4 km,6.27, 1.37
Modal (R,M,ε_0) = 43.8 km, 6.61, 1.67 (from peak R,M bin)
Modal (R,M,ε^*) = 31.7 km, 6.58, 1 to 2 sigma (from peak R,M,ε bin)
Binning: DeltaR=10. km, deltaM=0.2, Deltaε=1.0



GMT 2011 Apr 29 20:38:07 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 249. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

PSH Deaggregation on NEHRP D soil
SR4_Expansion 121.317° W, 37.940 N.

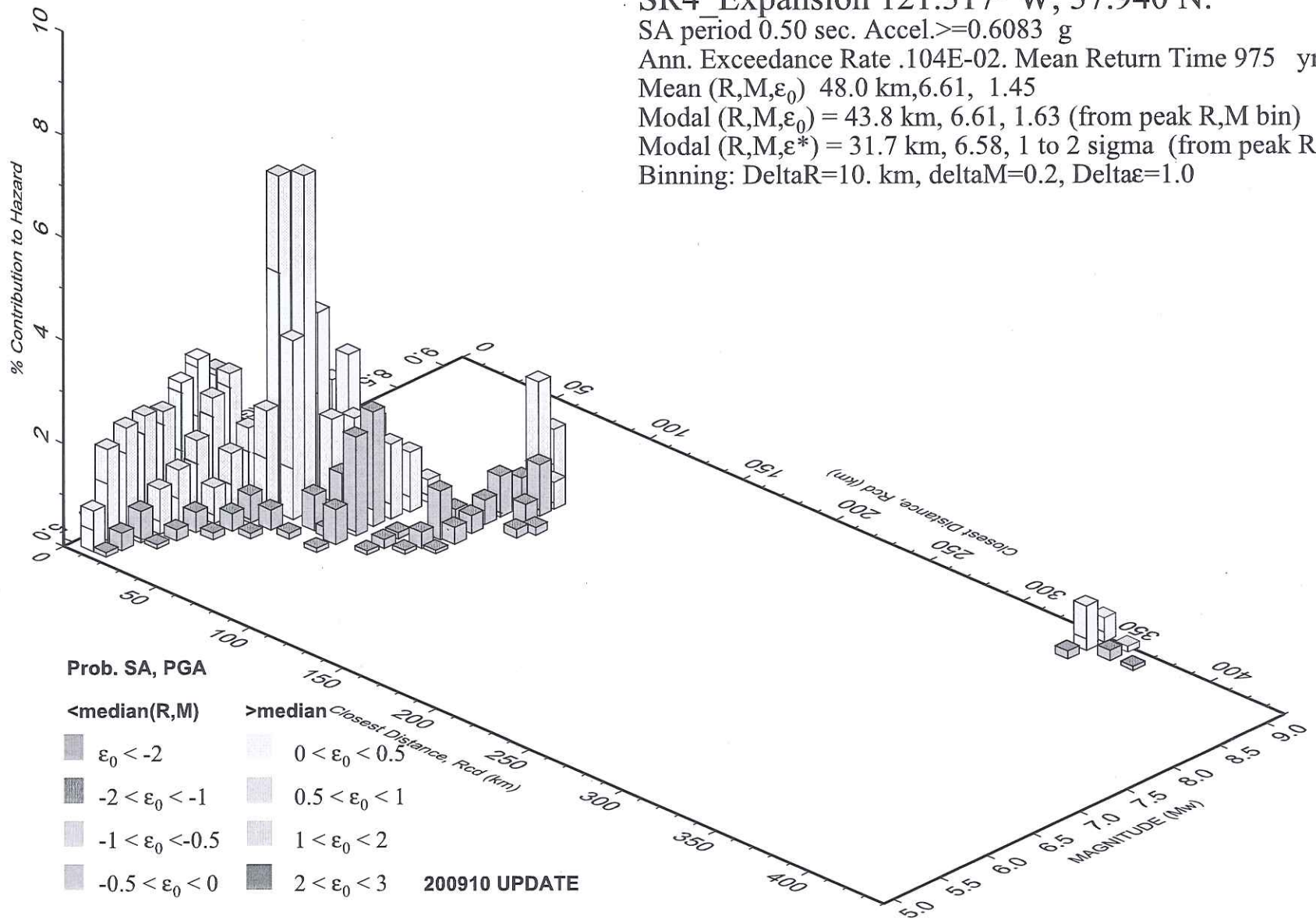
SA period 0.30 sec. Accel.>=0.7281 g
Ann. Exceedance Rate .103E-02. Mean Return Time 975 yrs
Mean (R,M,ε_0) 38.3 km,6.42, 1.42
Modal (R,M,ε_0) = 31.9 km, 6.58, 1.32 (from peak R,M bin)
Modal (R,M,ε^*) = 31.8 km, 6.58, 1 to 2 sigma (from peak R,M,ε bin)
Binning: DeltaR=10. km, deltaM=0.2, Deltaε=1.0



GMT 2011 Apr 29 20:38:47 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 249. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

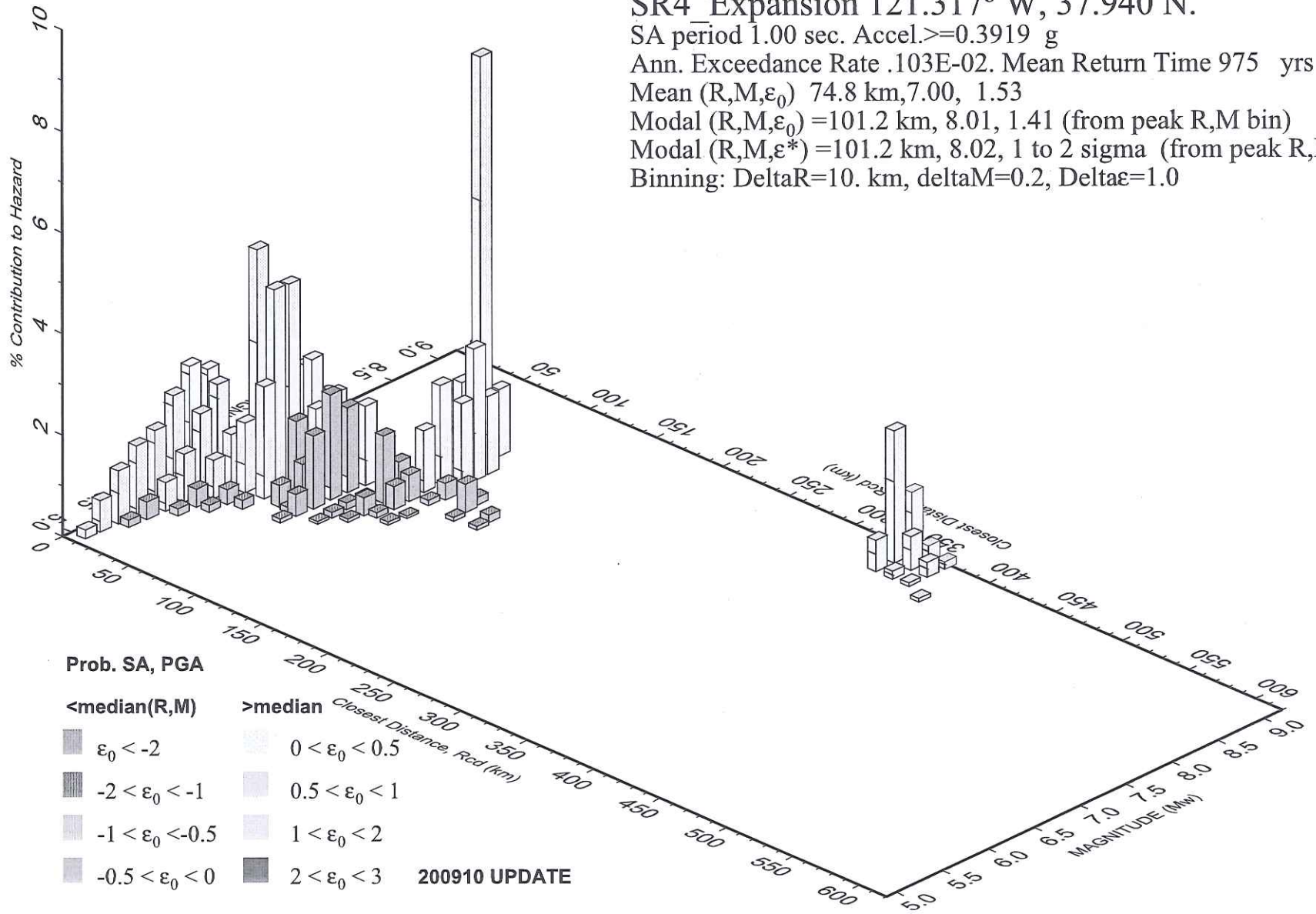
PSH Deaggregation on NEHRP D soil
SR4_Expansion 121.317° W, 37.940 N.

SA period 0.50 sec. Accel.>=0.6083 g
Ann. Exceedance Rate .104E-02. Mean Return Time 975 yrs
Mean (R,M,ε_0) 48.0 km,6.61, 1.45
Modal (R,M,ε_0) = 43.8 km, 6.61, 1.63 (from peak R,M bin)
Modal (R,M,ε^*) = 31.7 km, 6.58, 1 to 2 sigma (from peak R,M,ε bin)
Binning: DeltaR=10. km, deltaM=0.2, Deltaε=1.0



PSH Deaggregation on NEHRP D soil
SR4_Expansion 121.317° W, 37.940 N.

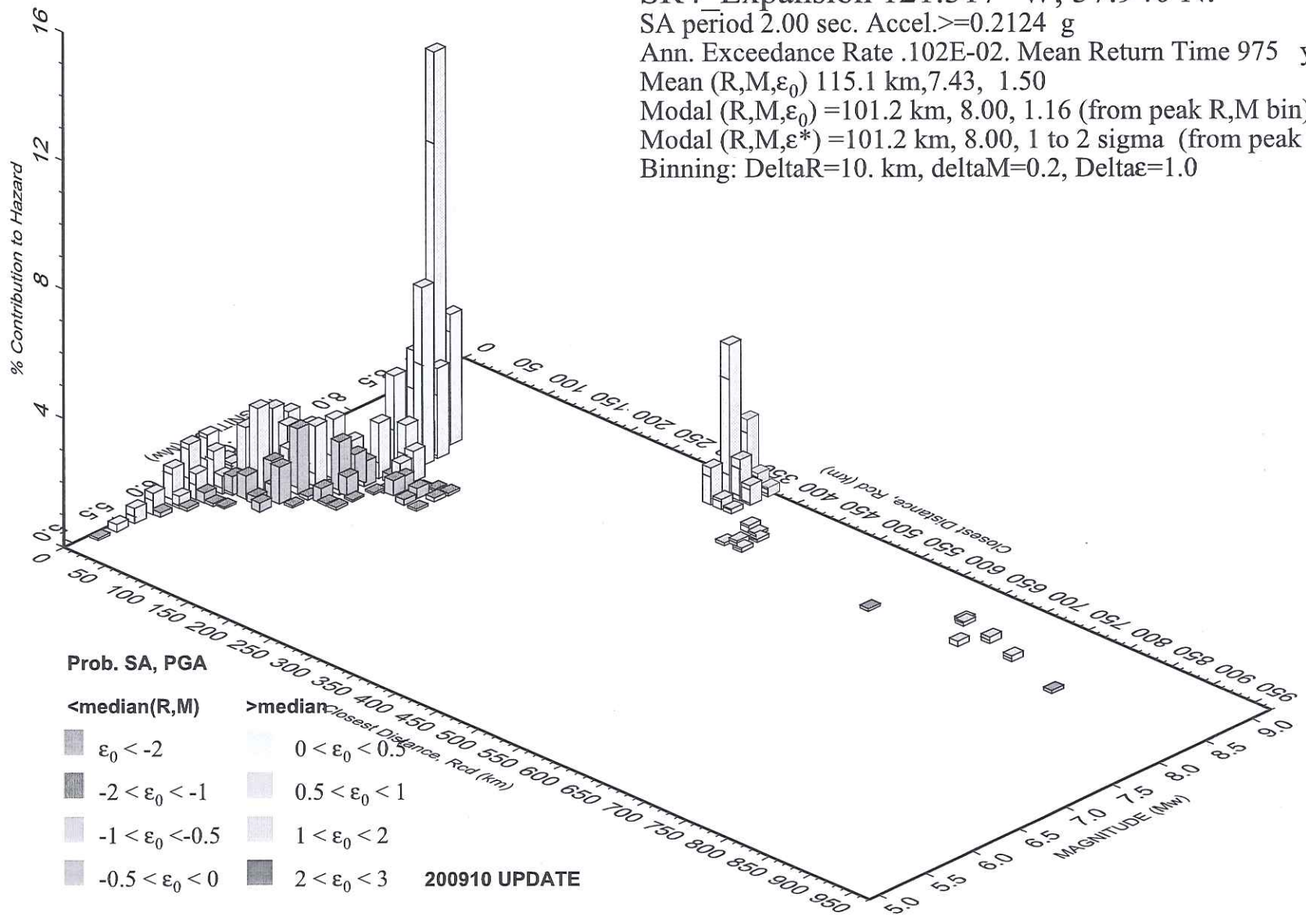
SA period 1.00 sec. Accel.>=0.3919 g
Ann. Exceedance Rate .103E-02. Mean Return Time 975 yrs
Mean (R,M,ε_0) 74.8 km,7.00, 1.53
Modal (R,M,ε_0) =101.2 km, 8.01, 1.41 (from peak R,M bin)
Modal (R,M,ε^*) =101.2 km, 8.02, 1 to 2 sigma (from peak R,M,ε bin)
Binning: DeltaR=10. km, deltaM=0.2, Deltaε=1.0

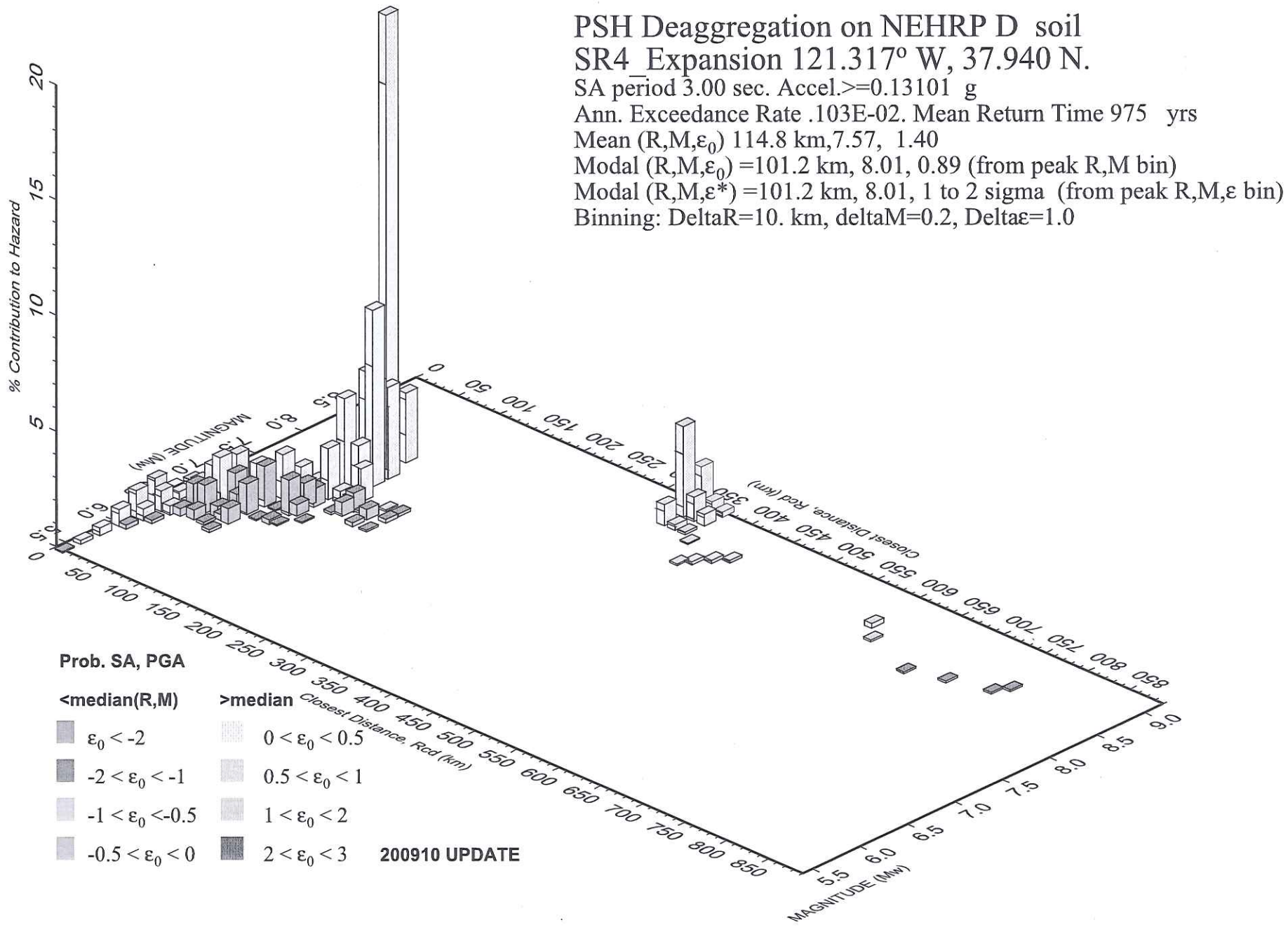


GMT 2011 Apr 29 20:39:52 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 249. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

PSH Deaggregation on NEHRP D soil
SR4_Expansion 121.317° W, 37.940 N.

SA period 2.00 sec. Accel.>=0.2124 g
Ann. Exceedance Rate .102E-02. Mean Return Time 975 yrs
Mean (R,M,ε_0) 115.1 km,7.43, 1.50
Modal (R,M,ε_0) =101.2 km, 8.00, 1.16 (from peak R,M bin)
Modal (R,M,ε*) =101.2 km, 8.00, 1 to 2 sigma (from peak R,M,ε bin)
Binning: DeltaR=10. km, deltaM=0.2, Deltaε=1.0

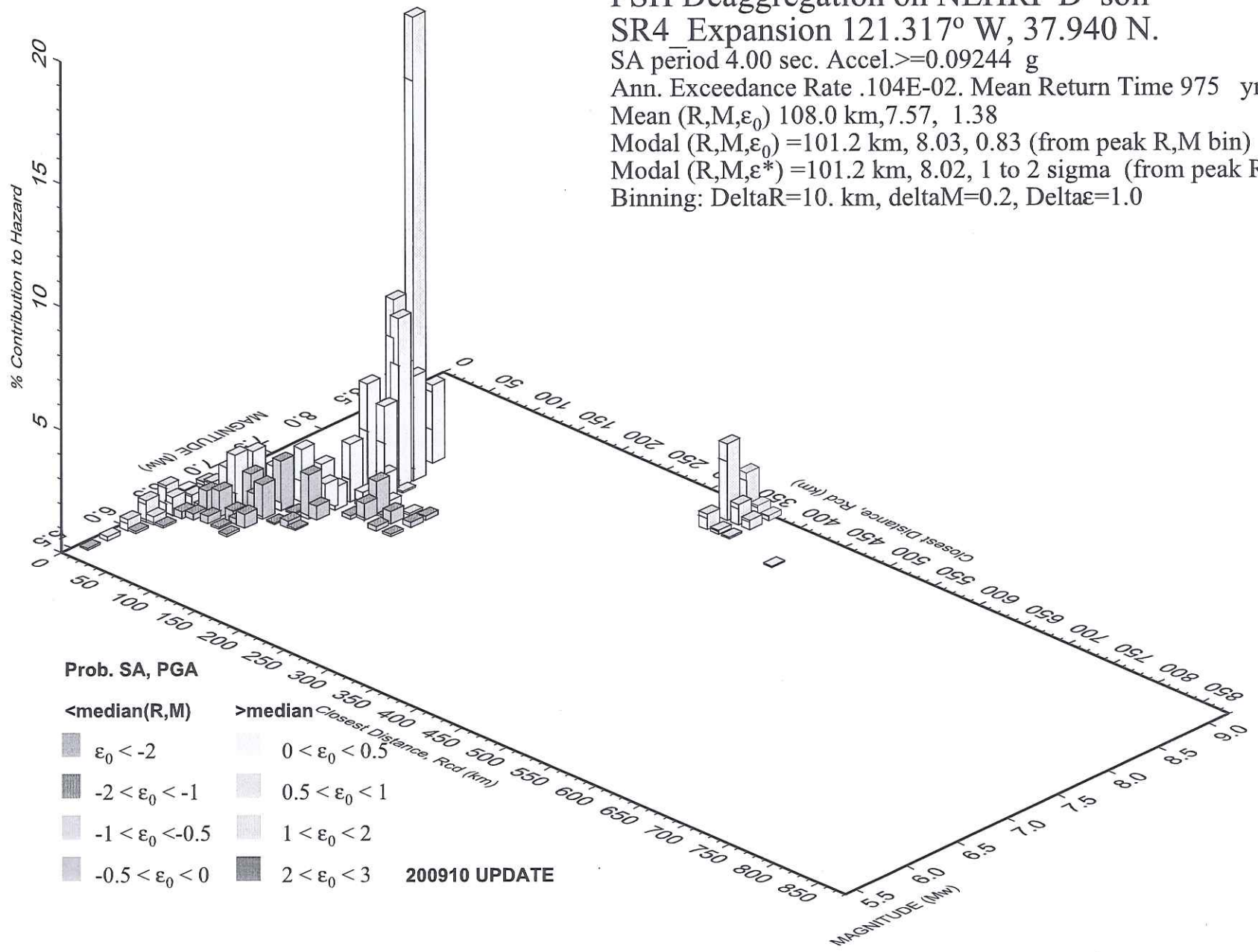


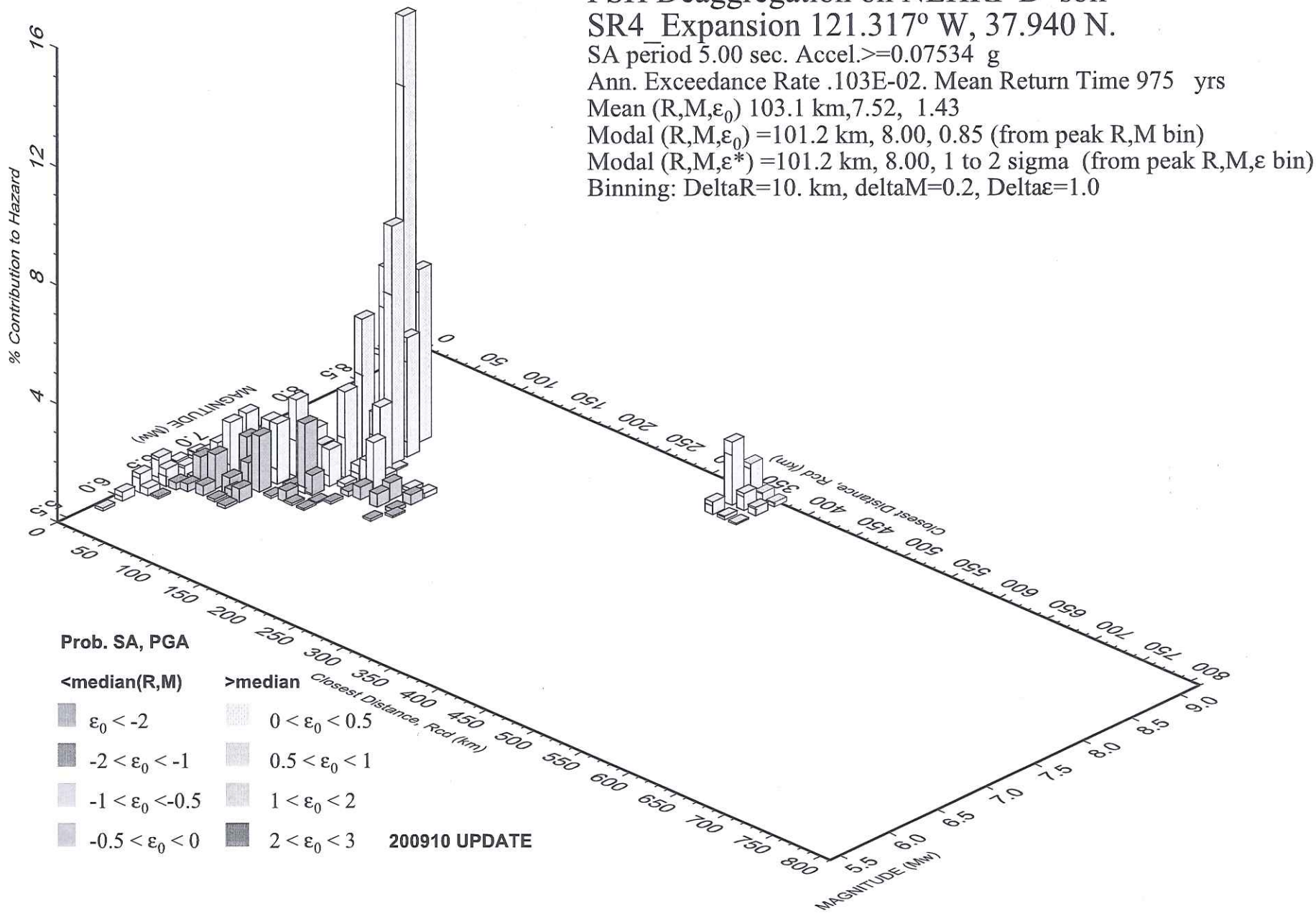


GMT 2011 Apr 29 20:40:57 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 249. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

PSH Deaggregation on NEHRP D soil
SR4_Expansion 121.317° W, 37.940 N.

SA period 4.00 sec. Accel.>=0.09244 g
Ann. Exceedance Rate .104E-02. Mean Return Time 975 yrs
Mean (R,M,ε_0) 108.0 km,7.57, 1.38
Modal (R,M,ε_0) =101.2 km, 8.03, 0.83 (from peak R,M bin)
Modal (R,M,ε*) =101.2 km, 8.02, 1 to 2 sigma (from peak R,M,ε bin)
Binning: DeltaR=10. km, deltaM=0.2, Deltaε=1.0





GMT 2011 Apr 29 20:41:56 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 249. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

APPENDIX G
LIQUEFACTION ANALYSIS RESULTS

LIST OF ATTACHMENTS

The following are attached and complete this appendix.

Example of Input Data for Liquefaction Assessment Spreadsheet
Example of Output Data from Liquefaction Assessment Spreadsheet
Liquefaction Assessment Results/Graphs

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Example of Input Data for Liquefaction Assessment Spreadsheet

STEP 1:

Project Name: Crosstown Freeway
Project No.: 111904.G01
Project Location: Stockton, CA

Analyzed by: Hamid Parsa Date: 4/28/2011
Checked by: Carl henderson Date:

STEP 2:

Mw= 6.7 5.5≤Mw≤8.5
PGA= 0.32

Field Ground Water Depth (ft)= 11.3
Design Ground Water Depth (ft)= 11

Existing Ground Surface Elevation (ft)= 0
New Fill Height (ft)= 0
Unit Weight of New Fill (pcf)= 120

STEP 3:

Probability to Liquefy (PL) (%)= 15 for Seed et al. (2003) approach

Factor of Safety (FOS) for Liquefaction = 1.00
Factor of Safety (FOS) for Settlement Calculation = 1.30

STEP 4:

CPT I.D.: CPT-1

Depth ft	Depth (m)	q_c T/ft^2	f_s T/ft^2	u_2 lb/in^2	R_f %	q_t T/ft^2	σ_{vo} T/ft^2	SBT	S_u(qc) T/ft^2	SPT N_{60} blows/ft	UW lb/ft^3
0.57		157.14	4.3		2.7			7			135
0.64		153.52	4.97		3.2			6			135
0.71		153.86	4.76		3.1			7			135
0.79		133.07	4.62		3.5			6			135
0.87		117.91	4.34		3.7			6			135
0.94		103.46	4.51		4.4			11			135
1.02		114.58	4.63		4			11			135
1.1		98.93	4.37		4.4			11			135
1.18		86.15	3.72		4.3			11			135
1.27		74.8	3.86		5.2			11			135
1.36		58.67	4.42		7.5			11			135
1.43		90.12	4.84		5.4			11			135
1.52		149.54	5.62		3.8			12			140
1.59		155.54	6.01		3.9			12			140
1.68		158.17	4.8		3			7			135
1.75		146.99	4.38		3			7			135
1.84		147.59	4.13		2.8			7			135
1.91		145	3.77		2.6			7			135
1.97		137.15	3.71		2.7			7			135
2.05		117.48	3.72		3.2			6			135
2.13		96.67	3.78		3.9			5			135
2.21		69.74	3.56		5.1			11			135
2.3		51.86	3.28		6.3			3			135
2.38		45.38	3.03		6.7			3			135
2.46		38.34	2.74		7.1			3			135
2.54		34.06	2.46		7.2			3			135
2.62		29.1	2.27		7.8			3			135
2.7		25.22	2.08		8.2			3			135
2.78		25.37	1.97		7.8			3			135
2.87		24.23	1.83		7.6			3			135
2.95		21.75	1.69		7.8			3			135
3.03		20.93	1.59		7.6			3			135
3.11		21.1	1.62		7.7			3			135
3.2		21.89	1.6		7.3			3			135
3.28		24.55	1.54		6.3			3			135
3.37		25.04	1.54		6.2			3			135
3.45		19.96	1.51		7.6			3			135
3.53		17.38	1.41		8.1			3			135
3.61		15.93	1.28		8.1			3			135
3.7		17.55	1.21		6.9			3			135
3.78		15.93	1.18		7.4			3			135
3.87		14.8	1.16		7.9			3			135
3.95		15.93	1.16		7.3			3			135
4.04		15.86	1.15		7.3			3			135
4.12		18.84	1.13		6			3			135
4.21		18.04	1.14		6.3			3			135
4.3		18.59	1.14		6.1			3			135
4.39		18.44	1.12		6.1			3			135
4.47		16.83	1.12		6.7			3			135
4.56		15.61	1.12		7.2			3			135
4.64		14.64	1.1		7.5			3			135
4.74		14.82	1.08		7.3			3			125
4.82		16.19	1.08		6.7			3			135
4.91		14.65	1.04		7.1			3			125
5		14.65	0.95		6.5			3			125
5.09		13.93	0.94		6.8			3			125
5.17		15.22	0.96		6.3			3			125
5.33		15.95	0.96		6			3			125
5.42		13.68	0.93		6.8			3			125
5.5		14.72	0.92		6.3			3			125
5.6		13.93	0.99		7.1			3			125
5.68		16.5	0.96		5.8			3			125
5.77		14.48	0.9		6.2			3			125
5.85		10.86	0.87		8			3			125
5.94		10.78	0.86		8			3			125
6.03		10.21	0.79		7.7			3			125
6.12		9.73	0.78		8			3			125

6.2	9.65	0.77	8	3	125
6.29	8.28	0.73	8.8	3	125
6.38	7.95	0.73	9.2	3	125
6.47	9.89	0.78	7.9	3	125
6.55	11.5	0.77	6.7	3	125
6.65	12.71	0.82	6.4	3	125
6.73	20.7	0.86	4.2	4	135
6.82	22.53	0.87	3.9	4	135
6.9	24.41	0.88	3.6	5	135
6.99	21.74	0.86	4	4	135
7.08	20.21	0.87	4.3	4	135
7.17	18.04	0.85	4.7	3	125
7.25	14.56	0.83	5.7	3	125
7.35	12.64	0.8	6.4	3	125
7.43	13.44	0.76	5.7	3	125
7.52	13.76	0.68	4.9	3	125
7.6	12.55	0.62	5	3	125
7.69	9.89	0.6	6.1	3	125
7.78	9.42	0.57	6	3	125
7.86	8.84	0.51	5.8	3	115
7.96	9.9	0.54	5.5	3	125
8.04	11.35	0.67	5.9	3	125
8.13	19.33	0.72	3.7	4	125
8.22	14.81	0.73	4.9	3	125
8.29	10.68	0.73	6.9	3	125
8.33	11.1	0.71	6.4	3	125
8.41	10.62	0.63	5.9	3	125
8.49	13.2	0.53	4	3	125
8.58	14.32	0.5	3.5	4	125
8.66	15.21	0.56	3.7	4	125
8.76	16.43	0.61	3.7	4	125
8.84	12.95	0.57	4.4	3	125
8.93	9.32	0.51	5.5	3	115
9.02	7.31	0.46	6.3	3	115
9.11	5.86	0.41	6.9	3	115
9.19	5.21	0.36	6.9	3	105
9.27	5.13	0.4	7.7	3	105
9.37	6.26	0.43	6.8	3	115
9.45	7.55	0.47	6.2	3	115
9.55	10.86	0.5	4.6	3	125
9.64	12.39	0.59	4.7	3	125
9.72	12.06	0.77	6.4	3	125
9.82	20.39	1.03	5	3	135
9.9	30.99	1.21	3.9	5	135
9.99	35.44	1.45	4.1	4	135
10.07	43.91	1.69	3.8	5	135
10.15	53.74	1.86	3.5	5	135
10.24	55.98	2.04	3.7	5	135
10.33	51.09	2.25	4.4	4	135
10.42	47.71	2.44	5.1	3	135
10.5	46.4	2.61	5.6	3	135
10.6	50.27	2.89	5.7	3	135
10.68	61.6	3.24	5.2	11	135
10.77	78.38	3.65	4.6	11	135
10.85	78.8	4.06	5.1	11	135
10.93	76.9	4.37	5.7	11	135
11	82.96	4.48	5.4	11	135
11.09	82.93	4.53	5.5	11	135
11.16	77.31	4.5	5.8	11	135
11.26	69.58	4.28	6.1	11	135
11.35	64.48	3.64	5.6	11	135
11.43	59.39	3.44	5.8	11	135
11.52	50.11	3.24	6.5	3	135
11.58	48.55	3.05	6.3	3	135
11.63	47.85	2.96	6.2	3	135
11.71	43.91	2.99	6.8	3	135
11.8	48.13	3.12	6.5	3	135
11.88	57.13	3.28	5.7	3	135
11.96	60.19	3.41	5.7	11	135
12.05	62.13	3.46	5.6	11	135
12.13	57.17	3.29	5.7	3	135
12.21	47.09	2.89	6.1	3	135
12.3	38.98	2.54	6.5	3	135
12.38	32.45	2.16	6.7	3	135
12.46	27.23	1.85	6.8	3	135
12.55	24.97	1.68	6.7	3	135
12.63	28.01	1.58	5.6	3	135
12.72	32.45	1.56	4.8	3	135
12.8	31.16	1.62	5.2	3	135
12.88	26.49	1.61	6.1	3	135
12.96	20.77	1.44	6.9	3	135
13.05	16.33	1.11	6.8	3	135
13.14	13.75	0.83	6	3	125
13.22	12.87	0.64	5	3	125

13.3	11.9	0.54	4.5	3	125
13.38	12.48	0.51	4.1	3	125
13.47	12.06	0.53	4.4	3	125
13.56	13.43	0.6	4.4	3	125
13.64	15.44	0.79	5.1	3	125
13.73	26.57	1.04	3.9	4	135
13.81	33.74	1.17	3.5	5	135
13.9	29.01	1.16	4	4	135
13.98	23.35	1.08	4.6	3	135
14.07	19	0.91	4.8	3	135
14.17	16.17	0.69	4.3	3	125
14.25	14.4	0.58	4	3	125
14.34	11.66	0.52	4.5	3	125
14.42	9.24	0.5	5.3	3	115
14.51	8.59	0.48	5.5	3	115
14.61	9.32	0.48	5.1	3	115
14.69	10.3	0.48	4.6	3	115
14.78	10.78	0.42	3.9	3	115
14.87	9.58	0.4	4.1	3	115
14.97	10.86	0.39	3.6	3	115
15.06	10.85	0.37	3.4	4	115
15.15	11.58	0.34	2.9	4	115
15.24	12.79	0.36	2.8	5	115
15.33	12.31	0.36	2.9	4	115
15.42	10.94	0.38	3.5	4	115
15.51	10.05	0.4	4	3	115
15.59	9.88	0.4	4	3	115
15.68	10.86	0.34	3.1	4	115
15.78	9.65	0.31	3.2	4	115
15.87	7.8	0.28	3.5	3	105
15.96	6.18	0.24	3.8	3	105
16.05	5.29	0.18	3.4	3	95
16.13	4.33	0.16	3.5	3	95
16.23	4.33	0.14	3.2	3	95
16.31	4.25	0.13	3	3	95
16.41	4.25	0.13	2.9	3	95
16.5	4.65	0.13	2.7	3	95
16.58	4.81	0.13	2.6	3	95
16.67	4.65	0.14	2.9	3	95
16.76	5.05	0.15	2.9	3	95
16.85	5.3	0.15	2.9	3	95
16.94	5.54	0.15	2.7	3	95
17.02	5.7	0.15	2.6	3	95
17.11	6.1	0.16	2.5	4	95
17.2	6.26	0.16	2.4	4	95
17.29	6.5	0.15	2.3	4	95
17.38	7.07	0.16	2.2	4	95
17.47	7.15	0.17	2.4	4	105
17.56	7.31	0.18	2.5	4	105
17.65	7.31	0.19	2.6	4	105
17.73	7.79	0.2	2.5	4	105
17.82	8.36	0.26	3.1	4	105
17.91	9.09	0.29	3.1	4	115
18	10.78	0.31	2.9	4	115
18.08	10.21	0.33	3.2	4	115
18.12	11.9	0.32	2.7	5	115
18.2	10.86	0.31	2.8	4	115
18.3	10.87	0.3	2.7	4	115
18.38	10.37	0.3	2.9	4	115
18.47	11.43	0.31	2.7	5	115
18.56	12.24	0.32	2.6	5	115
18.64	12.54	0.35	2.8	5	115
18.74	11.98	0.38	3.1	4	115
18.82	11.67	0.38	3.2	4	115
18.91	11.18	0.35	3.1	4	115
19	11.09	0.34	3.1	4	115
19.08	12.47	0.37	2.9	4	115
19.17	13.99	0.44	3.1	4	125
19.27	17.95	0.52	2.9	5	125
19.36	20.54	0.68	3.3	5	125
19.44	20.12	0.82	4	4	125
19.54	24.91	0.85	3.4	5	135
19.63	26.5	0.86	3.2	5	135
19.72	26.33	0.84	3.2	5	135
19.81	24.8	0.81	3.3	5	135
19.89	23.28	0.78	3.3	5	135
19.98	20.04	0.7	3.4	5	125
20.08	20.14	0.64	3.1	5	125
20.16	18.51	0.63	3.3	5	125
20.26	19.5	0.65	3.3	5	125
20.35	19.87	0.65	3.2	5	125
20.43	22.3	0.69	3.1	5	125
20.52	24.57	0.78	3.2	5	135
20.61	29.32	0.94	3.2	5	135

20.7	37.64	1.06	2.8	6	135
20.78	35.99	1.1	3	5	135
20.87	33.2	1.14	3.4	5	135
20.96	29.81	1.09	3.6	5	135
21.05	27.71	0.94	3.3	5	135
21.14	26.92	0.84	3.1	5	135
21.23	26.57	0.82	3	5	135
21.31	26.66	0.8	3	5	135
21.4	25.93	0.78	3	5	135
21.49	26.25	0.74	2.8	5	135
21.56	24	0.69	2.8	5	125
21.65	22.71	0.63	2.7	5	125
21.74	20.53	0.59	2.8	5	125
21.82	20.63	0.6	2.8	5	125
21.92	22.05	0.63	2.8	5	125
22	23.25	0.67	2.8	5	125
22.09	25.06	0.72	2.8	5	125
22.17	25.53	0.77	2.9	5	135
22.27	27.8	0.85	3	5	135
22.35	30.69	0.94	3	5	135
22.44	33.17	1.05	3.1	5	135
22.54	35.69	1.17	3.2	5	135
22.62	37.37	1.24	3.3	5	135
22.71	37.21	1.33	3.5	5	135
22.8	37.07	1.38	3.7	5	135
22.89	38.73	1.41	3.6	5	135
22.97	41.58	1.49	3.5	5	135
23.06	44.25	1.59	3.6	5	135
23.14	43.35	1.66	3.8	5	135
23.23	38.82	1.66	4.3	4	135
23.32	36.58	1.6	4.3	4	135
23.41	34.24	1.44	4.2	4	135
23.49	34.16	1.3	3.8	5	135
23.58	33.83	1.16	3.4	5	135
23.66	34.32	1.1	3.2	5	135
23.76	33.97	1.08	3.1	5	135
23.84	32.96	1.09	3.3	5	135
23.93	31.72	1.2	3.7	5	135
24.01	33.56	1.28	3.8	5	135
24.1	41.46	1.34	3.2	5	135
24.18	63.03	1.32	2.1	7	135
24.27	71.13	1.17	1.6	7	135
24.35	57.13	1.35	2.4	6	135
24.44	42.26	1.53	3.6	5	135
24.53	35.01	1.58	4.5	4	135
24.62	32.38	1.36	4.2	4	135
24.65	43.5	1.36	3.1	6	135
24.73	66.67	1.36	2	7	135
24.81	95.04	1.32	1.4	8	135
24.9	114.25	1.22	1.1	8	125
24.99	129.2	1.17	0.9	9	125
25.07	136.62	1.19	0.9	9	125
25.16	130.97	1.38	1.1	8	125
25.24	116.73	1.79	1.5	8	135
25.33	93.99	2.24	2.4	7	135
25.42	78.15	2.22	2.8	6	135
25.51	62.45	1.93	3.1	6	135
25.59	49.05	1.85	3.8	5	135
25.68	39.79	1.83	4.6	4	135
25.76	38.36	1.83	4.8	4	135
25.85	42.05	1.91	4.5	4	135
25.93	42.42	2	4.7	4	135
26.02	54.07	2.06	3.8	5	135
26.1	89.01	1.97	2.2	7	135
26.18	143.21	1.91	1.3	8	135
26.26	179.98	2.02	1.1	9	125
26.34	211.78	2.28	1.1	9	125
26.42	229.73	2.62	1.1	9	125
26.5	239.02	3.1	1.3	9	135
26.58	232.96	3.51	1.5	9	135
26.66	214.57	3.79	1.8	9	135
26.74	191.51	4.14	2.2	7	135
26.83	166.94	4.66	2.8	7	135
26.9	139.4	4.91	3.5	6	135
27	102.23	4.78	4.7	11	135
27.07	68.8	4.38	6.4	11	135
27.16	50.77	3.73	7.3	3	135
27.24	42.57	3.27	7.7	3	135
27.32	41.4	2.78	6.7	3	135
27.4	42.12	2.57	6.1	3	135
27.48	47.22	2.7	5.7	3	135
27.56	64.48	2.36	3.7	5	135
27.64	82.12	2.56	3.1	6	135
27.71	87.89	2.78	3.2	6	135

27.79	103.47	2.85	2.8	6	135
27.85	116.84	2.89	2.5	7	135
27.93	122.1	3.15	2.6	7	135
28	116.74	3.53	3	6	135
28.08	111.4	3.74	3.4	6	135
28.16	98.4	3.68	3.7	6	135
28.24	82.53	3.58	4.3	5	135
28.32	86.31	3.35	3.9	5	135
28.4	102.07	3.05	3	6	135
28.48	108.35	2.8	2.6	7	135
28.57	106.08	2.74	2.6	7	135
28.64	101.06	2.9	2.9	6	135
28.73	86.11	2.94	3.4	6	135
28.81	68.61	2.88	4.2	5	135
28.89	67.82	2.85	4.2	5	135
28.97	66.01	2.74	4.1	5	135
29.05	73.4	2.8	3.8	5	135
29.14	76.19	3.05	4	5	135
29.22	86.82	3.22	3.7	5	135
29.3	82.57	3.34	4	5	135
29.38	76.38	3.44	4.5	11	135
29.46	81.7	3.6	4.4	11	135
29.54	85.93	3.59	4.2	5	135
29.62	89.96	3.44	3.8	5	135
29.7	90.99	3.27	3.6	6	135
29.79	90.44	3.1	3.4	6	135
29.87	90.56	2.92	3.2	6	135
29.95	87.54	2.7	3.1	6	135
30.03	83.54	2.45	2.9	6	135
30.11	78.25	2.42	3.1	6	135
30.19	74.35	2.57	3.5	6	135
30.28	67.54	2.61	3.9	5	135
30.35	60.42	2.37	3.9	5	135
30.44	65.64	2.02	3.1	6	135
30.52	81.06	1.73	2.1	7	135
30.59	87.73	1.52	1.7	7	135
30.68	87.34	1.4	1.6	7	135
30.77	83.76	1.53	1.8	7	135
30.85	82.76	1.48	1.8	7	135
30.93	75.86	1.69	2.2	7	135
31.02	60.74	2.14	3.5	5	135
31.08	43.91	2.29	5.2	3	135
31.18	42.78	2.21	5.2	3	135
31.26	34.73	2.04	5.9	3	135
31.35	30.53	1.8	5.9	3	135
31.43	25.3	1.65	6.5	3	135
31.52	24.91	1.59	6.4	3	135
31.6	30.09	1.57	5.2	3	135
31.68	40.92	1.61	3.9	5	135
31.76	48.33	1.73	3.6	5	135
31.85	43.14	1.77	4.1	5	135
31.93	33.74	1.69	5	3	135
32.02	25.84	1.65	6.4	3	135
32.1	23.87	1.56	6.5	3	135
32.19	23.6	1.4	5.9	3	135
32.27	22.25	1.27	5.7	3	135
32.36	20.87	1.31	6.3	3	135
32.45	22.25	1.31	5.9	3	135
32.53	23.42	1.17	5	3	135
32.62	23.35	1.15	4.9	3	135
32.7	23.09	1.14	4.9	3	135
32.78	22.14	1.07	4.8	3	135
32.87	19.82	0.97	4.9	3	135
32.95	16.1	0.84	5.2	3	125
33.04	14.64	0.66	4.5	3	125
33.12	13.36	0.56	4.1	3	125
33.21	12.38	0.5	4	3	125
33.29	12.79	0.47	3.7	4	125
33.38	14.16	0.55	3.9	4	125
33.47	16.59	0.71	4.3	3	125
33.56	20.36	0.92	4.5	3	135
33.64	30.71	1.12	3.6	5	135
33.73	34.57	1.29	3.7	5	135
33.81	33.03	1.42	4.3	4	135
33.89	31.58	1.45	4.6	4	135
33.98	29.81	1.42	4.8	3	135
34.06	29.72	1.4	4.7	3	135
34.15	28.61	1.36	4.7	3	135
34.4	29.26	1.2	4.1	4	135
34.49	25.94	1.17	4.5	4	135
34.57	24.66	1.16	4.6	3	135
34.66	23.13	1.04	4.5	3	135
34.75	18.6	0.9	4.8	3	125
34.84	16.84	0.78	4.6	3	125

34.92	15.7	0.68	4.3	3	125
35.01	14.33	0.59	4.1	3	125
35.1	13.77	0.57	4.1	3	125
35.19	14.01	0.55	3.9	4	125
35.28	13.36	0.52	3.8	4	125
35.36	13.13	0.53	4	3	125
35.45	12.95	0.56	4.3	3	125
35.53	13.12	0.58	4.3	3	125
35.62	14.49	0.55	3.8	4	125
35.71	13.93	0.51	3.6	4	125
35.8	12.24	0.49	3.9	3	125
35.88	11.68	0.47	4	3	125
35.97	10.94	0.42	3.8	3	115
36.05	11.19	0.41	3.6	4	115
36.15	10.37	0.38	3.6	3	115
36.23	9.41	0.34	3.5	3	115
36.32	8.68	0.3	3.4	3	115
36.41	8.53	0.29	3.3	3	115
36.5	7.71	0.34	4.2	3	115
36.58	7.31	0.35	4.6	3	115
36.66	8.28	0.32	3.8	3	115
36.73	10.3	0.31	2.9	4	115
36.82	10.06	0.31	3	4	115
36.9	9.42	0.32	3.3	4	115
36.99	9.66	0.32	3.2	4	115
37.09	10.94	0.32	2.9	4	115
37.17	10.06	0.39	3.8	3	115
37.25	11.03	0.49	4.3	3	125
37.34	16.11	0.52	3.2	5	125
37.44	13.69	0.57	4.1	3	125
37.51	12.64	0.58	4.5	3	125
37.6	14.66	0.54	3.6	4	125
37.68	15.05	0.46	3	5	125
37.76	13.04	0.43	3.2	4	125
37.85	13.68	0.41	2.9	5	125
37.94	13.2	0.36	2.7	5	115
38.02	10.06	0.32	3.1	4	115
38.11	7.23	0.27	3.6	3	105
38.2	6.59	0.26	3.8	3	105
38.28	6.83	0.29	4.1	3	105
38.37	7.56	0.32	4.1	3	115
38.46	9.65	0.35	3.5	3	115
38.55	11.51	0.41	3.5	4	115
38.63	12.24	0.41	3.3	4	115
38.72	13.12	0.43	3.2	4	125
38.81	18.76	0.49	2.6	5	125
38.89	17.88	0.53	2.9	5	125
38.98	12.88	0.55	4.2	3	125
39.07	12.15	0.56	4.6	3	125
39.16	21.68	0.48	2.2	6	125
39.24	28.93	0.45	1.6	6	125
39.33	30.48	0.54	1.8	6	125
39.42	27.65	0.65	2.4	6	125
39.5	25.14	0.7	2.8	5	125
39.59	22.47	0.67	3	5	125
39.68	18.67	0.67	3.6	4	125
39.76	15.3	0.73	4.8	3	125
39.85	11.75	0.82	6.9	3	125
39.94	13.84	0.84	6	3	125
40.03	22.8	0.78	3.4	5	135
40.12	32.7	0.69	2.1	6	125
40.2	37.39	0.59	1.6	7	125
40.29	40.9	0.49	1.2	7	125
40.38	41.63	0.58	1.4	7	125
40.46	38.68	0.76	2	6	135
40.55	27.07	0.82	3	5	135
40.64	18.44	0.82	4.4	3	125
40.73	14.48	0.77	5.3	3	125
40.8	10.61	0.75	7.1	3	125
40.85	14.33	0.76	5.2	3	125
40.93	18.77	0.72	3.8	4	125
41.01	22.23	0.65	2.9	5	125
41.1	27.54	0.63	2.3	6	125
41.18	30.76	0.67	2.2	6	125
41.27	32.05	0.63	1.9	6	125
41.35	34.81	0.53	1.5	7	125
41.44	38.52	0.57	1.5	7	125
41.52	38.86	0.69	1.8	6	125
41.61	32.49	0.84	2.6	6	135
41.7	28.21	0.92	3.3	5	135
41.79	25.78	0.85	3.3	5	135
41.87	25.56	0.68	2.7	5	125
41.96	25.22	0.52	2	6	125
42.04	26.11	0.48	1.8	6	125

42.13	24.63	0.61	2.5	6	125
42.22	21.18	0.67	3.2	5	125
42.31	16.74	0.68	4	4	125
42.39	14.39	0.69	4.8	3	125
42.48	13.92	0.62	4.4	3	125
42.57	13.43	0.51	3.8	4	125
42.65	12.64	0.4	3.2	4	115
42.74	10.29	0.3	2.9	4	115
42.82	8.76	0.24	2.7	4	105
42.91	7.72	0.23	2.9	4	105
43	6.34	0.27	4.2	3	105
43.08	6.99	0.34	4.8	3	115
43.17	9.09	0.37	4	3	115
43.26	15.28	0.39	2.5	5	125
43.34	21.82	0.4	1.8	6	125
43.43	22.25	0.41	1.8	6	125
43.52	18.59	0.43	2.3	5	125
43.6	14.24	0.43	3	5	125
43.69	10.96	0.43	3.9	3	115
43.78	8.36	0.28	3.3	3	105
43.87	8.76	0.33	3.7	3	115
43.96	12.14	0.41	3.3	4	115
44.04	15.96	0.38	2.3	5	125
44.1	10.85	0.37	3.2	4	115
44.2	16.59	0.36	2.2	5	125
44.28	14.2	0.36	2.5	5	115
44.37	10.62	0.35	3.2	4	115
44.46	9.32	0.32	3.3	4	115
44.54	10.29	0.27	2.5	5	115
44.63	10.23	0.21	2	5	105
44.72	8.36	0.2	2.3	4	105
44.8	7.4	0.18	2.3	4	105
44.9	7.89	0.19	2.3	4	105
44.98	7.55	0.2	2.4	4	105
45.07	7.55	0.21	2.6	4	105
45.15	8.29	0.24	2.7	4	105
45.25	7.63	0.31	3.8	3	105
45.33	7.71	0.35	4.2	3	115
45.42	12.64	0.47	3.5	4	125
45.5	38.62	0.79	2	6	135
45.59	57.13	1.05	1.8	7	135
45.67	47.5	1.04	2.2	6	135
45.76	41.08	1.14	2.7	6	135
45.85	34.35	1.3	3.7	5	135
45.93	34.82	1.45	4.1	4	135
46.02	35.41	1.57	4.3	4	135
46.11	34.22	1.56	4.5	4	135
46.19	31.73	1.44	4.5	4	135
46.28	28.77	1.29	4.4	4	135
46.36	28.18	1.16	4.1	4	135
46.45	28.43	1.12	3.9	4	135
46.54	30.61	1.18	3.8	5	135
46.62	37.71	1.35	3.5	5	135
46.71	48.42	1.62	3.3	5	135
46.79	55.84	1.86	3.3	6	135
46.87	55.15	2.11	3.8	5	135
46.96	54.46	2.68	4.9	4	135
47.04	54.37	2.61	4.7	4	135
47.12	51.3	2.36	4.5	4	135
47.19	50.77	2.2	4.3	5	135
47.23	49.57	2.11	4.2	5	135
47.31	45.06	1.93	4.2	5	135
47.4	43.1	1.76	4	5	135
47.48	41.01	1.62	3.9	5	135
47.56	38.59	1.52	3.9	5	135
47.65	38.36	1.45	3.7	5	135
47.73	36.48	1.38	3.7	5	135
47.82	35.54	1.36	3.7	5	135
47.9	32.87	1.35	4	4	135
47.99	31.01	1.31	4.1	4	135
48.07	30.21	1.28	4.2	4	135
48.15	30.06	1.35	4.4	4	135
48.24	31.89	1.42	4.4	4	135
48.33	33.93	1.36	3.9	5	135
48.41	33.11	1.22	3.6	5	135
48.49	32.07	1.11	3.4	5	135
48.58	28.61	1.05	3.6	5	135
48.67	25.71	1.04	4	4	135
48.76	24.01	1.04	4.3	4	135
48.84	22.48	1.03	4.5	3	135
48.93	21.84	0.98	4.4	3	135
49.01	20.77	0.95	4.5	3	135
49.09	20.72	0.92	4.4	3	135
49.18	20.29	0.9	4.4	3	135

Example of Output Data From Liquefaction Assessment Spreadsheet

CPT-Based Liquefaction Triggering Analysis

Per recommendations by Youd et al. (2001), "LIQUEFACTION RESISTANCE OF SOILS: SUMMARY REPORT FROM THE 1996 NCEER AND 1998 NCEER/NSF WORKSHOPS ON EVALUATION OF LIQUEFACTION RESISTANCE OF SOILS", ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 132, No.8

Project Name: Crosstown Freeway
Project No.: 111004.G01
Project Location: Stockton, CA
CPT I.D.: CPT-1

Analyzed by: Hamid Parsa
Date : 4/28/2011
Checked by: Carl
Date:

Mw= 6.7
PGA= 0.32
Field Ground Water Depth (feet) 11.3
Design Ground Water Depth 11
Total Settlement = 0.55 inches

Fill Heigth (feet) = 0
Unit Weight of the Fill (pcf) = 120

Factor of Safety for Liquefaction = 1.0
Factor of Safety for Settlement Calculation = 1.3

Depth (feet)	Depth (meters) From	To	Thickness (meter)	Design Depth to Water (m)	Field Depth to Water (m)	Unit Weight (pcf)	SBT Zone	SBT Classification	Unit Wt. (kN/m³)	σ_{vo} (Kpa)	Design σ_{vo}' (Kpa)	Field σ_{vo}' (Kpa)	σ_{vo}'+fill (Kpa)	Dolsign σ_{vo}'+fill (Kpa)	Field σ_{vo}'+fill (Kpa)	f_s (tsf)	q_c (tsf)	f_s (kpa)	q_c (Mpa)	n Factor	Normalization Factor C_N	q_{c1N}	Q	R_f (%)	Ic	FC (%)	$(q_{c1N})_{cs}$	rd	CSR	K_σ	K_α	CSR $_{M=7.5,\sigma'=1}$	CRR $_{M=7.5,\sigma'=1}$	FOS	Notes	N_{60}	C_E	$(N)_{60}$	$(N_1)_{60cs}$ for S_r	S_r (psf)	$(N_1)_{60cs}$ for Settlement	Volumetric Strain (%)	Settlement (inch)	Accumulative Settlement (inch)
0.67	0.17	0.20	0.02	3.4	3.4	135.0	7	Silty Sand to Sandy Sill	21.2	3.7	3.7	3.7	3.68	3.68	3.68	4.30	157.14	411.8	15.05	0.5	1.70	252.5	778.7	2.7	1.7	7.0	270.5	1.00	0.208	1.00	1.00	0.156	1.00	NL	Above WT	29.8	1.70	50.7	50.7	>1600	51.1	0.0000	0.000	0.55
0.64	0.20	0.22	0.02	3.4	3.4	135.0	6	Sandy Silt to Clayey Silt	21.2	4.1	4.1	4.1	4.14	4.14	4.14	4.97	153.52	475.9	14.70	0.5	1.70	246.7	718.9	3.2	1.8	8.7	278.7	1.00	0.208	1.00	1.00	0.156	1.00	NL	Above WT	30.0	1.70	51.0	51.0	>1600	51.8	0.0000	0.000	0.55
0.71	0.22	0.24	0.02	3.4	3.4	135.0	7	Silty Sand to Sandy Sill	21.2	4.6	4.6	4.6	4.59	4.59	4.59	4.76	153.86	455.8	14.73	0.5	1.70	247.2	653.1	3.1	1.8	8.6	278.3	1.00	0.208	1.00	1.00	0.156	1.00	NL	Above WT	30.0	1.70	51.0	51.0	>1600	51.7	0.0000	0.000	0.55
0.79	0.24	0.27	0.02	3.4	3.4	135.0	6	Sandy Silt to Clayey Silt	21.2	5.1	5.1	5.1	5.10	5.10	5.10	4.02	133.07	442.4	12.74	0.5	1.70	213.8	560.1	3.5	1.9	10.5	255.4	1.00	0.208	1.00	1.00	0.156	1.03	NL	Above WT	26.7	1.70	45.4	46.4	>1600	46.5	0.0000	0.000	0.55
0.87	0.27	0.29	0.02	3.4	3.4	135.0	6	Sandy Silt to Clayey Silt	21.2	5.6	5.6	5.6	5.62	5.62	5.62	4.34	117.91	415.6	11.29	0.5	1.70	189.4	472.8	3.7	2.0	11.7	236.4	1.00	0.208	1.00	1.00	0.156	1.03	NL	Above WT	24.1	1.70	41.0	42.0	>1600	42.2	0.0000	0.000	0.55
0.94	0.29	0.31	0.02	3.4	3.4	135.0	11	Very Stiff Fine Grained	21.2	6.3	6.1	6.1	6.07	6.07	6.07	4.51	103.45	431.9	9.91	0.5	1.70	166.2	399.1	4.4	2.3	14.4	229.5	1.00	0.208	1.00	1.00	0.156	1.00	NL	Above WT	22.0	1.70	37.4	38.4	>1600	38.7	0.0000	0.000	0.55
1.02	0.31	0.34	0.02	3.4	3.4	135.0	11	Very Stiff Fine Grained	21.2	6.6	6.6	6.6	6.59	6.59	6.59	4.63	114.58	443.4	10.97	0.5	1.70	184.1	424.3	4.0	2.0	13.1	241.1	1.00	0.208	1.00	1.00	0.156	1.00	NL	Above WT	23.9	1.70	40.7	41.7	>1600	41.9	0.0000	0.000	0.55
1.10	0.34	0.36	0.02	3.4	3.4	135.0	11	Very Stiff Fine Grained	21.2	7.1	7.1	7.1	7.11	7.11	7.11	4.37	98.93	418.5	9.47	0.5	1.70	158.9	352.7	4.4	2.3	15.1	223.5	1.00	0.207	1.00	1.00	0.155	1.00	NL	Above WT	21.2	1.70	36.1	37.1	>1600	37.4	0.0000	0.000	0.55
1.18	0.36	0.39	0.03	3.4	3.4	135.0	11	Very Stiff Fine Grained	21.2	7.6	7.6	7.6	7.63	7.63	7.63	3.72	86.15	356.2	8.25	0.5	1.70	138.4	296.5	4.3	2.3	15.9	202.6	1.00	0.207	1.00	1.00	0.155	1.00	NL	Above WT	18.7	1.70	31.8	32.8	>1600	33.2	0.0000	0.000	0.54
1.27	0.39	0.41	0.03	3.4	3.4	135.0	11	Very Stiff Fine Grained	21.2	8.2	8.2	8.2	8.21	8.21	8.21	3.66	74.80	369.6	7.16	0.5	1.70	120.2	248.1	5.2	2.2	19.4	204.7	1.00	0.207	1.00	1.00	0.155	1.00	NL	Above WT	17.0	1.70	28.8	29.8	>1600	30.5	0.0000	0.000	0.54
1.36	0.41	0.44	0.02	3.4	3.4	135.0	11	Very Stiff Fine Grained	21.2	8.8	8.8	8.8	8.79	8.79	8.79	4.42	89.67	423.3	5.62	0.5	1.70	143.3	193.0	7.5	2.4	26.7	222.8	1.00	0.207	1.00	1.00	0.155	1.03	NL	Above WT	14.3	1.70	24.7	26.7	>1600	26.8	0.0000	0.000	0.53
1.43	0.44	0.46	0.03	3.4	3.4	135.0	11	Very Stiff Fine Grained	21.2	9.2	9.2	9.2	9.24	9.24	9.24	4.84	90.12	463.5	8.63	0.5	1.70	144.8	281.7	5.4	2.2	19.0	242.3	1.00	0.207	1.00	1.00	0.155	1.00	NL	Above WT	20.3	1.70	34.6	35.6	>1600	36.2	0.0000	0.000	0.54
1.52	0.46	0.48	0.02	3.4	3.4	140.0	12	Sand to Clayey Sand	22.0	9.8	9.8	9.8	9.84	9.84	9.84	5.62	149.54	539.2	14.32	0.5	1.70	240.3	453.1	3.8	2.0	12.2	304.9	1.00	0.207	1.00	1.00	0.155	1.00	NL	Above WT	30.8	1.70	52.4	53.4	>1600	53.6	0.0000	0.000	0.55
1.59	0.48	0.51	0.03	3.4	3.4	140.0	12	Sand to Clayey Sand	22.0	10.3	10.3	10.3	10.31	10.31	10.31	6.01	159.54	575.5	14.89	0.5	1.70	249.9	460.4	3.9	2.0	12.4	319.4	1.00	0.207	1.00	1.00	0.155	1.03	NL	Above WT	32.2	1.70	54.7	55.7	>1600	55.8	0.0000	0.000	0.55
1.68	0.51	0.53	0.02	3.4	3.4	135.0	7	Silty Sand to Sandy Sill	21.2	10.9	10.9	10.9	10.89	10.89	10.89	4.80	158.17	459.6	15.15	0.5	1.70	254.1	455.6	3.0	1.9	9.9	297.7	1.00	0.207	1.00	1.00	0.155	1.03	NL	Above WT	31.5	1.70	53.5	53.5	>1600	54.5	0.0000	0.000	0.55
1.75	0.53	0.56	0.03	3.4	3.4	135.0	7	Silty Sand to Sandy Sill	21.2	11.3	11.3	11.3	11.35	11.35	11.35	4.38	140.99	419.4	14.08	0.5	1.70	236.2	414.8	3.0	1.9	10.3	280.5	1.00	0.207	1.00	1.00	0.155	1.03	NL	Above WT	29.5	1.70	50.1	51.1	>1600	51.1	0.0000	0.000	0.55
1.84	0.56	0.58	0.02	3.4	3.4	135.0	7	Silty Sand to Sandy Sill	21.2	11.9	11.9	11.9	11.93	11.93	11.93	4.13	147.89	395.5	14.13	0.5	1.70	237.1	406.2	2.8	1.9	9.8	276.8	1.00	0.207	1.00	1.00	0.155	1.03	NL	Above WT	29.3	1.70	49.9	49.9	>1600	50.8	0.0000	0.000	0.55
1.91	0.58	0.60	0.02	3.4	3.4	135.0	7	Silty Sand to Sandy Sill	21.2	12.4	12.4	12.4	12.38	12.38	12.38	3.77	145.00	361.0	13.89	0.5	1.70	233.0	391.7	2.6	1.9	9.3	267.8	1.00	0.207	1.00	1.00	0.155	1.00	NL	Above WT	28.6	1.70	48.6	48.6	>1600	49.5	0.0000	0.000	0.55
1.97	0.60	0.63	0.02	3.4	3.4	135.0	7	Silty Sand to Sandy Sill	21.2	12.8	12.8	12.8	12.77	12.77	12.77	3.71	137.16	355.3	13.13	0.5	1.70	220.3	364.8	2.7	1.9	9.9	254.8	1.00	0.207	1.00	1.00	0.155	1.03	NL	Above WT	27.3	1.70	46.5	46.5	>1600	47.5	0.0000	0.000	0.55
2.05	0.63	0.65	0.02	3.4	3.4	135.0	6	Sandy Silt to Clayey Silt	21.2	13.3	13.3	13.3	13.29	13.29	13.29	3.72	117.48	356.2	11.25	0.5	1.70	188.7	306.3	3.2	2.0	12.5	242.0	1.00	0.207	1.00	1.00	0.155	1.00	NL	Above WT	24.3	1.70	41.3	42.3	>1600	42.5	0.0000	0.000	0.55
2.13	0.65	0.67	0.02	3.4	3.4	135.0	5	Clayey Silt to Silty Clay	21.2	13.8	13.8	13.8	13.80	13.80	13.80	3.78	99.67	362.0	9.26	0.5	1.70	155.3	247.2	3.9	2.1	15.9	228.0	1.00	0.207	1.00	1.00	0.155	1.03	NL	Above WT	21.0	1.70	35.7	36.7	>1600	37.1	0.0000	0.000	0.55
2.21	0.67	0.70	0.03	3.4	3.4	135.0	11	Very Stiff Fine Grained	21.2	14.3	14.3	14.3	14.32	14.32	14.32	3.56	69.74	340.9	6.48	0.5	1.70	112.0	175.9	5.1	2.3	21.8	215.1	1.00	0.207	1.00	1.00	0.155	1.03	NL	Above WT	16.3	1.70	27.7	28.7	>1600	29.5	0.0000	0.000	0.54
2.30	0.70	0.73	0.02	3.4	3.4	135.0	3	Clay	21.2	14.9	14.9	14.9	14.90	14.90	14.90	3.28	51.86	314.1	4.97	0.5	1.70	83.3	127.4	6.3	2.4	27.8	205.9	0.99	0.207	1.00	1.00	0.155	1.03	NL	Above WT	13.0	1.70	22.1	24.1	>1600	24.3	0.0000	0.000	0.53
2.38	0.73	0.75	0.03	3.4	3.4	135.0	3	Clay	21.2	15.4	15.4	15.4	15.41	15.41	15.41	3.03	45.38	290.5	4.35	0.5	1.70	72.9	100.6	6.7	2.5	30.3	200.5	0.99	0.207	1.00	1.00	0.155	1.03	NL	Above WT	11.7	1.70	19.8	21.8	1435.6	22.3	0.0000	0.000	0.53
2.46	0.75	0.77	0.02	3.4	3.4	135.0	3	Clay	21.2	15.9	15.9	15.9	15.93	15.93	15.93	2.74	38.34	262.4	3.67	0.5	1.70	61.6	91.0	7.1	2.6	33.4	191.7	0.99	0.207	1.00	1.00	0.155	1.03	NL	Above WT	10.2	1.70	17.3	19.3	1290.8	20.0	0.0000	0.000	0.58
2.64	0.77	0.80	0.02	3.4	3.4	135.0	3	Clay	21.2	16.5	16.5	16.5	16.45	16.45	16.45	2.46	34.06	235.6	3.26	0.7	1.70	54.7	114.3	7.2	2.5	31.0	134.8	0.99	0.207	1.00	1.00	0.155	0.42	NL	Above WT	8.8	1.70	15.0	17.0	900.0	17.5	0.0000	0.000	0.58
2.62	0.80	0.82	0.02	3.4	3.4	135.0	3	Clay	21.2	17.0	17.0	17.0	16.97	16.97	16.97	2.27	29.79	217.4	2.70	0.7	1.70	46.8	95.5	7.8	2.6	34.4	151.2	0.99	0.207	1.00	1.00	0.155	0.49	NL	Above WT	7.8	1.70	13.3	15.3	796.0	16.0	0.0000	0.000	0.58
2.70	0.82	0.85	0.02	3.4	3.4	135.0	3	Clay	21.2	17.5	17.5	17.5	17.49	17.49	17.49	2.08	25.22	199.2	2.42	0.7	1.70	40.5	80.9	8.2	2.6	37.3	146.2	0.99	0.207	1.00	1.00	0.155	NL	NL	Above WT	7.0	1.70	11.9	13.9	550.0	14.9	0.0000	0.000	0.58
2.78	0.85	0.88	0.03	3.4	3.4	135.0	3	Clay	21.2	18.0	18.0	18.0	18.00	18.00	18.00	1.97	25.37	188.6	2.43	0.7	1.70	40.8	79.8	7.8	2.6	36.6	143.5	0.99	0.207	1.00	1.00	0.155	NL	NL	Above WT	7.0	1.70	11.9	13.9	550.0	14.8	0.0000	0.000	0.58
2.87	0.88	0.90	0.02	3.4	3.4	135.0	3	Clay	21.2	18.6	18.6	18.6	18.58	18.58	18.58	1.83	24.23	175.2	2.32	0.7	1.70	38.9	74.5	7.6	2.6	37.1	139.3	0.99	0.207	1.00	1.00	0.155	NL	NL	Above WT	6.7	1.70	11.4	13.4	500.0	14.3	0.0000	0.000	0.58
2.95	0.90	0.92	0.02	3.4	3.4	135.0	3	Clay	21.2	19.1	19.1	19.1	19.10	19.10	19.10	1.69	21.75	161.8	2.08	0.7	1.70	34.9	65.5	7.8	2.7	39.3	135.3	0.99	0.207	1.00	1.00	0.155	NL	NL	Above WT	6.1	1.70	10.4	12.4	400.0	13.6	0.0000	0.000	0.58
3.03	0.92	0.95	0.02	3.4	3.4	135.0	3	Clay	21.2	19.6	19.6	19.6	19.62	19.62	19.62	1.59	20.93	152.3	2.00	0.7	1.70	33.6	61.8	7.6	2.7	39.6	131.8	0.99	0.207	1.00	1.00	0.155	NL	NL	Above WT	5.9	1.70	10.1	12.1	400.0	13.2	0.0000	0.000	0.58
3.11	0.95	0.98	0.03	3.4	3.4	135.0	3	Clay	21.2	20.1	20.1	20.1	20.13	20.13	20.13	1.62	21.10	155.1	2.02	0.7	1.70	33.9	61.2	7.7	2.7	40.0	134.6	0.99	0.206	1.00	1.00	0.155	NL	NL	Above WT	6.0	1.70	10.2	12.2	400.0	13.4	0.0000	0.000	0.58
3.20	0.98	1.00	0.02	3.4	3.4	135.0	3	Clay	21.2	20.7	20.7	20.7	20.72	20.72	20.72	1.60	21.09	153.2	2.10	0.7	1.70	35.2	62.2	7.3	2.7	38.8	134.0	0.99	0.206	1.00	1.00	0.155	NL	NL	Above WT	6.2	1.70	10.5	12.5	400.0	13.6	0.0000	0.000	0.58
3.28	1.00	1.03	0.03	3.4	3.4	135.0	3	Clay	21.2	21.2	21.2	21.2	21.23	21.23	21.23	1.54	24.95	147.5	2.35	0.7	1.70	39.4	69.4	6.3	2.6	35.0	130.5	0.99	0.206	1.00	1.00	0.155	0.29	NL	Above WT	6.6	1.70	11.3	13.3	500.0	14.1	0.0000	0.000	0.58
3.37	1.03	1.05	0.02	3.4	3.4	135.0	3	Clay	21.2	21.8	21.8	21.8	21.81	21.81	21.81	1.54	25.04	147.5	2.40	0.7	1.70	40.2	68.7	6.2	2.6	34.7	131.8	0.99	0.206	1.00	1.00	0.155	0.29	NL	Above WT	6.7	1.70	11.5	13.5	500.0	14.2	0.0000	0.000	0.58
3.46	1.05	1.08	0.02	3.4	3.4	135.0	3	Clay	21.2	22.3	22.3	22.3	22.33	22.33	22.33	1.51	19.90	144.6	1.91	1.0	1.70	32.1	54.6	7.6	2.6	35.4	105.9	0.99	0.206	1.00	1.00	0.155	NL	NL	Above WT	5.4	1.70	9.2	11.2	350.0	12.0	0.0000	0.000	0.58
3.53	1.08	1.10	0.02	3.4	3.4	135.0	3	Clay	21.2	22.8	22.8	22.8	22.85	22.85	22.85	1.41	17.38	135.0	1.66	1.0	1.70	27.9	71.8	8.1	2.7	38.7	105.8	0.99	0.206	1.00	1.00	0.155	NL	NL	Above WT	4.9	1.70	8.3	10.3	300.0	11.4	0.0000	0.000	0.58
3.61	1.10	1.13	0.03	3.4	3.4	135.0	3	Clay	21.2	23.4	23.4	23.4	23.37	23.37	23.37	1.28	15.93	122.6	1.53	1.0	1.70	25.6	64.3	8.1	2.7	40.2	102.4	0.99	0.206	1.00	1.00	0.155	NL	NL	Above WT	4.5	1.70	7.7	9.7	275.0	10.9	0.0000	0.000	0.58
3.70	1.13	1.15	0.02	3.4	3.4	135.0	3	Clay	21.2	23.9	23.9	23.9	23.95	23.95	23.95	1.21	17.55	115.9	1.61	1.0	1.70	24.7	69.2	6.9	2.6	36.4	98.4	0.99	0.206	1.00	1.00	0.155	NL	NL	Above WT	4.8	1.70	9.2	10.2	300.0	11.1	0.0000	0.000	0.58
3.78	1.15	1.18	0.03	3.4	3.4	135.0	3	Clay	21.2	24.5	24.5	24.5	24.46	24.46	24.46	1.18	15.93	113.0	1.53	1.0	1.70	25.6	61.4	7.4	2.7	39.3	99.0	0.99	0.206	1.00	1.00	0.155	NL	NL	Above WT	4.5	1.70	7.7	9.7	275.0	10.9	0.0000	0.000	0.58
3.87	1.18	1.20	0.02	3.4	3.4	135.0	3	Clay	21.2	25.0	25.0	25.0	25.05	25.05	25.05	1.16	14.00	111.1	1.42	1.0	1.70	23.8	55.8	7.4	2.7	41.9	100.6	0.99	0.206	1.00	1.00	0.155	NL	NL	Above WT	4.3	1.70	7.3	9.3	250.0	10.6	0.0000	0.000	0.58
3.95	1.20	1.23	0.03	3.4	3.4	135.0	3	Clay	21.2	25.6	25.6	25.6	25.56	25.56	25.56	1.16	15.93	111.1	1.53	1.0	1.70	25.6	58.7	7.3	2.7	39.7	100.4	0.99	0.206	1.00	1.00	0.154	NL	NL	Above WT	4.5	1.70	7.7	9.7	275.0	10.8	0.0000	0.000	0.58
4.04	1.23	1.26	0.02	3.4	3.4	135.0	3	Clay	21.2	26.1	26.1	26.1	26.14	26.14	26.14	1.15	15.86	110.1	1.52	1.0	1.70	25.5	57.1	7.3	2.7	40.1	101.3	0.99	0.206	1.00	1.00	0.154	NL	NL	Above WT	4.5	1.70	7.7	9.7	275.0	10.9	0.0000	0.000	0.58
4.12	1.26	1.28	0.03	3.4	3.4	135.0	3	Clay	21.2	26.7	26.7	26.7	26.66	26.66	26.66	1.13	18.44	108.2	1.80	0.7	1.70	30.3	44.7	6.0	2.7	40.3	121.6	0.99	0.206	1.00	1.00	0.154	NL	NL	Above WT	5.4	1.70	9.1	11.1	350.0	12.4	0.0000	0.000	0.58
4.21	1.28	1.31	0.03	3.4	3.4	135.0	3	Clay	21.2	27.2	27.2	27.2	27.24	27.24	27.24	1.14	18.04	109.2	1.73	1.0	1.70	29.0	62.4	6.3	2.6	36.3	100.7	0.99	0.206	1.00	1.00	0.154	NL	NL	Above WT	4.9	1.70	8.4	10.4	300.0	11.3	0.0000	0.000	0.58
4.30	1.31	1.34	0.03	3.4	3.4	135.0	3	Clay	21.2	27.8	27.8	27.8	27.82	27.82	27.82	1.14	18.09	109.2	1.78	1.0	1.70	29.9	63.0	6.1	2.6	35.6	101.2	0.99	0.206	1.00	1.00	0.154	NL	NL	Above WT	5.1	1.70	8.6	10.6	325.0	11.4	0.0000	0.000	0.58
4.39	1.34	1.36	0.02	3.4	3.4	135.0	3	Clay	21.2	28.4	28.4	28.4	28.41	28.41	28.41	1.12	18.44	107.3	1.77	1.0	1.70	29.6	61.2	6.1	2.6	36.0	101.9	0.99	0.206	1.00	1.00	0.154	NL	NL	Above WT	5.0	1.70	8.6	10.6	325.0	11.4	0.0000	0.000	0.58
4.47	1.36	1.39	0.03	3.4	3.4	135.0	3	Clay	21.2	28.9	28.9	28.9	28.92	28.92	28.92	1.12	16.83	107.3	1.61	1.0	1.70	27.0	54.7	6.7	2.7	39.2	104.3	0.99	0.206	1.00	1.00	0.154	NL	NL	Above WT	4.7	1.70	8.1	10.1	300.0	11.2	0.0000	0.000	0.58
4.58	1.39	1.41	0.02	3.4	3.4	135.0	3	Clay	21.2	29.5	29.5	29.5	29.50	29.50	29.50	1.12	15.65	107.3	1.49	1.0	1.70	25.1	49.7	7.2	2.7	41.9	106.2	0.99	0.206	1.00	1.00	0.154	NL	NL	Above WT	4.5	1.70	7.7	9.7	275.0	11.0	0.0000	0.000	0.58
4.64	1.41	1.45	0.03	3.4	3.4	135.0	3	Clay	21.2	30.0	30.0	30.0	30.02	30.02	30.02	1.10	14.64	105.3	1.40	1.0	1.70	23.5	45.7	7.5	2.8	43.9	106.4	0.99	0.206	1.00	1.00	0.154	NL	NL	Above WT	4.3	1.70	7.4	9.4	250.0	10.9	0.0000	0.000	0.58
4.74	1.45	1.47	0.02	3.4	3.4	135.0	3	Clay	19.6	30.6	30.6	30.6	30.62	30.62	30.62	1.08	14.62	103.4	1.42	1.0	1.70	23.8	45.3	7.3	2.8	43.6	106.4	0.99	0.206	1.00	1.00	0.154	NL	NL	Above WT	4.4	1.70	7.4	9.4	250.0	10.9	0.0000	0.000	0.58
4.82	1.47	1.50	0.03	3.4	3.4	135.0	3	Clay	21.2	31.1	31.1	31.1	31.14	31.14	31.14	1.08	16.76	103.4	1.61	1.0	1.70	26.0	53.8	6.7	2.7	40.9	104.4	0.99	0.206	1.00	1.00	0.154	NL	NL	Above WT	4.6	1.70	7.9	9.9	275.0	11.2	0.0000	0.000	0.58
4.91	1.50	1.52	0.03	3.4	3.4	129.0	3	Clay	19.6	31.7	31.7	31.7	31.68	31.68	31.68	1.04	14.65	99.6	1.40	1.0	1.70	23.5	43.3	7.1	2.8	43.8	106.0	0.99	0.206	1.00	1.00	0.154	NL	NL	Above WT	4.3	1.70	7.4	9.4	250.0	10.9	0.0000	0.000	0.58
5.00	1.52	1.55	0.03	3.4	3.4	129.0	3	Clay	19.6	32.2	32.2	32.2	32.21	32.21	32.21	1.04	14.65	99.6	1.40	1.0	1.70	23.5	43.3	7.1	2.8	43.8	106.0	0.99	0.206	1.00	1.00	0.154	NL	NL	Above WT	4.3	1.70	7.3	9.3	250.0	10.7	0.0000	0.000	0.58
5.09	1.55	1.58	0.02	3.4	3.4	129.0	3	Clay	19.6	32.8	32.8	32.8	32.75	32.75	32.75	0.94	13.93	90.0	1.33	1.0	1.70	22.4	39.7	6.8	2.8	44.4	102.8	0.99	0.206	1.00	1.00	0.154	NL	NL	Above WT	4.1	1.70	7.0	9.0	250.0	10.6	0.0

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Liquefaction Assessment Results/Graphs

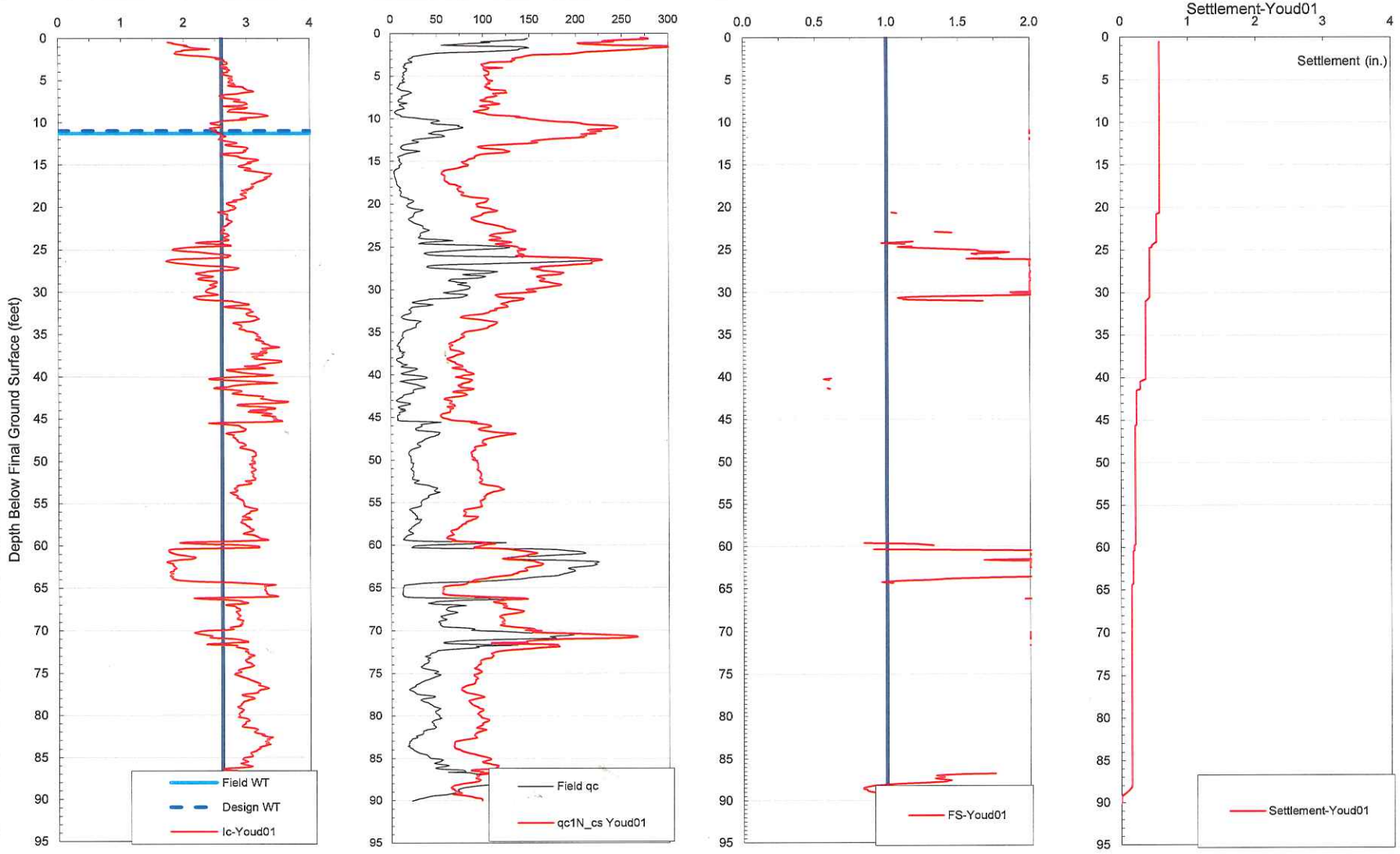
CPT ID: **CPT-11-001**

M_w= **6.7**
PGA= **0.32g**

Field Ground Water Depth (ft)= **11.3 ft**
Design Ground Water Depth (ft)= **11.0 ft**

Existing Ground Elevation= **0.0 ft**
Final Ground Elevation= **0.0 ft**

Ana. by: Hamid Parsa
Checked by: Carl henderson



Note: Factor of safety is capped at 2.0 for liquefiable soils and not plotted for non-liquefiable soils & soils above the ground water; $CRR_{M=7.5,\ \sigma v'=1tsf}$ is capped at 1.0 for liqeufiable soils and not plotted for non-liquefiable soils and soils above the ground water.

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2001 Arch Airport Road, #100
Stockton, California 95206
t: 209 948 1345 f: 209 234 4700

Project Name: Crosstown Freeway
Project No.: 111904.G01
Project Location: Stockton, CA

LIQUEFACTION ANALYSIS

Date 4/28/2011

CPT ID: **CPT-11-002**

M_w= **6.7**

PGA= **0.32g**

Field Ground Water Depth (ft)= **10.1 ft**

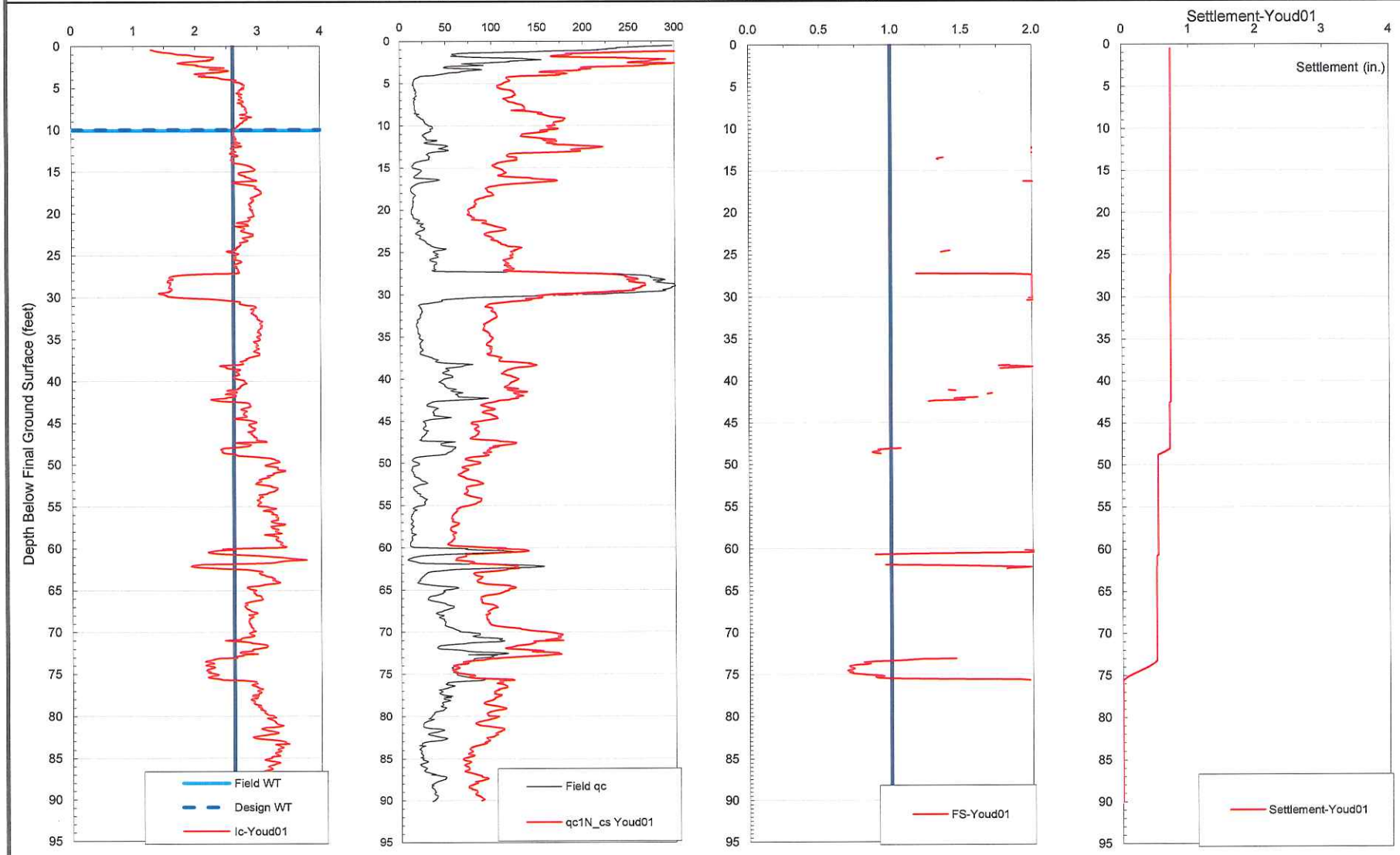
Design Ground Water Depth (ft)= **10.0 ft**

Existing Ground Elevation= **0.0 ft**

Final Ground Elevation= **0.0 ft**

Ana. by: Hamid Parsa

Checked by: Carl Henderson



Note: Factor of safety is capped at 2.0 for liquefiable soils and not plotted for non-liquefiable soils & soils above the ground water; $CRR_{M=7.5,\ \sigma v'=1tsf}$ is capped at 1.0 for liqeufiable soils and not plotted for non-liquefiable soils and soils above the ground water.

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Project Name: Crosstown Freeway
Project No.: 111904.G01
Project Location: Stockton, CA

LIQUEFACTION ANALYSIS

Date 4/28/2011

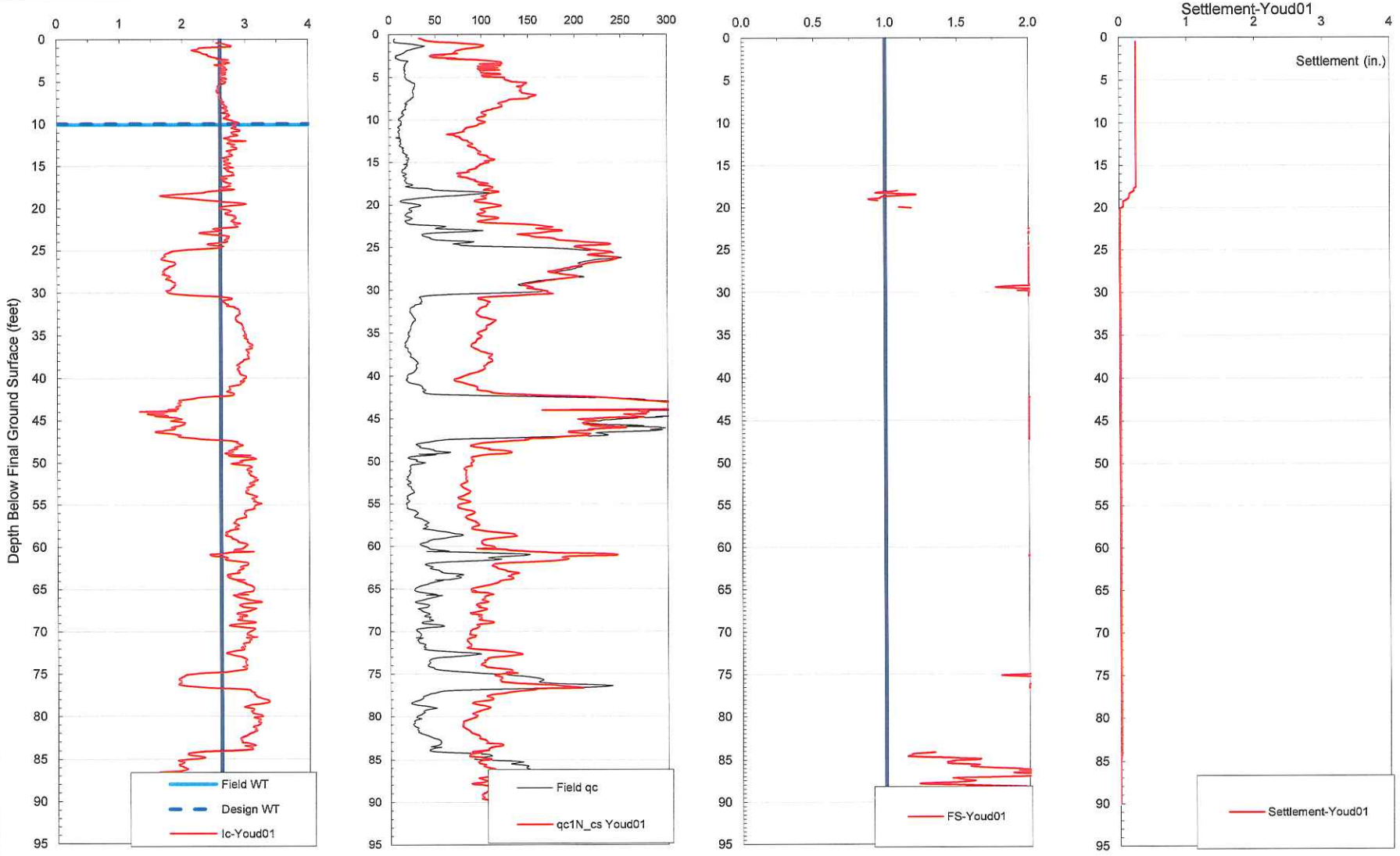
CPT ID: **CPT-11-003**

M_w= **6.7**
PGA= **0.32g**

Field Ground Water Depth (ft)= **10.1 ft**
Design Ground Water Depth (ft)= **10.0 ft**

Existing Ground Elevation= **0.0 ft**
Final Ground Elevation= **0.0 ft**

Ana. by: Hamid Parsa
Checked by: Carl Henderson



Note: Factor of safety is capped at 2.0 for liquefiable soils and not plotted for non-liquefiable soils & soils above the ground water; $CRR_{M=7.5,\ \sigma v'=1tsf}$ is capped at 1.0 for liqeufiable soils and not plotted for non-liquefiable soils and soils above the ground water.

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t: 209 948 1345 f: 209 234 4700

Project Name: Crosstown Freeway
Project No.: 111904.G01
Project Location: Stockton, CA

LIQUEFACTION ANALYSIS

Date 4/28/2011

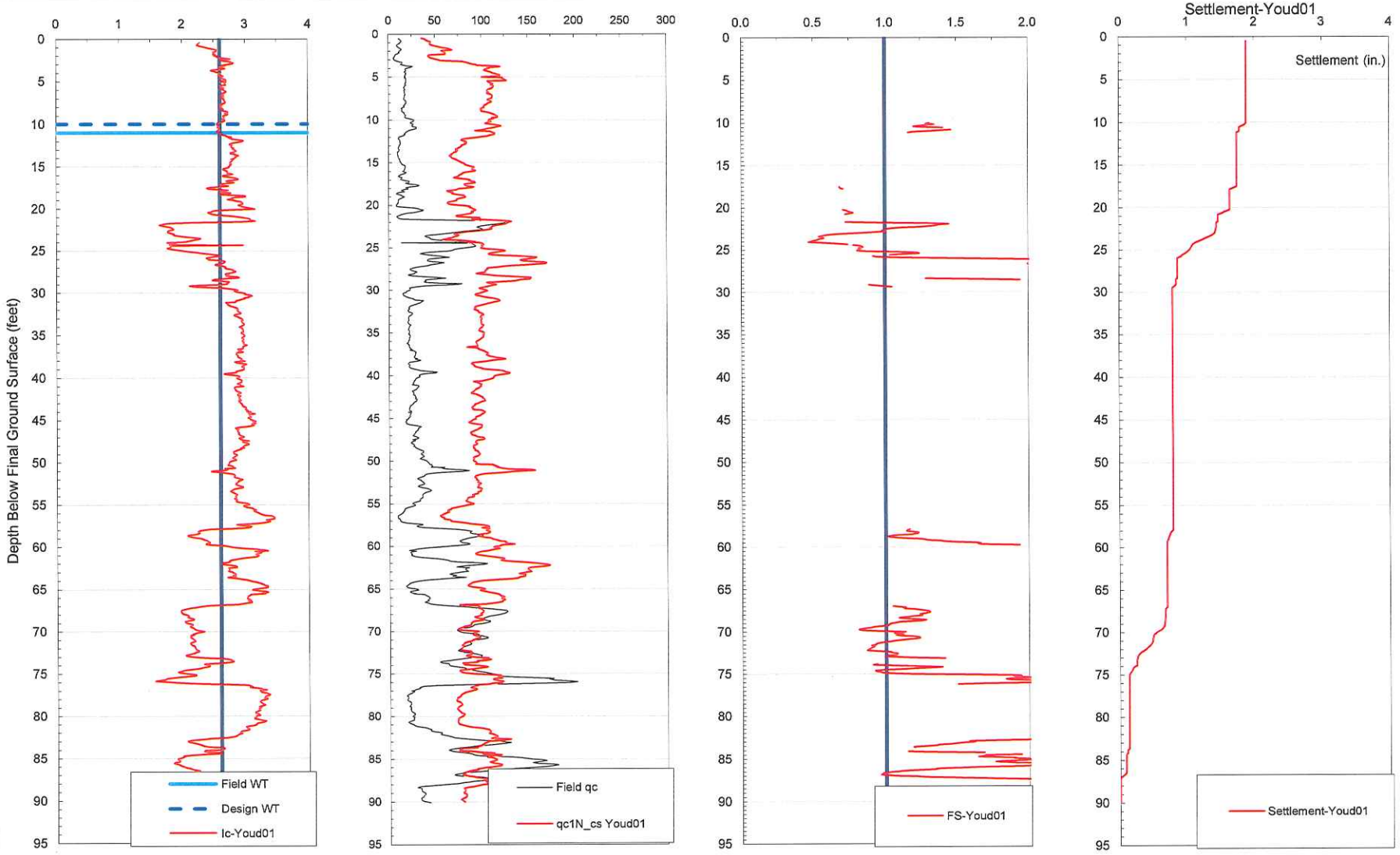
CPT ID: **CPT-11-004**

M_w= **6.7**
PGA= **0.32g**

Field Ground Water Depth (ft)= **11.0 ft**
Design Ground Water Depth (ft)= **10.0 ft**

Existing Ground Elevation= **0.0 ft**
Final Ground Elevation= **0.0 ft**

Ana. by: Hamid Parsa
Checked by: Carl Henderson



Note: Factor of safety is capped at 2.0 for liquefiable soils and not plotted for non-liquefiable soils & soils above the ground water; $CRR_{M=7.5,\ \sigma v'=\ 1tsf}$ is capped at 1.0 for liqeufiable soils and not plotted for non-liquefiable soils and soils above the ground water.

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Project Name: Crosstown Freeway
Project No.: 111904.G01
Project Location: Stockton, CA

LIQUEFACTION ANALYSIS

Date 4/29/2011

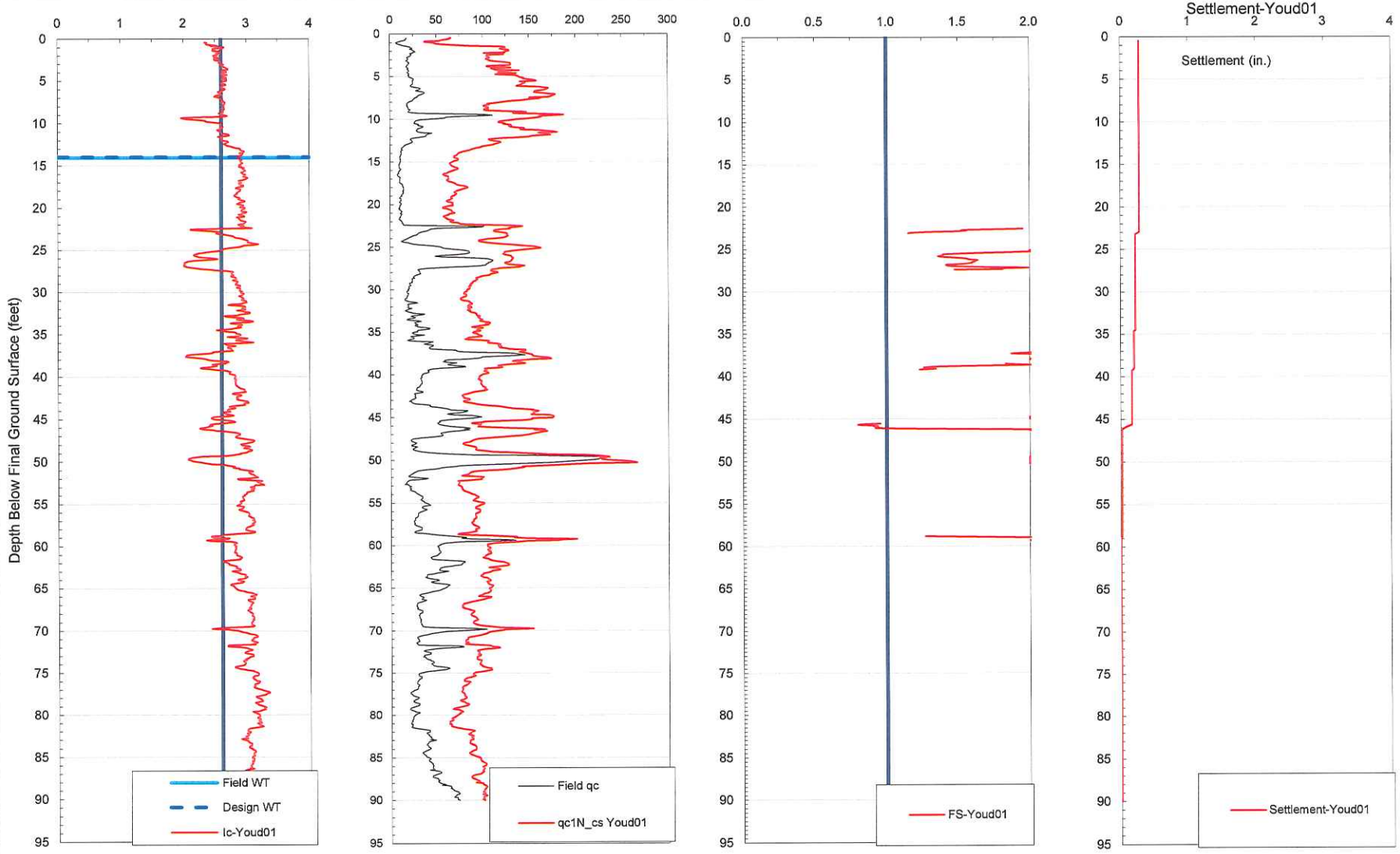
CPT ID: **CPT-11-005**

M_w= **6.7**
PGA= **0.32g**

Field Ground Water Depth (ft)= **14.1 ft**
Design Ground Water Depth (ft)= **14.0 ft**

Existing Ground Elevation= **0.0 ft**
Final Ground Elevation= **0.0 ft**

Ana. by: Hamid Parsa
Checked by: Carl Henderson



Note: Factor of safety is capped at 2.0 for liquefiable soils and not plotted for non-liquefiable soils & soils above the ground water; $CRR_{M=7.5,\ \sigma v'=1tsf}$ is capped at 1.0 for liqeufiable soils and not plotted for non-liquefiable soils and soils above the ground water.

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Project Name: Crosstown Freeway
Project No.: 111904.G01
Project Location: Stockton, CA

LIQUEFACTION ANALYSIS

Date 4/28/2011

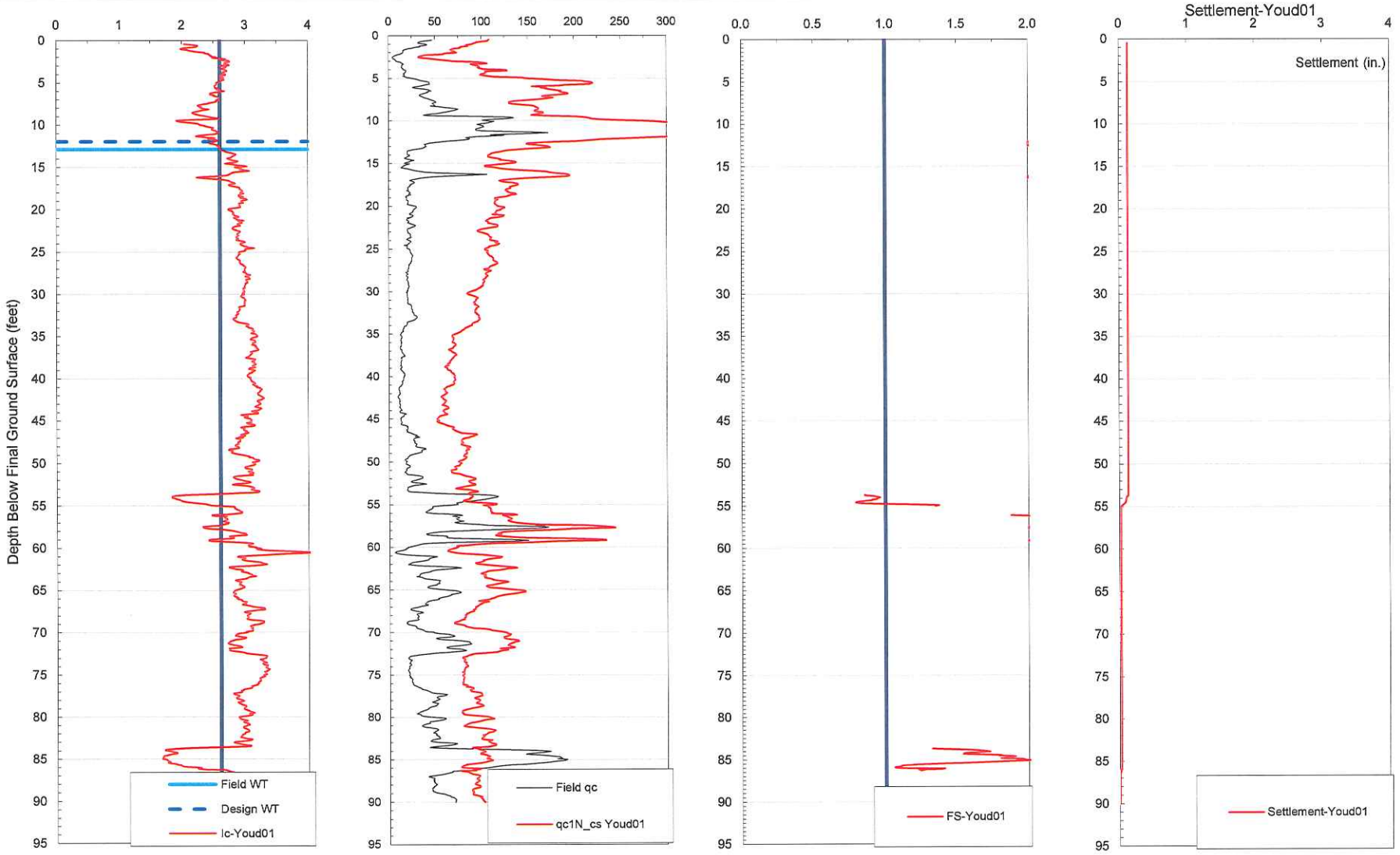
CPT ID: **CPT-11-006**

M_w= **6.7**
PGA= **0.32g**

Field Ground Water Depth (ft)= **12.9 ft**
Design Ground Water Depth (ft)= **12.0 ft**

Existing Ground Elevation= **0.0 ft**
Final Ground Elevation= **0.0 ft**

Ana. by: Hamid Parsa
Checked by: Carl Henderson



Note: Factor of safety is capped at 2.0 for liquefiable soils and not plotted for non-liquefiable soils & soils above the ground water; $CRR_{M=7.5,\ \sigma v'=1tsf}$ is capped at 1.0 for liqeufiable soils and not plotted for non-liquefiable soils and soils above the ground water.

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Project Name: Crosstown Freeway
Project No.: 111904.G01
Project Location: Stockton, CA

LIQUEFACTION ANALYSIS

Date 4/28/2011

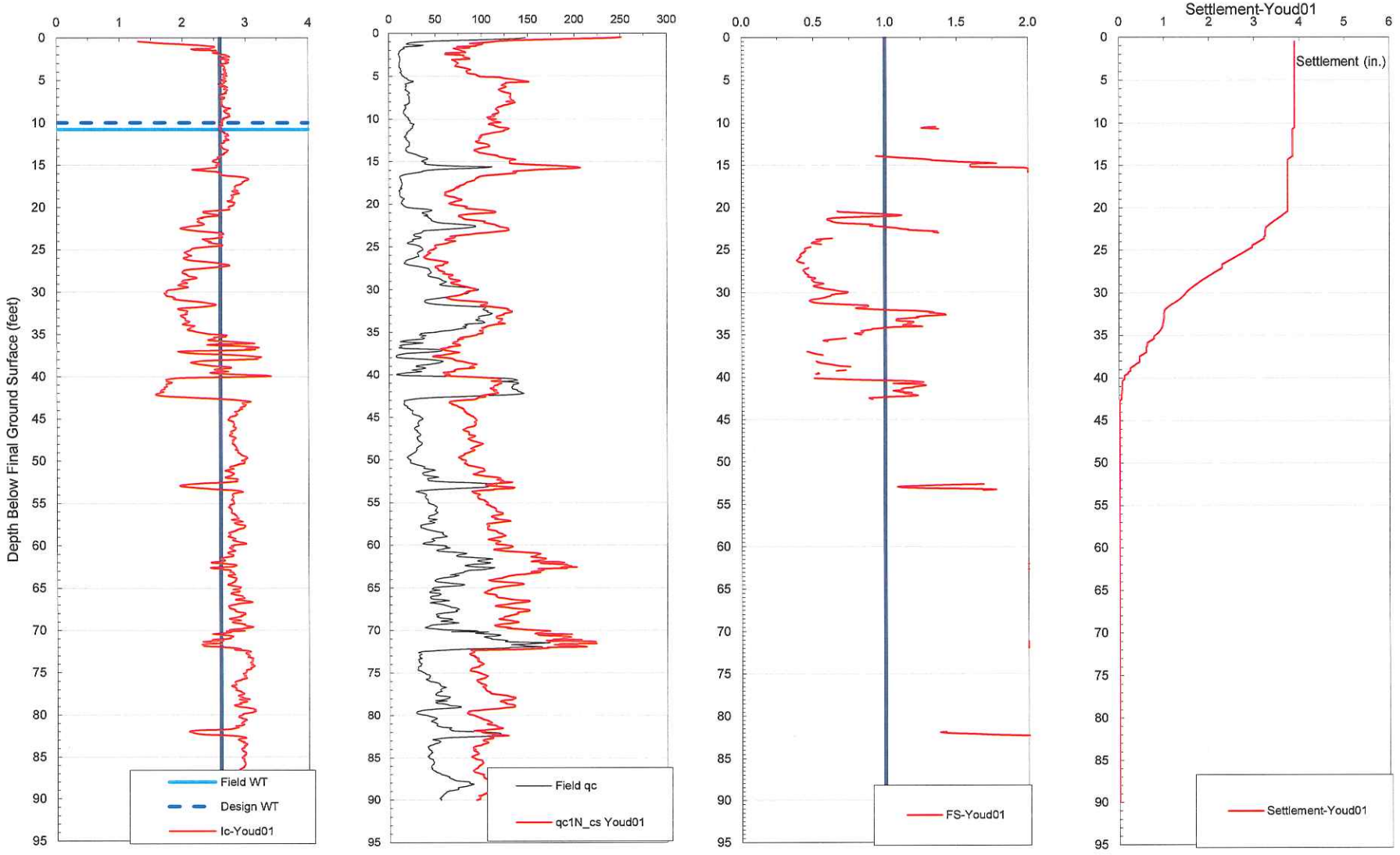
CPT ID: **CPT-11-007**

M_w= **6.7**
PGA= **0.32g**

Field Ground Water Depth (ft)= **10.8 ft**
Design Ground Water Depth (ft)= **10.0 ft**

Existing Ground Elevation= **0.0 ft**
Final Ground Elevation= **0.0 ft**

Ana. by: Hamid Parsa
Checked by: Carl Henderson



Note: Factor of safety is capped at 2.0 for liquefiable soils and not plotted for non-liquefiable soils & soils above the ground water; $CRR_{M=7.5,\ \sigma v'=\ 1tsf}$ is capped at 1.0 for liqeufiable soils and not plotted for non-liquefiable soils and soils above the ground water.

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Project Name: Crosstown Freeway
Project No.: 111904.G01
Project Location: Stockton, CA

LIQUEFACTION ANALYSIS

Date 4/29/2011

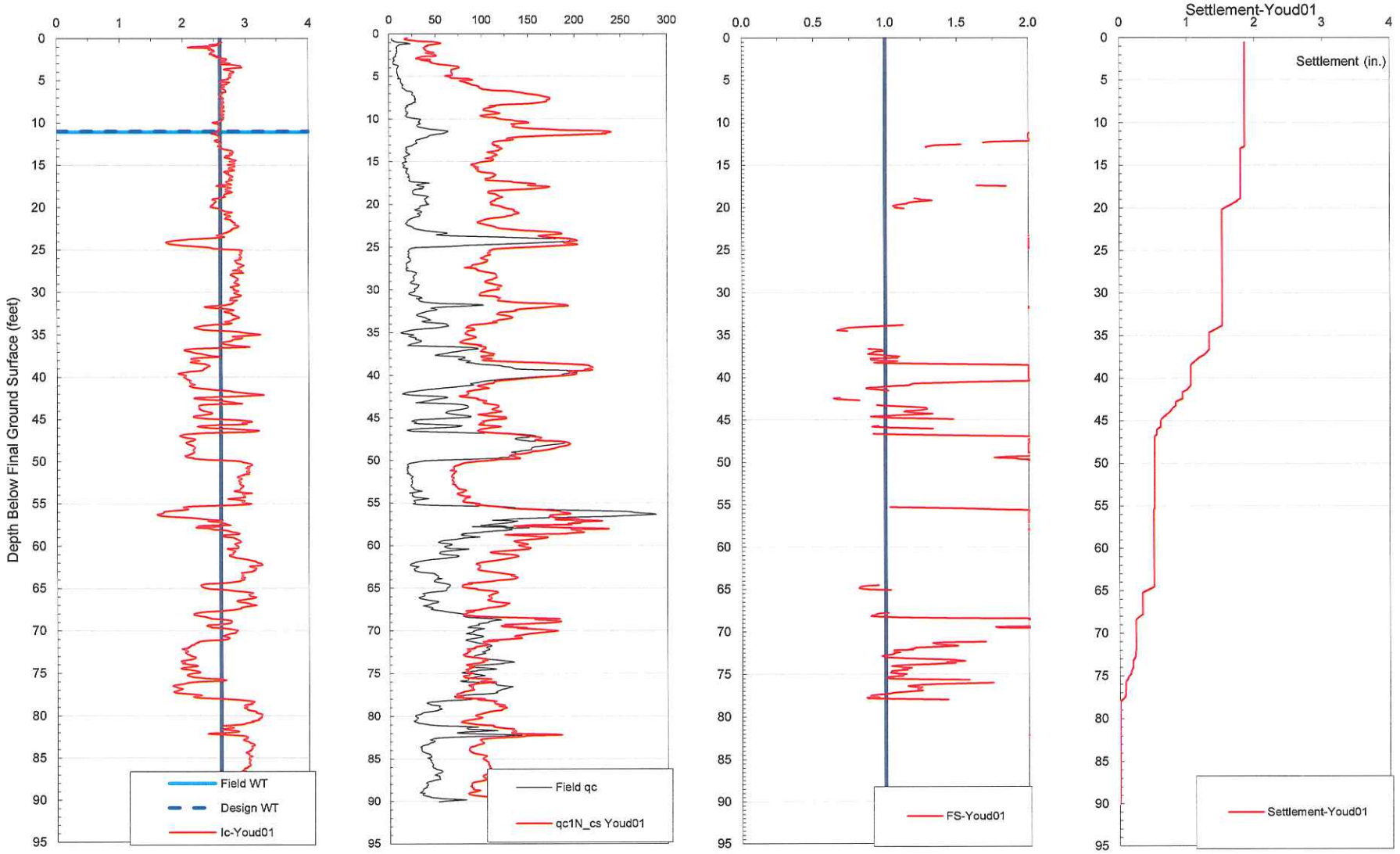
CPT ID: **CPT-11-008**

M_w= **6.7**
PGA= **0.32g**

Field Ground Water Depth (ft)= **11.1 ft**
Design Ground Water Depth (ft)= **11.0 ft**

Existing Ground Elevation= **0.0 ft**
Final Ground Elevation= **0.0 ft**

Ana. by: Hamid Parsa
Checked by: Carl Henderson



Note: Factor of safety is capped at 2.0 for liquefiable soils and not plotted for non-liquefiable soils & soils above the ground water; $CRR_{M=7.5,\ \sigma v'=1tsf}$ is capped at 1.0 for liqeufiable soils and not plotted for non-liquefiable soils and soils above the ground water.

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Project Name: Crosstown Freeway
Project No.: 111904.G01
Project Location: Stockton, CA

LIQUEFACTION ANALYSIS

Date 4/29/2011

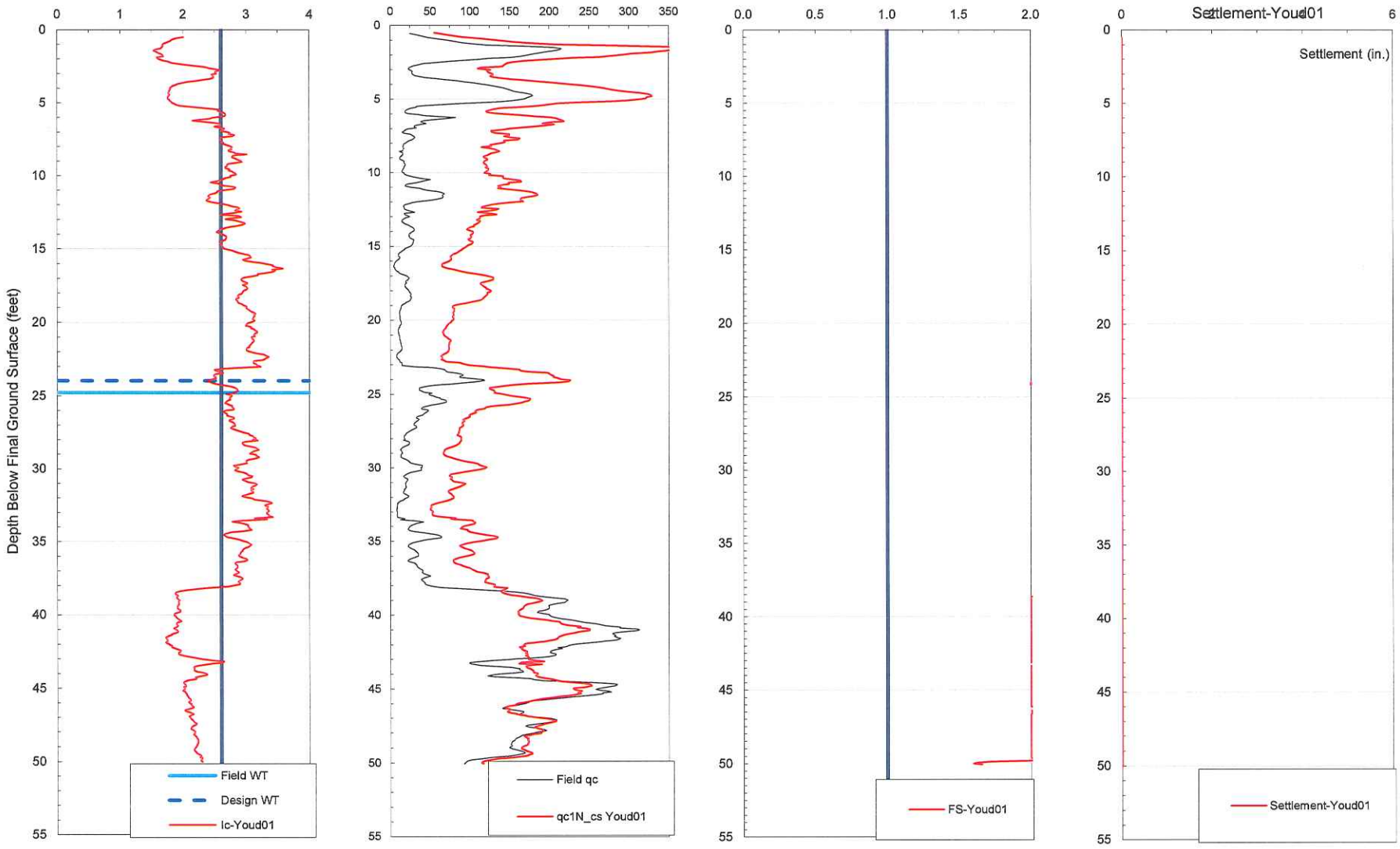
CPT ID: **CPT-11-009**

M_w= **6.7**
PGA= **0.32g**

Field Ground Water Depth (ft)= **24.8 ft**
Design Ground Water Depth (ft)= **24.0 ft**

Existing Ground Elevation= **0.0 ft**
Final Ground Elevation= **0.0 ft**

Ana. by: Hamid Parsa
Checked by: Cral Henderson



Note: Factor of safety is capped at 2.0 for liquefiable soils and not plotted for non-liquefiable soils & soils above the ground water; $CRR_{M=7.5,\ \sigma v'=\ 1tsf}$ is capped at 1.0 for liqeufiable soils and not plotted for non-liquefiable soils and soils above the ground water.

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Project Name: Crosstown Freeway
Project No.: 111904.G01
Project Location: Stockton, CA

LIQUEFACTION ANALYSIS

Date 5/2/2011

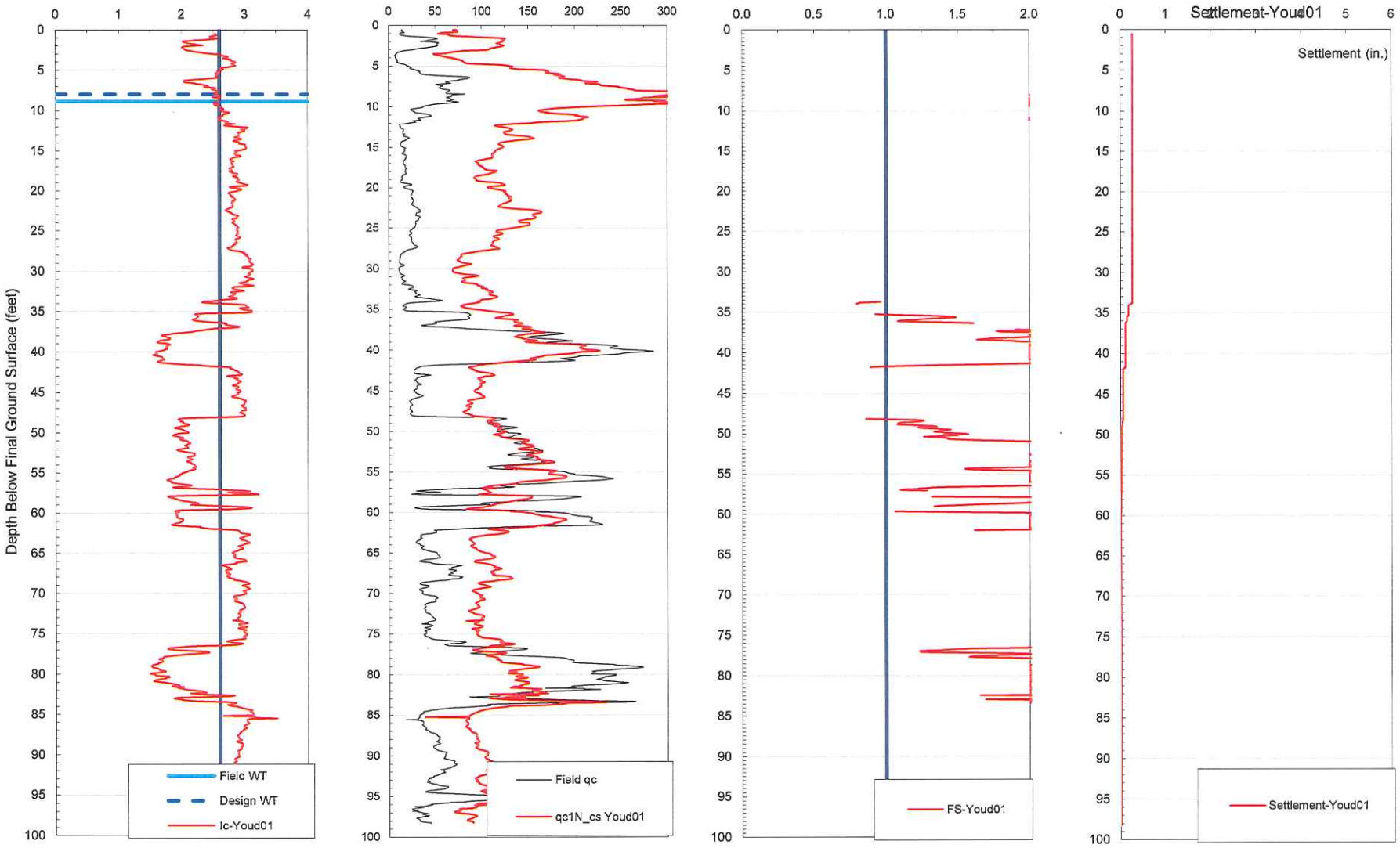
CPT ID: **CPT-11-10**

M_w= **6.7**
PGA= **0.32g**

Field Ground Water Depth (ft)= **8.9 ft**
Design Ground Water Depth (ft)= **8.0 ft**

Existing Ground Elevation= **0.0 ft**
Final Ground Elevation= **0.0 ft**

Ana. by: Hamid Parsa
Checked by: Carl Henderson



Note: Factor of safety is capped at 2.0 for liquefiable soils and not plotted for non-liquefiable soils & soils above the ground water; $CRR_{M=7.5,\ \sigma v'=\ 1tsf}$ is capped at 1.0 for liqeufiable soils and not plotted for non-liquefiable soils and soils above the ground water.

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Project Name: Crosstown Freeway
Project No.: 111904.G01
Project Location: Stockton, CA

LIQUEFACTION ANALYSIS

Date 5/2/2011

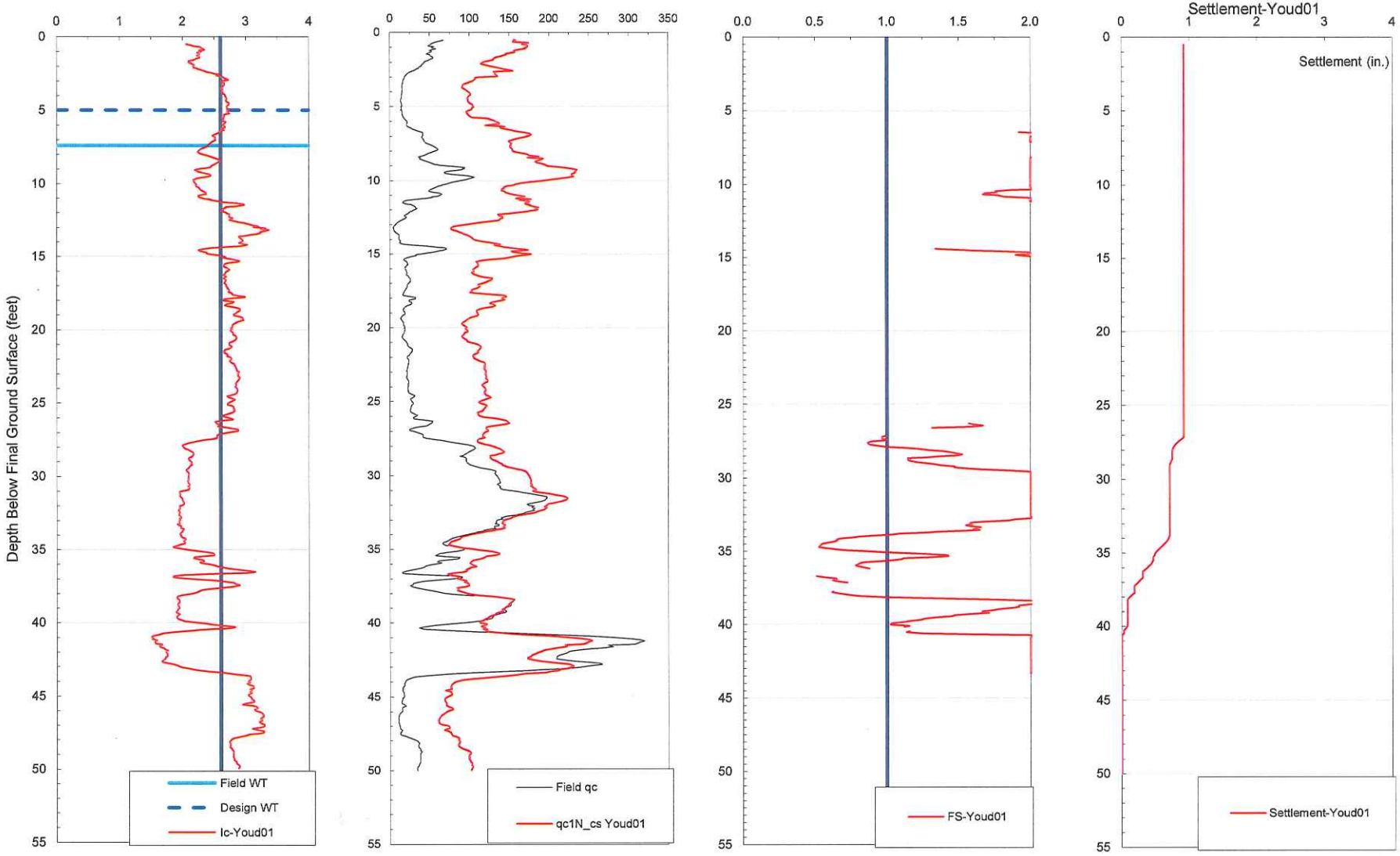
CPT ID: **CPT-11-011**

M_w= **6.7**
PGA= **0.32g**

Field Ground Water Depth (ft)= **7.4 ft**
Design Ground Water Depth (ft)= **5.0 ft**

Existing Ground Elevation= **0.0 ft**
Final Ground Elevation= **0.0 ft**

Ana. by: Hamid Parsa
Checked by: Carl Henderson



Note: Factor of safety is capped at 2.0 for liquefiable soils and not plotted for non-liquefiable soils & soils above the ground water; $CRR_{M=7.5,\ \sigma v'=\ 1tsf}$ is capped at 1.0 for liqeufiable soils and not plotted for non-liquefiable soils and soils above the ground water.

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2001 Arch Airport Road, #100
Stockton, California 95206
t: 209 948 1345 f: 209 234 4700

Project Name: Crosstown Freeway
Project No.: 111904.G01
Project Location: Stockton, CA

LIQUEFACTION ANALYSIS

Date 5/2/2011

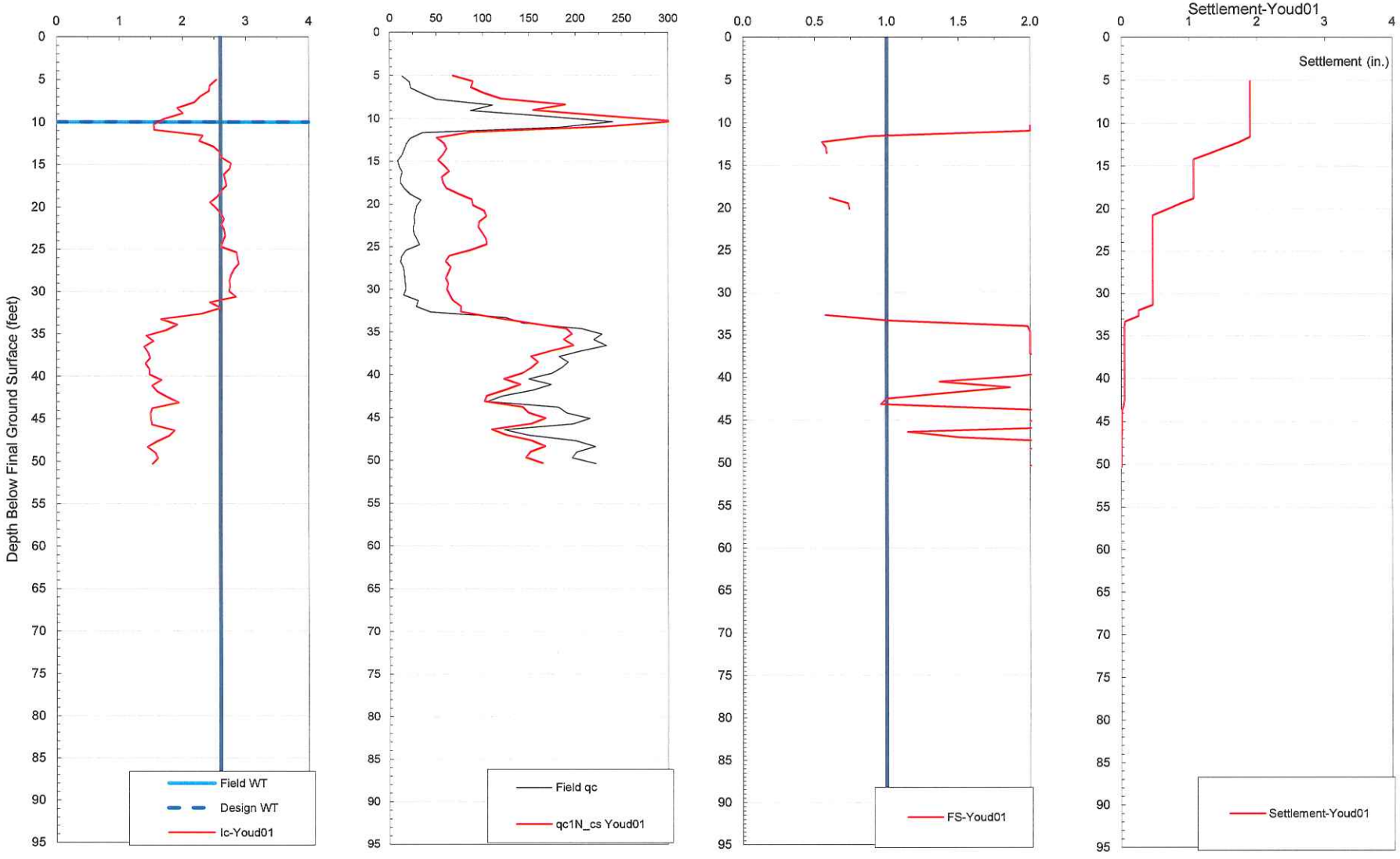
CPT ID: **CPT-09-001**

M_w= **6.7**
PGA= **0.32g**

Field Ground Water Depth (ft)= **10.0 ft**
Design Ground Water Depth (ft)= **10.0 ft**

Existing Ground Elevation= **0.0 ft**
Final Ground Elevation= **0.0 ft**

Ana. by: Carl Henderson
Checked by: Hamid Parsa



Note: Factor of safety is capped at 2.0 for liquefiable soils and not plotted for non-liquefiable soils & soils above the ground water; $CRR_{M=7.5,\ \sigma v'=\ 1tsf}$ is capped at 1.0 for liqeufiable soils and not plotted for non-liquefiable soils and soils above the ground water.

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2001 Arch Airport Road, #100
Stockton, California 95206
t: 209 948 1345 f: 209 234 4700

Project Name: Crosstown Freeway
Project No.: 111904.G01
Project Location: Stockton, CA

LIQUEFACTION ANALYSIS

Date 5/5/2011

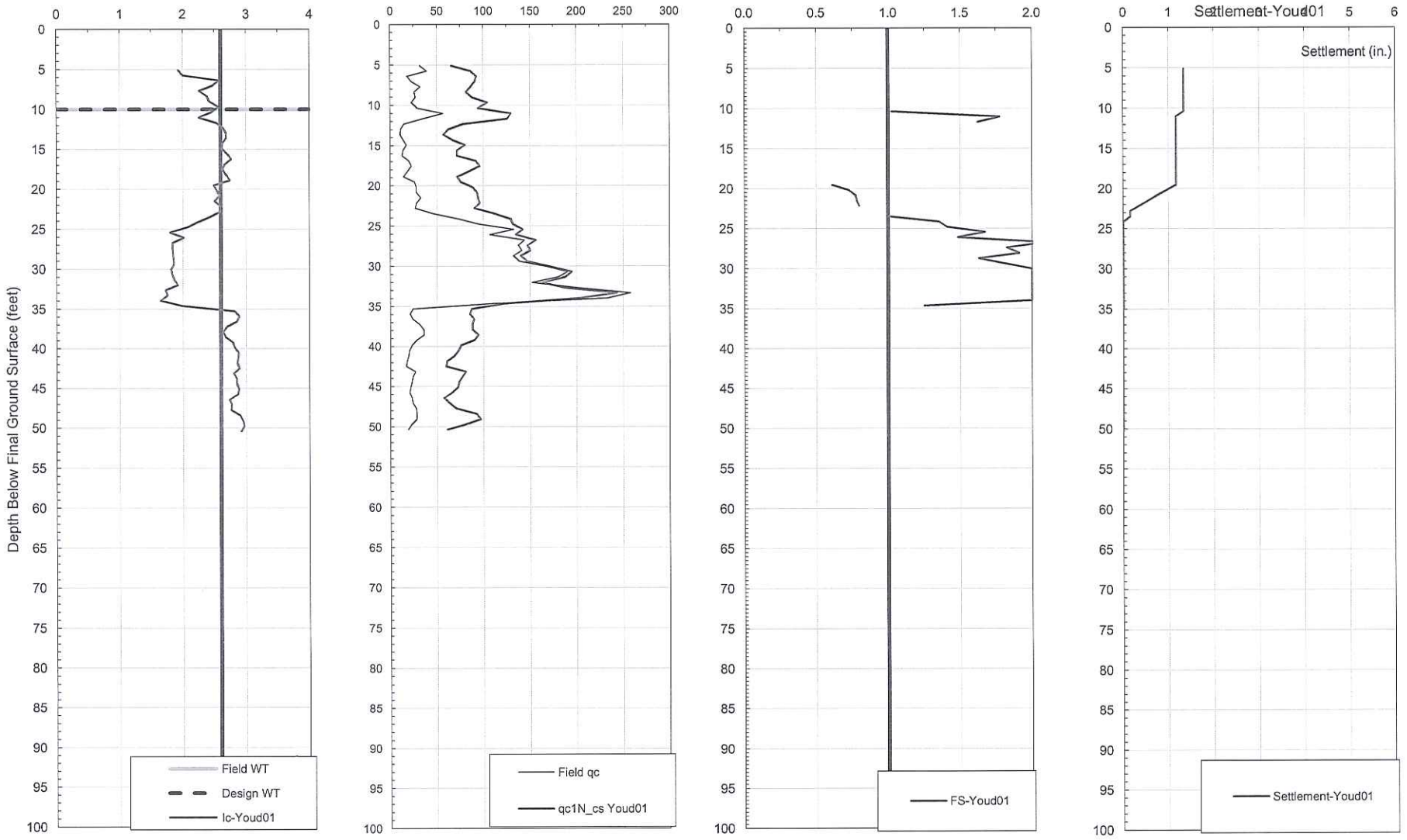
CPT ID: **CPT-09-002**

M_w= **6.7**
PGA= **0.32g**

Field Ground Water Depth (ft)= **10.0 ft**
Design Ground Water Depth (ft)= **10.0 ft**

Existing Ground Elevation= **0.0 ft**
Final Ground Elevation= **0.0 ft**

Ana. by: Hamid Parsa
Checked by: Carl Henderson



Note: Factor of safety is capped at 2.0 for liquefiable soils and not plotted for non-liquefiable soils & soils above the ground water; $CRR_{M=7.5,\ \sigma v'=1tsf}$ is capped at 1.0 for liqeufiable soils and not plotted for non-liquefiable soils and soils above the ground water.

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2001 Arch Airport Road, #100
Stockton, California 95206
t: 209 948 1345 f: 209 234 4700

Project Name: Crosstown Freeway
Project No.: 111904.G01
Project Location: Stockton, CA

LIQUEFACTION ANALYSIS

Date 7/13/2011

APPENDIX H
EMBANKMENT SETTLEMENT ANALYSIS AND GROUND IMPROVEMENT ALTERNATIVES

LIST OF ATTACHMENTS

The following are attached and complete this appendix.

Settlement Analysis and Ground Improvement Alternatives Summary
Subsurface Soil Model Profile and Analysis
Wick Drain Analysis
Ground Improvement Assessment
Settlement Reduction – Distance From Crest of Embankment/Toe of MSE Wall

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Settlement Analysis and Ground Improvement Alternatives Summary

SR-4 Crosstown Freeway Extension Project
Consolidation Settlement and Rate Analysis
and
Ground Improvement Cost Estimates

1. Introduction

The SR-4 project consists of construction of a main viaduct structure and approach embankments at its eastern and western ends. The eastern approach embankments will be constructed with side slopes of 4 (Horizontal):1(vertical). The western approach embankments (maximum height of 42 feet) will be constructed of three MSE walls (Nos. 1, 2, and 3) over existing grade with a total length of approximately 790 feet at the north edge (Wall #2) and 640 feet at the south edge (Walls #1 and 3).

2. Subsurface Soil Conditions

Review of boring logs indicates that the western and eastern approach embankments are generally underlain by stiff to very stiff lean clay/silt/clayey sand, interbedded with medium dense silty sand, to a depth of about 40 to 60 feet below existing grades. Below this layer are layers of very stiff to hard silt/lean clay and/or dense to very dense sandy soils to the maximum depths explored. Groundwater was encountered at a depth of about 7 to 14 feet at the western approach embankment below existing ground surface, and 11 to 36 feet below existing ground surface at the eastern approach embankment. Logs of borings and CPT's are presented in Appendices A and B. Generalized soil profiles for the settlement analysis (eastern approach embankment) are presented at the rear of this appendix.

Kleinfelder and Prikh, peformed consolidation tests on five selected soil samples. Results of consolidation testing are summarized in the table below.

Boring No.	Depth (ft)	Compression Ratio		Preconsolidation Pressure (psf)	Coeff. Of Consolidation C_v (ft2/day)	Approx. OCR
		NC	OC			
R-11-012	20	0.05	0.01	1,700	0.9	1.0
R-11-013	15	0.05	0.015	1,500	0.9	1.0
A-09-001	16	0.05	0.005	1,500	0.73 to 0.94	1.0
A-09-003	16	0.05	0.005	1,800	0.83 to 1.08	1.2
A-09-003	41	0.2	0.03	11,000	0.17 to 0.37	5.0

Notes:
1. Normally consolidated
2. Over consolidated

Based on correlations with moisture content per Lambe & Whitman (1969), soil compression ratios were estimated to be 0.05 to 0.15, which is slightly higher than the lab test results. In our settlement analysis later, we used compression ratios as summarized in the table above.

3. Estimation of Consolidation Settlements without Ground Improvement

We anticipate the settlement criteria for the proposed approach embankments will be established based on the required performance of pavement structure and adjacent bridge structures. On a preliminary basis, the settlement criteria for the approach embankments include a total post-construction settlement of 1 ½ inches over 100 feet.

Settlement of approach embankments typically consist of settlement of embankment itself due to self-weight, immediate settlement of foundations, and primary and secondary consolidation settlements of clayey soils. On a preliminary basis, the following assumptions have been made in the estimation of settlement of approach embankments:

- Embankment settlement due to its self weight is not included. In other words, settlement of embankment itself will complete during the construction phase, and will not contribute to the post-construction settlement of approach embankment;
- Embankment and surcharge loadings are applied instantaneously;
- Immediate settlement of foundations is not considered. We assumed that immediate settlement will occur during the construction phase, and its effect will be mitigated by overbuild the embankment sections during construction;
- Settlement of sandy soils (e.g., sand with silt, silty sand) is not estimated. We assumed the settlement of sandy soils will occur during the construction phase.
- Only primary consolidation of clayey soils will be considered. Secondary settlement of clayey soils is discounted in the estimation.

In the estimation of primary consolidation of clays/silts, the subsurface clay/silts are classified into two layers: The upper zone and the lower zone. We estimated consolidation settlements along the project alignment for soil profiles obtained from Borings R-11-013, R-11-012, and A-09-001; and CPT-11-011 and CPT-11-001. For illustration purposes, the table below presents post-construction settlement at MSE wall abutment #1, which corresponds to an embankment height of about 42 feet. Details of consolidation settlement analysis are presented later in this appendix.

West Approach Embankment.	Total Settlement (Upper Zone + Lower Zone) [1] (inches)	Post-Construction Settlement after 90 percent Consolidation (inches)	Time required for 90 percent consolidation [2] (days)		Remarks
			Upper Zone	Lower Zone	
A-09-001	13.2 (8+5)	1.3	109	109	

Notes:

1. Settlements considered for the Upper Zone and Lower Zone are primary consolidation only
2. Conservative values were used for Borings R-11-012, R-11-013 and A-09-001.

3. A fraction of settlements should occur during the construction of embankments. Therefore waiting periods are estimated by applying a reduction factor of 0.7 to the time required for 90 percent degree of consolidation.

It can be seen from the table above that:

1. Estimated maximum consolidation settlement is about 13 inches, depending on the underlying soil conditions. Considering the difference between actual field conditions and assumed soil conditions in our analysis, a reduction factor of about 0.70 to 0.85 should be adopted for a better estimation of consolidation settlements. Therefore, we anticipated that consolidation settlement will be on the order of about 10 inches for a 42-foot-high embankment.
2. The contribution of the lower clay/silt layer to the total settlement accounts for about 40 percent for the Borings R-11-013/R-11-012/A-09-001 profile.
3. Post-construction settlements after 90 percent consolidation are on the order of about 1.5 inches, which exceeds the tolerable settlements of 1 inch. Surcharge may be considered to deal with issue for high embankments (keep in mind that post-settlements for lower embankments may meet the post-construction settlement criteria).
4. Required time for 90 percent consolidation varies is about 3 to 4 months, depending on the permeability of underlying soils. It is also important to note that the field permeability of underlying soils is usually higher than tested in the laboratory.

4. Ground Improvement Recommendations

Typically, the following options have been considered in the past to control short- and long-term settlement of roadway embankment due to the consolidation of underlying soft to medium stiff silt/clay:

- Lightweight embankment fill (e.g., geoform, shredded tires)
- Wick drain with or without surcharge
- Stone Columns
- Geopiers
- Jetting Columns

The principal involved with the use of lightweight embankment fill (e.g., geofoam) is to reduce the magnitude of consolidation pressure and thus reduce the consolidation settlement. In the construction practice, geofoam has seen wide application and acceptance as a roadway construction material in the past years. The unit weight of geofoam is only about 1 to 2 pcf compared to typical soil unit weight of about 120 to 130 pcf. The use of geofoam as fill material needs to consider potential fire-related issues and

buoancy due to ground water. The geofoam is usually covered by a 3- to 5- foot layer of soils to ensure the appropriate performance of embankment.

The use of wick drains (prefabricated vertical drain) will accelerate the consolidation settlement rate. Depending on the spacing of wick drains, the time needed for primary consolidation to occur can be reduced by up to two thirds. It should be noted that the magnitude of consolidation settlement will not be reduced even with application of wick drains. Most commonly used wick drains are 4 inch wide with an equivalent drain diameter of 0.22 feet.

Stone columns and geopiers are similar in terms of expected performance. Application of stone columns and geopiers not only reduce the magnitude of consolidation of clays due to the increased stiffness of soils but also increase the bearing capacity of soils, which is beneficial especially for the stability of roadway approach embankment.

The approach embankments for the proposed project will be constructed predominately of MSE walls. Therefore, the use of lightweight fill such as geofoam will not be appropriate. Wick drains will be the most cost effective solution if time is not a significant factor in the project construction. Based on the estimation of consolidation settlement and rate, we recommend on a preliminary basis that wick drains and surcharge (if necessary, depending on the height of embankments) be used at the project site to improve the in-situ ground conditions).

Wick Drains

Effects of wick drains depend primarily on the spacing of wick drains. On a preliminary basis, we assumed a wick drain spacing of 6 feet to estimate the time required for 90 percent consolidation, based on the soil profiles obtained from R-11-012, R-11-013 and A-09-001. Results of our calculations are summarized in the table below. Details of consolidation rate analysis are presented later in this appendix.

West Approach Embankment.	Sample Depth (ft)	T_{90} (minutes)	Coeff. Of Consol. (ft2/day)	Time for 90% Consol. Without Wick drains (days)	Time for 90% Consol. With Wick drains (days)
Upper Bound Rate	16	2.25	0.94	85	32
Lower Bound Rate	16	2.89	0.73	109	41

It can be seen from the table above that time required for 90 percent degree of consolidation with wick drains varies from 32 to 41 days. In our analysis, we assumed the horizontal and vertical coefficients of consolidation are equal to each other. In the field condition, however, the horizontal permeability of the soils is usually 2 to 3 times

higher than the vertical permeability, resulting horizontal coefficient of consolidation 2 to 3 times higher than vertical one. Therefore, we anticipate that 90 percent degree of consolidation can be achieved along the most parts of the alignment by the end of the construction of embankments.

Geopiers

We considered 30-inch geopiers installed to a depth of about 30 feet below ground level at a center-to-center spacing of 5 to 7 feet. The settlement of the upper zone will be reduced to be about 2 inches due to the effects of geopier reinforcement. According to the consolidation settlement estimated for the "No Ground Improvement" option, we anticipated that the settlement of geopier-supported embankment would be on the order of about 4 to 7 inches.

5. Cost Estimation

The table below presents approximate cost estimates for the three ground improvement techniques. For the Preloading option, the cost will involve the placement and removal of earthen fill used to preload the foundation soils. The cost for this option varies, depending on the construction schedule, and sizes and sequences of areas selected for preloading.

Option No.	Ground Improvement technique	Approximate Costs ($)	Remarks
1	Preloading	N/A	
2	Wick Drains	305,000	
3	Geopiers	2,160,000	

Detailed cost estimation for Options No. 2 and 3 are presented later in the appendix.

Subsurface Soil Model Profile and Analysis

Prepared By: HP Date: 6/9/2011
Checked By: CH Date: 6/10/2011

Boring No. :	R-11-013		CPT-11-011	R-11-012		CPT-11-001
	Undrained Shear Strength (psf) Based on					
Depth (feet)	Blow Counts/ft	*Pocket Penetrometer data	CPT-11-011 Data	Blow Counts/ft	*Pocket Penetrometer data	CPT-11-001 Data
3	NT	NT	4,000	15	3,500	3,600
5	13	3,000	4,000	24	NT	2,100
10	NT	NT	7,000	9	2,500	4,900
15	37	3,500	3,200	10	3,500	1,700
20	8	3,500	2,800	NT	3,500	2,400
25	13	NT	5,000	30	NT	
30	16	3,500		9	NT	
35				15	4,000	2,200
40				12	NT	2,600
45			1,700	23	4,000	2,600
50			4,500	23	4,000	3,100
55						4,400
60						
65						4,300
70						8,000
75						4,500
80						4,800
85						5,000
90						2,360
95						N/T
100						N/T
110						N/T
120						

* Highest reading on pocket penetrometer for unconfined compression strength is

NT= Not Tested

= Sand

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Sheet 1 of 1

PROJECT CR 4 CROSSTOWN PROJECT NO. 111904. G01

SUBJECT R-11-013 / R-11-012 / A-09-001 BY YLI DATE 9/26/10

IDEALIZED SOIL PROFILE REVIEWED BY CFH DATE 10/07/10

REVISED BY CFH 7/15/11

Elev. 6

0 ▽ G.W (ASSUMED)

Sandy lean clay (CL)
Silt (ML)
Lean clay with sand (CL)
N = 9 ~ 20
LL = 26 - 30
PI = 3 - 20 w = (24 - 32)%

CR = 0.05 ~ 0.09 (Lambe and Whitman, 1969)

CR = 0.05 (KLEINFELDER & PARIKH CONSOLIDATION TEST DATA)

-20

Silty sand (SM) / Sand with Silt (SW-SM)
N = 10 ~ > 50/6"
w = (13 - 15)%

-30

Lean clay (CL) / with sand
N = 11 ~ 16
LL = 36 ~ 47
PI = 14 ~ 23
w = (28 - 33)%

CR = 0.07 ~ 0.1

-55

Non-Compressible
Silty Sand (SM)
N = 48 ~ > 63/6"

-70

SILT (ML), very stiff
N = 24
w = 31%

CR_recompression = 0.01

Elev. -80

Non-Compressible Base

SR-4 Crosstwon Freeway Extension Project

Consolidation Settlement Analysis for Approach Embankment

Prepared By: YLI Date 10/8/2010

Checked By: CFH Date 10/8/2010

REVISED BY: CFH DATE 7/13/2011

Based on Boring A-09-001 /R-11-012 / R-11-013

45 feet, Fill Height

100 feet, Fill Width

130 pcf, Fill density

Soil No.	Depth (ft) Top	Depth (ft) Bottom	Thickness (ft)	Mid Depth (ft)	Effective Unit Weight (pcf)	Weight of the Layer (psf)	Midlayer Effective Overburden pressure (psf)	Applied Pressure (psf)	Compression Ratio	Settlement (inches)
1	0	6	6	3	125	750	375	5680	0.0001	0.0
2	6	26	20	16	62.6	1252	1376	5043	0.05	8.0
3	26	36	10	31	62.6	626	2315	4466	0.0001	0.0
4	36	61	25	48.5	62.6	1565	3410.5	3939	0.05	5.0
5	61	76	15	68.5	62.6	939	4662.5	3472	0.0001	0.0
6	76	86	10	81	62.6	626	5445	3232	0.01	0.2
7	86	110	24	98	62.6	1502.4	6509.2	2955	0.0001	0.0
									Total Settlement	13.3

Fill Height (ft)	Consol. Settlement (inches)	70% of Estimated Consol. Settlement (inches)
5	2.6	1.82
10	4.7	3.29
15	6.4	4.48
20	7.9	5.53
25	9.2	6.44
30	10.4	7.28
35	11.4	7.98
40	12.4	8.68
45	13.3	9.31
50	14.1	9.87

Wick Drain Analysis

CONSOLIDATION-TIME CALCULATIONS ACCORDING TO BARRON-HANSBO EQUATION

ASSUMPTIONS:

1. Radial drainage only
2. Uniform soil deposit
3. No backpressure from horizontal drainage
4. Soil parameters constant in time and position
5. No drain resistance considered
6. No disturbance considered

INPUT			
Ch =	0.94	ft^2/day	
U =	90	%	# Days
Spacing 1 =	8	ft	63
Spacing 2 =	6	ft	32
Spacing 3 =	5	ft	20
Spacing 4 =	3	ft	6
Triangular		x	
Square			
Equivalent drain diameter =	0.22	ft	
(Recommend = 0.22 ft)			

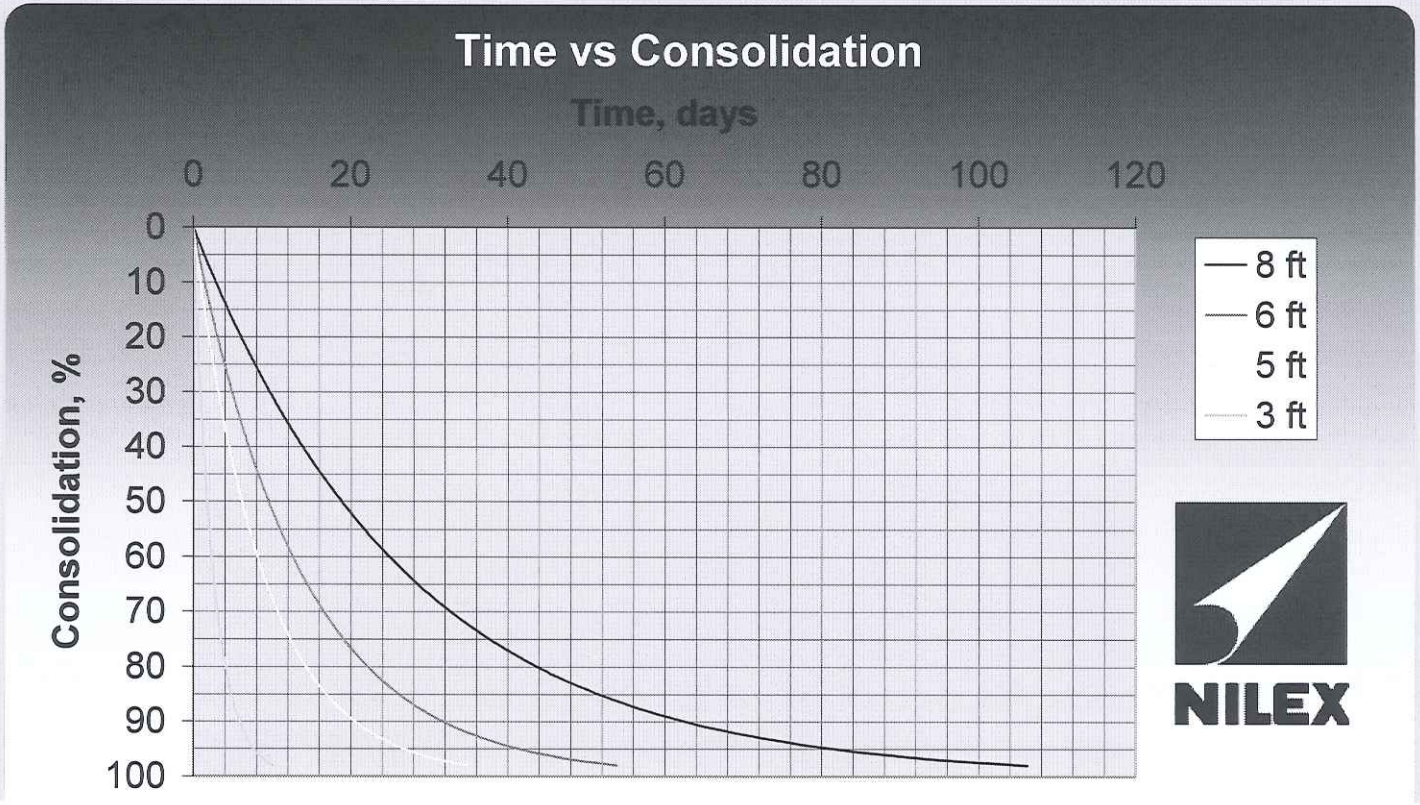

PLATE C-5

CONSOLIDATION-TIME CALCULATIONS ACCORDING TO BARRON-HANSBO EQUATION

ASSUMPTIONS:

1. Radial drainage only
2. Uniform soil deposit
3. No backpressure from horizontal drainage
4. Soil parameters constant in time and position
5. No drain resistance considered
6. No disturbance considered

INPUT			
Ch =	0.73	ft^2/day	
U =	90	%	# Days
Spacing 1 =	8	ft	81
Spacing 2 =	6	ft	41
Spacing 3 =	5	ft	26
Spacing 4 =	3	ft	8
Triangular	x		
Square			
Equivalent drain diameter =	0.22	ft	
(Recommend = 0.22 ft)			

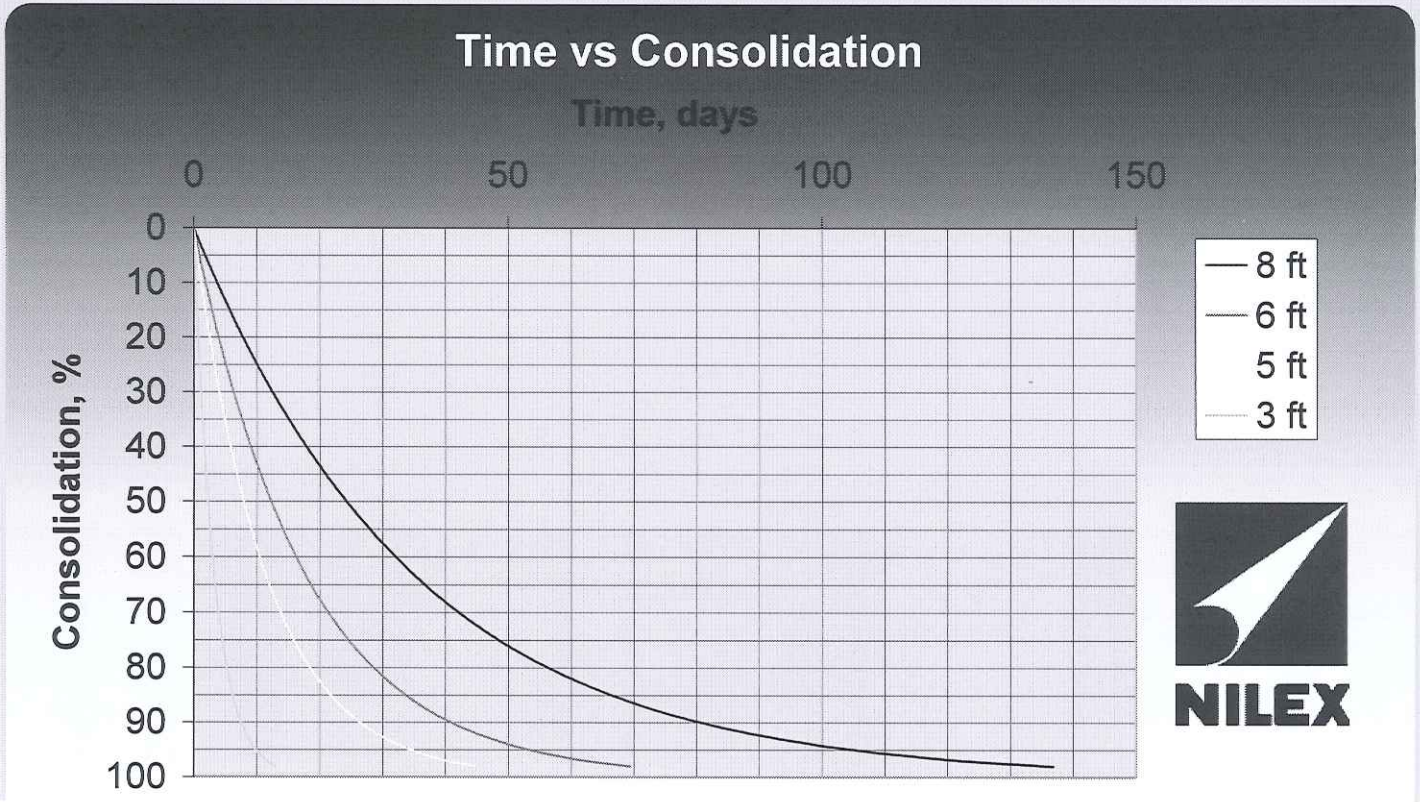

PLATE C-6

Ground Improvement Assessment

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Sheet 1 of 2

PROJECT SR-4 crosstown PROJECT NO. 111904. G01

SUBJECT Ground improvement BY YLi DATE 10/8/10

REVIEWED BY CFH DATE 10/8/10

Wick Drains

* 60 feet long each
* 5 feet spacing
* Western embankment: about 1000 ft long, wide 75 to 110 ft
  Eastern embankment: about 300 ft long wide 170 ft

Embankment	Row	column (along the alignment	PV length (ft)
WEST	100/5 = 20	1000/5 = 200	20 (201) (60) = 241,200
EAST	170/5 = 34	300/5 = 60	34 (61) (60) = 124,440
			= 365,640 ft

use Strip drains to discharge the water (instead of drain rock or sandy blanet):

Strip drains: 20,000 + 10200 = 30,200

west Embank East embank

Cost Analysis:

ITEM	UNIT PRICE	Quantity	COST
PV DRAIN	0.6/ft	365,640	219,384
STRIP DRAIN	1.31 ft	30,200	39,260
MOB	—	—	20,000
			278,644 (1.1) 306,508

Amplification factor 1.1

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Sheet 1 of 2

PROJECT SR-4 PROJECT NO. 111904.G01

SUBJECT Ground Improvement BY YC DATE 10/8/10

REVIEWED BY CEH DATE 10/8/10

Geopiers

* 30 feet long each
* 6 feet spacing (average)
* West Embankment: 1000 feet long by 100 feet wide (average)

East Embankment: 300 feet long by 170 feet wide

Embankment	Row	Column	Pier lengths (ft)
WEST	100/6 = 17	1000/6 = 167	17(168)(30) = 85,680
East	170/6 = 28	300/6 = 50	28(50)(30) = 42,840
			Total: 128,520 ft

Item	Unit price	Quantity	Cost ($)
Geopier	42 / ft	128,520	5397840

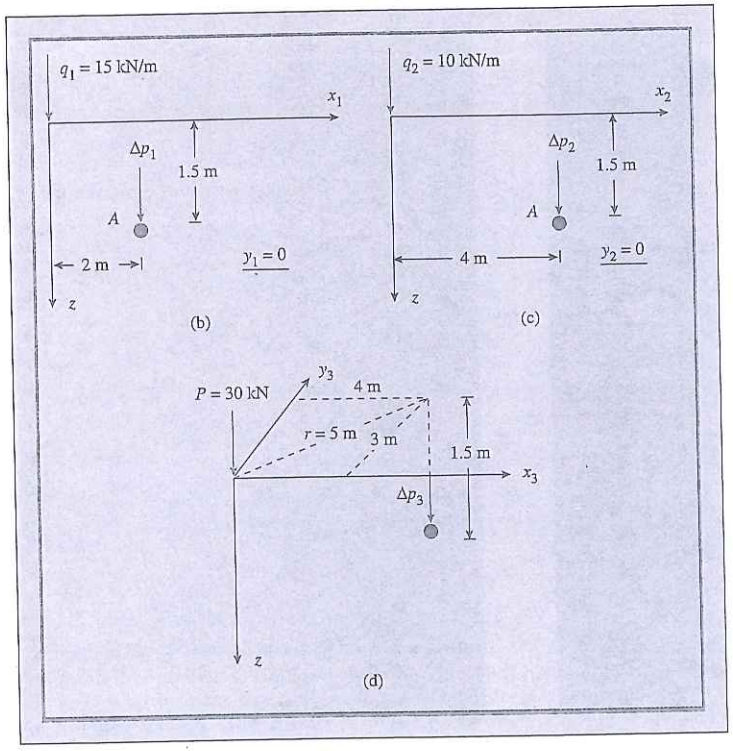
the calculations above are based on improvement across embankment foot prints. We anticipate foundations for embankment less than 15 ft to 20 high may not require ground improvement.

Embankment 15-20 ft high accounts for about 40 ~~to~~ 50 percent of all the approach embankments therefore,

(5,397,840)(40%) = $2,159,136

ENG-40 REV 07/09

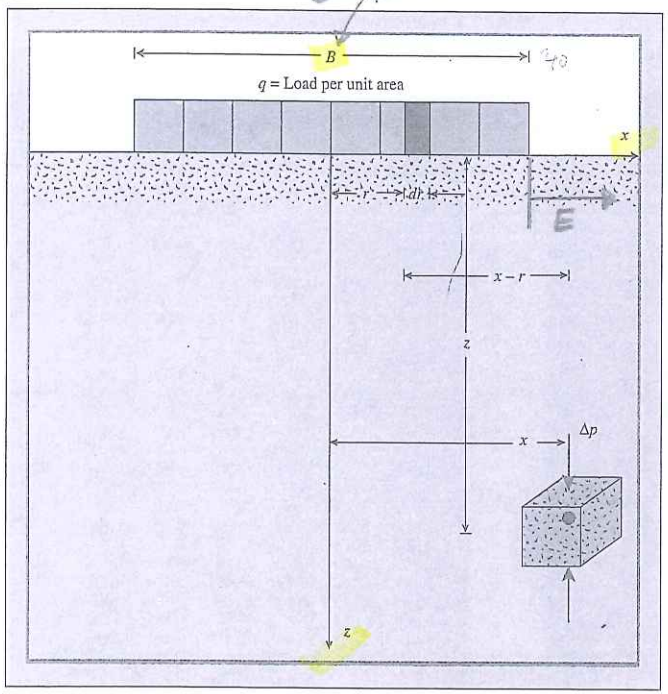
Settlement Reduction – Distance From Crest of Embankment/Toe of MSE Wall



▼ FIGURE 7.12 Continued

7.6 VERTICAL STRESS CAUSED BY A STRIP LOAD (FINITE WIDTH AND INFINITE LENGTH)

The fundamental equation for the vertical stress increase at a point in a soil mass as the result of a line load (Section 7.5) can be used to determine the vertical stress at a point caused by a flexible strip load of width B (Figure 7.13). Let the load per unit area of the strip shown in Figure 7.13 be equal to q. If we consider an elemental strip of width dr, the load per unit length of this strip is equal to $q\,dr$. This elemental strip can be treated as a line load. Equation (7.18) gives the vertical stress increase, dp, at point A inside the soil mass caused by this elemental strip load. To calculate the vertical stress increase, we need to substitute $q\,dr$ for q and $(x - r)$ for x. So,



▼ FIGURE 7.13 Vertical stress caused by a flexible strip load

The total increase in the vertical stress (Δp) at point A caused by the entire strip load of width B can be determined by integration of Eq. (7.20) with limits of r from $-B/2$ to $+B/2$, or

$$\Delta p = \int dp = \int_{-B/2}^{+B/2} \left(\frac{2q}{\pi}\right)\left\{\frac{z^3}{[(x-r)^2+z^2]^2}\right\} dr$$

$$= \frac{q}{\pi}\left\{\tan^{-1}\left[\frac{z}{x-(B/2)}\right] - \tan^{-1}\left[\frac{z}{x+(B/2)}\right] - \frac{Bz[x^2 - z^2 - (B^2/4)]}{[x^2+z^2-(B^2/4)]+B^2z^2}\right\} \quad (7.21)$$

Table 7.3 shows the variation of $\Delta p/q$ with $2z/B$ for $2x/B$. This table can be

BELOW SOIL DEPTH OF 90 FT SOIL SETTLEMENT IS ASSUMED AS NEGLIGIBLE

B=110 FT

E = DISTANCE FROM EDGE OF MSE WALL/EMBANKMENT (FT)

▼ TABLE 7.3 Variation of $\Delta p/q$ with $2z/B$ and $2x/B$ [Eq. (7.21)]

	2x/B										
2z/B	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
0.00	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	0.000
0.10	1.000	1.000	0.999	0.999	0.999	0.998	0.997	0.993	0.980	0.909	0.500
0.20	0.997	0.997	0.996	0.995	0.992	0.988	0.979	0.959	0.909	0.775	0.500
0.30	0.990	0.989	0.987	0.984	0.978	0.967	0.947	0.908	0.833	0.697	0.499
0.40	0.977	0.976	0.973	0.966	0.955	0.937	0.906	0.855	0.773	0.651	0.498
0.50	0.959	0.958	0.953	0.943	0.927	0.902	0.864	0.808	0.727	0.620	0.497
0.60	0.937	0.935	0.928	0.915	0.896	0.866	0.825	0.767	0.691	0.598	0.495
0.70	0.910	0.908	0.899	0.885	0.863	0.831	0.788	0.732	0.662	0.581	0.492
0.80	0.881	0.878	0.869	0.853	0.829	0.797	0.755	0.701	0.638	0.566	0.489
0.90	0.850	0.847	0.837	0.821	0.797	0.765	0.724	0.675	0.617	0.552	0.485
1.00	0.818	0.815	0.805	0.789	0.766	0.735	0.696	0.650	0.598	0.540	0.480
1.10	0.787	0.783	0.774	0.758	0.735	0.706	0.670	0.628	0.580	0.529	0.474
1.20	0.755	0.752	0.743	0.728	0.707	0.679	0.646	0.607	0.564	0.517	0.468
1.30	0.725	0.722	0.714	0.699	0.679	0.654	0.623	0.588	0.548	0.506	0.462
1.40	0.696	0.693	0.685	0.672	0.653	0.630	0.602	0.569	0.534	0.495	0.455
1.50	0.668	0.666	0.658	0.646	0.629	0.607	0.581	0.552	0.519	0.484	0.448
1.60	0.642	0.639	0.633	0.621	0.605	0.586	0.562	0.535	0.506	0.474	0.440
1.70	0.617	0.615	0.608	0.598	0.583	0.565	0.544	0.519	0.492	0.463	0.433
1.80	0.593	0.591	0.585	0.576	0.563	0.546	0.526	0.504	0.479	0.453	0.425
1.90	0.571	0.569	0.564	0.555	0.543	0.528	0.510	0.489	0.467	0.443	0.417
2.00	0.550	0.548	0.543	0.535	0.524	0.510	0.494	0.475	0.455	0.433	0.409
2.10	0.530	0.529	0.524	0.517	0.507	0.494	0.479	0.462	0.443	0.423	0.401
2.20	0.511	0.510	0.506	0.499	0.490	0.479	0.465	0.449	0.432	0.413	0.393
2.30	0.494	0.493	0.489	0.483	0.474	0.464	0.451	0.437	0.421	0.404	0.385
2.40	0.477	0.476	0.473	0.467	0.460	0.450	0.438	0.425	0.410	0.395	0.378
2.50	0.462	0.461	0.458	0.452	0.445	0.436	0.426	0.414	0.400	0.386	0.370
2.60	0.447	0.446	0.443	0.439	0.432	0.424	0.414	0.403	0.390	0.377	0.363
2.70	0.433	0.432	0.430	0.425	0.419	0.412	0.403	0.393	0.381	0.369	0.355
2.80	0.420	0.419	0.417	0.413	0.407	0.400	0.392	0.383	0.372	0.360	0.348
2.90	0.408	0.407	0.405	0.401	0.396	0.389	0.382	0.373	0.363	0.352	0.341
3.00	0.396	0.395	0.393	0.390	0.385	0.379	0.372	0.364	0.355	0.345	0.334
3.10	0.385	0.384	0.382	0.379	0.375	0.369	0.363	0.355	0.347	0.337	0.327
3.20	0.374	0.373	0.372	0.369	0.365	0.360	0.354	0.347	0.339	0.330	0.321
3.30	0.364	0.363	0.362	0.359	0.355	0.351	0.345	0.339	0.331	0.323	0.315
3.40	0.354	0.354	0.352	0.350	0.346	0.342	0.337	0.331	0.324	0.316	0.308
3.50	0.345	0.345	0.343	0.341	0.338	0.334	0.329	0.323	0.317	0.310	0.302
3.60	0.337	0.336	0.335	0.333	0.330	0.326	0.321	0.316	0.310	0.304	0.297
3.70	0.328	0.328	0.327	0.325	0.322	0.318	0.314	0.309	0.304	0.298	0.291
3.80	0.320	0.320	0.319	0.317	0.315	0.311	0.307	0.303	0.297	0.292	0.285
3.90	0.313	0.313	0.312	0.310	0.307	0.304	0.301	0.296	0.291	0.286	0.280
4.00	0.306	0.305	0.304	0.303	0.301	0.298	0.294	0.290	0.285	0.280	0.275
4.10	0.299	0.299	0.298	0.296	0.294	0.291	0.288	0.284	0.280	0.275	0.270
4.20	0.292	0.292	0.291	0.290	0.288	0.285	0.282	0.278	0.274	0.270	0.265
4.30	0.286	0.286	0.285	0.283	0.282	0.279	0.276	0.273	0.269	0.265	0.260
4.40	0.280	0.280	0.279	0.278	0.276	0.274	0.271	0.268	0.264	0.260	0.256
4.50	0.274	0.274	0.273	0.272	0.270	0.268	0.266	0.263	0.259	0.255	0.251
4.60	0.268	0.268	0.268	0.266	0.265	0.263	0.260	0.258	0.254	0.251	0.247
4.70	0.263	0.263	0.262	0.261	0.260	0.258	0.255	0.253	0.250	0.246	0.243
4.80	0.258	0.258	0.257	0.256	0.255	0.253	0.251	0.248	0.245	0.242	0.239
4.90	0.253	0.253	0.252	0.251	0.250	0.248	0.246	0.244	0.241	0.238	0.235
5.00	0.248	0.248	0.247	0.246	0.245	0.244	0.242	0.239	0.237	0.234	0.231

(continued)

▼ TABLE 7.3 Continued

	2x/B									
2z/B	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.9	2.0
0.00	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.10	0.091	0.020	0.007	0.003	0.002	0.001	0.001	0.000	0.000	0.000
0.20	0.225	0.091	0.040	0.020	0.011	0.007	0.004	0.003	0.002	0.002
0.30	0.301	0.165	0.090	0.052	0.031	0.020	0.013	0.009	0.007	0.005
0.40	0.346	0.224	0.141	0.090	0.059	0.040	0.027	0.020	0.014	0.011
0.50	0.373	0.267	0.185	0.128	0.089	0.063	0.046	0.034	0.025	0.019
0.60	0.391	0.298	0.222	0.163	0.120	0.088	0.066	0.050	0.038	0.030
0.70	0.403	0.321	0.250	0.193	0.148	0.113	0.087	0.068	0.053	0.042
0.80	0.411	0.338	0.273	0.218	0.173	0.137	0.108	0.086	0.069	0.056
0.90	0.416	0.351	0.291	0.239	0.195	0.158	0.128	0.104	0.085	0.070
1.00	0.419	0.360	0.305	0.256	0.214	0.177	0.147	0.122	0.101	0.084
1.10	0.420	0.366	0.316	0.271	0.230	0.194	0.164	0.138	0.116	0.098
1.20	0.419	0.371	0.325	0.282	0.243	0.209	0.178	0.152	0.130	0.111
1.30	0.417	0.373	0.331	0.291	0.254	0.221	0.191	0.166	0.143	0.123
1.40	0.414	0.374	0.335	0.298	0.263	0.232	0.203	0.177	0.155	0.135
1.50	0.411	0.374	0.338	0.303	0.271	0.240	0.213	0.188	0.165	0.146
1.60	0.407	0.373	0.339	0.307	0.276	0.248	0.221	0.197	0.175	0.155
1.70	0.402	0.370	0.339	0.309	0.281	0.254	0.228	0.205	0.183	0.164
1.80	0.396	0.368	0.339	0.311	0.284	0.258	0.234	0.212	0.191	0.172
1.90	0.391	0.364	0.338	0.312	0.286	0.262	0.239	0.217	0.197	0.179
2.00	0.385	0.360	0.336	0.311	0.288	0.265	0.243	0.222	0.203	0.185
2.10	0.379	0.356	0.333	0.311	0.288	0.267	0.246	0.226	0.208	0.190
2.20	0.373	0.352	0.330	0.309	0.288	0.268	0.248	0.229	0.212	0.195
2.30	0.366	0.347	0.327	0.307	0.288	0.268	0.250	0.232	0.215	0.199
2.40	0.360	0.342	0.323	0.305	0.287	0.268	0.251	0.234	0.217	0.202
2.50	0.354	0.337	0.320	0.302	0.285	0.268	0.251	0.235	0.220	0.205
2.60	0.347	0.332	0.316	0.299	0.283	0.267	0.251	0.236	0.221	0.207
2.70	0.341	0.327	0.312	0.296	0.281	0.266	0.251	0.236	0.222	0.208
2.80	0.335	0.321	0.307	0.293	0.279	0.265	0.250	0.236	0.223	0.210
2.90	0.329	0.316	0.303	0.290	0.276	0.263	0.249	0.236	0.223	0.211
3.00	0.323	0.311	0.299	0.286	0.274	0.261	0.248	0.236	0.223	0.211
3.10	0.317	0.306	0.294	0.283	0.271	0.259	0.247	0.235	0.223	0.212
3.20	0.311	0.301	0.290	0.279	0.268	0.256	0.245	0.234	0.223	0.212
3.30	0.305	0.296	0.286	0.275	0.265	0.254	0.243	0.232	0.222	0.211
3.40	0.300	0.291	0.281	0.271	0.261	0.251	0.241	0.231	0.221	0.211
3.50	0.294	0.286	0.277	0.268	0.258	0.249	0.239	0.229	0.220	0.210
3.60	0.289	0.281	0.273	0.264	0.255	0.246	0.237	0.228	0.218	0.209
3.70	0.284	0.276	0.268	0.260	0.252	0.243	0.235	0.226	0.217	0.208
3.80	0.279	0.272	0.264	0.256	0.249	0.240	0.232	0.224	0.216	0.207
3.90	0.274	0.267	0.260	0.253	0.245	0.238	0.230	0.222	0.214	0.206
4.00	0.269	0.263	0.256	0.249	0.242	0.235	0.227	0.220	0.212	0.205
4.10	0.264	0.258	0.252	0.246	0.239	0.232	0.225	0.218	0.211	0.203
4.20	0.260	0.254	0.248	0.242	0.236	0.229	0.222	0.216	0.209	0.202
4.30	0.255	0.250	0.244	0.239	0.233	0.226	0.220	0.213	0.207	0.200
4.40	0.251	0.246	0.241	0.235	0.229	0.224	0.217	0.211	0.205	0.199
4.50	0.247	0.242	0.237	0.232	0.226	0.221	0.215	0.209	0.203	0.197
4.60	0.243	0.238	0.234	0.229	0.223	0.218	0.212	0.207	0.201	0.195
4.70	0.239	0.235	0.230	0.225	0.220	0.215	0.210	0.205	0.199	0.194
4.80	0.235	0.231	0.227	0.222	0.217	0.213	0.208	0.202	0.197	0.192
4.90	0.231	0.227	0.223	0.219	0.215	0.210	0.205	0.200	0.195	0.190
5.00	0.227	0.224	0.220	0.216	0.212	0.207	0.203	0.198	0.193	0.188

DEPTH (FT)	2z/B
5	0.09
10	0.18
15	0.27
20	0.36
25	0.45
30	0.55
35	0.64
40	0.73
45	0.82
50	0.91
55	1
60	1.1
65	1.18
70	1.27
75	1.36
80	1.45
85	1.55
90	1.64
95	1.73
100	

E	X	2X/B
0	55	1
25	80	1.45
50	105	1.91
75	130	2.36
100	155	2.82
125	180	3.27
150	205	3.72
10	60	1.09

* E(FT) | APPROXIMATE ASSUMED REDUCTION IN SETTLEMENT

E(FT)	
10	50%
25	75%
50	90%

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APPENDIX I
MSE WALL EXTERNAL AND GLOBAL STABILITY ANALYSIS

LIST OF ATTACHMENTS

The following are attached and complete this appendix.

Conceptual Retaining Wall Profile
Subsurface Soil Model Profile
External Stability Check Calculations
Global Stability Analysis – Static Condition
Global Stability Analysis – Seismic Condition

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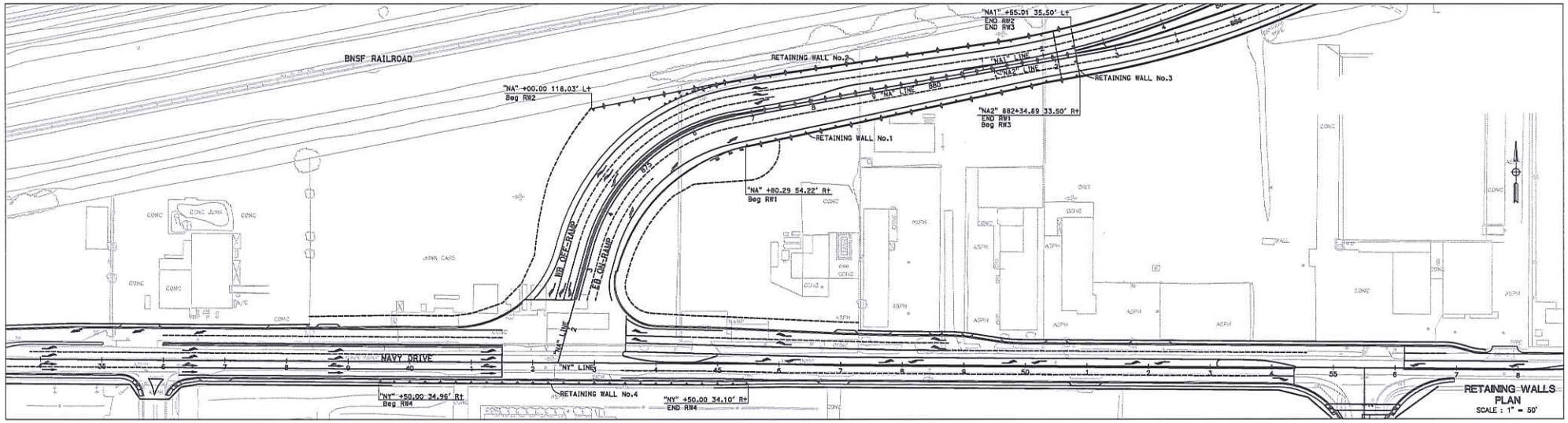
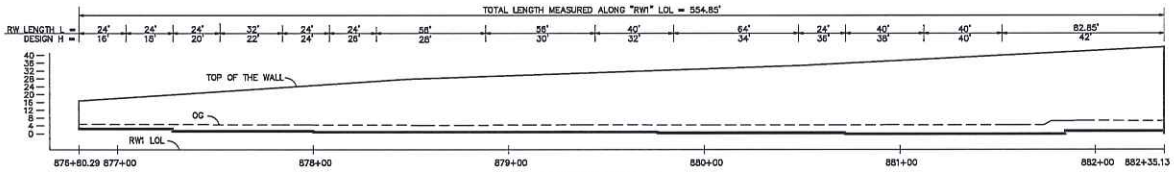
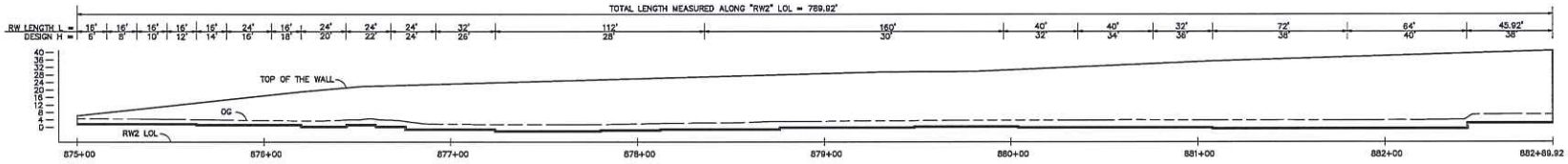
Conceptual Retaining Wall Profile



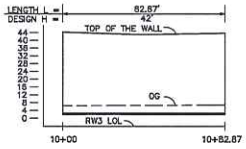
RETAINING WALLS PLAN
SCALE : 1" = 50'



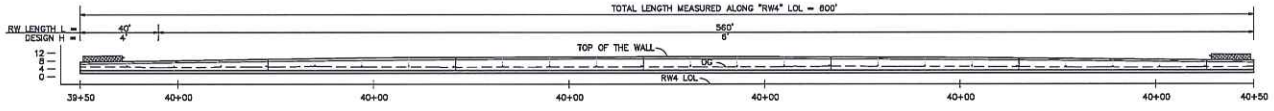
RETAINING WALL 1



RETAINING WALL 2



RETAINING WALL 3



RETAINING WALL 4

RETAINING WALLS PROFILE
SCALE : 1" = 20'

Subsurface Soil Model Profile

Prepared By: HP Date: 6/9/2011
Checked By: CH Date: 6/10/2011

Boring No. :	R-11-013		CPT-11-011	R-11-012		CPT-11-001
	Undrained Shear Strength (psf) Based on					
Depth (feet)	Blow Counts/ft	*Pocket Penetrometer data	CPT-11-011 Data	Blow Counts/ft	*Pocket Penetrometer data	CPT-11-001 Data
3	NT	NT	4,000	15	3,500	3,600
5	13	3,000	4,000	24	NT	2,100
10	NT	NT	7,000	9	2,500	4,900
15	37	3,500	3,200	10	3,500	1,700
20	8	3,500	2,800	NT	3,500	2,400
25	13	NT	5,000	30	NT	
30	16	3,500		9	NT	
35				15	4,000	2,200
40				12	NT	2,600
45			1,700	23	4,000	2,600
50			4,500	23	4,000	3,100
55						4,400
60						
65						4,300
70						8,000
75						4,500
80						4,800
85						5,000
90						2,360
95						N/T
100						N/T
110						N/T
120						

* Highest reading on pocket penetrometer for unconfined compression strength is

NT= Not Tested

= Sand

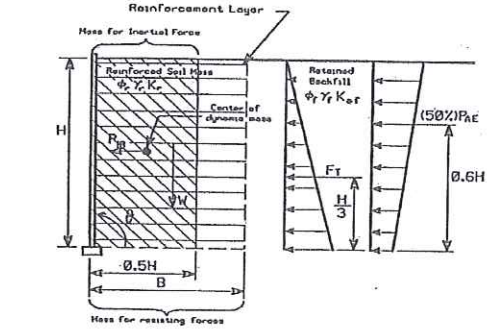
External Stability Check Calculations

MSE WALL EXTERNAL STABILITY CHECK CALCULATIONS

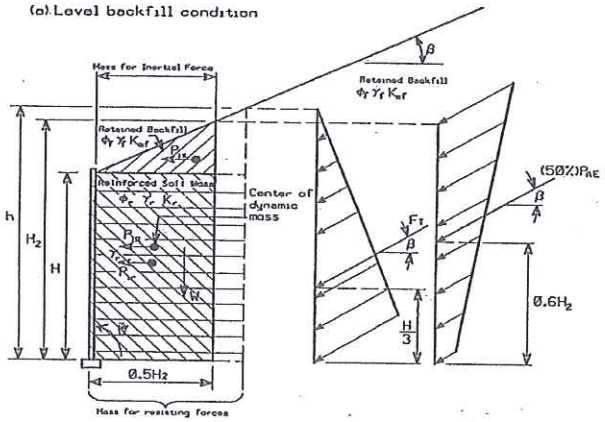
(EZ_MSEWall Version R1.0, 2007)

PROJECT INFORMATION	
Project Name	Crosstown Freeway, Retaining Wall 1, Stn. 877+00
Project No.	111904.G01
Location	Stockton, CA

INPUT PARAMETERS		
WALL DIMENSIONS		
Design Wall Height, H	16.00 ft	Maximum wall height
Wall Face Slope Angle, θ	90.00 degrees	Vertical wall =90 degrees
Wall Friction Angle, δ	0.00 degrees	Enter 0 for vertical walls
Backfill Slope Angle, β	0.00 degrees	
Wall Embedment Depth, D	2.00 ft	Minimum is 2 feet or 0.1H whichever is greater
Reinforcement Length, L	11.20 ft	Minimum is 70% of H or 8, whichever is greater
LOADS		
Traffic Surcharge, q	240.00 psf	Equiv. to 2-ft of overburden soil pressure
Horizontal Seismic Coefficient, k_h	0.16	kh = 1/2 PGA for Mononobe-Okabe Pressure
Vertical Seismic Coefficient, k_v	0.10	
FOUNDATION SOIL		
Friction Angle, ϕ_b	30.00 degrees	For standard design use 30 deg. (BDS 5.9.2)
Cohesion, c_b	0.00 psf	
Total Unit Weight, γ_b	110.00 pcf	
REINFORCED SOIL MASS		
Friction Angle, ϕ_r	30.00 degrees	To be specified as a minimum
Cohesion, c_r	0.00 psf	Imported material
Total Unit Weight, γ_r	120.00 pcf	
RETAINED BACKFILL		
Friction Angle, ϕ_f	34.00 degrees	For standard design use 34 deg. (BDS 5.9.2)
Cohesion, c_f	0.00 psf	Imported structural backfill
Total Unit Weight, γ_f	120.00 pcf	



(a) Level backfill condition



(b) Sloping backfill condition

References:

1. **Federal Highway Administration (FHWA), "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design & Construction Guidelines," Publication No. FHWA-NHI-00-043, March 2001.**
2. **Caltrans Memo to Designers 5-8, "Mechanically Stabilized Embankment," January 1991.**
3. **Caltrans Bridge Design Aids, "Mechanically Stabilized Embankment," pp. 3-8.1 - 3-8.10, April 2002.**

MSE WALL EXTERNAL STABILITY CHECK CALCULATIONS

(EZ_MSEWall Version R1.0, 2007)

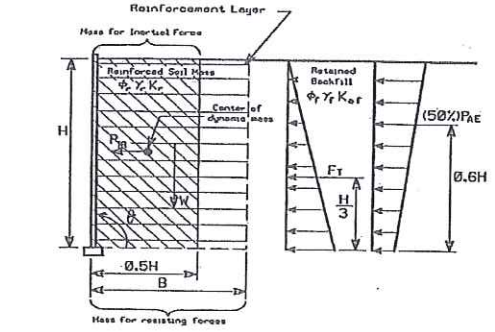
PROJECT INFORMATION		
Project Name	Crosstown Freeway, Retaining Wall 1, Stn. 877+00	
Project No.	111904.G01	
Wall Description	Stockton, CA	
INPUT PARAMETERS		
WALL DIMENSIONS:		
Design Height, H	16.00 ft	Maximum wall height
Wall Face Slope Angle, θ	90.00 degrees	Vertical wall =90 degrees
Wall Friction Angle, δ	0.00	Enter 0 for vertical walls
Backfill Slope Angle, β	0.00 degrees	
Wall Embedment Depth, D	2.00 ft	Minimum is 2 feet or 0.1H whichever is greater
Reinforcement Length, L	11.20 ft	Minimum is 70% of H or 8, whichever is greater
LOADS:		
Traffic Surcharge, q	240.00 psf	Equiv. to 2-ft of overburden soil pressure
Horizontal Seismic Coefficient, k_h	0.16	kh = 1/2 PGA for Mononobe-Okabe Pressure
Vertical Seismic Coefficient, k_v	0.10	
FOUNDATION SOIL:		
Friction Angle, ϕ_b	30.00 degrees	For standard design use 30 deg. (BDS 5.9.2)
Cohesion, c_b	0.00 psf	
Total Unit Weight, γ_b	110.00 pcf	
REINFORCED SOIL MASS:		
Friction Angle, ϕ_r	30.00 degrees	To be specified as a minimum
Cohesion, c_r	0.00 psf	Imported material
Total Unit Weight, γ_r	120.00 pcf	
RETAINED BACKFILL:		
Friction Angle, ϕ_f	34.00 degrees	For standard design use 34 deg. (BDS 5.9.2)
Cohesion, c_f	0.00 psf	Imported structural backfill
Total Unit Weight, γ_f	120.00 pcf	
CALCULATED PARAMETERS		
Static (Active) Earth Pressure Coefficient, K_a	0.283	Based on Rankine Theory (BDS C5.5.5.3)
Dynamic (Active) Earth Pressure Coefficient, K_{ae}	0.394	From Mononobe-Okabe Equation (Kramer p478-479)
Effective Wall Height for Static Active Thrust Force, H_1	16.00 ft	$H_1 = H + L \tan\beta$
Effective Wall Height for Dynamic Active Thrust Force, H_2	16.00 ft	$H_2 = H + (0.5\ H \tan\beta)/(1 - 0.5 \tan\beta)$ (FHWA-NHI-00-043 p.100)
Static Active Thrust Force From Retained Fill, F_T	4.34 kips/ft of wall	$F_T = 1/2\ \gamma_f K_a\ (H_1)^2$
Dynamic Active Thrust Force From Retained Fill, P_{ae}	1.70 kips/ft of wall	$P_{ae} = 1/2\ \gamma_f\ (K_{ae} - K_a)\ (H_2)^2$ (FHWA-NHI-00-043 p.100)
Horizontal Component of Static Active Thrust Force, F_1	4.34 kips/ft of wall	$F_1 = F_T \cos\beta$
Horizontal Force From Traffic Surcharge, F_2	1.09 kips/ft of wall	$F_2 = q\ H\ K_a$
Horizontal Inertial Force From Reinforced Soil Mass, P_{ir}	2.46 kips/ft of wall	$P_{ir} = 0.5\ k_h\ \gamma_f\ H_2\ H$ (FHWA-NHI-00-043 p.100)
Horizontal Inertial Force From Sloping Soil Surcharge, P_{is}	0.00 kips/ft of wall	$P_{is} = 0.125\ k_h\ \gamma_f\ (H_2)^2 \tan\beta$
Weight of Reinforced Soil Mass, V_1	21.50 kips/ft of wall	$V_1 = \gamma_r\ H\ L$
Weight of Sloping Backfill, V_2	0.00 kips/ft of wall	$V_2 = 1/2\ \gamma_f\ L\ (L \tan\beta)$
Sum of Resisting Moments, M_R	120.42 kip-ft/ft of wall	$M_R = V_1\ (L/2) + V_2\ (1/2\ L+1/6\ L)$
Sum of Overturning Moments (Static), M_{OS}	31.85 kip-ft/ft of wall	$M_{OS} = F_1\ (H1/3) + F_2\ (H1/2)$
Sum of Overturning Moments (Static + Dynamic), M_{OD}	67.86 kip-ft/ft of wall	$M_{OD} = Mos + P_{ir}\ (H/2) + P_{is}\ [H+ 1/3\ (H_2-H)] + P_{ae} \cos\beta\ (0.6\ H_2)$
Sum of Vertical Forces, V_T	21.50 kips/ft of wall	$V_T = V_1 + V_2 + F_T \sin\beta$
Sum of Horizontal Resisting Forces, P_R	12.42 kips/ft of wall	$P_R = V_T\ \tan[\min(\phi_r, \phi_b)]$
Sum of Horizontal Driving Forces (Static), P_{DS}	5.43 kips/ft of wall	$P_{DS} = F_1 + F_2$
Sum of Horizontal Driving Forces (Static + Dynamic), P_{DD}	9.59 kips/ft of wall	$P_{DD} = P_{DS} + P_{ir} + P_{is} + P_{ae}$
CHECK SLIDING FAILURE:		
Static Sliding Factor of Safety, $FS_{(static\ sliding)} = P_R/P_{DS}$	2.29 ≥ 1.5	**OK**
Dynamic Sliding Factor of Safety, $FS_{(dynamic\ sliding)} = P_R/P_{DD}$	1.29 ≥ 1.125	**OK**
CHECK OVERTURNING FAILURE:		
Static Overturning Factor of Safety, $FS_{(static\ overturning)} = M_R/M_{OS}$	3.78 ≥ 2.0	**OK**
Dynamic Overturning Factor of Safety, $FS_{(dynamic\ overturning)} = M_R/M_{OD}$	1.77 ≥ 1.5	**OK**
Static Eccentricity, $e_s = [L/2 - (M_R-M_{OS})/V_T]$	1.48 ft < L/6	**OK**
Dynamic Eccentricity, $e_d = [L/2 - (M_R-M_{OD})/V_T]$	3.16 ft < L/3	**OK**
CHECK BEARING FAILURE:		
Bearing Capacity Factor, Nc	30.14	Based on Caltrans DBS Table 4.4.7.1A (2003)
Bearing Capacity Factor, Nγ	22.40	Based on Caltrans DBS Table 4.4.7.1A (2003)
Max. Applied Bearing Pressure at the Base of the Wall, σ_v	2.61 ksf	$\sigma_v = V_T / (L - 2\ e_s)$
Ultimate Bearing Capacity, q_{ult}	10.15 ksf	$q_{ult} = c_b\ N_c + 0.5\ \gamma_b\ N_\gamma\ (L - 2\ e_s)$ (Embedment is neglected)
Bearing Capacity Factor of Safety, $FS_{(bearing\ capacity)}$	3.89 ≥ 2.0	**OK**

MSE WALL EXTERNAL STABILITY CHECK CALCULATIONS

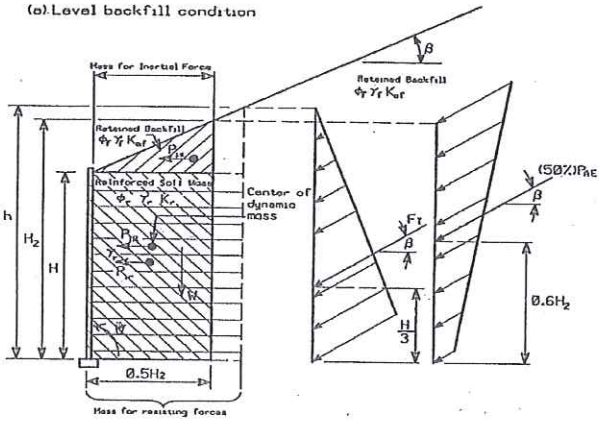
(EZ_MSEWall Version R1.0, 2007)

PROJECT INFORMATION	
Project Name	Crosstown Freeway, Retaining Wall 1, Stn. 878+50
Project No.	111904.G01
Location	Stockton, CA

INPUT PARAMETERS		
WALL DIMENSIONS		
Design Wall Height, H	28.00 ft	Maximum wall height
Wall Face Slope Angle, θ	90.00 degrees	Vertical wall =90 degrees
Wall Friction Angle, δ	0.00 degrees	Enter 0 for vertical walls
Backfill Slope Angle, β	0.00 degrees	
Wall Embedment Depth, D	2.80 ft	Minimum is 2 feet or 0.1H whichever is greater
Reinforcement Length, L	19.60 ft	Minimum is 70% of H or 8, whichever is greater
LOADS		
Traffic Surcharge, q	240.00 psf	Equiv. to 2-ft of overburden soil pressure
Horizontal Seismic Coefficient, k_h	0.16	kh = 1/2 PGA for Mononobe-Okabe Pressure
Vertical Seismic Coefficient, k_v	0.10	
FOUNDATION SOIL		
Friction Angle, ϕ_b	30.00 degrees	For standard design use 30 deg. (BDS 5.9.2)
Cohesion, c_b	0.00 psf	
Total Unit Weight, γ_b	110.00 pcf	
REINFORCED SOIL MASS		
Friction Angle, ϕ_r	30.00 degrees	To be specified as a minimum
Cohesion, c_r	0.00 psf	Imported material
Total Unit Weight, γ_r	120.00 pcf	
RETAINED BACKFILL		
Friction Angle, ϕ_f	34.00 degrees	For standard design use 34 deg. (BDS 5.9.2)
Cohesion, c_f	0.00 psf	Imported structural backfill
Total Unit Weight, γ_f	120.00 pcf	



(a) Level backfill condition



(b) Sloping backfill condition

References:

1. **Federal Highway Administration (FHWA), "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design & Construction Guidelines," Publication No. FHWA-NHI-00-043, March 2001.**
2. **Caltrans Memo to Designers 5-8, "Mechanically Stabilized Embankment," January 1991.**
3. **Caltrans Bridge Design Aids, "Mechanically Stabilized Embankment," pp. 3-8.1 - 3-8.10, April 2002.**

MSE WALL EXTERNAL STABILITY CHECK CALCULATIONS

(EZ_MSEWall Version R1.0, 2007)

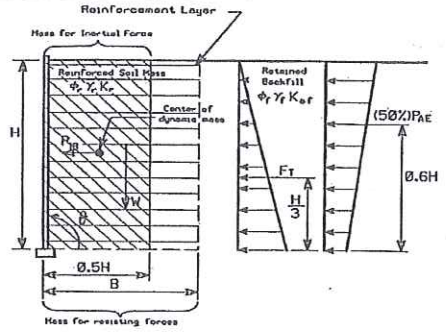
PROJECT INFORMATION		
Project Name	Crosstown Freeway, Retaining Wall 1, Stn. 878+50	
Project No.	111904.G01	
Wall Description	Stockton, CA	
INPUT PARAMETERS		
WALL DIMENSIONS:		
Design Height, H	28.00 ft	Maximum wall height
Wall Face Slope Angle, θ	90.00 degrees	Vertical wall =90 degrees
Wall Friction Angle, δ	0.00	Enter 0 for vertical walls
Backfill Slope Angle, β	0.00 degrees	
Wall Embedment Depth, D	2.80 ft	Minimum is 2 feet or 0.1H whichever is greater
Reinforcement Length, L	19.60 ft	Minimum is 70% of H or 8, whichever is greater
LOADS:		
Traffic Surcharge, q	240.00 psf	Equiv. to 2-ft of overburden soil pressure
Horizontal Seismic Coefficient, k_h	0.16	kh = 1/2 PGA for Mononobe-Okabe Pressure
Vertical Seismic Coefficient, k_v	0.10	
FOUNDATION SOIL:		
Friction Angle, ϕ_b	30.00 degrees	For standard design use 30 deg. (BDS 5.9.2)
Cohesion, c_b	0.00 psf	
Total Unit Weight, γ_b	110.00 pcf	
REINFORCED SOIL MASS:		
Friction Angle, ϕ_r	30.00 degrees	To be specified as a minimum
Cohesion, c_r	0.00 psf	Imported material
Total Unit Weight, γ_r	120.00 pcf	
RETAINED BACKFILL:		
Friction Angle, ϕ_f	34.00 degrees	For standard design use 34 deg. (BDS 5.9.2)
Cohesion, c_f	0.00 psf	Imported structural backfill
Total Unit Weight, γ_f	120.00 pcf	
CALCULATED PARAMETERS		
Static (Active) Earth Pressure Coefficient, K_a	0.283	Based on Rankine Theory (BDS C5.5.5.3)
Dynamic (Active) Earth Pressure Coefficient, K_{ae}	0.394	From Mononobe-Okabe Equation (Kramer p478-479)
Effective Wall Height for Static Active Thrust Force, H_1	28.00 ft	$H_1 = H + L \tan \beta$
Effective Wall Height for Dynamic Active Thrust Force, H_2	28.00 ft	$H_2 = H + (0.5\ H \tan \beta)/(1 - 0.5 \tan \beta)$ (FHWA-NHI-00-043 p.100)
Static Active Thrust Force From Retained Fill, F_T	13.30 kips/ft of wall	$F_T = 1/2\ \gamma_f K_a\ (H_1)^2$
Dynamic Active Thrust Force From Retained Fill, P_{ae}	5.22 kips/ft of wall	$P_{ae} = 1/2\ \gamma_f\ (K_{ae} - K_a)\ (H_2)^2$ (FHWA-NHI-00-043 p.100)
Horizontal Component of Static Active Thrust Force, F_1	13.30 kips/ft of wall	$F_1 = F_T \cos \beta$
Horizontal Force From Traffic Surcharge, F_2	1.90 kips/ft of wall	$F_2 = q\ H\ K_a$
Horizontal Inertial Force From Reinforced Soil Mass, P_{ir}	7.53 kips/ft of wall	$P_{ir} = 0.5\ k_h\ \gamma_f\ H_2\ H$ (FHWA-NHI-00-043 p.100)
Horizontal Inertial Force From Sloping Soil Surcharge, P_{is}	0.00 kips/ft of wall	$P_{is} = 0.125\ k_h\ \gamma_f\ (H_2)^2 \tan \beta$
Weight of Reinforced Soil Mass, V_1	65.86 kips/ft of wall	$V_1 = \gamma_r\ H\ L$
Weight of Sloping Backfill, V_2	0.00 kips/ft of wall	$V_2 = 1/2\ \gamma_f\ L\ (L \tan \beta)$
Sum of Resisting Moments, M_R	645.39 kip-ft/ft of wall	$M_R = V_1\ (L/2) + V_2\ (1/2\ L+1/6\ L)$
Sum of Overturning Moments (Static), M_{OS}	150.72 kip-ft/ft of wall	$M_{OS} = F_1\ (H1/3) + F_2\ (H1/2)$
Sum of Overturning Moments (Static + Dynamic), M_{OD}	343.73 kip-ft/ft of wall	$M_{OD} = Mos + P_{ir}\ (H/2) + P_{is}\ [H+ 1/3\ (H_2-H)] + P_{ae} \cos \beta\ (0.6\ H_2)$
Sum of Vertical Forces, V_T	65.86 kips/ft of wall	$V_T = V_1 + V_2 + F_T \sin \beta$
Sum of Horizontal Resisting Forces, P_R	38.02 kips/ft of wall	$P_R = V_T\ \tan [\min\ (\phi_r, \phi_b)]$
Sum of Horizontal Driving Forces (Static), P_{DS}	15.20 kips/ft of wall	$P_{DS} = F_1 + F_2$
Sum of Horizontal Driving Forces (Static + Dynamic), P_{DD}	27.94 kips/ft of wall	$P_{DD} = P_{DS} + P_{ir} + P_{is} + P_{ae}$
CHECK SLIDING FAILURE:		
Static Sliding Factor of Safety, $FS_{(static\ sliding)} = P_R/P_{DS}$	2.50 ≥ 1.5	**OK**
Dynamic Sliding Factor of Safety, $FS_{(dynamic\ sliding)} = P_R/P_{DD}$	1.36 ≥ 1.125	**OK**
CHECK OVERTURNING FAILURE:		
Static Overturning Factor of Safety, $FS_{(static\ overturning)} = M_R/M_{OS}$	4.28 ≥ 2.0	**OK**
Dynamic Overturning Factor of Safety, $FS_{(dynamic\ overturning)} = M_R/M_{OD}$	1.88 ≥ 1.5	**OK**
Static Eccentricity, $e_s = [L/2 - (M_R-M_{OS})/V_T]$	2.29 ft < L/6	**OK**
Dynamic Eccentricity, $e_d = [L/2 - (M_R-M_{OD})/V_T]$	5.22 ft < L/3	**OK**
CHECK BEARING FAILURE:		
Bearing Capacity Factor, Nc	30.14	Based on Caltrans DBS Table 4.4.7.1A (2003)
Bearing Capacity Factor, Nγ	22.40	Based on Caltrans DBS Table 4.4.7.1A (2003)
Max. Applied Bearing Pressure at the Base of the Wall, σ_v	4.38 ksf	$\sigma_v = V_T / (L - 2\ e_s)$
Ultimate Bearing Capacity, q_{ult}	18.51 ksf	$q_{ult} = c_b\ N_c + 0.5\ \gamma_b\ N_\gamma\ (L - 2\ e_s)$ (Embedment is neglected)
Bearing Capacity Factor of Safety, $FS_{(bearing\ capacity)}$	4.22 ≥ 2.0	**OK**

MSE WALL EXTERNAL STABILITY CHECK CALCULATIONS

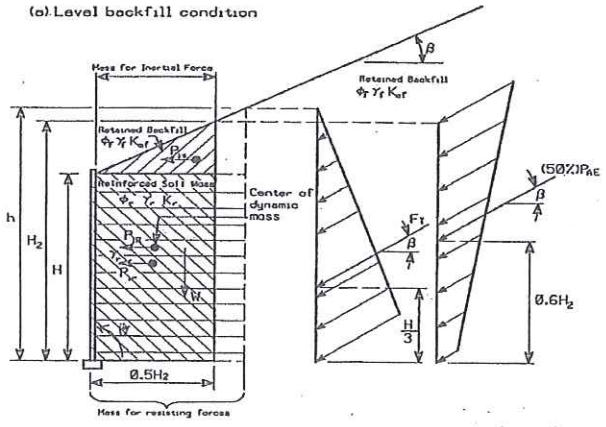
(EZ_MSEWall Version R1.0, 2007)

PROJECT INFORMATION	
Project Name	Crosstown Freeway, Retaining wall 1, Stn. 880+50
Project No.	111904.G01
Location	Stockton, CA

INPUT PARAMETERS		
WALL DIMENSIONS		
Design Wall Height, H	36.00 ft	Maximum wall height
Wall Face Slope Angle, θ	90.00 degrees	Vertical wall =90 degrees
Wall Friction Angle, δ	0.00 degrees	Enter 0 for vertical walls
Backfill Slope Angle, β	0.00 degrees	
Wall Embedment Depth, D	3.60 ft	Minimum is 2 feet or 0.1H whichever is greater
Reinforcement Length, L	25.20 ft	Minimum is 70% of H or 8, whichever is greater
LOADS		
Traffic Surcharge, q	240.00 psf	Equiv. to 2-ft of overburden soil pressure
Horizontal Seismic Coefficient, k_h	0.16	kh = 1/2 PGA for Mononobe-Okabe Pressure
Vertical Seismic Coefficient, k_v	0.10	
FOUNDATION SOIL		
Friction Angle, ϕ_b	30.00 degrees	For standard design use 30 deg. (BDS 5.9.2)
Cohesion, c_b	0.00 psf	
Total Unit Weight, γ_b	110.00 pcf	
REINFORCED SOIL MASS		
Friction Angle, ϕ_r	30.00 degrees	To be specified as a minimum
Cohesion, c_r	0.00 psf	Imported material
Total Unit Weight, γ_r	120.00 pcf	
RETAINED BACKFILL		
Friction Angle, ϕ_f	34.00 degrees	For standard design use 34 deg. (BDS 5.9.2)
Cohesion, c_f	0.00 psf	Imported structural backfill
Total Unit Weight, γ_f	120.00 pcf	



(a) Level backfill condition



(b) Sloping backfill condition

References:

1. **Federal Highway Administration (FHWA), "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design & Construction Guidelines," Publication No. FHWA-NHI-00-043, March 2001.**
2. **Caltrans Memo to Designers 5-8, "Mechanically Stabilized Embankment," January 1991.**
3. **Caltrans Bridge Design Aids, "Mechanically Stabilized Embankment," pp. 3-8.1 - 3-8.10, April 2002.**

MSE WALL EXTERNAL STABILITY CHECK CALCULATIONS

(EZ_MSEWall Version R1.0, 2007)

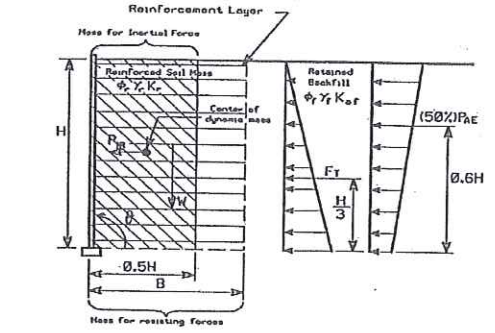
PROJECT INFORMATION		
Project Name	Crosstown Freeway, Retaining wall 1, Stn. 880+50	
Project No.	111904.G01	
Wall Description	Stockton, CA	
INPUT PARAMETERS		
WALL DIMENSIONS:		
Design Height, H	36.00 ft	Maximum wall height
Wall Face Slope Angle, θ	90.00 degrees	Vertical wall =90 degrees
Wall Friction Angle, δ	0.00	Enter 0 for vertical walls
Backfill Slope Angle, β	0.00 degrees	
Wall Embedment Depth, D	3.60 ft	Minimum is 2 feet or 0.1H whichever is greater
Reinforcement Length, L	25.20 ft	Minimum is 70% of H or 8, whichever is greater
LOADS:		
Traffic Surcharge, q	240.00 psf	Equiv. to 2-ft of overburden soil pressure
Horizontal Seismic Coefficient, k_h	0.16	kh = 1/2 PGA for Mononobe-Okabe Pressure
Vertical Seismic Coefficient, k_v	0.10	
FOUNDATION SOIL:		
Friction Angle, ϕ_b	30.00 degrees	For standard design use 30 deg. (BDS 5.9.2)
Cohesion, c_b	0.00 psf	
Total Unit Weight, γ_b	110.00 pcf	
REINFORCED SOIL MASS:		
Friction Angle, ϕ_r	30.00 degrees	To be specified as a minimum
Cohesion, c_r	0.00 psf	Imported material
Total Unit Weight, γ_r	120.00 pcf	
RETAINED BACKFILL:		
Friction Angle, ϕ_f	34.00 degrees	For standard design use 34 deg. (BDS 5.9.2)
Cohesion, c_f	0.00 psf	Imported structural backfill
Total Unit Weight, γ_f	120.00 pcf	
CALCULATED PARAMETERS		
Static (Active) Earth Pressure Coefficient, K_a	0.283	Based on Rankine Theory (BDS C5.5.5.3)
Dynamic (Active) Earth Pressure Coefficient, K_{ae}	0.394	From Mononobe-Okabe Equation (Kramer p478-479)
Effective Wall Height for Static Active Thrust Force, H_1	36.00 ft	$H_1 = H + L \tan\beta$
Effective Wall Height for Dynamic Active Thrust Force, H_2	36.00 ft	$H_2 = H + (0.5\ H \tan\beta)/(1 - 0.5 \tan\beta)$ (FHWA-NHI-00-043 p.100)
Static Active Thrust Force From Retained Fill, F_T	21.98 kips/ft of wall	$F_T = 1/2\ \gamma_f K_a\ (H_1)^2$
Dynamic Active Thrust Force From Retained Fill, P_{ae}	8.62 kips/ft of wall	$P_{ae} = 1/2\ \gamma_f\ (K_{ae} - K_a)\ (H_2)^2$ (FHWA-NHI-00-043 p.100)
Horizontal Component of Static Active Thrust Force, F_1	21.98 kips/ft of wall	$F_1 = F_T \cos\beta$
Horizontal Force From Traffic Surcharge, F_2	2.44 kips/ft of wall	$F_2 = q\ H\ K_a$
Horizontal Inertial Force From Reinforced Soil Mass, P_{ir}	12.44 kips/ft of wall	$P_{ir} = 0.5\ k_h\ \gamma_f\ H_2\ H$ (FHWA-NHI-00-043 p.100)
Horizontal Inertial Force From Sloping Soil Surcharge, P_{is}	0.00 kips/ft of wall	$P_{is} = 0.125\ k_h\ \gamma_f\ (H_2)^2 \tan\beta$
Weight of Reinforced Soil Mass, V_1	108.86 kips/ft of wall	$V_1 = \gamma_r\ H\ L$
Weight of Sloping Backfill, V_2	0.00 kips/ft of wall	$V_2 = 1/2\ \gamma_f\ L\ (L \tan\beta)$
Sum of Resisting Moments, M_R	1,371.69 kip-ft/ft of wall	$M_R = V_1\ (L/2) + V_2\ (1/2\ L+1/6\ L)$
Sum of Overturning Moments (Static), M_{OS}	307.77 kip-ft/ft of wall	$M_{OS} = F_1\ (H1/3) + F_2\ (H1/2)$
Sum of Overturning Moments (Static + Dynamic), M_{OD}	717.99 kip-ft/ft of wall	$M_{OD} = Mos + P_{ir}\ (H/2) + P_{is}\ [H+ 1/3\ (H_2-H)] + P_{ae} \cos\beta\ (0.6\ H_2)$
Sum of Vertical Forces, V_T	108.86 kips/ft of wall	$V_T = V_1 + V_2 + F_T \sin\beta$
Sum of Horizontal Resisting Forces, P_R	62.85 kips/ft of wall	$P_R = V_T\ \tan\ [\min\ (\phi_r, \phi_b)]$
Sum of Horizontal Driving Forces (Static), P_{DS}	24.43 kips/ft of wall	$P_{DS} = F_1 + F_2$
Sum of Horizontal Driving Forces (Static + Dynamic), P_{DD}	45.49 kips/ft of wall	$P_{DD} = P_{DS} + P_{ir} + P_{is} + P_{ae}$
CHECK SLIDING FAILURE:		
Static Sliding Factor of Safety, $FS_{(static\ sliding)} = P_R/P_{DS}$	2.57 ≥ 1.5	**OK**
Dynamic Sliding Factor of Safety, $FS_{(dynamic\ sliding)} = P_R/P_{DD}$	1.38 ≥ 1.125	**OK**
CHECK OVERTURNING FAILURE:		
Static Overturning Factor of Safety, $FS_{(static\ overturning)} = M_R/M_{OS}$	4.46 ≥ 2.0	**OK**
Dynamic Overturning Factor of Safety, $FS_{(dynamic\ overturning)} = M_R/M_{OD}$	1.91 ≥ 1.5	**OK**
Static Eccentricity, $e_s = [L/2 - (M_R-M_{OS})/V_T]$	2.83 ft < L/6	**OK**
Dynamic Eccentricity, $e_d = [L/2 - (M_R-M_{OD})/V_T]$	6.60 ft < L/3	**OK**
CHECK BEARING FAILURE:		
Bearing Capacity Factor, Nc	30.14	Based on Caltrans DBS Table 4.4.7.1A (2003)
Bearing Capacity Factor, Nγ	22.40	Based on Caltrans DBS Table 4.4.7.1A (2003)
Max. Applied Bearing Pressure at the Base of the Wall, σ_v	5.57 ksf	$\sigma_v = V_T / (L - 2\ e_s)$
Ultimate Bearing Capacity, q_{ult}	24.08 ksf	$q_{ult} = c_b\ N_c + 0.5\ \gamma_b\ N_\gamma\ (L - 2\ e_s)$ (Embedment is neglected)
Bearing Capacity Factor of Safety, $FS_{(bearing\ capacity)}$	4.32 ≥ 2.0	**OK**

MSE WALL EXTERNAL STABILITY CHECK CALCULATIONS

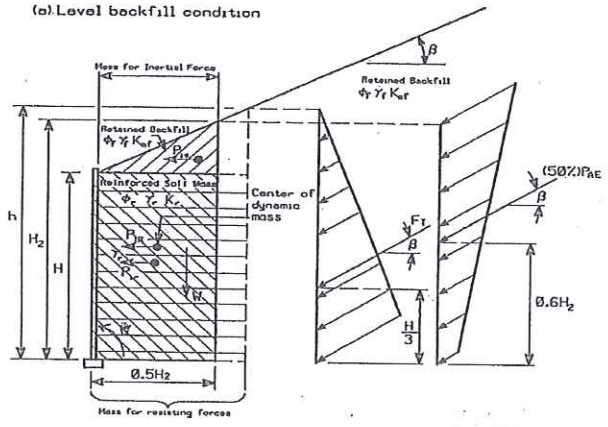
(EZ_MSEWall Version R1.0, 2007)

PROJECT INFORMATION	
Project Name	Crosstown Freeway-Retaining Wall 1, Stn. 882+00
Project No.	111904.G01
Location	Stockton, CA

INPUT PARAMETERS		
WALL DIMENSIONS		
Design Wall Height, H	42.00 ft	Maximum wall height
Wall Face Slope Angle, θ	90.00 degrees	Vertical wall =90 degrees
Wall Friction Angle, δ	0.00 degrees	Enter 0 for vertical walls
Backfill Slope Angle, β	0.00 degrees	
Wall Embedment Depth, D	4.20 ft	Minimum is 2 feet or 0.1H whichever is greater
Reinforcement Length, L	29.40 ft	Minimum is 70% of H or 8, whichever is greater
LOADS		
Traffic Surcharge, q	240.00 psf	Equiv. to 2-ft of overburden soil pressure
Horizontal Seismic Coefficient, k_h	0.16	kh = 1/2 PGA for Mononobe-Okabe Pressure
Vertical Seismic Coefficient, k_v	0.10	
FOUNDATION SOIL		
Friction Angle, ϕ_b	30.00 degrees	For standard design use 30 deg. (BDS 5.9.2)
Cohesion, c_b	0.00 psf	
Total Unit Weight, γ_b	110.00 pcf	
REINFORCED SOIL MASS		
Friction Angle, ϕ_r	30.00 degrees	To be specified as a minimum
Cohesion, c_r	0.00 psf	Imported material
Total Unit Weight, γ_r	120.00 pcf	
RETAINED BACKFILL		
Friction Angle, ϕ_f	34.00 degrees	For standard design use 34 deg. (BDS 5.9.2)
Cohesion, c_f	0.00 psf	Imported structural backfill
Total Unit Weight, γ_f	120.00 pcf	



(a) Level backfill condition



(b) Sloping backfill condition

References:

1. **Federal Highway Administration (FHWA), "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design & Construction Guidelines," Publication No. FHWA-NHI-00-043, March 2001.**
2. **Caltrans Memo to Designers 5-8, "Mechanically Stabilized Embankment," January 1991.**
3. **Caltrans Bridge Design Aids, "Mechanically Stabilized Embankment," pp. 3-8.1 - 3-8.10, April 2002.**

MSE WALL EXTERNAL STABILITY CHECK CALCULATIONS

(EZ_MSEWall Version R1.0, 2007)

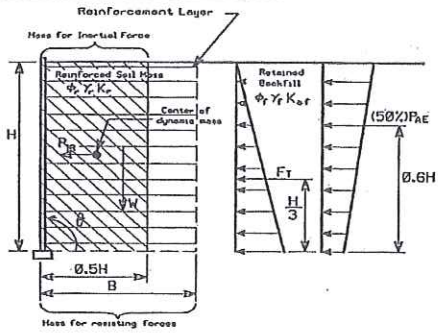
PROJECT INFORMATION		
Project Name	Crosstown Freeway-Retaining Wall 1, Stn. 882+00	
Project No.	111904.G01	
Wall Description	Stockton, CA	
INPUT PARAMETERS		
WALL DIMENSIONS:		
Design Height, H	42.00 ft	Maximum wall height
Wall Face Slope Angle, θ	90.00 degrees	Vertical wall =90 degrees
Wall Friction Angle, δ	0.00	Enter 0 for vertical walls
Backfill Slope Angle, β	0.00 degrees	
Wall Embedment Depth, D	4.20 ft	Minimum is 2 feet or 0.1H whichever is greater
Reinforcement Length, L	29.40 ft	Minimum is 70% of H or 8, whichever is greater
LOADS:		
Traffic Surcharge, q	240.00 psf	Equiv. to 2-ft of overburden soil pressure
Horizontal Seismic Coefficient, k_h	0.16	kh = 1/2 PGA for Mononobe-Okabe Pressure
Vertical Seismic Coefficient, k_v	0.10	
FOUNDATION SOIL:		
Friction Angle, ϕ_b	30.00 degrees	For standard design use 30 deg. (BDS 5.9.2)
Cohesion, c_b	0.00 psf	
Total Unit Weight, γ_b	110.00 pcf	
REINFORCED SOIL MASS:		
Friction Angle, ϕ_r	30.00 degrees	To be specified as a minimum
Cohesion, c_r	0.00 psf	Imported material
Total Unit Weight, γ_r	120.00 pcf	
RETAINED BACKFILL:		
Friction Angle, ϕ_f	34.00 degrees	For standard design use 34 deg. (BDS 5.9.2)
Cohesion, c_f	0.00 psf	Imported structural backfill
Total Unit Weight, γ_f	120.00 pcf	
CALCULATED PARAMETERS		
Static (Active) Earth Pressure Coefficient, K_a	0.283	Based on Rankine Theory (BDS C5.5.5.3)
Dynamic (Active) Earth Pressure Coefficient, K_{ae}	0.394	From Mononobe-Okabe Equation (Kramer p478-479)
Effective Wall Height for Static Active Thrust Force, H_1	42.00 ft	$H_1 = H + L \tan\beta$
Effective Wall Height for Dynamic Active Thrust Force, H_2	42.00 ft	$H_2 = H + (0.5\ H \tan\beta)/(1 - 0.5 \tan\beta)$ (FHWA-NHI-00-043 p.100)
Static Active Thrust Force From Retained Fill, F_T	29.92 kips/ft of wall	$F_T = 1/2\ \gamma_f K_a\ (H_1)^2$
Dynamic Active Thrust Force From Retained Fill, P_{ae}	11.74 kips/ft of wall	$P_{ae} = 1/2\ \gamma_f\ (K_{ae} - K_a)\ (H_2)^2$ (FHWA-NHI-00-043 p.100)
Horizontal Component of Static Active Thrust Force, F_1	29.92 kips/ft of wall	$F_1 = F_T \cos\beta$
Horizontal Force From Traffic Surcharge, F_2	2.85 kips/ft of wall	$F_2 = q\ H\ K_a$
Horizontal Inertial Force From Reinforced Soil Mass, P_{ir}	16.93 kips/ft of wall	$P_{ir} = 0.5\ k_h\ \gamma_f\ H_2\ H$ (FHWA-NHI-00-043 p.100)
Horizontal Inertial Force From Sloping Soil Surcharge, P_{is}	0.00 kips/ft of wall	$P_{is} = 0.125\ k_h\ \gamma_f\ (H_2)^2 \tan\beta$
Weight of Reinforced Soil Mass, V_1	148.18 kips/ft of wall	$V_1 = \gamma_r\ H\ L$
Weight of Sloping Backfill, V_2	0.00 kips/ft of wall	$V_2 = 1/2\ \gamma_f\ L\ (L \tan\beta)$
Sum of Resisting Moments, M_R	2,178.19 kip-ft/ft of wall	$M_R = V_1\ (L/2) + V_2\ (1/2\ L+1/6\ L)$
Sum of Overturning Moments (Static), M_{OS}	478.76 kip-ft/ft of wall	$M_{OS} = F_1\ (H1/3) + F_2\ (H1/2)$
Sum of Overturning Moments (Static + Dynamic), M_{OD}	1,130.16 kip-ft/ft of wall	$M_{OD} = Mos + P_{ir}\ (H/2) + P_{is}\ [H+ 1/3\ (H_2-H)] + P_{ae} \cos\beta\ (0.6\ H_2)$
Sum of Vertical Forces, V_T	148.18 kips/ft of wall	$V_T = V_1 + V_2 + F_T \sin\beta$
Sum of Horizontal Resisting Forces, P_R	85.55 kips/ft of wall	$P_R = V_T\ \tan[\min(\phi_r, \phi_b)]$
Sum of Horizontal Driving Forces (Static), P_{DS}	32.77 kips/ft of wall	$P_{DS} = F_1 + F_2$
Sum of Horizontal Driving Forces (Static + Dynamic), P_{DD}	61.44 kips/ft of wall	$P_{DD} = P_{DS} + P_{ir} + P_{is} + P_{ae}$
CHECK SLIDING FAILURE:		
Static Sliding Factor of Safety, $FS_{(static\ sliding)} = P_R/P_{DS}$	2.61 ≥ 1.5	**OK**
Dynamic Sliding Factor of Safety, $FS_{(dynamic\ sliding)} = P_R/P_{DD}$	1.39 ≥ 1.125	**OK**
CHECK OVERTURNING FAILURE:		
Static Overturning Factor of Safety, $FS_{(static\ overturning)} = M_R/M_{OS}$	4.55 ≥ 2.0	**OK**
Dynamic Overturning Factor of Safety, $FS_{(dynamic\ overturning)} = M_R/M_{OD}$	1.93 ≥ 1.5	**OK**
Static Eccentricity, $e_s = [L/2 - (M_R-M_{OS})/V_T]$	3.23 ft < L/6	**OK**
Dynamic Eccentricity, $e_d = [L/2 - (M_R-M_{OD})/V_T]$	7.63 ft < L/3	**OK**
CHECK BEARING FAILURE:		
Bearing Capacity Factor, Nc	30.14	Based on Caltrans DBS Table 4.4.7.1A (2003)
Bearing Capacity Factor, Nγ	22.40	Based on Caltrans DBS Table 4.4.7.1A (2003)
Max. Applied Bearing Pressure at the Base of the Wall, σ_v	6.46 ksf	$\sigma_v = V_T / (L - 2\ e_s)$
Ultimate Bearing Capacity, q_{ult}	28.26 ksf	$q_{ult} = c_b\ N_c + 0.5\ \gamma_b\ N_\gamma\ (L - 2\ e_s)$ (Embedment is neglected)
Bearing Capacity Factor of Safety, $FS_{(bearing\ capacity)}$	4.37 ≥ 2.0	**OK**

MSE WALL EXTERNAL STABILITY CHECK CALCULATIONS

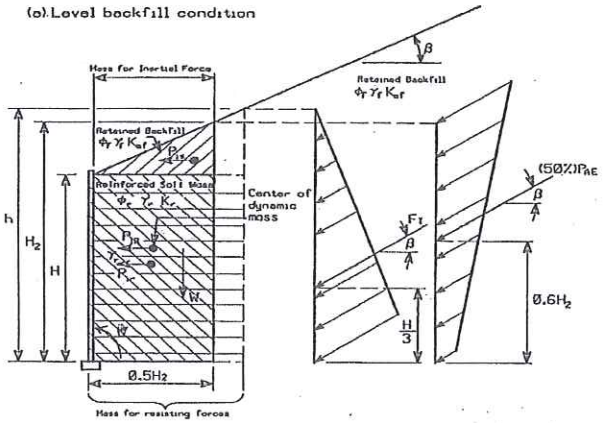
(EZ_MSEWall Version R1.0, 2007)

PROJECT INFORMATION	
Project Name	Crosstown Freeway, Retaining Wall 2, Stn. 876+50
Project No.	111904.G01
Location	Stockton, CA

INPUT PARAMETERS		
WALL DIMENSIONS		
Design Wall Height, H	22.00 ft	Maximum wall height
Wall Face Slope Angle, θ	90.00 degrees	Vertical wall =90 degrees
Wall Friction Angle, δ	0.00 degrees	Enter 0 for vertical walls
Backfill Slope Angle, β	0.00 degrees	
Wall Embedment Depth, D	2.20 ft	Minimum is 2 feet or 0.1H whichever is greater
Reinforcement Length, L	15.40 ft	Minimum is 70% of H or 8, whichever is greater
LOADS		
Traffic Surcharge, q	240.00 psf	Equiv. to 2-ft of overburden soil pressure
Horizontal Seismic Coefficient, k_h	0.16	kh = 1/2 PGA for Mononobe-Okabe Pressure
Vertical Seismic Coefficient, k_v	0.10	
FOUNDATION SOIL		
Friction Angle, ϕ_b	30.00 degrees	For standard design use 30 deg. (BDS 5.9.2)
Cohesion, c_b	0.00 psf	
Total Unit Weight, γ_b	110.00 pcf	
REINFORCED SOIL MASS		
Friction Angle, ϕ_r	30.00 degrees	To be specified as a minimum
Cohesion, c_r	0.00 psf	Imported material
Total Unit Weight, γ_r	120.00 pcf	
RETAINED BACKFILL		
Friction Angle, ϕ_f	34.00 degrees	For standard design use 34 deg. (BDS 5.9.2)
Cohesion, c_f	0.00 psf	Imported structural backfill
Total Unit Weight, γ_f	120.00 pcf	



(a) Level backfill condition



(b) Sloping backfill condition

References:

1. **Federal Highway Administration (FHWA), "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design & Construction Guidelines," Publication No. FHWA-NHI-00-043, March 2001.**
2. **Caltrans Memo to Designers 5-8, "Mechanically Stabilized Embankment," January 1991.**
3. **Caltrans Bridge Design Aids, "Mechanically Stabilized Embankment," pp. 3-8.1 - 3-8.10, April 2002.**

MSE WALL EXTERNAL STABILITY CHECK CALCULATIONS

(EZ_MSEWall Version R1.0, 2007)

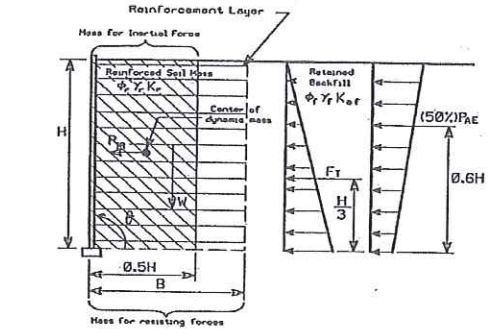
PROJECT INFORMATION		
Project Name	Crosstown Freeway, Retaining Wall 2, Stn. 876+50	
Project No.	111904.G01	
Wall Description	Stockton, CA	
INPUT PARAMETERS		
WALL DIMENSIONS:		
Design Height, H	22.00 ft	Maximum wall height
Wall Face Slope Angle, θ	90.00 degrees	Vertical wall =90 degrees
Wall Friction Angle, δ	0.00	Enter 0 for vertical walls
Backfill Slope Angle, β	0.00 degrees	
Wall Embedment Depth, D	2.20 ft	Minimum is 2 feet or 0.1H whichever is greater
Reinforcement Length, L	15.40 ft	Minimum is 70% of H or 8, whichever is greater
LOADS:		
Traffic Surcharge, q	240.00 psf	Equiv. to 2-ft of overburden soil pressure
Horizontal Seismic Coefficient, k_h	0.16	kh = 1/2 PGA for Mononobe-Okabe Pressure
Vertical Seismic Coefficient, k_v	0.10	
FOUNDATION SOIL:		
Friction Angle, ϕ_b	30.00 degrees	For standard design use 30 deg. (BDS 5.9.2)
Cohesion, c_b	0.00 psf	
Total Unit Weight, γ_b	110.00 pcf	
REINFORCED SOIL MASS:		
Friction Angle, ϕ_r	30.00 degrees	To be specified as a minimum
Cohesion, c_r	0.00 psf	Imported material
Total Unit Weight, γ_r	120.00 pcf	
RETAINED BACKFILL:		
Friction Angle, ϕ_f	34.00 degrees	For standard design use 34 deg. (BDS 5.9.2)
Cohesion, c_f	0.00 psf	Imported structural backfill
Total Unit Weight, γ_f	120.00 pcf	
CALCULATED PARAMETERS		
Static (Active) Earth Pressure Coefficient, K_a	0.283	Based on Rankine Theory (BDS C5.5.5.3)
Dynamic (Active) Earth Pressure Coefficient, K_{ae}	0.394	From Mononobe-Okabe Equation (Kramer p478-479)
Effective Wall Height for Static Active Thrust Force, H_1	22.00 ft	$H_1 = H + L \tan\beta$
Effective Wall Height for Dynamic Active Thrust Force, H_2	22.00 ft	$H_2 = H + (0.5\ H \tan\beta)/(1 - 0.5 \tan\beta)$ (FHWA-NHI-00-043 p.100)
Static Active Thrust Force From Retained Fill, F_T	8.21 kips/ft of wall	$F_T = 1/2\ \gamma_f K_a\ (H_1)^2$
Dynamic Active Thrust Force From Retained Fill, P_{ae}	3.22 kips/ft of wall	$P_{ae} = 1/2\ \gamma_f\ (K_{ae} - K_a)\ (H_2)^2$ (FHWA-NHI-00-043 p.100)
Horizontal Component of Static Active Thrust Force, F_1	8.21 kips/ft of wall	$F_1 = F_T \cos\beta$
Horizontal Force From Traffic Surcharge, F_2	1.49 kips/ft of wall	$F_2 = q\ H\ K_a$
Horizontal Inertial Force From Reinforced Soil Mass, P_{ir}	4.65 kips/ft of wall	$P_{ir} = 0.5\ k_h\ \gamma_f\ H_2\ H$ (FHWA-NHI-00-043 p.100)
Horizontal Inertial Force From Sloping Soil Surcharge, P_{is}	0.00 kips/ft of wall	$P_{is} = 0.125\ k_h\ \gamma_f\ (H_2)^2 \tan\beta$
Weight of Reinforced Soil Mass, V_1	40.66 kips/ft of wall	$V_1 = \gamma_r\ H\ L$
Weight of Sloping Backfill, V_2	0.00 kips/ft of wall	$V_2 = 1/2\ \gamma_f\ L\ (L \tan\beta)$
Sum of Resisting Moments, M_R	313.05 kip-ft/ft of wall	$M_R = V_1\ (L/2) + V_2\ (1/2\ L + 1/6\ L)$
Sum of Overturning Moments (Static), M_{OS}	76.63 kip-ft/ft of wall	$M_{OS} = F_1\ (H1/3) + F_2\ (H1/2)$
Sum of Overturning Moments (Static + Dynamic), M_{OD}	170.25 kip-ft/ft of wall	$M_{OD} = Mos + P_{ir}\ (H/2) + P_{is}\ [H + 1/3\ (H_2 - H)] + P_{ae} \cos\beta\ (0.6\ H_2)$
Sum of Vertical Forces, V_T	40.66 kips/ft of wall	$V_T = V_1 + V_2 + F_T \sin\beta$
Sum of Horizontal Resisting Forces, P_R	23.47 kips/ft of wall	$P_R = V_T \tan[\min(\phi_r, \phi_b)]$
Sum of Horizontal Driving Forces (Static), P_{DS}	9.70 kips/ft of wall	$P_{DS} = F_1 + F_2$
Sum of Horizontal Driving Forces (Static + Dynamic), P_{DD}	17.57 kips/ft of wall	$P_{DD} = P_{DS} + P_{ir} + P_{is} + P_{ae}$
CHECK SLIDING FAILURE:		
Static Sliding Factor of Safety, $FS_{(static\ sliding)} = P_R/P_{DS}$	2.42 ≥ 1.5	**OK**
Dynamic Sliding Factor of Safety, $FS_{(dynamic\ sliding)} = P_R/P_{DD}$	1.34 ≥ 1.125	**OK**
CHECK OVERTURNING FAILURE:		
Static Overturning Factor of Safety, $FS_{(static\ overturning)} = M_R/M_{OS}$	4.09 ≥ 2.0	**OK**
Dynamic Overturning Factor of Safety, $FS_{(dynamic\ overturning)} = M_R/M_{OD}$	1.84 ≥ 1.5	**OK**
Static Eccentricity, $e_s = [L/2 - (M_R - M_{OS})/V_T]$	1.88 ft < L/6	**OK**
Dynamic Eccentricity, $e_d = [L/2 - (M_R - M_{OD})/V_T]$	4.19 ft < L/3	**OK**
CHECK BEARING FAILURE:		
Bearing Capacity Factor, Nc	30.14	Based on Caltrans DBS Table 4.4.7.1A (2003)
Bearing Capacity Factor, Nγ	22.40	Based on Caltrans DBS Table 4.4.7.1A (2003)
Max. Applied Bearing Pressure at the Base of the Wall, σ_v	3.50 ksf	$\sigma_v = V_T / (L - 2\ e_s)$
Ultimate Bearing Capacity, q_{ult}	14.33 ksf	$q_{ult} = c_b\ N_c + 0.5\ \gamma_b\ N_\gamma\ (L - 2\ e_s)$ (Embedment is neglected)
Bearing Capacity Factor of Safety, $FS_{(bearing\ capacity)}$	4.10 ≥ 2.0	**OK**

MSE WALL EXTERNAL STABILITY CHECK CALCULATIONS

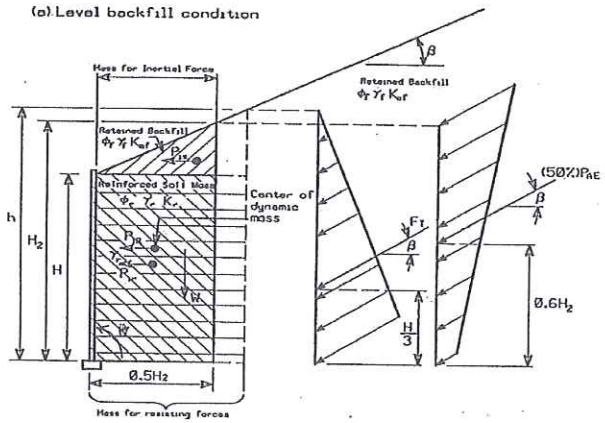
(EZ_MSEWall Version R1.0, 2007)

PROJECT INFORMATION	
Project Name	Crosstown Freeway, Retaining Wall 2, Stn. 878+50
Project No.	111904.G01
Location	Stockton, CA

INPUT PARAMETERS		
WALL DIMENSIONS		
Design Wall Height, H	30.00 ft	Maximum wall height
Wall Face Slope Angle, θ	90.00 degrees	Vertical wall =90 degrees
Wall Friction Angle, δ	0.00 degrees	Enter 0 for vertical walls
Backfill Slope Angle, β	0.00 degrees	
Wall Embedment Depth, D	3.00 ft	Minimum is 2 feet or 0.1H whichever is greater
Reinforcement Length, L	21.00 ft	Minimum is 70% of H or 8, whichever is greater
LOADS		
Traffic Surcharge, q	240.00 psf	Equiv. to 2-ft of overburden soil pressure
Horizontal Seismic Coefficient, k_h	0.16	kh = 1/2 PGA for Mononobe-Okabe Pressure
Vertical Seismic Coefficient, k_v	0.10	
FOUNDATION SOIL		
Friction Angle, ϕ_b	30.00 degrees	For standard design use 30 deg. (BDS 5.9.2)
Cohesion, c_b	0.00 psf	
Total Unit Weight, γ_b	110.00 pcf	
REINFORCED SOIL MASS		
Friction Angle, ϕ_r	30.00 degrees	To be specified as a minimum
Cohesion, c_r	0.00 psf	Imported material
Total Unit Weight, γ_r	120.00 pcf	
RETAINED BACKFILL		
Friction Angle, ϕ_f	34.00 degrees	For standard design use 34 deg. (BDS 5.9.2)
Cohesion, c_f	0.00 psf	Imported structural backfill
Total Unit Weight, γ_f	120.00 pcf	



(a) Level backfill condition



(b) Sloping backfill condition

References:

1. **Federal Highway Administration (FHWA), "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design & Construction Guidelines," Publication No. FHWA-NHI-00-043, March 2001.**
2. **Caltrans Memo to Designers 5-8, "Mechanically Stabilized Embankment," January 1991.**
3. **Caltrans Bridge Design Aids, "Mechanically Stabilized Embankment," pp. 3-8.1 - 3-8.10, April 2002.**

MSE WALL EXTERNAL STABILITY CHECK CALCULATIONS

(EZ_MSEWall Version R1.0, 2007)

PROJECT INFORMATION		
Project Name	Crosstown Freeway, Retaining Wall 2, Stn. 878+50	
Project No.	111904.G01	
Wall Description	Stockton, CA	

INPUT PARAMETERS		
WALL DIMENSIONS:		
Design Height, H	30.00 ft	Maximum wall height
Wall Face Slope Angle, θ	90.00 degrees	Vertical wall =90 degrees
Wall Friction Angle, δ	0.00	Enter 0 for vertical walls
Backfill Slope Angle, β	0.00 degrees	
Wall Embedment Depth, D	3.00 ft	Minimum is 2 feet or 0.1H whichever is greater
Reinforcement Length, L	21.00 ft	Minimum is 70% of H or 8, whichever is greater
LOADS:		
Traffic Surcharge, q	240.00 psf	Equiv. to 2-ft of overburden soil pressure
Horizontal Seismic Coefficient, k_h	0.16	kh = 1/2 PGA for Mononobe-Okabe Pressure
Vertical Seismic Coefficient, k_v	0.10	
FOUNDATION SOIL:		
Friction Angle, ϕ_b	30.00 degrees	For standard design use 30 deg. (BDS 5.9.2)
Cohesion, c_b	0.00 psf	
Total Unit Weight, γ_b	110.00 pcf	
REINFORCED SOIL MASS:		
Friction Angle, ϕ_r	30.00 degrees	To be specified as a minimum
Cohesion, c_r	0.00 psf	Imported material
Total Unit Weight, γ_r	120.00 pcf	
RETAINED BACKFILL:		
Friction Angle, ϕ_f	34.00 degrees	For standard design use 34 deg. (BDS 5.9.2)
Cohesion, c_f	0.00 psf	Imported structural backfill
Total Unit Weight, γ_f	120.00 pcf	

CALCULATED PARAMETERS		
Static (Active) Earth Pressure Coefficient, K_a	0.283	Based on Rankine Theory (BDS C5.5.5.3)
Dynamic (Active) Earth Pressure Coefficient, K_{ae}	0.394	From Mononobe-Okabe Equation (Kramer p478-479)
Effective Wall Height for Static Active Thrust Force, H_1	30.00 ft	$H_1 = H + L \tan \beta$
Effective Wall Height for Dynamic Active Thrust Force, H_2	30.00 ft	$H_2 = H + (0.5\ H \tan \beta)/(1 - 0.5 \tan \beta)$ (FHWA-NHI-00-043 p.100)
Static Active Thrust Force From Retained Fill, F_T	15.27 kips/ft of wall	$F_T = 1/2\ \gamma_f K_a\ (H_1)^2$
Dynamic Active Thrust Force From Retained Fill, P_{ae}	5.99 kips/ft of wall	$P_{ae} = 1/2\ \gamma_f\ (K_{ae} - K_a)\ (H_2)^2$ (FHWA-NHI-00-043 p.100)
Horizontal Component of Static Active Thrust Force, F_1	15.27 kips/ft of wall	$F_1 = F_T \cos \beta$
Horizontal Force From Traffic Surcharge, F_2	2.04 kips/ft of wall	$F_2 = q\ H\ K_a$
Horizontal Inertial Force From Reinforced Soil Mass, P_{ir}	8.64 kips/ft of wall	$P_{ir} = 0.5\ k_h\ \gamma_f\ H_2\ H$ (FHWA-NHI-00-043 p.100)
Horizontal Inertial Force From Sloping Soil Surcharge, P_{is}	0.00 kips/ft of wall	$P_{is} = 0.125\ k_h\ \gamma_f\ (H_2)^2 \tan \beta$
Weight of Reinforced Soil Mass, V_1	75.60 kips/ft of wall	$V_1 = \gamma_r\ H\ L$
Weight of Sloping Backfill, V_2	0.00 kips/ft of wall	$V_2 = 1/2\ \gamma_f\ L\ (L \tan \beta)$
Sum of Resisting Moments, M_R	793.80 kip-ft/ft of wall	$M_R = V_1\ (L/2) + V_2\ (1/2\ L + 1/6\ L)$
Sum of Overturning Moments (Static), M_{OS}	183.20 kip-ft/ft of wall	$M_{OS} = F_1\ (H1/3) + F_2\ (H1/2)$
Sum of Overturning Moments (Static + Dynamic), M_{OD}	420.59 kip-ft/ft of wall	$M_{OD} = Mos + P_{ir}\ (H/2) + P_{is}\ [H + 1/3\ (H_2 - H)] + P_{ae} \cos \beta\ (0.6\ H_2)$
Sum of Vertical Forces, V_T	75.60 kips/ft of wall	$V_T = V_1 + V_2 + F_T \sin \beta$
Sum of Horizontal Resisting Forces, P_R	43.65 kips/ft of wall	$P_R = V_T \tan [\min (\phi_r, \phi_b)]$
Sum of Horizontal Driving Forces (Static), P_{DS}	17.30 kips/ft of wall	$P_{DS} = F_1 + F_2$
Sum of Horizontal Driving Forces (Static + Dynamic), P_{DD}	31.93 kips/ft of wall	$P_{DD} = P_{DS} + P_{ir} + P_{is} + P_{ae}$

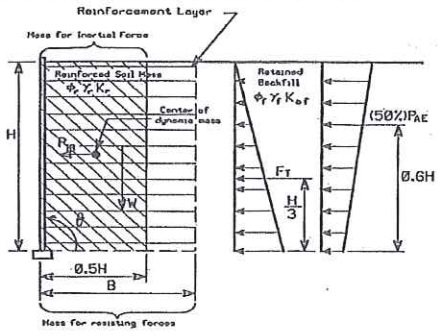
CHECK SLIDING FAILURE:		
Static Sliding Factor of Safety, $FS_{(static\ sliding)} = P_R/P_{DS}$	2.52 $\geq$ 1.5	**OK**
Dynamic Sliding Factor of Safety, $FS_{(dynamic\ sliding)} = P_R/P_{DD}$	1.37 $\geq$ 1.125	**OK**
CHECK OVERTURNING FAILURE:		
Static Overturning Factor of Safety, $FS_{(static\ overturning)} = M_R/M_{OS}$	4.33 $\geq$ 2.0	**OK**
Dynamic Overturning Factor of Safety, $FS_{(dynamic\ overturning)} = M_R/M_{OD}$	1.89 $\geq$ 1.5	**OK**
Static Eccentricity, $e_s = [L/2 - (M_R - M_{OS})/V_T]$	2.42 ft < L/6	**OK**
Dynamic Eccentricity, $e_d = [L/2 - (M_R - M_{OD})/V_T]$	5.56 ft < L/3	**OK**
CHECK BEARING FAILURE:		
Bearing Capacity Factor, Nc	30.14	Based on Caltrans DBS Table 4.4.7.1A (2003)
Bearing Capacity Factor, Nγ	22.40	Based on Caltrans DBS Table 4.4.7.1A (2003)
Max. Applied Bearing Pressure at the Base of the Wall, σ_v	4.68 ksf	$\sigma_v = V_T / (L - 2\ e_s)$
Ultimate Bearing Capacity, q_{ult}	19.90 ksf	$q_{ult} = c_b\ N_c + 0.5\ \gamma_b\ N_\gamma\ (L - 2\ e_s)$ (Embedment is neglected)
Bearing Capacity Factor of Safety, $FS_{(bearing\ capacity)}$	4.25 $\geq$ 2.0	**OK**

MSE WALL EXTERNAL STABILITY CHECK CALCULATIONS

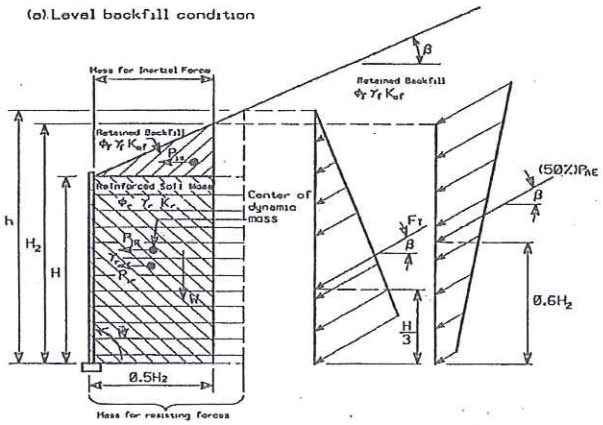
(EZ_MSEWall Version R1.0, 2007)

PROJECT INFORMATION	
Project Name	Crosstown Freeway, Retaining Wall 2, Stn. 880+50
Project No.	111904.G01
Location	Stockton, CA

INPUT PARAMETERS		
WALL DIMENSIONS		
Design Wall Height, H	34.00 ft	Maximum wall height
Wall Face Slope Angle, θ	90.00 degrees	Vertical wall =90 degrees
Wall Friction Angle, δ	0.00 degrees	Enter 0 for vertical walls
Backfill Slope Angle, β	0.00 degrees	
Wall Embedment Depth, D	3.40 ft	Minimum is 2 feet or 0.1H whichever is greater
Reinforcement Length, L	23.80 ft	Minimum is 70% of H or 8, whichever is greater
LOADS		
Traffic Surcharge, q	240.00 psf	Equiv. to 2-ft of overburden soil pressure
Horizontal Seismic Coefficient, k_h	0.16	kh = 1/2 PGA for Mononobe-Okabe Pressure
Vertical Seismic Coefficient, k_v	0.10	
FOUNDATION SOIL		
Friction Angle, ϕ_b	30.00 degrees	For standard design use 30 deg. (BDS 5.9.2)
Cohesion, c_b	0.00 psf	
Total Unit Weight, γ_b	110.00 pcf	
REINFORCED SOIL MASS		
Friction Angle, ϕ_r	30.00 degrees	To be specified as a minimum
Cohesion, c_r	0.00 psf	Imported material
Total Unit Weight, γ_r	120.00 pcf	
RETAINED BACKFILL		
Friction Angle, ϕ_f	34.00 degrees	For standard design use 34 deg. (BDS 5.9.2)
Cohesion, c_f	0.00 psf	Imported structural backfill
Total Unit Weight, γ_f	120.00 pcf	



(a) Level backfill condition



(b) Sloping backfill condition

References:

1. **Federal Highway Administration (FHWA), "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design & Construction Guidelines," Publication No. FHWA-NHI-00-043, March 2001.**
2. **Caltrans Memo to Designers 5-8, "Mechanically Stabilized Embankment," January 1991.**
3. **Caltrans Bridge Design Aids, "Mechanically Stabilized Embankment," pp. 3-8.1 - 3-8.10, April 2002.**

MSE WALL EXTERNAL STABILITY CHECK CALCULATIONS

(EZ_MSEWall Version R1.0, 2007)

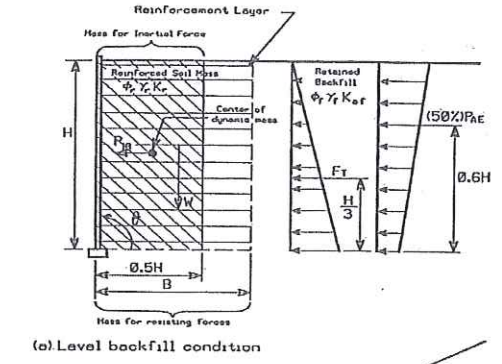
PROJECT INFORMATION		
Project Name	Crosstown Freeway, Retaining Wall 2, Stn. 880+50	
Project No.	111904.G01	
Wall Description	Stockton, CA	
INPUT PARAMETERS		
WALL DIMENSIONS:		
Design Height, H	34.00 ft	Maximum wall height
Wall Face Slope Angle, θ	90.00 degrees	Vertical wall =90 degrees
Wall Friction Angle, δ	0.00	Enter 0 for vertical walls
Backfill Slope Angle, β	0.00 degrees	
Wall Embedment Depth, D	3.40 ft	Minimum is 2 feet or 0.1H whichever is greater
Reinforcement Length, L	23.80 ft	Minimum is 70% of H or 8, whichever is greater
LOADS:		
Traffic Surcharge, q	240.00 psf	Equiv. to 2-ft of overburden soil pressure
Horizontal Seismic Coefficient, k_h	0.16	kh = 1/2 PGA for Mononobe-Okabe Pressure
Vertical Seismic Coefficient, k_v	0.10	
FOUNDATION SOIL:		
Friction Angle, ϕ_b	30.00 degrees	For standard design use 30 deg. (BDS 5.9.2)
Cohesion, c_b	0.00 psf	
Total Unit Weight, γ_b	110.00 pcf	
REINFORCED SOIL MASS:		
Friction Angle, ϕ_r	30.00 degrees	To be specified as a minimum
Cohesion, c_r	0.00 psf	Imported material
Total Unit Weight, γ_r	120.00 pcf	
RETAINED BACKFILL:		
Friction Angle, ϕ_f	34.00 degrees	For standard design use 34 deg. (BDS 5.9.2)
Cohesion, c_f	0.00 psf	Imported structural backfill
Total Unit Weight, γ_f	120.00 pcf	
CALCULATED PARAMETERS		
Static (Active) Earth Pressure Coefficient, K_a	0.283	Based on Rankine Theory (BDS C5.5.5.3)
Dynamic (Active) Earth Pressure Coefficient, K_{ae}	0.394	From Mononobe-Okabe Equation (Kramer p478-479)
Effective Wall Height for Static Active Thrust Force, H_1	34.00 ft	$H_1 = H + L \tan\beta$
Effective Wall Height for Dynamic Active Thrust Force, H_2	34.00 ft	$H_2 = H + (0.5\ H \tan\beta)/(1 - 0.5 \tan\beta)$ (FHWA-NHI-00-043 p.100)
Static Active Thrust Force From Retained Fill, F_T	19.61 kips/ft of wall	$F_T = 1/2\ \gamma_f K_a\ (H_1)^2$
Dynamic Active Thrust Force From Retained Fill, P_{ae}	7.69 kips/ft of wall	$P_{ae} = 1/2\ \gamma_f\ (K_{ae} - K_a)\ (H_2)^2$ (FHWA-NHI-00-043 p.100)
Horizontal Component of Static Active Thrust Force, F_1	19.61 kips/ft of wall	$F_1 = F_T \cos\beta$
Horizontal Force From Traffic Surcharge, F_2	2.31 kips/ft of wall	$F_2 = q\ H\ K_a$
Horizontal Inertial Force From Reinforced Soil Mass, P_{ir}	11.10 kips/ft of wall	$P_{ir} = 0.5\ k_h\ \gamma_f\ H_2\ H$ (FHWA-NHI-00-043 p.100)
Horizontal Inertial Force From Sloping Soil Surcharge, P_{is}	0.00 kips/ft of wall	$P_{is} = 0.125\ k_h\ \gamma_f\ (H_2)^2 \tan\beta$
Weight of Reinforced Soil Mass, V_1	97.10 kips/ft of wall	$V_1 = \gamma_r\ H\ L$
Weight of Sloping Backfill, V_2	0.00 kips/ft of wall	$V_2 = 1/2\ \gamma_f\ L\ (L \tan\beta)$
Sum of Resisting Moments, M_R	1,155.54 kip-ft/ft of wall	$M_R = V_1\ (L/2) + V_2\ (1/2\ L+1/6\ L)$
Sum of Overturning Moments (Static), M_{OS}	261.45 kip-ft/ft of wall	$M_{OS} = F_1\ (H1/3) + F_2\ (H1/2)$
Sum of Overturning Moments (Static + Dynamic), M_{OD}	607.02 kip-ft/ft of wall	$M_{OD} = Mos + P_{ir}\ (H/2) + P_{is}\ [H+ 1/3\ (H_2-H)] + P_{ae} \cos\beta\ (0.6\ H_2)$
Sum of Vertical Forces, V_T	97.10 kips/ft of wall	$V_T = V_1 + V_2 + F_T \sin\beta$
Sum of Horizontal Resisting Forces, P_R	56.06 kips/ft of wall	$P_R = V_T \tan[\min(\phi_r, \phi_b)]$
Sum of Horizontal Driving Forces (Static), P_{DS}	21.92 kips/ft of wall	$P_{DS} = F_1 + F_2$
Sum of Horizontal Driving Forces (Static + Dynamic), P_{DD}	40.71 kips/ft of wall	$P_{DD} = P_{DS} + P_{ir} + P_{is} + P_{ae}$
CHECK SLIDING FAILURE:		
Static Sliding Factor of Safety, $FS_{(static\ sliding)} = P_R/P_{DS}$	2.56 ≥ 1.5	**OK**
Dynamic Sliding Factor of Safety, $FS_{(dynamic\ sliding)} = P_R/P_{DD}$	1.38 ≥ 1.125	**OK**
CHECK OVERTURNING FAILURE:		
Static Overturning Factor of Safety, $FS_{(static\ overturning)} = M_R/M_{OS}$	4.42 ≥ 2.0	**OK**
Dynamic Overturning Factor of Safety, $FS_{(dynamic\ overturning)} = M_R/M_{OD}$	1.90 ≥ 1.5	**OK**
Static Eccentricity, $e_s = [L/2 - (M_R-M_{OS})/V_T]$	2.69 ft < L/6	**OK**
Dynamic Eccentricity, $e_d = [L/2 - (M_R-M_{OD})/V_T]$	6.25 ft < L/3	**OK**
CHECK BEARING FAILURE:		
Bearing Capacity Factor, Nc	30.14	Based on Caltrans DBS Table 4.4.7.1A (2003)
Bearing Capacity Factor, Nγ	22.40	Based on Caltrans DBS Table 4.4.7.1A (2003)
Max. Applied Bearing Pressure at the Base of the Wall, σ_v	5.27 ksf	$\sigma_v = V_T / (L - 2\ e_s)$
Ultimate Bearing Capacity, q_{ult}	22.69 ksf	$q_{ult} = c_b\ N_c + 0.5\ \gamma_b\ N_\gamma\ (L - 2\ e_s)$ (Embedment is neglected)
Bearing Capacity Factor of Safety, $FS_{(bearing\ capacity)}$	4.30 ≥ 2.0	**OK**

MSE WALL EXTERNAL STABILITY CHECK CALCULATIONS

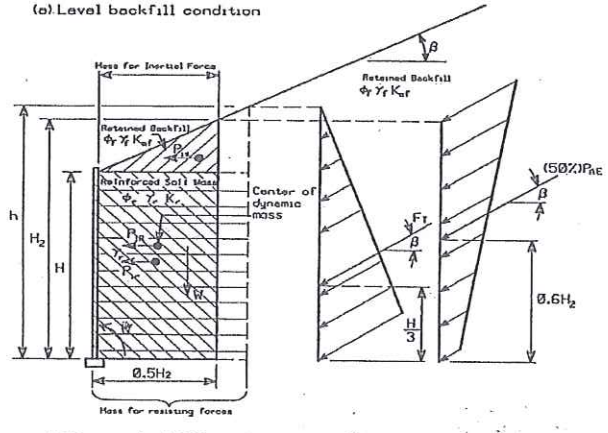
(EZ_MSEWall Version R1.0, 2007)

PROJECT INFORMATION	
Project Name	Crosstown Freeway, Retaining Wall 2, Stn. 882+50
Project No.	111904.G01
Location	Stockton, CA

INPUT PARAMETERS		
WALL DIMENSIONS		
Design Wall Height, H	38.00 ft	Maximum wall height
Wall Face Slope Angle, θ	90.00 degrees	Vertical wall =90 degrees
Wall Friction Angle, δ	0.00 degrees	Enter 0 for vertical walls
Backfill Slope Angle, β	0.00 degrees	
Wall Embedment Depth, D	3.80 ft	Minimum is 2 feet or 0.1H whichever is greater
Reinforcement Length, L	26.60 ft	Minimum is 70% of H or 8, whichever is greater
LOADS		
Traffic Surcharge, q	240.00 psf	Equiv. to 2-ft of overburden soil pressure
Horizontal Seismic Coefficient, k_h	0.16	kh = 1/2 PGA for Mononobe-Okabe Pressure
Vertical Seismic Coefficient, k_v	0.10	
FOUNDATION SOIL		
Friction Angle, ϕ_b	30.00 degrees	For standard design use 30 deg. (BDS 5.9.2)
Cohesion, c_b	0.00 psf	
Total Unit Weight, γ_b	110.00 pcf	
REINFORCED SOIL MASS		
Friction Angle, ϕ_r	30.00 degrees	To be specified as a minimum
Cohesion, c_r	0.00 psf	Imported material
Total Unit Weight, γ_r	120.00 pcf	
RETAINED BACKFILL		
Friction Angle, ϕ_f	34.00 degrees	For standard design use 34 deg. (BDS 5.9.2)
Cohesion, c_f	0.00 psf	Imported structural backfill
Total Unit Weight, γ_f	120.00 pcf	



(a) Level backfill condition



(b) Sloping backfill condition

References:

1. **Federal Highway Administration (FHWA), "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design & Construction Guidelines," Publication No. FHWA-NHI-00-043, March 2001.**
2. **Caltrans Memo to Designers 5-8, "Mechanically Stabilized Embankment," January 1991.**
3. **Caltrans Bridge Design Aids, "Mechanically Stabilized Embankment," pp. 3-8.1 - 3-8.10, April 2002.**

MSE WALL EXTERNAL STABILITY CHECK CALCULATIONS

(EZ_MSEWall Version R1.0, 2007)

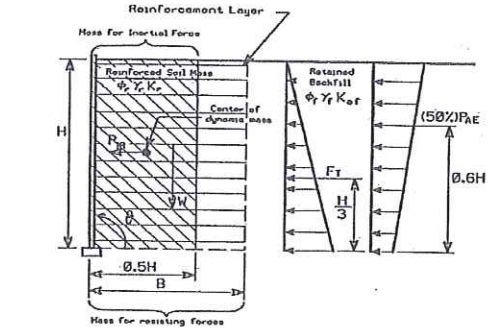
PROJECT INFORMATION		
Project Name	Crosstown Freeway, Retaining Wall 2, Stn. 882+50	
Project No.	111904.G01	
Wall Description	Stockton, CA	
INPUT PARAMETERS		
WALL DIMENSIONS:		
Design Height, H	38.00 ft	Maximum wall height
Wall Face Slope Angle, θ	90.00 degrees	Vertical wall =90 degrees
Wall Friction Angle, δ	0.00	Enter 0 for vertical walls
Backfill Slope Angle, β	0.00 degrees	
Wall Embedment Depth, D	3.80 ft	Minimum is 2 feet or 0.1H whichever is greater
Reinforcement Length, L	26.60 ft	Minimum is 70% of H or 8, whichever is greater
LOADS:		
Traffic Surcharge, q	240.00 psf	Equiv. to 2-ft of overburden soil pressure
Horizontal Seismic Coefficient, k_h	0.16	kh = 1/2 PGA for Mononobe-Okabe Pressure
Vertical Seismic Coefficient, k_v	0.10	
FOUNDATION SOIL:		
Friction Angle, ϕ_b	30.00 degrees	For standard design use 30 deg. (BDS 5.9.2)
Cohesion, c_b	0.00 psf	
Total Unit Weight, γ_b	110.00 pcf	
REINFORCED SOIL MASS:		
Friction Angle, ϕ_r	30.00 degrees	To be specified as a minimum
Cohesion, c_r	0.00 psf	Imported material
Total Unit Weight, γ_r	120.00 pcf	
RETAINED BACKFILL:		
Friction Angle, ϕ_f	34.00 degrees	For standard design use 34 deg. (BDS 5.9.2)
Cohesion, c_f	0.00 psf	Imported structural backfill
Total Unit Weight, γ_f	120.00 pcf	
CALCULATED PARAMETERS		
Static (Active) Earth Pressure Coefficient, K_a	0.283	Based on Rankine Theory (BDS C5.5.5.3)
Dynamic (Active) Earth Pressure Coefficient, K_{ae}	0.394	From Mononobe-Okabe Equation (Kramer p478-479)
Effective Wall Height for Static Active Thrust Force, H_1	38.00 ft	$H_1 = H + L \tan\beta$
Effective Wall Height for Dynamic Active Thrust Force, H_2	38.00 ft	$H_2 = H + (0.5\ H \tan\beta)/(1 - 0.5 \tan\beta)$ (FHWA-NHI-00-043 p.100)
Static Active Thrust Force From Retained Fill, F_T	24.49 kips/ft of wall	$F_T = 1/2\ \gamma_f K_a\ (H_1)^2$
Dynamic Active Thrust Force From Retained Fill, P_{ae}	9.61 kips/ft of wall	$P_{ae} = 1/2\ \gamma_f\ (K_{ae} - K_a)\ (H_2)^2$ (FHWA-NHI-00-043 p.100)
Horizontal Component of Static Active Thrust Force, F_1	24.49 kips/ft of wall	$F_1 = F_T \cos\beta$
Horizontal Force From Traffic Surcharge, F_2	2.58 kips/ft of wall	$F_2 = q\ H\ K_a$
Horizontal Inertial Force From Reinforced Soil Mass, P_{ir}	13.86 kips/ft of wall	$P_{ir} = 0.5\ k_h\ \gamma_f\ H_2\ H$ (FHWA-NHI-00-043 p.100)
Horizontal Inertial Force From Sloping Soil Surcharge, P_{is}	0.00 kips/ft of wall	$P_{is} = 0.125\ k_h\ \gamma_f\ (H_2)^2 \tan\beta$
Weight of Reinforced Soil Mass, V_1	121.30 kips/ft of wall	$V_1 = \gamma_r\ H\ L$
Weight of Sloping Backfill, V_2	0.00 kips/ft of wall	$V_2 = 1/2\ \gamma_f\ L\ (L \tan\beta)$
Sum of Resisting Moments, M_R	1,613.24 kip-ft/ft of wall	$M_R = V_1\ (L/2) + V_2\ (1/2\ L+1/6\ L)$
Sum of Overturning Moments (Static), M_{OS}	359.25 kip-ft/ft of wall	$M_{OS} = F_1\ (H1/3) + F_2\ (H1/2)$
Sum of Overturning Moments (Static + Dynamic), M_{OD}	841.70 kip-ft/ft of wall	$M_{OD} = Mos + P_{ir}\ (H/2) + P_{is}\ [H+ 1/3\ (H_2-H)] + P_{ae} \cos\beta\ (0.6\ H_2)$
Sum of Vertical Forces, V_T	121.30 kips/ft of wall	$V_T = V_1 + V_2 + F_T \sin\beta$
Sum of Horizontal Resisting Forces, P_R	70.03 kips/ft of wall	$P_R = V_T \tan[\min(\phi_r, \phi_b)]$
Sum of Horizontal Driving Forces (Static), P_{DS}	27.07 kips/ft of wall	$P_{DS} = F_1 + F_2$
Sum of Horizontal Driving Forces (Static + Dynamic), P_{DD}	50.54 kips/ft of wall	$P_{DD} = P_{DS} + P_{ir} + P_{is} + P_{ae}$
CHECK SLIDING FAILURE:		
Static Sliding Factor of Safety, $FS_{(static\ sliding)} = P_R/P_{DS}$	2.59 ≥ 1.5	**OK**
Dynamic Sliding Factor of Safety, $FS_{(dynamic\ sliding)} = P_R/P_{DD}$	1.39 ≥ 1.125	**OK**
CHECK OVERTURNING FAILURE:		
Static Overturning Factor of Safety, $FS_{(static\ overturning)} = M_R/M_{OS}$	4.49 ≥ 2.0	**OK**
Dynamic Overturning Factor of Safety, $FS_{(dynamic\ overturning)} = M_R/M_{OD}$	1.92 ≥ 1.5	**OK**
Static Eccentricity, $e_s = [L/2 - (M_R-M_{OS})/V_T]$	2.96 ft < L/6	**OK**
Dynamic Eccentricity, $e_d = [L/2 - (M_R-M_{OD})/V_T]$	6.94 ft < L/3	**OK**
CHECK BEARING FAILURE:		
Bearing Capacity Factor, Nc	30.14	Based on Caltrans DBS Table 4.4.7.1A (2003)
Bearing Capacity Factor, Nγ	22.40	Based on Caltrans DBS Table 4.4.7.1A (2003)
Max. Applied Bearing Pressure at the Base of the Wall, σ_v	5.87 ksf	$\sigma_v = V_T / (L - 2\ e_s)$
Ultimate Bearing Capacity, q_{ult}	25.47 ksf	$q_{ult} = c_b\ N_c + 0.5\ \gamma_b\ N_\gamma\ (L - 2\ e_s)$ (Embedment is neglected)
Bearing Capacity Factor of Safety, $FS_{(bearing\ capacity)}$	4.34 ≥ 2.0	**OK**

MSE WALL EXTERNAL STABILITY CHECK CALCULATIONS

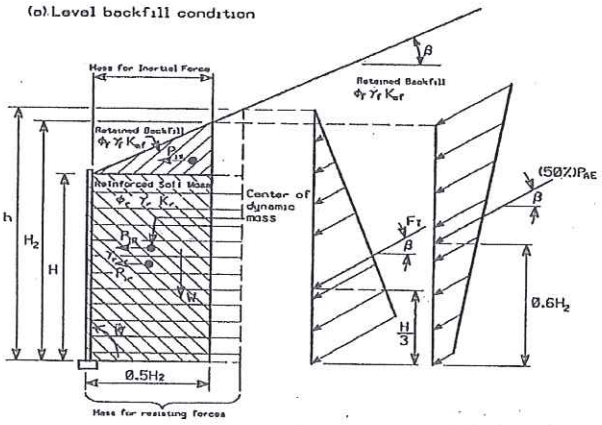
(EZ_MSEWall Version R1.0, 2007)

PROJECT INFORMATION	
Project Name	Crosstown Freeway, Retaining Wall 3, Stn. 10+50
Project No.	111904.G01
Location	Stockton, CA

INPUT PARAMETERS		
WALL DIMENSIONS		
Design Wall Height, H	42.00 ft	Maximum wall height
Wall Face Slope Angle, θ	90.00 degrees	Vertical wall =90 degrees
Wall Friction Angle, δ	0.00 degrees	Enter 0 for vertical walls
Backfill Slope Angle, β	0.00 degrees	
Wall Embedment Depth, D	4.20 ft	Minimum is 2 feet or 0.1H whichever is greater
Reinforcement Length, L	29.40 ft	Minimum is 70% of H or 8, whichever is greater
LOADS		
Traffic Surcharge, q	240.00 psf	Equiv. to 2-ft of overburden soil pressure
Horizontal Seismic Coefficient, k_h	0.16	kh = 1/2 PGA for Mononobe-Okabe Pressure
Vertical Seismic Coefficient, k_v	0.10	
FOUNDATION SOIL		
Friction Angle, ϕ_b	30.00 degrees	For standard design use 30 deg. (BDS 5.9.2)
Cohesion, c_b	0.00 psf	
Total Unit Weight, γ_b	110.00 pcf	
REINFORCED SOIL MASS		
Friction Angle, ϕ_r	30.00 degrees	To be specified as a minimum
Cohesion, c_r	0.00 psf	Imported material
Total Unit Weight, γ_r	120.00 pcf	
RETAINED BACKFILL		
Friction Angle, ϕ_f	34.00 degrees	For standard design use 34 deg. (BDS 5.9.2)
Cohesion, c_f	0.00 psf	Imported structural backfill
Total Unit Weight, γ_f	120.00 pcf	



(a) Level backfill condition



(b) Sloping backfill condition

References:

1. **Federal Highway Administration (FHWA), "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design & Construction Guidelines," Publication No. FHWA-NHI-00-043, March 2001.**
2. **Caltrans Memo to Designers 5-8, "Mechanically Stabilized Embankment," January 1991.**
3. **Caltrans Bridge Design Aids, "Mechanically Stabilized Embankment," pp. 3-8.1 - 3-8.10, April 2002.**

MSE WALL EXTERNAL STABILITY CHECK CALCULATIONS

(EZ_MSEWall Version R1.0, 2007)

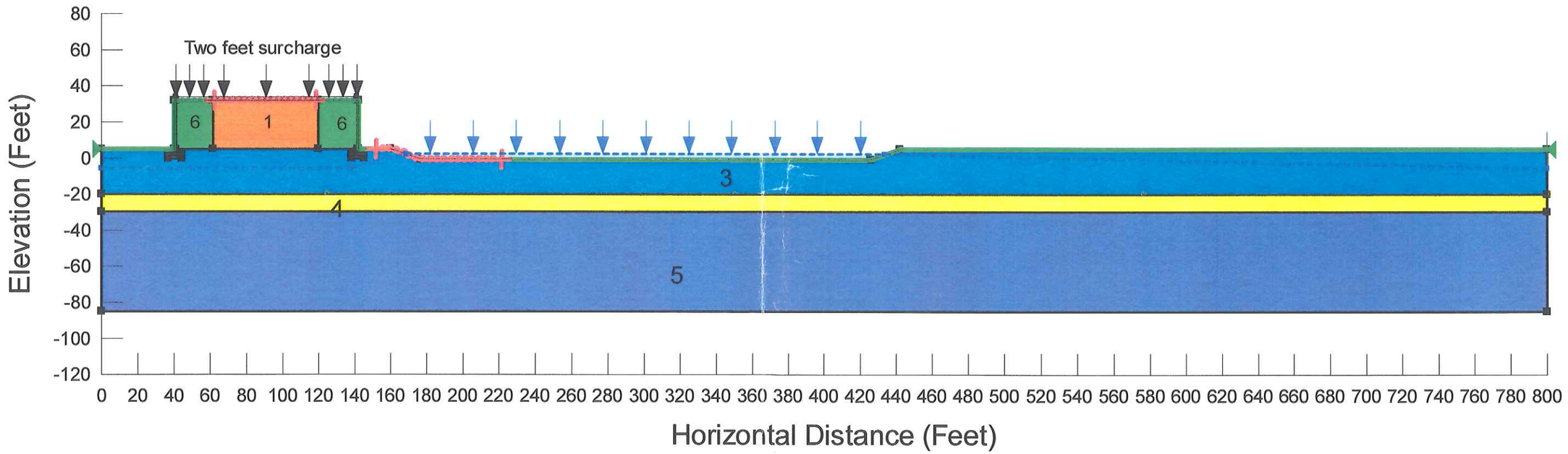
PROJECT INFORMATION		
Project Name	Crosstown Freeway, Retaining Wall 3, Stn. 10+50	
Project No.	111904.G01	
Wall Description	Stockton, CA	
INPUT PARAMETERS		
WALL DIMENSIONS:		
Design Height, H	42.00 ft	Maximum wall height
Wall Face Slope Angle, θ	90.00 degrees	Vertical wall =90 degrees
Wall Friction Angle, δ	0.00	Enter 0 for vertical walls
Backfill Slope Angle, β	0.00 degrees	
Wall Embedment Depth, D	4.20 ft	Minimum is 2 feet or 0.1H whichever is greater
Reinforcement Length, L	29.40 ft	Minimum is 70% of H or 8, whichever is greater
LOADS:		
Traffic Surcharge, q	240.00 psf	Equiv. to 2-ft of overburden soil pressure
Horizontal Seismic Coefficient, k_h	0.16	kh = 1/2 PGA for Mononobe-Okabe Pressure
Vertical Seismic Coefficient, k_v	0.10	
FOUNDATION SOIL:		
Friction Angle, ϕ_b	30.00 degrees	For standard design use 30 deg. (BDS 5.9.2)
Cohesion, c_b	0.00 psf	
Total Unit Weight, γ_b	110.00 pcf	
REINFORCED SOIL MASS:		
Friction Angle, ϕ_r	30.00 degrees	To be specified as a minimum
Cohesion, c_r	0.00 psf	Imported material
Total Unit Weight, γ_r	120.00 pcf	
RETAINED BACKFILL:		
Friction Angle, ϕ_f	34.00 degrees	For standard design use 34 deg. (BDS 5.9.2)
Cohesion, c_f	0.00 psf	Imported structural backfill
Total Unit Weight, γ_f	120.00 pcf	
CALCULATED PARAMETERS		
Static (Active) Earth Pressure Coefficient, K_a	0.283	Based on Rankine Theory (BDS C5.5.5.3)
Dynamic (Active) Earth Pressure Coefficient, K_{ae}	0.394	From Mononobe-Okabe Equation (Kramer p478-479)
Effective Wall Height for Static Active Thrust Force, H_1	42.00 ft	$H_1 = H + L \tan\beta$
Effective Wall Height for Dynamic Active Thrust Force, H_2	42.00 ft	$H_2 = H + (0.5\ H \tan\beta)/(1 - 0.5 \tan\beta)$ (FHWA-NHI-00-043 p.100)
Static Active Thrust Force From Retained Fill, F_T	29.92 kips/ft of wall	$F_T = 1/2\ \gamma_f K_a\ (H_1)^2$
Dynamic Active Thrust Force From Retained Fill, P_{ae}	11.74 kips/ft of wall	$P_{ae} = 1/2\ \gamma_f\ (K_{ae} - K_a)\ (H_2)^2$ (FHWA-NHI-00-043 p.100)
Horizontal Component of Static Active Thrust Force, F_1	29.92 kips/ft of wall	$F_1 = F_T \cos\beta$
Horizontal Force From Traffic Surcharge, F_2	2.85 kips/ft of wall	$F_2 = q\ H\ K_a$
Horizontal Inertial Force From Reinforced Soil Mass, P_{ir}	16.93 kips/ft of wall	$P_{ir} = 0.5\ k_h\ \gamma_f\ H_2\ H$ (FHWA-NHI-00-043 p.100)
Horizontal Inertial Force From Sloping Soil Surcharge, P_{is}	0.00 kips/ft of wall	$P_{is} = 0.125\ k_h\ \gamma_f\ (H_2)^2 \tan\beta$
Weight of Reinforced Soil Mass, V_1	148.18 kips/ft of wall	$V_1 = \gamma_r\ H\ L$
Weight of Sloping Backfill, V_2	0.00 kips/ft of wall	$V_2 = 1/2\ \gamma_f\ L\ (L \tan\beta)$
Sum of Resisting Moments, M_R	2,178.19 kip-ft/ft of wall	$M_R = V_1\ (L/2) + V_2\ (1/2\ L + 1/6\ L)$
Sum of Overturning Moments (Static), M_{OS}	478.76 kip-ft/ft of wall	$M_{OS} = F_1\ (H1/3) + F_2\ (H1/2)$
Sum of Overturning Moments (Static + Dynamic), M_{OD}	1,130.16 kip-ft/ft of wall	$M_{OD} = Mos + P_{ir}\ (H/2) + P_{is}\ [H + 1/3\ (H_2 - H)] + P_{ae} \cos\beta\ (0.6\ H_2)$
Sum of Vertical Forces, V_T	148.18 kips/ft of wall	$V_T = V_1 + V_2 + F_T \sin\beta$
Sum of Horizontal Resisting Forces, P_R	85.55 kips/ft of wall	$P_R = V_T \tan[\min(\phi_r, \phi_b)]$
Sum of Horizontal Driving Forces (Static), P_{DS}	32.77 kips/ft of wall	$P_{DS} = F_1 + F_2$
Sum of Horizontal Driving Forces (Static + Dynamic), P_{DD}	61.44 kips/ft of wall	$P_{DD} = P_{DS} + P_{ir} + P_{is} + P_{ae}$
CHECK SLIDING FAILURE:		
Static Sliding Factor of Safety, $FS_{(static\ sliding)} = P_R/P_{DS}$	2.61 ≥ 1.5	**OK**
Dynamic Sliding Factor of Safety, $FS_{(dynamic\ sliding)} = P_R/P_{DD}$	1.39 ≥ 1.125	**OK**
CHECK OVERTURNING FAILURE:		
Static Overturning Factor of Safety, $FS_{(static\ overturning)} = M_R/M_{OS}$	4.55 ≥ 2.0	**OK**
Dynamic Overturning Factor of Safety, $FS_{(dynamic\ overturning)} = M_R/M_{OD}$	1.93 ≥ 1.5	**OK**
Static Eccentricity, $e_s = [L/2 - (M_R - M_{OS})/V_T]$	3.23 ft < L/6	**OK**
Dynamic Eccentricity, $e_d = [L/2 - (M_R - M_{OD})/V_T]$	7.63 ft < L/3	**OK**
CHECK BEARING FAILURE:		
Bearing Capacity Factor, Nc	30.14	Based on Caltrans DBS Table 4.4.7.1A (2003)
Bearing Capacity Factor, Nγ	22.40	Based on Caltrans DBS Table 4.4.7.1A (2003)
Max. Applied Bearing Pressure at the Base of the Wall, σ_v	6.46 ksf	$\sigma_v = V_T / (L - 2\ e_s)$
Ultimate Bearing Capacity, q_{ult}	28.26 ksf	$q_{ult} = c_b\ N_c + 0.5\ \gamma_b\ N_\gamma\ (L - 2\ e_s)$ (Embedment is neglected)
Bearing Capacity Factor of Safety, $FS_{(bearing\ capacity)}$	4.37 ≥ 2.0	**OK**

Global Stability Analysis – Static Condition

Slope stability Analysis with Seismic Load for MSE Wall RW-1
File Name: Slope Stability-MSE Wall-RW1-Water in Basin-Seismic.gsz
Date: 7/14/2011
Original grouns Surface Elevation: 5.08 ft
Bottom of Basin elevation: -0.5 ft
Height of Retaining Wall: 30 ft
Distance between retaining wall and the Basin: 15 ft
Seismic Load = 0.16

Material Number, Description, Unit Weight, Cohesion, Friction Angle

Material # 1: Retained Backfill Unit Weight: 120 pcf; Cohesion: 0 psf; Friction Angle: Value: 34 degrees
Material # 2: MSE Wall Unit Weight: 150 pcf; Cohesion: 2000 psf; Friction Angle: Value: 40 degrees
Material # 3: Clay Unit Weight: 120 pcf; Cohesion: 100 psf; Friction Angle: Value: 25 degrees
Material # 4: Sand Unit Weight: 120 pcf; Cohesion: 0 psf; Friction Angle: Value: 33 degrees
Material # 5: Very Stiff Clay Unit Weight: 125 pcf; Cohesion: 250 psf; Friction Angle: Value: 28 degrees
Material # 6: Reinforced Material Unit Weight: 120 pcf; Cohesion: 2000 psf; Friction Angle: Value: 42 degrees
Parameters selected to force failure circles outside of reinforced area



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PROJECT NO.	111904.G0
DATE:	7/14/11
DRAWN BY:	H. Parsa
CHECKED BY:	C. Henderson

Station 878+50
Near the Proposed Basin

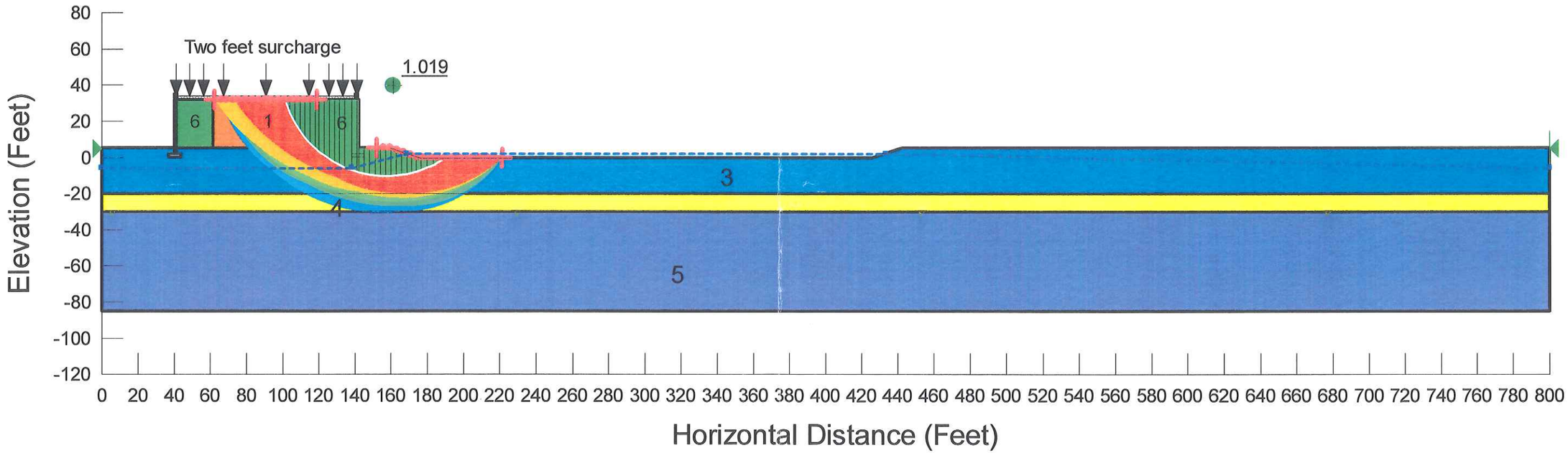
MSE Wall Global Stability Evaluation
Crosstown Freeway DR 4
Stockton, CALIFORNIA

PLATE

Slope stability Analysis with Seismic Load for MSE Wall RW-1
File Name: Slope Stability-MSE Wall-RW1-Water in Basin-Seismic.gsz
Date: 7/14/2011
Original grouns Surface Elevation: 5.08 ft
Bottom of Basin elevation: -0.5 ft
Height of Retaining Wall: 30 ft
Distance between retaining wall and the Basin: 15 ft
Seismic Load = 0.16

Material Number, Description, Unit Weight, Cohesion, Friction Angle

Material # 1: Embankment Unit Weight: 120 pcf; Cohesion: 0 psf; Friction Angle: Value: 34 degrees
Material # 2: MSE Wall Unit Weight: 150 pcf; Cohesion: 2000 psf; Friction Angle: Value: 40 degrees
Material # 3: Clay Unit Weight: 120 pcf; Cohesion: 100 psf; Friction Angle: Value: 25 degrees
Material # 4: Sand Unit Weight: 120 pcf; Cohesion: 0 psf; Friction Angle: Value: 33 degrees
Material # 5: Very Stiff Clay Unit Weight: 125 pcf; Cohesion: 250 psf; Friction Angle: Value: 28 degrees
Material # 6: Reinforced Material Unit Weight: 120 pcf; Cohesion: 2000 psf; Friction Angle: Value: 42 degrees
Parameters selected to force failure circles outside of reinforced area



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PROJECT NO.	111904.G0
DATE:	7/14/11
DRAWN BY:	H. Parsa
CHECKED BY:	C. Henderson

Station 878+50
Near the Proposed Basin

MSE Wall Global Stability Evaluation
Crosstown Freeway DR 4
Stockton, CALIFORNIA

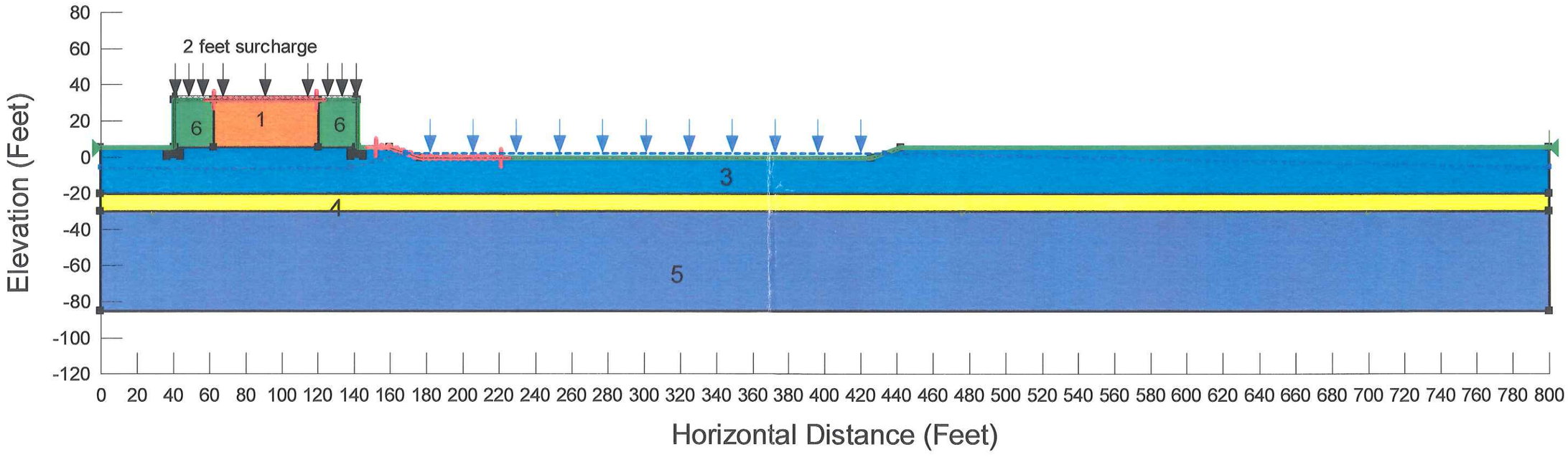
PLATE

Global Stability Analysis – Seismic Condition

Slope stability Analysis for MSE Wall RW-1
File Name: Slope Stability-MSE Wall-RW1-Water in Basin-Static.gsz
Date: 7/14/2011
Original grouns Surface Elevation: 5.08 ft
Bottom of Basin elevation: -0.5 ft
Height of Retaining Wall: 30 ft
Distance between retaining wall and the Basin: 15 ft
Seismic Load = 0

Material Number, Description, Unit Weight, Cohesion, Friction Angle

Material # 1: Retained Backfill Unit Weight: 120 pcf; Cohesion: 0 psf; Friction Angle: Value: 34 degrees
Material # 2: MSE Wall Unit Weight: 150 pcf; Cohesion: 2000 psf; Friction Angle: Value: 40 degrees
Material # 3: Clay Unit Weight: 120 pcf; Cohesion: 100 psf; Friction Angle: Value: 25 degrees
Material # 4: Sand Unit Weight: 120 pcf; Cohesion: 0 psf; Friction Angle: Value: 33 degrees
Material # 5: Very Stiff Clay Unit Weight: 125 pcf; Cohesion: 250 psf; Friction Angle: Value: 28 degrees
Material # 6: Reinforced Material Unit Weight: 120 pcf; Cohesion: 2000 psf; Friction Angle: Value: 42 degrees
Parameters selected to force failure circles outside of reinforced area



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Station 878+50
Near the Proposed Basin

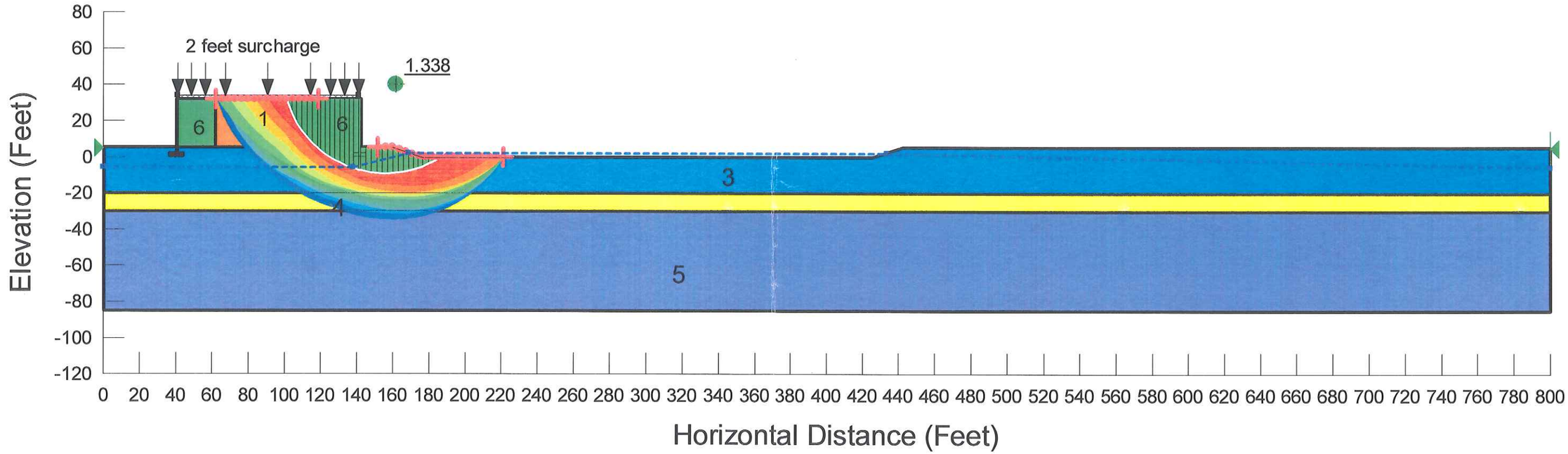
MSE Wall Global Stability Evaluation
Crosstown Freeway DR 4
Stockton, CALIFORNIA

PLATE

Slope stability Analysis for MSE Wall RW-1
File Name: Slope Stability-MSE Wall-RW1-Water in Basin-Static.gsz
Date: 7/14/2011
Original grouns Surface Elevation: 5.08 ft
Bottom of Basin elevation: -0.5 ft
Height of Retaining Wall: 30 ft
Distance between retaining wall and the Basin: 15 ft
Seismic Load = 0

Material Number, Description, Unit Weight, Cohesion, Friction Angle

Material # 1: Embankment Unit Weight: 120 pcf; Cohesion: 0 psf; Friction Angle: Value: 34 degrees
Material # 2: MSE Wall Unit Weight: 150 pcf; Cohesion: 2000 psf; Friction Angle: Value: 40 degrees
Material # 3: Clay Unit Weight: 120 pcf; Cohesion: 100 psf; Friction Angle: Value: 25 degrees
Material # 4: Sand Unit Weight: 120 pcf; Cohesion: 0 psf; Friction Angle: Value: 33 degrees
Material # 5: Very Stiff Clay Unit Weight: 125 pcf; Cohesion: 250 psf; Friction Angle: Value: 28 degrees
Material # 6: Reinforced Material Unit Weight: 120 pcf; Cohesion: 2000 psf; Friction Angle: Value: 42 degrees
Parameters selected to force failure circles outside of reinforced area



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PROJECT NO.	111904.G0
DATE:	7/14/11
DRAWN BY:	H. Parsa
CHECKED BY:	C. Henderson

Station 878+50
Near the Proposed Basin

MSE Wall Global Stability Evaluation
Crosstown Freeway DR 4
Stockton, CALIFORNIA

PLATE

APPENDIX J
GABION BASKET WALL ANALYSIS

LIST OF ATTACHMENTS

The following are attached and complete this appendix.

Conceptual Plan – Partial Filling of LKQ Property Basin

Terra Aqua Gabions Gravity Retaining Wall Calculations

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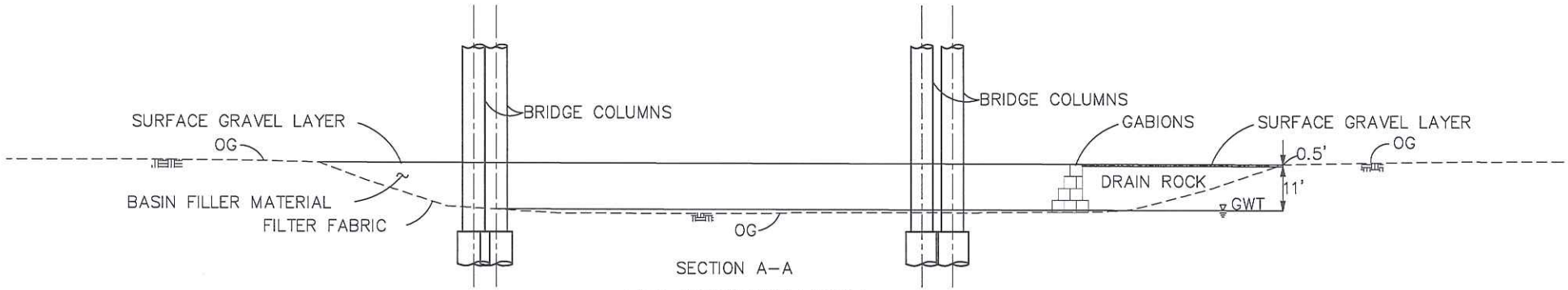
Conceptual Plan – Partial Filling of LKQ Property Basin



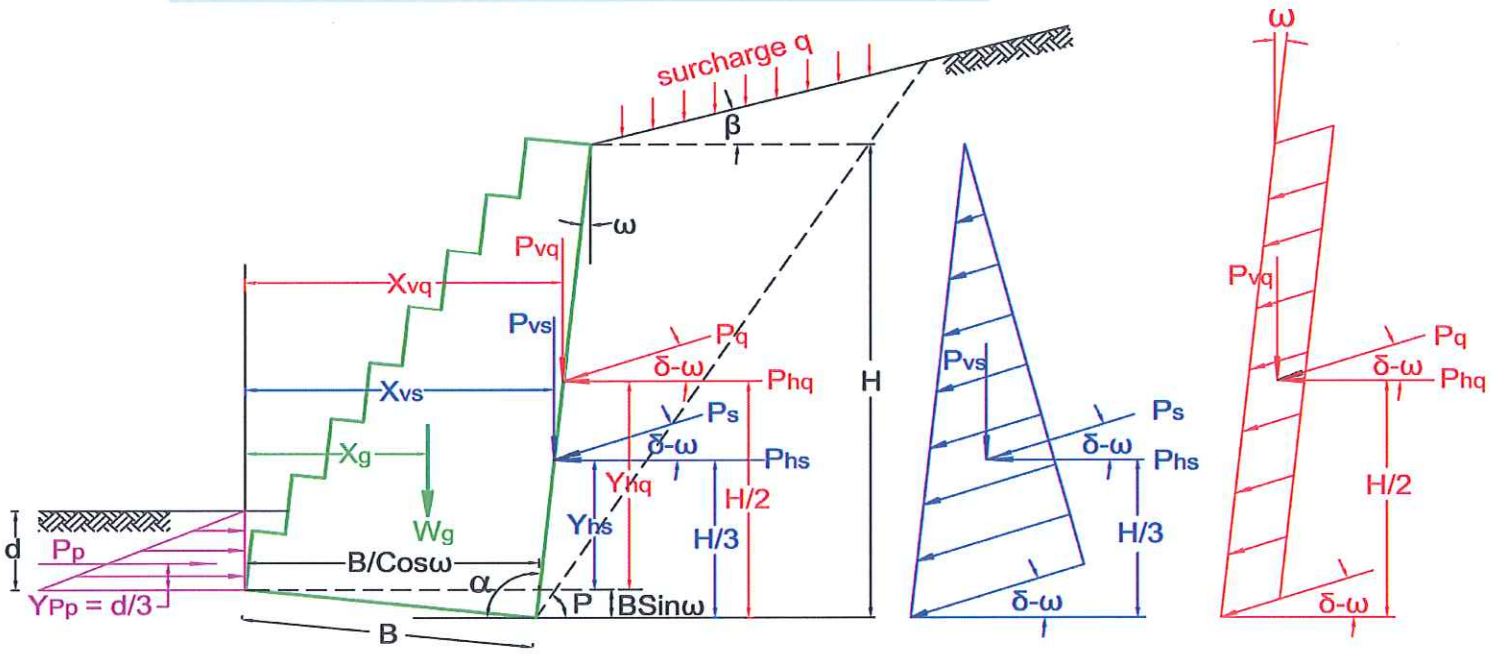
SECTION A–A

LKQ PROPERTY BASIN

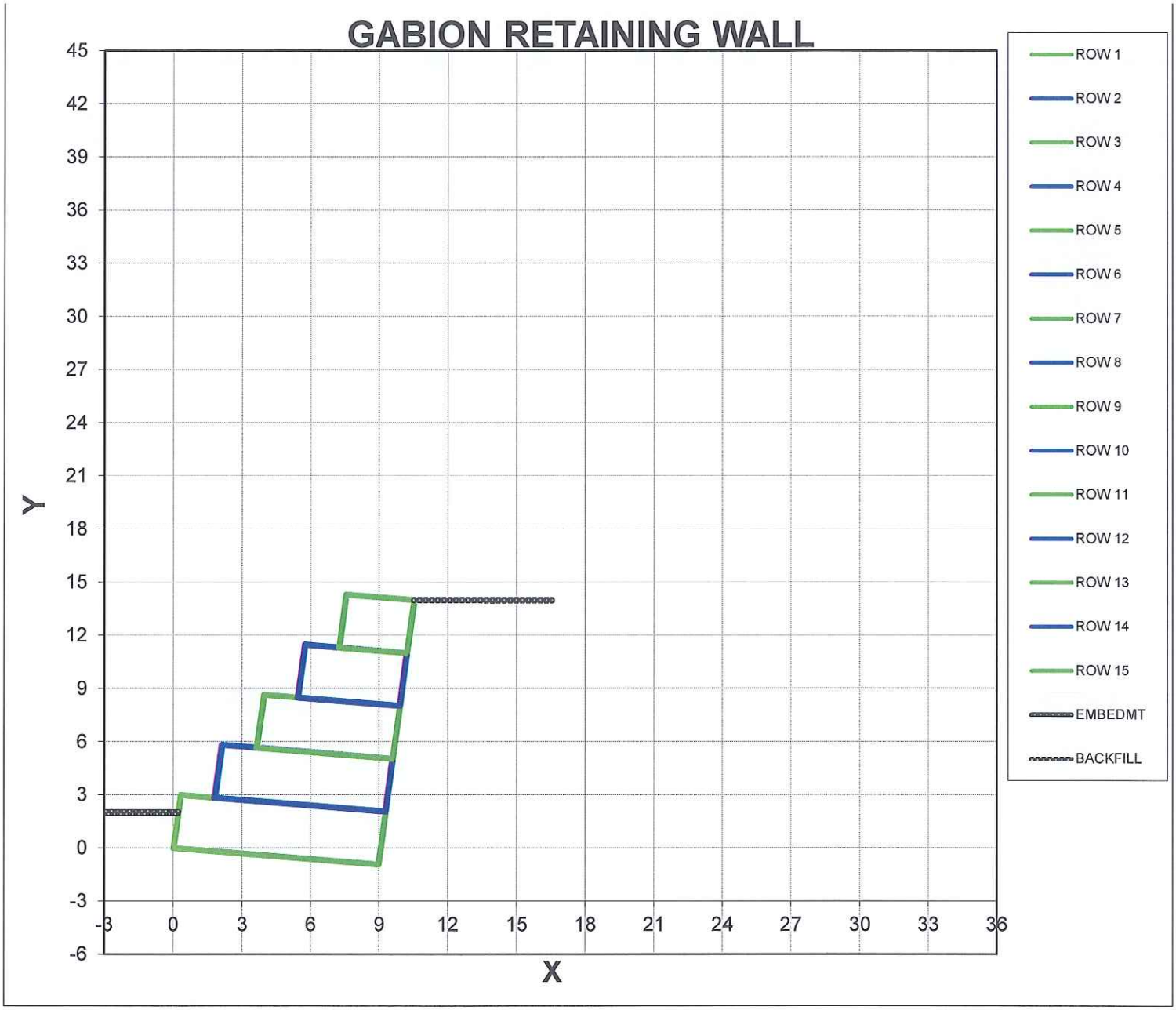
Terra Aqua Gabions Gravity Retaining Wall Calculations

TERRA AQUA GABIONS GRAVITY RETAINING WALL CALCULATIONS (STRAIGHT LINE BACK)

PROJECT NAME:	RT 4 Crosstown Freeway Extension	**PROJECT #:**	111904.G01
LOCATION:	San Joaquin County, California	**SECTION:**	
GEOTECHNICAL ENGINEER:	Carl Henderson	**REPORT #:**	
NOTES:	Proposed Gabion Wall at LKQ Basin	**DRAWING #:**	
		DATE:	10-May-12



Descriptions	symbols	Input Values	Units	Notes
Backfill slope angle above wall	β	0.000	°	< Φ
Angle of internal friction	Φ	34.000	°	
Wall friction reduction by geotextile	fr	20.000	%	Back of wall
Angle of wall friction	δ	27.200	°	Φ(100-fr)/100
Inclination angle to vertical plane	ω	6.000	°	for wall with straight back (no offsets)
Back of wall angle to horizontal	α	96.000	°	90+ω
Cohesion	c	0	psf	Ignore cohesion
Surcharge	q	240.000	psf	
Soil density	γs	120.000	pcf	
Rock density	γr	140.000	pcf	
Void in gabion	v	30.000	%	
Gabion density	γg	98.000	pcf	γr(100-v)/100
Actual height of wall	H	14.918	ft	(hCosω) Corrected for inclination
Embedment	d	2.000	ft	0 ft to ignore passive thrust
Width of base	B	9	ft	
Allowable soil bearing capacity	qa	2000.000	psf	determined by Geotechnical Engineer



Row #	Width (ft)	Height (ft)	offset (from toe) (ft)	Area (ft²)	X (ft)	Moment (ft³)	Y (ft)	Moment (ft³)
15				0.000	0.000	0.000	0.000	0.000
14				0.000	0.000	0.000	0.000	0.000
13				0.000	0.000	0.000	0.000	0.000
12				0.000	0.000	0.000	0.000	0.000
11				0.000	0.000	0.000	0.000	0.000
10				0.000	0.000	0.000	0.000	0.000
9				0.000	0.000	0.000	0.000	0.000
8				0.000	0.000	0.000	0.000	0.000
7				0.000	0.000	0.000	0.000	0.000
6				0.000	0.000	0.000	0.000	0.000
5	3.0	3.0	6.00	9.000	7.500	67.500	13.500	121.500
4	4.5	3.0	4.50	13.500	6.750	91.125	10.500	141.750
3	6.0	3.0	3.00	18.000	6.000	108.000	7.500	135.000
2	7.5	3.0	1.50	22.500	5.250	118.125	4.500	101.250
1	9.0	3.0	0.00	27.000	4.500	121.500	1.500	40.500
	h=	15.0		90.000	5.625	506.250	6.000	540.000

1 **COULOMB'S THEORY**

BACK

Active earth pressure coefficient

$$K_a = \frac{\sin^2(\alpha+\Phi)}{\sin^2\alpha \cdot \sin(\alpha-\delta)\left[1+\sqrt{\frac{\sin(\Phi+\delta)\sin(\Phi-\beta)}{\sin(\alpha-\delta)\sin(\alpha+\beta)}}\right]^2}$$

$$= \frac{0.587}{0.989 \cdot 0.932\left[1+\sqrt{\frac{0.876 \cdot 0.559}{0.932 \cdot 0.995}}\right]^2}$$

$$= \frac{0.587}{2.750}$$

$$= 0.213$$

Active soil thrust $P_s = 0.5K_a\gamma_s H^2$

$= 2849.074$ lb/ft

Active surcharge thrust $P_q = \frac{\sin\alpha}{\sin(\alpha+\beta)} K_{aq} H$

$= \frac{0.995}{0.995} 763.938$

$= 763.938$ lb/ft

Horizontal active soil thrust $P_{hs} = P_s\cos(\delta-\omega)$

$= 2656.260$ lb/ft

Horizontal active surcharge thrust $P_{hq} = P_q\cos(\delta-\omega)$

$= 712.238$ lb/ft

Vertical active soil thrust $P_{vs} = P_s\sin(\delta-\omega)$

$= 1030.295$ lb/ft

Vertical active surcharge thrust $P_{vq} = P_q\sin(\delta-\omega)$

$= 276.259$ lb/ft

FRONT

Inclination angle to vertical $\omega_p = 0.000$

Front face angle to horizontal $\alpha_p = 90-\omega_p$

$= 90.000$

Backfill slope $\beta_p = 0.000$

Angle of wall friction $\delta_p = 0.000$

Passive earth pressure coefficient

$$K_p = \frac{\sin^2(\alpha-\Phi)}{\sin^2\alpha \cdot \sin(\alpha+\delta)\left[1-\sqrt{\frac{\sin(\Phi+\delta)\sin(\Phi+\beta)}{\sin(\alpha+\delta)\sin(\alpha+\beta)}}\right]^2}$$

$$= \frac{0.687}{1.000 \cdot 1.000\left[1-\sqrt{0.559 \cdot 0.559}\right]^2}$$

| | √ | 1.000 | 1.000 |

$$\frac{0.687}{0.194}$$

3.537

Passive soil thrust $P_p = 0.5K_p\gamma_s \; d^2$

$= 848.912$ lb/ft

2 Check Overturning:

Vertical distance to P_{hs} $Y_{hs} = H/3-B\sin\omega$

$= 4.032$ ft

Vertical distance to P_{hq} $Y_{hq} = H/2-B\sin\omega$

$= 6.518$ ft

Overturning moment $\sum M_o = Y_{hs}P_{hs} + Y_{hq}P_{hq}$

$= 15352.128$ ft-lb / ft

Weight of Gabion $W_g = \sum A\,\gamma_g$

$= 8820.000$ lb / ft

Horizontal distance to W_g $X_g = Y\sin\omega + X\cos\omega$

$= 6.221$ ft

Horizontal distance to P_{vs} $X_{vs} = B/\cos\omega + (H/3-B\sin\omega)\tan\omega$

9.473

Horizontal distance to P_{vq} $X_{vq} = B/\cos\omega + (H/2-B\sin\omega)\tan\omega$

9.735

Vertical distance to P_p $Y_{Pp} = d/3$

$= 0.667$

Resisting moment $\sum M_r = W_g X_g + P_{vs} X_{vs} + P_{vq} X_{vq} + P_p Y_{Pp}$

$= 67887.928$ ft-lb / ft

Overturning factor of safety $SF_o = \frac{\sum M_r}{\sum M_o}$

4.422 ≥ 2.000 **O.K**

3 Check Sliding

Total Normal forces $\sum W = W_g\cos\omega + P_s\sin\delta + P_q\sin\delta - P_p\sin\omega$

		=	10334.448	lb/ft				
Frictional force	Ff	=	∑W	TanΦ				
		=	10334.448	0.675				
		=	6970.673					
Total Resisting Forces	∑Fr	=	Ff + CosωPp					
		=	7814.935					
Total Driving Forces at base	∑Fd	=	PsCosδ +	PqCosδ -	WgSinω			
			2291.531					
Siding factor of safety	SFs	=	∑Fr / ∑Fd					
			3.410	≥	1.500	**O.K**		

4 Check the Eccentricity of Resultant Force
(Resultant is in middle one third)

Eccentricity	e	=	0.5 B	-	(∑Mr / ∑W	-	Mo)	
		=	4.500	-	5.084			
		=	-B/6	≤	e	≤	+B/6	
			-1.500	≤ **O.K**	-0.584	≤ **O.K**	1.500	

5 Check Bearing

Applied bearing pressure	P	=	∑W / B	(1±	6e/B)			
		=	1148.272	(1±	0.389	)		
	Right	=	1594.997	psf	≤	2000.000	**O.K**	
	Left	=	701.547	psf	≤	2000.000	**O.K**	

Additional calculations may have to be performed including and not limiting to global stability analysis (by (Geotechnical Engineer), seismic forces (by Seismic Engineer) and hydraulic forces (by Hydraulic Engineer). Please check local, State and Federal requirements. The calculator assumes drained uniformed retained and foundation soil properties. The calculations should be reviewed, checked and certified by a Professional Engineer. To the best of our knowledge, the calculator and information was prepared accurately. Terra Aqua is not responsible for the reliability and validity of the geotechnical parameters assumed in the calculations. The calculator is intended to provided design design assistance to the engineer for the purpose of designing with Terra Aqua products.

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APPENDIX K
EXCERPTS FROM 1999 BOGGS TRACT DRAINAGE STUDY

LIST OF ATTACHMENTS

The following are attached and complete this appendix.

Boggs Tract Site Map
Boring Logs

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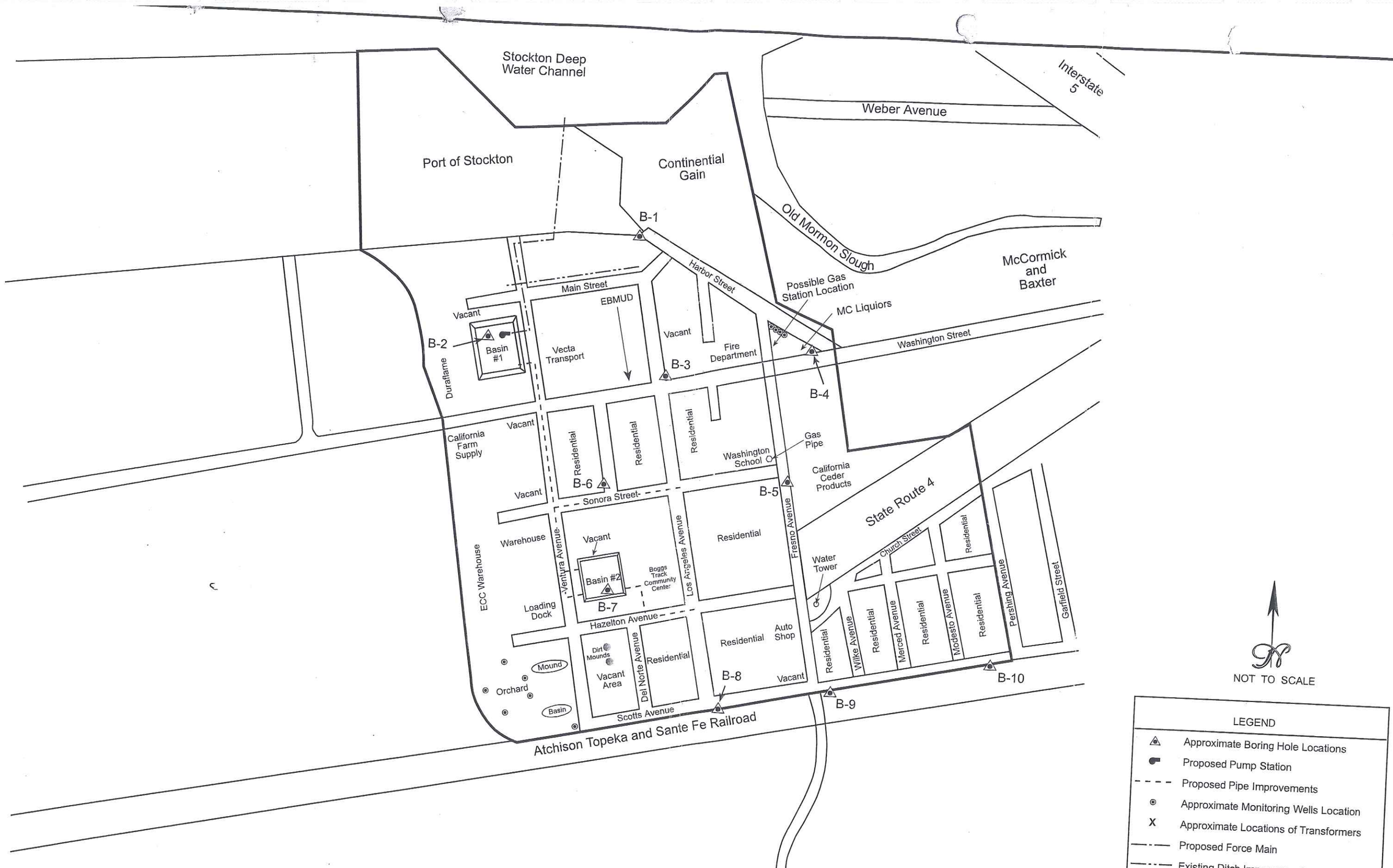
Boggs Tract Site Map



NOT TO SCALE

LEGEND

- Approximate Boring Hole Locations
- Proposed Pump Station
- Proposed Pipe Improvements
- Approximate Monitoring Wells Location
- X Approximate Locations of Transformers
- Proposed Force Main
- Existing Ditch Improvements

KLEINFELDER

DATE PRODUCED: 10/1/99	DRAFTED BY: J YOUNG
PROJ. NO: 20-2631-08.G01	FILE NO: 2019d091

SITE MAP
BOGGS TRACT
STOCKTON, CALIFORNIA

PLATE
A-1a

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Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			USCS SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS (MORE THAN HALF OF MATERIAL IS LARGER THAN THE #200 SIEVE)	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN THE #4 SIEVE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
			GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
		GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVELS, GRAVEL-SILT-SAND MIXTURES
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN THE #4 SIEVE	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
			SP	POORLY-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
		SANDS WITH OVER 12% FINES	SM	SILTY SANDS, SAND-GRAVEL-SILT MIXTURES
			SC	CLAYEY SANDS, SAND-GRAVEL-CLAY MIXTURES
FINE GRAINED SOILS (MORE THAN HALF OF MATERIAL IS SMALLER THAN THE #200 SIEVE)	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS & VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS, CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILT
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS & ORGANIC SILTS OF MEDIUM-TO-HIGH PLASTICITY
HIGHLY ORGANIC SOILS			PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

KLEINFELDER

Drafted By: JMY	Project No.: 20-2631-08.G01
Date: 9/8/99	File Number: 20-2631-08.G01

SOIL CLASSIFICATION SYSTEM
BOGGS TRACT
DRAINAGE STUDY
STOCKTON, CALIFORNIA

PLATE

A-2

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LOG SYMBOLS

	BULK / BAG SAMPLE	-4	PERCENT FINER THAN THE NO. 4 SIEVE (ASTM Test Method C 136)
	MODIFIED CALIFORNIA SAMPLER (2-1/2 inch outside diameter)	-200	PERCENT FINER THAN THE NO. 200 SIEVE (ASTM Test Method C 117)
	CALIFORNIA SAMPLER (3 inch outside diameter)	LL	LIQUID LIMIT (ASTM Test Method D 4318)
	STANDARD PENETRATION SPLIT SPOON SAMPLER (2 inch outside diameter)	PI	PLASTICITY INDEX (ASTM Test Method D 4318)
	SHELBY TUBE (3 inch outside diameter)	EI	EXPANSION INDEX (UBC Standard 29-2)
	GRAB SAMPLE	COL	COLLAPSE POTENTIAL
	WATER LEVEL (level after completion)	UC	UNCONFINED COMPRESSION
	WATER LEVEL (level where first encountered)	MC	MOISTURE CONTENT

GENERAL NOTES

1. Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual.
2. No warranty is provided as to the continuity of soil conditions between individual sample locations.
3. Logs represent general soil conditions observed at the point of exploration on the date indicated.
4. In general, Unified Soil Classification designations presented on the logs were evaluated by visual methods only. Therefore, actual designations (based on laboratory tests) may vary.

KLEINFELDER		LOG KEY BOGGS TRACT DRAINAGE STUDY STOCKTON, CALIFORNIA	PLATE A-3
Drafted By: JMY Date: 9/8/99	Project No.: 20-2631-08.G01 File Number: 20-2631-08.G01		

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Date Completed: 9/8/99

Logged By: JMY

Total Depth: 20 feet

Surface Conditions: Asphaltic concrete on gravel lot next to road.

Groundwater: Groundwater encountered at 8-1/2 foot depth. Caving to 17-1/2 foot depth.

Depth (ft)	FIELD: Sample	FIELD: Blows/ft	LABORATORY: Dry Density (pcf)	LABORATORY: Moisture Content (%)	LABORATORY: Compress. Strength (ksf)	LABORATORY: Other Tests	Pen (ksf)	DESCRIPTION / USCS Classification
								ASPHALTIC CONCRETE AND GRAVEL - 1-1/2 feet
								(SC) CLAYEY SAND - Light Brown, Clay Balls, Fine Grained, Moist
								(CL) SILTY CLAY - Gray-Brown, Moist
5		9						**(CL) SILTY CLAY** - Brown, Stiff, Moist
								(CL) CLAY - Brown, Moist
						Groundwater Encountered		
10		9						**(CL) SILTY CLAY** - Light Gray-Brown, Stiff, Moist
15		8						**(CL) SILTY CLAY** - Gray-Orange-Brown, Very Silty, Moist
								(ML) SANDY SILT - Blue-Gray, Very Fine Grained, Stiff, Moist
								(SM) SILTY SAND - Blue-Gray, Very Fine to Fine-Grained, Moist
								(SW) SAND - Blue-Gray, Fine Grained, Moist
20								**(SW) SAND** - Blue-Gray, Fine to Medium-Grained, Moist
								END OF BORING
25								

KLEINFELDER

PROJECT NO. 20-2631-08.G01

BOGGS TRACT
DRAINAGE STUDY
STOCKTON, CALIFORNIA
LOG OF BORING B-1

PLATE 1 of 1

A-4

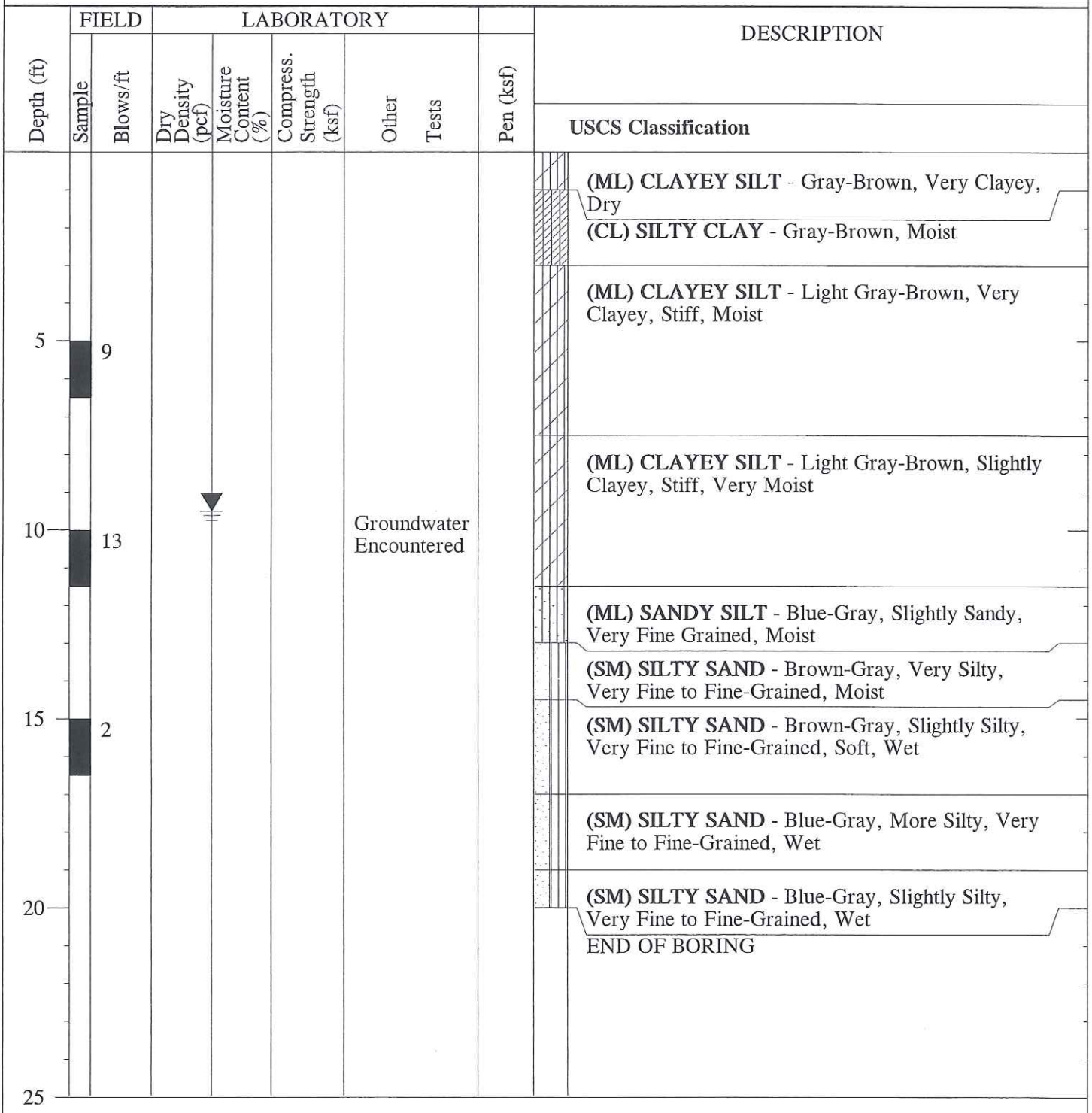
Date Completed: 9/8/99

Logged By: JMY

Total Depth: 20 feet

Surface Conditions: In field 80 feet west of Ventura Avenue.

Groundwater: Groundwater encountered to 9-1/2 foot depth. Caving to 15 foot depth.



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PROJECT NO. 20-2631-08.G01

BOGGS TRACT
DRAINAGE STUDY
STOCKTON, CALIFORNIA
LOG OF BORING B-2

PLATE
1 of 1

A-5

Date Completed: 9/8/99

Logged By: JMY

Total Depth: 20 feet

Surface Conditions: Next to road.

Groundwater: Groundwater encountered at 10 feet 10 inch depth. Caving to 18-1/2 foot depth.

Depth (ft)	FIELD: Sample	FIELD: Blows/ft	LABORATORY: Dry Density (pcf)	LABORATORY: Moisture Content (%)	LABORATORY: Compress. Strength (ksf)	LABORATORY: Other Tests	Pen (ksf)	DESCRIPTION / USCS Classification
								(ML) SANDY SILT - Brown, Very Fine Grained, Moist
								(ML) GRAVELLY SANDY SILT - Brown, Very Fine Grained, Moist
								(SM) SILTY SAND - Light Brown, Very Fine to Fine-Grained, Moist
								(ML) SANDY SILT - Brown, Very Fine Grained, Moist
5								**(CL) CLAY** - Light Brown-Gray, Stiff, Moist
		10						**(ML) CLAYEY SILT** - Light Brown-Gray, Moist
								(CL) SILTY CLAY - Gray-Brown, Very Silty, Moist
								(ML) CLAYEY SILT - Light Gray-Brown, Moist
10		13				Groundwater Encountered		**(ML) SANDY SILT** - Light Gray-Brown, Very Fine Grained, Stiff, Moist
								(ML) SILT - Light Brown, Moist
15								**(ML) SILT** - Light Brown-Gray, Moist
								(SM) SILTY SAND - Blue-Gray, Very Silty, Very Fine to Fine-Grained, Moist
20								**(SM) SILTY SAND** - Blue-Gray, Less Silty, Very Fine to Fine-Grained, Moist
								END OF BORING
25								

KLEINFELDER

PROJECT NO. 20-2631-08.G01

BOGGS TRACT
DRAINAGE STUDY
STOCKTON, CALIFORNIA
LOG OF BORING B-3

PLATE 1 of 1

A-6

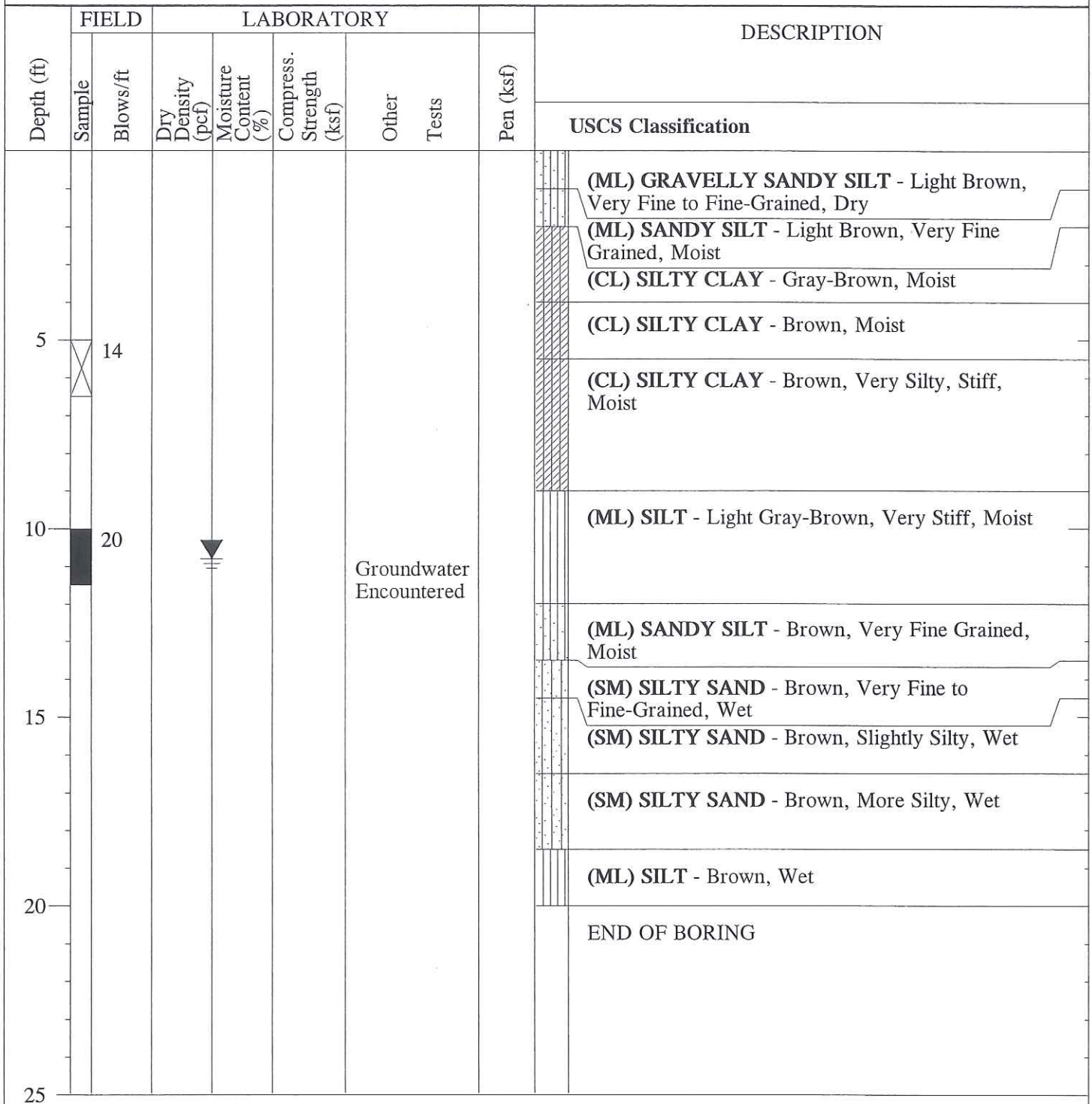
Date Completed: 9/8/99

Logged By: JMY

Total Depth: 20 feet

Surface Conditions: Dirt lot.

Groundwater: Groundwater encountered at 10 feet 8 inch depth.



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PROJECT NO. 20-2631-08.G01

BOGGS TRACT
DRAINAGE STUDY
STOCKTON, CALIFORNIA
LOG OF BORING B-4

PLATE 1 of 1

A-7

Date Completed: 9/9/99

Logged By: JMY

Total Depth: 15 feet

Surface Conditions: Dirt area along road.

Groundwater: Not encountered.

Depth (ft)	FIELD: Sample	FIELD: Blows/ft	LABORATORY: Dry Density (pcf)	LABORATORY: Moisture Content (%)	LABORATORY: Compress. Strength (ksf)	LABORATORY: Other Tests	Pen (ksf)	DESCRIPTION USCS Classification
0								**(ML) GRAVELLY SANDY SILT** - Brown, Dry
								(SM) SILTY SAND - Gray-Brown, Very Silty, Mix, Very Fine Grained, Moist
								(ML) SANDY SILT - Gray-Brown, Very Silty, Mix, Very Fine Grained, Soft, Very Moist
5		15						**(CL) CLAY** - Gray-Brown, Moist
								(CL) SILTY CLAY - Gray-Brown, Stiff, Moist
10		26						**(CL) SILTY CLAY** - Gray-Brown, Very Silty, Very stiff, Moist
								(ML) SILT - Gray-Brown, Moist
15								**(ML) CLAYEY SILT** - Gray-Brown, Moist
								END OF BORING
20								
25								

KLEINFELDER

PROJECT NO. 20-2631-08.G01

BOGGS TRACT
DRAINAGE STUDY
STOCKTON, CALIFORNIA
LOG OF BORING B-5

PLATE 1 of 1

A-8

Date Completed: 9/8/99

Logged By: JMY

Total Depth: 15 feet

Surface Conditions: Dry weed area next to road.

Groundwater: Groundwater encountered at 8 feet 8 inch depth.

Depth (ft)	FIELD: Sample	FIELD: Blows/ft	LABORATORY: Dry Density (pcf)	LABORATORY: Moisture Content (%)	LABORATORY: Compress. Strength (ksf)	LABORATORY: Other Tests	Pen (ksf)	DESCRIPTION / USCS Classification
								(CL) SANDY SILTY CLAY - Gray-Brown, Very Fine Grained, Dry
								(CL) CLAY - Gray-Brown, Slightly Moist
								(ML) CLAYEY SILT - Light Brown, Very Clayey, Slightly Moist
5		30						
								(ML) SANDY SILT -Light Gray-Brown, Very Fine Grained, Very Stiff, Moist
								(ML) SILT - Light Gray-Brown, Moist
						Groundwater Encountered		**(ML) CLAYEY SILT** - Light Gray-Brown, Moist
10		37						**(ML) SILT** - Light Gray-Brown, Hard, Moist
								(ML) CLAYEY SILT - Light Gray-Brown, Moist
15								**(CL) CLAY** - Light Brown, Moist
								END OF BORING
20								
25								

KLEINFELDER

PROJECT NO. 20-2631-08.G01

BOGGS TRACT
DRAINAGE STUDY
STOCKTON, CALIFORNIA
LOG OF BORING B-6

PLATE 1 of 1

A-9

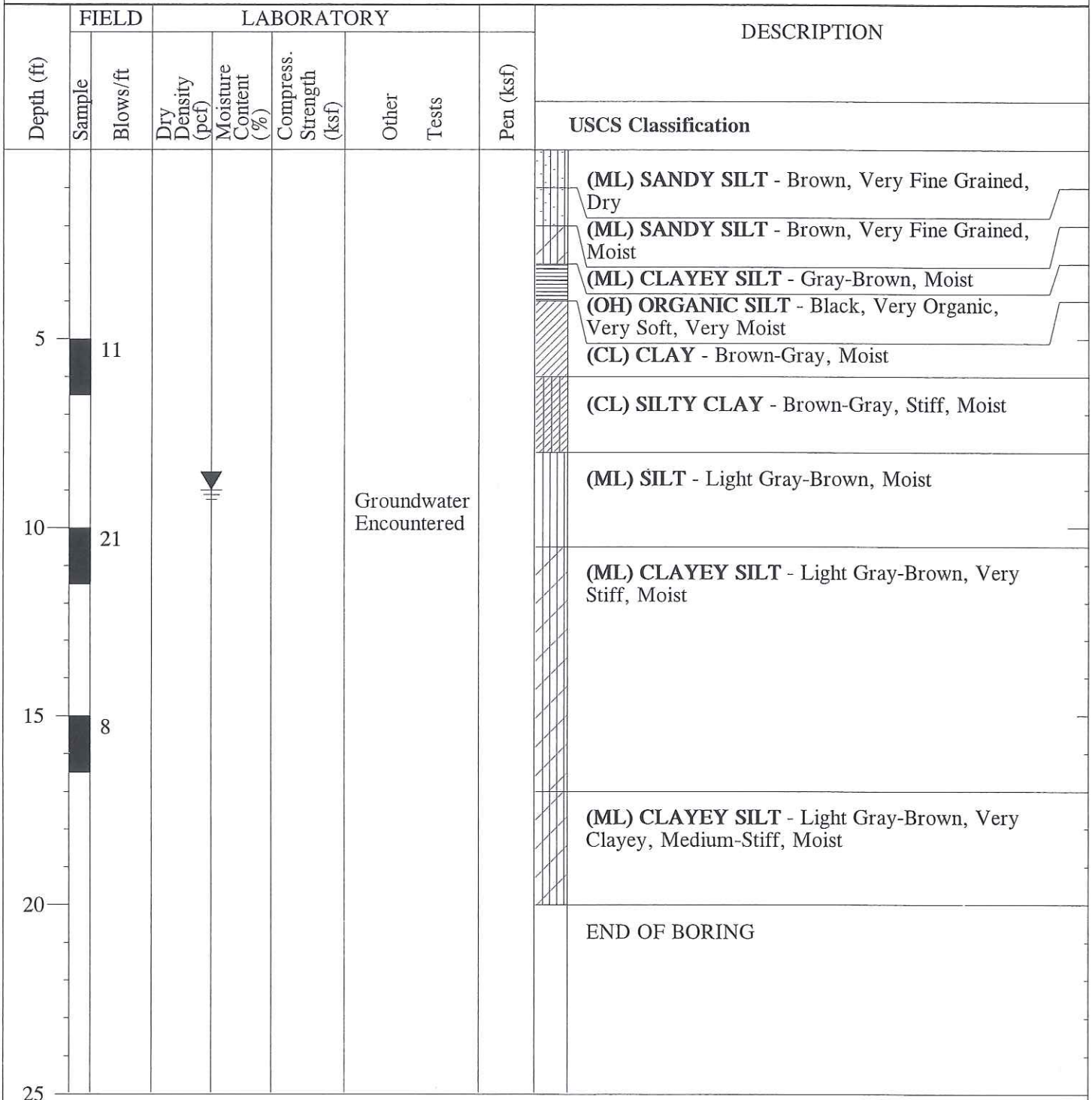
Date Completed: 9/9/99

Logged By: JMY

Total Depth: 20 feet

Surface Conditions: Dirt lot 100 feet east of Ventura Avenue.

Groundwater: Groundwater encountered at 9 foot depth.



KLEINFELDER

PROJECT NO. 20-2631-08.G01

BOGGS TRACT
DRAINAGE STUDY
STOCKTON, CALIFORNIA
LOG OF BORING B-7

PLATE 1 of 1

A-10

Date Completed: 9/9/99

Logged By: JMY

Total Depth: 20 feet

Surface Conditions: Dry weeds next to road.

Groundwater: Groundwater encountered at 11 feet 3 inch depth. Caving to 13 feet 3 inch depth.

Depth (ft)	FIELD: Sample	FIELD: Blows/ft	LABORATORY: Dry Density (pcf)	LABORATORY: Moisture Content (%)	LABORATORY: Compress. Strength (ksf)	LABORATORY: Other Tests	Pen (ksf)	DESCRIPTION / USCS Classification
								(CL) SILTY CLAY - Gray-Brown, Dry
								(ML) SANDY SILT - Brown, Very Fine Grained, Slightly-Moist
								(ML) SANDY SILT - Brown, with Gravel and Broken Glass, Very Fine Grained, Slightly Moist
5		21						**(CL) SILTY CLAY** - Light Gray-Brown, Very Stiff, Moist
								(ML) CLAYEY SANDY SILT - Light Brown, Slightly Clayey, Very Fine Grained, Soft, Very Moist
10		5						**(ML) SANDY SILT** - Light Gray-Brown, Very Fine Grained, Very Moist
						Groundwater Encountered		**(SM) SILTY SAND** - Light Gray-Brown, Very Fine to Fine-Grained, Medium-Stiff, Very Moist
								(SM) SILTY SAND - Light Gray-Brown, Less Silty, Fine Grained, Very Moist
15								**(SP) SAND** - Brown, Fine to Medium-Grained, Very Moist
								(CL) SILTY CLAY - Brown-Gray, Very Moist
								(ML) CLAYEY SILT - Brown-Gray, Very Moist
20								END OF BORING
25								

KLEINFELDER

PROJECT NO. 20-2631-08.G01

BOGGS TRACT
DRAINAGE STUDY
STOCKTON, CALIFORNIA
LOG OF BORING B-8

PLATE 1 of 1

A-11

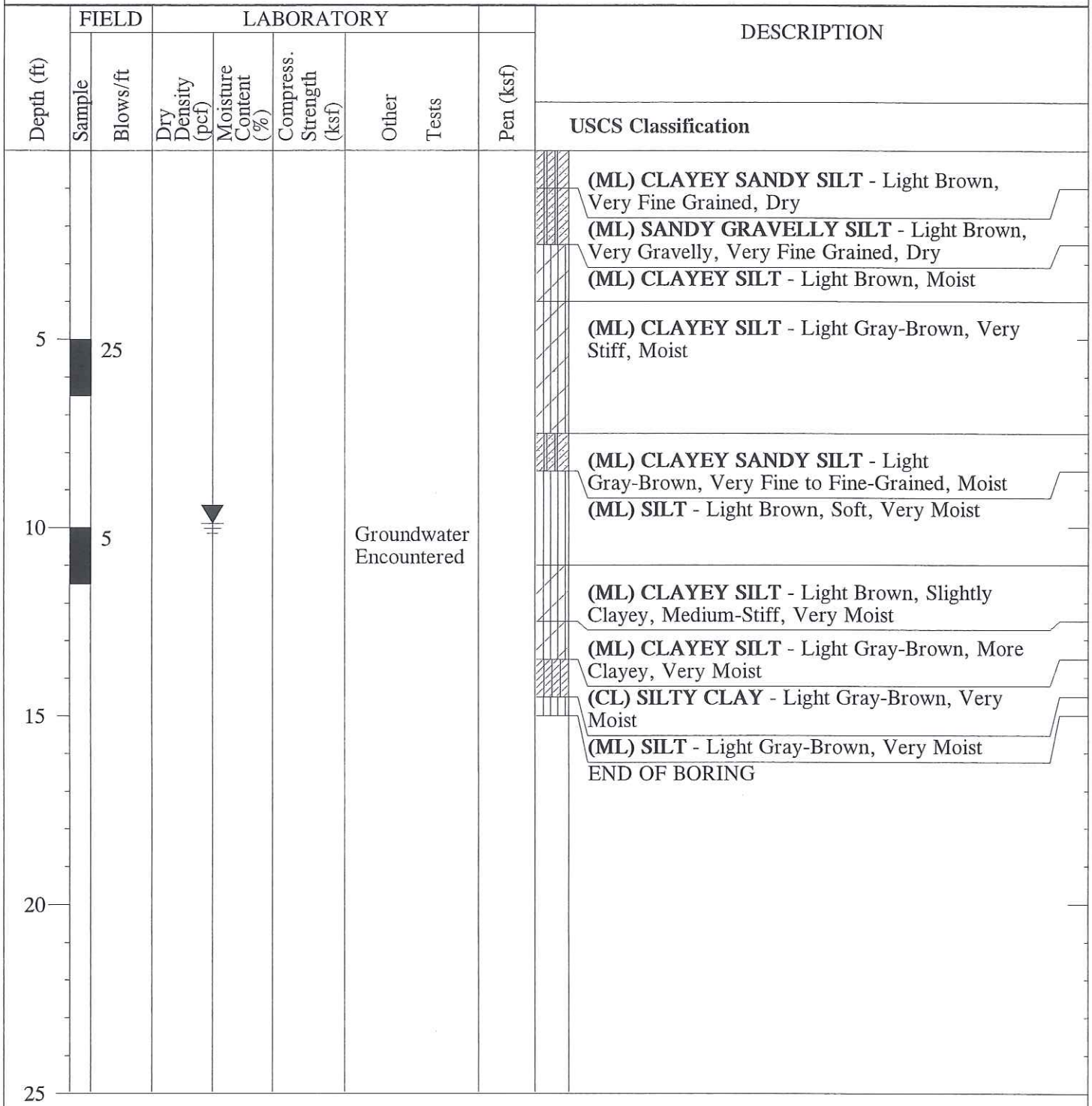
Date Completed: 9/9/99

Logged By: JMY

Total Depth: 15 feet

Surface Conditions: Dry weeds next to road.

Groundwater: Groundwater encountered at 9 feet 9 inch depth.



KLEINFELDER

PROJECT NO. 20-2631-08.G01

BOGGS TRACT
DRAINAGE STUDY
STOCKTON, CALIFORNIA
LOG OF BORING B-9

PLATE 1 of 1

A-12

Date Completed: 9/9/99

Logged By: JMY

Total Depth: 20 feet

Surface Conditions: Dry weeds next to road.

Groundwater: Groundwater encountered at 13 foot depth.

Depth (ft)	FIELD: Sample	FIELD: Blows/ft	LABORATORY: Dry Density (pcf)	LABORATORY: Moisture Content (%)	LABORATORY: Compress. Strength (ksf)	LABORATORY: Other Tests	Pen (ksf)	DESCRIPTION / USCS Classification
								(CL) SANDY SILTY CLAY - Light Gray-Brown, Dry
								(CL) CLAY - Brown, Moist
5		20						**(CL) SILTY CLAY** - Light Gray-Brown, Very Stiff, Moist
								(ML) CLAYEY SILT - Light Gray-Brown, Slightly Clayey, Moist
10		22						**(ML) SANDY SILT** - Light Gray-Brown, Very Fine Grained, Very Moist
								(SM) SILTY SAND - Light Gray-Brown, Very Fine Grained, Very Stiff, Very Moist
						Groundwater Encountered		**(ML) CLAYEY SANDY SILT** - Light Gray-Brown, Very Fine Grained, Very Moist
								(ML) SILT - Light Brown, Very Moist
15		9						**(ML) CLAYEY SILT** - Light Gray-Brown, Slightly Clayey, Stiff, Very Moist
								(ML) CLAYEY SILT - Light Brown-Gray, Very Clayey, Very Moist
20								**(ML) SANDY CLAYEY SILT** - Light Brown-Gray, Very Fine Grained, Very Moist
								END OF BORING
25								

KLEINFELDER

PROJECT NO. 20-2631-08.G01

BOGGS TRACT
DRAINAGE STUDY
STOCKTON, CALIFORNIA
LOG OF BORING B-10

PLATE
1 of 1

A-13

APPENDIX L
PAVEMENT ANALYSIS

LIST OF ATTACHMENTS

The following are attached and complete this appendix.

R-Value Sample Locations
Pavement Core Locations and Results
Pavement Design

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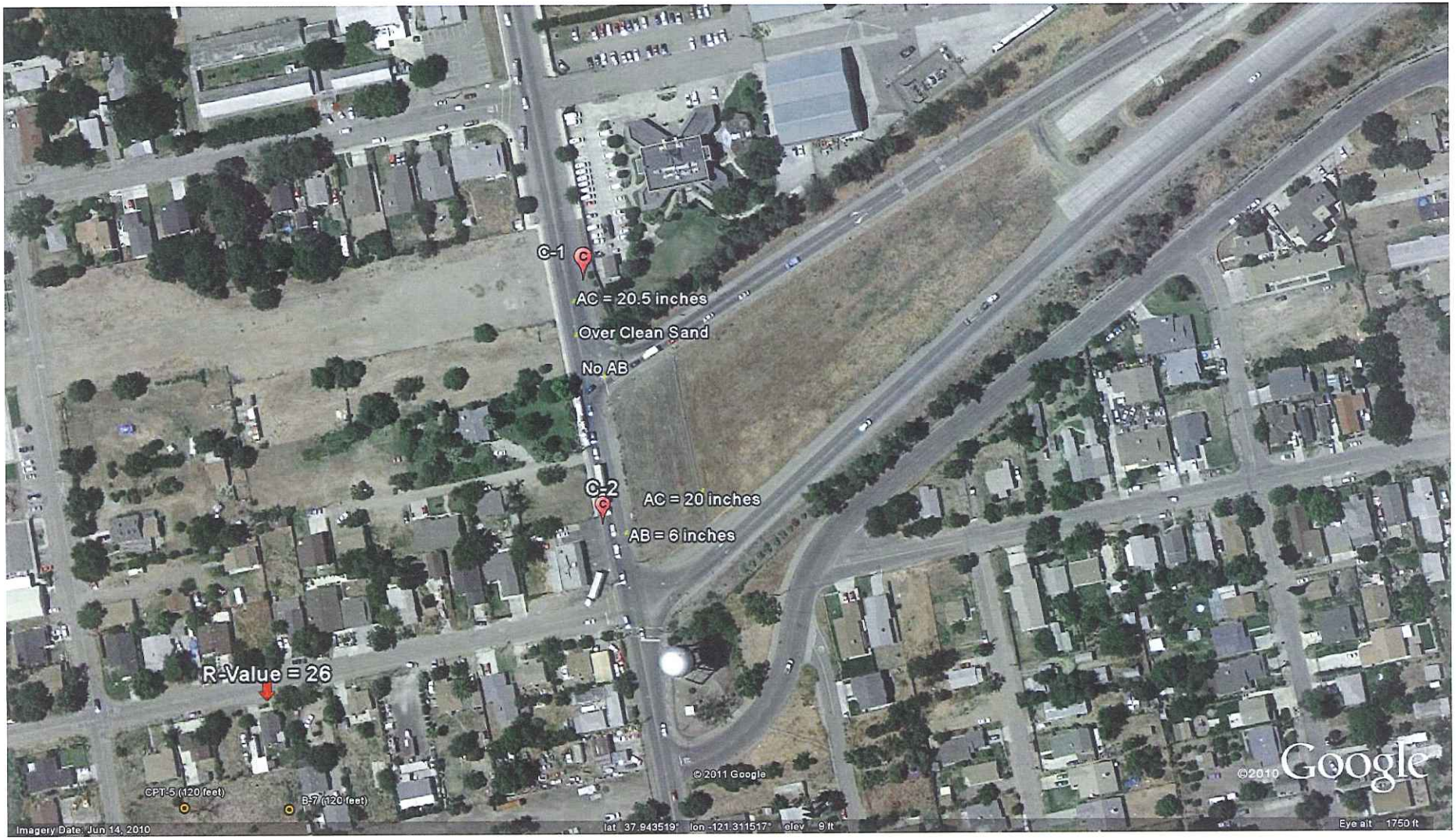
R-Value Sample Locations

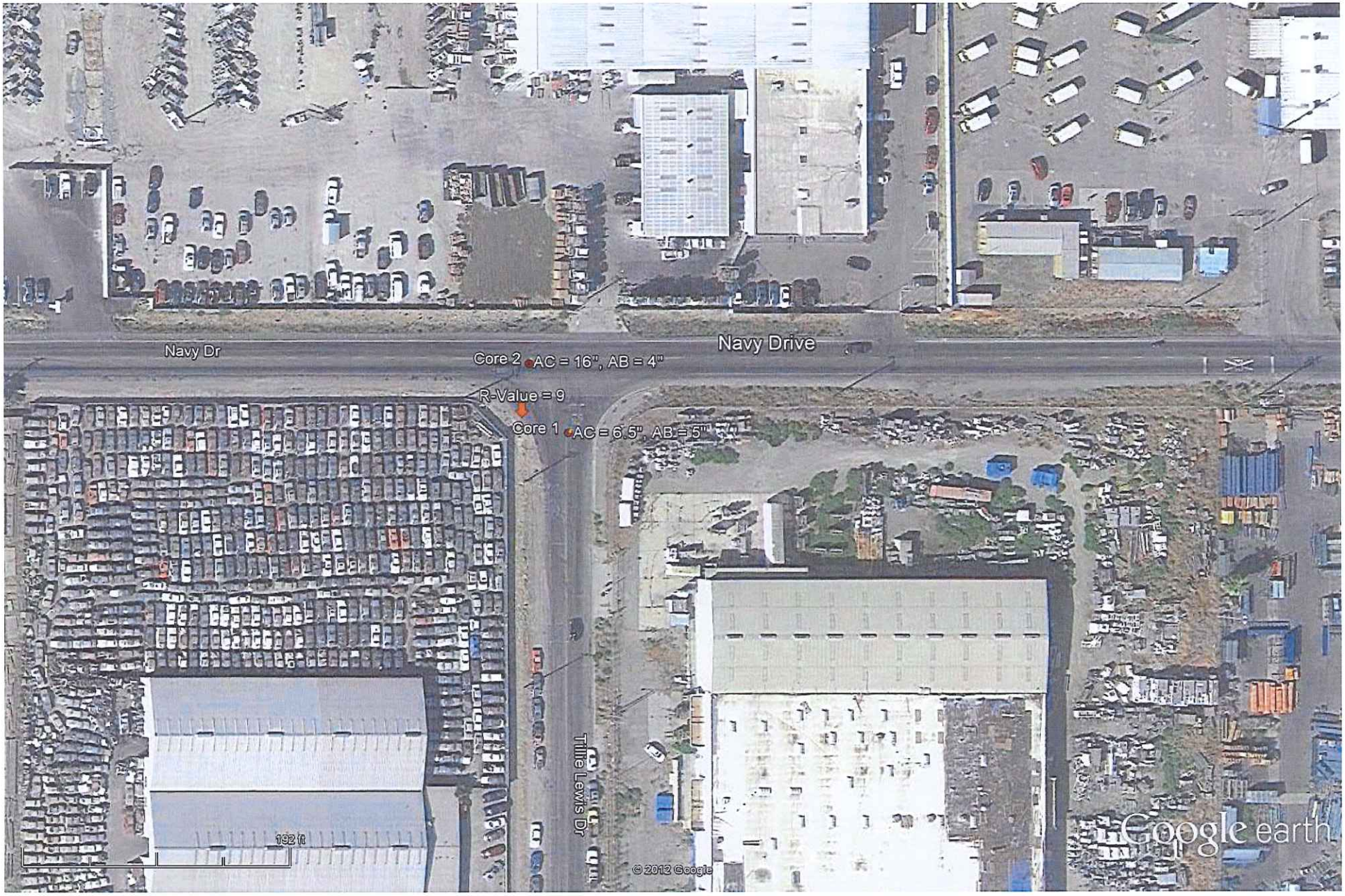
Google earth

feet 3000

meters 900

Pavement Core Locations and Results


C-1
AC = 20.5 inches
Over Clean Sand
No AB
C-2
AC = 20 inches
AB = 6 inches
R-Value = 26
CPT-5 (120 feet)
B-7 (120 feet)
© 2011 Google
©2010 Google
Imagery Date: Jun 14, 2010
lat 37.943519° lon -121.311517° elev 9 ft
Eye alt 1750 ft



Google earth

feet 600

meters 100

Pavement Design

RT 4 Crosstown Freeway

Performed By: Carl H. Date: 3/29/2012
Checked By : Ron H. Date: 3/29/2012

Temporary Full Depth AC Construction

Assumptions:

* Class 2 AB R-value = 78
* Subgrade R-value = 5
* GE = 0.0032(TI)(100-R)
* HMA estimated per Table 633.1 of Caltrans Highway Design Manual
* 20 Year Design Life

TI (20 Yrs Life)	R	GE	GE + 0.1	HMA (ft)	HMA (in)	HMA (in)
7	5	2.128	2.228	0.9	10.8	***11***
7.5	5	2.28	2.38	1	12	***12***
8	5	2.432	2.532	1.05	12.6	***13***
8.5	5	2.584	2.684	1.15	13.8	***14***
9	5	2.736	2.836	1.2	14.4	**14.5**
9.5	5	2.888	2.988	1.25	15	**15**
10	5	3.04	3.14	1.3	15.6	**16**
10.5	5	3.192	3.292	1.4	16.8	**17**
11	5	3.344	3.444	1.45	17.4	***17.5***
11.5	5	3.496	3.596	1.55	18.6	***19***
12	5	3.648	3.748	1.6	19.2	***19.5***
12.5	5	3.8	3.9	1.7	20.4	***20.5***
13	5	3.952	4.052	1.75	21	***21***
13.5	5	4.104	4.204	1.85	22.2	***22.5***
14	5	4.256	4.356	1.9	22.8	***23***
14.5	5	4.408	4.508	1.95	23.4	***23.5***
15	5	4.56	4.66	2	24	***24***

Per Section 613.3 of the Caltrans Highway Design Manual

* TI = 9.0 x [(ESAL X LDF) / 10^6]^0.119
* Assume ESAL Constant = 13,780 per Table 613.3A (5 or more-axle trucks, 20 year-design life)
* Assume AADT = ESAL/ESAL Constant (5 or more-axle trucks, 20-year design life)
* Assuming LDF = 1, ESAL is summarized as follows from Table 613.3C

TI	ESAL	Calculated AADTT	Total Trucks Over 20 years	Total Trucks Over 2 years	Total Trucks Over 3 years
	487,000	35	257,990	25,799	38,698
8.5					
	798,000	58	422,743	42,274	63,411
9					
	1,270,000	92	672,787	67,279	100,918
9.5					
	1,980,000	144	1,048,911	104,891	157,337
10					
	3,020,000	219	1,599,855	159,985	239,978
10.5					
	4,500,000	327	2,383,890	238,389	357,583
11					
	6,600,000	479	3,496,372	349,637	524,456
11.5					
	9,490,000	689	5,027,358	502,736	754,104
12					
	13,500,000	980	7,151,669	715,167	1,072,750
12.5					
	18,900,000	1372	10,012,337	1,001,234	1,501,851
13					
	26,100,000	1894	13,826,560	1,382,656	2,073,984
13.5					
	35,600,000	2583	18,859,216	1,885,922	2,828,882
14					
	48,100,000	3491	25,481,132	2,548,113	3,822,170
14.5					
	64,300,000	4666	34,063,135	3,406,313	5,109,470
15					
	84,700,000	6147	44,870,102	4,487,010	6,730,515
15.5					
	112,000,000	8128	59,332,366	5,933,237	8,899,855
16					
	144,000,000	10450	76,284,470	7,628,447	11,442,671
16.5					
	186,000,000	13498	98,534,107	9,853,411	14,780,116

Conclusion

The following full depth pavement thickness can be assumed for temporary pavements for TI's (20 year design life) between 11 and 16.5

Temporary Pavement For Two Years

TI (20 Yr Life)	Total Trucks Over 2 Years	TI (2 YR Life)	Temporary HMA (in)
11	349,637	8.5	***14***
11.5	502,736	9	***14.5***
12	715,167	9.5	***15***
12.5	1,001,234	9.5	***15.5***
13	1,382,656	10	***16***
13.5	1,885,922	10.5	***17***
14	2,548,113	11	***17.5***
14.5	3,406,313	11	***18***
15	4,487,010	11.5	***19***
15.5	5,933,237	12	***19.5***
16	7,628,447	12.5	***20.5***
16.5	9,853,411	12.5	***21***

Temporary Pavement For Three Years

TI (20 Yr Life)	Total Trucks Over 3 Years	TI (3 YR Life)	Temporary HMA (in)
11	524,456	9	***14.5***
11.5	754,104	9.5	***15***
12	1,072,750	10	***16***
12.5	1,501,851	10	***16.5***
13	2,073,984	10.5	***17***
13.5	2,828,882	11	***17.5***
14	3,822,170	11.5	***19***
14.5	5,109,470	12	***19.5***
15	6,730,515	12	***20***
15.5	8,899,855	12.5	***20.5***
16	11,442,671	13	***21***
16.5	14,780,116	13.5	***22.5***

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Sheet 1 of 2

PROJECT SR-4 PROJECT NO. 111904-001

SUBJECT Pavement BY S. Parra DATE 9/6/2012

REVIEWED BY C. Henderson DATE

Reference Highway Design Manual - Caltrans 620-14 2008

Route 4, TI = 16.5, ramps

constructed over at least 3' embankment with materials with a minimum R-value of 10

CRCP - without lateral support

CRCP = 1.10 ft over
HMA = 0.5 ft over
Class 2 AS = 0.7 ft

JRCP - without lateral support

JPCP = 1.25 ft over
HMA-A = 0.5 ft over
Class 2 AS = 0.7 ft

or

JPCP = 1.25 ft over
LCB = 0.5 ft over
Class 2 AS = 0.7 ft

Navy Drive, TI = 16, over material with a minimum R-value of 10

JPCP - without Lateral support

JPCP = 1.20 ft over
HMA-A = 0.5 ft over
Class 2 AS = 0.7 ft

or

JPCP = 1.2 ft over
LCB = 0.5 ft over
Class 2 AS = 0.7 ft

for subgrade soils with R-value less than 10, a geotextile fabric (TX-5 or TX-7) should be placed over the ground surface below the pavement

PROJECT SR-4 PROJECT NO.

SUBJECT Pavement BY S. Pandya DATE

REVIEWED BY DATE

Navy Drive (Tillie Lewis Intersection)

TI = 14, JPCP - without Lateral support

JPCP = 1.05 ft over
HMA-A = 0.5 ft over
Class 2 AS = 0.7 ft

or

JPCP = 1.05 ft over
LCB = 0.5 ft over
Class 2 AS = 0.7 ft

If the pavement will be constructed over subgrade soils with R-values less than 10, a geogrid (TX-5 or TX-7) should be place over the ground surface

Charter Way (TI = 15) $R\text{-Value}_{AB} = 78$

R-Value = 5, ⇒

$GE_{HMA} = 0.0032 \times (TI)(100 - R\text{-Value}_{AB}) = 0.0032 \times 15 \times (100-78) + 0.2$ (S.F)

$GE_{HMA} = 1.26$ ft ⇒ (from table 633.1) $t_{AC} = 0.75$ ft $\cong 9''$

(1.23 is closest to 1.26)

$GE_{HMA+AB} = 0.0032(TI)(100-5) = 4.56$ ft

$GE_{AB} = 4.56 - 1.23 = 3.33$ ft

$\Rightarrow t_{AB} = \frac{3.33}{1.1} = 3.03' \times 12 = 36.5''$

Tillie Lewis - TI = 13.5, HMA R-value = 5

$GE_{AC} = 0.0032 \times 13.5 \times (100-78) + 0.2 = 1.15$ ft

⇒ from table 633.1, Closest value = 1.16 = $t_{AC} = 0.7' = 8.5''$

$GE_{AB+AC} = 0.0032 \times (TI) \times (100-5) = 4.10$ ft

$GE_{AB} = 4.10 - 1.16 = 2.94$ ft ⇒ $t_{AB} = \frac{2.94}{1.1} = 2.68$ ft $= 32.0''$

Table 633.1
Gravel Equivalents of Structural Layers (ft)

	HMA[1], [2]											Base and Subbase					
	Traffic Index (TI)																
	5.0 & below	5.5 6.0	6.5 7.0	7.5 8.0	8.5 9.0	9.5 10.0	10.5 11.0	11.5 12.0	12.5 13.0	13.5 14.0	14.5 15.0	HMAB; LCB	CTPB; CTB (Cl. A)	ATPB	CTB (Cl. B)	AB	AS
Actual HMA Thickness	G_f (varies with TI and HMA thickness greater than 0.5 ft)											G_f (constant)					
(ft)	2.54	2.32	2.14	2.01	1.89	1.79	1.71	1.64	1.57	1.52	1.46	1.9	1.7	1.4	1.2	1.1	1.0
0.10	0.25	0.23	0.21	0.20	0.19	0.18	0.17	0.16	0.16	0.15	0.15	--	--	--	--	--	--
0.15	0.38	0.35	0.32	0.30	0.28	0.27	0.26	0.25	0.24	0.23	0.22	--	--	--	--	--	--
0.20	0.51	0.46	0.43	0.40	0.38	0.36	0.34	0.33	0.31	0.30	0.29	--	--	--	--	--	--
0.25	0.63	0.58	0.54	0.50	0.47	0.45	0.43	0.41	0.39	0.38	0.37	--	--	0.35	--	--	--
0.30	0.76	0.69	0.64	0.60	0.57	0.54	0.51	0.49	0.47	0.45	0.44	--	--	0.42	--	--	--
0.35	0.89	0.81	0.75	0.70	0.66	0.63	0.60	0.57	0.55	0.53	0.51	0.67	0.60	0.49	0.42	0.39	0.35
0.40	1.01	0.93	0.86	0.80	0.76	0.72	0.68	0.65	0.63	0.61	0.59	0.76	0.68	0.56	0.48	0.44	0.40
0.45	1.14	1.04	0.96	0.90	0.85	0.81	0.77	0.74	0.71	0.68	0.66	0.86	0.77	0.63	0.54	0.50	0.45
0.50	1.27	1.16	1.07	1.00	0.94	0.90	0.85	0.82	0.79	0.76	0.73	0.95	0.85	0.70	0.60	0.55	0.50
0.55	1.41	1.29	1.19	1.12	1.05	1.00	0.95	0.91	0.87	0.84	0.81	1.05	0.94	0.77	0.66	0.61	0.55
0.60	1.58	1.45	1.34	1.25	1.18	1.12	1.07	1.02	0.98	0.95	0.91	1.14	1.02	0.84	0.72	0.66	0.60
0.65	1.76	1.61	1.49	1.39	1.31	1.25	1.19	1.14	1.09	1.05	1.02	1.24	1.11	0.91	0.78	0.72	0.65
0.70	--	1.78	1.64	1.54	1.45	1.38	1.31	1.26	1.21	1.16	1.12	1.33	1.19	--	0.84	0.77	0.70
0.75	--	1.95	1.80	1.69	1.59	1.51	1.44	1.38	1.32	1.27	1.23	1.43	1.28	--	0.90	0.83	0.75
0.80	--	2.12	1.96	1.84	1.73	1.64	1.57	1.50	1.44	1.39	1.34	1.52	1.36	--	0.96	0.88	0.80
0.85	--	--	2.13	1.99	1.88	1.78	1.70	1.63	1.56	1.51	1.46	1.62	1.45	--	1.02	0.94	0.85
0.90	--	--	2.30	2.15	2.03	1.92	1.83	1.76	1.69	1.63	1.57	1.71	1.53	--	1.08	0.99	0.90
0.95	--	--	--	2.31	2.18	2.07	1.97	1.89	1.81	1.75	1.69	1.81	1.62	--	1.14	1.05	0.95
1.00	--	--	--	2.47	2.33	2.21	2.11	2.02	1.94	1.87	1.81	1.90	1.70	--	1.20	1.10	1.00
1.05	--	--	--	2.64	2.49	2.36	2.25	2.16	2.07	2.00	1.93	2.00	1.79	--	1.26	1.16	1.05
1.10	--	--	--	--	2.65	2.51	2.40	2.29	2.20	2.12	2.05	--	--	--	--	--	--
1.15	--	--	--	--	2.81	2.67	2.54	2.43	2.34	2.25	2.18	--	--	--	--	--	--
1.20	--	--	--	--	2.98	2.82	2.69	2.58	2.48	2.39	2.30	--	--	--	--	--	--
1.25	--	--	--	--	--	2.98	2.84	2.72	2.61	2.52	2.43	--	--	--	--	--	--
1.30	--	--	--	--	--	3.14	2.99	2.87	2.75	2.65	2.56	--	--	--	--	--	--
1.35	--	--	--	--	--	3.30	3.15	3.01	2.90	2.79	2.70	--	--	--	--	--	--
1.40	--	--	--	--	--	--	3.31	3.16	3.04	2.93	2.83	--	--	--	--	--	--
1.45	--	--	--	--	--	--	3.46	3.32	3.19	3.07	2.97	--	--	--	--	--	--
1.50	--	--	--	--	--	--	3.62	3.47	3.33	3.21	3.10	--	--	--	--	--	--
1.55	--	--	--	--	--	--	--	3.62	3.48	3.36	3.24	--	--	--	--	--	--
1.60	--	--	--	--	--	--	--	3.78	3.63	3.50	3.38	--	--	--	--	--	--
1.65	--	--	--	--	--	--	--	3.94	3.79	3.65	3.52	--	--	--	--	--	--
1.70	--	--	--	--	--	--	--	--	3.94	3.80	3.67	--	--	--	--	--	--
1.75	--	--	--	--	--	--	--	--	4.09	3.95	3.81	--	--	--	--	--	--
1.80	--	--	--	--	--	--	--	--	4.25	4.10	3.96	--	--	--	--	--	--
1.85	--	--	--	--	--	--	--	--	--	4.25	4.10	--	--	--	--	--	--
1.90	--	--	--	--	--	--	--	--	--	4.40	4.25	--	--	--	--	--	--
1.95	--	--	--	--	--	--	--	--	--	4.56	4.40	--	--	--	--	--	--
2.00	--	--	--	--	--	--	--	--	--	--	4.55	--	--	--	--	--	--

Notes:

(1) Open Graded Friction Course (conventional and rubberized) is a non-structural wearing course and provides no structural value.

(2) Top portion of HMA surface layer (maximum 0.20 ft.) may be replaced with equivalent RAC-G thickness. See Topic 631.3 for additional details.

Table 623.1G
Rigid Pavement Catalog (Inland Valley, Type II Subgrade Soil) (1), (2), (3), (4)

	Rigid Pavement Structural Depth							
TI	With Lateral Support (ft)				Without Lateral Support (ft)			
< 9	0.70 JPCP 0.35 LCB 0.50 AS	0.70 JPCP 0.35 HMA-A 0.50 AS	0.75 JPCP 1.00 AB	0.70 JPCP 0.35 ATPB 0.80 AB	0.75 JPCP 0.35 LCB 0.50 AS	0.75 JPCP 0.35 HMA-A 0.50 AS	0.80 JPCP 1.00 AB	0.75 JPCP 0.35 ATPB 0.80 AB
9.5 to 10	0.70 JPCP 0.35 LCB 0.50 AS	0.70 JPCP 0.35 HMA-A 0.50 AS	0.80 JPCP 1.00 AB	0.75 JPCP 0.35 ATPB 0.80 AB	0.80 JPCP 0.35 LCB 0.50 AS	0.85 JPCP 0.35 HMA-A 0.50 AS	0.90 JPCP 1.00 AB	0.85 JPCP 0.35 ATPB 0.80 AB
10.5 to 11	0.75 JPCP 0.40 LCB 0.60 AS	0.75 JPCP 0.40 HMA-A 0.60 AS	0.85 JPCP 1.30 AB		0.85 JPCP 0.40 LCB 0.60 AS	0.90 JPCP 0.40 HMA-A 0.60 AS	0.95 JPCP 1.30 AB	
11.5 to 12	0.85 JPCP 0.40 LCB 0.60 AS	0.85 JPCP 0.40 HMA-A 0.60 AS	0.80 CRCP 0.40 HMA-A 0.60 AS		0.95 JPCP 0.40 LCB 0.60 AS	0.95 JPCP 0.40 HMA-A 0.60 AS	0.85 CRCP 0.40 HMA-A 0.60 AS	
12.5 to 13	0.85 JPCP 0.50 LCB 0.70 AS	0.90 JPCP 0.50 HMA-A 0.70 AS	0.80 CRCP 0.50 HMA-A 0.70 AS		1.00 JPCP 0.50 LCB 0.70 AS	1.00 JPCP 0.50 HMA-A 0.70 AS	0.90 CRCP 0.50 HMA-A 0.70 AS	
13.5 to 14	0.95 JPCP 0.50 LCB 0.70 AS	0.95 JPCP 0.50 HMA-A 0.70 AS	0.85 CRCP 0.50 HMA-A 0.70 AS		1.05 JPCP 0.50 LCB 0.70 AS	1.05 JPCP 0.50 HMA-A 0.70 AS	0.95 CRCP 0.50 HMA-A 0.70 AS	
14.5 to 15	1.00 JPCP 0.50 LCB 0.70 AS	1.00 JPCP 0.50 HMA-A 0.70 AS	0.90 CRCP 0.50 HMA-A 0.70 AS		1.15 JPCP 0.50 LCB 0.70 AS	1.15 JPCP 0.50 HMA-A 0.70 AS	1.00 CRCP 0.50 HMA-A 0.70 AS	
15.5 to 16	1.05 JPCP 0.50 LCB 0.70 AS	1.05 JPCP 0.50 HMA-A 0.70 AS	0.95 CRCP 0.50 HMA-A 0.70 AS		1.20 JPCP 0.50 LCB 0.70 AS	1.20 JPCP 0.50 HMA-A 0.70 AS	1.05 CRCP 0.50 HMA-A 0.70 AS	
16.5 to 17	1.10 JPCP 0.50 LCB 0.70 AS	1.10 JPCP 0.50 HMA-A 0.70 AS	0.95 CRCP 0.50 HMA-A 0.70 AS		1.25 JPCP 0.50 LCB 0.70 AS	1.25 JPCP 0.50 HMA-A 0.70 AS	1.10 CRCP 0.50 HMA-A 0.70 AS	
> 17	1.15 JPCP 0.50 LCB 0.70 AS	1.15 JPCP 0.50 HMA-A 0.70 AS	1.00 CRCP 0.50 HMA-A 0.70 AS		1.30 JPCP 0.50 LCB 0.70 AS	1.30 JPCP 0.50 HMA-A 0.70 AS	1.10 CRCP 0.50 HMA-A 0.70 AS	

Notes:
(1) Thicknesses shown for JPCP are for doweled pavement only. The thickness shown in these tables are not valid for nondoweled JPCP.
(2) Includes 0.03 ft sacrificial wearing course for future grinding of JPCP/CRCP.
(3) Portland cement concrete may be substituted for LCB when justified for constructibility or traffic handling. If Portland cement concrete is used in lieu of LCB, it must be placed in a separate lift than JPCP and must not be bonded to the JPCP.
(4) If ATPB is needed for TIs > 10.0 to perpetuate an existing treated permeable layer, place the ATPB between the surface layer (JPCP or CRCP) and the base layer. No deduction is made to the thickness of the base and subbase layers on account of the ATPB.

Legend:
JPCP = Jointed Plain Concrete Pavement
CRCP = Continuously Reinforced Concrete Pavement
LCB = Lean Concrete Base
HMA-A = Hot Mix Asphalt (Type A)
ATPB = Asphalt Treated Permeable Base
AB = Class 2 Aggregate Base
AS = Class 2 Aggregate Subbase
TI = Traffic Index

Crosstown Freeway SR-4, Stockton, CA

TI = 14 & R = 5

Tensar. THE COMPANY YOU CAN BUILD ON™

Preliminary

Date: 5/9/11

UNREINFORCED DESIGN	
Traffic Index	14
Subgrade R-value	5

Current Section (AC/AB)

Pavement Layer	R-value	Gravel Factor	Required GE (ft)	Actual GE (ft)	Required Thickness (ft)	Required Thickness (inch)	Unreinfoced Pavement Profile
Asphalt Concrete	N/A	1.67	4.26	4.25	0.71	8.5	8.5 inch AC
Class 2 AB	78	1.1			2.79	33.5	33.5 inch AB
Total:					3.50	42.0	Subgrade

TENSAR TX7 Alternative (1)	
TriAx Geogrid Reinforced T.I.	14
TriAx Geogrid R-value[1]	35

Pavement Layer	R-value	Gravel Factor	Required GE (ft)	Actual GE (ft)	Required Thickness (ft)	Required Thickness (inch)	Geogrid Pavement Profile
Asphalt Concrete	N/A	1.67	2.91	3.01	0.71	8.5	8.5 inch AC
Class 2 AB	78	1.1			1.67	20.0	20.0 inch AB
Total:					2.38	28.5	Subgrade

Notes:

1. Reinforced TriAx geogrid (TX7) Subgrade layer: Increase R value by 30.

TENSAR TX7 Alternative (2)	
TriAx Geogrid Reinforced T.I.	11.5
TriAx Geogrid R-value[1]	35

2-Layer System

Pavement Layer	R-value	Gravel Factor	Required GE (ft)	Actual GE (ft)	Required Thickness (ft)	Required Thickness (inch)	Geogrid Pavement Profile
Asphalt Concrete	N/A	1.84	2.39	2.40	0.71	8.5	8.5 inch AC
Class 2 AB	78	1.1			0.50	6.0	6.0 inch AB
Class 2 AB	78	1.1			0.50	6.0	6.0 inch AB
Total:					1.71	20.5	Subgrade

Notes:

1. Reinforced TriAx geogrid (TX7) Subgrade layer: Increase R value by 30.
2. Top layer of TriAx geogrid increases the stiffness of the section (Such that a conventional TI = 14 is equivalent to a TriAx TI = 11.5)

APPENDIX M
ALTPIPE ANALYSIS

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AltPipe

Project EA: **RT 4 Crosstown Freeway Extension**
Project Engineer: **Carl Henderson**
Location: **Stockton, CA**
Description:

Steel Pipes

DSN/U		12-Inch Diameter APC	18-Inch Diameter APC	24-Inch Diameter APC
Pipe Diameter (in)		12	18	24
Pipe Type	**Coat**	**Minimum Thickness (in)**		
Corrugated Steel Pipe - Helical Corrugations - 2⅔" X ½" Corrugations	PA	0.052	0.052	0.052
	PS	0.052	0.052	0.052
Corrugated Steel Pipe - Annular Corrugations - 2⅔" X ½" Corrugations	PA		0.064	0.064
	PS		0.064	0.064
Steel Spiral Rib Pipe - ¾" X 1" Ribs At 11½" Pitch	BCI			0.109
	PA			0.064
	PS			0.064
Steel Spiral Rib Pipe - ¾" X 1" Ribs At 8½" Pitch	BCI			0.109
	PA			0.064
	PS			0.064
Steel Spiral Rib Pipe - ¾" X ¾" Ribs At 7½" Pitch	BCI			0.109
	PA			0.064
	PS			0.064

Plastic Pipes

DSN/U	12-Inch Diameter APC	18-Inch Diameter APC	24-Inch Diameter APC
Pipe Diameter (in)	12	18	24
Pipe Type	**Availability**		
PVC Corrugated	Allowable	Allowable	Allowable
PVC Ribbed		Allowable	Allowable
HDPE Corrugated - Type S		Allowable	Allowable
HDPE Ribbed		Allowable	Allowable
HDPE Corrugated - Type C		Allowable	Allowable

Reinforced Concrete Pipes

DSN/U	12-Inch Diameter APC	18-Inch Diameter APC	24-Inch Diameter APC
Pipe Diameter (in)	**12**	**18**	**24**
Steel Cover (in)	0.75	0.75	1
Sacks of Cement	5	5	5
Percentage Water	10.0	10.0	10.0

Other Information

DSN/U	12-Inch Diameter APC	18-Inch Diameter APC	24-Inch Diameter APC
Soil pH	7.4	7.4	7.4
Minimum Soil Resistivity (ohm-cm)	470	470	470
Sulfate Concentration (ppm)	460	460	460
Chloride Concentration (ppm)	340	340	340
Abrasion Level	1	1	1
2–5 Year Flow Velocity (ft/sec)	5	5	5
Design Service Life (years)	50	50	50
Height of Cover (ft)	5	5	5

AltPipe Version 6.08 © State of California

Dist	COUNTY	LOCATION CODE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS

Theresa Gabriel — REGISTERED CIVIL ENGINEER — 12-20-07 DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER — Theresa A. Gabriel — No. E15129 — Exp. 6-30-10 — ELECT — STATE OF CALIFORNIA

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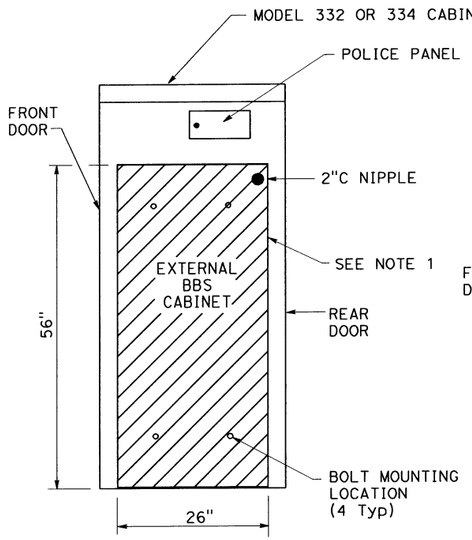
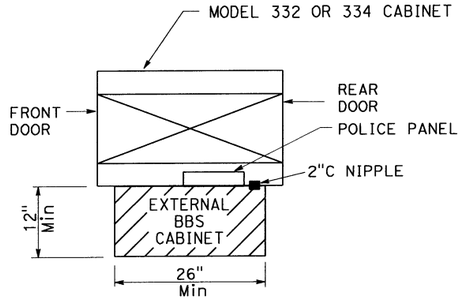


SIDE VIEW

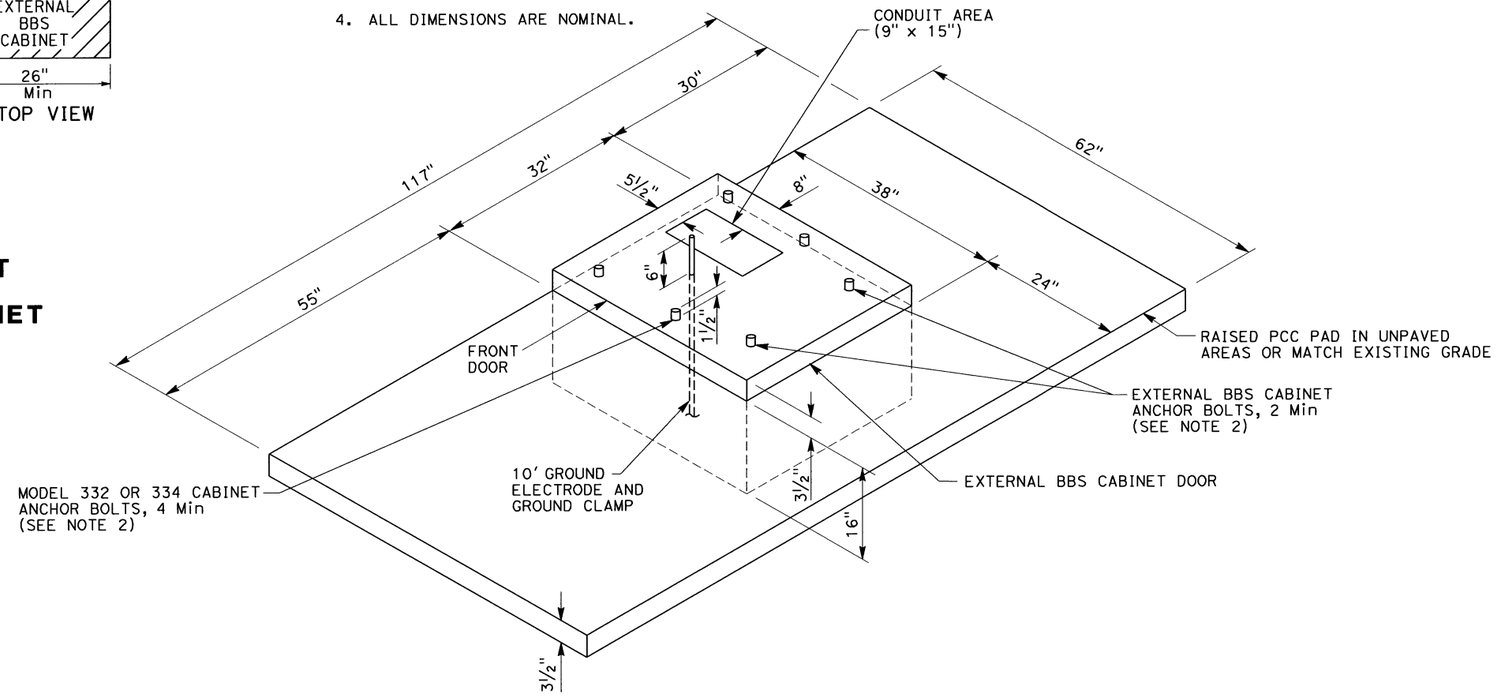


TOP VIEW

EXTERNAL BBS CABINET MOUNTED TO THE MODEL 332 OR 334 CABINET

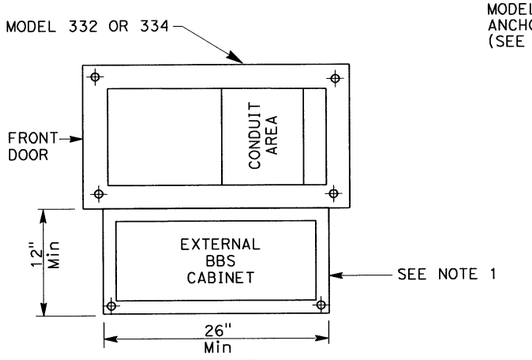
NOTE: (THIS SHEET ONLY)

1. THE EXTERNAL BBS CABINET SHALL BE MOUNTED TO THE MODEL 332 OR 334 CABINET WITH FOUR 18-8 STAINLESS STEEL HEX HEAD, FULLY-THREADED, ⅜"-16 X 1" BOLTS; TWO WASHERS PER BOLT, DESIGNED FOR ⅜" BOLTS AND ARE 18-8 STAINLESS STEEL, 1" OUTSIDE DIAMETER, ROUND, AND FLAT; AND ONE K-LOCK NUT PER BOLT THAT IS 18-8 STAINLESS STEEL AND A HEX-NUT. THE ENGINEER WILL HAVE TO APPROVE THE BOLT MOUNTING LOCATION PRIOR TO INSTALLATION.
2. THE ANCHOR BOLTS SHALL BE ¾" Dia X 15" WITH A 2"-90° BEND. THE CABINET MANUFACTURER'S SPECIFICATION SHALL DETERMINE THE LOCATION OF THE ANCHOR BOLTS IN THE FOUNDATION. THE ENGINEER WILL HAVE TO APPROVE THE ANCHOR BOLTS AND ITS LOCATION IN THE FOUNDATION PRIOR TO CONSTRUCTION.
3. THE CONTRACTOR SHALL VERIFY THE DIMENSIONS OF THE BBS CABINET PRIOR TO CONSTRUCTING THE FOUNDATION OF THE MODIFIED PORTION OF THE Std MODEL 332 AND 334 CABINET FOUNDATION. THE ENGINEER WILL HAVE TO APPROVE ANY NECESSARY DEVIATIONS PRIOR TO CONSTRUCTION.
4. ALL DIMENSIONS ARE NOMINAL.



MODIFIED MODEL 332 AND 334 CABINET FOUNDATION DETAIL FOR BATTERY BACKUP SYSTEM (BBS)

(FOR DIMENSIONS AND DETAILS NOT SHOWN AND ADDITIONAL NOTES, SEE SHEET ES-3C OF THE STANDARD PLANS FOR MODEL 332 AND 334 CABINETS)



BASE PLAN FOR BBS MOUNTED TO THE MODEL 332 OR 334 CABINET

(FOR DIMENSIONS AND DETAILS NOT SHOWN, SEE SHEET A6-1 TO A6-4, CABINET HOUSING DETAILS OF THE TRANSPORTATION ELECTRICAL EQUIPMENT SPECIFICATION (TEES))

ELECTRICAL SYSTEMS
(BBS FOUNDATION DETAILS)

NO SCALE

THIS PLAN IS ACCURATE FOR ELECTRICAL WORK ONLY.

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Theresa Gabriel — 12-20-07
REGISTERED ~~CIVIL~~ ENGINEER — DATE

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Registered Professional Engineer — Theresa A. Gabriel — No. E15129 — Exp. 6-30-10 — ELECT — State of California

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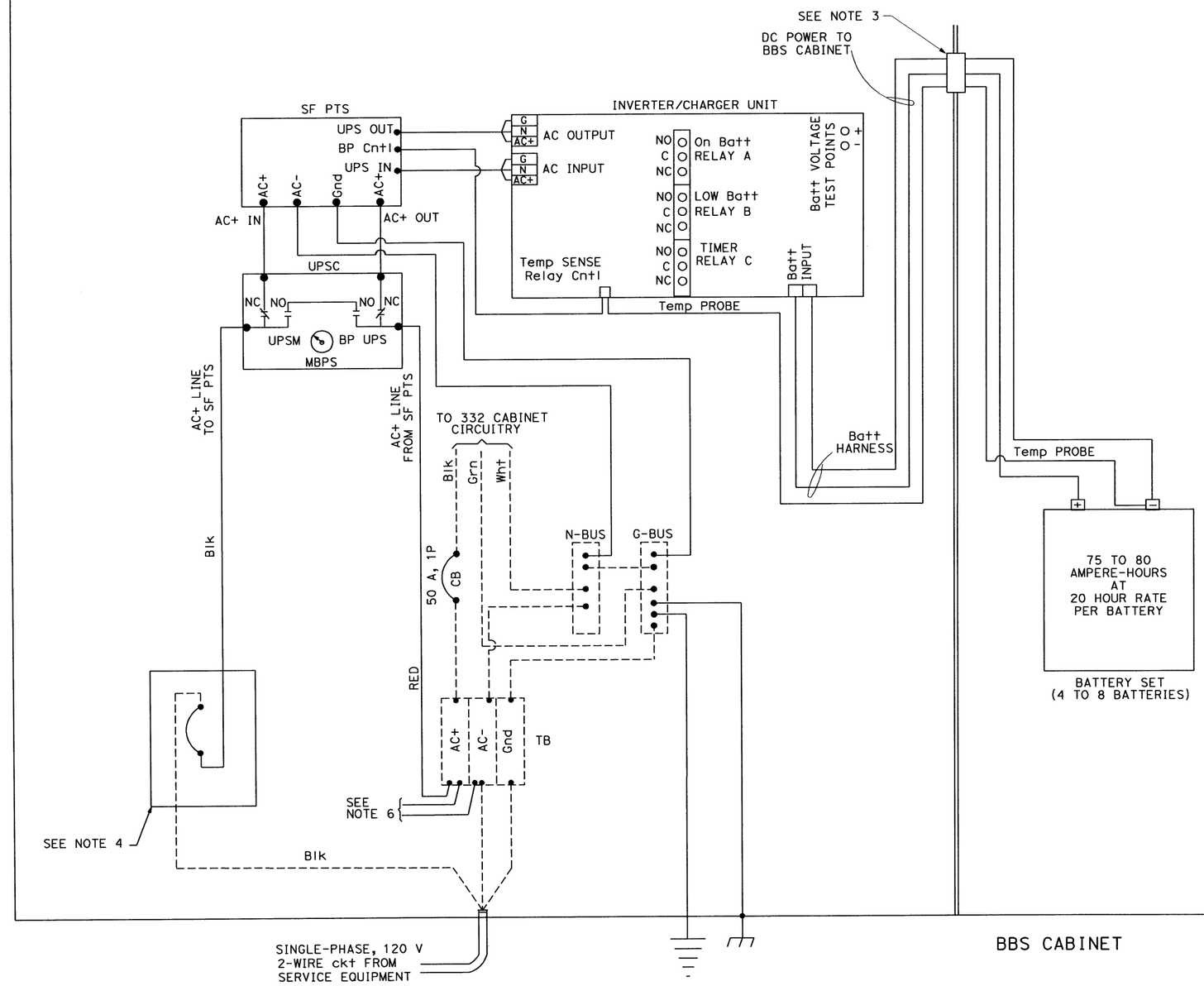

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UPS = UNINTERRUPTIBLE POWER SUPPLY
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UPSM = UPS MODE
BP = BYPASS
MBPS = MANUAL BYPASS SWITCH
AC+ = UNGROUNDED CONDUCTOR
AC- = GROUNDED CONDUCTOR
C = COMMON
Grn = GREEN
Blk = BLACK
Wht = WHITE
SF = STATE-FURNISHED
Batt = BATTERY
Temp = TEMPERATURE
TB = TERMINAL BOARD
Cntl = CONTROL
Gnd = GROUND

NOTES: (THIS SHEET ONLY)

1. TYPE B REFERS TO THE BBS EQUIPMENT FROM MANUFACTURER B.
2. CASE-2 REFERS TO THE SITUATION WHEN ONLY THE BATTERIES ARE INSTALLED IN THE BBS CABINET. THE REMAINING EQUIPMENT IS PLACED IN THE 332 CONTROLLER CABINET.
3. THE LOCATION OF THE 2"C NIPPLE WILL BE DETERMINED BY THE ENGINEER IN THE FIELD.
4. THE CONTRACTOR SHALL FURNISH AND INSTALL A NEMA-1 ENCLOSURE WITH 30 A, 1P, 120/240 VOLTS RATED CIRCUIT BREAKER MANUFACTURED PER UL STANDARD 489.
5. A TEMPERATURE PROBE SHALL BE ATTACHED TO THE BATTERY BY TAPE OR ATTACHED TO THE NEGATIVE TERMINAL OF THE BATTERY.
6. THE ELECTRICAL POWER FOR THE COOLING FAN FOR THE BBS CABINET SHALL BE TAPPED FROM THE BOTTOM OF THE TB IN THE 332 CABINET.
7. THE CONTRACTOR SHALL PROVIDE A 9-WIRE WIRING HARNESS OR BUNDLED 9 MULTICOLOR CONDUCTORS, #18 AWG WIRES FROM THE RELAY ON THE INVERTER/CHARGER UNIT TO THE CONTROLLER. THE ENDS OFTHE CONDUCTORS SHALL BE INSULATED WITH TAPE AND A SIX-FOOT COIL ON EACH END.

ELECTRICAL SYSTEM

(BBS POWER CONNECTION DIAGRAM, TYPE B, CASE-2)

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Theresa Gabriel — REGISTERED CIVIL ENGINEER — 12-20-07 DATE

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Cntl = CONTROL
Gnd = GROUND
Temp = TEMPERATURE
Batt = BATTERY

NOTES: (THIS SHEET ONLY)

1. TYPE A REFERS TO THE BBS EQUIPMENT FROM MANUFACTURER A.
2. CASE-1 REFERS TO THE SITUATION WHEN THE ENTIRE BBS EQUIPMENT INCLUDING THE BATTERIES ARE INSTALLED IN THE BBS CABINET.
3. THE LOCATION OF THE 2"C NIPPLE WILL BE DETERMINED BY THE ENGINEER IN THE FIELD.
4. THE CONTRACTOR SHALL FURNISH AND INSTALL A NEMA-1 ENCLOSURE WITH 30 A, 1P, 120/240 VOLTS RATED CIRCUIT BREAKER MANUFACTURED PER UL STANDARD 489.
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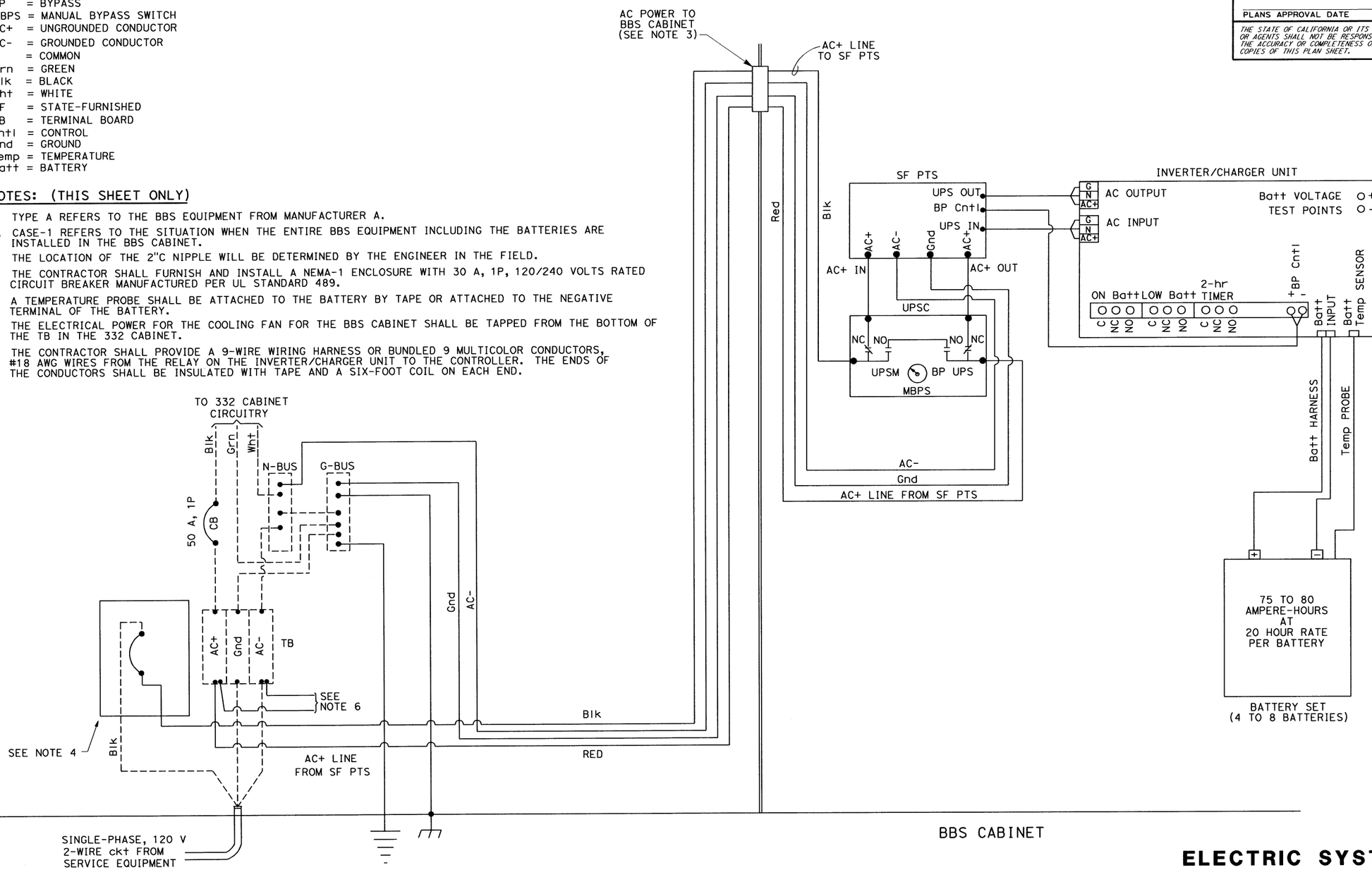

ELECTRIC SYSTEM
(BBS POWER CONNECTION DIAGRAM, TYPE A, CASE-1)

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NOTES: (THIS SHEET ONLY)

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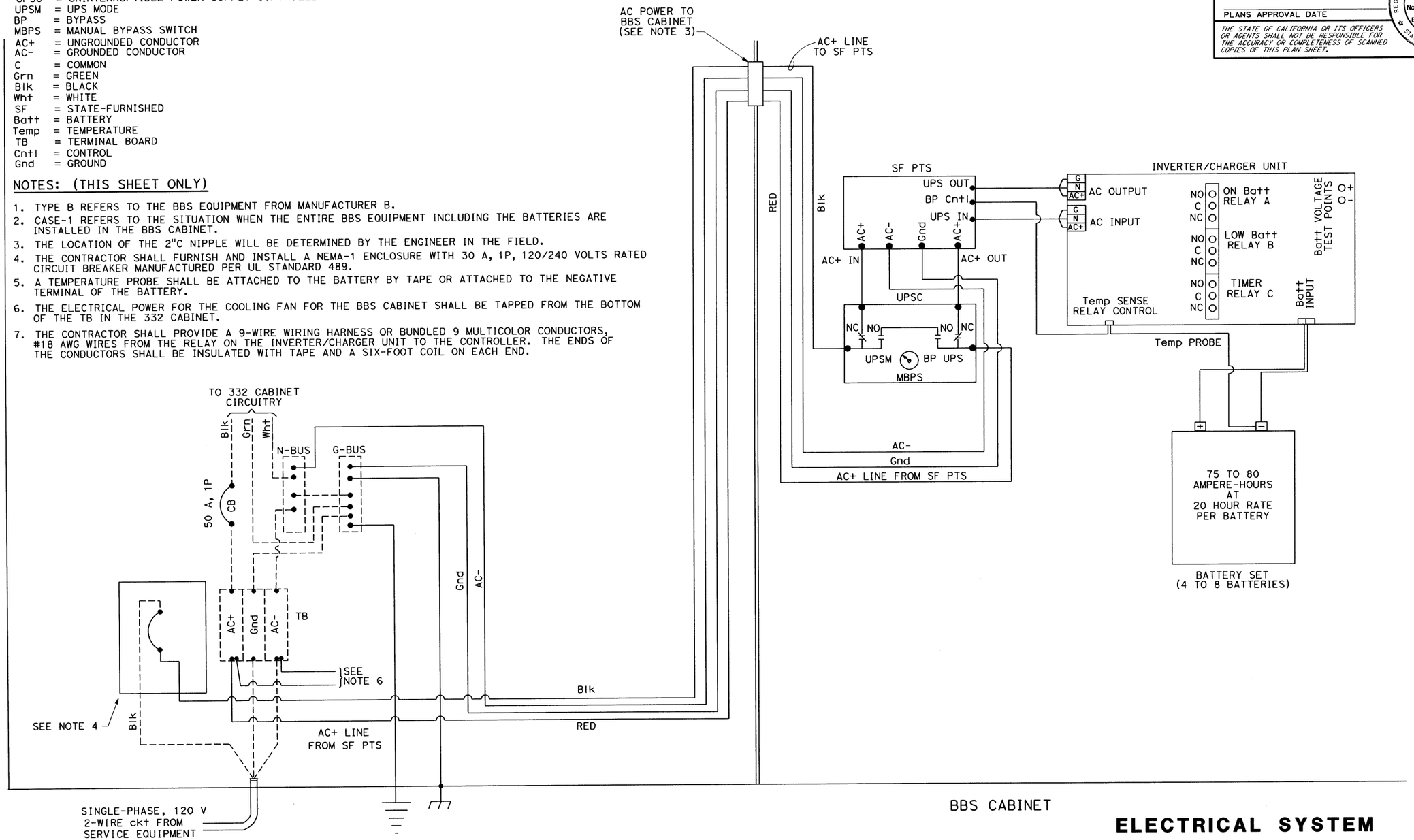

ELECTRICAL SYSTEM

(BBS POWER CONNECTION DIAGRAM, TYPE B, CASE-1)

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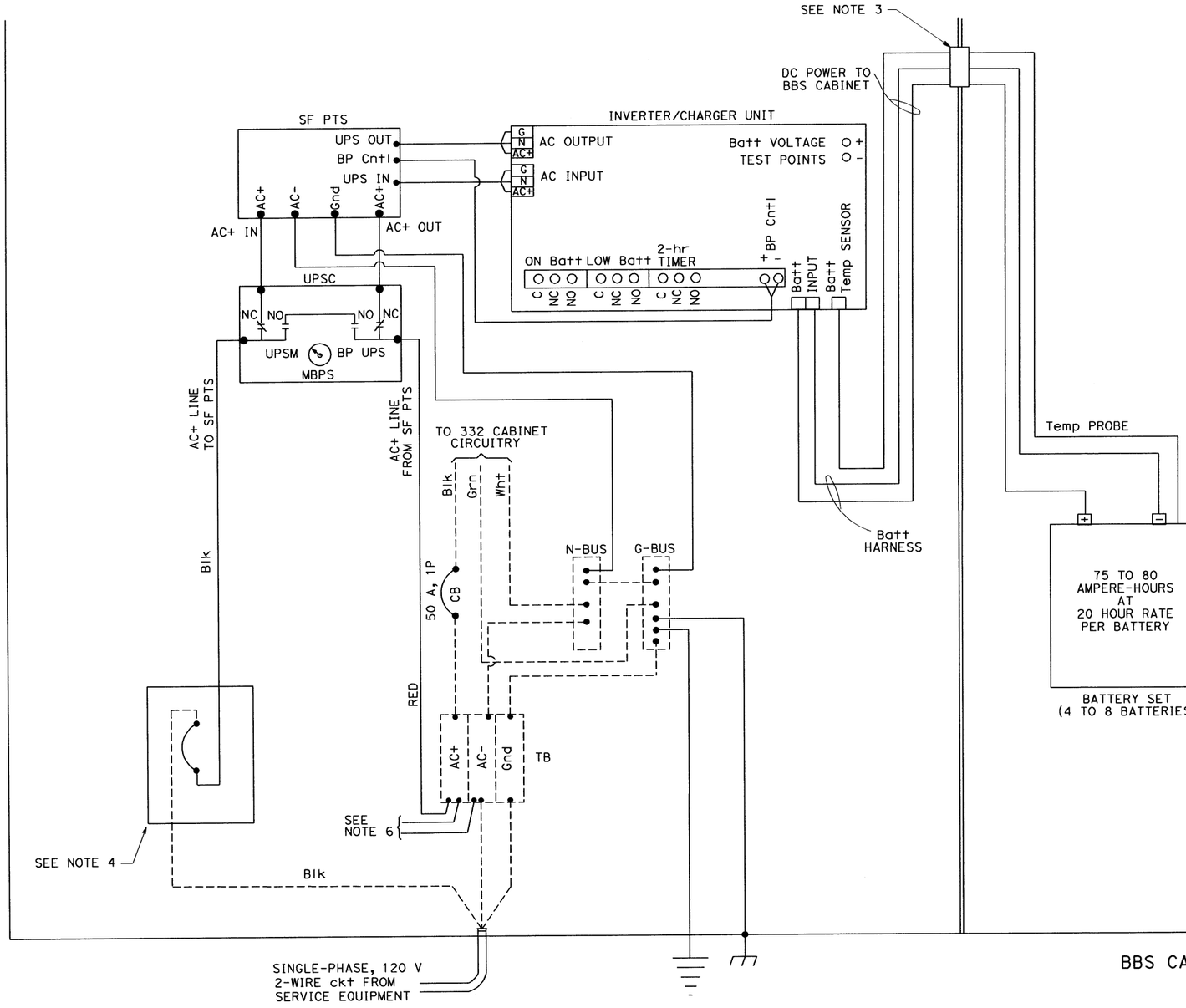


LEGEND: (THIS SHEET ONLY)

PTS = POWER TRANSFER SWITCH
UPS = UNINTERRUPTIBLE POWER SUPPLY
UPSC = UNINTERRUPTIBLE POWER SUPPLY CONTROLLER
UPSM = UPS MODE
BP = BYPASS
MBPS = MANUAL BYPASS SWITCH
AC+ = UNGROUNDED CONDUCTOR
AC- = GROUNDED CONDUCTOR
C = COMMON
Grn = GREEN
Blk = BLACK
Wht = WHITE
SF = STATE-FURNISHED
Batt = BATTERY
Temp = TEMPERATURE
TB = TERMINAL BOARD
Cntl = CONTROL
Gnd = GROUND

NOTES: (THIS SHEET ONLY)

1. TYPE B REFERS TO THE BBS EQUIPMENT FROM MANUFACTURER B.
2. CASE-2 REFERS TO THE SITUATION WHEN ONLY THE BATTERIES ARE INSTALLED IN THE BBS CABINET. THE REMAINING EQUIPMENT IS PLACED IN THE 332 CONTROLLER CABINET.
3. THE LOCATION OF THE 2"C NIPPLE WILL BE DETERMINED BY THE ENGINEER IN THE FIELD.
4. THE CONTRACTOR SHALL FURNISH AND INSTALL A NEMA-1 ENCLOSURE WITH 30 A, 1P, 120/240 VOLTS RATED CIRCUIT BREAKER MANUFACTURED PER UL STANDARD 489.
5. A TEMPERATURE PROBE SHALL BE ATTACHED TO THE BATTERY BY TAPE OR ATTACHED TO THE NEGATIVE TERMINAL OF THE BATTERY.
6. THE ELECTRICAL POWER FOR THE COOLING FAN FOR THE BBS CABINET SHALL BE TAPPED FROM THE BOTTOM OF THE TB IN THE 332 CABINET.
7. THE CONTRACTOR SHALL PROVIDE A 9-WIRE WIRING HARNESS OR BUNDLED 9 MULTICOLOR CONDUCTORS, #18 AWG WIRES FROM THE RELAY ON THE INVERTER/CHARGER UNIT TO THE CONTROLLER. THE ENDS OFTHE CONDUCTORS SHALL BE INSULATED WITH TAPE AND A SIX-FOOT COIL ON EACH END.

ELECTRICAL SYSTEMS
(BBS POWER CONNECTION DIAGRAM, TYPE A, CASE-2)

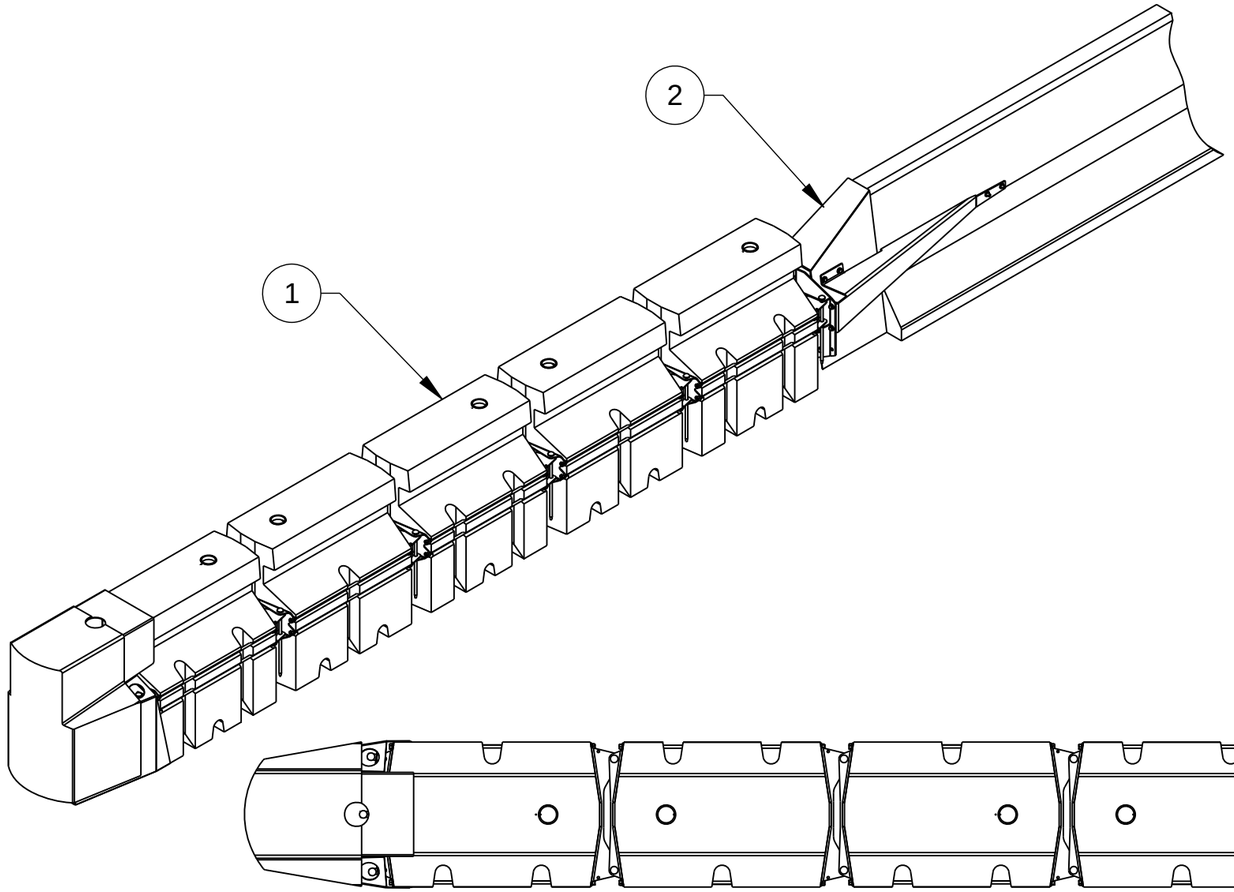
NO SCALE

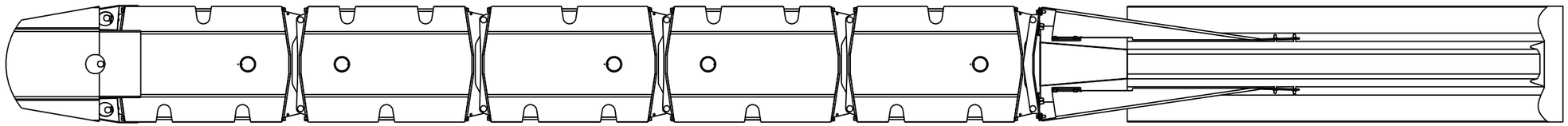
REVISED BY | DATE REVISED
CALCULATED-DESIGNED BY
CHECKED BY
FUNCTIONAL SUPERVISOR
STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION
Caltrans

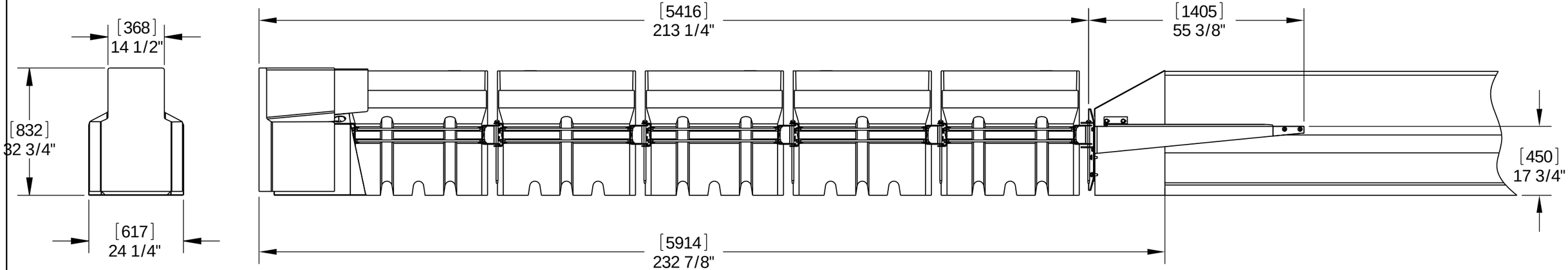
LAST REVISION 2-2-09
DATE PLOTTED => 13-MAR-2009
TIME PLOTTED => 09:07

BORDER LAST REVISED 4/11/2008
RELATIVE BORDER SCALE IS IN INCHES 0 1 2 3
USERNAME => trcarol
DGN FILE => BBS 1250FSP.dgn
CU 00000
EA 000000

Item	Qty	Part Description	Part#	U/M
1	1.00	AB350 70km PCB ARRAY, TL-2	AB070P	EACH
2	1.00	AB350 PCB ATTACHMENT ASSY	SBPCB	EACH







NOTE:
THICKNESS OF WELD TO BE EQUAL TO THE THINNER OF 2 PIECES BEING JOINED. WELD TO BE ALL AROUND UNLESS OTHERWISE SPECIFIED

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REV.	CHANGES	DATE	BY	REQ'D	NEXT ASSY.	ITEM

SCALE: 1:25

	DATE	INIT.
DRAWN BY	6/06/05	CMB
		GAD

Standard Tolerance
Angular +/- 1/2 Deg.
Fractional +/- 1/16
Dec. XXX= +/- .010
Dec. .XX= +/- .030

TITLE: AB350 70km SYSTEM, PCB, TL-2

MODEL	DRAWING NUMBER	REV
	AB070PSB	