NAPA RIVER WEST
HATT TO 1ST STREET FLOODWALL

NAPA RIVER/NAPA CREEK FLOOD PROTECTION PROJECT
NAPA, CALIFORNIA
NLD SYSTEM ID NO. 5305000050 SEGMENT ID NO. 5304000050

PERIODIC INSPECTION REPORT NO. 2
SEPTEMBER 2020

US Army Corps
of Engineers ®
San Francisco District

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QUALITY CONTROL CERTIFICATION

COMPLETION OF QUALITY CONTROL ACTIVITIES

The Walla Walla District has completed the Periodic Inspection Report No. 2 for Napa River Hatt Building to 1st Street Floodwall, Napa, California. Notice is hereby given that the DQC Review has been conducted in accordance with District policy. During this review, compliance with established policy principles and procedures, utilizing justified and valid assumptions, was verified.

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Susan Kelly, P.E.
Levee Safety Officer Engineering
& Technical Services
San Francisco District

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ACRONYMS AND ABBREVIATIONS

A	Acceptable
ASTM	American Society for Testing and Materials
cfs	cubic feet per second
CGS	California Geological Survey
DDR	Design Documentation Report
EM	Engineering Manual
ER	Engineering Regulation
ETL	Engineering Technical Letter
FEMA	Federal Emergency Management Agency
FWHA	Federal Highway Administration
FESWMS	Finite-Element Surface-Water Modeling System
FIRM	Flood Insurance Rate Map
FOUO	For Official Use Only
ft	foot or feet
gpm	gallons per minute
GPS	Global Positioning System
H:V	Horizontal:Vertical
in.	inch or inches
ITR	Independent Technical Review
lb	Pounds
LIS	Levee Inspection System
LSO	Levee Safety Officer
M	Minimally Acceptable
MLLW	Mean Lower Low Water
MSL	Mean Sea Level
n	Coefficient of Roughness
NA	Not Applicable
NAVD88	North American Vertical Datum of 1988
NCFCWCD	Napa County Flood Control and Water Conservation District

NGS	National Geodetic Survey
NSD	Napa Sanitation District
NLD	National Levee Database
NWW	Walla Walla District
O&M	Operations & Maintenance
Project	Napa River/Napa Creek Flood Protection Project
pcf	pounds per cubic foot
PGA	Peak Ground Acceleration
PI	Periodic Inspection
PL	Public Law
psf	pounds per square foot
psi	pounds per square inch
ROW	Right-Of-Way
SGDM	Supplemental General Design Memorandum
SPN	San Francisco District
U	Unacceptable
USACE	United States Army Corps of Engineers
USGS	United States Geological Survey

PART 1 - EXECUTIVE SUMMARY

This Executive Summary provides the scope and purpose of the periodic inspection (PI), an overview of the Napa River West, Hatt to 1st Street Floodwall System, a summary of the major findings of the PI, and the overall levee system rating.

1.1 **Scope and Purpose of Periodic Inspection**

The purpose of the Napa River West, Hatt to 1st Street Floodwall System PI is to identify deficiencies that pose hazards to human life or property, and to determine design adequacy relative to present day criteria. The inspection is intended to identify the issues in order to facilitate future studies and associated repairs, as appropriate.

This assessment of the general condition of the Napa River West, Hatt to 1st Street Floodwall System is only based on available data and visual inspections. Detailed investigation and analysis involving hydrologic design, topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of this PI.

1.2 **System Summary**

The Napa River/Napa Creek Flood Protection Project is a federally authorized, multiphase urban project that was designed to provide 100-year level of flood protection and also referred to as the 1% annual chance of exceedance (ACE) flood event to the city of Napa, California. Herein, this overall flood protection project will be simply be referred to as the "Project". The Project spans almost 7 miles of the Napa River from Trancas Street to the Highway 29 crossing. The Napa River, right bank system (System 0050) extends from First Street to Imola Avenue along the right bank of the Napa River, a distance of about 1.38 miles. The Napa River West, Hatt Building to First Street floodwall segment (Hatt to 1st Street Floodwall) extends from the Hatt Building to First Street, a distance of about 0.34 miles. The remainder of the system downstream from the Hatt Building has yet to be designed or constructed. This report covers only the Napa River West, Hatt Building to First Street Floodwall segment of the flood protection project.

The entire flood protection project is within the city of Napa, California. This flood protection system protects the city of Napa from the Napa River. The floodwall segment from the Napa River West, Hatt Building to First Street, is identified hereinafter as "Hatt to 1st Street Floodwall" or simply as the "Floodwall". A general location map is shown in Figure 1-1.

The local sponsor is the Napa County Flood Control and Water Conservation District (NCFCWCD). The U.S. Army Corps of Engineers (USACE) Sacramento District recently transferred Hatt to 1st Street to NCFCWCD for long-term operation and maintenance. A final inspection or PI is required for the transfer of all levee/floodwall segments.

The Project was authorized by the Flood Control Act of 1965 (Public Law 89-298). Recreation features were included as an allied purpose in the authorizing document, House Document 222, 89th Congress, 1st Session, and are also an authorized purpose for the Project. The recreational elements within the Hatt to 1st Street Floodwall include 2700 feet of promenade, Veterans Park and marshplain terrace covered with stone protection.

1.3 **Summary of Major Deficiencies**

There were no major deficiencies that were observed by the inspection team or issues rated as "unacceptable" for this PI.

1.4 **Overall Rating**

The overall rating of the Hatt to 1st Street Floodwall Segment is "acceptable" based on USACE Levee Safety Program rating criteria and the results of this periodic inspection. The Floodwall appears to have the ability to continue safe operation as a flood reduction system and function as authorized. See Appendix B, Flood Damage Reduction Segment/System Inspection Report, and Part 5 of this report for more information.

Figure 1-1: Location Map of the Napa River Hatt to 1st Street Floodwall System

PART 2 - INSPECTION TEAM AND DATE OF INSPECTION

The following section contains a summary of general information pertaining to the inspection team and conditions during the PI of the Hatt to 1st Street Floodwall System. The information presented below was obtained through readily available data sources and is accurate and complete to the best of our knowledge at the time of preparation of this report.

2.1 **Inspection Team**

The inspection team consisted of one representative from NCFCWCD and three representatives from USACE. Mr. Jeremy Sarrow represented NCFCWCD and is their designated lead point of contact for the Project. Mr. John Conway represented USACE San Francisco District and is the Levee Safety Program Manager. Mr. Michael Franssen, USACE Walla Walla District served as the inspection team lead, and has a background in Civil Engineering. Mr. Nathan DeLannoy, USACE Walla Walla District, served as the inspection recorder and has a background as a Civil Engineering Technician.

2.2 **Date of Inspection**

The PI was conducted on 22 July 2020

2.3 **Weather During Inspection**

The weather on the day of the PI was partly cloudy, with light winds and temperatures in the mid to high 70's (degrees Fahrenheit).

2.4 **River Gauge or Elevation Readings During Inspection**

The closest stream gage to the Hatt to 1st Street Floodwall System is USGS stream gage 11458000 the recorded gage height was approximately 1.97 feet (ft) during the PI, which results in no apparent discharge on the Napa River.

PART 3 - SYSTEN BACKGROUND INFORMATION

The following section contains detailed information pertaining to the Hatt to 1st Street Floodwall relating to design and expected project performance. Additional information, including as-built drawings, is in the appendices of this inspection report.

3.1 **Project Description**

The Hatt Building to 1st Street Floodwall is about 0.34 miles long, located in downtown Napa and consists of floodwalls and associated features. The surrounding area is heavily developed with business, local government offices, and housing units. Access to the segment floodwalls is by walkways from Brown, Fifth, and Third Streets and Veteran's Park. Major roadways that cross the floodwall alignment are Third Street and First Street.

3.1.1 Project Type

The Project is a federally authorized urban flood protection project. The Project will be locally operated and maintained after transfer to the local sponsor.

3.1.2 Authority

Construction of the local flood protection measures along the Napa River from Edgerly Island to Trancas Street was authorized by the Flood Control Act of 1965 (Public Law 89-298). Recreation features were included as an allied purpose in the authorizing document, House Document 222, 89th Congress, 1st Session, and are also an authorized purpose for the Project. Napa Creek was added to the Project authorization by the Flood Control Act of 1976 (Public Law 94-587).

3.1.3 Cost

The Operations, Maintenance, Repair, Replacement and Rehabilitation Manual for the Napa River / Napa Creek Flood Protection Project (USACE 2018) indicates that the Construction cost of the CT 2West Hatt Building to 1st Street, floodwall segment was $35,872,136.. Herein, the manual will simply be referred to as the "OMRR&R Manual".

3.1.4 Completion Date

Construction of the CT 2West Hatt Building to 1st Street, floodwall segment was accomplished under Contract No. W91238-05-C-0020 by R&L Brosamer, Inc. of Alamo, California during the period from 2005 to 2008.

3.1.5 Public Sponsor

NCFCWCD is the public sponsor and will operate and maintain the Hatt to 1st Street Floodwall. The point-of-contacts for NCFCWCD are referenced in Table 3-1.

Table 3-1: NCFCWCD Points of Contact

Name	Address	Phone	Email
Jeremy Sarrow (Primary Point of Contact)	804 First Street Napa, California 94559-2623	(707) 259-8204	Jeremy.Sarrow@CountyofNapa.org
Andrew Butler	804 First Street Napa, California 94559-2623	(707) 259-8671	Andrew.Butler@CountyofNapa.org
Richard Thomasser	804 First Street Napa, California 94559-2623	(707) 259-0407	Richard.Thomasser@CountyofNapa.org

3.1.6 Location

The Napa River/Napa Creek Flood Protection Project is located in Napa County, California, with the majority of the project work occurring within the city of Napa. The limits of the Project start at the State Highway 29 bridge over the Napa River and extends approximately 6.9 miles upriver (north) to Trancas Street. The Project also includes approximately two-thirds of a mile of Napa Creek starting at its confluence with the Napa River and extending upstream to Jefferson Street. This flood protection project protects the City of Napa against flooding from the Napa River and Napa Creek. This Periodic Inspection report only covers the CT 2West Hatt Building to First Street, floodwall segment of the flood protection project, which is located on the left (west) bank of the Napa River in downtown Napa. This segment is 0.34 miles long and consists of floodwalls and associated drainage, irrigation, walkways, and ramp/stair facilities. The Hatt to First Street Floodwall System (System 50) part of the Project is shown in Figure 3-1 below.

3.1.7 Potential Consequences

The *Supplemental General Design Memorandum* (USACE 1998) identified average annual flood damages of $27,704,000 for the "largest floodplain" (1430 to 500-year) and $163,834,000 for the "medium floodplain (65 to 50-year), in October 1997 dollars, for the Project. Herein, the *Supplemental General Design Memorandum* will simply be referred to as the "*SGDM*". Average annual flood damages specific to the Hatt to 1st Street Floodwall system are not given in the *SGDM*.

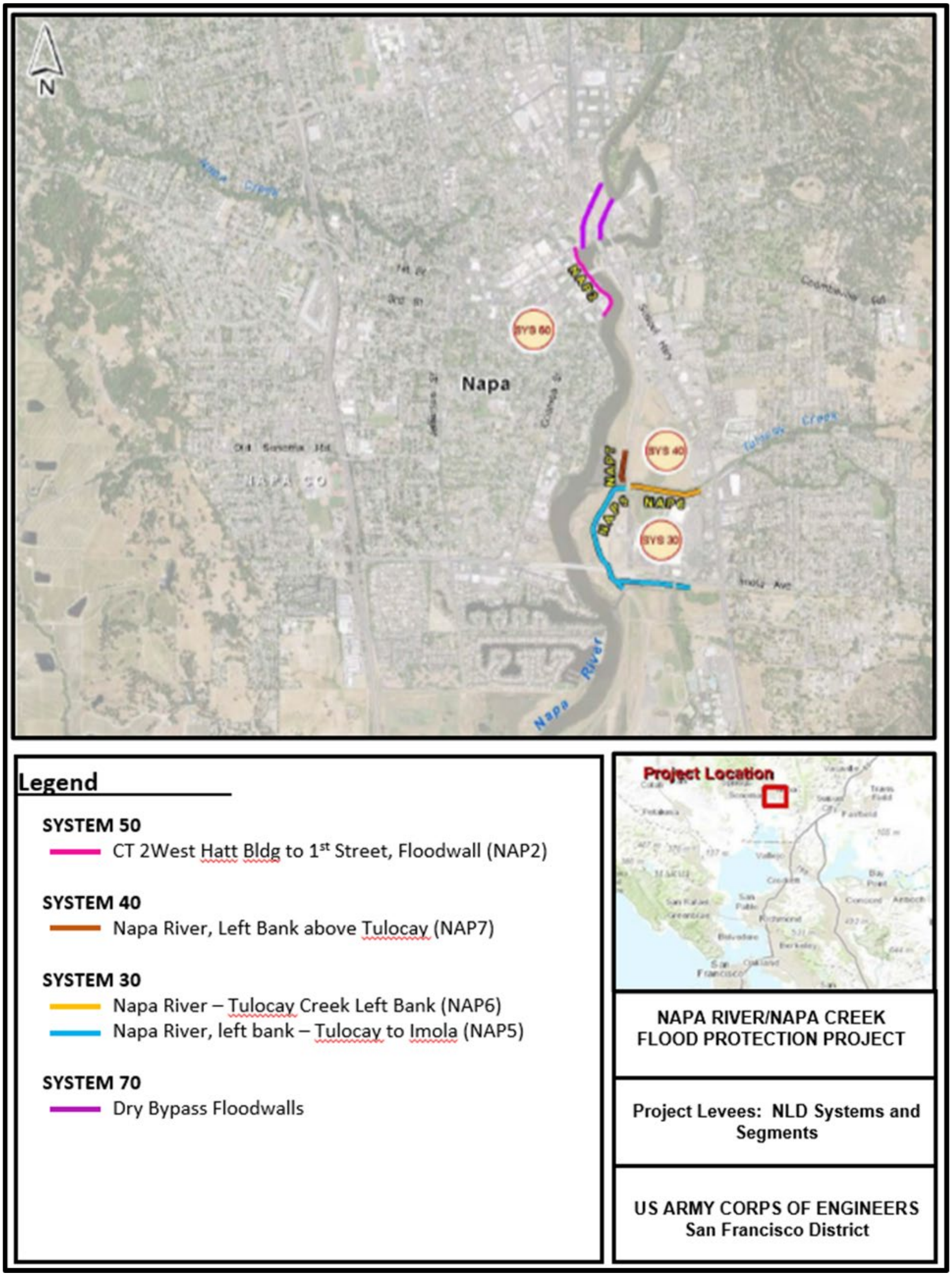

Figure 3-1: Napa Levee Safety System Map

3.1.8 Investigations Prior to Construction

USACE, SACRAMENTO DISTRICT (OCTOBER 1998). Napa River/Napa Creek Flood Protection Project, Supplemental General Design Memorandum (SGDM). This document presents feasibility-level plans for the entire Napa River/Napa Creek Flood Protection Project and serves as the project authorization document. The Geotechnical Appendix includes a detailed discussion of regional geology and seismic sources, soil boring logs and laboratory test data as of the document publication date, a general description of foundation conditions, preliminary values of unit weight and shear strength, slope stability, preliminary floodwall design, and a preliminary evaluation of liquefaction potential.

U.S. Army Corps of Engineers (USACE), Sacramento District (May 2005) Napa River/Napa Creek Flood Protection Project, Contract 2 West Hatt Building to 1st Street, Geotechnical Design Document Report (DDR).

The DDR is a detailed document covering only the Hatt to 1st Street Floodwall segment of the flood protection project and serves as the document of record for the geotechnical design of this segment. Included in the DDR are soil boring logs (from before and after the SGDM date), laboratory test data, a subsurface profile, a description of the soil and groundwater conditions along the floodwall alignment, floodwall foundation design details (deep and shallow), slope stability analysis, seismic analysis using a PGA of 0.5g, dewatering, the impact of construction on nearby structures, and settlement/vibration monitoring of nearby structures during construction.

The geotechnical evaluations included the following:

- Historical data collection and review.
- Field exploration program (SPT, CPT).
- Laboratory testing
- Data interpretation and site characterization.
- Shallow floodwall foundation design
- Deep floodwall foundation design
- Global slope stability of the floodwall system
- Seismic analysis
- Evaluation of construction impact on nearby structures

3.1.9 History of Remedial Measures

The Hatt to 1st Street Floodwall segment of the Napa River/Napa Creek Flood Protection Project was constructed between 2005 and 2008. The only flood events experienced since construction completion occurred in March 2011 and December 2012. No signs of distress were observed in the floodwalls during or after the events. Due to the recent construction of this segment and the lack of flood history since construction, no remedial measures have been performed on this segment.

3.2 **Description of Pertinent Features**

The CT 2West, Napa River, right bank system currently consists of 1 segment; CT 2West

Hatt Building to 1st Street, floodwall (or Hatt to 1st Street Floodwall). According to the SGDM, the Napa River/Napa Creek Flood Protection Project upstream of Imola Avenue protects approximately 1,308 acres of urban and industrial development. No estimate of the area protected specifically by the Hatt to 1st Street Floodwall segment is given in the SGDM. Flood protection for the Floodwall segment is provided by a combination of floodwalls, concrete walkways, concrete planter areas and a stop-log closure.

3.2.1 Cantilever Floodwall

The floodwall (inverted cantilever-wall founded on cast-in-drilled-hole (CIDH) piles) begins at the south (downstream) end of the project at the Hatt building. The concrete walkways attached to the wall have been constructed to an elevation to provide the necessary protection at key locations where floodwaters may outflank the beginning of the wall. The floodwall continues north around the Napa River Inn at the Hatt complex to 5th Street, where the primary floodwall separates into lower (I-wall) and upper (inverted cantilever-wall) walls with a pedestrian promenade behind the upper wall and a pedestrian walkway behind the lower floodwall. There is a concrete stairway over the floodwall at the terminus of 5th Street allowing pedestrians access from the upper to lower promenade. The elevation of the upper promenade has been set above the 100-year design flood elevation. Upper and Lower floodwalls continue from 5th Street to the 3rd Street Bridge. A break in the upper floodwalls between 5th and 3rd Streets provides pedestrian stairs and ramps to access the lower promenade and river docks. The top of the ramps and stairs have been constructed above the 100-year design flood elevation. The upper wall and promenade continue north to and connects with the 3rd Street abutment providing flood protection. Approaching the 3rd Street Bridge, the lower promenade passes beneath the 3rd Street Bridge and will be flooded during the 10-year event and greater. See Figure 3-2 for a typical cross section of the floodwall.

3.2.2 Veteran’s Park

Just north of the 3rd Street Bridge is Veteran's Park. The lower floodwall (I-wall) is the primary line of flood protection for 10-year and lesser flow events. Veteran’s Park consists of a terraced amphitheater with vehicular and pedestrian access along the south side of the amphitheater, just north of the 3rd Street Bridge. The vehicle/pedestrian ramp is constructed at a crest elevation exceeding the 100-year flow event. This ramp connects the park with Main Street to the west and the lower promenade trail to the east.

3.2.3 Concrete Seat Walls

The concrete seat walls along the upper terrace have been constructed to an elevation that exceeds the 100-year flood event. Immediately west of the amphitheater is an ADA accessible pedestrian ramp with access to Main Street and the lower promenade. This ramp does not meet the 100-year flow event and has been outfitted with a Stop Log structure to be installed during high water events to prevent floodwater from flowing onto Main Street at this location.

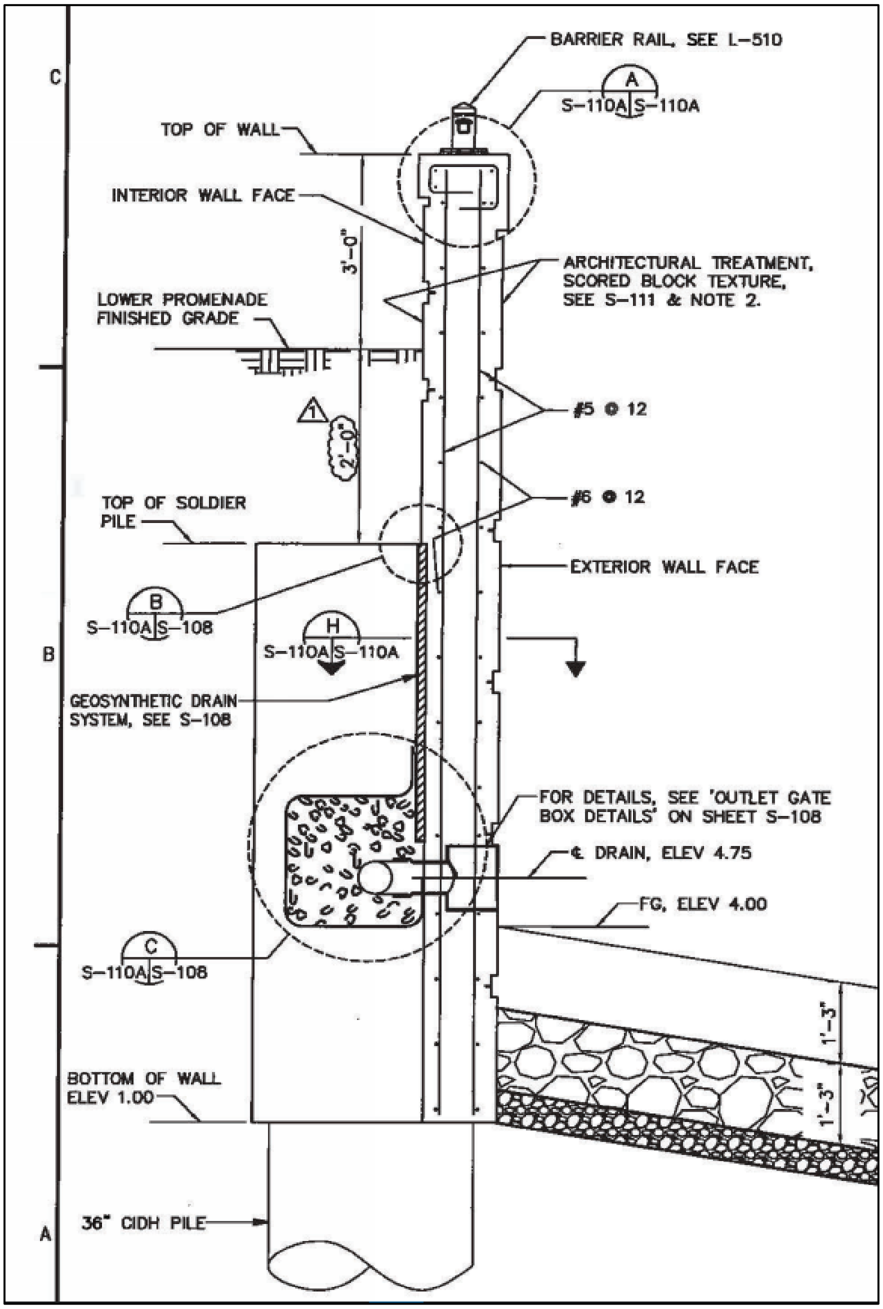

Figure 3-2: Typical Cross-Section (Type C Modified) of the Floodwall

3.3 **Topography, Geology, Seismicity, and Groundwater**

The topographic, geologic, and foundation conditions for the Hatt to 1st Street Floodwall System are characterized in the *SGDM*, the *USACE (2005) Geotechnical Design Document Report* and the as-built drawings (USACE 2006 and 2007). They are summarized below. The Napa Dry Bypass DDR (USACE 2011) also discusses seismic analysis and some of the information from that report is included in the following.

3.3.1 Regional Geologic Setting, Site Specific Geology, and Topography

The Project is located in the Coast Ranges Physiographic Province, which is composed of the Southern Coast Ranges and Northern Coast Ranges, extend to the Great Valley Province to the east, the Pacific Ocean to the west, the Klamath Mountains Province to the north, and Transverse Ranges in the south. The Northern Coast Ranges Physiographic Province typically trend parallel to the California coastline with north-to-south trending mountain ranges and valleys, including the Napa Valley. The Northern Coast Ranges are dominated by extensive hills with landside characteristics from the Franciscan Complex. In several areas, Franciscan rocks are overlain by volcanic cones and flows of the Quian Sabe, Sonoma, and Clear Lake volcanic fields (California Geological Survey [CGS] 2002).

The Napa Valley is a northwest-trending with the Napa River flowing south through the Napa Valley and into San Francisco Bay. The valley is bounded to the west by sedimentary rocks of the Late Jurassic/Early Cretaceous Franciscan Formation and Late Jurassic to Cretaceous Great Valley Formation. To the north and east, the valley by overlying Pliocene and early Miocene volcanic rocks (United States Geological Survey [USGS], 2006). The valley floor is covered by alluvium and older alluvium composed of sediment derived from both sides of the valley.

3.3.1.1 Seismicity

The Napa Valley is in an area containing many active fault zones. Major faults in the area are the San Andreas (capable of producing an earthquake of magnitude 8.25), Hayward (capable of producing an earthquake of magnitude 7.5), and Concord (capable of producing an earthquake of magnitude 6.5) faults. The (smaller) faults closest to the project are the Soda Creek (capable of producing an earthquake of magnitude 6.25) and West Napa (capable of producing an earthquake of magnitude 6.5) faults, located on the east and west edges of the Napa Valley respectively. A design peak ground acceleration (PGA) of 0.5g was selected for the project (USACE 1998 Paragraph 18.2.5) Soils along the Napa River in the area of the Hatt to 1st Street Floodwall segment are shown as having a high liquefaction potential on the Liquefaction Susceptibility Map, Napa, California.

According to the *Napa Dry Bypass DDR*, an estimated peak ground acceleration of 0.27g was estimated for a 100-year event (estimated magnitude 6.7) from the 2008 Probabilistic Seismic Hazard Analysis (PSHA) USGS model. This peak ground acceleration was used for the seismic evaluation of the Dry Bypass and is appropriate for the other Napa River Flood Protection Project features.

On August 24, 2014, the Main Street USGS Station N016 measured a 6.0 magnitude earthquake, 9.1 miles from the epicenter, with a peak ground acceleration of 0.61g. This monitoring station is within 1 mile of the Hatt to 1st Street Floodwall System. (Strong-Motion Center 2016).

3.3.1.2 Groundwater Conditions

The various exploratory programs performed for the Project indicate that the groundwater elevation for the Hatt to 1st Street Floodwall system varied between 14 and 20 feet below ground surface (USACE (2005) Geotechnical Design Document Report). Groundwater levels are expected to vary depending on time of year, rainfall, river stage, and irrigation/pumping activities.

3.3.2 Subsurface Investigation and Foundation Conditions

The Hatt Building to 1st Street Geotechnical Design Document report (USACE 2005) is a detailed document describing the foundation conditions and the geotechnical design of all the elements in the Floodwall segment. Included in the document are soil boring logs, laboratory test data, a subsurface profile, a description of the soil and groundwater conditions, floodwall foundation design details (deep and shallow), slope stability analysis, seismic analysis, dewatering, the impact of construction on nearby structures, and settlement/vibration monitoring of nearby structures during construction.

The soil borings within the Floodwall segment indicate a soil profile of 20-22 feet of silts, sandy clays, and clayey sands of medium plasticity, underlain by 8 to 30 feet of a dense clayey sand and gravel, underlain by 12 to 36 feet of clay and sandy clay of medium to occasionally high plasticity, underlain by 8 to 10 feet of dense clayey sand and gravel, underlain by lean clay. The upper dense clayey sand and gravel is thicker at the downstream end of the segment and the "middle" fine-grained layer is thicker at the upstream end of the segment. Clays in the Napa Valley are overconsolidated with a typical overconsolidation ratio (OCR) of 2.

3.3.3 Floodwall Design and Construction

Referencing the USACE (2005) Geotechnical Design Document Report, section 6.2.4 states the following: "MGE submitted calculations of the wall loadings, design values, and deflections in each of their submittals. The final values are in the Structural Design Calculations (100% Submittal) report (reference 6). For hydraulic structures, EM 1110-2-2502 (Reference 3) recommends the use of the coefficient of earth pressure at rest (K_o) rather than the active earth pressure coefficient (K_A) for calculating horizontal soil pressures on retaining and flood walls. This is because hydraulic structures are often critical features, and since K_o is greater than K_A, the calculated loadings will be higher, resulting in a more conservative design. For each wall type, the station with the greatest free wall height was chosen for design. The soil and water loadings were calculated for four different cases: end-of-construction, long-term with no flood, long-term with a flood, and long-term with an earthquake and no flood. The case which produced the highest loadings was selected for structural design purposes. The small passive wedge above the bottom of the soldier pile wall was ignored in all the calculations, simulating erosion at the toe of the wall. A rapid drawdown case was not examined because rapid drawdown conditions are highly unlikely to develop in this project. The 100-year hydrograph for the Napa River indicates the river level rises and falls relatively quickly (2 days). The vertical concrete wall faces, the pavements on the upper and lower promenade, and the trench drains will reduce water infiltration into the soils behind the retaining walls. The lower wall has a drainage system consisting of a geocomposite drainage net, gravelly sand structural backfill, and a collector pipe surrounded in gravel with weep holes about 1 foot above the mean high tide water level. Any excess water that infiltrates the backfill material will drain relatively quickly."

3.3.4 Hydrologic/Hydraulic

The Napa River Basin lies in California's Central Coast Mountain Range, draining 426 square miles in Napa and Solano County. The headwaters of the basin are on the southeast slope of Mount Saint Helena. The basin is approximately 50 miles long and 10 miles wide (USACE 1998).

3.3.5 Past Project Performance

The Hatt to 1st Street Floodwall segment of the Napa River/Napa Creek Flood Protection Project was constructed between 2005 and 2008. The only flood events experienced since construction completion occurred in March 2011 and December 2012. The maximum recorded river level was 22.6 feet NGVD 29 on March 20, 2011 at the USGS stream gage11458000, located approximately 5 miles upstream of the Hatt to 1st Street Floodwall segment. This corresponds to a flow of 12,290 cubic feet per second (cfs) and a return period of just under 3 years. The recorded river stage on December 2, 2012 was 23.75 feet NGVD 29, corresponding to a flow of 10,802 cfs. The recorded river stage on December 24, 2012 was 23.83 feet NGVD 29, corresponding to a flow of 13, 509 cfs. The December 24, 2012 event corresponds to a return period of about 3 years. The largest flow recorded at this gage was 32,580 cfs in March 1995, which corresponds to a return period of about 70 years. No signs of distress were observed in the floodwalls during or after the event. Due to the recent construction of this segment and the lack of flood history since construction, no remedial measures have been performed on this segment. Flood Insurance Study

The Federal Emergency Management Agency (FEMA) Flood Insurance Rate Map (FIRM) 06055C0516F covers the Hatt to 1st Street Floodwall System. Both FEMA FIRMs indicate that area behind the Hatt to 1st Street Floodwall System is classified in the Zone X floodplain. The Zone X floodplain is defined by FEMA as areas subject to inundation by the 0.2% annual chance (500-year) flood event. However, the map was last updated in September 2010, prior to construction of the Dry Bypass. It is anticipated that a revision to the map would indicate the area be only within Zone AE. The Zone AE floodplain is defined by FEMA as areas subjected to inundation by the 1% annual chance flood event.

3.4 **Previous Periodic Inspection Findings**

The previous periodic Inspection was performed in 2013 by the Sacramento District (SPK). The inspection assessed the ability of each feature and overall system to function as authorized with respect to hydraulic and geotechnical issues. The 2013 PI found the overall system to have the ability to continue safe operations as a flood reduction system.

The floodwalls associated with the segment were inspected on 20 July 2011 by a team from SPK and the San Francisco District (SPN). NCFCWCD has been performing basic maintenance of the floodwalls. The following items were noted during the inspection:

- Vegetation growth was present at several locations for most of the floodwall segment. The growth was generally on the face of the floodwall and in the vegetation free zone (VFZ). A majority of the vegetation was designed to be a part of the project and should be maintained per the OMRR&R Manual.
- Minor separation was found along the edge of a few pilasters in the lower wall and the connection between the sidewalk and back of the lower wall in the Veteran’s Park Area. These separations are currently being monitored by the Sponsor and have not shown signs of continued movement.

3.5 **References**

Below is a list of references that are used in this report. Note: these do not include the USACE design references (such engineering manuals and engineering regulations) that are included at the end of Part 4 of this report.

- American Society of Testing and Materials (ASTM), 2012. *D1557-12e1, Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft3 (2,700 kN-m/m3)), ASTM International, West Conshohocken, PA.*
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- Jennings, C.W., and Bryant, W.A., 2010. *Fault activity map of California: California Geological Survey, Geologic Data Map No. 6, Map Scale 1:750,000.*
- Strong Motion Center, 2016. *CESMD, Information for Strong-Motion Station, Main St, Napa, CA, USGS-NCSN Station N016.* http://www.strongmotioncenter.org/cgi-bin/CESMD/stationhtml.pl?stationID=NCN016&network=NCSN
- U.S. Army Corps of Engineers (USACE), 1998. *Napa River/Napa Creek Flood Protection Project, Final Supplemental General Design Memorandum.*
- U.S. Army Corps of Engineers (USACE), Sacramento District, 2005. *Napa River/Napa Creek Flood Protection Project, Contract 2 West Hatt Building to First Street, Geotechnical Design Document Report.*
- U.S. Army Corps of Engineers (USACE), 2014. *Napa River/Napa Creek Flood Protection Project, Napa, California – Contract 2 East Geotechnical Design Document Report.*
- U.S. Army Corps of Engineers (USACE), 2011. *Napa Dry Bypass Plans and Specifications for the Napa River Flood Protection Project, Napa, California – 100% Design Submittal Design Documentation Report.* Prepared by McMillen.
- U.S. Geological Survey (USGS), 2019. Napa River, Near Napa, California Stream Gage.

PART 4 - DESIGN CRITERIA REVIEW

Design for the features in the Contract 2 West Hatt Building to First Street portion of the Napa River/Napa Creek Flood Project began in 2004 and was complete in 2005. Geotechnical Design was performed by the US Army Corps of Engineers, Sacramento District. Structural and Civil Design was performed by MGE Engineering, Inc., of Sacramento, California. Landscape and Electrical Design was performed by The HLA Group of Sacramento, California.

The inspection team reviewed the documentation referenced in the Introduction section and evaluated the levee system's documented design criteria against current design criteria. The purpose of the evaluation is to assess the ability of each feature and overall system to function as authorized and identify potential needs to update system design. The results of the design criteria review demonstrate no concerns with the design and specifics for each feature are described in the following sections.

4.1 **Geotechnical**

The Geotechnical Design Document Report by the Army Corps of Engineers, Sacramento District (May 2005) provides detailed geotechnical analyses for the Hatt to 1st Street Floodwall segment. The Sacramento District performed slope stability, shallow foundation bearing capacity, CIDH pile axial capacity, and filter analyses for the design of the floodwalls at the project design flood. Seepage analysis was not conducted because the Hatt to 1st Street Floodwall floodwalls will have no differential head across them at the project design flood and will have 0-3 feet (average 2 feet) of differential head across them when the water is at the floodwall crest elevation. The upper "pervious" subsurface layer is a silty/clayey sand and gravel with 9-45 percent fines (i.e. semipervious). The project design hydrograph shows a flood duration of 4 days. Given the low differential head across the floodwalls, the short duration of flood events, the lack of a highly pervious subsurface layer, and the impermeable nature of the concrete floodwalls, seepage is not expected to be a problem with this segment. Settlement analyses was also not conducted because the clays in the Napa area are overconsolidated. The additional loadings supplied by the floodwalls are lower than the preconsolidation pressure of the clays, so settlements are expected to be less than one inch.

4.1.1 Soil Investigations

The subsurface investigation and laboratory testing program supporting the project basis of design is summarized in Part 3. The following paragraph was taken from Contract 2 West Hatt Building to 1st Street GDDR (USACE 2005), section 2.0.

At the time of the SGDM preparation, Soil Design section had the following explorations in the Hatt to 1st contract area, from south to north: 2F-90-29, 2F-30 (just south of the Napa Mill); 2F-94-14 (just north of the Napa Mill); 2F-29, CPT-94-2, and 2F-94-15 (near the Third Street bridge). For plans and specifications, more subsurface information was needed, so the following deep explorations were conducted by the Corps: 2F-03-3, 2F-03-4, 2F-04-51 (from a barge in the Napa River near the Napa Mill); 2F-03-5, 2F-03-6, 2F-03-7 (between Fifth Street and Third Street); and 2F-03-8 (in the parking lot north of Downtown Joes). Numerous shallow exploration logs at the Napa Mill, many conducted for an environmental assessment, were obtained from Raney Geotechnical. Two boring logs for the construction of the Third Street Bridge (B-3 and B-4) were obtained from AGS, Inc.

EM 1110-2-1913 states that Phase 1 spacing for borings usually varies from 200 to 1,000 ft. In Phase 2, additional locations of borings are selected based on Phase 1 results. *EM 1110-2-1913* also states that borings should be drilled to depths at least equal to the height of the proposed levee at its highest points but not less than 10 ft. The level of investigation is compliant with a Phase 2 exploration and testing parameters described in *EM 1110-2-1913*.

4.1.2 Slope Stability

The global slope stability of the dual-wall floodwall system was evaluated with the UTEXAS4 computer program using a composite section of the tallest walls combined with the weakest subsurface soil profile. The conditions evaluated were end of construction, long term with no flood, long term with the design flood, and earthquake. The rapid drawdown condition was not evaluated because rapid drawdown conditions will not develop at the floodwalls. The concrete walls will block river water infiltration into the soils behind the walls and, as stated previously, the floodwalls have low differential head, no highly pervious subsurface soil layer, and short duration flood events. The Corps of Engineers has no criteria for global stability of retaining/flood walls, so the criteria for levees and for base sliding of flood walls was used to evaluate the factors of safety. The levee slope stability factor of safety criteria is given in EM 1110-2-1913, Design and Construction of Levee, dated 30 April 2000. The base sliding factors of safety criteria are given in EM 1110-2-2502, Retaining and Flood Walls, dated 29 September 1989. The levee and base sliding criteria have not changed since the floodwalls were designed. The following table shows the slope stability factors of safety for the floodwalls. The lateral loading on the piles was evaluated by the civil/structural A/E firm using the LPILE computer program. The LPILE analysis confirmed that the piles can withstand the lateral loads without a shearing failure and without sufficient deformation to negatively impact the axial capacity of the piles.

Table 4-1: Results of Slope Stability Analysis

Condition	F.S. (Calculated)	Minimum F.S. (Base Sliding)	Minimum F.S. (Flood Control Levee)
End of Construction	1.89	1.33	1.3
Long Term	2.65	1.5	None Listed
Long Term w/ Flood	4.80	1.5	1.4
Earthquake	1.22	1.1	None Listed (1.1 Typically Used)

4.1.3 Seismic

ER 1110-2-1806 outlines current USACE seismic design practice. There are three levels of design earthquakes and ground motions mentioned in *ER 1110-2-1806*:

- Maximum Credible Earthquake (MCE) is the greatest earthquake that can reasonably be expected to be generated by a specific source on the basis of seismological and geological evidence. The MCE is determined by a Deterministic Seismic-Hazard Analysis (DSHA).
- Maximum Design Earthquake (MDE) is the maximum level of ground motion for which

a structure is designed or evaluated. The associated performance requirement is that the project performs without catastrophic failure, although, severe damage or economic loss may be tolerated. For critical features, the MDE is the same as the MCE. For all other features, the MDE shall be selected as a lesser earthquake than the MCE that provides economical designs meeting appropriate safety standards. *EM 1110-2-2100* describes this earthquake as generally having a 10% probability of exceedance in 100 years, or a 950-year return period.

- Operating Basis Earthquake (OBE) is an earthquake that can reasonably be expected to occur within the service life of the project, that is, with a 50% probability of exceedance within its service life of 100 years (a 144-year return period). The associated performance requirement is that the project should function with little or no damage, and without interruption of function. The OBE is determined by a PSHA.

A reevaluation of the seismic design criteria might be required if a modification to a project feature also changes the loading of the same project feature or if it would change the normal water surface elevation. If no changes occur, a reevaluation of the seismic design criteria is recommended every third PI or every 15 years, whichever comes first. The seismic design criteria in the *DDR* (USACE 2011) is within recommended timeframe shown in *ER 1110-2-1806* and seismic events (e.g. 6.0 magnitude earthquake on August 24, 2014) after construction should be evaluated during the next reevaluation phase.

4.1.4 Bearing Capacity

The bearing capacity of the shallow foundation of the upper wall (T-wall) was analyzed in accordance with EM 1110-1-1905, Bearing Capacity of Soils, dated 30 October 1992. A Factor of Safety of 3.0 was used to determine the allowable bearing capacity. EM 1110-2-1905 gives a minimum factor of safety of 2.0 for cohesionless soils and 3.0 for cohesive soils. EM 1110-2-2502, Retaining and Flood Walls, dated 29 September 1989 gives different minimum bearing capacity factors of safety for various loading conditions, but the maximum listed is 3.0. The shallow foundation bearing capacity criteria have not changed since the floodwall was designed.

4.1.5 Cast-in-Drilled-Hole (CIDH) Pile Axial Capacity

Axial capacity of the CIDH piles on which the lower wall was founded was calculated using 4 different references (EM 1110-1-1905, Bearing Capacity of Soils, dated 30 October 1992; Federal Highway Administration FHWA-IF-99-025 Drilled Shafts: Construction Procedures and Design Methods, dated August 1999; NAVFAC DM 7.2, Foundations and Earth Structures, dated September 1986; and Engineering Manual for Drilled Shafts, Virginia Polytechnic Institute and State University, August 1992.). The four calculated capacities were averaged for the design ultimate capacity. A Factor of Safety of 3.0 was used to determine the allowable axial capacity. All references used recommend a Factor of Safety of 3.0 for axial capacity when the capacity is not checked with a pile load test. The Federal Highway Administration Manual has been updated since the CIDH piles were designed, but the axial capacity factor of safety is unchanged.

4.1.6 Liquefaction

Most of the SPT N-values obtained in the sand and gravel layers are above 30, indicating the soils are extremely unlikely to liquefy during an earthquake. A few zones of lower SPT N-values do exist. A liquefaction analysis using the simplified procedure of Seed and Idriss was conducted. Results are given in Appendix 2 of the DDR. This analysis showed there is no potential for liquefaction in the sand and gravel layers in the project area.

4.1.7 Sliding and Overturning

For the floodwalls resting on soil, an overturning factor of safety of 1.5 was maintained for all of the load cases. For the sliding analyses, a minimum FOS of 1.3 was employed for the flood cases, 1.5 was instituted for the non-flood events, and a 1.1 minimum was the standard for seismic events. The maximum allowable bearing capacity of 2 ksf was not exceeded, even though the allowable bearing capacity amplifications permitted in seismic loading conditions were excluded. All of these factors met or exceed those required from EM 1102-2-2502. Reference Structural Design Calculations Contract Number DACW05-01-D-0011 (MGE 2005).

4.2 **Structural**

A *Final Supplemental General Design Memorandum* dated October 1998, along with a document entitled *Soldier Pile and Sheet Pile Wall Load Conditions & Load Diagrams*, provided the structural design criteria for the Hatt Building to First Street segment of the Napa Valley Flood Control Project floodwalls. These documents appear to be project specific interpretations of *Engineering Manual (EM) 1110-2-2502, Retaining and Flood Walls*. In general, EM 1110-2-2502 instructs the designer to select applicable load cases from the following conditions: the design flood event, a typical non-flood event with water on the unprotected side of the floodwall, a seismic event and the construction event. All of these load cases are considered for the project floodwalls. For the floodwalls anchored to CIDH piles, the criterion set forth in *EM 1100-2-2100, Stability Analysis of Concrete Structures* was applied.

4.2.1 Concrete Structures

Flood protection for the Hatt to 1st Street Floodwall segment is provided by a combination of floodwalls, concrete walkways, concrete planter areas and a stop-log closure.

4.2.1.1 Concrete Strength

EM 1110-2-2007 states that concrete shall have a minimum compressive strength (f'_c) of 3,000 pounds per square in. (psi) at 28-days. Section 2 of the *DDR* (MGE 2005) shows that the concrete strength used was 4,000 psi. The concrete compressive strength meets the current design criteria.

4.2.1.2 Reinforcing Steel Strength

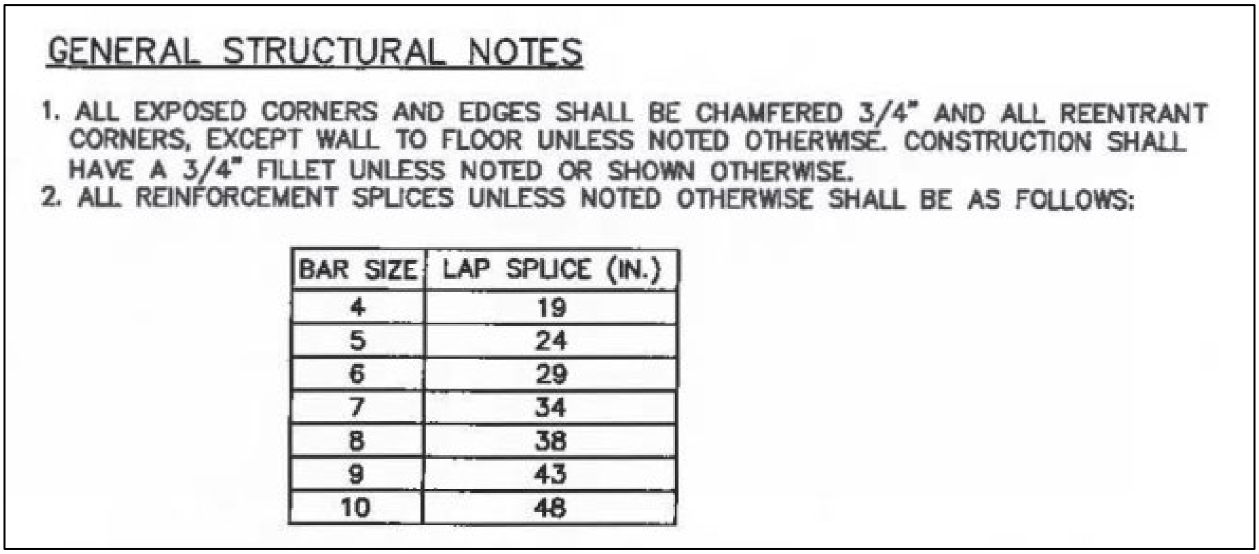
EM 1110-2-2104 & *EM 1110-2-2007* state that reinforcing steel should be limited to ASTM A615 (Billet Steel), Grade 60. The detailed calculations in the Structural Design Calculations Contract Number DACW05-01-D-0011 (MGE 2005) show that a steel yield strength of 60,000 psi was used for the design of the reinforced concrete structures in accordance with the current requirements recommended in *EM 1110-2-2104* & *EM 1110-2-2007.*

4.2.1.3 Temperature and Shrinkage Reinforcing

EM 1110-2-2104 states that the area of temperature and shrinkage reinforcement steel should be at least 0.003 times the gross cross-sectional area of the concrete, with half in each face. Generally, temperature and shrinkage reinforcement for thin sections should be no less than the equivalent of #4 bars spaced at 12 in. on center. The as built construction drawings for Hatt to 1st Street Floodwall show that sufficient reinforcement-to- concrete area proportion was provided to ensure the concrete will be well confined and to prevent excessive temperature and shrinkage cracks. The maximum spacing observed for the #5 reinforcement bars was 12 in. and meets the required design criteria specified in the *EM 1110-2-2104*.

4.2.1.4 Splices for Reinforcement

Figure 4-1 below was taken from the Hatt to 1st Street Floodwall As-Built drawings and shows splice values for different bar sizes used in construction.



GENERAL STRUCTURAL NOTES

1. ALL EXPOSED CORNERS AND EDGES SHALL BE CHAMFERED 3/4" AND ALL REENTRANT CORNERS, EXCEPT WALL TO FLOOR UNLESS NOTED OTHERWISE. CONSTRUCTION SHALL HAVE A 3/4" FILLET UNLESS NOTED OR SHOWN OTHERWISE.
2. ALL REINFORCEMENT SPLICES UNLESS NOTED OTHERWISE SHALL BE AS FOLLOWS:

BAR SIZE	LAP SPLICE (IN.)
4	19
5	24
6	29
7	34
8	38
9	43
10	48

Figure 4-1: Lap Splice Lengths

4.2.1.5 Hooks and Bends

EM 1110-2-2104 states that all hooks and bends should follow the guidelines provided in *ACI318*. The general notes on the structural detail as-built drawing (USACE 2008 [sheet S- 100]) indicates that all hook lengths are per *ACI standards*, which indicates that the hooks and bends meet the current requirement.

4.2.1.6 Bar Spacing

EM 1110-2-2104 states that the minimum clear distance between parallel bars should not be less than 1-1/2 times the nominal diameter of the bars nor less than 1-1/2 times the maximum size of coarse aggregate. The Maximum center-to-center spacing of both primary and secondary reinforcement shouldn't exceed 18 in. Structural details of the reinforcement bars shown on the as-built drawings (USACE 2008) indicate that all parallel bars were spaced with a minimum clearance of 6 in and a maximum of 18 in.

4.2.1.7 Minimum Reinforcement Cover

EM 1110-2-2104 and *EM 1110-2-2007* state that reinforcement should be placed in such a manner that the steel will have a minimum cover of 3 in. *EM 1110-2-2007* further expands for paving subjected to high-velocity flow or heavy sand scouring should be increased to provide 4 in. of clear cover. The minimum reinforcement cover utilized in the Hatt to 1st Street Floodwall structural features compared against the current minimum reinforced cover required in *EM 1110-2-2104*, *EM 1110-2-2007* and *ACI 318-08* are shown in Table 4-4.

Table 4-2: Minimum Reinforcing Concrete Cover

Requirement type	Current Design Criteria	System Documentation
Unformed surfaces in contact with foundation.	4 in.	The floodwall footing has a 4 in. clear cover per the as-built drawings (sheet S-107). This meets the current design criteria.
Formed and screened surfaces such as stilling basin walls, chute spillway slabs, and channel lining slabs on grade;	4 in.	N/A
Equal or greater than 24 in. of thickness		
Greater than 12 in. and less than 24 in. of thickness	3 in.	N/A
Equal or less than 12 in. of thickness	Per *ACI 318*, min of 2 in.	No clear cover less than 2 in. was provided on all concrete structures according to the details on the as-built drawings, consistent with the current design criteria.

4.2.1.8 Minimum Thickness of Walls

EM 1110-2-2502 and *EM 1110-2-2007* state, “The top thickness of the stem for a cantilever wall or concrete walls more than 8 ft tall and for the base slab should be a minimum of 12 in. to facilitate concrete placement.” Floodwalls more than 8 ft tall have a base slab thickness between 12 in. and 20 in. and are compliant with the current design criteria. All the footings are 12 in. thick or greater per the as-built drawings (USACE 2008 [sheets S-107, S-110, and S-110A]).

4.2.1.9 Seismic Design

In accordance with the requirements in *EM 1110-2-2104* and *ER 1110-2-1150*, seismic loading was considered during the design. For each wall height, the design assumed backfill to be at the final elevation with Wall 1 when the earthquake loading was applied. The detailed calculations in Structural Design Calculations (100% Submittal (MGE 2005) show that the seismic force was applied to the land side of the floodwall face.

4.2.2 Floodwall Joint

EM 1110-2-2502 states that expansion joints are needed to prevent spalling, displacement, buckling and warping and sometimes to break continuity between two monolith structures with different configurations. Per the as-built drawings (USACE 2008 [sheet S-100]), expansion joints with pre-molded joint fillers were provided between the floodwalls that have different height and depth configurations consistent with the requirements. The reinforcement bars were discontinued at the joint and polyvinyl chloride waterstops with sealants were also provided per the requirement of the *EM 1110-2-2502*. *EM 1110-2-2502* also requires that contraction joints be provided to regulate cracking and be spaced at a minimum from 20 to 30 ft apart. The contraction joints along the floodwalls were spaced at 24 ft intervals per the as-built drawings and are in accordance with the current design criteria.

4.2.3 Subdrainage Structures

Per *EM 1110-2-2502*, all inland floodwalls should be provided with a landside toe drain. The details on the structural as-built show floodwall toe drainage was provided.

4.3 **Hydrologic/Hydraulic**

4.3.1 Design Capacity

The Napa River/Napa Creek Flood Protection Project, which includes the Hatt to 1st Street Floodwall System, is designed to provide protection to the city of Napa for the 1% annual chance of exceedance event. The current design-flood peak discharge for projects is based on the project-specific National Economic Development plan, as specified in *ER 1105-2-100*. Section 4.5 of the Napa River/Napa Creek Flood Protection Project OMRR&R Manual contains a table showing a 1%-event discharge of 42,410 cfs for the reach of Napa River that includes the Contract 2W Floodwall .

4.3.2 Hydraulic Analysis

According to the Napa River/Napa Creek Flood Protection Project OMRR&R Manual, the amount of distance between the top of the floodwall and the design profile (design profile distance) to be provided on levees, floodwalls, and incised channels was determined based on the uncertainties inherent in the water surface profile computations. The design profile distance adheres to USACE ETL 1110-2-299, “Overtopping of Flood Control Levees and Floodwalls” for providing superiority. For the Napa Project, the design profile distance was set above the base water surface profile based on an increase in the hydraulic head loss parameters. The design profile distance assumed channel configurations, sediment deposition, and bridge debris loading.

An analysis was conducted in August 2008 to evaluate the Napa River/Napa Creek Flood Protection Project’s ability to contain the 1% event based on risk and uncertainty methods, in order to meet FEMA certification requirements. The memorandum for record for this analysis stated that the top of feature profile was developed using the superiority concept after the sponsor expressed the desire to have low floodwalls and levees while still having a high performing flood control project. The top of protection profiles was set based on the superiority concept with a minimum of freeboard of 2.0 feet. (EM 1110-2-2502, “Retaining and Flood Walls,” recommends freeboard default values of 2 feet on agricultural and 3 feet on urban flood walls.)

Current USACE guidance provided in ER 1105-2-101, "Risk Analysis for Flood Damage Reduction Studies" (USACE, 2006), states that all flood damage reduction studies will adopt risk analysis. The Napa River/Napa Creek Flood Protection project had a waiver from the requirement to do risk analysis at the time the SGDM (USACE, 1998) was prepared and the initial portions of the project were designed and constructed (2000 to circa 2006). That waiver was removed and the August 2008 analysis evaluated the Project's 1% event performance with risk and uncertainty taken into consideration, and concluded that based on information available at the time, floodwall and levee features built to date met minimum top of feature elevation for FEMA certification. Index Point 3, at Main Street Landing (River Station 769+00) had a 1% event conditional nonexceedance probability (CNP) of 96.1% with 2.7 feet of freeboard, while Index Point 4, upstream of the Third Street Bridge (Station 773+00), had a 1% CNP of 98.32% with 3.3 feet of freeboard. Both locations meet the National Flood Insurance Program levee system evaluation requirements for 1% annual chance exceedance flood assurance specified in EC 1110-2-6067, "USACE Process for the National Flood Insurance Program (NFIP) Levee System Evaluation." (USACE, 2010).

Appendix H, Section 4.1 of the OMRR&R Manual states that the high tide elevation within the floodwall project limits is approximately 3.77 feet NGVD29, well below all floodwall improvements, and that tidal influence is not expected to have significant impact on the performance of the floodwall improvements. The project, however, has not been evaluated for sea-level change in accordance with EC 1165-2-212, "Sea-Level Change Considerations for Civil Works Programs." (USACE, 2011)

The floodwalls have been constructed with an independent subdrain and surface drain system. The subdrain system was designed to relieve hydrostatic pressure on the landside of the floodwall, while various surfaces have been designed to allow efficient runoff collection within the surface drainage system. Detailed descriptions of the drainage facilities associated with the Contract 2W Floodwall is provided in Appendix H of the OMRR&R Manual, under Sections 8.6 and 8.7.

4.4 **Survey Datum**

The floodwalls were surveyed during construction for measurement and payment purposes and that survey is reflected in the as-built drawings. The NGVD 29 vertical datum was used for the design and construction of this segment. A survey to determine the conversion between NGVD29 and NAVD88 datums has not been completed as required in *ER 1110-2-8160 Policies for Referencing Project Evaluation Grades to Nationwide Vertical Datums*

4.5 **Design Criterial Review Conclusions**

Based on the findings of the design criteria review, each feature and the overall system appear to be able to function as originally authorized.

PART 5 - INSPECTION FINDINGS AND EVALUATIONS

The PI was conducted on 22 July 2020. Table 5-1 shows the key team members and the role each assumed during the PI. The inspection team lead was Mr. Michael Franssen.

Table 5-1: List of Key Inspection Staff

Title	Name
Local Sponsor Representative (NCFCWCD)	Jeremy Sarrow
Civil/Team Lead (USACE Walla Walla District)	Michael Franssen, PE
Geotechnical/LSPM (USACE San Francisco District)	John Conway, PG
Civil Technician (USACE Walla Walla District)	Nathan DeLannoy

5.1 **Inspection Summary**

An overall summary of the PI ratings is shown in Table 5-2. Specific detailed related to acceptable, minimally acceptable, and unacceptable rated items are discussed in the subsequent sections.

5.2 **General Items for All Flood Damage Reduction Segments/Systems**

A summary of the rated items contained in the checklist titled "General Items for All Flood Damage Reduction Segments/Systems" is shown in Table 5-2. The following subsections provide additional detail on these items.

5.2.1 Operation and Maintenance Manuals

The operation and maintenance (O&M) manual for the Napa River / Napa Creek Flood Protection Project was made final in April 2018 by USACE Sacramento District and provided to NCFCWCD. The Hatt to 1st Street Floodwall System is a component of the Project.

5.2.2 Emergency Supplies and Equipment

NCFCWCD maintains a supply of empty sandbags, stockpile sand, chain saws, various hand tools, and other emergency supplies at the maintenance yard located on 933 Water Street in Napa, CA. The majority of sand that would be used for sands bags is stored at 770 Jackson Street in Napa, CA. Both of these locations are within 1.5 miles of the Levees. NCFCWCD has emergency contracts with general contractors when emergency services are needed. NCFCWCD informed the inspection team that the location on 933 Water Street may be bought out or leased to an external organization in the near future.

5.2.3 Flood Preparedness and Training

NCFCWCD has developed a flood emergency operation plan. Annual flood fight training program is conducted by the California Department of Water Resources at the Napa Sheriff's Department each fall. NCFCWCD has previously attended the USACE San Francisco District's Levee Owner Workshop in Sausalito, CA.

5.3 Concrete Floodwall

A summary of the rated items contained in the checklist titled “Floodwalls” is shown in Table 5-2. The following subsections provide additional detail on these items. Items listed as non-applicable (NA) in Table 5-2 are not included in the following paragraphs.

Table 5-2: PI Rated Summary

Category	Rated Item	Rating[1]
General Items for All Flood Damage Reduction Segments/Systems	1. Operation and Maintenance Manuals	A
	2. Emergency Supplies and Equipment	A
	3. Flood Preparedness and Training	A
Floodwalls	1. Non-Compliant Vegetation Growth	A
	2. Encroachments	A
	3. Closure Structures (Stop Log Closures and Gates)	A
	4. Concrete Surfaces	A
	5. Tilting, Sliding or Settlement of Concrete Structures	A
	6. Foundation of Concrete Structures	A
	7. Monolith Joints	A
	8. Underseepage Relief Wells/Toe Drainage Systems	A
	9. Seepage	A
Interior Drainage System	1. Vegetation and Obstructions	M
	2. Encroachments	A
	3. Ponding Areas	NA
	4. Fencing and Gates	NA
	5. Concrete Surfaces	A
	6. Tilting, Sliding or Settlement of Concrete and Sheet Pile Structures	A
	7. Foundation of Concrete Structures	A
	8. Monolith Joints	A
	9. Culvert/Discharge Pipes	A
	10. Sluice/Slide Gates	NA
	11. Flap Gates/Flap Valves/Pinch Valves	A
	12. Trash Racks	NA
	13. Other Metallic Items	NA
	14. Riprap Revetments of Inlet/ Discharge Areas	NA
	15. Revetments other than Riprap	NA

[1] Note: Acceptable (A), Minimally Acceptable (M), Unacceptable (U), Not Applicable (NA)

5.3.1 Non-Compliant Vegetation Growth

This item was rated “acceptable”. The floodwall project is maintained very well with only minor grass and small plant type vegetation noted in the riprap observed during the inspection.

5.3.2 Encroachments

This item was rated “acceptable”. Access to a city dock on the river side and landscape anchors in the wall were both noted during the inspection. The anchors support ivy plants along the base of the upper wall which were installed during construction.

5.3.3 Closure Structures

This item was rated “acceptable”. No action required at this time

5.3.4 Concrete Surfaces

This item was rated “acceptable”. Spalling was observed on concrete floor. Minor spall has no bearing on the integrity of the floodwall.

5.3.5 Tilting, Sliding or Settlement of Concrete Structures

This item was rated “acceptable”. No tilting, sliding, or settlement of the concrete floodwall was observed during the PI.

5.3.6 Foundation of Concrete Structures

This item was rated “acceptable”. No foundation concerns were observed during the PI.

5.3.7 Monolith Joints

This item was rated “acceptable”. Expansion and contraction joints were in good condition.

5.3.8 Underseepage Relief Wells/ Toe Drainage Systems

This item was rated “acceptable”. The drain system was in good condition with no signs of corrosion, deterioration or any blockages to prevent water from landside floodwall to Napa River.

5.3.9 Seepage

This item was rated “acceptable”. No seepage concerns were observed during the PI.

5.4 Interior Drainage System

A summary of the rated items contained in the checklist titled “Interior Drainage System” is shown in Table 5-2. The following subsections provide additional detail on these items. Items listed as non-applicable (NA) in Table 5-2 are not included in the following paragraphs.

5.4.1 Vegetation and Obstructions

This item was rated “minimally acceptable”. Plantings that were observed on the PI were part of the original construction contract of the levee and have minimal risk the integrity of the floodwall. Grasses and small plants are present in the riprap at the toe of the floodwall. This has been removed in the past and should be monitored and controlled to prevent establishment of trees.

5.4.2 Encroachments

This item was rated “acceptable”. All landside structures have been approved and pose no threat to the floodwall.

5.4.3 Concrete Surfaces

This item was rated “acceptable”.

5.4.4 Tilting, Sliding or Settlement of Concrete and Sheet Pile Structures

This item was rated “acceptable”. No tilting, sliding or settlement of concrete floodwall was observed during PI.

5.4.5 Foundation of Concrete Structures

This item was rated “acceptable”. No foundation concerns were observed during the PI.

5.4.6 Monolith Joints

This item was rated “acceptable”. No monolith concerns were observed during the PI.

5.4.7 Culverts/ Discharge Pipes

This item was rated “acceptable”. No culvert obstructions, breaks or cracks were observed during the PI.

5.4.8 Flap Gates/ Flap Valves/ Pinch Valves

This item was rated “acceptable”. Sponsor indicates the flap gates and pinch valves are exercised twice a year. Gates all appeared to be in good order during the inspection.

PART 6 - CONCLUSIONS AND RECOMMENDATIONS

This section summarizes items that received either "minimally acceptable" or "unacceptable" ratings for each feature of the Hatt to 1st Street Floodwall System, and it includes the recommended actions for each of these items. A discussion of levee safety issues and a summary of the needs related to the design criteria review follow the inspection recommendations.

7.1 **Recommendations**

7.1.1 General Items for All Flood Damage Reduction Segments/Systems

All of the General Items for All Flood Damage Reduction Segments/Systems items received an "acceptable" rating.

7.1.2 Concrete Floodwall

All of the Concrete Floodwall items received an "acceptable" rating.

7.1.3 Interior Drainage System

The only item that received a minimally and/or unacceptable rating was Vegetation and Obstructions, which received a rating of "minimally acceptable". Plantings that were observed on the PI were part of the original construction contract of the levee and have minimal risk to the integrity of the floodwall. Grasses and small plants are present in the riprap at the toe of the floodwall. This has been removed in the past and should be monitored and controlled to prevent establishment of trees.

7.2 **Rating**

The overall rating of the Hatt to 1st Street Floodwall System is "minimally acceptable".

7.3 **Future Periodic Inspection**

The next PI of the Hatt to 1st Street Floodwall System should be at 5 years from the levee screening to take place in 2021.

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Appendix A

Pertinent Plates and Drawings

US Army Corps
of Engineers
Sacramento District

NAPA RIVER / NAPA CREEK FLOOD PROTECTION PROJECT
CONTRACT 2 WEST
(BETWEEN HATT BUILDING & FIRST STREET)

NAPA
CALIFORNIA

AS-BUILT

CONTRACT NO.: W91238-05-R0024

This project was designed by the Sacramento District of the U.S. Army Corps of Engineer. The initials or signatures and registration designations or individuals appear on these project documents within the scope of their employment as required by ER 1110-1-8152

Approved
/S/ THOMAS E. TRAINER 04/27/2005
Chief, Engineering Division Date
Prepared Under the Direction of
COL. RONALD N. LIGHT
Col. Corps of Engineers District Engineer

Drawing Code:
Designed by
JEFF S. CROVITZ
Drawn By
KERWIN G. LI
Spec No.
1407
Design File No:
NA-25-030

Approved Functional Adequacy
/S/ RONALD F. MULLER 04/27/2005
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NAPA CALIFORNIA
NAPA RIVER/NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (HATT BUILDING
TO FIRST STREET)

TITLE SHEET

Sheet reference number:
G-001
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DRAWING INDEX

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SHEET REFERENCE NUMBER LEGEND

C-101
- DISCIPLINE SEQUENCE NUMBER
- DRAWING TYPE CODE
- DISCIPLINE CODE

DISCIPLINE CODES:

G – GENERAL
B – GEOTECHNICAL
C – CIVIL
S – STRUCTURAL

L – LANDSCAPING
P – PLANTING
I – IRRIGATION
E – ELECTRICAL

DRAWING TYPE CODE:

0 – GENERAL
1 – PLAN
2 – PROFILE
3 – CROSS SECTION
4 – LARGE SCALE PLAN
5 – DETAILS
6 – SCHEDULES AND DIAGRAMS

US Army Corps
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Sacramento District

Symbol	Description	Date	Approved
△	NEW SHEET	[illegible]	JSC
△	5TH STREET UTILITY MODIFICATIONS	[illegible]	JSC
△	REVISED SIDEWALK	[illegible]	JSC
△	REVISED STONE PROTECTION AND LOWERED WALL FOOTING	[illegible]	JSC
△	LINE REFERENCES ADDED	[illegible]	JSC
△	STONE PROTECTION REVISION	[illegible]	JSC
△	REVISED GRADING OF SIDEWALK	[illegible]	JSC

Designed by: JEFF S. CROVITZ
Dwn by: KLI
Spec No.: 1407
Reviewed by: R. SENNETT
Submitted by: /S/ J. CROVITZ
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NAPA CALIFORNIA
NAPA RIVER/NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (HATT BUILDING
TO FIRST STREET)

SHEET INDEX - 1

Sheet reference number:
G–002A
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△7

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Symbol	Description	Date	Approved
△	NEW SHEET		JSC
△	5TH STREET UTILITY MODIFICATIONS		JSC
△	REVISED SIDEWALK		JSC
△	REVISED STONE PROTECTION AND LOWERED WALL FOOTING		JSC
△	LINE REFERENCES ADDED		JSC
△	REVISED GRADING OF SIDEWALK		JSC

Designed by: JEFF S. CROWITZ
Dwn by: KJ
Spec No.: 1407
Reviewed by: R. SENNETT
Submitted by: /S/ J. CROWITZ
Date: 03/10/2006
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NAPA CALIFORNIA
NAPA RIVER/NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (HATT BUILDING
TO FIRST STREET)

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Sheet
reference
number:
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Symbol	Description	Date	Approved
1	NEW SHEET	[illegible]	JSC
2	5TH STREET UTILITY MODIFICATIONS	[illegible]	JSC
3	REVISED SIDEWALK	[illegible]	JSC
4	REVISED STONE PROTECTION AND LOWERED WALL FOOTING	[illegible]	JSC

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Spec No.: 1407
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NAPA RIVER/NAPA CREEK FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (HATT BUILDING TO FIRST STREET)
NAPA CALIFORNIA

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DRAWING INDEX

REF#	TITLE	SHEET#	AMENDMENT#1 12/23/05	AMENDMENT#2 03/10/06	AMENDMENT#3 05/30/06	VALUE ENGINEERING 09/06/06	AMENDMENT#4 11/09/06	AMENDMENT#5 12/28/06	AMENDMENT#6 03/02/07
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FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (HATT BUILDING
TO FIRST STREET)

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Sheet reference number:
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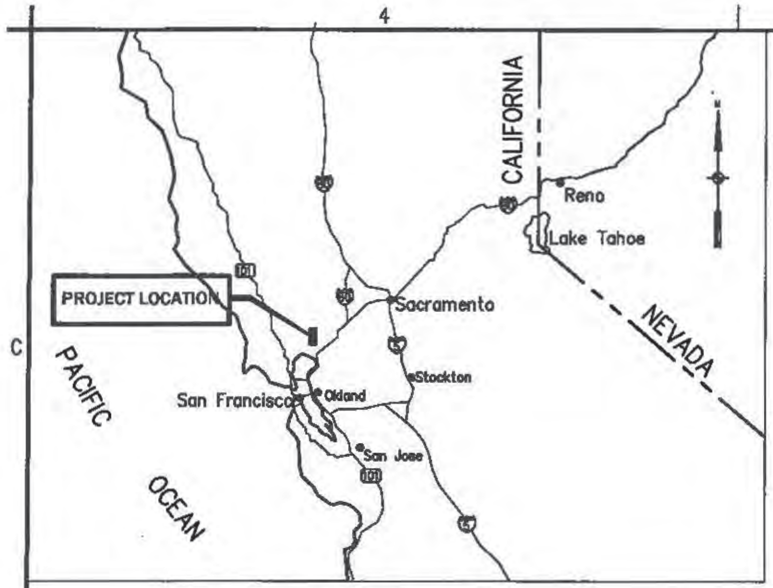
Sheet 2D of 166

AM-005

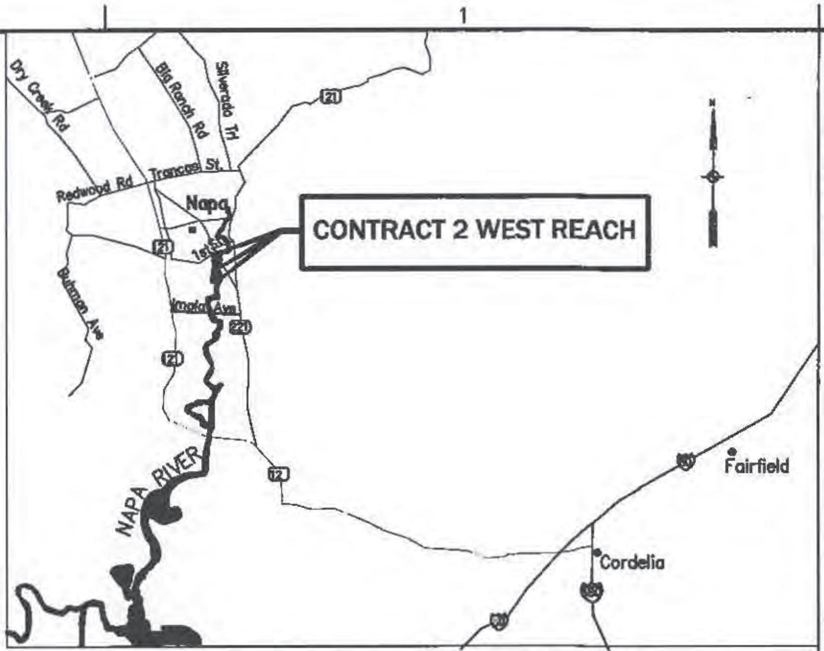


VICINITY MAP
NOT TO SCALE

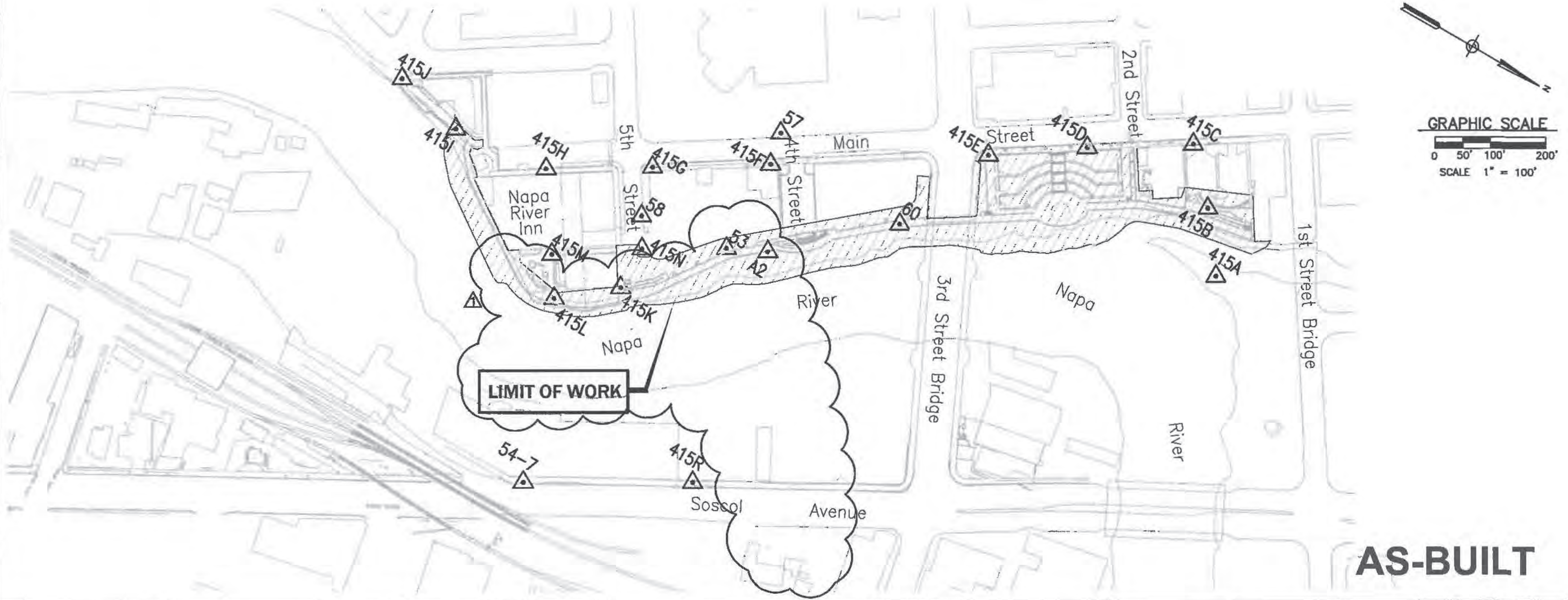
SURVEY NOTES:

1. GRID COORDINATES IN U.S. FEET REFER TO CALIFORNIA STATE PLANE ZONE 2, NAD 83 AND ARE BASED ON CALTRANS HPGN D CA 04 LG, NGS BLUE, QUARRY, AND USC&GS DRY
2. ELEVATIONS IN U.S. FEET REFER TO NGVD 29 BASED ON CITY OF NAPA 11-B, 54-C, 54-7, 54-8, 75-A, 75-C, 78-B, USE NA2, USC&GS TIDAL, AND USCE SAC.DIST. N1.
3. NA 2000 ELECTRONIC LEVEL RUN RESET 54-E HOLDING N1, 54-7, 54-8 AND CHECKED WITH NA 2, TIDAL
4. UNLESS NOTED "*" IS LEVEL WITH GROUND NAPA RIVER NAD 83 STATE PLANE ZONE 2 NGVD 29 U.S. FEET
5. STATION 53, 57, 58, AND 60 ARE FROM "CHANNEL DEVELOPMENT" SURVEYING AND REFER TO NGVD 29.

STATION	NORTHING	EASTING	MON. ELVE.	RIM ELEV.	MARKER
415A	1870918.39	6480045.40	14.13	*	SPIKE & WASHER
415B	1870845.40	6479946.95	17.97	*	SPIKE & WASHER
415C	1870765.44	6479860.26	17.49	*	SPIKE & WASHER
415D	1870604.06	6479959.42	17.50	*	SPIKE & WASHER
415E	1870458.45	6480058.43	17.08	*	SPIKE & WASHER
415F	1870127.62	6480266.42	16.17	*	SPIKE & WASHER
415G	1869945.61	6480377.45	15.55	*	SPIKE & WASHER
415H	1869782.07	6480475.05	16.68	*	SPIKE & WASHER
415I	1869607.21	6480493.30	15.42	*	SPIKE & WASHER
415J	1869477.47	6480464.04	13.42	*	SPIKE & WASHER
415K	1870003.40	6480593.20	16.45	*	SPIKE & WASHER
415L	1869910.96	6480668.80	16.91	*	SPIKE & WASHER
415M	1869867.81	6480602.55	16.27	*	SPIKE & WASHER
415N	1870002.14	6480513.95	15.39	*	SPIKE & WASHER
415R	1870290.31	6480830.14	13.43	*	SPIKE & WASHER
A2	1870201.61	6480404.95	15.85	*	SPIKE & WASHER
54-7	1870028.18	6480981.15	14.38	*	CITY OF NAPA BRASS CAP
53	1870132.55	6480437.37	15.57	*	SPIKE & WASHER
57	1870112.23	6480209.39	16.31	*	SPIKE & WASHER
58	1869972.98	6480462.57	15.48	*	SPIKE & WASHER
60	1870376.25	6480243.58	17.14	*	SPIKE & WASHER



LOCATION MAP
NOT TO SCALE



AS-BUILT

US Army Corps of Engineers
Sacramento District

Symbol	Descriptions	Date	Approved
1	UPDATES ASSOCIATED WITH WALL NO. 1 ALIGNMENT CHANGE	[illegible]	JSC

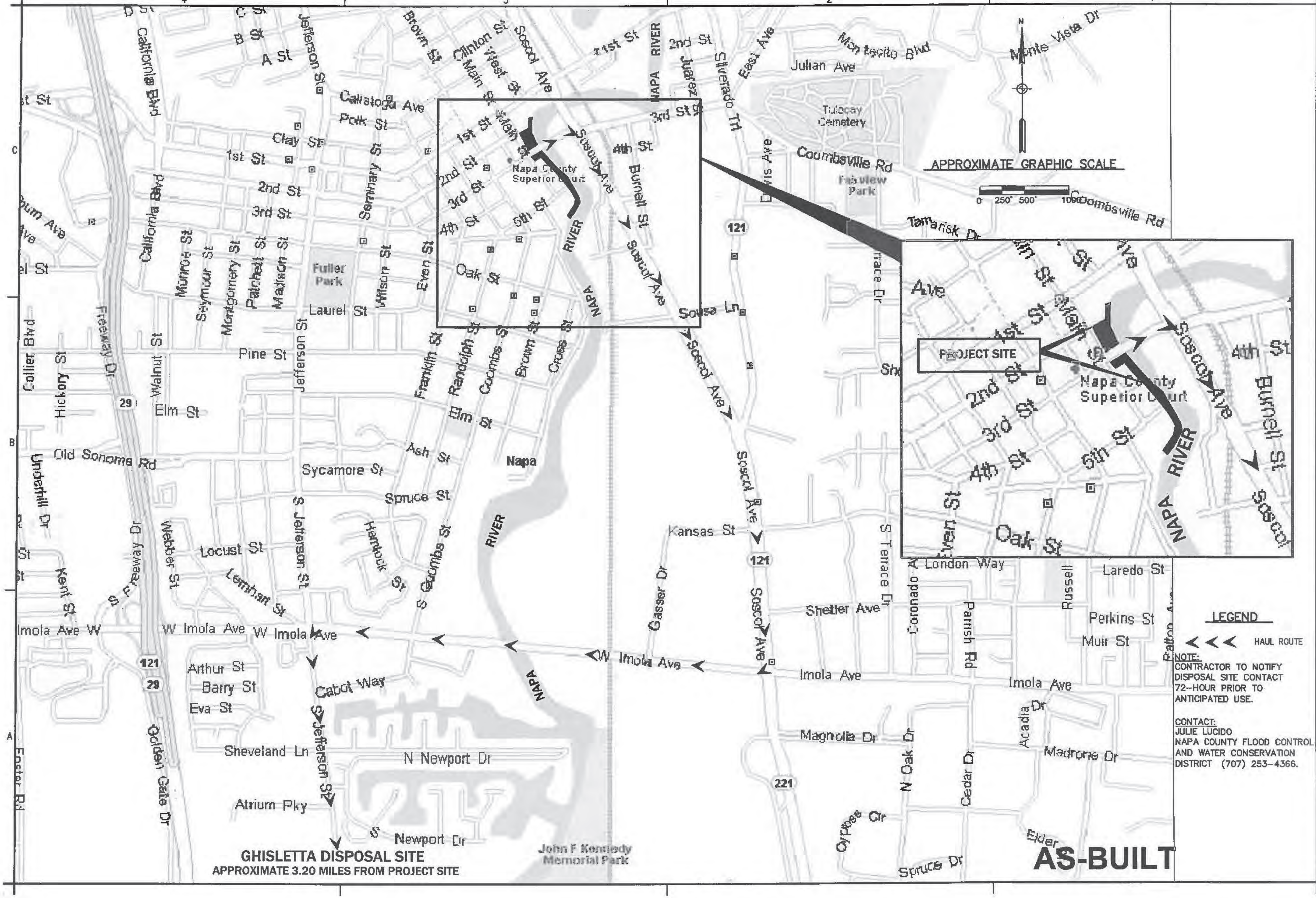
Designed by: JEFF S. CROWITZ
Date: 04/27/2005
Rev.
Dwn by: KU
Spec No.: 1407
Design file no: NA-25-030
Reviewed by: R. SENNETT
Drawing Code:
Submitted by: /S/ J. CROWITZ

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

MGE ENGINEERING INC.
7415 Greenhaven Dr. Suite 100
Sacramento, California 95831
916.421.1000

NAPA CALIFORNIA
NAPA RIVER/NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (HATT BUILDING TO FIRST STREET)
PROJECT LOCATIONS,
VICINITY MAP, SURVEY
CONTROL POINTS AND SURVEY NOTES

Sheet reference number:
G-004
Sheet 4 of 166



LEGEND

< < < HAUL ROUTE

NOTE:
CONTRACTOR TO NOTIFY DISPOSAL SITE CONTACT 72–HOUR PRIOR TO ANTICIPATED USE.

CONTACT:
JULIE LUCIDO
NAPA COUNTY FLOOD CONTROL AND WATER CONSERVATION DISTRICT (707) 253–4366.

AS-BUILT

US Army Corps of Engineers
Sacramento District

Designed by: JEFF S. CROMITZ	Date: 04/27/2005	Rev.
Dwn by: KU	Spec No.: 1407	Design file no: NA-25-030
Reviewed by: R. SENNETT		Drawing Code:
Submitted by: /S/ J. CROMITZ		File name: Plot date: Dwg scale:

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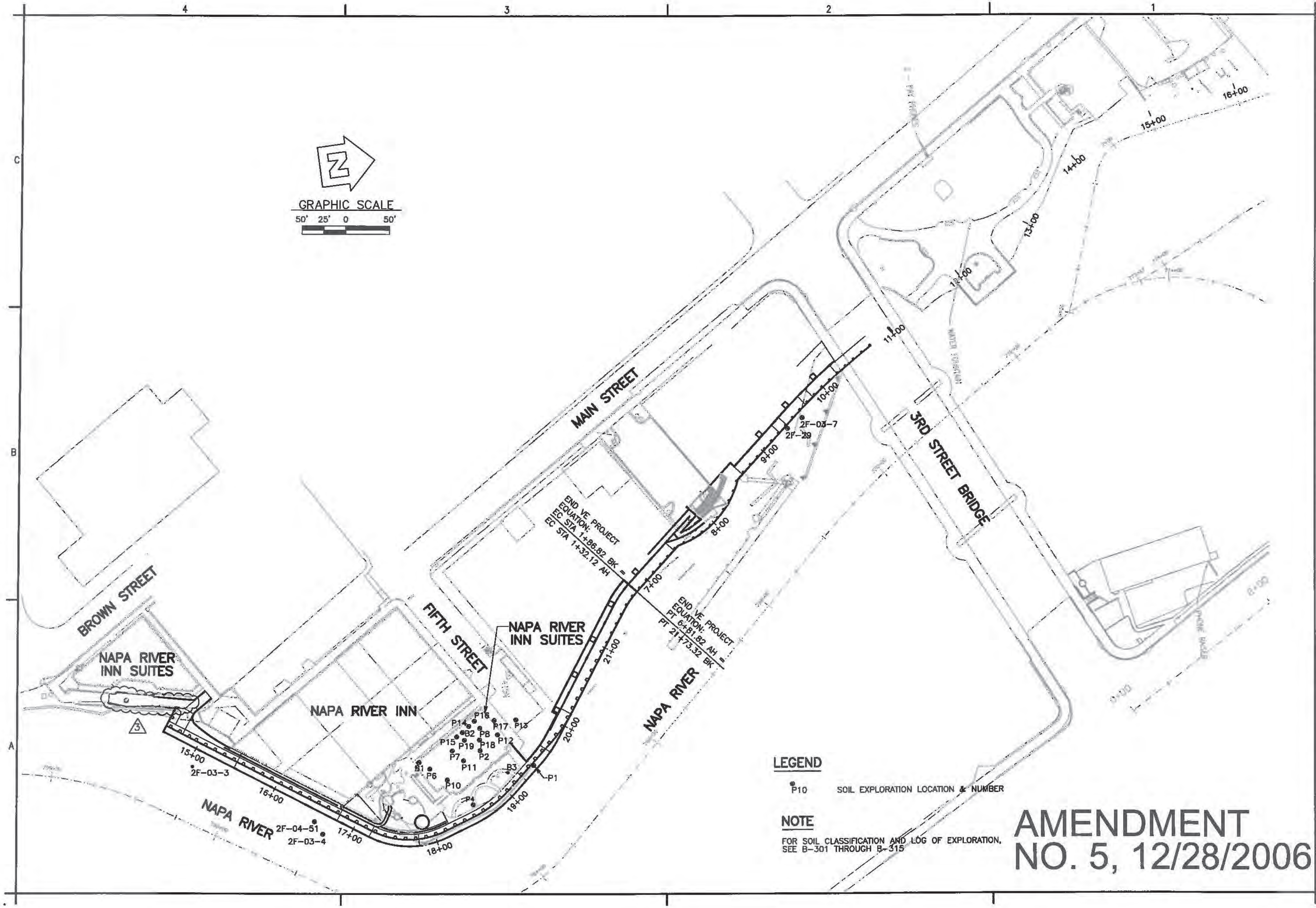
NAPA
NAPA RIVER/NAPA CREEK CALIFORNIA
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (HATT BUILDING TO FIRST STREET)

DISPOSAL SITES AND HAUL ROUTE

Sheet reference number:
G–006
Sheet 6 of 166



NOTE

FOR SOIL CLASSIFICATION AND LOG OF EXPLORATION, SEE B-301 THROUGH B-315

AMENDMENT
NO. 5, 12/28/2006

US Army Corps
of Engineers
Sacramento District

Symbol	Description	Date	Approved
3	AM/005 - REVISED SIDEWALK	JSC	12/28/06
2	REVISED STATION		
1	MOVED STATION EQUATION TO VE LIMITS		

Designed by:		Date:	Rev.
Dwn by:	Spec No.:	Design file no:	
Reviewed by:		Drawing Code:	
Submitted by:		File name: Plot date: Dwg scale:	

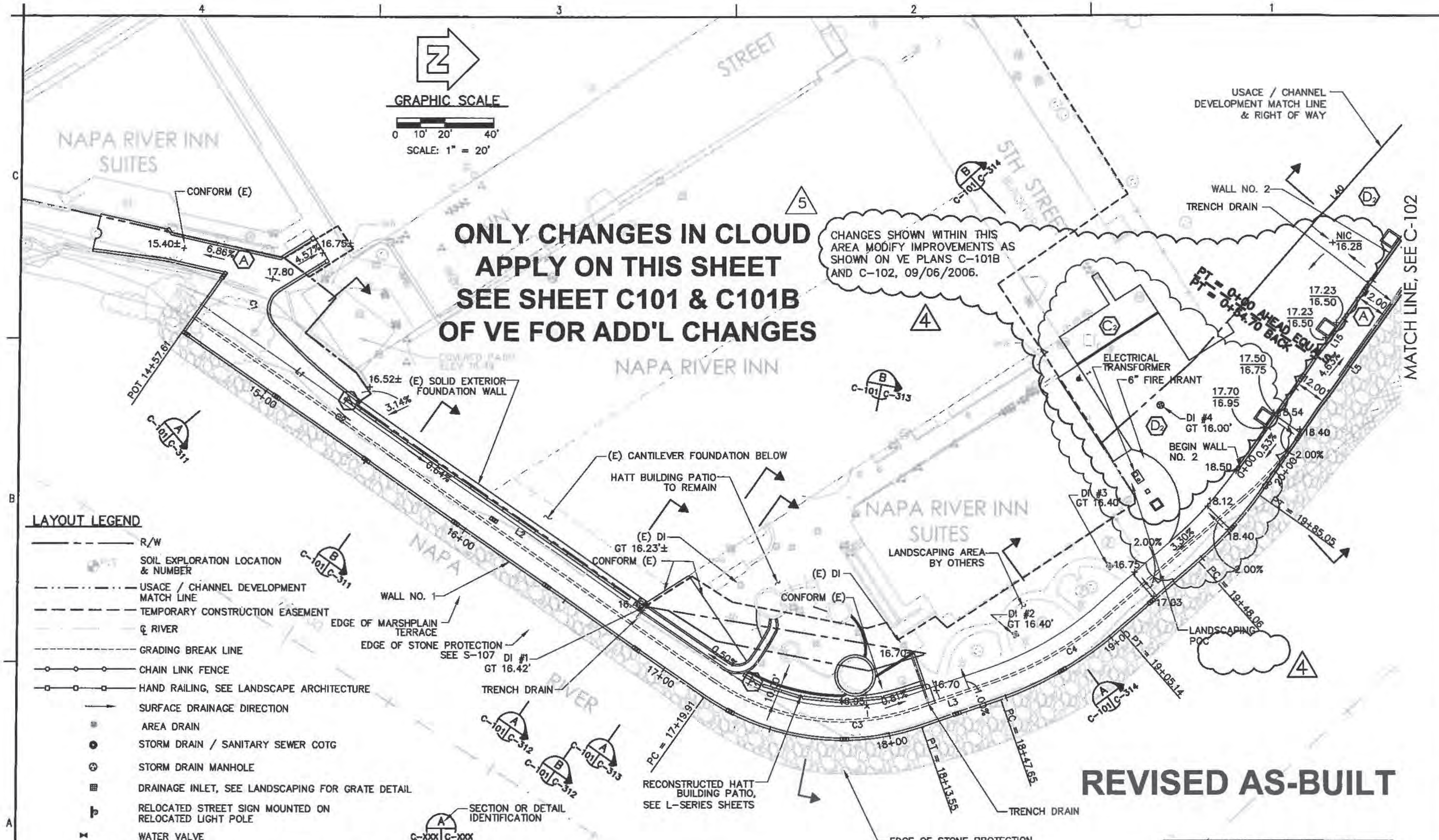
DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

BEI ENGINEERING INC.
900 East Main Street, Suite 100
Grass Valley, California 95945
530.477.9960

NAPA CALIFORNIA
NAPA RIVER/NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (BROWN STREET
TO FIFTH STREET)
(VALUE ENGINEERING PLANS)
GEOTECHNICAL EXPLORATION
BORING LOCATION PLAN

Sheet reference number:
G-008
Sheet 8 of 166

AM-005



ONLY CHANGES IN CLOUD APPLY ON THIS SHEET SEE SHEET C101 & C101B OF VE FOR ADD'L CHANGES

CHANGES SHOWN WITHIN THIS AREA MODIFY IMPROVEMENTS AS SHOWN ON VE PLANS C-101B AND C-102, 09/06/2006.

REVISED AS-BUILT

LAYOUT LEGEND

	R/W
	SOIL EXPLORATION LOCATION & NUMBER
	USACE / CHANNEL DEVELOPMENT MATCH LINE
	TEMPORARY CONSTRUCTION EASEMENT
	℄ RIVER
	GRADING BREAK LINE
	CHAIN LINK FENCE
	HAND RAILING, SEE LANDSCAPE ARCHITECTURE
	SURFACE DRAINAGE DIRECTION
	AREA DRAIN
	STORM DRAIN / SANITARY SEWER COTG
	STORM DRAIN MANHOLE
	DRAINAGE INLET, SEE LANDSCAPING FOR GRATE DETAIL
	RELOCATED STREET SIGN MOUNTED ON RELOCATED LIGHT POLE
	WATER VALVE
X 16.45	SPOT ELEVATION
15	(N) MAJOR CONTOUR
17	(N) MINOR CONTOUR
	(N) MARSHPLAIN TERRACE
	(N) STONE PROTECTION
	TRENCH DRAIN

A C-XXX C-XXX — SECTION OR DETAIL IDENTIFICATION; REFERENCE SHEET NO. FROM WHICH SECTION OR DETAIL IS TAKEN; REFERENCE SHEET NO. ON WHICH SECTION OR DETAIL IS DRAWN

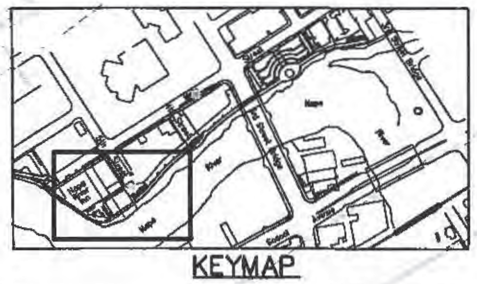
S-2 — STANDARD DWG # PER CITY OF NAPA PUBLIC WORKS DEPARTMENT STANDARD PLAN (JULY, 2001)

A — TYPICAL PAVEMENT SECTION # IN SHEET G-007

X 17.23 — ULTIMATE FG, NIC
16.48 — CONSTRUCTION PHASE I FG

LAYOUT NOTES

1. SEE C-207 THROUGH C-209 FOR FLOODWALL & PROMENADE PROFILES.
2. SEE C-210 FOR USACE / CHANNEL DEVELOPMENT MATCH LINE PROFILES.
3. SEE C-648 FOR WALL ALIGNMENTS LAYOUT DATA.
4. SEE B-301 THROUGH B-315 FOR PROJECT SOILS EXPLORATION INFORMATION.
5. SEE C-104 THROUGH C-106 FOR UNDERGROUND UTILITIES.
6. SEE C-419 THROUGH C-427 FOR DETAILS & LIMITS OF REMOVAL.
7. SEE C-547 FOR TRENCH DRAIN DETAILS.



KEYMAP

US Army Corps
of Engineers
Sacramento District

Symbol	Description	Date	Approved
△	WALL NO. 1 ALIGNMENT CHANGE	[illegible]	JSC
△	UPDATES ASSOCIATED WITH WALL NO. 1 ALIGNMENT CHANGE	[illegible]	JSC
△	REVISION PER STORM DRAIN CHANGE	[illegible]	JSC
△	5TH STREET UTILITY MODIFICATIONS	[illegible]	JSC
△	UPDATES ASSOCIATED W/ 5TH ST UTILITY MOD		

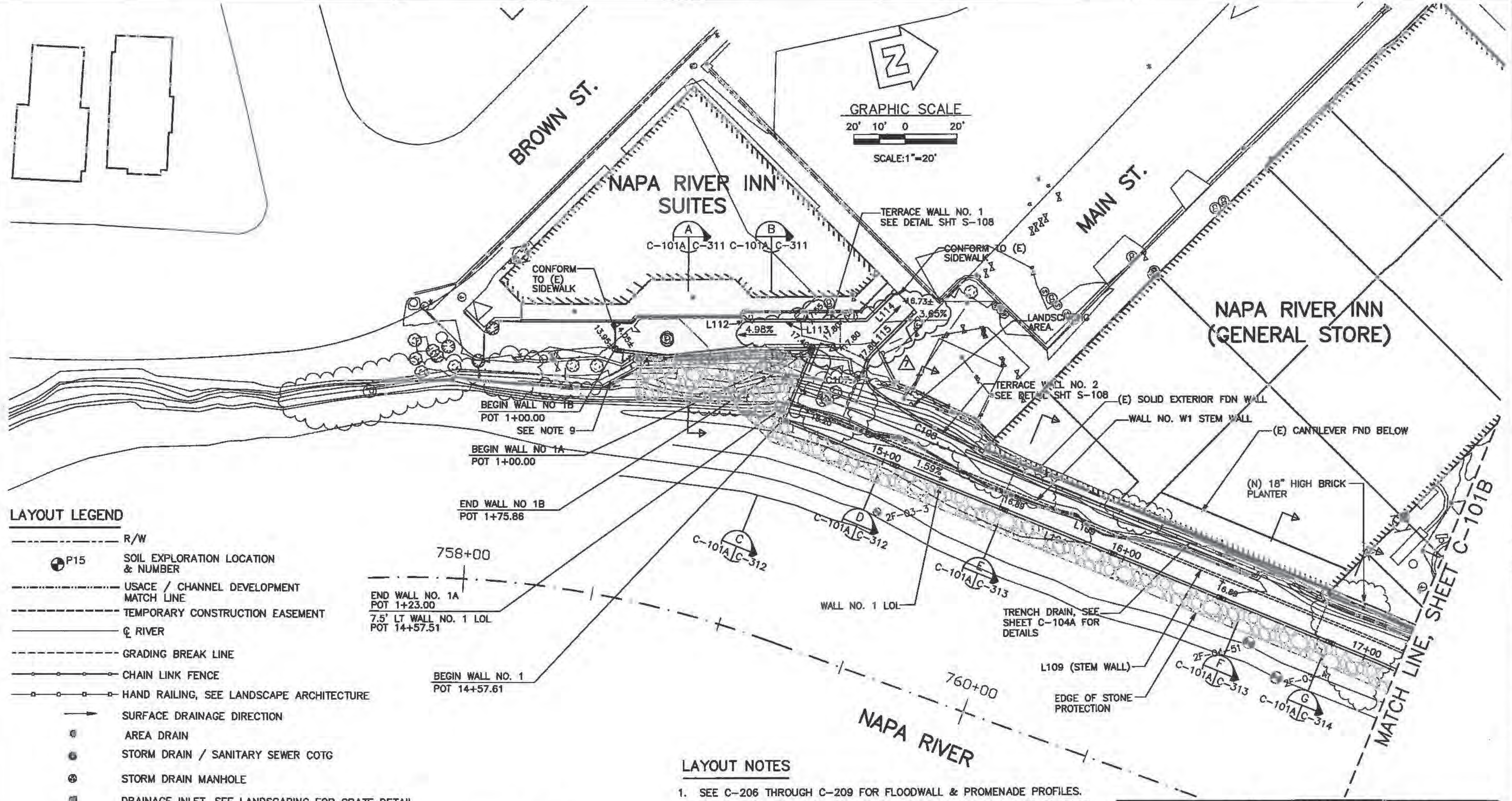
DEPARTMENT OF THE ARMY CORPS OF ENGINEERS SACRAMENTO, CALIFORNIA	Designed by: JEFF S. CROMITZ		Date: 04/27/2005
	Dwn by: KJJ	Spec No.: 1407	Design file no: NA-25-030
MGE ENGINEERING INC. 7415 Greenhaven Dr., Suite 100 Sacramento, California 95831 916.421.1000	Reviewed by: R. SENNETT		Drawing Code:
	Submitted by: /S/ J. CROMITZ		

NAPA
NAPA RIVER/NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (HATT BUILDING
TO FIRST STREET)
CALIFORNIA

LAYOUT & PROMENADE
GRADING PLAN NO. 1

Sheet reference number:
C-101
Sheet 24 of 166

AM-004



LAYOUT LEGEND

- R/W
- P15 SOIL EXPLORATION LOCATION & NUMBER
- USACE / CHANNEL DEVELOPMENT MATCH LINE
- TEMPORARY CONSTRUCTION EASEMENT
- ℄ RIVER
- GRADING BREAK LINE
- CHAIN LINK FENCE
- HAND RAILING, SEE LANDSCAPE ARCHITECTURE
- SURFACE DRAINAGE DIRECTION
- AREA DRAIN
- STORM DRAIN / SANITARY SEWER COTG
- STORM DRAIN MANHOLE
- DRAINAGE INLET, SEE LANDSCAPING FOR GRATE DETAIL
- RELOCATED STREET SIGN MOUNTED ON RELOCATED LIGHT POLE
- WATER VALVE
- X 16.45 SPOT ELEVATION
- 15 (N) MAJOR CONTOUR
- 17 (N) MINOR CONTOUR
- ▨ (N) MARSHPLAIN TERRACE
- ▨ (N) STONE PROTECTION
- TRENCH DRAIN

A C-XXX C-XXX
SECTION OR DETAIL IDENTIFICATION
REFERENCE SHEET NO. FROM WHICH SECTION OR DETAIL IS TAKEN
REFERENCE SHEET NO. ON WHICH SECTION OR DETAIL IS DRAWN

S-2 STANDARD DWG # PER CITY OF NAPA PUBLIC WORKS DEPARTMENT STANDARD PLAN (JULY, 2001)

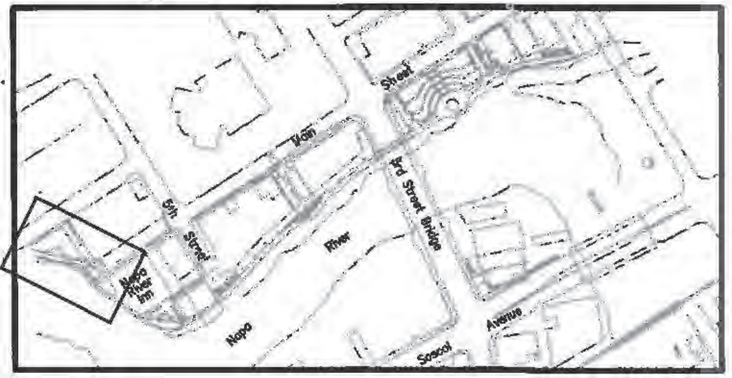
Ⓐ TYPICAL PAVEMENT SECTION # IN SHEET G-008

X 17.23 ULTIMATE FG, NIC
16.48 CONSTRUCTION PHASE I FG

LAYOUT NOTES

1. SEE C-206 THROUGH C-209 FOR FLOODWALL & PROMENADE PROFILES.
2. SEE C-210 FOR USACE / CHANNEL DEVELOPMENT MATCH LINE PROFILES.
3. SEE C-648A FOR WALL ALIGNMENTS LAYOUT DATA.
4. SEE B-301 THROUGH B-315 FOR PROJECT SOILS EXPLORATION INFORMATION.
5. SEE C-104A THROUGH C-106 FOR UNDERGROUND UTILITIES.
6. SEE C-419 THROUGH C-427 FOR DETAILS & LIMITS OF REMOVAL.
7. SEE C-547 FOR TRENCH DRAIN DETAILS.
8. SEE C-430A THROUGH C-432 FOR GRADING DETAILS.
9. EXISTING RETAINING WALL TO REMAIN AND BE PROTECTED IN PLACE. CONTRACTOR TO PROVIDE SHORING TO PROTECT THE WALL IF NECESSARY.

AMENDMENT NO. 9, 07/03/2007



KEYMAP

US Army Corps of Engineers Sacramento District

Symbol	Description	Date	Approval
△	AM/009 – REVISED GRADING OF SIDEWALK	JSC	07/03/07
△	AM/006 – REVISED STONE PROTECTION AND LOWERED WALL FOOTING	JSC	03/15/07
△	AM/005 – REVISED SIDEWALK	JSC	12/28/06
△	REVISED NOTE		
△	ADDED BRICK PLANTER		
△	CORRECTED SECTION LABEL		
△	ADDED SECTION B-B		

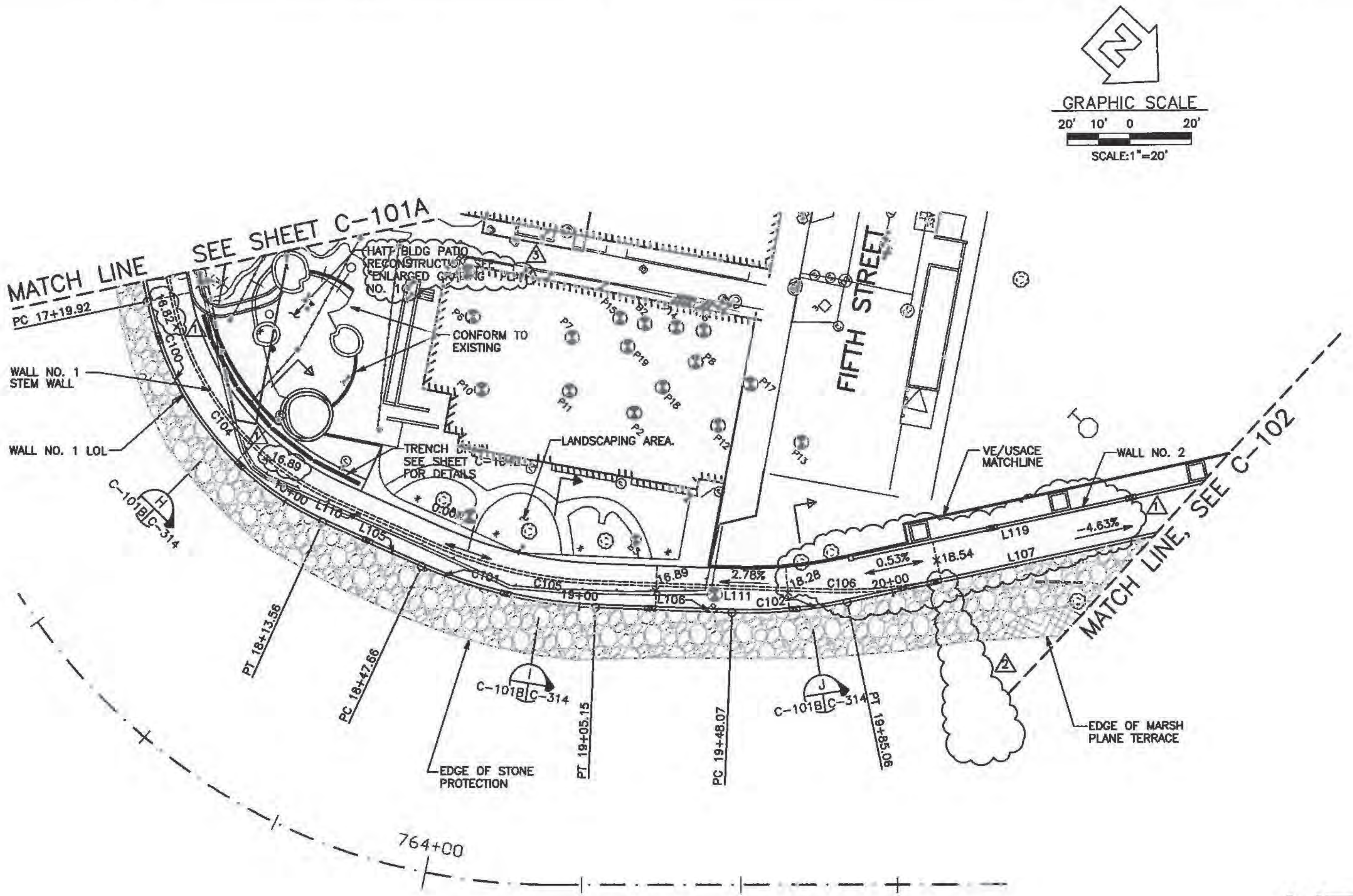
Designed by: | Date: | Rev.
Dwn by: | Spec No.: | Design file no:
Reviewed by: | Drawing Code:
Submitted by: | File name: Plot date: Plot scale:

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

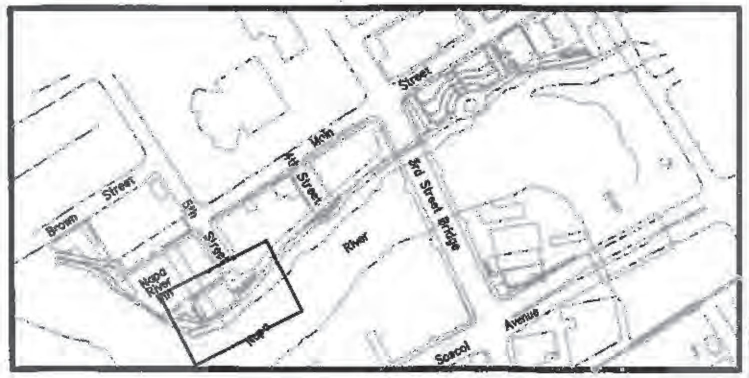
BEI ENGINEERING INC.
900 East Main Street, Suite 100
Grass Valley, California 95945
530.477.9960

NAPA CALIFORNIA
NAPA RIVER/NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (BROWN STREET
TO FIFTH STREET)
(VALUE ENGINEERING PLANS)
LAYOUT & PROMENADE
GRADING PLAN NO. 1A

Sheet reference number:
C-101A
Sheet 24A of 166
AM-009



VECP, 09/06/2006



KEYMAP

SEE C-101A FOR LAYOUT LEGEND AND NOTES

US Army Corps of Engineers
Sacramento District

Symbol	Description	Date	Approved
3	REVISED ELEVATIONS AND GRADES		
2	REMOVED STATION EQUATION		
1	REVISED NOTE		

Designed by: | Date: | Rev.
Dwn by: | Spec No.: | Design file no:
Reviewed by: | Drawing Code:
Submitted by: | File name: Plot date: Dwg scale:

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

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900 East Main Street, Suite 100
Grass Valley, California 95945
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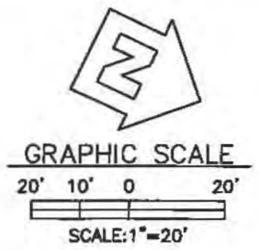
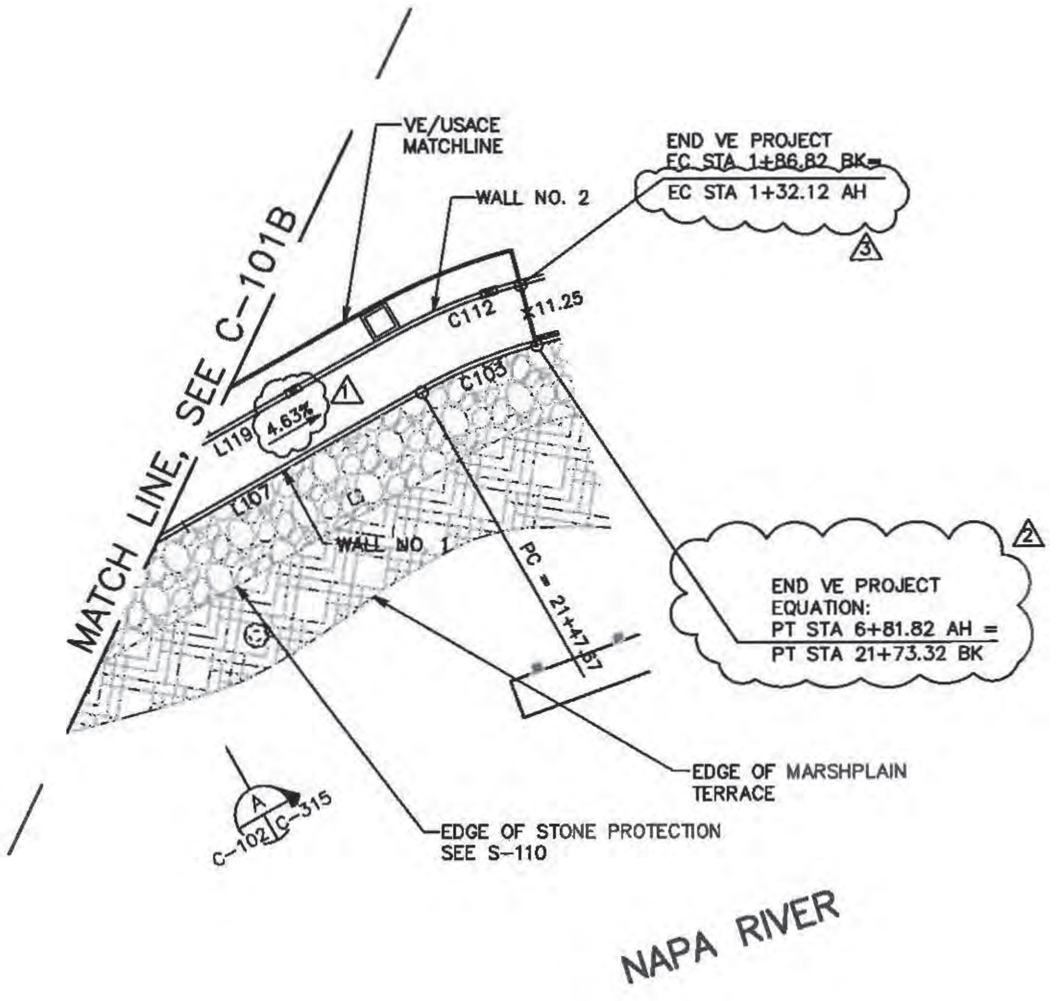
NAPA CALIFORNIA
NAPA RIVER/NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (BROWN STREET
TO FIFTH STREET)
(VALUE ENGINEERING PLANS)
LAYOUT & PROMENADE
GRADING PLAN NO. 1B

Sheet
reference
number:
C-101B
Sheet of 166

VE





VECP, 09/06/2006

KEYMAP

SEE C-101A FOR LAYOUT LEGEND AND NOTES

US Army Corps of Engineers Sacramento District

Symbol	Description	Date	Approved
3	REVISED STATION		
2	MOVED STATION EQUATION TO VE LIMITS		
1	REVISED GRADES		

Designed by:	Date:	Rev.
Dwn by: Spec No.:	Design file no:	
Reviewed by:	Drawing Code:	
Submitted by:	File name: Plot date: Dwn scale:	

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

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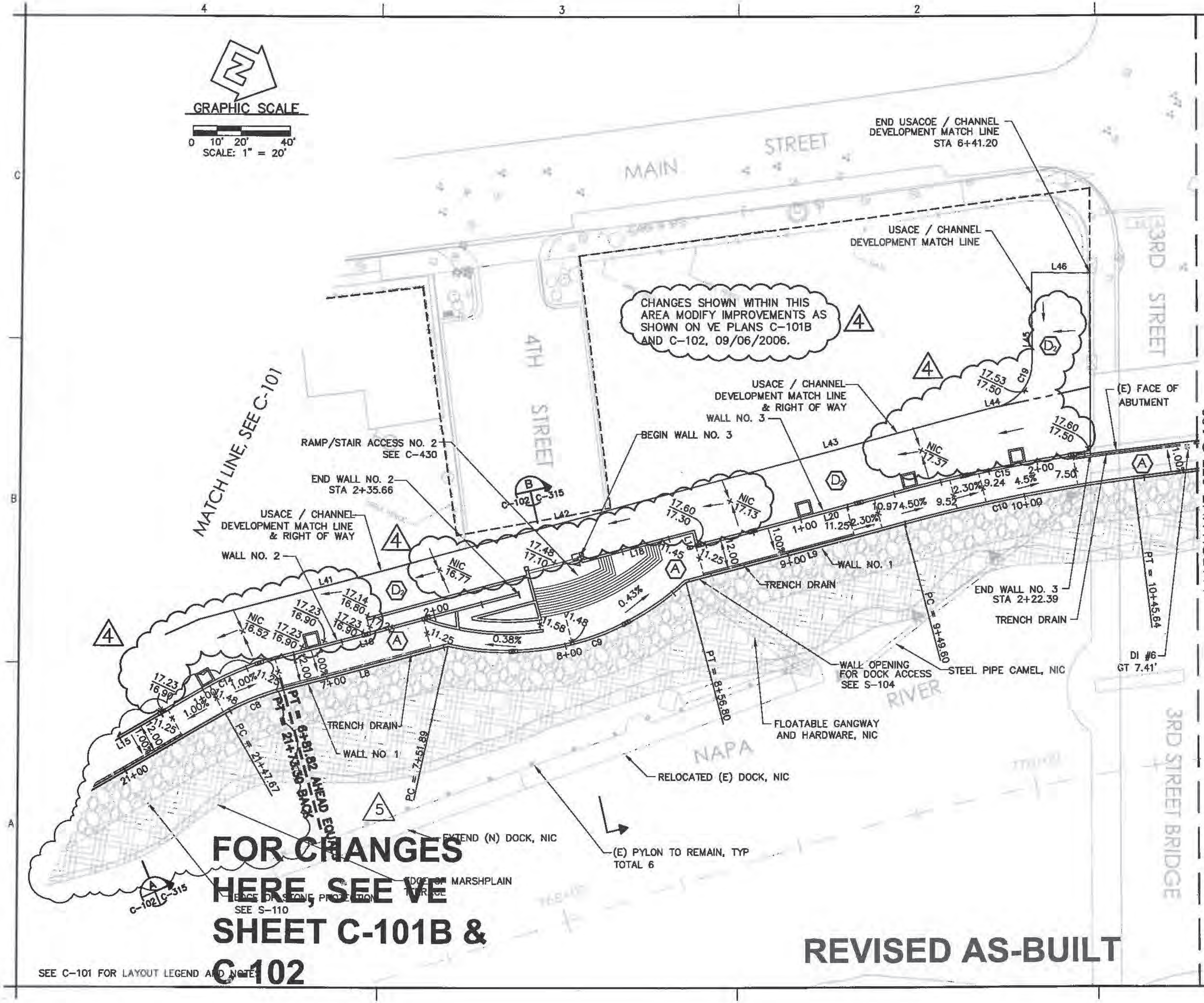
900 East Main Street, Suite 100
Grass Valley, California 95945
530.477.9960

NAPA CALIFORNIA
NAPA RIVER/NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (BROWN STREET
TO FIFTH STREET)
(VALUE ENGINEERING PLANS)
LAYOUT & PROMENADE
GRADING PLAN NO. 2

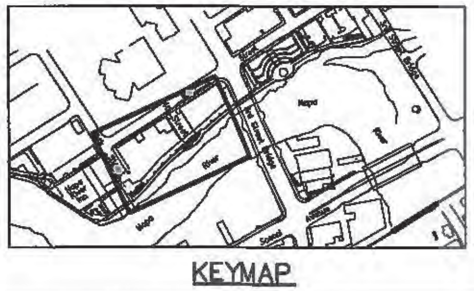
Sheet reference number:
C-102
Sheet of 166
VE



FOR CHANGES HERE, SEE VE SHEET C-101B & C-102

REVISED AS-BUILT

SEE C-101 FOR LAYOUT LEGEND AND NOTES



KEYMAP

US Army Corps of Engineers
Sacramento District

Symbol	Description	Date	Approved
1	WALL NO. 1 ALIGNMENT CHANGE	[illegible]	JSC
2	UPDATES ASSOCIATED WITH WALL NO. 1 ALIGNMENT CHANGE	[illegible]	JSC
3	REVISION PER STORM DRAIN CHANGE	[illegible]	JSC
4	5TH STREET UTILITY MODIFICATIONS	[illegible]	JSC
5	CHANGES PER VE DESIGN		

Designed by: JEFF S. CROMITZ
Dwn by: KLI
Spec No.: 1407
Reviewed by: R. SENNETT
Submitted by: /S/ J. CROMITZ
Date: 04/27/2005
Rev.
Design file no: NA-25-030
Drawing Code:

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

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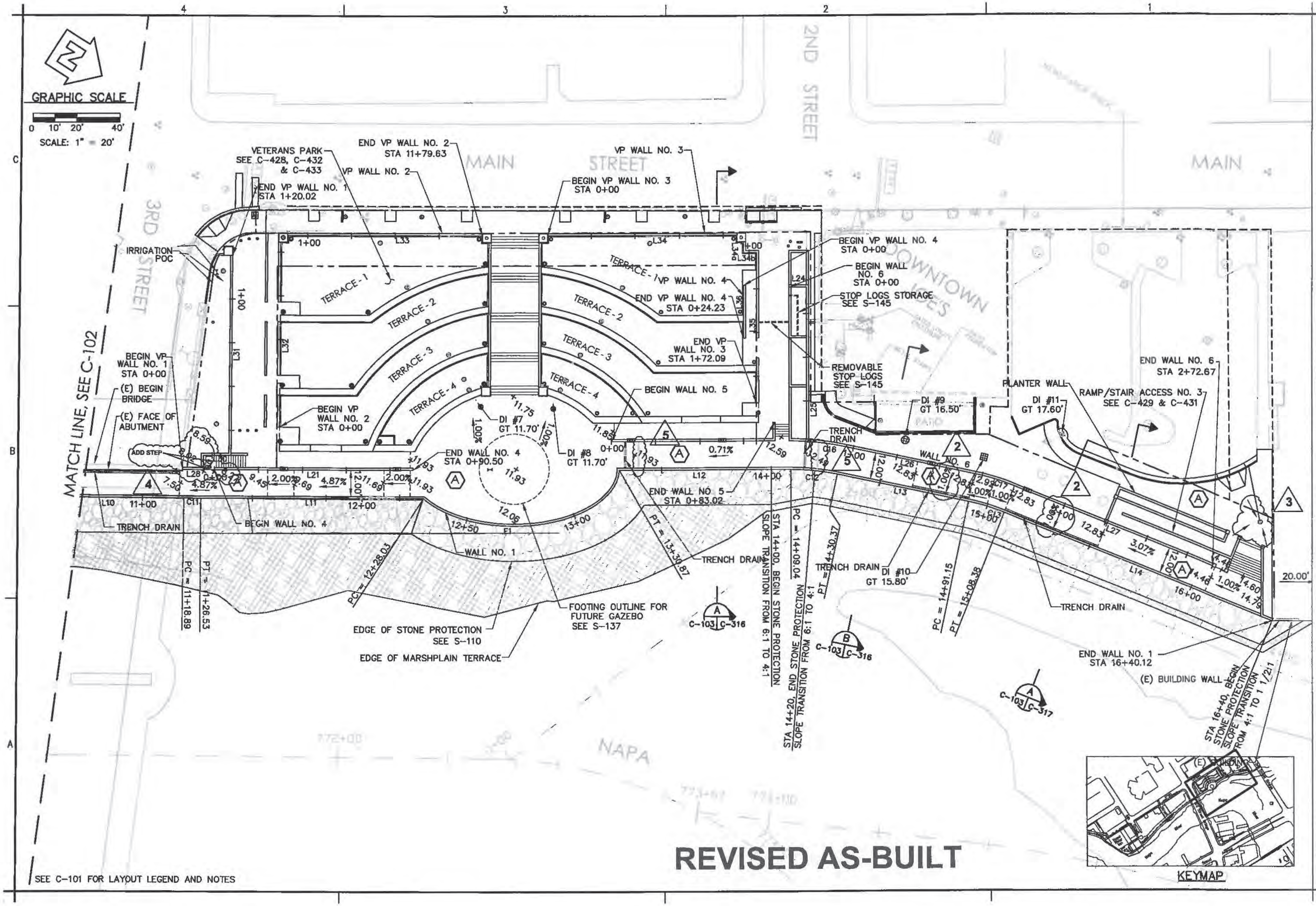
916.421.1000

NAPA CALIFORNIA
NAPA RIVER/NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (HATT BUILDING TO FIRST STREET)

LAYOUT & PROMENADE
GRADING PLAN NO. 2

Sheet reference number:
C-102
Sheet 25 of 166
AM-004



REVISED AS-BUILT

SEE C-101 FOR LAYOUT LEGEND AND NOTES

US Army Corps
of Engineers
Sacramento District

Symbol	Description	Date	Approved
1	STONE PROTECTION REVISION		JSC
2	ADD CROSS-SLOPE AT TRENCH DRAINS		
3	ADD 3" AD TO BETTER DRAIN PAVEMENT		
4	CHANGES PER RFI 98 – GRADE BUST @ VP STAIR 1		
5	ADD ADD'L TRENCH DRAIN FOR BETTER DRAINAGE		

DEPARTMENT OF THE ARMY
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SACRAMENTO, CALIFORNIA

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Designed by: JEFF S. CROWITZ
Dwn by: KJ
Spec No.: 1407
Reviewed by: R. SENNETT
Submitted by: /S/ J. CROWITZ
Date: 04/27/2005
Rev.
Design file no: NA-25-030
Drawing Code:

NAPA
NAPA RIVER/NAPA CREEK FLOOD PROTECTION PROJECT
CALIFORNIA
CONTRACT 2 WEST (HATT BUILDING TO FIRST STREET)

LAYOUT & PROMENADE
GRADING PLAN NO. 3

Sheet reference number:
C-103
Sheet 26 of 166

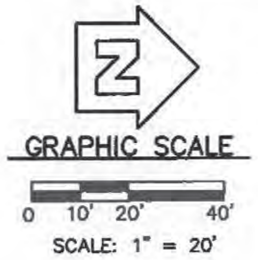
AM-008

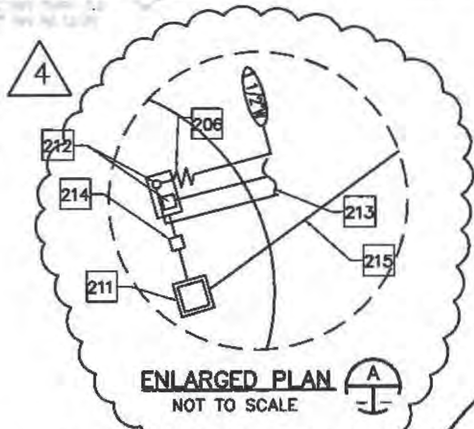
DRAINAGE INLET SCHEDULE – 1

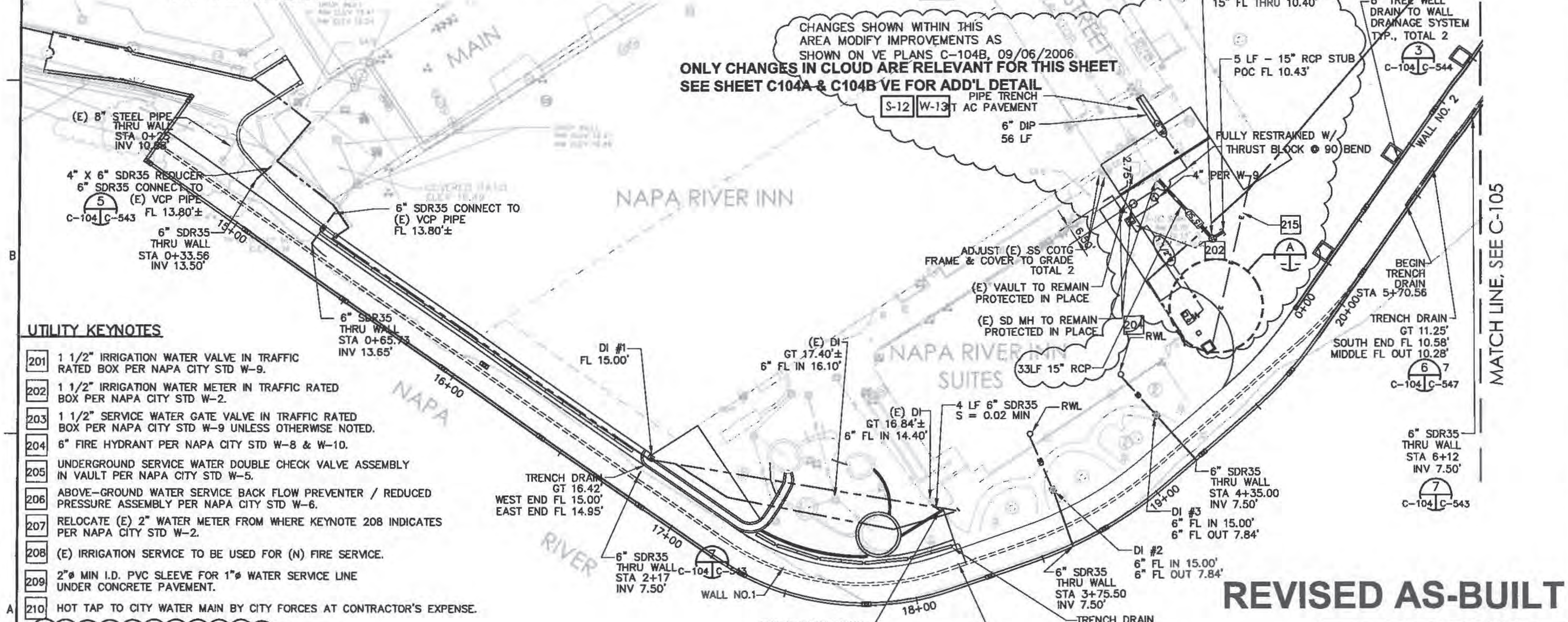
NO.	"WALL #1" STA / OFF	GRATE ELEV	COMMENT
~~1~~	~~2+17.02 / 7.72' L~~	~~16.42'~~	~~18" X 18" X 18"D "TEICHERT" OR EQUAL W/ 18" X 18" DECORATIVE GRATE~~
~~2~~	~~3+75.50 / 11.00' L~~	~~16.40'~~	~~18" X 18" X 8'–9"D "TEICHERT" OR EQUAL W/ 18" X 18" DOUBLE GALVANIZED GRATE~~
~~3~~	~~4+35 / 11.00' L~~	~~16.40'~~	~~18" X 18" X 8'–9"D "TEICHERT" OR EQUAL W/ 18" X 18" DOUBLE GALVANIZED GRATE~~
4	19+92.32/ 56.57' L	16.00'	48"ø X 5'–8"D MH BARREL SECTION "TEICHERT" OR EQUAL W/ 48"ø DOUBLE GALVANIZED GRATE (FUTURE MH RIM 17.20')

NOTE

SEE "MANHOLE / INLET COLLAR DETAIL" FOR MANHOLE/ INLET TO RCP COLLAR CONNECTION DETAIL.







ONLY CHANGES IN CLOUD ARE RELEVANT FOR THIS SHEET
SEE SHEET C104A & C104B VE FOR ADD'L DETAIL

UTILITY KEYNOTES

201	1 1/2" IRRIGATION WATER VALVE IN TRAFFIC RATED BOX PER NAPA CITY STD W–9.
202	1 1/2" IRRIGATION WATER METER IN TRAFFIC RATED BOX PER NAPA CITY STD W–2.
203	1 1/2" SERVICE WATER GATE VALVE IN TRAFFIC RATED BOX PER NAPA CITY STD W–9 UNLESS OTHERWISE NOTED.
204	6" FIRE HYDRANT PER NAPA CITY STD W–8 & W–10.
205	UNDERGROUND SERVICE WATER DOUBLE CHECK VALVE ASSEMBLY IN VAULT PER NAPA CITY STD W–5.
206	ABOVE–GROUND WATER SERVICE BACK FLOW PREVENTER / REDUCED PRESSURE ASSEMBLY PER NAPA CITY STD W–6.
207	RELOCATE (E) 2" WATER METER FROM WHERE KEYNOTE 208 INDICATES PER NAPA CITY STD W–2.
208	(E) IRRIGATION SERVICE TO BE USED FOR (N) FIRE SERVICE.
209	2"ø MIN I.D. PVC SLEEVE FOR 1"ø WATER SERVICE LINE UNDER CONCRETE PAVEMENT.
210	HOT TAP TO CITY WATER MAIN BY CITY FORCES AT CONTRACTOR'S EXPENSE.
211	TRANSFORMER PER ELECTRICAL PLAN.
212	IRRIGATION POC AND CONTROLLER PEDESTAL, SEE I–102.
213	IRRIGATION AND IRRIGATION CONTROL CONDUITS, SEE I–102 FOR CONTINUATION.
214	"SPA" ELCTRICAL PEDESTAL, SEE E–102 FOR CONTINUATION.

3

ULILITY NOTES

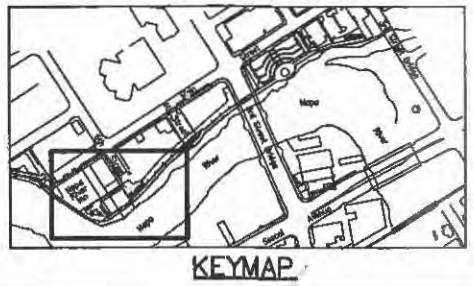
1. SEE C–210 FOR USACE / CHANNEL DEVELOPMENT MATCHLINE PROFILES
2. 12"ø AND LARGER RCP TO BE CLASS III PER CITY OF NAPA STD SPEC.
3. SEE S–108 FOR "OUTLET GATE BOX" DETAIL TO BE CONSTRUCTED AT ALL DRAIN OUTLETS, PENETRATING WALL, UON.
4. SEE G–005 FOR UTILITY NOTES AND CONTACT INFORMATION.
5. SEE C–547 FOR TRENCH DRAIN DETAILS.

UTILITY LEGEND

━18"SD━	RCP STORM DRAIN
— SD —	SDR35 STORM DRAIN
— W —	COPPER WATER
— SS —	SDR35 SANITARY SEWER

SEE C–101 THROUGH C–103 FOR LEGEND AND LAYOUT NOT SHOWN HERE

REVISED AS-BUILT



KEYMAP

US Army Corps of Engineers
Sacramento District

Symbol	Description	Date	Approved
1	UPDATES ASSOCIATED WITH WALL NO. 1 ALIGNMENT CHANGE	[illegible]	JSC
2	REVISION PER STORM DRAIN CHANGE	[illegible]	JSC
3	5TH STREET UTILITY MODIFICATIONS	[illegible]	JSC
4	CHANGES FOR WATERLINE & HYDRANT LOC BY CITY	[illegible]	

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

MGE ENGINEERING INC.
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Sacramento, California 95831
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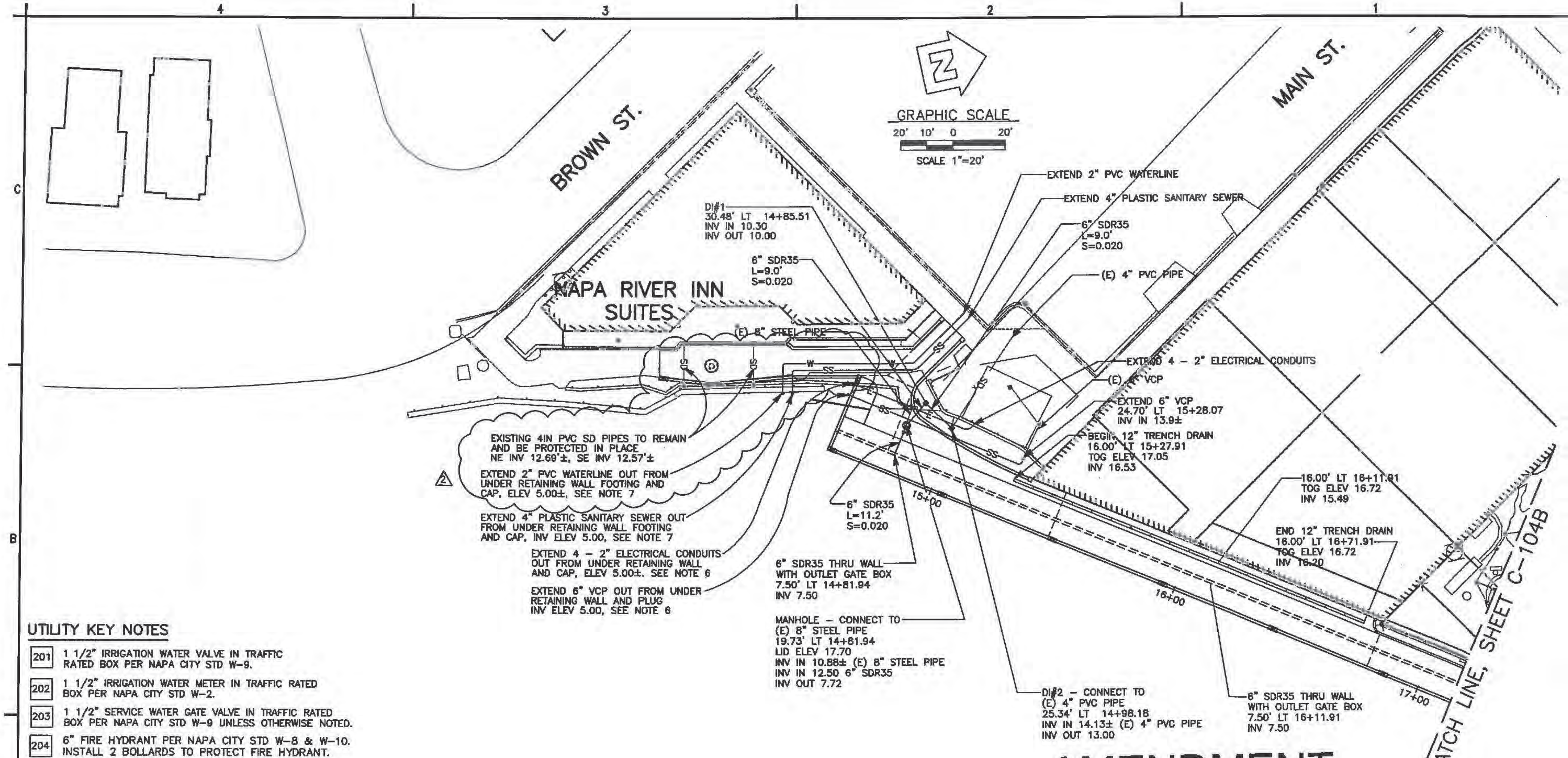
Designed by: JEFF S. CROVITZ		Date: 04/27/2005
Dwn by: KLI	Spec No.: 1407	Design file no: NA–25–030
Reviewed by: R. SENNETT		Drawing Code:
Submitted by: /S/ J. CROVITZ		File name: Plot date: Dwg scale:

NAPA, CALIFORNIA
NAPA RIVER/NAPA CREEK FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (HATT BUILDING TO FIRST STREET)

UNDERGROUND UTILITY PLAN NO

Sheet reference number:
C–104
Sheet 27 of 166



AMENDMENT NO. 5, 12/28/2006

UTILITY KEY NOTES

201	1 1/2" IRRIGATION WATER VALVE IN TRAFFIC RATED BOX PER NAPA CITY STD W-9.
202	1 1/2" IRRIGATION WATER METER IN TRAFFIC RATED BOX PER NAPA CITY STD W-2.
203	1 1/2" SERVICE WATER GATE VALVE IN TRAFFIC RATED BOX PER NAPA CITY STD W-9 UNLESS OTHERWISE NOTED.
204	6" FIRE HYDRANT PER NAPA CITY STD W-8 & W-10. INSTALL 2 BOLLARDS TO PROTECT FIRE HYDRANT.
205	UNDERGROUND SERVICE WATER DOUBLE CHECK VALVE ASSEMBLY IN VAULT PER NAPA CITY STD W-5.
206	ABOVE-GROUND WATER SERVICE BACK FLOW PREVENTER / REDUCED PRESSURE ASSEMBLY PER NAPA CITY STD W-2 & W-6.
207	RELOCATE (E) 2" WATER METER FROM WHERE KEYNOTE 208 INDICATES PER NAPA CITY STD W-2.
208	(E) IRRIGATION SERVICE TO BE USED FOR (N) FIRE SERVICE.
209	2"ø MIN I.D. PVC SLEEVE FOR 1"ø WATER SERVICE LINE UNDER CONCRETE PAVEMENT.
210	HOT TAP TO CITY WATER MAIN BY CITY FORCES AT CONTRACTOR'S EXPENSE.

UTILITY LEGEND

18"SD	RCP STORM DRAIN
▬	SDR35 STORM DRAIN
▬	COPPER WATER
▬	SDR35 SANITARY SEWER

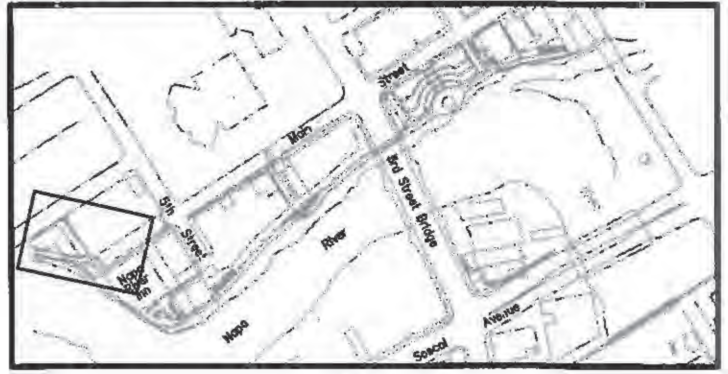
SEE C-101A THROUGH C-103 FOR LEGEND AND LAYOUT NOT SHOWN HERE

DRAINAGE INLET SCHEDULE - 1B

NO.	"WALL #1" STA / OFF	GRATE ELEV	COMMENT
1	14+85.51 / 30.48' L	16.00'	18" X 18" X 8'-9" D "TEICHERT" OR EQUAL W/ 18" X 18" DOUBLE GALVANIZED GRATE
2	14+98.18 / 25.34' L	16.00'	18" X 18" X 8'-9" D "TEICHERT" OR EQUAL W/ 18" X 18" DOUBLE GALVANIZED GRATE

UTILITY NOTES

1. SEE C-210 FOR USACE / CHANNEL DEVELOPMENT MATCH LINE PROFILES.
2. 12" ØAND LARGER RCP TO BE CLASS III PER CITY OF NAPA STD SPEC.
3. SEE S-108 FOR "OUTLET GATE BOX" DETAIL TO BE CONSTRUCTED AT ALL DRAIN OUTLETS PENETRATING WALL.
4. SEE G-005 FOR UTILITY NOTES AND CONTACT INFORMATION.
5. SEE C-547 FOR TRENCH DRAIN DETAILS.
6. SEE S-101A "WALL NO. 1A ELEVATION" FOR ADDITIONAL DETAILS.
7. SEE S-101A "WALL NO. 1B ELEVATION" FOR ADDITIONAL DETAILS.



KEYMAP

US Army Corps of Engineers
Sacramento District

Symbol	Description	Date	Approved
1	AM/005 - REVISED SIDEWALK	JSC	12/28/06
2	REVISED TRENCH DRAIN		

Designed by:
Dwn by:
Spec No.:
Reviewed by:
Submitted by:
Date:
Rev.
Design file no:
Drawing Code:
File name:
Plot date:
Dwg scale:

DEPARTMENT OF THE ARMY
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NAPA CALIFORNIA
NAPA RIVER/NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (BROWN STREET
TO FIFTH STREET)
(VALUE ENGINEERING PLANS)

UNDERGROND UTILITY PLAN NO. 1A

Sheet reference number:
C-104A
Sheet 27A of 166
AM-005

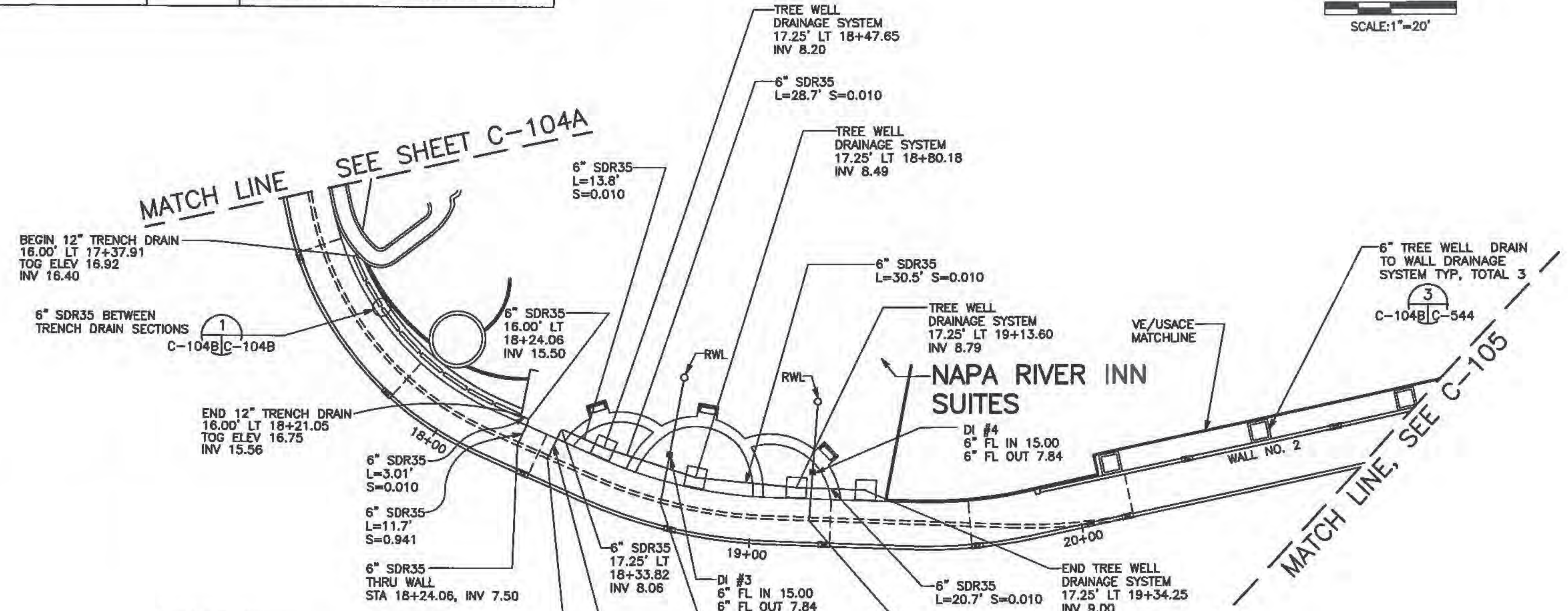
DRAINAGE INLET SCHEDULE - 1B

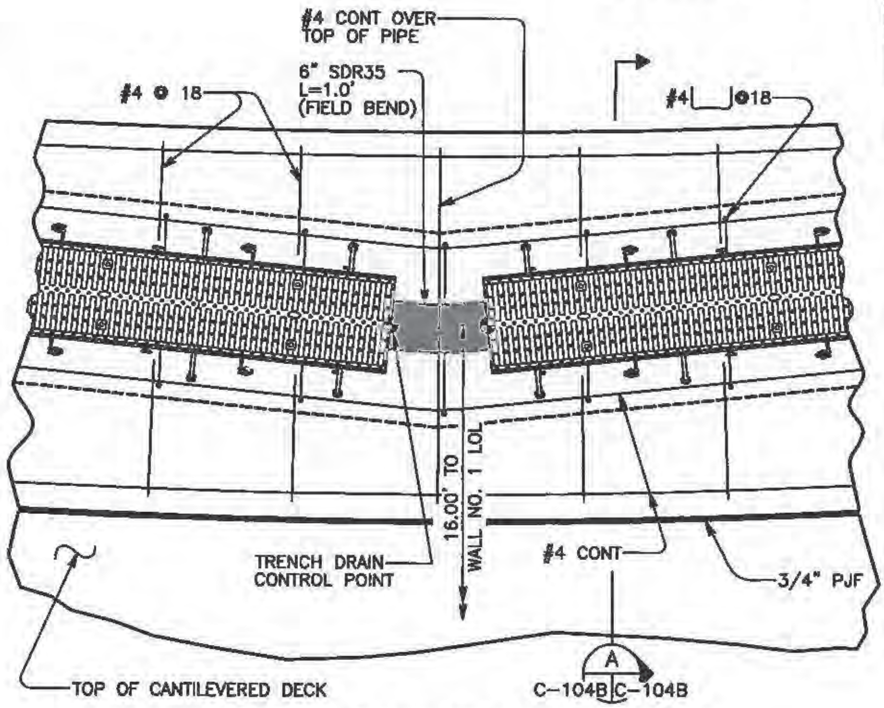
NO.	"WALL #1" STA / OFF	GRATE ELEV	COMMENT
3	18+69.50 / 22.00' L	16.59'	18" X 18" X 8'-9" D "TEICHERT" OR EQUAL W/ 18" X 18" DOUBLE GALVANIZED GRATE
4	19+18.27 / 22.00' L	16.59'	18" X 18" X 8'-9" D "TEICHERT" OR EQUAL W/ 18" X 18" DOUBLE GALVANIZED GRATE

GRAPHIC SCALE

20' 10' 0 20'

SCALE:1"=20'

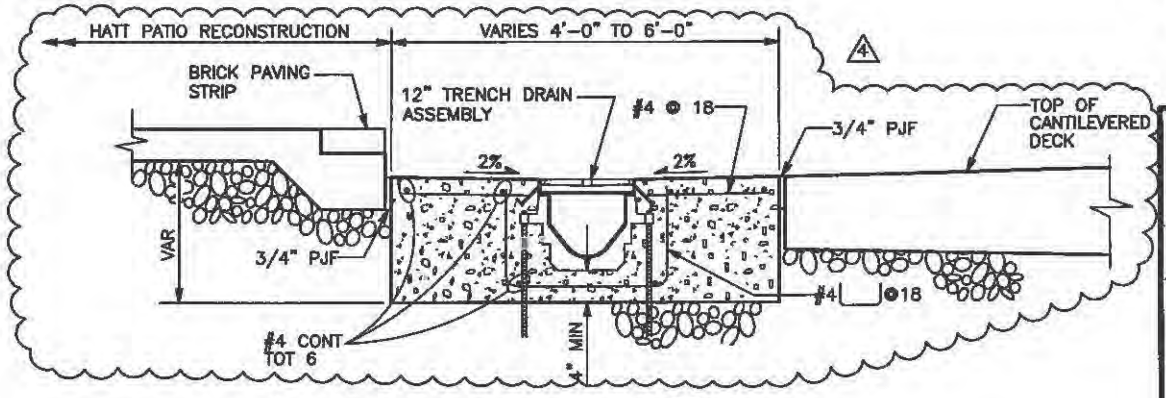




TRENCH DRAIN DETAIL

SCALE: 1"=1'-0"

1 C-104B C-104B

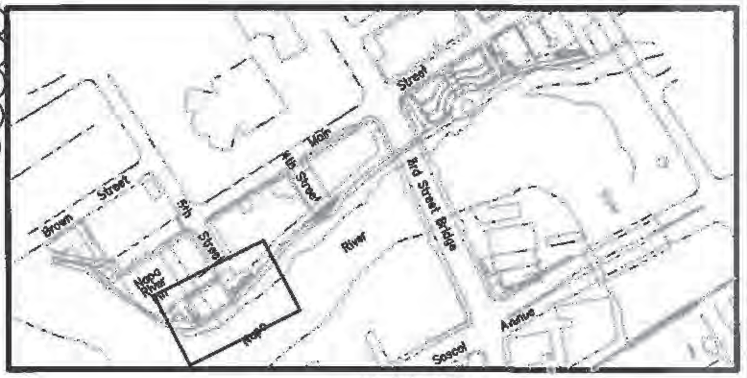


SECTION

SCALE: 1"=1'-0"

A C-104B C-104B

VECP, 02/01/2007



KEYMAP

SEE C-104A FOR LAYOUT LEGEND AND NOTES

US Army Corps of Engineers Sacramento District

Symbol	Description	Date	Approved
	REVISED SECTION	02/01/07	GGB
	PLAZA RECONSTRUCTION NOT PART OF THE VECP	12/22/06	GGB
	REVISED DRAINAGE SYSTEM LAYOUT	12/22/06	GGB
	FIELD BEND 6" SDR35	09/06/06	GGB

Designed by: Date: Rev.
Dwn by: Spec No.: Design file no:
Reviewed by: Drawing Code:
Submitted by: File name: Plot date: Dwg scale:

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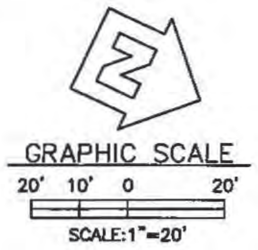
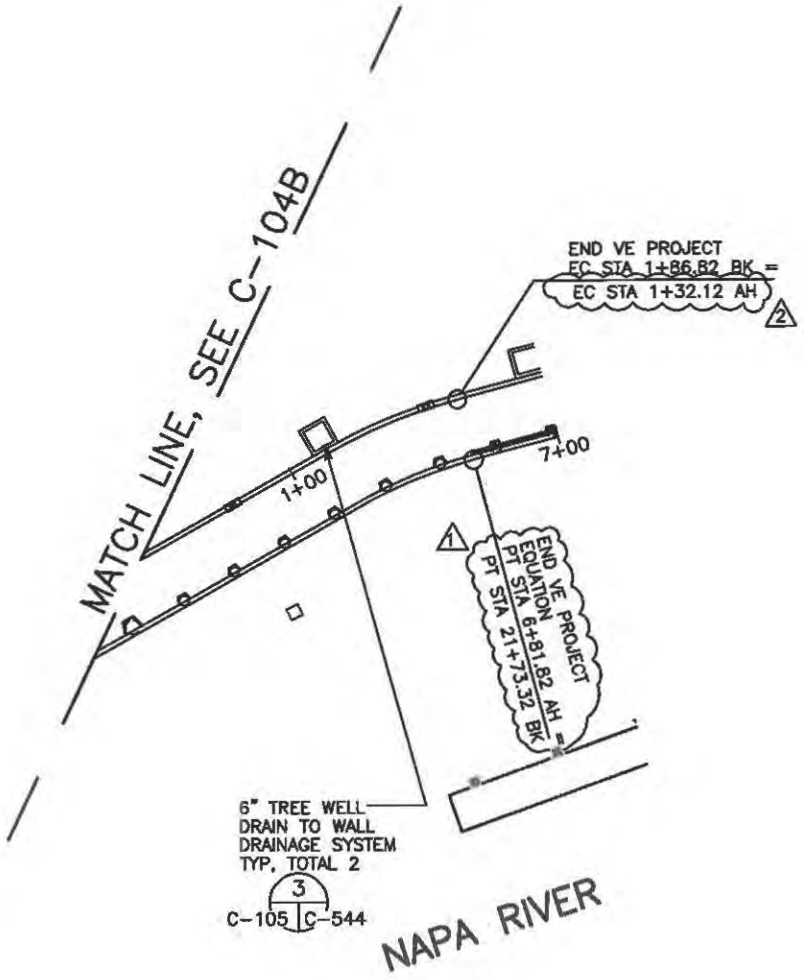
NAPA CALIFORNIA
NAPA RIVER/NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (BROWN STREET TO FIFTH STREET)
(VALUE ENGINEERING PLANS)

UNDERGROUND UTILITY PLAN NO. 1B

Sheet reference number:
C-104B
Sheet of 166
VE





VECP, 09/06/2006

KEYMAP

SEE C-104A FOR UNDERGROUND UTILITY LEGEND AND NOTES

US Army Corps
of Engineers
Sacramento District

Symbol	Description	Date	Approved
△	REVISED STATION		
△	MOVED STATION EQUATION TO VE LIMITS		

Designed by:		Date:	Rev.
Dwn by:	Spec No.:	Design file no:	
Reviewed by:		Drawing Code:	
Submitted by:		File name: Plot date: Plot scale:	

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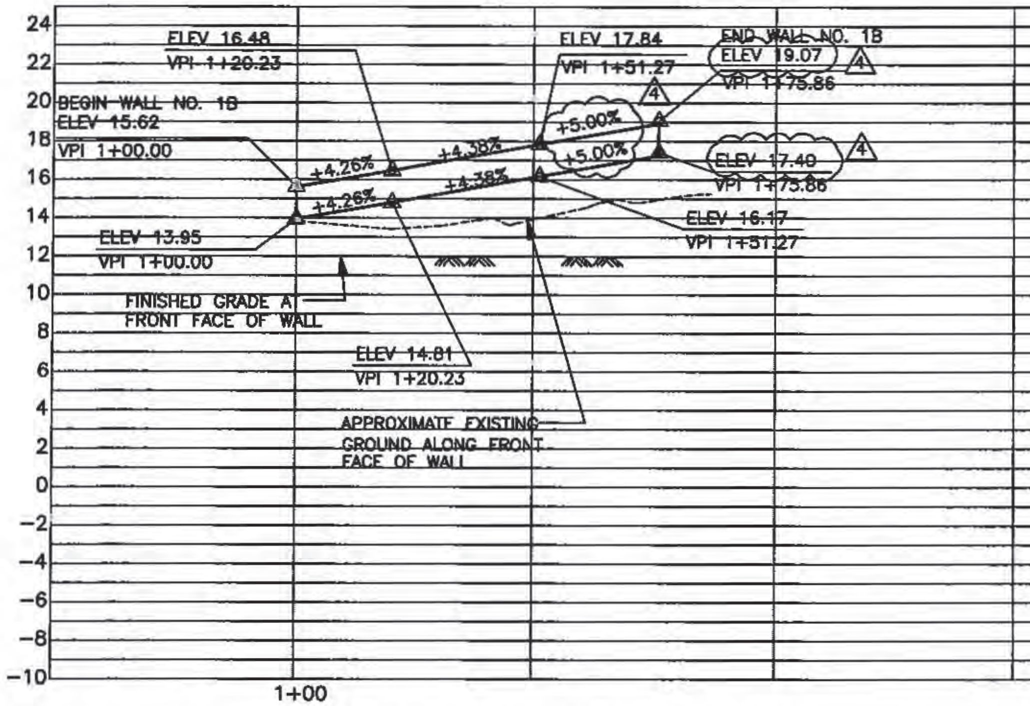
NAPA CALIFORNIA
NAPA RIVER/NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (BROWN STREET
TO FIFTH STREET)
(VALUE ENGINEERING PLANS)

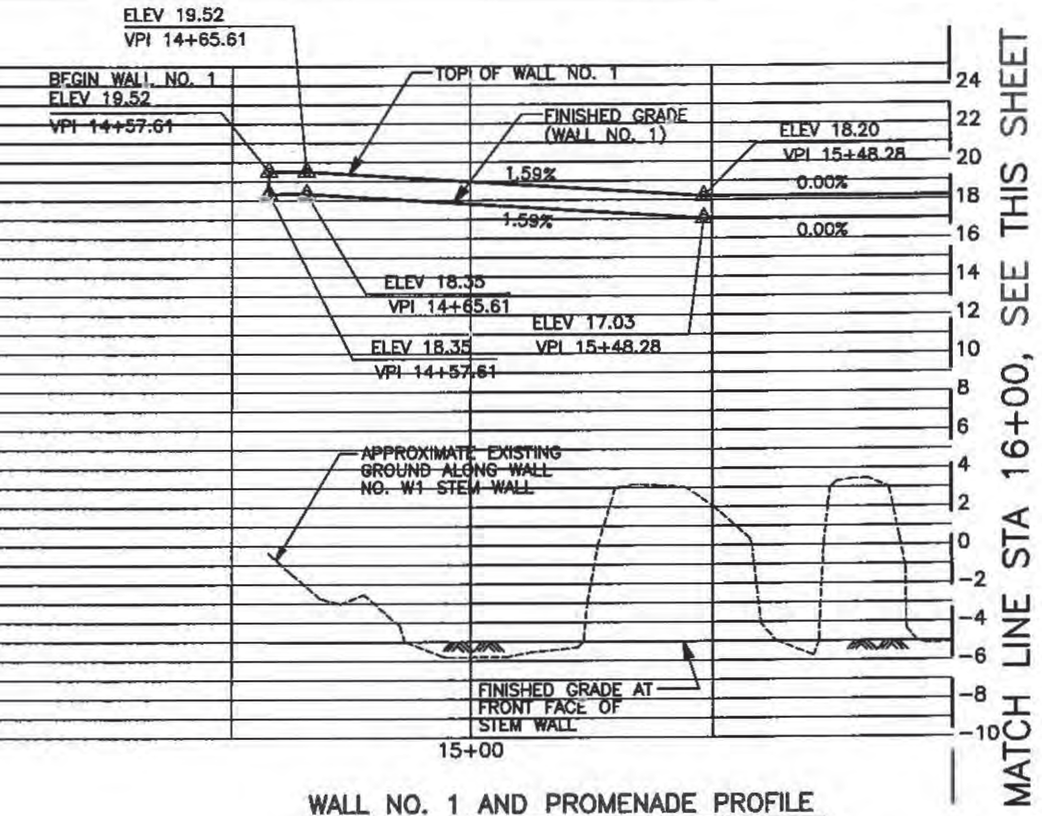
UNDERGROUND UTILITY PLAN NO. 2

Sheet
reference
number:
C-105
Sheet of 166
VE



WALL NO. 1B AND PROMENADE PROFILE

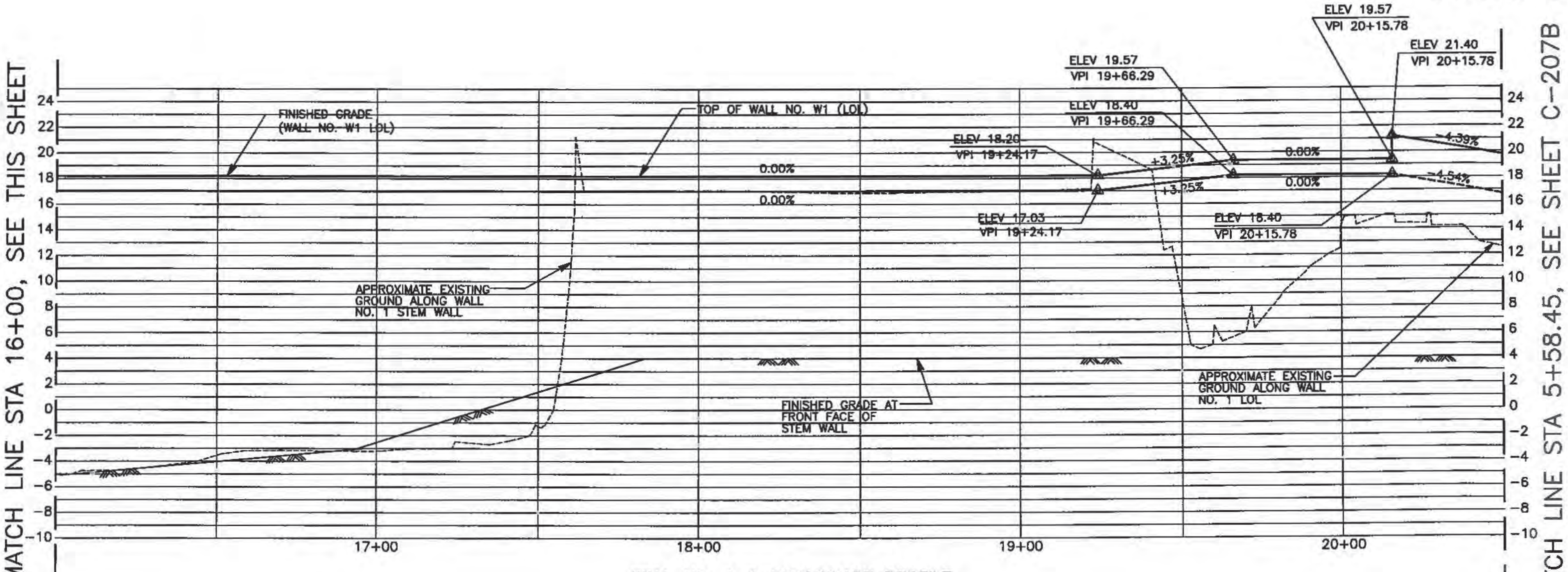
SCALE: 1"=20' H
1"=5' V



WALL NO. 1 AND PROMENADE PROFILE

SCALE: 1"=20' H
1"=5' V

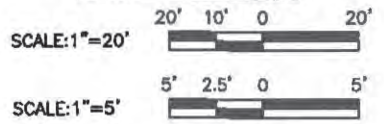
AMENDMENT
NO. 9, 07/03/2007



WALL NO. 1 & PROMENADE PROFILE

SCALE: 1"=20' H
1"=5' V

GRAPHIC SCALE



US Army Corps
of Engineers
Sacramento District

Symbol	Descriptions	Date	Approved
4	AM/009 - REVISED GRADING OF SIDEWALK	JSC	07/03/07
3	AM/006 - REVISED STONE PROTECTION AND LOWERED WALL FOOTING	JSC	03/16/07
2	AM/005 - REVISED SIDEWALK	JSC	12/28/06
1	REVISED ELEVATIONS AND GRADES		

Designed by: | Date: | Rev.
Dwn by: | Spec No.: | Design file no:
Reviewed by: | Drawing Code:
Submitted by: | File name: | Plot date: | Dwg scale:

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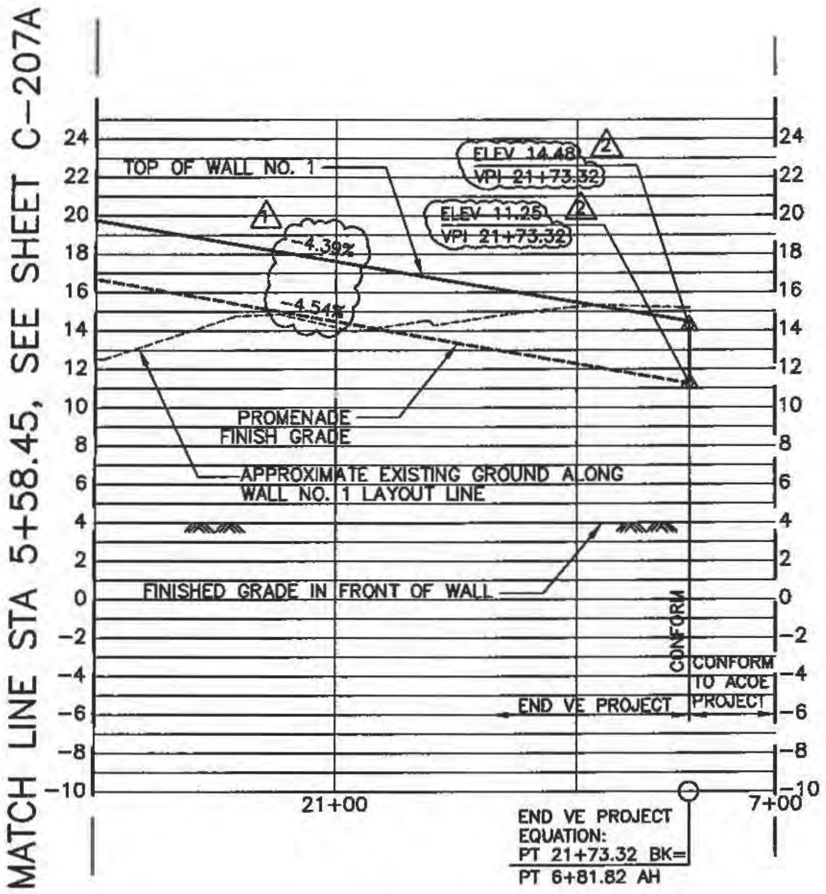
NAPA CALIFORNIA
NAPA RIVER/NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (BROWN STREET
TO FIFTH STREET)
(VALUE ENGINEERING PLANS)
WALL AND PROMENADE
PROFILES NO. 1A

Sheet
reference
number:
C-207A
Sheet 30Aof 166

AM-009



WALL NO. 1 & LOWER PROMENADE PROFILE

SCALE: 1"=20' H
1"=5' V

VECP, 09/06/2006

GRAPHIC SCALE

SCALE:1"=20' 20' 10' 0 20'

SCALE:1"=5' 5' 2.5' 0 5'

US Army Corps
of Engineers
Sacramento District

Symbol	Description	Date	Approved
1	REVISED STATIONS		
2	REVISED GRADES		

Designed by: Dwn by: Reviewed by: Submitted by:
Date: Spec No.: Design file no: Drawing Code:
Rev.

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SACRAMENTO, CALIFORNIA

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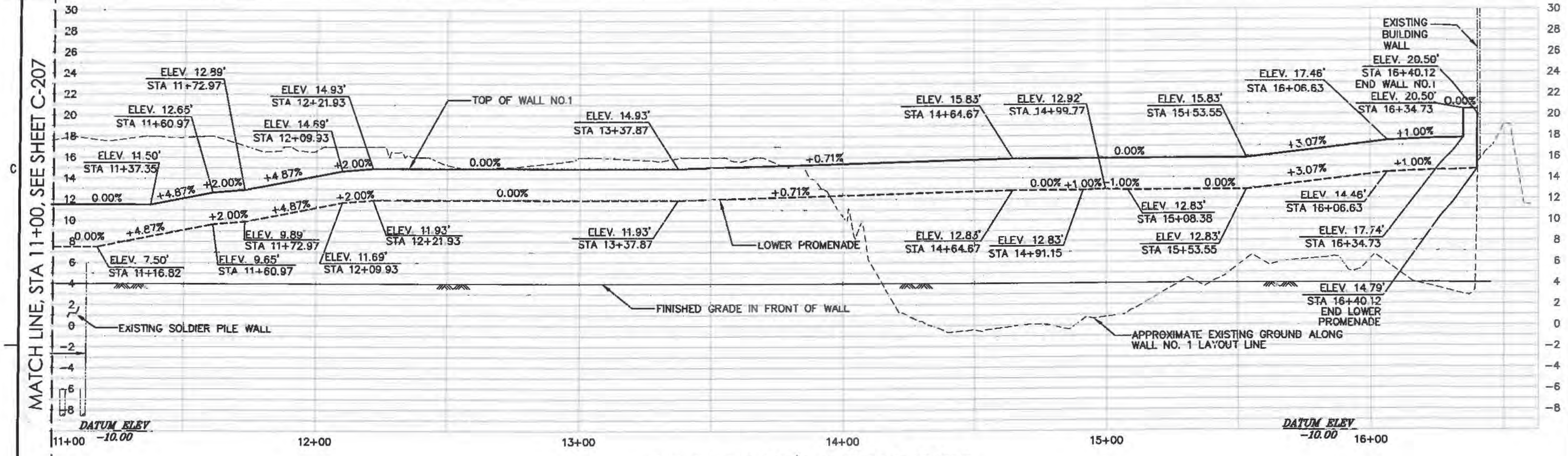
NAPA RIVER/NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (BROWN STREET
TO FIFTH STREET)
(VALUE ENGINEERING PLANS)
WALL AND PROMENADE
PROFILES NO. 1B

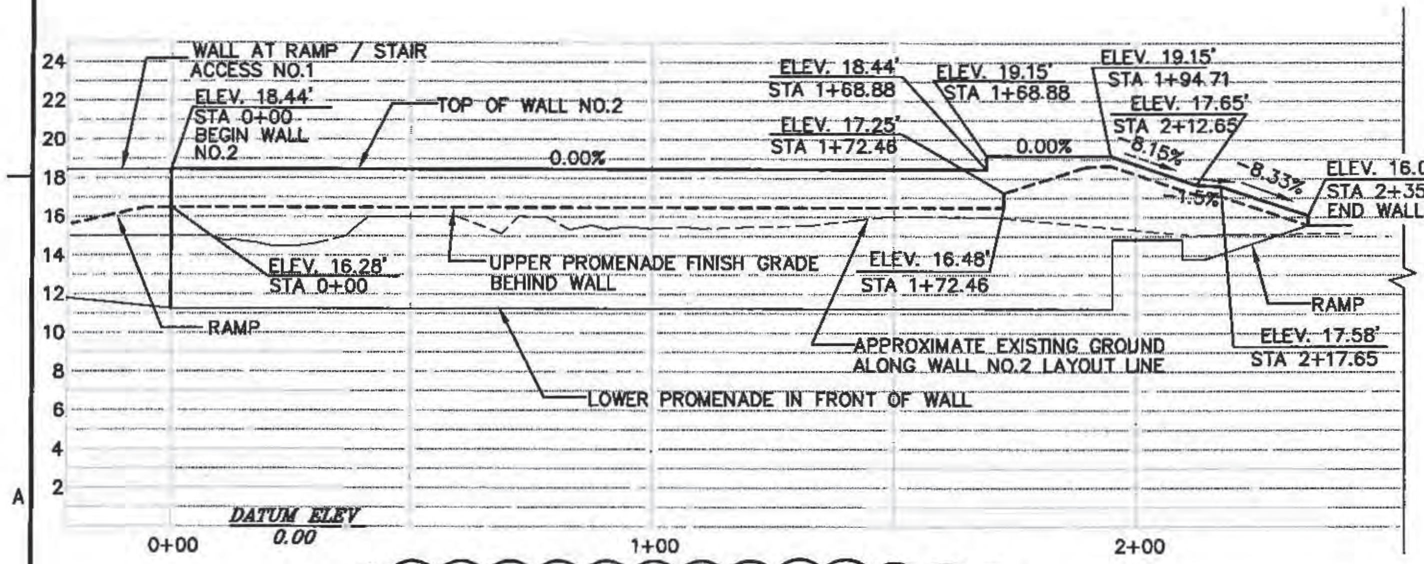
NAPA CALIFORNIA

Sheet reference number:
C-207B
Sheet of 166
VE



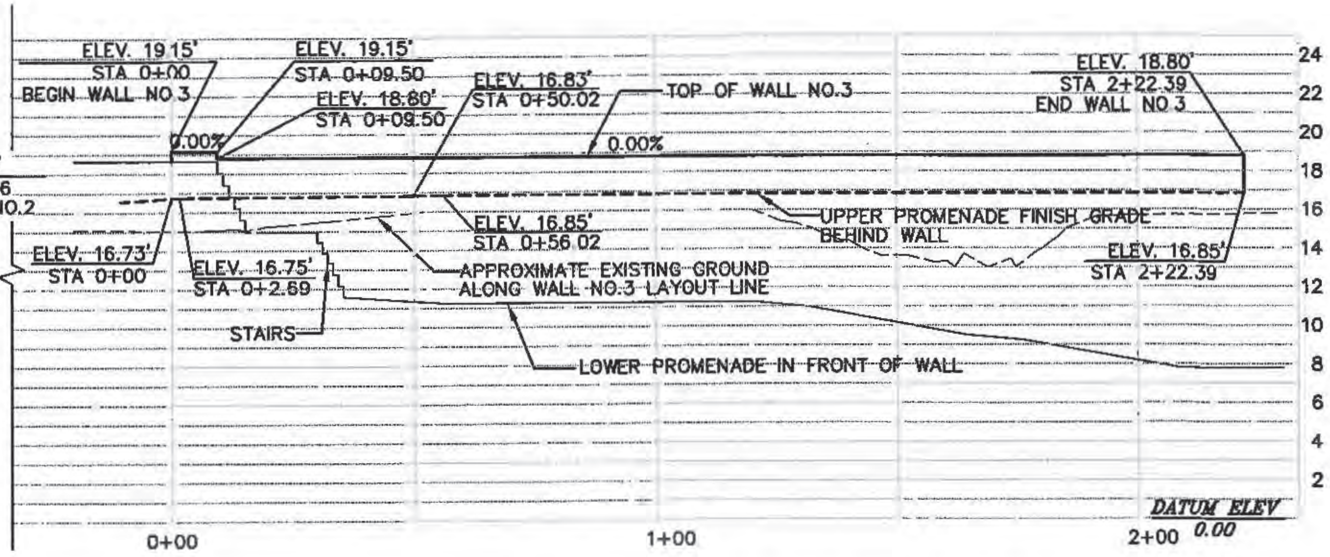
WALL NO. 1 & LOWER PROMENADE PROFILE

SCALE: 1" = 20' H
1" = 5' V



WALL NO. 2 & UPPER PROMENADE PROFILE

SCALE: 1" = 20' H
1" = 5' V



WALL NO. 3 & UPPER PROMENADE PROFILE

SCALE: 1" = 20' H
1" = 5' V

GRAPHIC SCALE

SCALE: 1" = 20' 0 10' 20' 40'

SCALE: 1" = 5' 0 2'-6" 5' 10'

AS-BUILT

SEE C-207 FOR PROFILE LEGEND

US Army Corps of Engineers
Sacramento District

Symbol	Description	Date	Approved
△	UPDATES ASSOCIATED WITH WALL NO. 1 ALIGNMENT CHANGE	[illegible]	JSC

Designed by: JEFF S. CROWITZ
Dwn by: KJ
Spec No.: 1407
Reviewed by: R. SENNETT
Submitted by: /S/ J. CROWITZ
Date: 04/27/2005
Design file no: NA-25-030
Drawing Code:
Rev.

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
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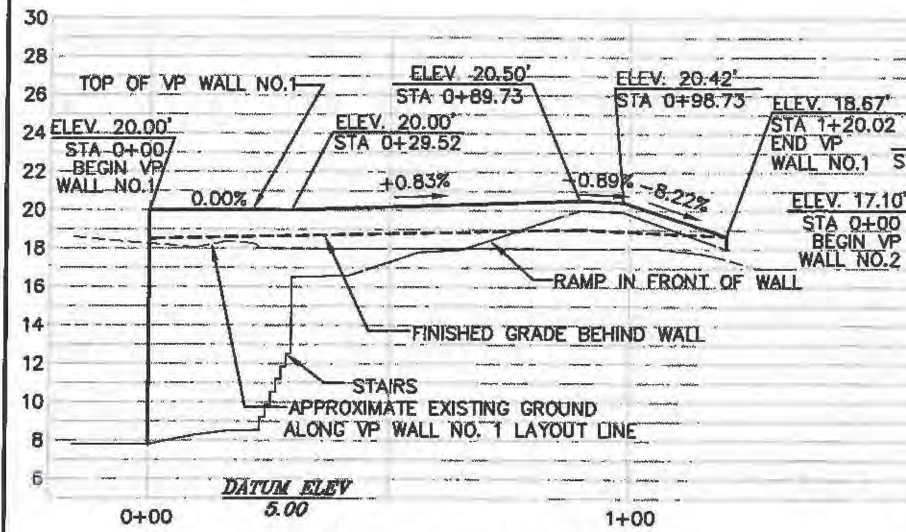
NAPA CALIFORNIA
NAPA RIVER/NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (HATT BUILDING
TO FIRST STREET)

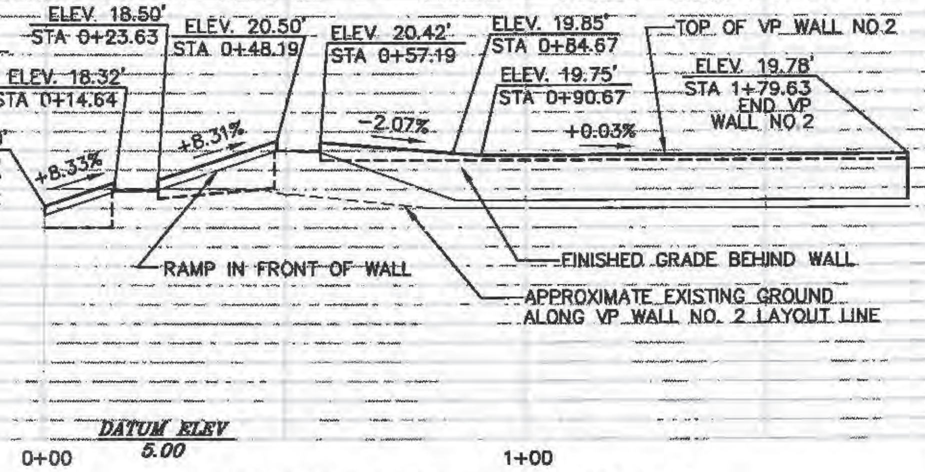
WALL & PROMENADE
PROFILES NO. 2

Sheet reference number:
C-208
Sheet 31 of 166



VETERANS PARK WALL NO. 1 PROFILE

SCALE: 1" = 20' H
1" = 5' V



VETERANS PARK WALL NO. 2 PROFILE

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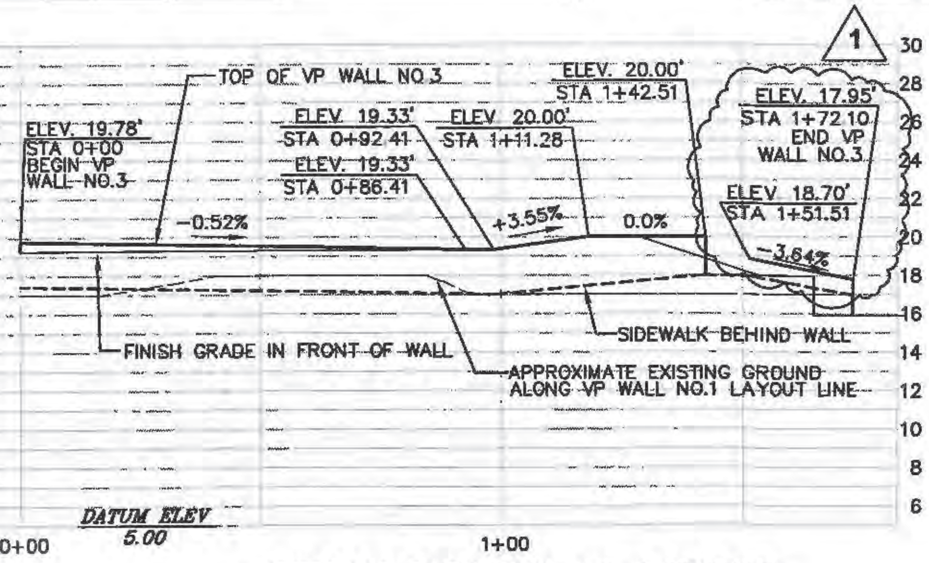
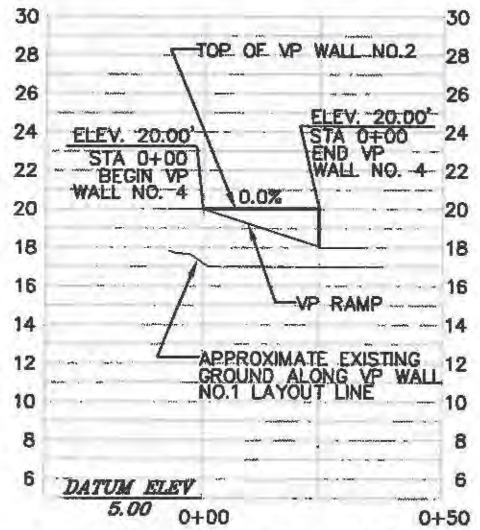
1" = 5' V



VETERANS PARK WALL NO. 3 PROFILE

SCALE: 1" = 20' H
1" = 5' V



VETERANS PARK WALL NO. 4 PROFILE

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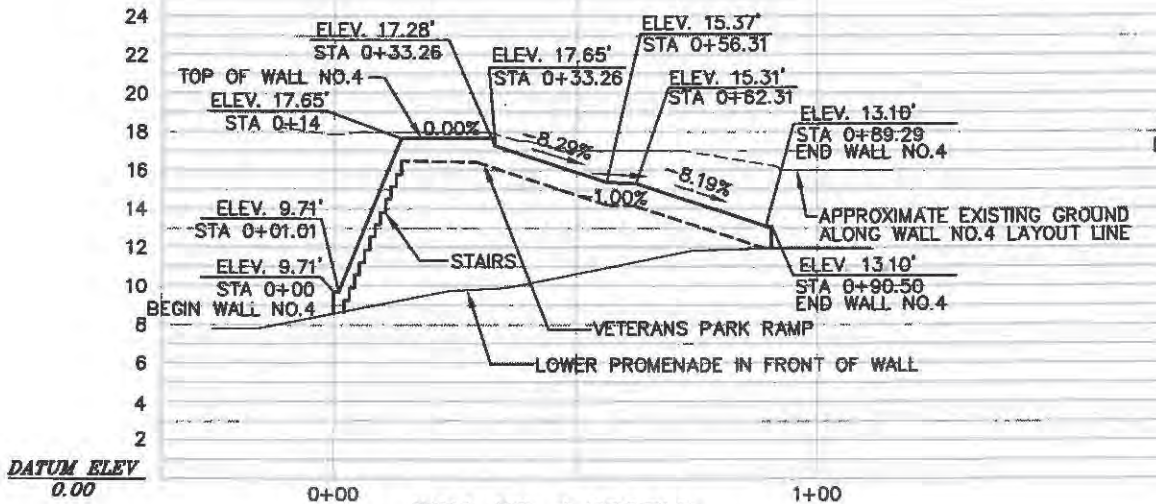
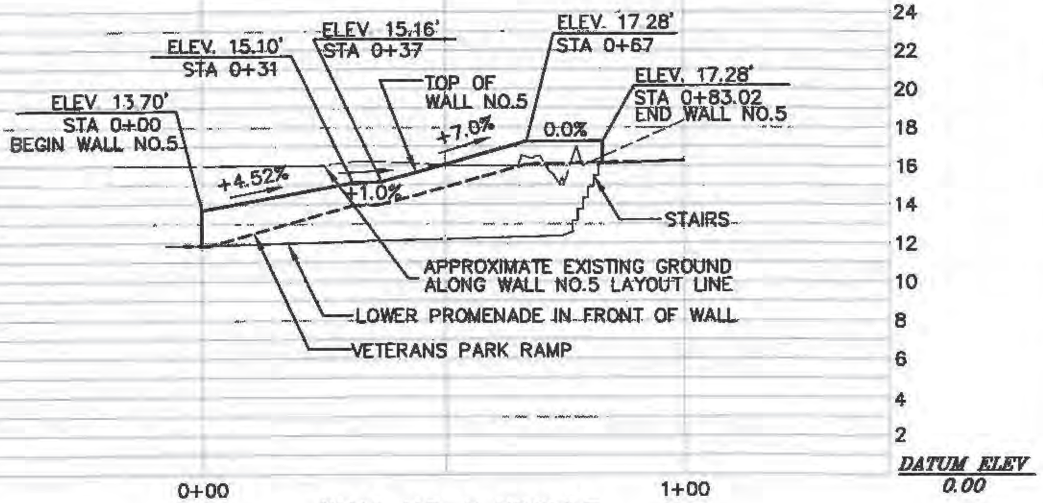


WALL NO. 4 PROFILE

SCALE: 1" = 20' H
1" = 5' V



WALL NO. 5 PROFILE

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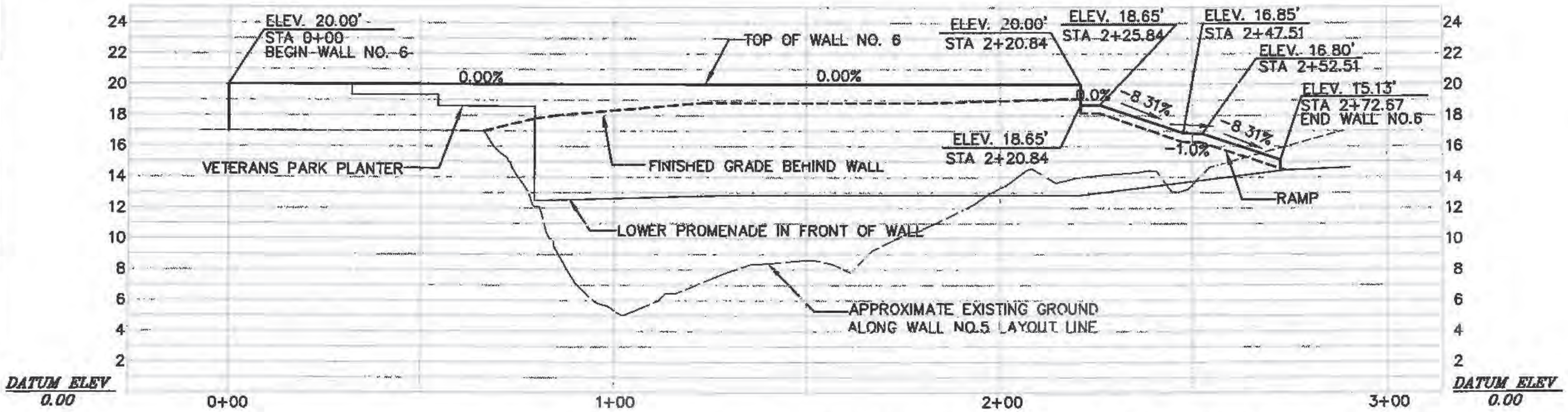
1" = 5' V

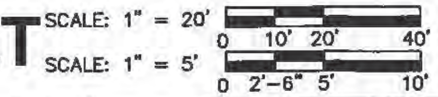


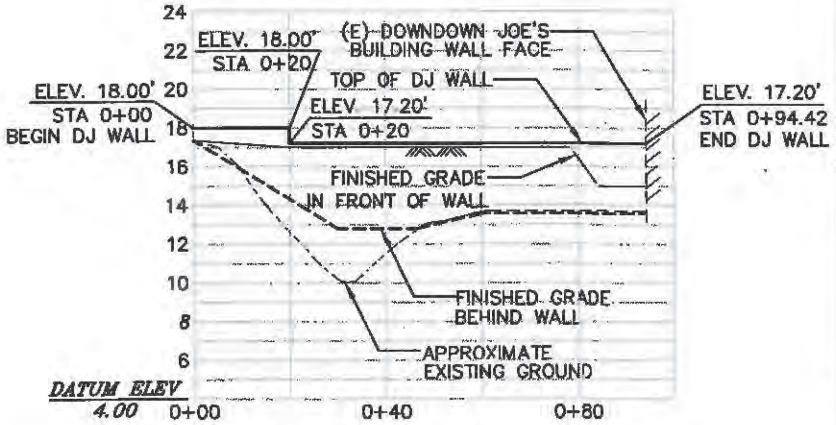
WALL NO. 6 PROFILE

SCALE: 1" = 20' H
1" = 5' V

REVISED AS-BUILT

GRAPHIC SCALE





DOWNTOWN JOE'S RETAINING WALL

SCALE: 1" = 20' H
1" = 5' V

SEE C-207 FOR PROFILE LEGEND

US Army Corps
of Engineers
Sacramento District

Symbol	Description	Date	Approved
1	CHANGES PER RLB – TO BETTER SUIT WALL PROFILE ELEV.	12/02/06	

Designed by: JEFF S. CROMITZ
Dwn by: KLI
Spec No.: 1407
Reviewed by: R. SENNETT
Submitted by: /S/ J. CROMITZ
Date: 04/27/2005
Design file no: NA-25-030
Drawing Code:
Rev.

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

MGE ENGINEERING INC.
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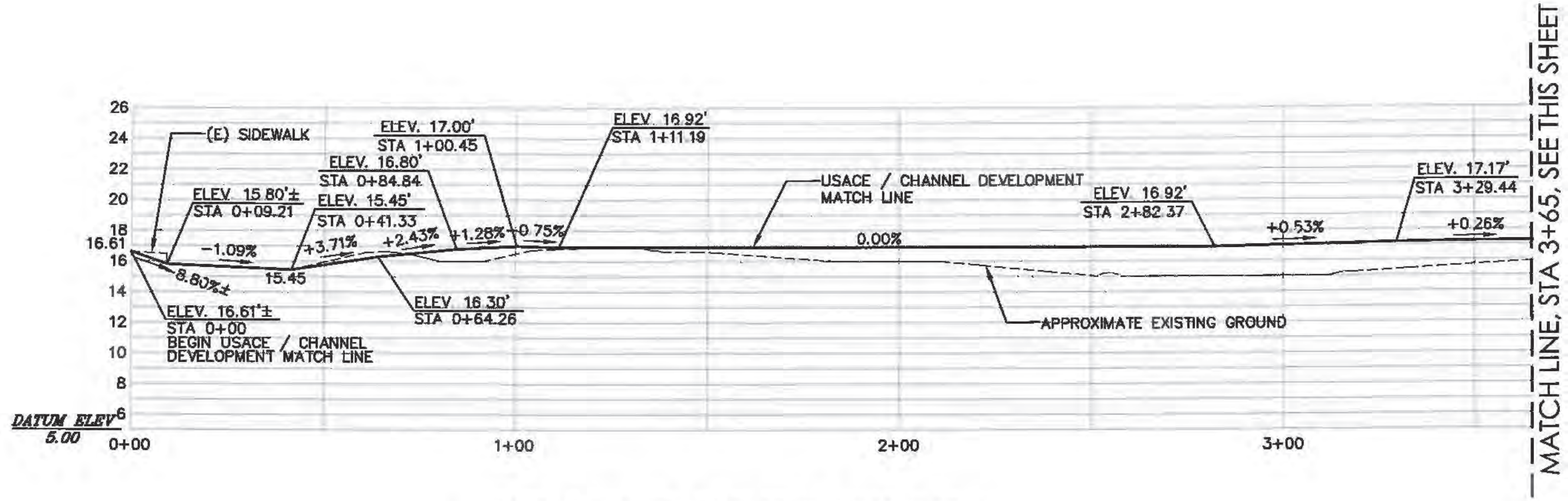
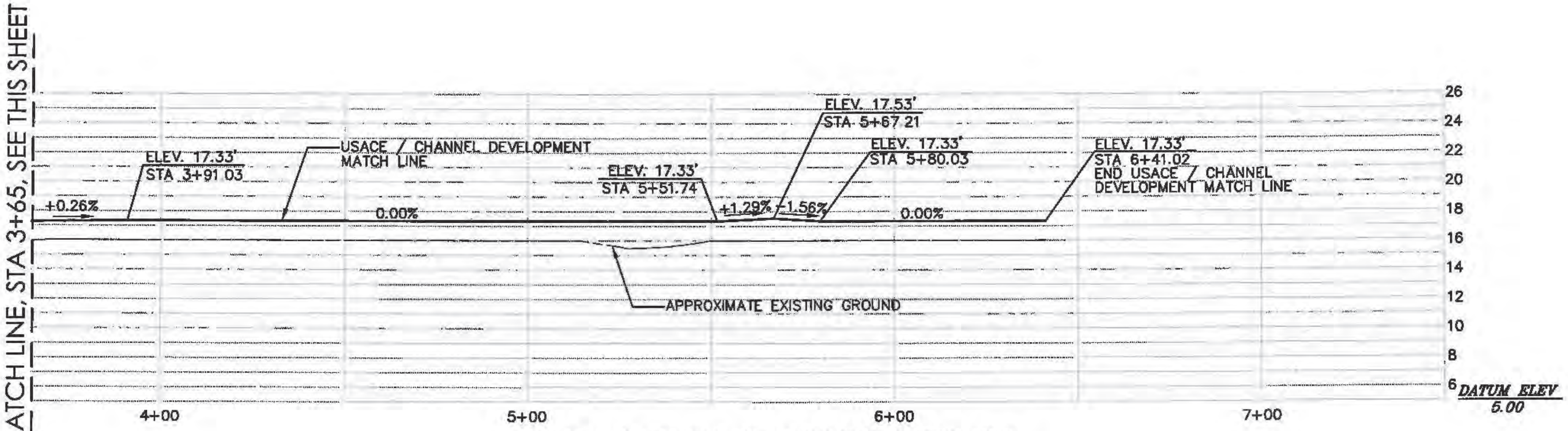
NAPA CALIFORNIA
NAPA RIVER/NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (HATT BUILDING
TO FIRST STREET)
WALL & PROMENADE
PROFILES NO. 3

Sheet reference number:
C-209
Sheet 32 of 166



USACE / CHANNEL DEVELOPMENT MATCH LINE

SCALE: 1" = 20' H
1" = 5' V



USACE / CHANNEL DEVELOPMENT MATCH LINE

SCALE: 1" = 20' H
1" = 5' V

AS-BUILT

GRAPHIC SCALE

SCALE: 1" = 20' 0 10' 20' 40'

SCALE: 1" = 5' 0 2'-6" 5' 10'

SEE C-207 FOR PROFILE LEGEND

US Army Corps
of Engineers
Sacramento District

Designed by: JEFF S. CROWITZ
Dwn by: KLI
Spec No.: 1407
Date: 04/27/2005
Design file no: NA-25-030
Drawing Code:
Reviewed by: R. SENNETT
Submitted by: /S/ J. CROWITZ

DEPARTMENT OF THE ARMY
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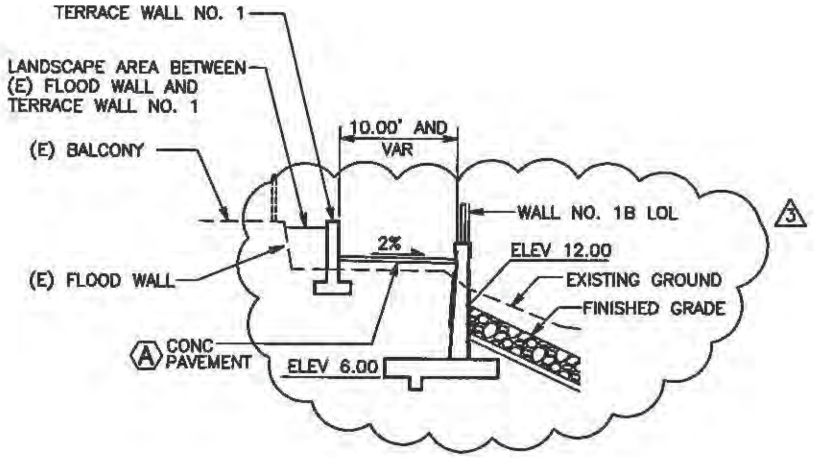
NAPA NAPA RIVER/NAPA CREEK CALIFORNIA
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (HATT BUILDING
TO FIRST STREET)

USACE / CHANNEL DEVELOPMENT
MATCH LINE PROFILES

Sheet reference number:
C-210
Sheet 33 of 166



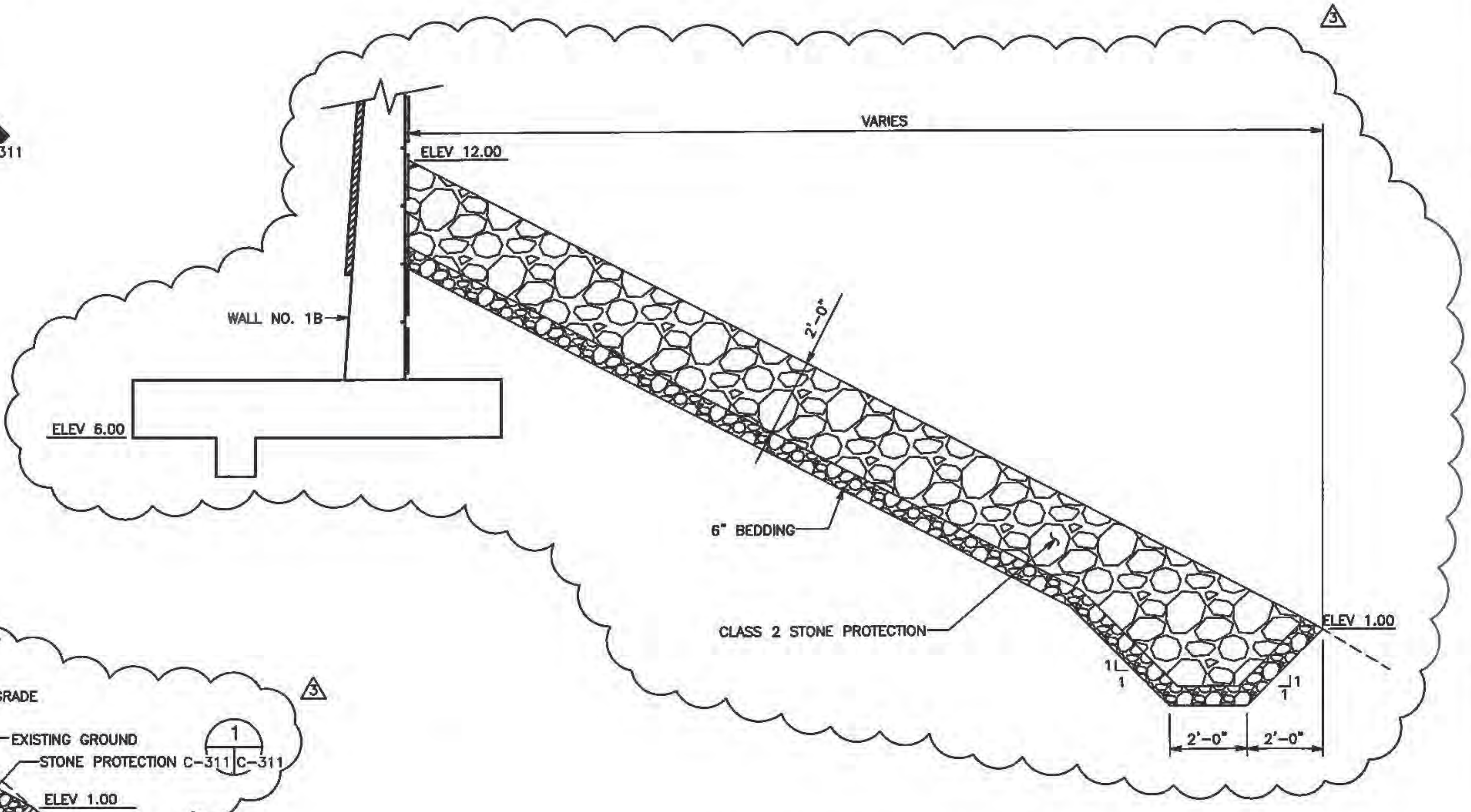
SECTION @ STA 1+31.00 ALONG WALL NO. 1B LOL

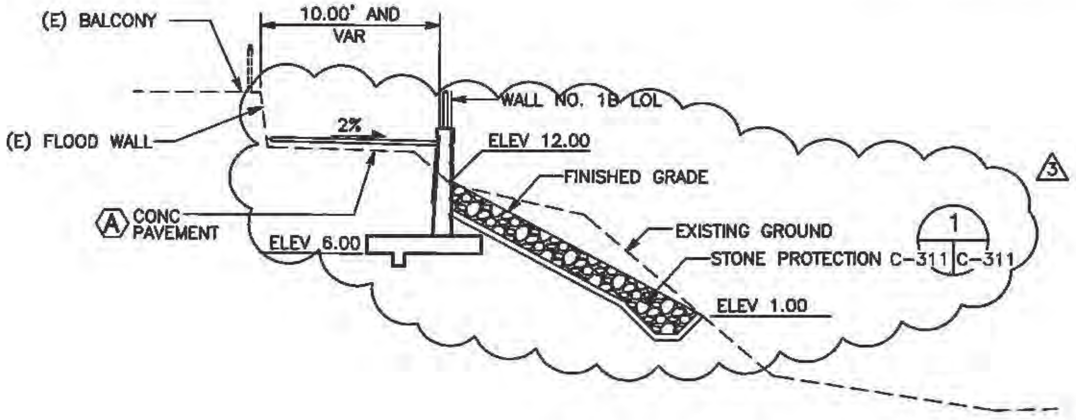
SCALE: 1/8"=1'-0"

B C-101A C-311

TABLE OF ANTICIPATED WATER TIDAL DATUMS (NOAA 1994)	
EVENT	WATER SURFACE ELEV. (FEET)
100-YEAR FLOOD (NO FREEBOARD)	16.60
10-YEAR FLOOD	10.00
MEAN HIGHER HIGH WATER (MHHW)	3.96
MEAN HIGH WATER (MHW)	3.44
MEAN TIDAL LEVEL (MTL)	0.87
MEAN LOW WATER (MLW)	-1.70
MEAN LOWER LOW WATER (MLLW)	-2.64

NOTE:
CONSTRUCTION OF FLOOD WALLS MAY REQUIRE SHORING AND CONTROL OF WATER





STONE PROTECTION DETAIL

SCALE: 1/2"=1'-0"

1 C-311 C-311

SECTION @ STA 1+10.00 ALONG WALL NO. 1B LOL

SCALE: 1/8"=1'-0"

A C-101A C-311

AMENDMENT
NO. 6, 03/15/2007

GRAPHIC SCALE
8' 4' 0 8'
SCALE:1/8"=1'-0"
2' 1' 0 2'
SCALE:1/2"=1'-0"

US Army Corps
of Engineers
Sacramento District

Symbol	Description		Date	Approved
3	AM/006 - REVISED STONE PROTECTION AND LOWERED WALL FOOTING	JSC	03/15/07	
2	AM/005 - REVISED SIDEWALK	JSC	12/28/06	
1	ADDED HANDRAIL & NOTE			

Designed by: Date: Rev.
Dwn by: Spec No.: Design file no:
Reviewed by: Drawing Code:
Submitted by: File name: Plot date: Dwg scale:

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

BEI ENGINEERING INC.
900 East Main Street, Suite 100
Grass Valley, California 95945
530.477.9960

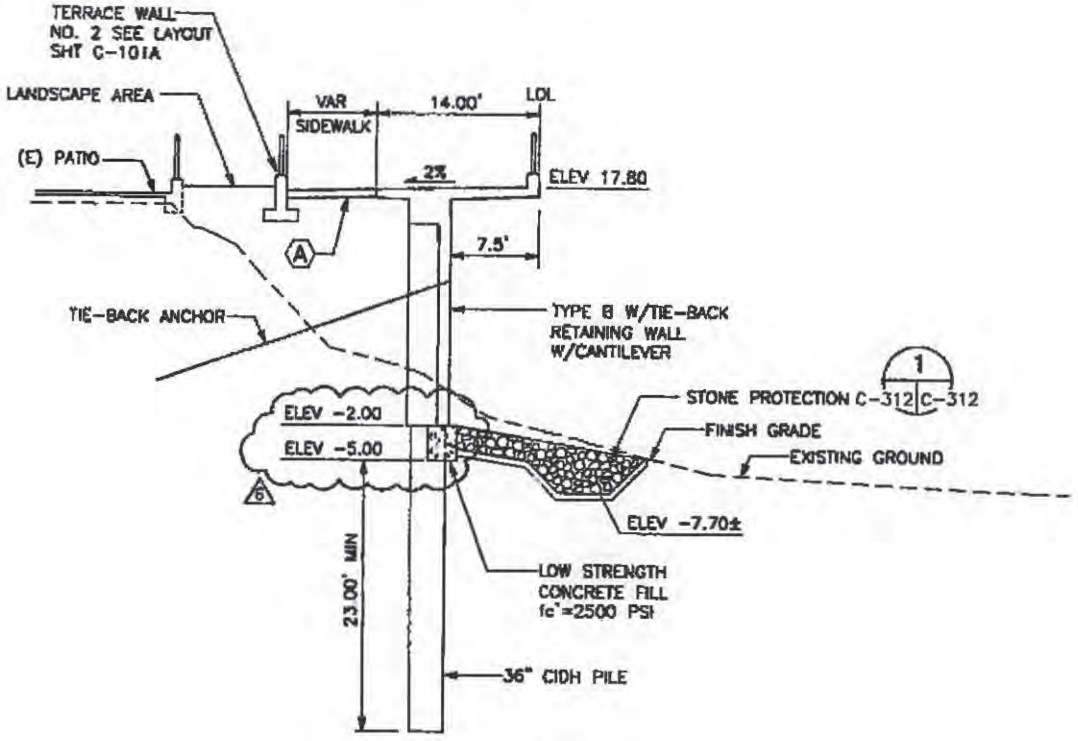
NAPA CALIFORNIA
NAPA RIVER/NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (BROWN STREET
TO FIFTH STREET)
(VALUE ENGINEERING PLANS)

TYPICAL CROSS SECTIONS NO. 1

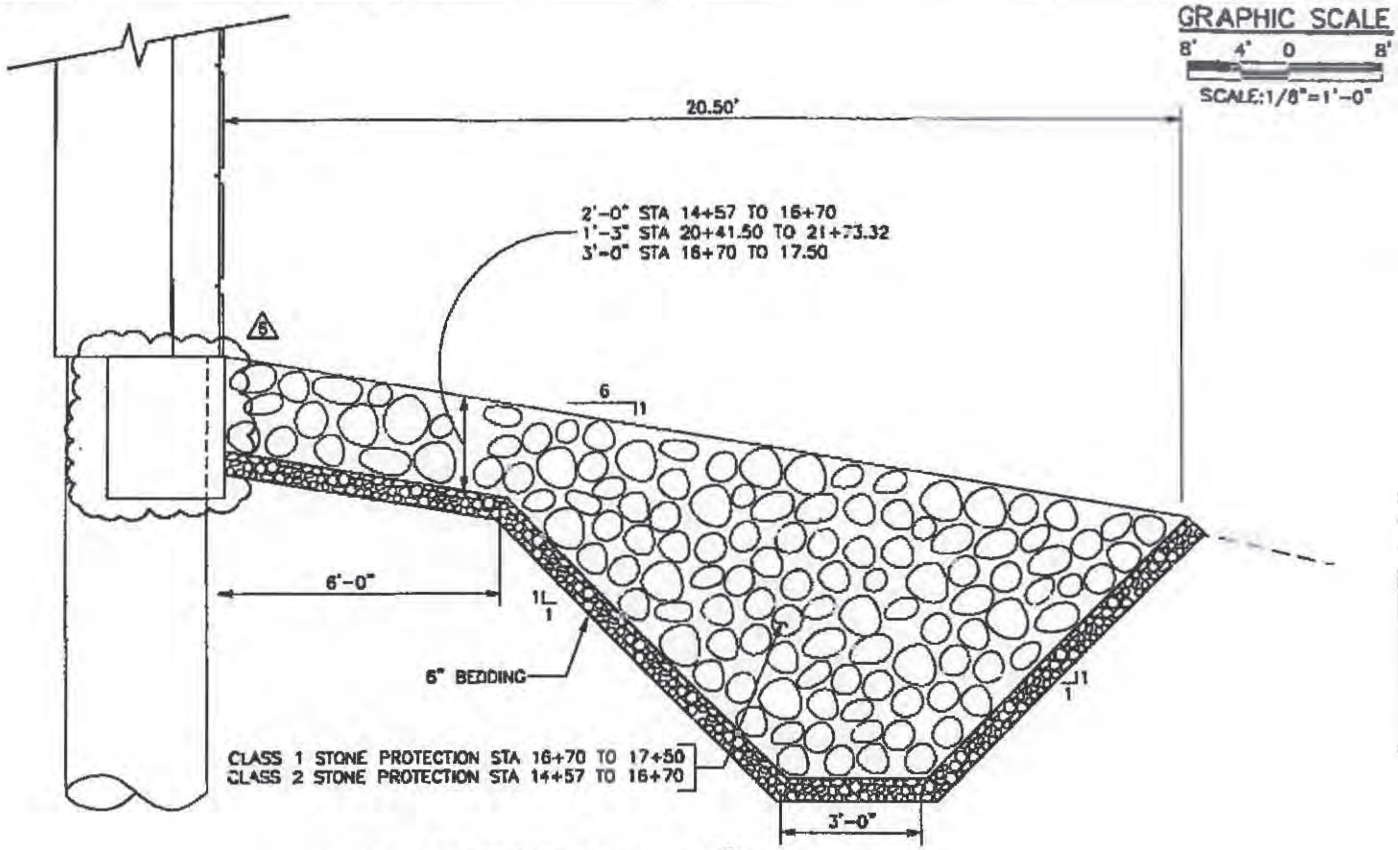
Sheet reference number:
C-311
Sheet 34 of 166

AM-006

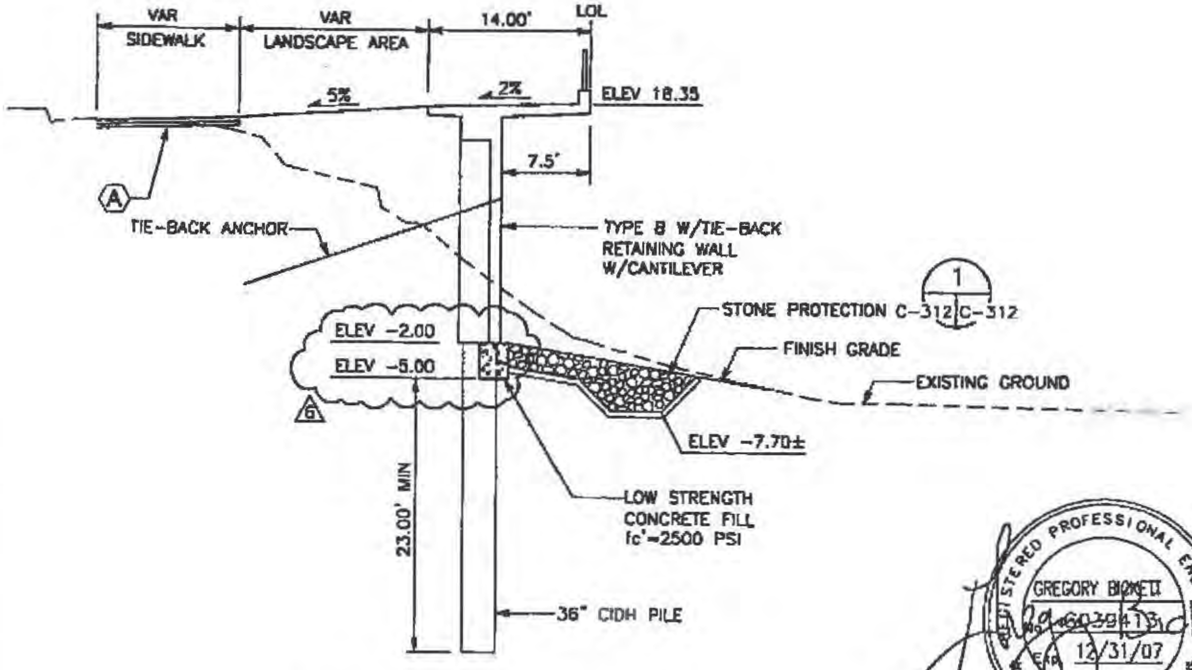
GRAPHIC SCALE
8' 4' 0 8'
SCALE: 1/8"=1'-0"



SECTION @ STA 15+00 D
SCALE: 1/8"=1'-0" C-101A C-312



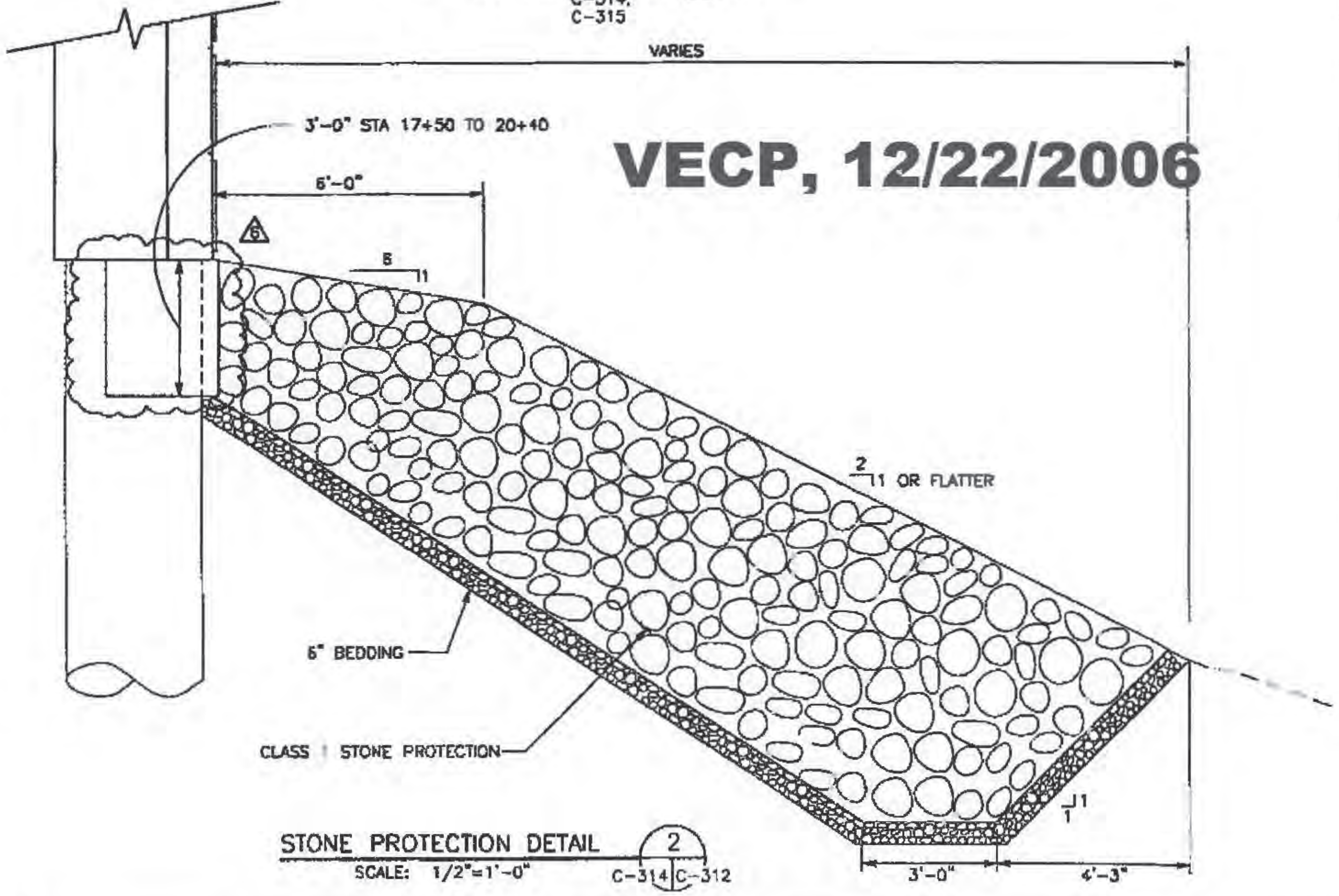
STONE PROTECTION DETAIL 1
SCALE: 1/2"=1'-0" C-312, C-313 C-312
C-314,
C-315



SECTION @ STA 14+60 C
SCALE: 1/8"=1'-0" C-101A C-312

REGISTERED PROFESSIONAL ENGINEER
GREGORY BICKETT
EXP 12/31/07
CIVIL
STATE OF CALIFORNIA

VECP, 12/22/2006



STONE PROTECTION DETAIL 2
SCALE: 1/2"=1'-0" C-314 C-312

US Army Corps of Engineers Sacramento District

Symbol	Descriptions	Date	Approved
△	REVISED GRADE BEAM GEOMETRY	12/22/06	GGB
△	ADDED DIMENSION	09/08/06	GGB
△	REVISED STONE PROTECTION CONFIGURATION	09/08/06	GGB
△	REVISED STATION	09/08/06	GGB
△	REVISED WIDTH	09/08/06	GGB
△	REVISED GEOMETRY	09/08/06	GGB

Designed by:		Date:	Rev.
Dwn by:	Spec No.:	Design file no:	
Reviewed by:		Drawing Code:	
Submitted by:		Plot scale: Plot date: Dwg units:	

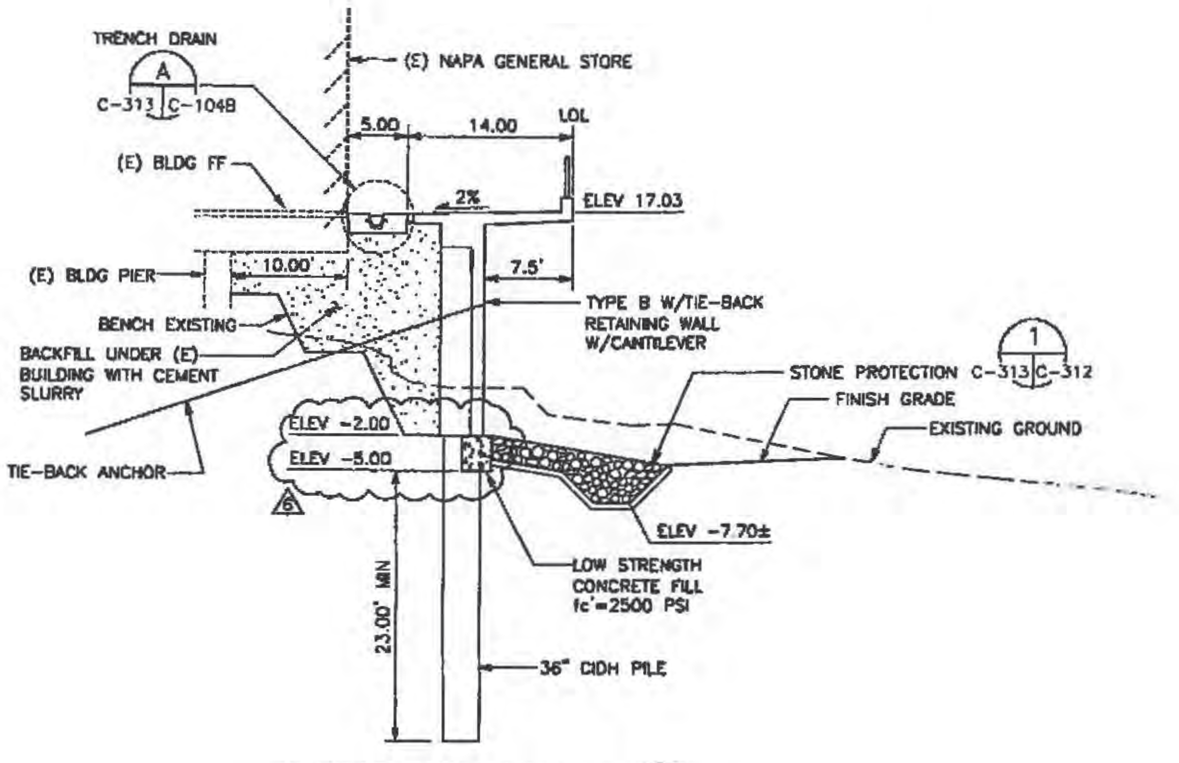
DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

BEI ENGINEERING INC.
300 East Main Street, Suite 100
Grass Valley, California 95945
530.477.9980

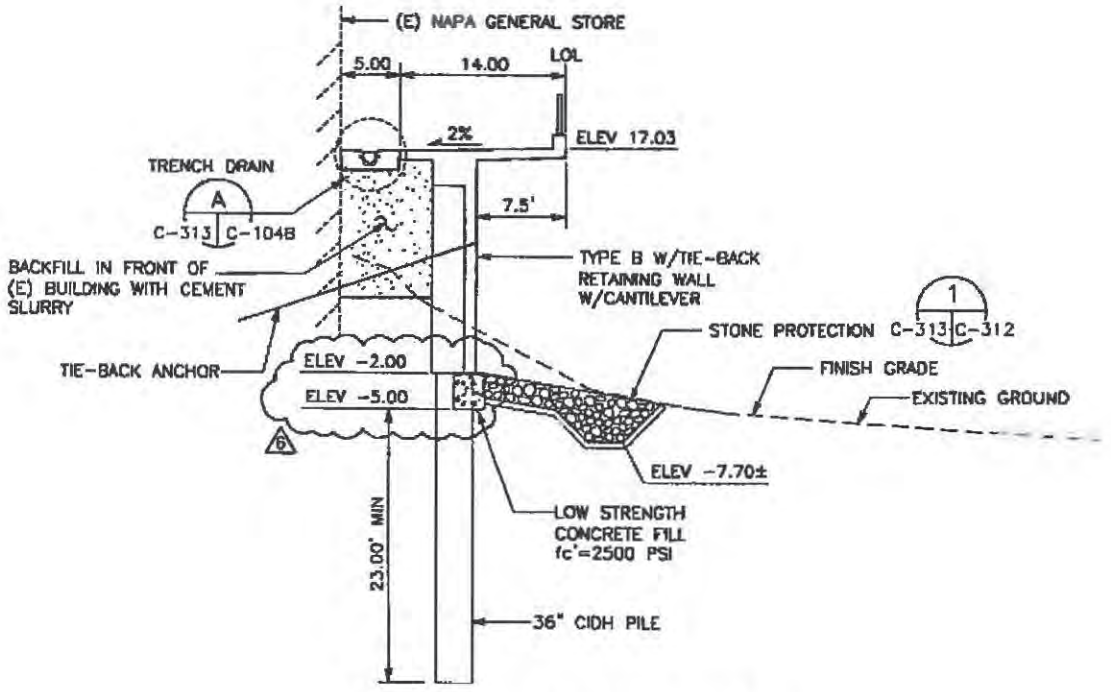
NAPA CALIFORNIA
NAPA RIVER/NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (BROWN STREET
TO FIFTH STREET)
(VALUE ENGINEERING PLANS)

TYPICAL CROSS SECTIONS NO. 2

Sheet reference number:
C-312
Sheet of



SECTION @ STA 16+50
SCALE: 1/8"=1'-0"
F
C-101A | C-313



SECTION @ STA 15+50
SCALE: 1/8"=1'-0"
E
C-101A | C-313

VECP, 12/22/2006

REGISTERED PROFESSIONAL ENGINEER
GREGORY BICKET
C039413
Exp. 12/31/07
CIVIL
STATE OF CALIFORNIA

GRAPHIC SCALE
8' 4' 0 8'
SCALE 1/8"=1'-0"

US Army Corps of Engineers
Sacramento District

Symbol	Descriptions	Date	Approved
A	REVISED GRADE BEAM GEOMETRY	12/22/06	GB3

Designed by:
Dwn by:
Spec No.:
Date:
Rev.
Reviewed by:
Design file no:
Drawing Code:
Submitted by:
File name:
Plot date:

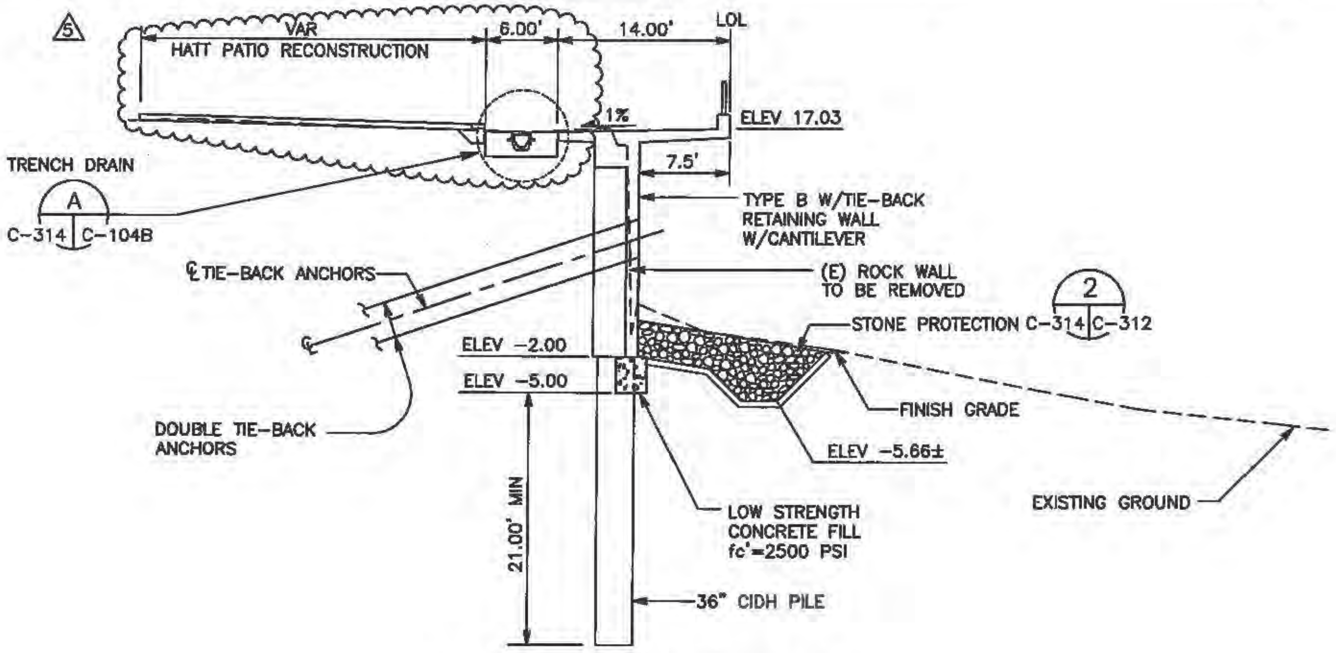
DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

3E ENGINEERING INC.
901 East Main Street, Suite 100
Grass Valley, California 95945
530.477.8080

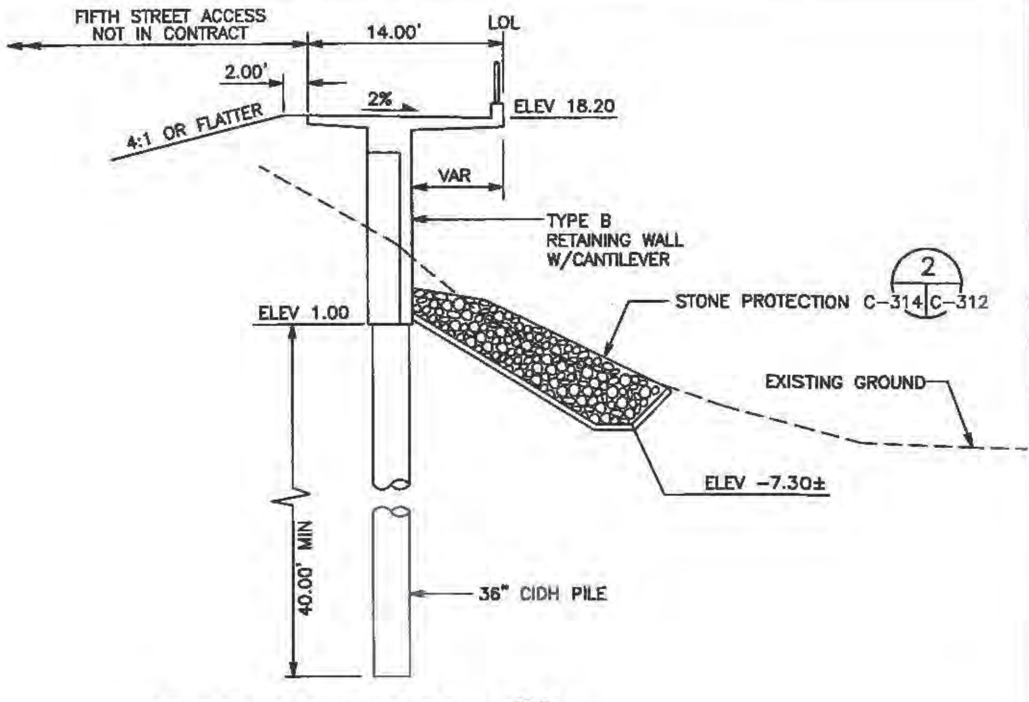
NAPA
CALIFORNIA
NAPA RIVER/NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (BROWN STREET
TO FIFTH STREET)
(VALUE ENGINEERING PLANS)

TYPICAL CROSS SECTIONS NO. 3

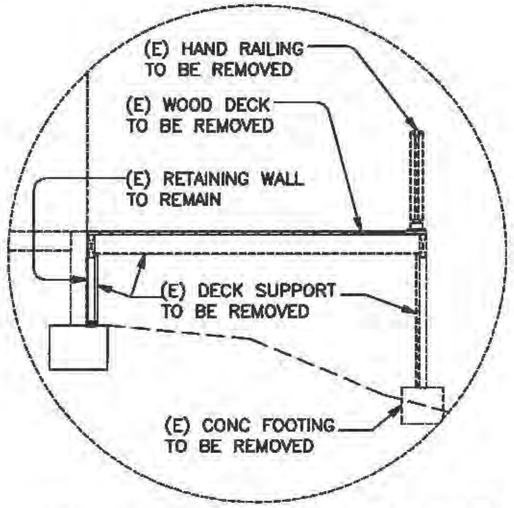
Sheet reference number:
C-313
Sheet of



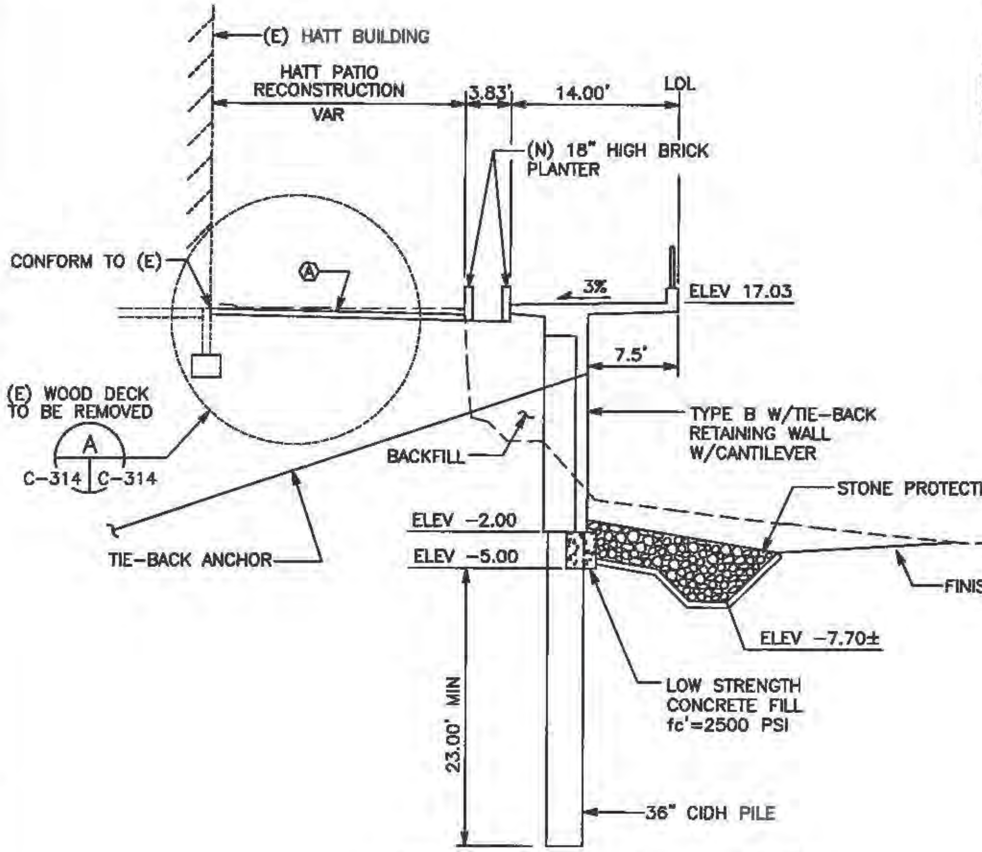
SECTION @ STA 17+68
SCALE: 1/8"=1'-0"
H
C-101B | C-314



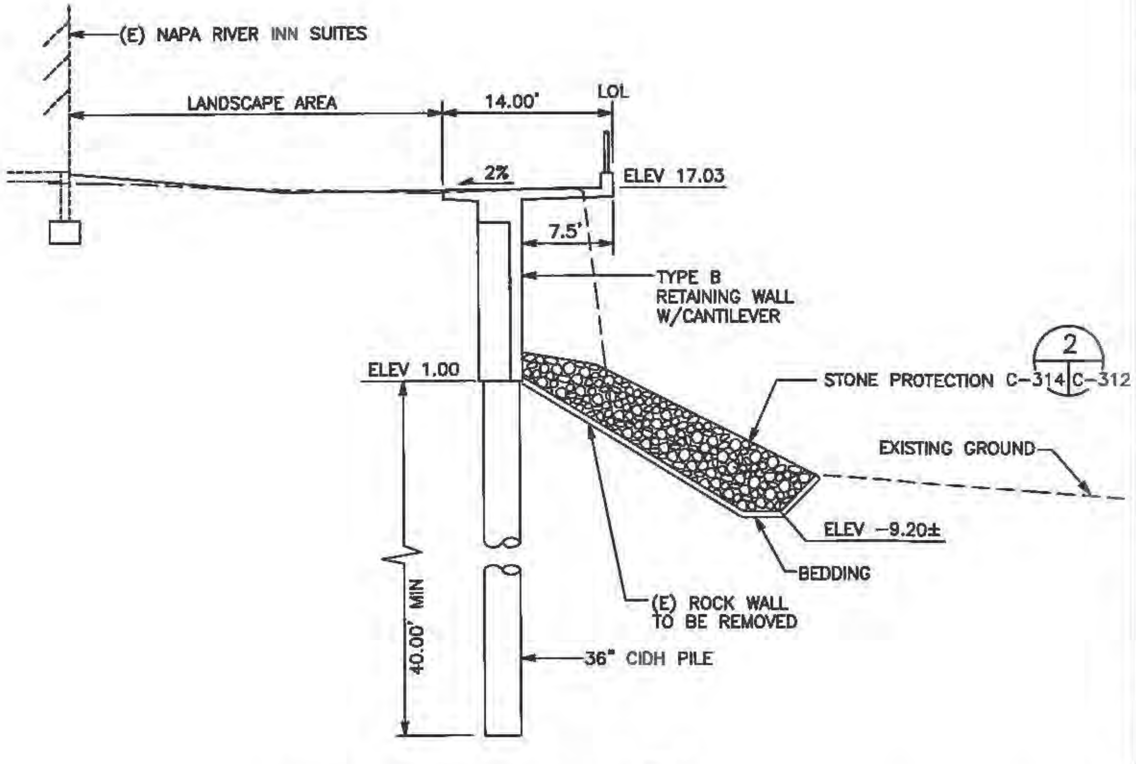
SECTION @ STA 19+73
SCALE: 1/8"=1'-0"
1
J
C-101B | C-314



EXISTING DECK AT HATT PATIO
NOT TO SCALE
A
C-314 | C-314



SECTION @ STA 16+85
SCALE: 1/8"=1'-0"
G
C-101A | C-314



SECTION @ STA 18+75
SCALE: 1/8"=1'-0"
I
C-101B | C-314

GRAPHIC SCALE
8' 4' 0 8'
SCALE: 1/8"=1'-0"

VECP, 02/01/2007

US Army Corps of Engineers
Sacramento District

Symbol	Description	Date	Approved
1	REVISED SECTION	02/01/07	GGB
2	REVISED GRADE BEAM GEOMETRY	12/22/08	GGB
3	REVISED GEOMETRY	08/08/08	GGB
4	REVISED STONE PROTECTION CONFIGURATION	08/08/08	GGB
5	CORRECTED SHEET REFERENCE	08/08/08	GGB

Designed by:
Dwn by:
Spec No.:
Date:
Rev.
Design file no:
Reviewed by:
Drawing Code:
Submitted by:
File name:
Plot date:
Dwg scale:

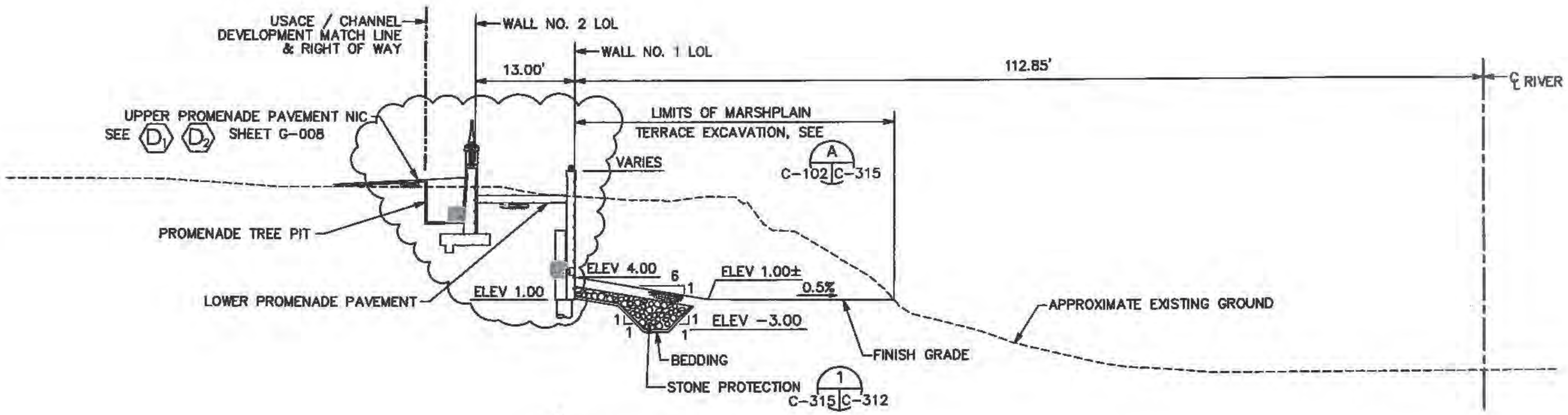
DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

BEI ENGINEERING INC.
900 East Main Street, Suite 100
Grass Valley, California 95945
530.477.9960

NAPA CALIFORNIA
NAPA RIVER/NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (BROWN STREET
TO FIFTH STREET)
(VALUE ENGINEERING PLANS)

TYPICAL CROSS SECTIONS NO. 4

Sheet reference number:
C-314
Sheet of 166
VE

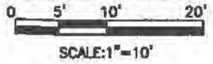


2

SECTION ⊕ STA 20+91.50 A
SCALE: 1"=10' C-102 C-315

VECP, 09/06/2006

GRAPHIC SCALE



SCALE:1"=10'

US Army Corps of Engineers
Sacramento District

2 REVISED STATION
1 REVISED GEOMETRY

Symbol
Description
Date
Approval

Designed by:
Dwn by:
Reviewed by:
Submitted by:
Spec No.:
Date:
Design file no:
Drawing Code:
Rev.

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

BEI ENGINEERING INC.
900 East Main Street, Suite 100
Grass Valley, California 95945
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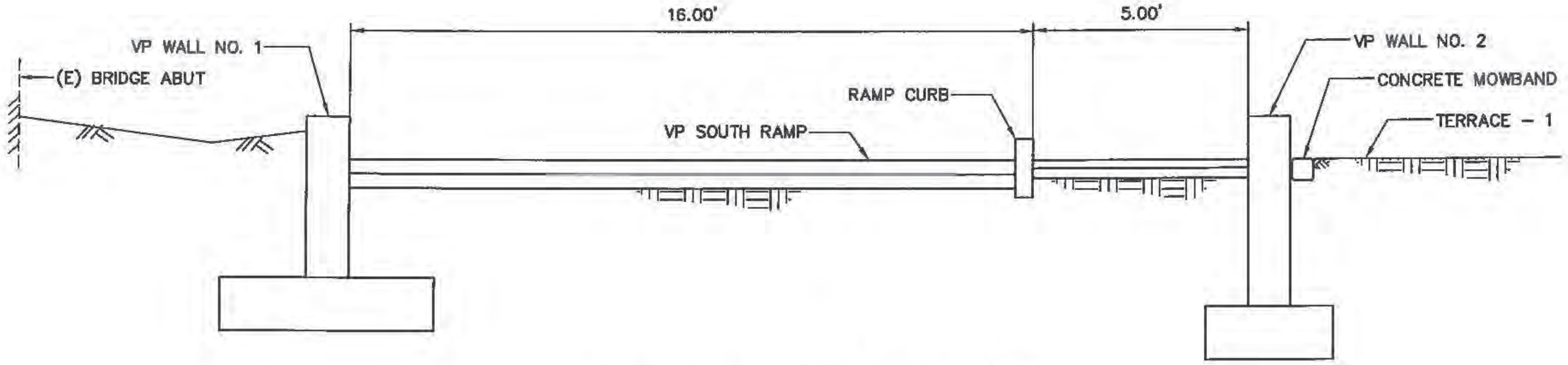
NAPA CALIFORNIA
NAPA RIVER/NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (BROWN STREET
TO FIFTH STREET)
(VALUE ENGINEERING PLANS)

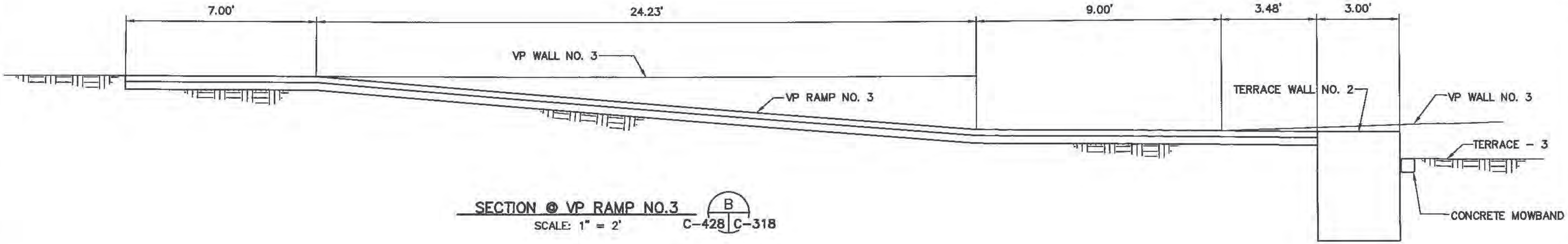
TYPICAL CROSS SECTIONS NO. 5

Sheet reference number:
C-315
Sheet of 166

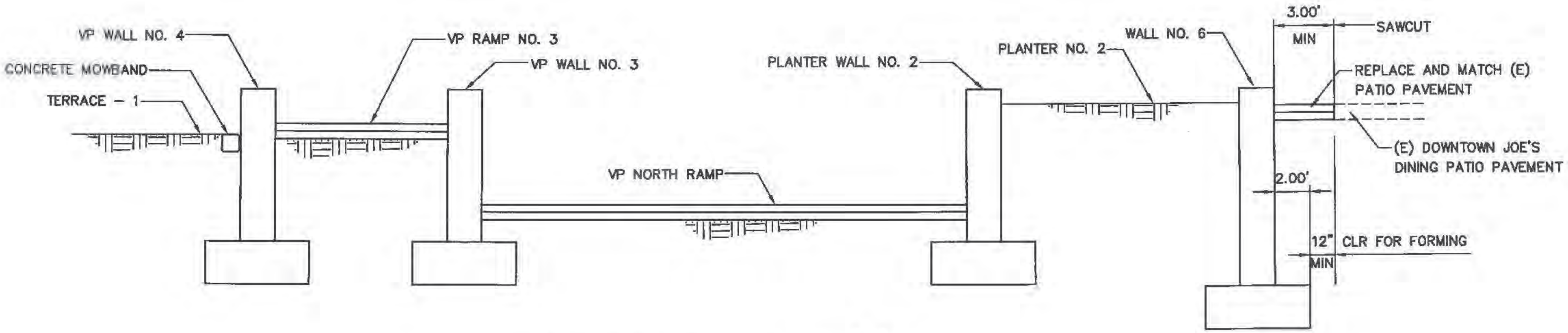
VE



SECTION @ VP SOUTH RAMP
SCALE: 1" = 2'
A
C–428 | C–318



SECTION @ VP RAMP NO.3
SCALE: 1" = 2'
B
C–428 | C–318



SECTION @ VP NORTH RAMP
SCALE: 1" = 2'
C
C–428 | C–318

AS-BUILT

GRAPHIC SCALE
SCALE: 1" = 2'
0 1' 2' 4'

US Army Corps of Engineers
Sacramento District

Symbol	Description	Date	Approved

Designed by: JEFF S. CROWITZ
Dwn by: KJ
Spec No.: 1407
Reviewed by: R. SENNETT
Submitted by: /S/ J. CROWITZ
Date: 04/27/2005
Rev.
Design file no: NA-25-030
Drawing Code:
File name:
Plot date:
Dwg scale:

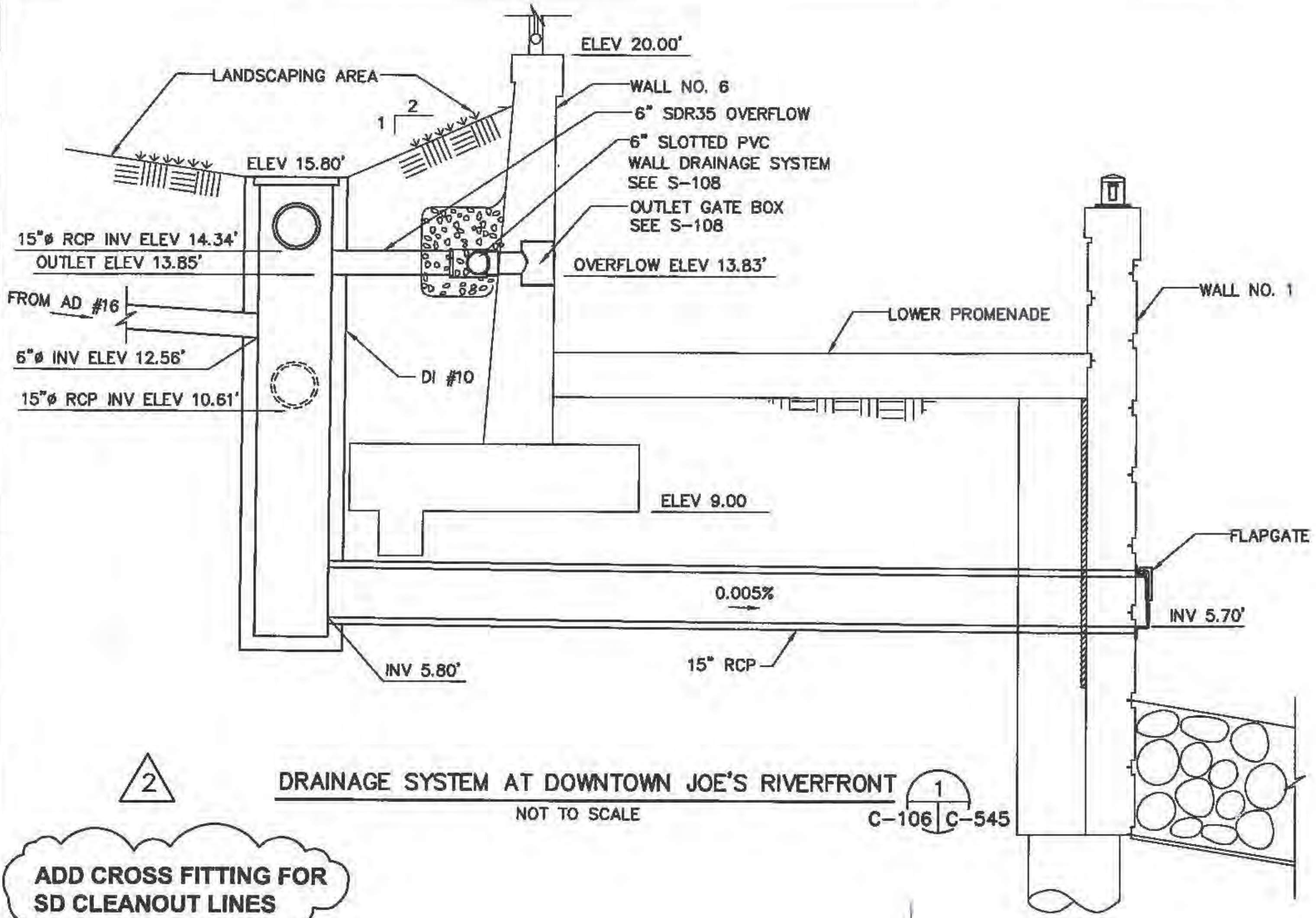
DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

MGE ENGINEERING INC.
PARK 5 Greenhaven Dr. Suite 100
Sacramento, California 95831
916.421.1000

NAPA RIVER/NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (HATT BUILDING TO FIRST STREET)
NAPA CALIFORNIA

TYPICAL SECTIONS AT VETERANS PARK

Sheet reference number:
C–318
Sheet 41 of 166

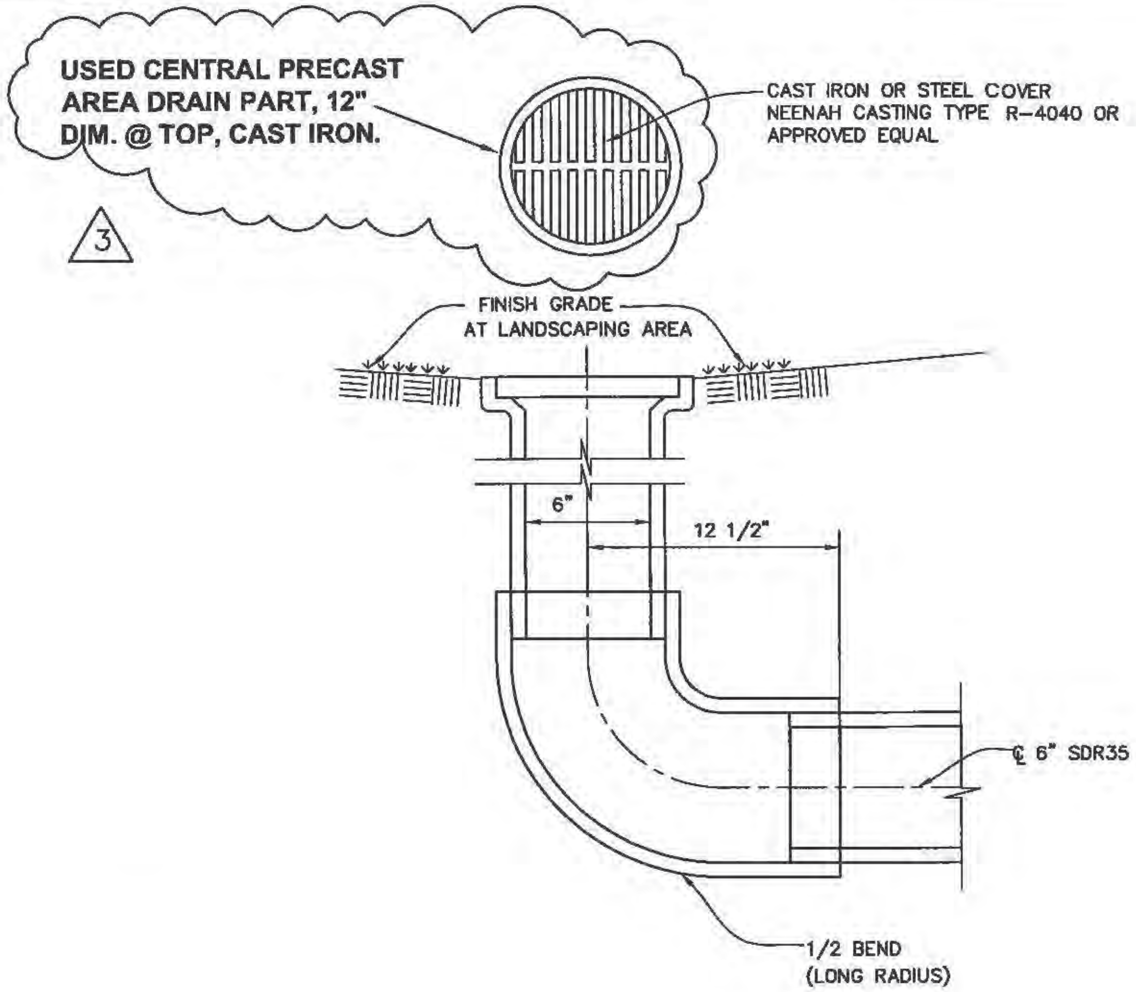


△2

DRAINAGE SYSTEM AT DOWNTOWN JOE'S RIVERFRONT

NOT TO SCALE

1 C-106 | C-545



USED CENTRAL PRECAST AREA DRAIN PART, 12" DIM. @ TOP, CAST IRON.

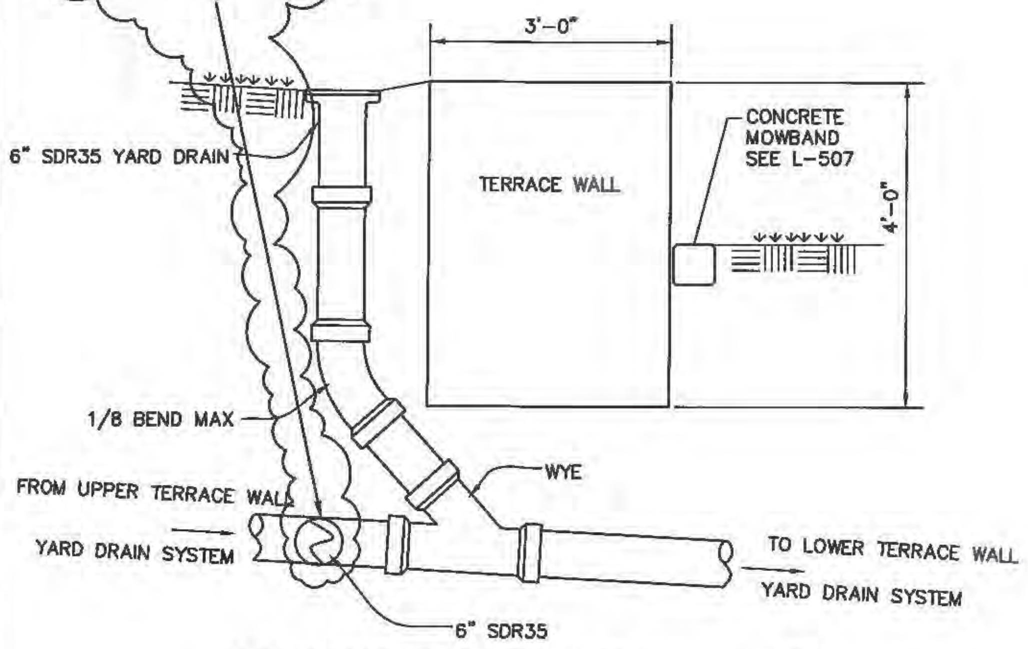
△3

AREA DRAIN

NOT TO SCALE

2 C-104 | C-545
C-106

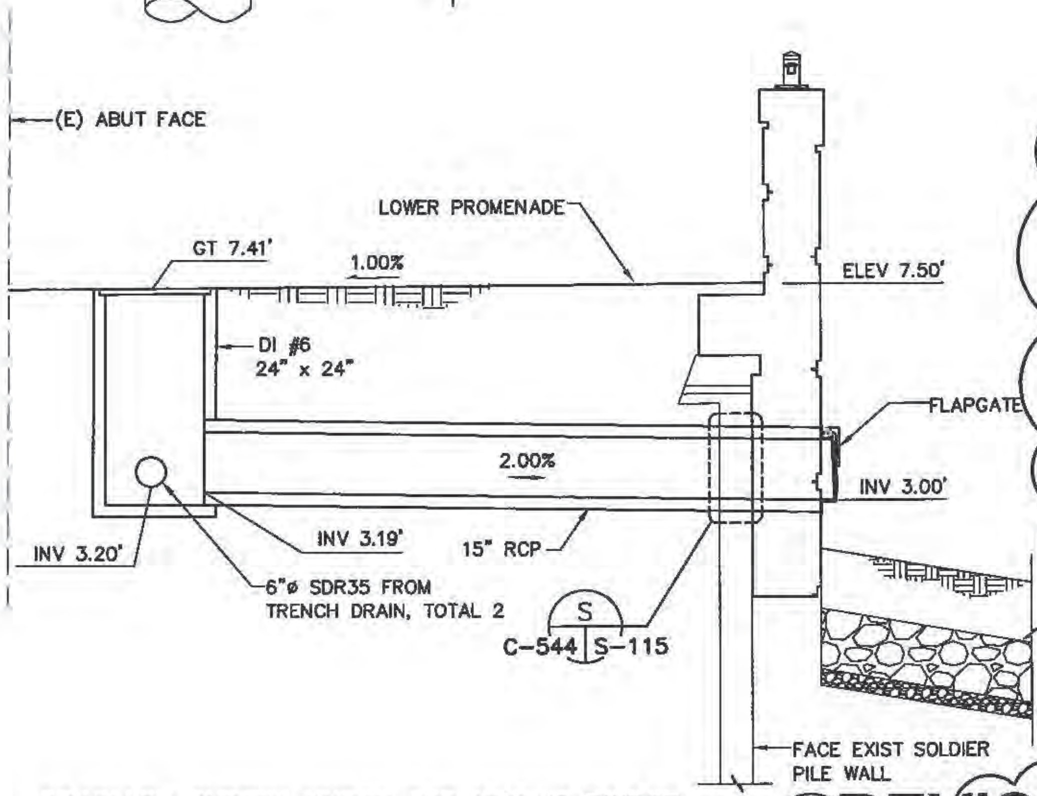
ADD CROSS FITTING FOR SD CLEANOUT LINES



AREA DRAIN BEHIND TERRACE WALL

NOT TO SCALE

3 C-106 | C-545

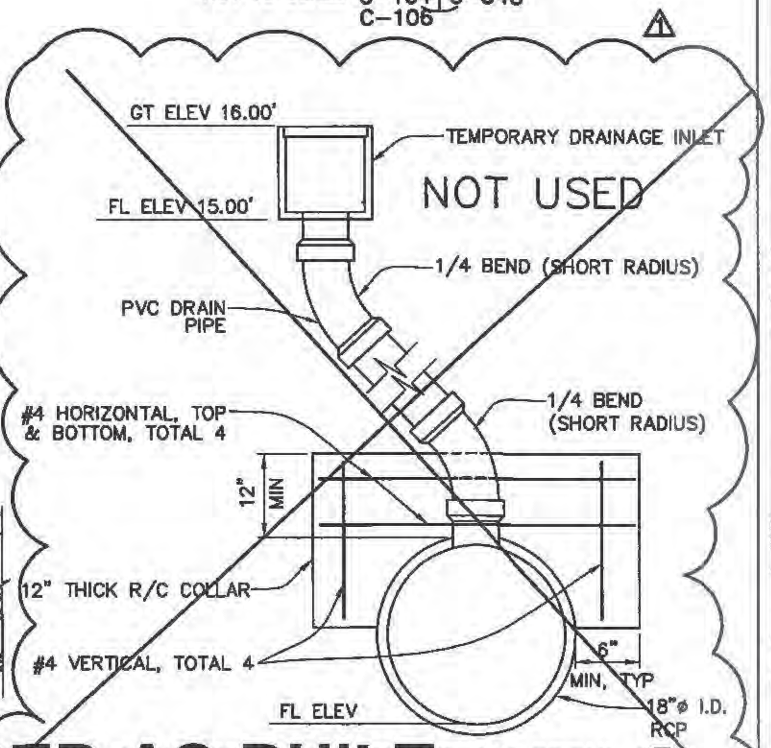


DRAINAGE SYSTEM UNDER 3RD STREET BRIDGE

NOT TO SCALE

4 C-105 | C-545

△1



TEMPORARY DRAINAGE INLET TAPPING RCP DETAIL

NOT TO SCALE

5 C-104 | C-545

REVISED AS-BUILT

US Army Corps of Engineers
Sacramento District

Symbol	Description	Date	Approved
△	REVISION PER STORM DRAIN CHANGE	8/9/06	JSC
△	CHANGES PER RFI 91 - @ SD AND CO CROSSING		
△	SEE SUBMITTAL SEC 2630-13 FOR ADD'L DATA		

Designed by: JEFF S. CROMITZ
Dwn by: KLJ
Spec No.: 1407
Reviewed by: R. SENNETT
Submitted by: /S/ J. CROMITZ
Date: 04/27/2005
Design file no: NA-25-030
Drawing Code:
Rev.

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

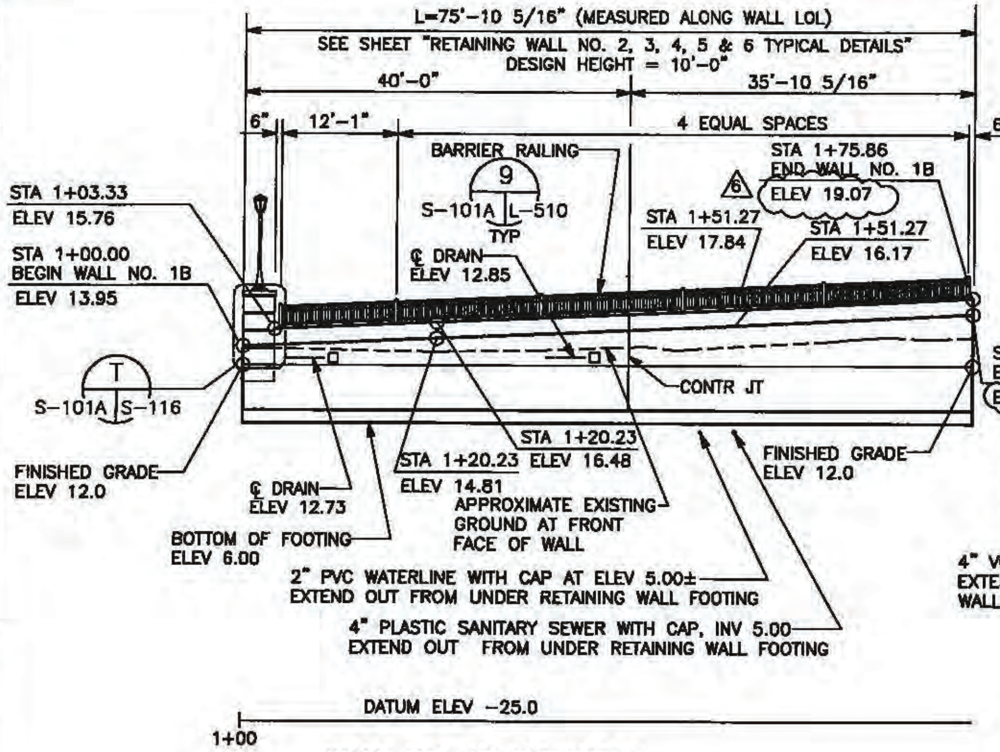
MGE ENGINEERING INC.
7415 Greenhaven Dr, Suite 100
Sacramento, California 95831
916.421.1000

NAPA
NAPA RIVER/NAPA CREEK CALIFORNIA
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (HATT BUILDING
TO FIRST STREET)

CONSTRUCTION DETAILS NO. 3

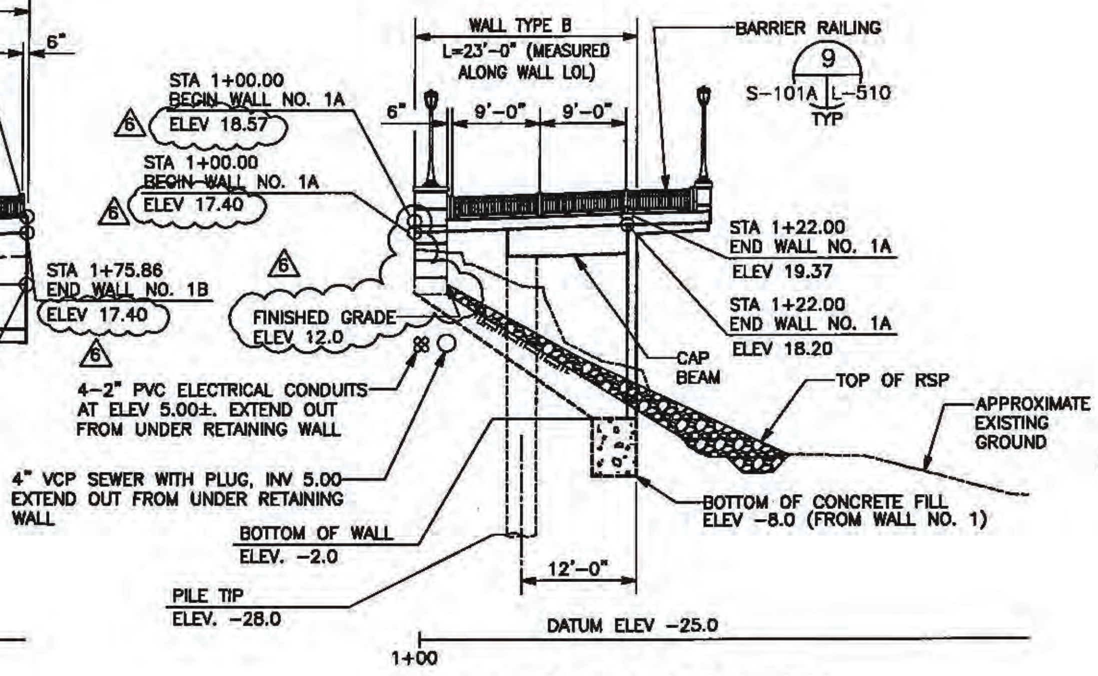
Sheet reference number:
C-545
Sheet 68 of 166

AM-003



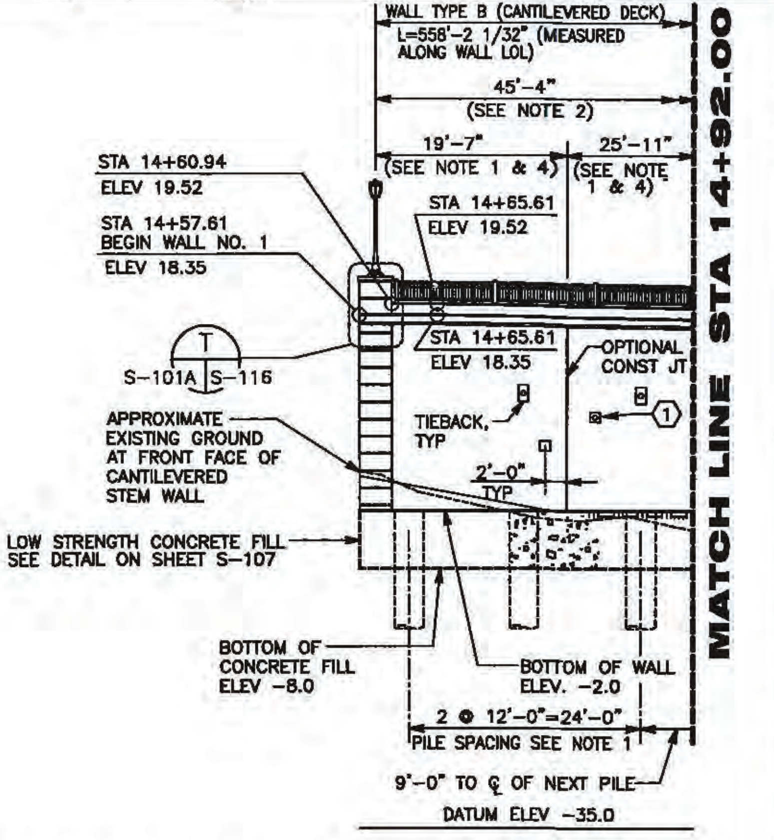
WALL NO. 1B ELEVATION

SCALE: 1"=10'-0"



WALL NO. 1A ELEVATION

SCALE: 1"=10'-0"



WALL NO. 1 ELEVATION

SCALE: 1"=10'-0"

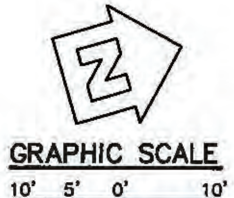
AMENDMENT NO. 9
07/03/2007

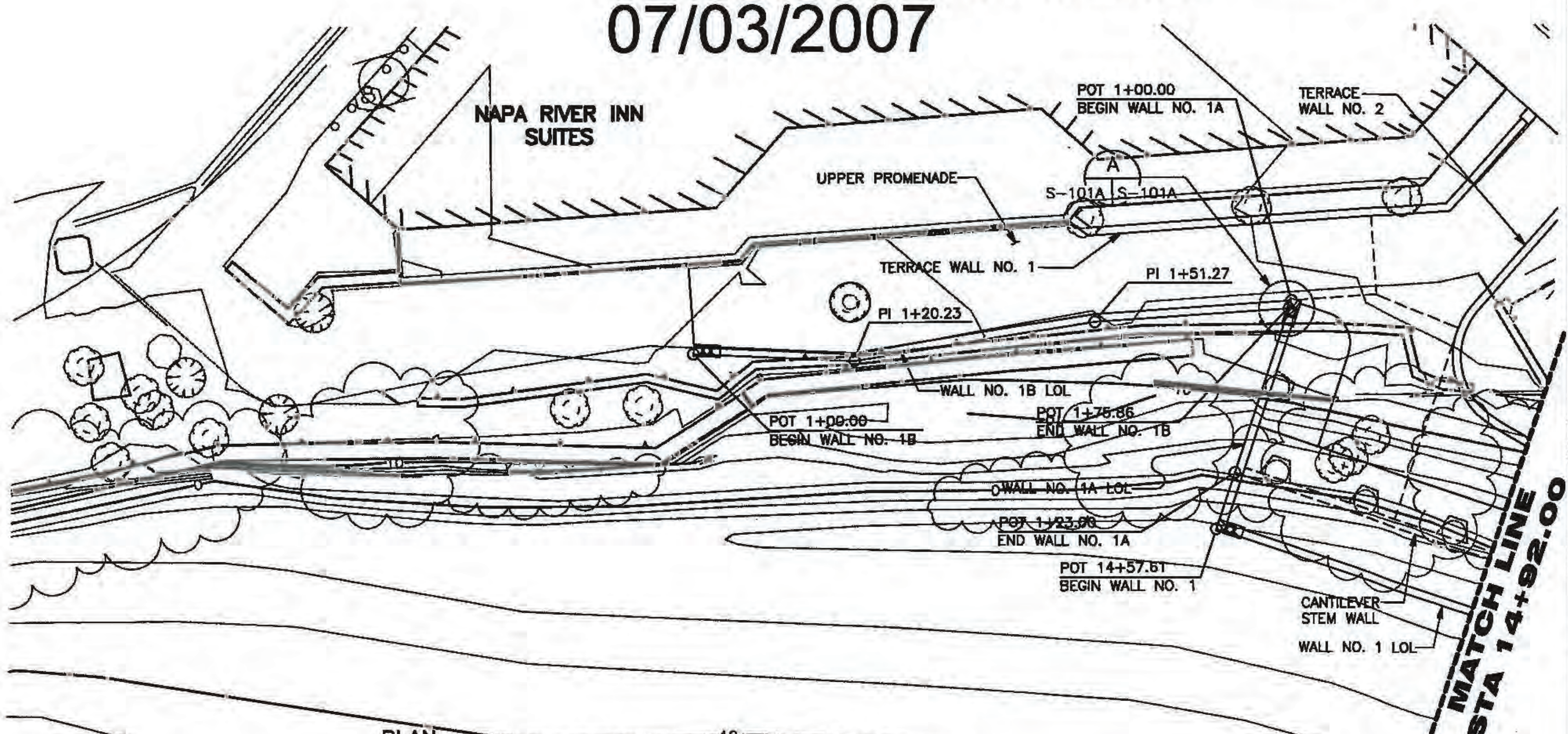
NOTES:

1. MEASURED ALONG FRONT FACE OF CANTILEVERED STEM WALL
2. LIGHT PILASTERS MEASURED ALONG WALL LOL. PILASTERS ON FRONT FACE OF CANTILEVERED STEM WALL ARE SPACED AT 45'-4" O.C. MEASURED ALONG FRONT FACE OF CANTILEVERED STEM WALL.
3. FOR LAYOUT OF SCORED BLOCK TEXTURED PATTERN, SEE SHEETS L519A THROUGH L-520
4. CONTR JT MAY BE SHIFTED 3'-0" FOR CLEARANCE WITH PILE & TIEBACK.

WALL PENETRATION REFERENCE NOTES:

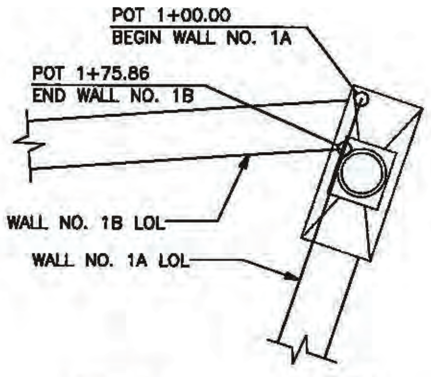
① 6" SDR35 STORM DRAIN PIPE, SEE C-104A





PLAN

SCALE: 1"=10'-0"



DETAIL A S-101A S-101A

SCALE: 1/2"=1'-0"

US Army Corps of Engineers Sacramento District

Symbol	Description	Date	Approved
△	AM#009 - REVISED GRADING OF SIDEWALK	JSC	07/03/07
△	AM#006 - REVISED STONE PROTECTION AND LOWERED WALL FOOTING	JSC	03/15/07
△	AM#005 - REVISED SIDEWALK	JSC	12/28/06
△	REVISE ELEVATIONS AND GRADES		
△	REMOVED PILASTER BELOW BOTTOM OF WALL		
△	ADDED NOTE		

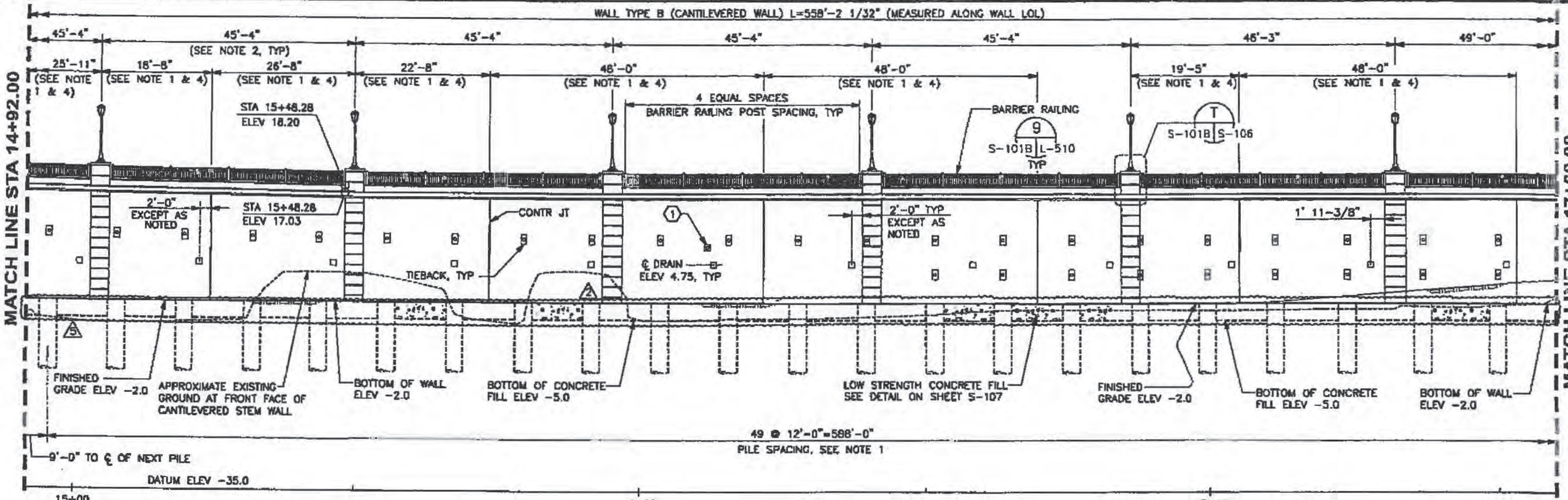
Designed by:		Date:	Rev.
Dwn by:	Spec No.:	Design file no:	
Reviewed by:		Drawing Code:	
Submitted by:		File name: Plot date: Dwg scale:	

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

BEI ENGINEERING INC.
900 East Main Street, Suite 100
Grass Valley, California 95945
530.477.9960

NAPA CALIFORNIA
NAPA RIVER/NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (BROWN STREET TO FIFTH STREET)
(VALUE ENGINEERING PLANS)
WALL NO. 1 AND 1A
PLAN AND ELEVATION NO. 1A

Sheet reference number:
S-101A
Sheet 73A of 166
AM-009



ELEVATION
SCALE: 1"=10'-0"

NOTES:

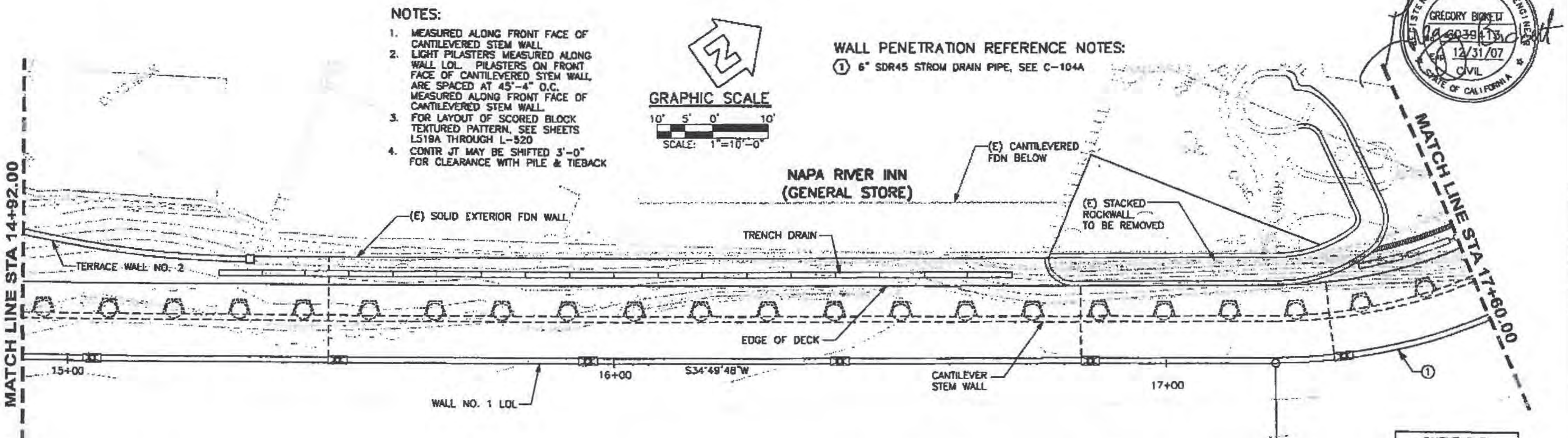
1. MEASURED ALONG FRONT FACE OF CANTILEVERED STEM WALL
2. LIGHT PILASTERS MEASURED ALONG WALL LOL. PILASTERS ON FRONT FACE OF CANTILEVERED STEM WALL, ARE SPACED AT 45'-4" O.C. MEASURED ALONG FRONT FACE OF CANTILEVERED STEM WALL.
3. FOR LAYOUT OF SCORED BLOCK TEXTURED PATTERN, SEE SHEETS L519A THROUGH L-520
4. CONTR JT MAY BE SHIFTED 3'-0" FOR CLEARANCE WITH PILE & TIEBACK

GRAPHIC SCALE
10' 5' 0' 10'
SCALE: 1"=10'-0"

WALL PENETRATION REFERENCE NOTES:

① 6" SDR45 STROM DRAIN PIPE, SEE C-104A

GREGORY BIRKETT
C 60394
12/31/07
CIVIL



PLAN
SCALE: 1"=10'-0"

CURVE DATA	
	①
R	100.00'
L	93.64'
T	50.57'
Δ	53° 39' 14"

VECP, 12/22/2006

US Army Corps of Engineers
Sacramento District

△	REVISED GRADE BEAM GEOMETRY	12/22/06	DGB
△	ADDED NOTE	09/06/06	DGB
△	REMOVED EXP JT	09/06/06	DGB
△	REMOVED PILASTER BELOW BOTTOM OF WALL	09/06/06	DGB
△	MOVE STORM DRAIN	09/06/06	DGB
Symbol	Descriptions	Date	Approved

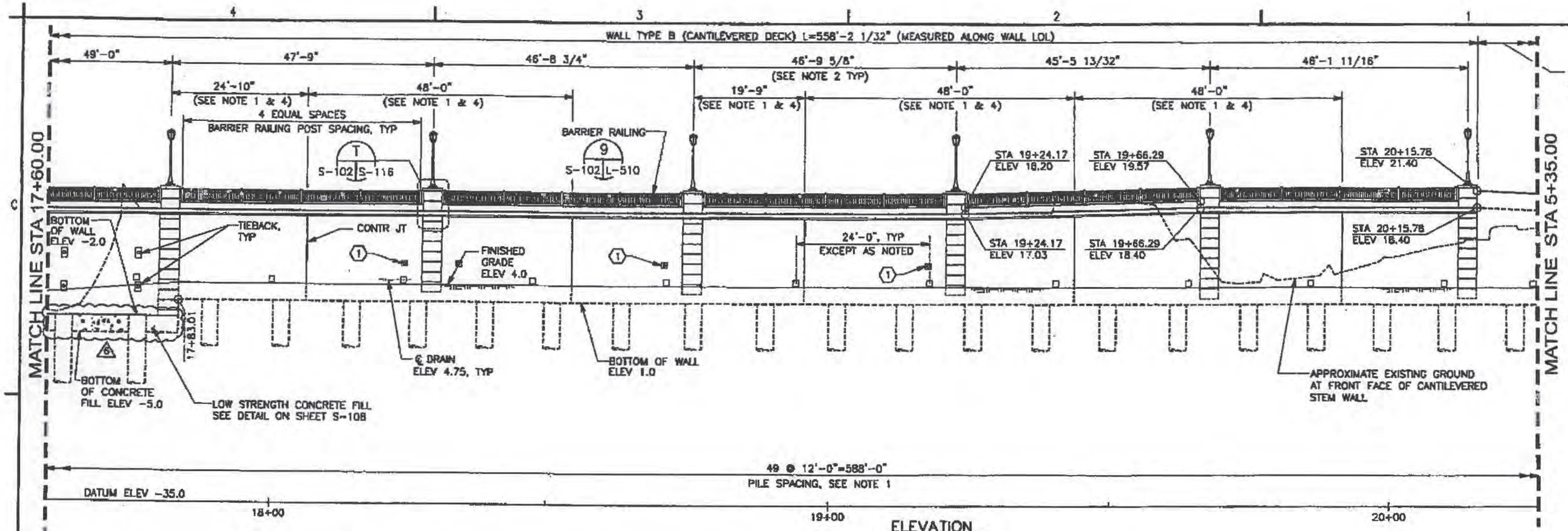
Designed by: | Date: | Rev.:
Dwn by: | Spec No.: | Design file no.:
Reviewed by: | Drawing Code:
Submitted by:

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

BEI ENGINEERING INC.
900 East Main Street, Suite 100
Grass Valley, California 95945
530.477.9960

NAPA CALIFORNIA
NAPA RIVER/NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (BROWN STREET TO FIFTH STREET)
(VALUE ENGINEERING PLANS)
WALL NO. 1
PLAN AND ELEVATION NO. 1B

Sheet reference number:
S-101B
Sheet of



ELEVATION
SCALE: 1"=10'-0"

NOTES:

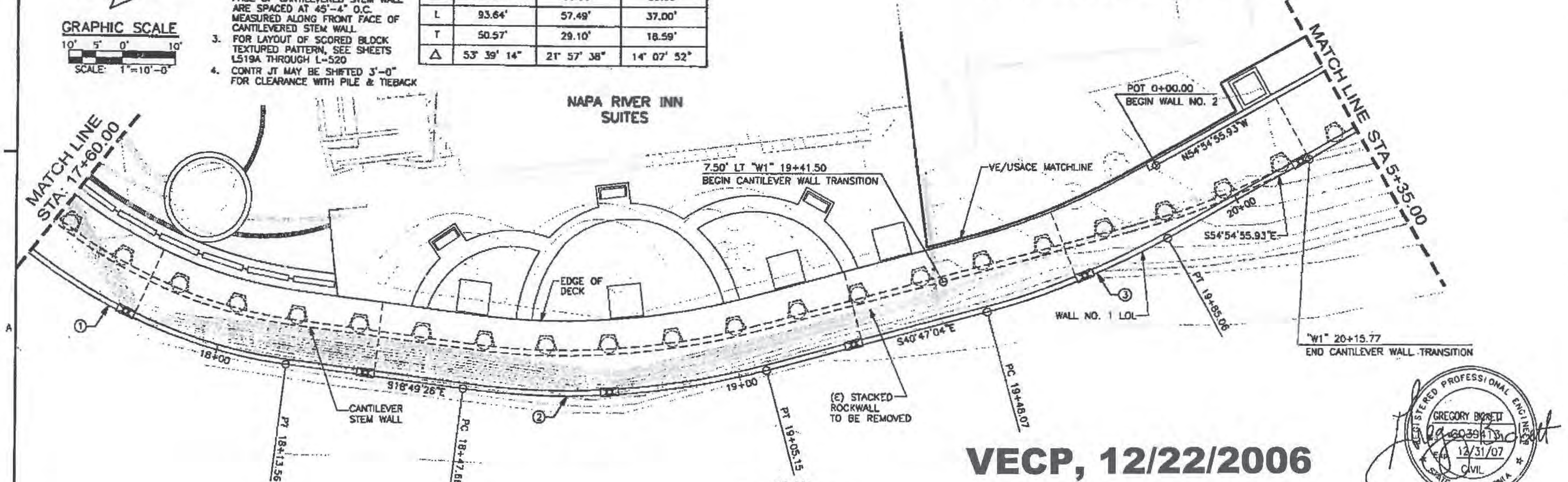
1. MEASURED ALONG FRONT FACE OF CANTILEVERED STEM WALL
2. LIGHT PILASTERS MEASURED ALONG WALL LOL. PILASTERS ON FRONT FACE OF CANTILEVERED STEM WALL ARE SPACED AT 45'-4" O.C. MEASURED ALONG FRONT FACE OF CANTILEVERED STEM WALL
3. FOR LAYOUT OF SCORED BLOCK TEXTURED PATTERN, SEE SHEETS L519A THROUGH L-520
4. CONTR JT MAY BE SHIFTED 3'-0" FOR CLEARANCE WITH PILE & TIEBACK

CURVE DATA			
	①	②	③
R	100.00'	150.00'	150.00'
L	93.64'	57.49'	37.00'
T	50.57'	29.10'	18.59'
Δ	53° 39' 14"	21° 57' 38"	14° 07' 52"

WALL PENETRATION REFERENCE NOTES:

① 6" SDR35 STORM DRAIN PIPE, SEE C-104A

GRAPHIC SCALE
10' 5' 0' 10'
SCALE: 1"=10'-0"



VECP, 12/22/2006

GREGORY BONETTI
C039413
12/31/07
CIVIL

PLAN
SCALE: 1"=10'-0"

US Army Corps of Engineers
Sacramento District

△	REVISED GRADE BEAM GEOMETRY	12/22/08	GGB
△	ADD NOTE	08/06/08	GGB
△	REMOVED EXP JT	08/06/08	GGB
△	REMOVED PLASTER BELOW BOTTOM OF WALL	08/06/08	GGB
△	ADDED STORM DRAIN	08/06/08	GGB
△	REVISED ELEVATIONS	08/06/08	GGB
Symbol	Description	Date	Approved

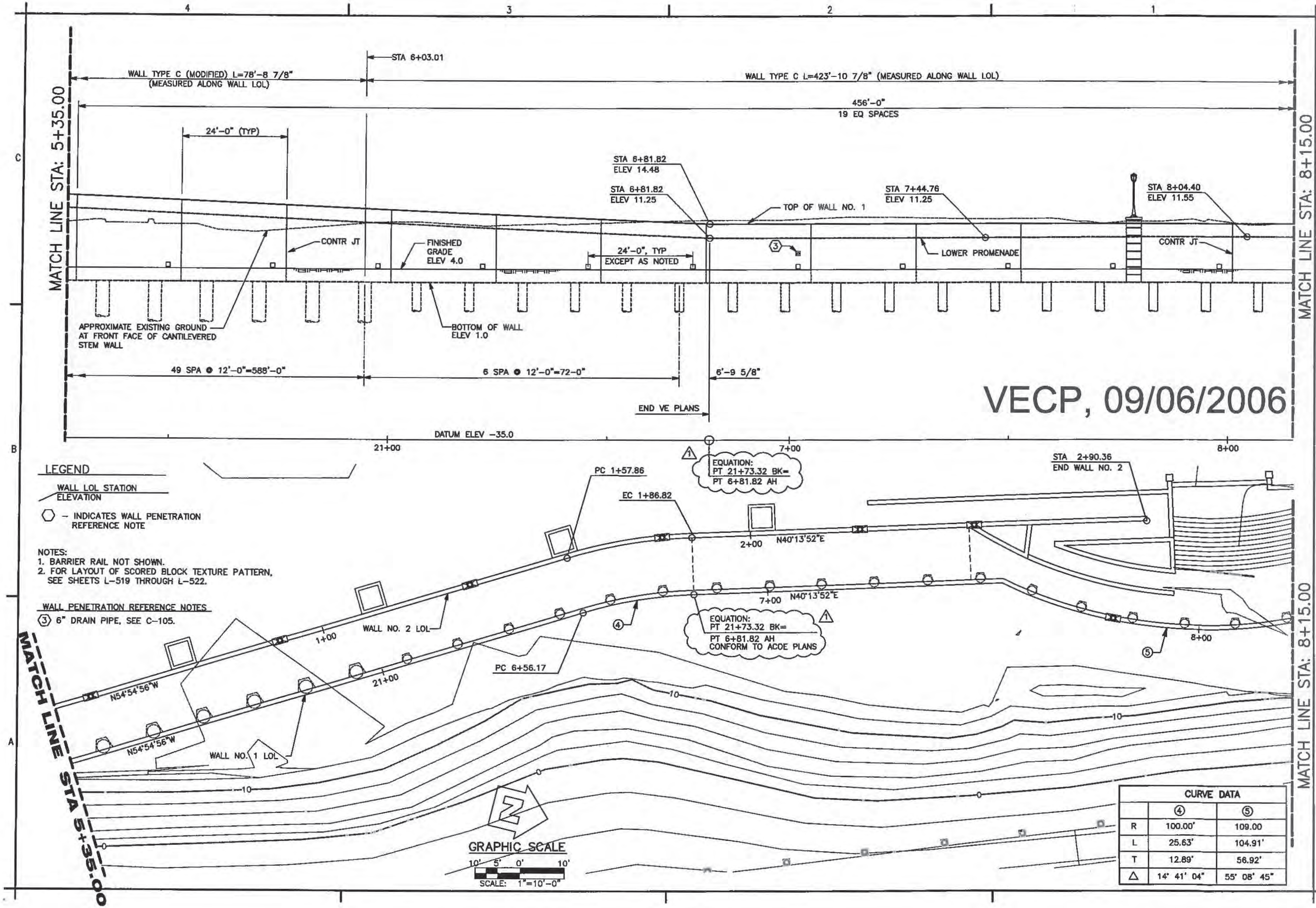
Designed by: | Date: | Rev.
Dwn by: | Spec No.: | Design file no:
Reviewed by: | Drawing Code:
Submitted by: | Plot scale: | Plot date: | Dwg scale:

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

BEI ENGINEERING INC.
900 East Main Street, Suite 100
Grass Valley, California 95945
530.477.9660

NAPA — CALIFORNIA
NAPA RIVER/NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (BROWN STREET
TO FIFTH STREET)
(VALUE ENGINEERING PLANS)
WALL NO. 1
PLAN AND ELEVATION NO. 2

Sheet reference number:
S-102
Sheet of



VECP, 09/06/2006

LEGEND

WALL LOL STATION
ELEVATION

⬡ – INDICATES WALL PENETRATION REFERENCE NOTE

NOTES:
1. BARRIER RAIL NOT SHOWN.
2. FOR LAYOUT OF SCORED BLOCK TEXTURE PATTERN, SEE SHEETS L-519 THROUGH L-522.

WALL PENETRATION REFERENCE NOTES
③ 6" DRAIN PIPE, SEE C-105.

CURVE DATA		
	④	⑤
R	100.00'	109.00
L	25.63'	104.91'
T	12.89'	56.92'
△	14° 41' 04"	55° 08' 45"

US Army Corps
of Engineers
Sacramento District

Symbol	Description	Date	Approved
△	MOVED STATION EQUATION TO VE LIMITS		

Designed by:
Dwn by:
Reviewed by:
Submitted by:
Spec No.:
Date:
Design file no:
Drawing Code:
Rev.
Plot scale:
Plot date:
File name:

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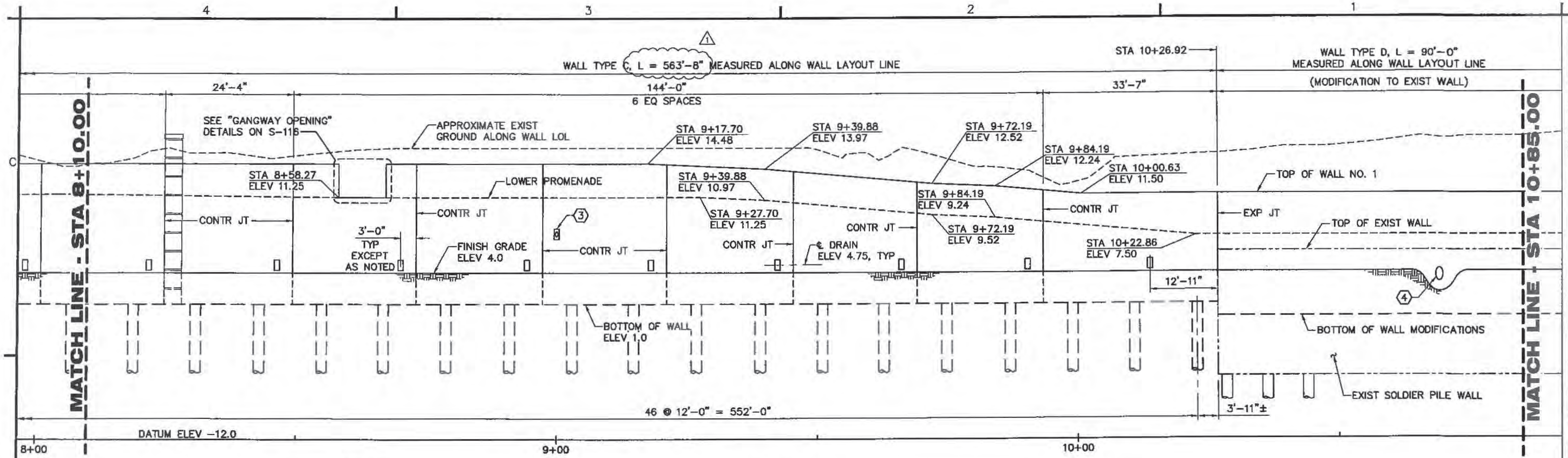
SACRAMENTO, CALIFORNIA

BEI ENGINEERING INC.
900 East Main Street, Suite 100
Grass Valley, California 95945
530.477.9960

NAPA
CALIFORNIA
NAPA RIVER/NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (BROWN STREET
TO FIFTH STREET)
(VALUE ENGINEERING PLANS)
WALL NO. 1
PLAN & ELEVATION NO. 3

Sheet
reference
number:
S-103
Sheet of 166
VE



ELEVATION

SCALE: 1" = 10'-0" (H)
1" = 5'-0" (V)

LEGEND

WALL LOL STATION
TOP OF WALL ELEVATION

⬡ - INDICATES WALL PENETRATION REFERENCE NOTE

NOTES:
1. BARRIER RAIL NOT SHOWN.
2. FOR LAYOUT OF SCORED BLOCK TEXTURE PATTERN, SEE SHEETS L-519 THROUGH L-522.

WALL PENETRATION REFERENCE NOTES

③ 6" DRAIN PIPE, SEE C-105.

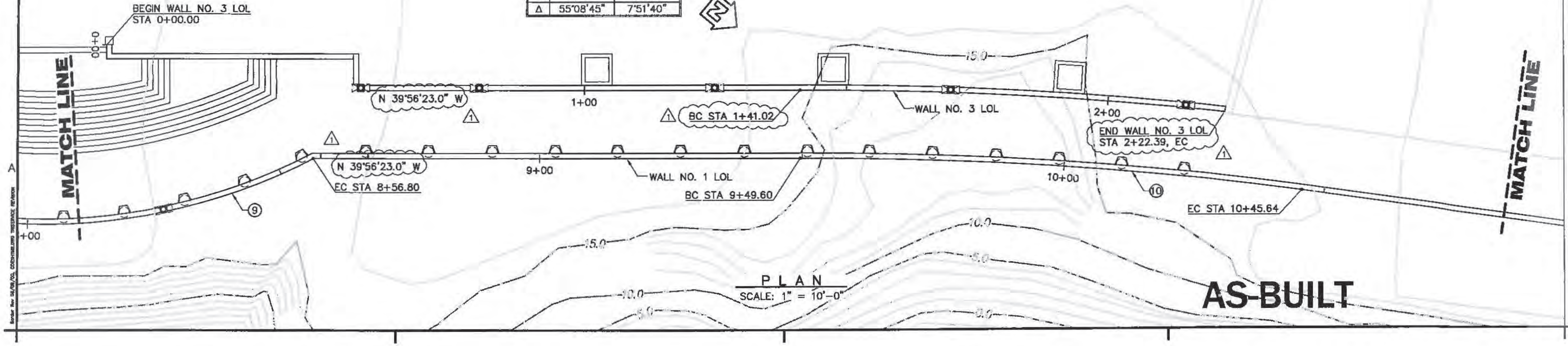
④ 15" RCP DRAIN.

GRAPHIC SCALE

5' 0' 5' 10'
SCALE: 1" = 5'-0"

10' 0' 10' 20'
SCALE: 1" = 10'-0"

CURVE DATA		
No.	⑨	⑩
R	109.00'	700.00'
L	104.91'	96.04'
T	56.92'	48.10'
Δ	55°08'45"	7°51'40"



PLAN

SCALE: 1" = 10'-0"

AS-BUILT

US Army Corps
of Engineers
Sacramento District

Symbol	Description	Date	Approved
△	UPDATES ASSOCIATED WITH WALL NO. 1 ALIGNMENT CHANGE	03/10/06	R. S.

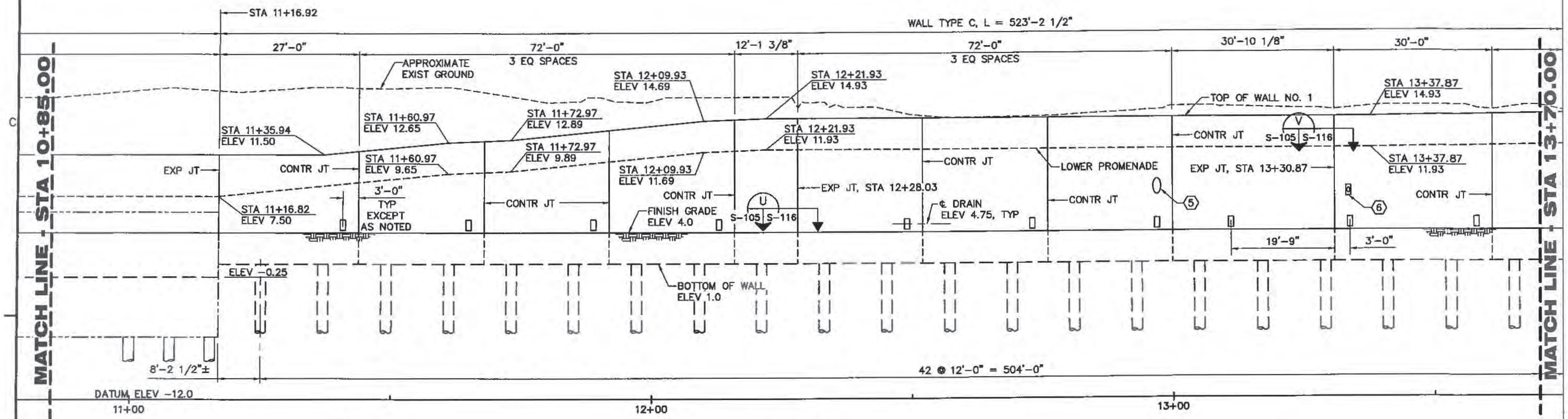
DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

MGE ENGINEERING INC.
7415 Greenhaven Dr. Suite 100
Sacramento, California 95831
916.421.1000

Designed by: D. AN
Dwn by: X. SUN
Spec No.: 1407
Reviewed by: R. SENNETT
Submitted by: /s/ R. SENNETT
Chief, XXXX Des. Sec.
Date: 04/27/05
Design file no: NA-25-030
Drawing Code:
File name:
Plot date:
Plot scale: 1:1
Rev.

NAPA CALIFORNIA
NAPA RIVER / NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST
(HATT BUILDING TO FIRST STREET)
WALL NO. 1
PLAN AND ELEVATION NO. 4

Sheet
reference
number:
S-104
Sheet 76 of 166



ELEVATION

SCALE: 1" = 10'-0" (H)
1" = 5'-0" (V)

LEGEND

WALL LOL STATION
TOP OF WALL ELEVATION

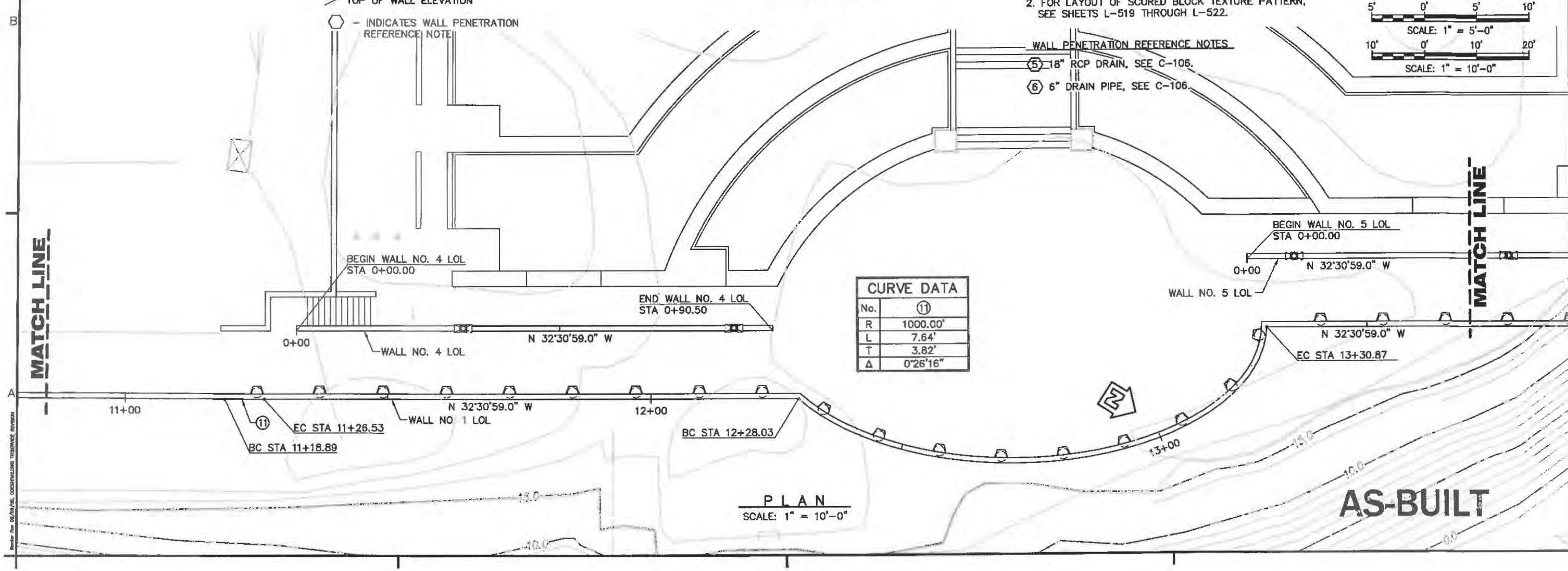
⬡ - INDICATES WALL PENETRATION REFERENCE NOTE

NOTES:
1. BARRIER RAIL NOT SHOWN.
2. FOR LAYOUT OF SCORED BLOCK TEXTURE PATTERN, SEE SHEETS L-519 THROUGH L-522.

WALL PENETRATION REFERENCE NOTES
⑤ 18" RCP DRAIN, SEE C-106.
⑥ 6" DRAIN PIPE, SEE C-106.

GRAPHIC SCALE

SCALE: 1" = 5'-0"
SCALE: 1" = 10'-0"



CURVE DATA	
No.	⑪
R	1000.00'
L	7.64'
T	3.82'
Δ	0°26'16"

P L A N

SCALE: 1" = 10'-0"

US Army Corps
of Engineers
Sacramento District

Symbol	Description	Date	Approval
△	WALL NO. 1 ALIGNMENT REVISED	02/27/06	R. S.

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

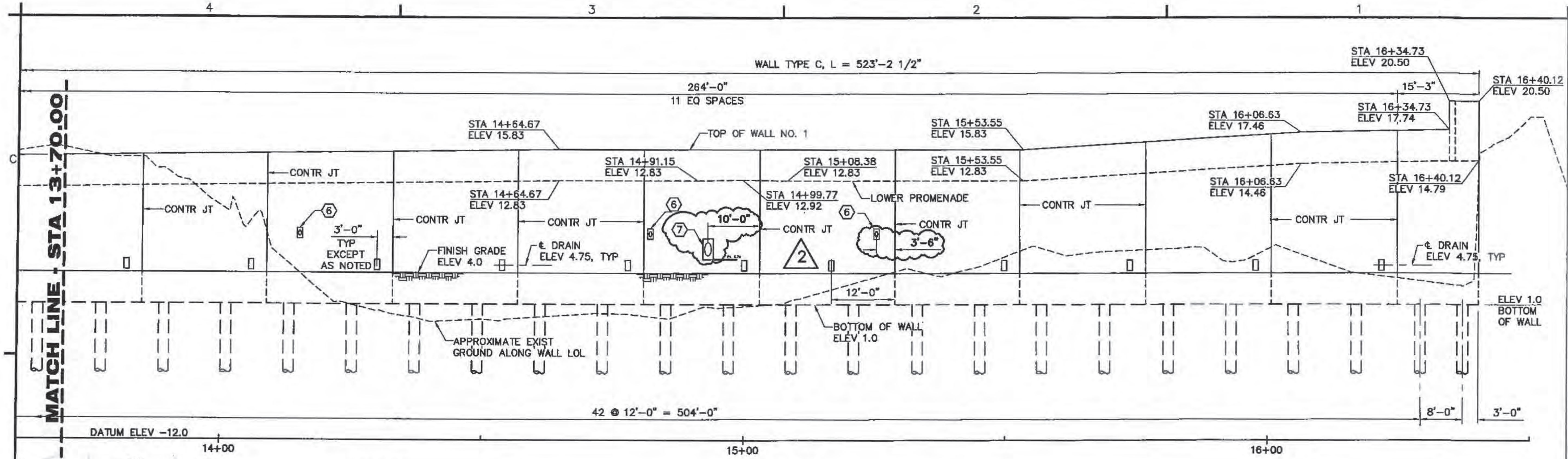
MGE ENGINEERING INC.
7415 Greenhaven Dr. Suite 100
Sacramento, California 95831
916.421.1000

Designed by: D. AN
Dwn by: X. SUN
Spec No.: 1407
Reviewed by: R. SENNETT
Submitted by: /s/ R. SENNETT
Chief, XXXX Des. Sec.
Date: 04/27/05
Rev.
Design file no: NA-25-030
Drawing Code:
File name:
Plot date:
Plot scale: 1:1

NAPA
CALIFORNIA
NAPA RIVER / NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST
(HATT BUILDING TO FIRST STREET)

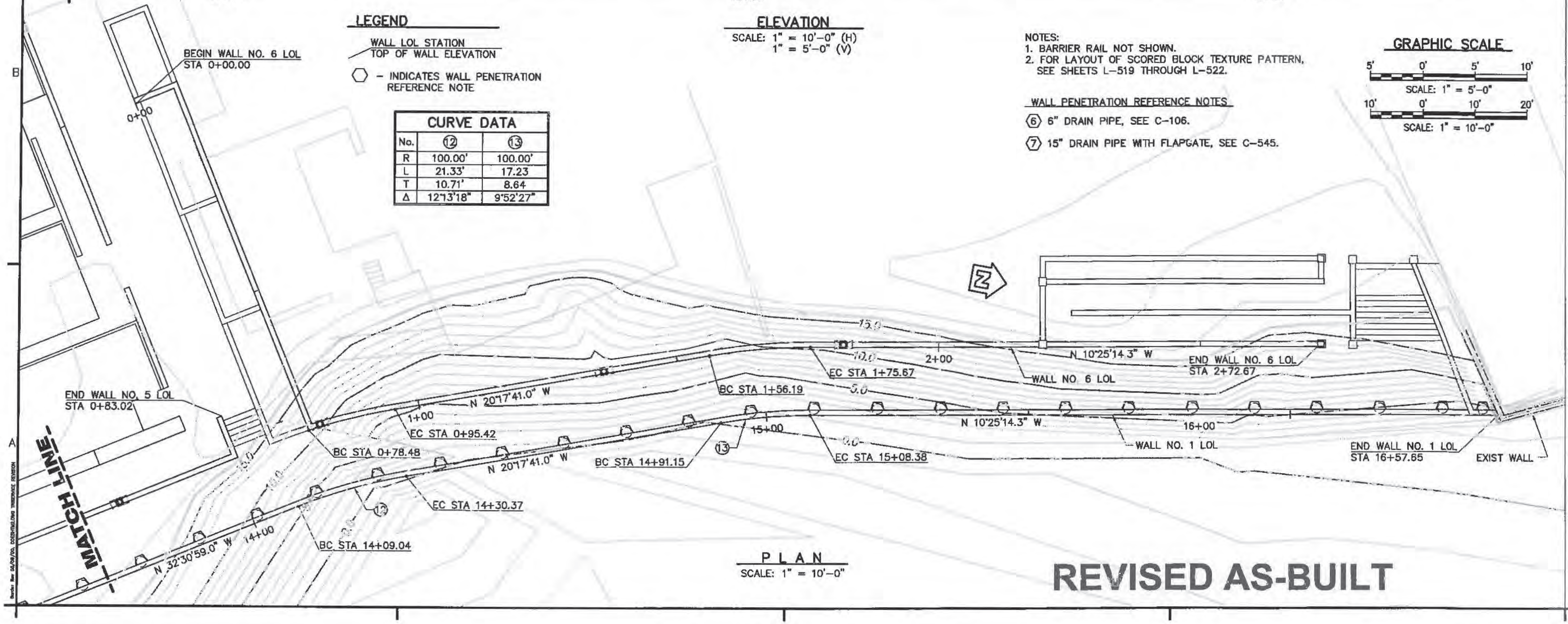
WALL NO. 1
PLAN AND ELEVATION NO. 5

Sheet reference number:
S-105
Sheet 77 of 166



ELEVATION

SCALE: 1" = 10'-0" (H)
1" = 5'-0" (V)



LEGEND

WALL LOL STATION
TOP OF WALL ELEVATION

⬡ - INDICATES WALL PENETRATION REFERENCE NOTE

CURVE DATA		
No.	⑫	⑬
R	100.00'	100.00'
L	21.33'	17.23
T	10.71'	8.64
Δ	12°13'18"	9°52'27"

NOTES:
1. BARRIER RAIL NOT SHOWN.
2. FOR LAYOUT OF SCORED BLOCK TEXTURE PATTERN, SEE SHEETS L-519 THROUGH L-522.

WALL PENETRATION REFERENCE NOTES

⑥ 6" DRAIN PIPE, SEE C-106.

⑦ 15" DRAIN PIPE WITH FLAPGATE, SEE C-545.

GRAPHIC SCALE

5' 0' 5' 10'
SCALE: 1" = 5'-0"

10' 0' 10' 20'
SCALE: 1" = 10'-0"

PLAN

SCALE: 1" = 10'-0"

REVISED AS-BUILT

US Army Corps
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Sacramento District

Symbol	Description	Date	Approved
2	FIELD FIT BLOCKOUTS TO MATCH DRAINAGE ON CIVIL PLANS		
1	WALL NO. 1 ALIGNMENT REVISED	02/27/06	R. S.

Designed by: D. AN		Date: 04/27/05	Rev.
Dwn by: X. SUN	Spec No.: 1407	Design file no: NA-25-030	
Reviewed by: R. SENNETT		Drawing Code:	
Submitted by: /s/ R. SENNETT		File name: Plot date: Plot scale: 1:1	

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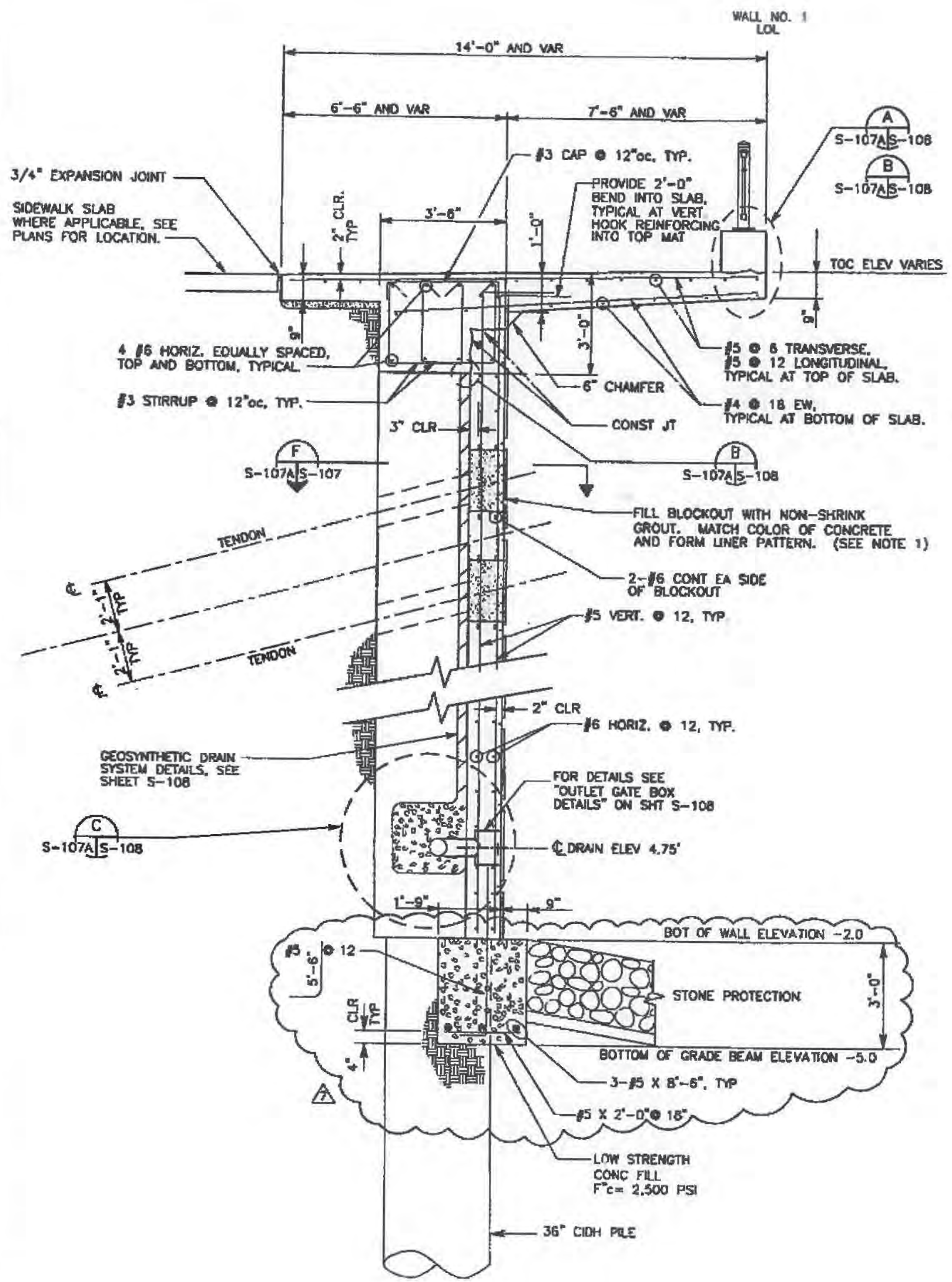
916.421.1000

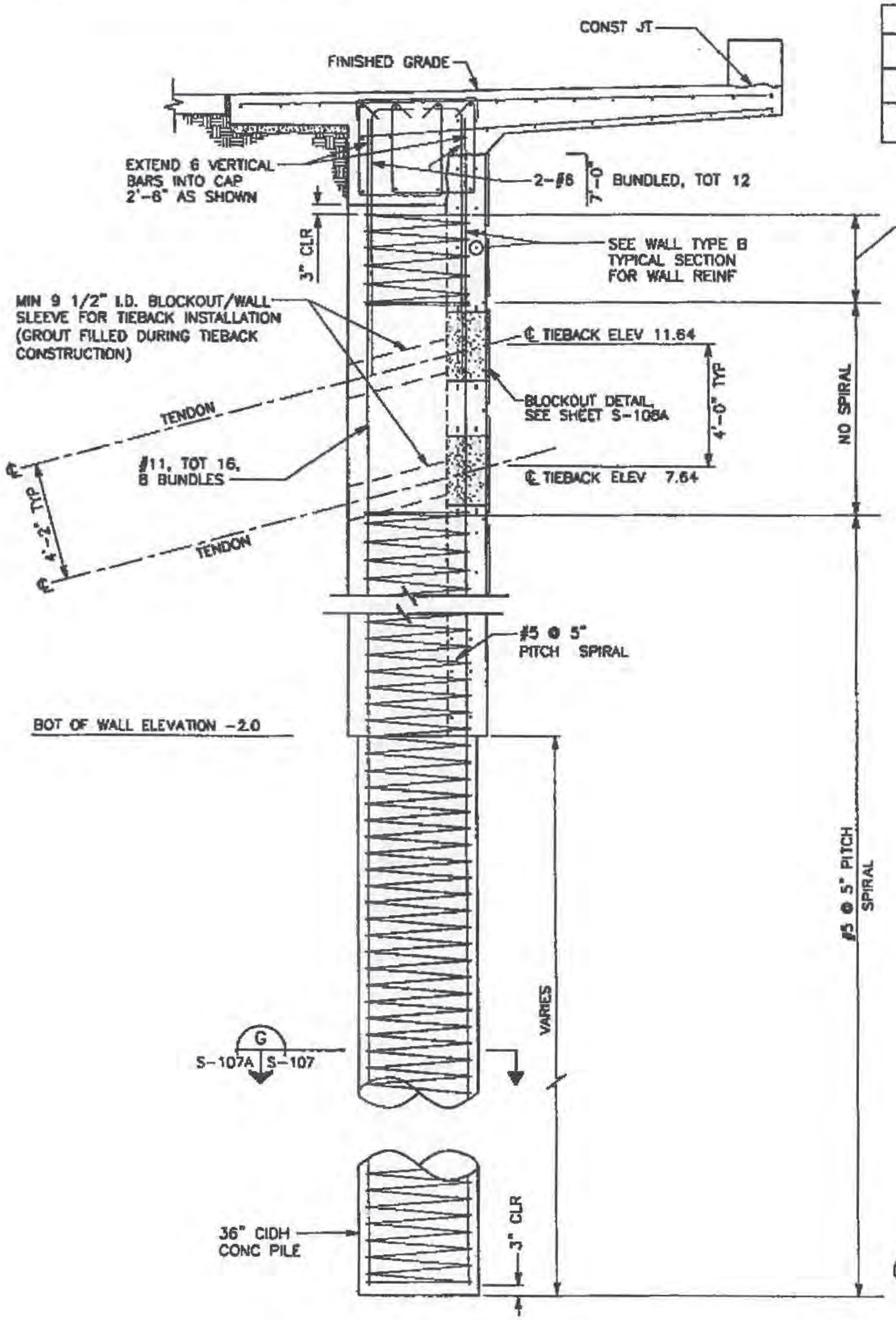
NAPA CALIFORNIA
NAPA RIVER / NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST
(HATT BUILDING TO FIRST STREET)
WALL NO. 1
PLAN AND ELEVATION NO. 6

Sheet
reference
number:
S-106
Sheet 78 of 166



WALL TYPE B (CANTILEVER DECK) W/DOUBLE TIEBACK-TYPICAL SECTION

SCALE: 1/2" = 1'-0"



36" CIDH PILE ELEVATION

SCALE: 1/2" = 1'-0"

PILE TIP ELEVATION TABLE

STATION RANGE	PILE TIP ELEVATION
14+62.66 TO 16+87.66	-28.0
16+99.67 TO 17+75.66	-26.0

NOTE

1. CONTRACTOR MAY REQUEST PERMISSION FROM ENGINEER TO USE ONE BLOCK-OUT INSTEAD OF TWO INDIVIDUAL BLOCKOUTS.

GREGORY BICKETT
12/31/07
CIVIL

VECP, 12/22/2006

2' 1' 0 2'
SCALE:1/2"= 1'-0"

US Army Corps of Engineers
Sacramento District

Symbol	Description	Date	Approved
7	REVISED GRADE BEAM GEOMETRY	12/22/06	GCB
	REMOVED SPIRAL	09/22/06	GCB
	ADDED PILE TIP TABLE	09/07/06	GCB
	REVISED NOTE	09/07/06	GCB
	REVISED STEEL TUBE TIEBACK	09/06/06	GCB
	REMOVED CORBEL #1	09/06/06	GCB
	REMOVED PARAPET WALLS	09/06/06	GCB

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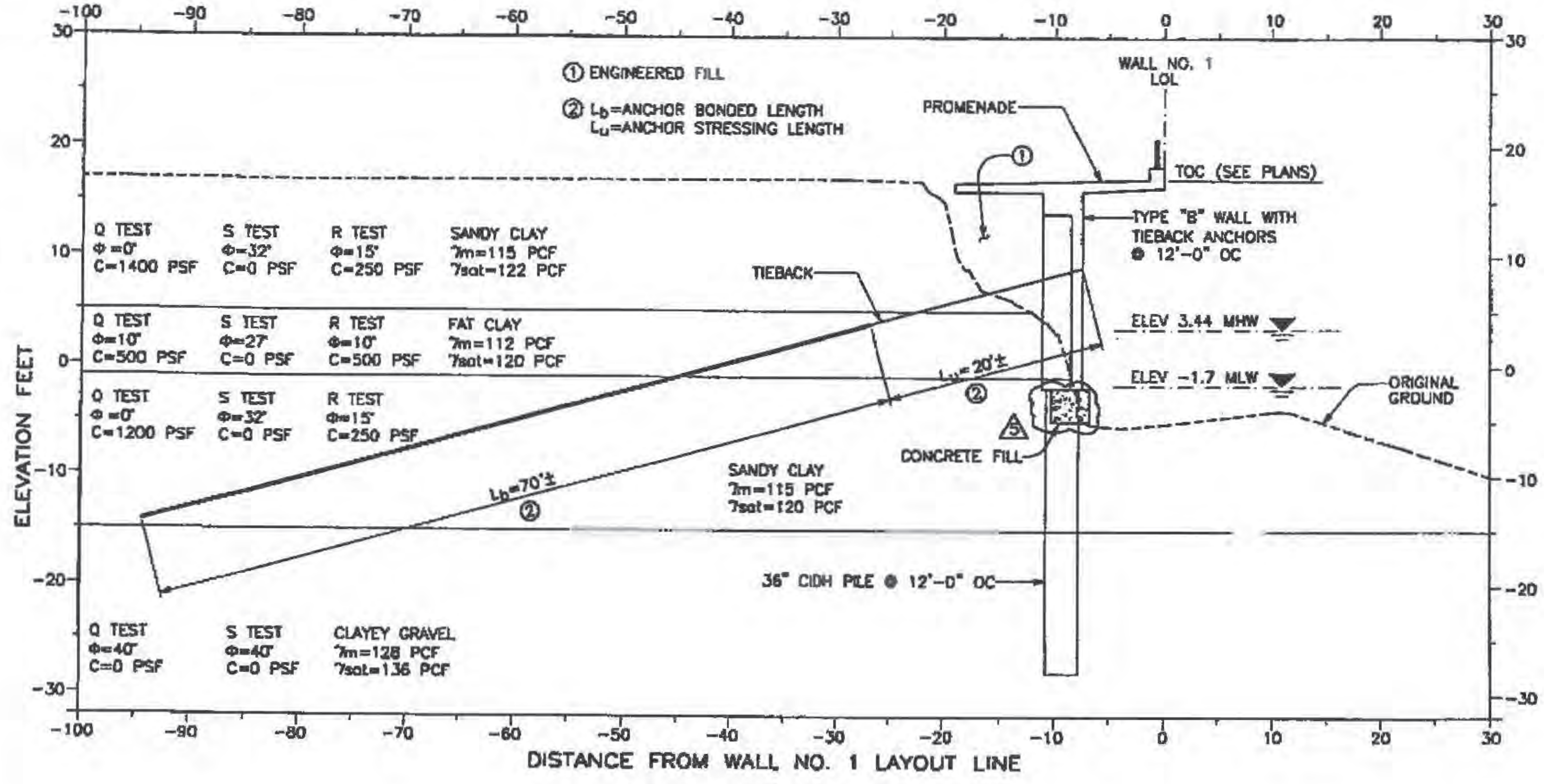
Grass Valley, California 95945
530.477.4960

Designed by:
Dwn by:
Spec No.:
Reviewed by:
Submitted by:
Date:
Rev:
Design file no:
Drawing Code:

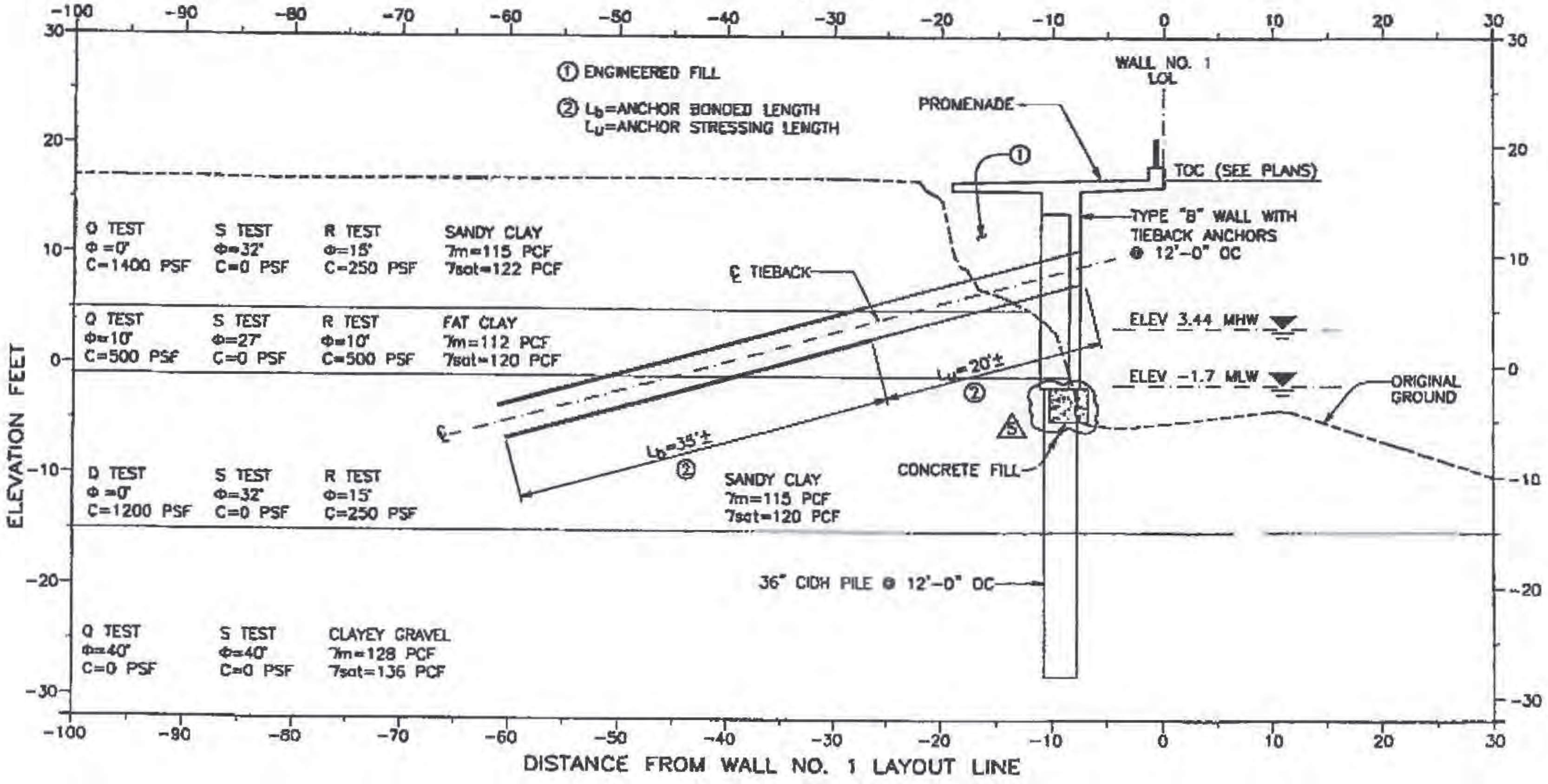
NAPA
NAPA RIVER/NAPA CREEK CALIFORNIA
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (BROWN STREET TO FIFTH STREET)
(VALUE ENGINEERING PLANS)
WALL NO. 1 CANTILEVER WALL
DETAILS NO. 2

Sheet reference number:
S-107A
Sheet of



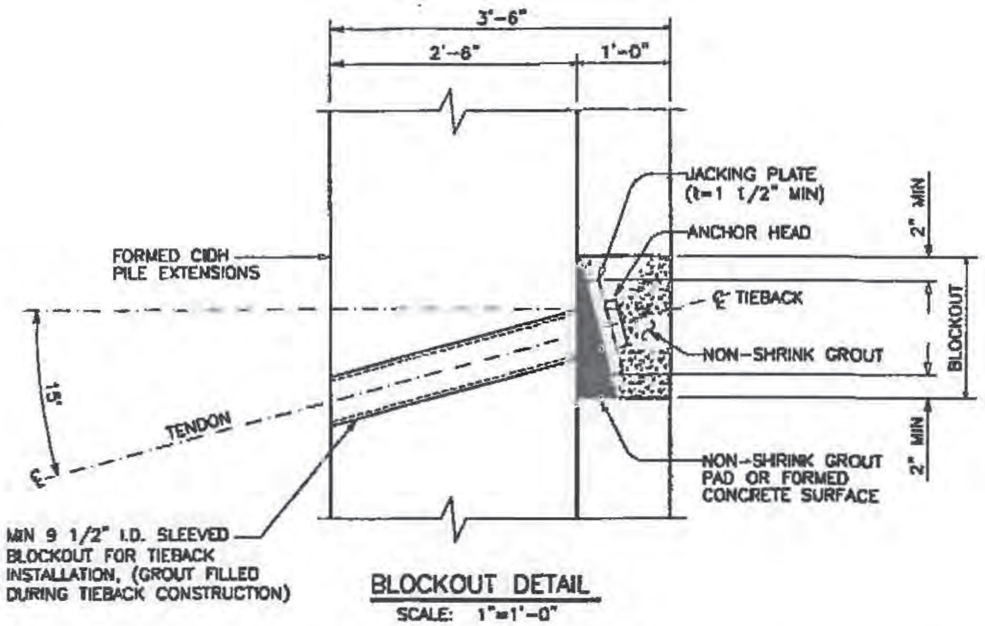
SINGLE TIEBACK TYPICAL SECTION

SCALE: 1/8"=1'-0"



DOUBLE TIEBACK TYPICAL SECTION

SCALE: 1/8"=1'-0"



BLOCKOUT DETAIL

SCALE: 1"=1'-0"

GENERAL NOTES – WORKING STRESS DESIGN

DESIGN: AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS (AASHTO), SECTION 5 STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, 15th EDITION, AND THE CALIFORNIA DEPARTMENT OF TRANSPORTATION.

SOIL PARAMETERS: (FOR DETERMINATION OF DESIGN LATERAL EARTH PRESSURE ON WALL).

$\Phi=32°$ SANDY CLAY $\gamma_m=115$ PCF $\gamma_{sat}=120$ PCF

REINFORCED CONCRETE: f_y = 60 ksi (YEILD STRENGTH OF REINFORCEMENT)

f'_c = 3000 psi (COMPRESSIVE STRENGTH AT 28 DAYS)

PRESTRESSING STEEL: (TIEBACKS)
STRANDS – ASTM DESIGNATION A416

SINGLE TENDON: T = 150k DESIGN (ASD)
T LOCKOFF=80k OR AS DIRECTED BY ENGINEER

DOUBLE TENDON: T = 75k DESIGN (ASD)
T LOCKOFF=37.5k OR AS DIRECTED BY ENGINEER

fpu= MINIMUM TENSILE STRENGTH OF PRESTRESSING STEEL
(270 ksi)

$$As(min) = \frac{1.0\ T}{0.75\ fpu}$$

CONSTRUCTION STAGES

STAGE I: CONSTRUCT WALL WITHOUT PROMENADE.

STAGE II: BACKFILL WALL TO ELEVATION OF TIEBACK TENDON PLUS 1 FOOT OF COVER.

STAGE III: INSTALL TIEBACK TENDON, COMPLETE. DO NOT TEST TENDON AT THIS TIME. PULL TENDON TIGHT (INITIAL LOCKOFF) AND ALIGN WALL, IF NECESSARY, BY PULLING WALL INTO POSITION WITH TENDON(S).

STAGE IV: COMPLETE BACKFILL OPERATIONS AND TEST TENDON(S) TO SPECIFIED LOAD. CONTRACTOR SHALL INSTALL HYDRAULIC JACK AND PULL ON TENDON TO DETERMINE, AND NOTE IN QC TESTING RECORDS, THE TENSION IN TENDON(S) AS A RESULT OF COMPLETING STAGE IV, BACKFILL OPERATIONS. NOTIFY ENGINEER OF THIS TENSION VALUE. COMPLETE TENDON QC TESTING. CONTRACTOR SHALL ADJUST TENSION IN TENDON(S) TO T LOCKOFF OR AS DIRECTED BY THE ENGINEER AND THEN SET WEDGES.

STAGE V: COMPLETE CONSTRUCTION OF PROMENADE AND RAILING.

VECP, 12/22/2006

SCALE: AS SHOWN

REGISTERED PROFESSIONAL ENGINEER
GREGORY BICKETT
C039413
Exp. 12/31/07
CIVIL
STATE OF CALIFORNIA

US Army Corps
of Engineers
Sacramento District

Symbol	Description	Date	Approved
△	REVISED GRADE BEAM GEOMETRY	12/22/06	GBB
△	VERIFY LOAD FOR DOUBLE TENDON	09/08/06	GBB
△	REVISED STEEL TUBE TIEBACK	09/08/06	GBB
△	REVISED LIMITS OF BLOCKOUT	08/08/06	GBB
△	REMOVED DIMENSION	08/08/06	GBB

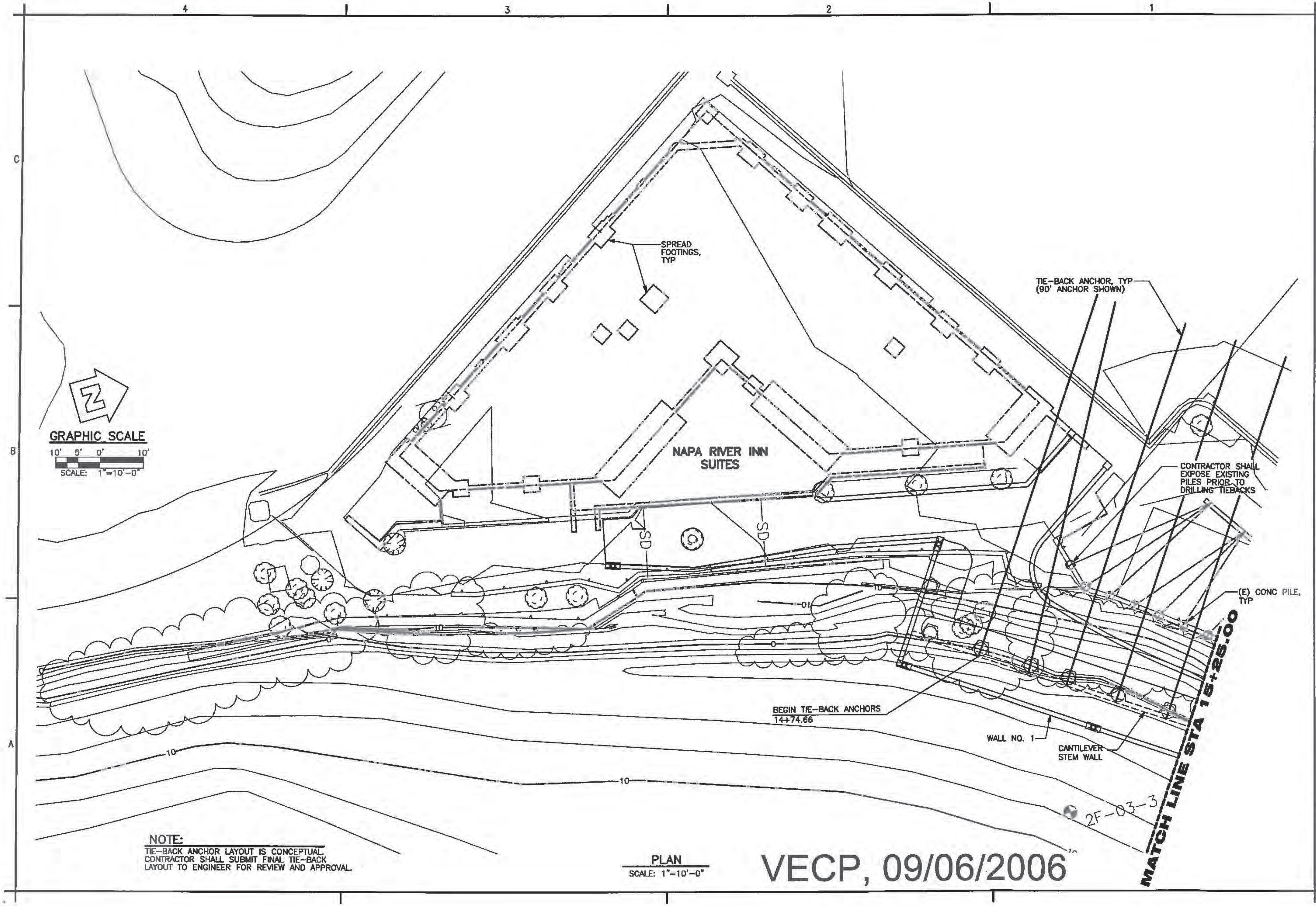
DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

BEI ENGINEERING INC.
900 East Main Street, Suite 100
Grass Valley, California 95945
530.477.9980

Designed by:
Date:
Rev.
Dwn by:
Spec No.:
Design file no.:
Reviewed by:
Drawing Code:
Submitted by:

NAPA CALIFORNIA
NAPA RIVER/NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (BROWN STREET
TO FIFTH STREET)
(VALUE ENGINEERING PLANS)
WALL NO. 1
DETAILS NO. 5

Sheet reference number:
S-108A
Sheet of



NOTE:
TIE-BACK ANCHOR LAYOUT IS CONCEPTUAL. CONTRACTOR SHALL SUBMIT FINAL TIE-BACK LAYOUT TO ENGINEER FOR REVIEW AND APPROVAL.

PLAN
SCALE: 1"=10'-0"

VECP, 09/06/2006

US Army Corps
of Engineers
Sacramento District

Symbol	Description	Date	Approval

Designed by:		Date:	Rev.
Dwn by:	Spec No.:	Design file no:	
Reviewed by:		Drawing Code:	
Submitted by:		File name: Plot date: Dwg scale:	

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SACRAMENTO, CALIFORNIA

BEI ENGINEERING INC.
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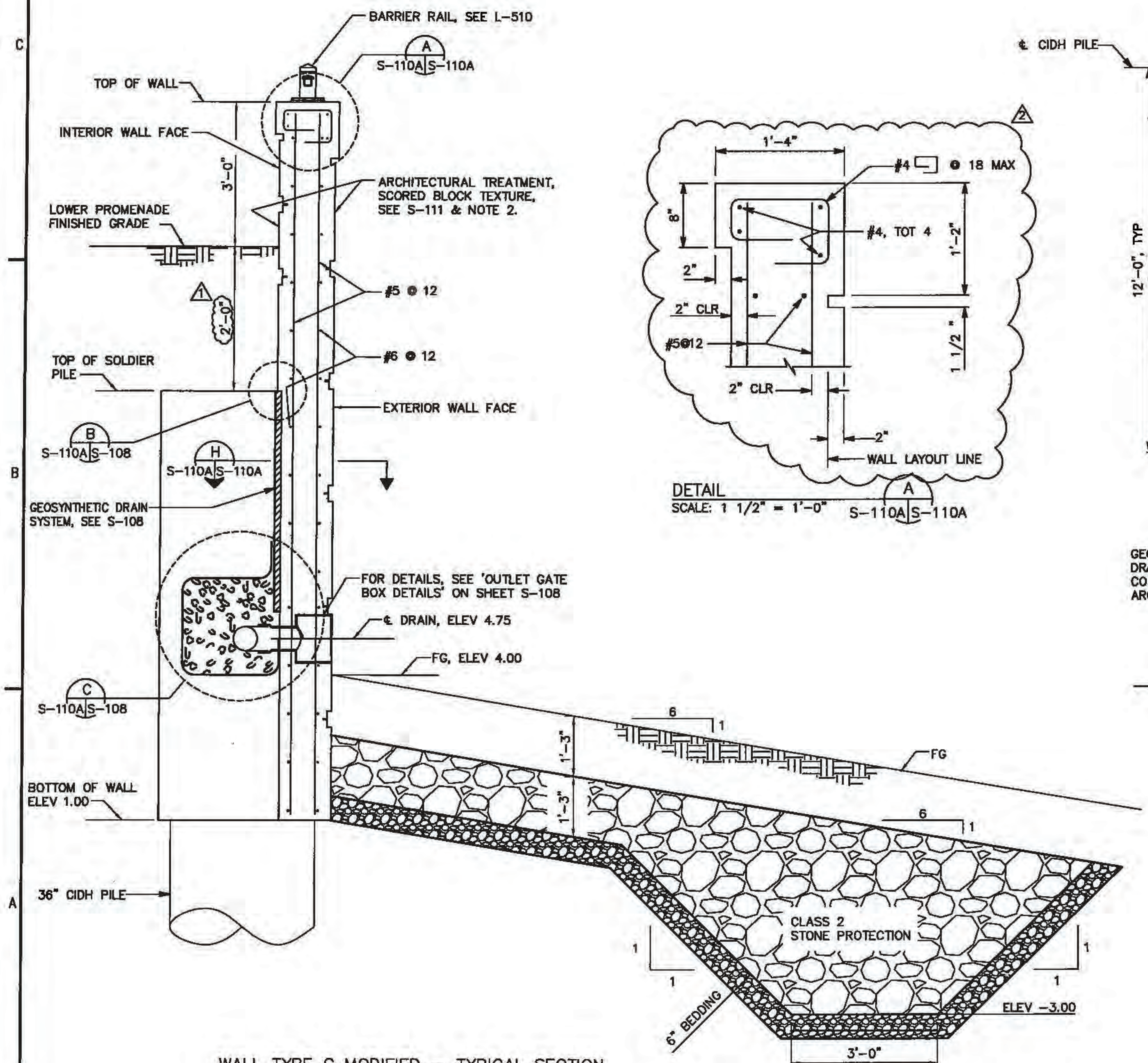
Grass Valley, California 95945
530.477.9960

NAPA CALIFORNIA
NAPA RIVER/NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (BROWN STREET
TO FIFTH STREET)
(VALUE ENGINEERING PLANS)
WALL NO. 1
TIE-BACK ANCHOR LAYOUT NO. 1

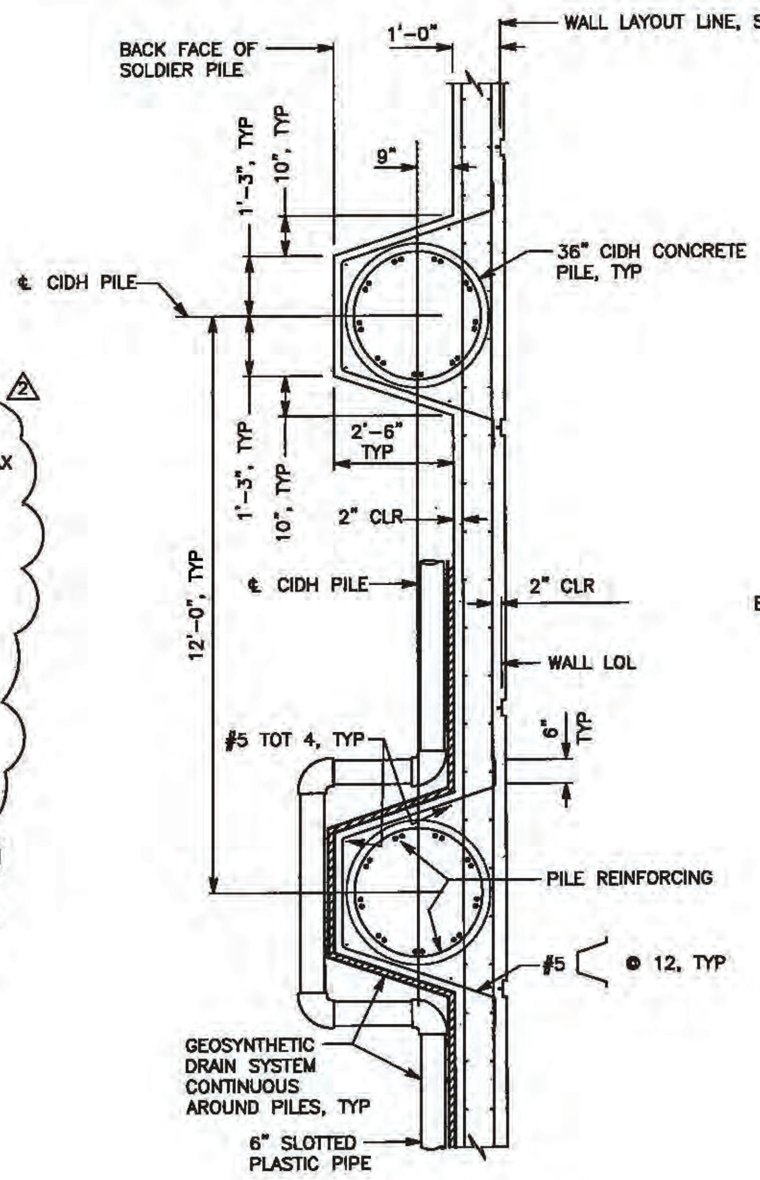
Sheet
reference
number:
S–108B
Sheet of 166
VE

NOTES:
1. FOR GEOSYNTHETIC DRAIN SYSTEM DETAILS NOT SHOWN, SEE SHEET S-108.
2. USE SCORED BLOCK TEXTURE PATTERN FOR WALLS 2-6 ON INTERIOR FACE OF WALL.
3. FOR WALL OFF-SET VALUES NOT SHOWN, SEE S-100.

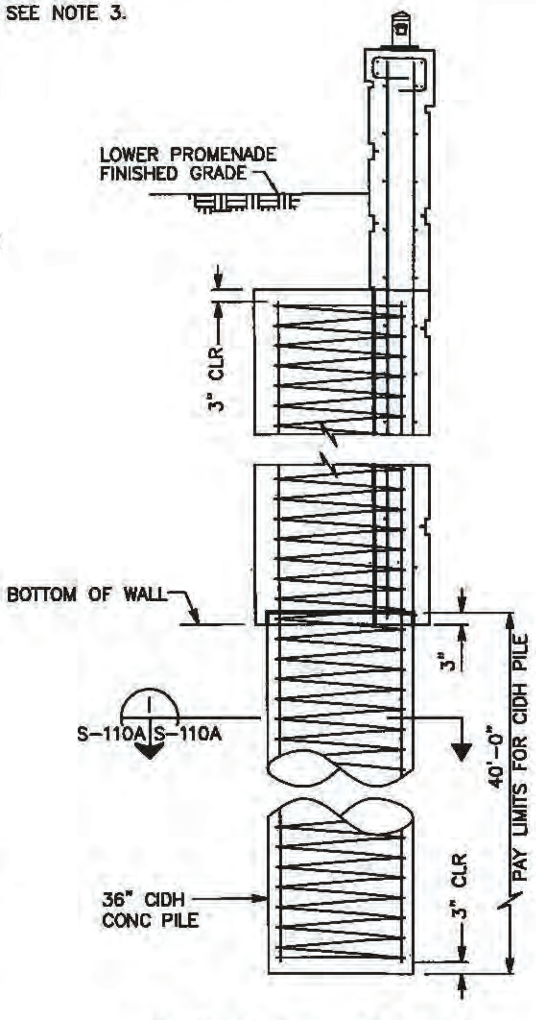


DETAIL A
SCALE: 1 1/2" = 1'-0" S-110A S-110A

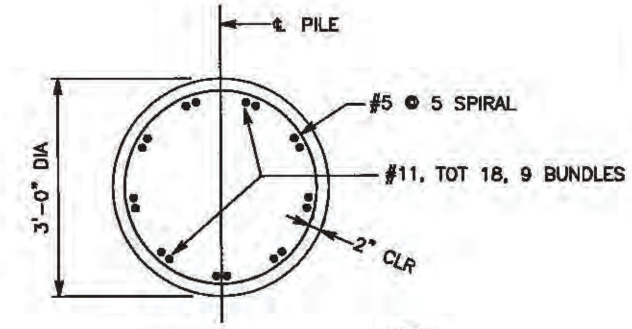
WALL TYPE C MODIFIED - TYPICAL SECTION
SCALE: 3/4" = 1'-0"



SECTION H
SCALE: 1/2" = 1'-0" S-110A S-110A



36" CIDH PILE ELEVATION
SCALE: 1/2" = 1'-0"



SECTION I
SCALE: 3/4" = 1'-0" S-110A S-110A

VECP, 09/06/2006

US Army Corps of Engineers
Sacramento District

Symbol	Descriptions	Date	Approval
1	REVISED DIMENSION		
2	REVISED PARAPET WALL		

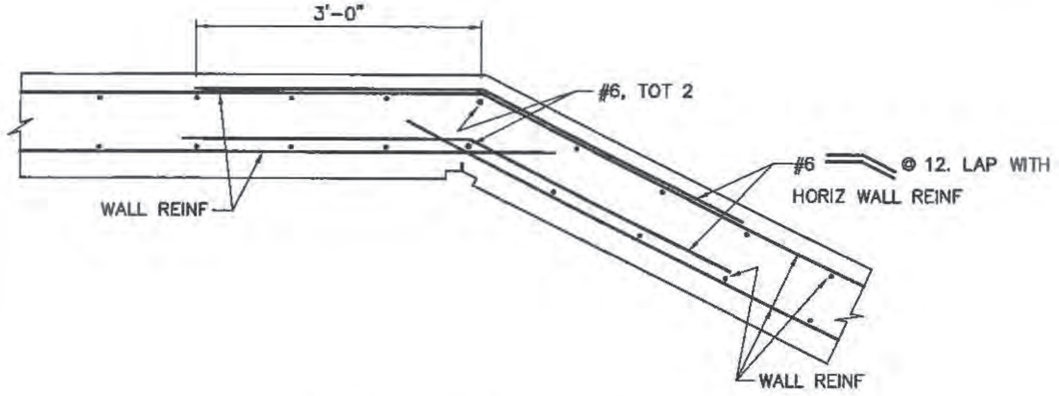
Designed by:
Dwn by:
Spec No.:
Date:
Rev.
Design file no:
Reviewed by:
Drawing Code:
Submitted by:
File name:
Plot date:
Dwg scale:

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

BEI ENGINEERING INC.
900 East Main Street, Suite 100
Grass Valley, California 95945
530.477.9960

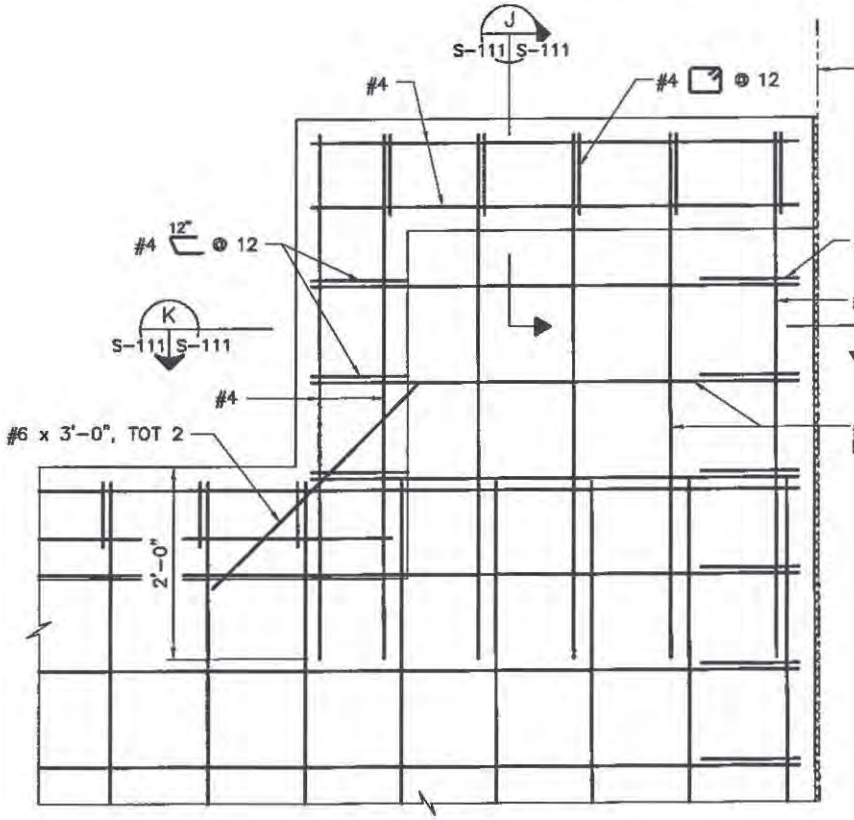
NAPA CALIFORNIA
NAPA RIVER/NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (BROWN STREET
TO FIFTH STREET)
(VALUE ENGINEERING PLANS)
WALL NO. 1
TYPE C MODIFIED WALL DETAILS

Sheet reference number:
S-110A
Sheet of 166
VE



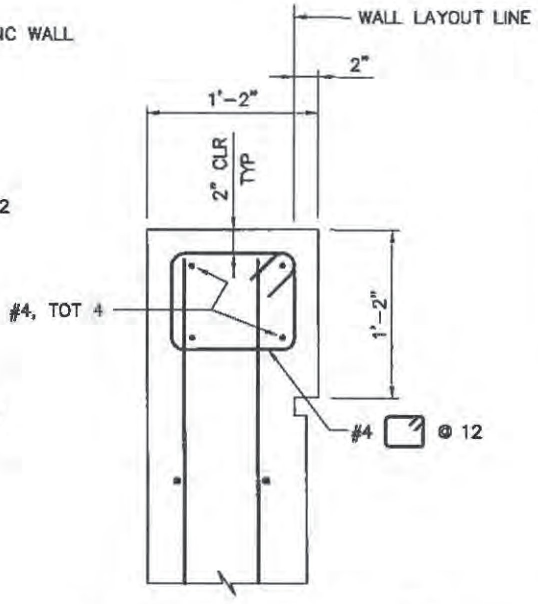
WALL ANGLE POING REINFORCEMENT DETAILS

SCALE: 1" = 1'-0"



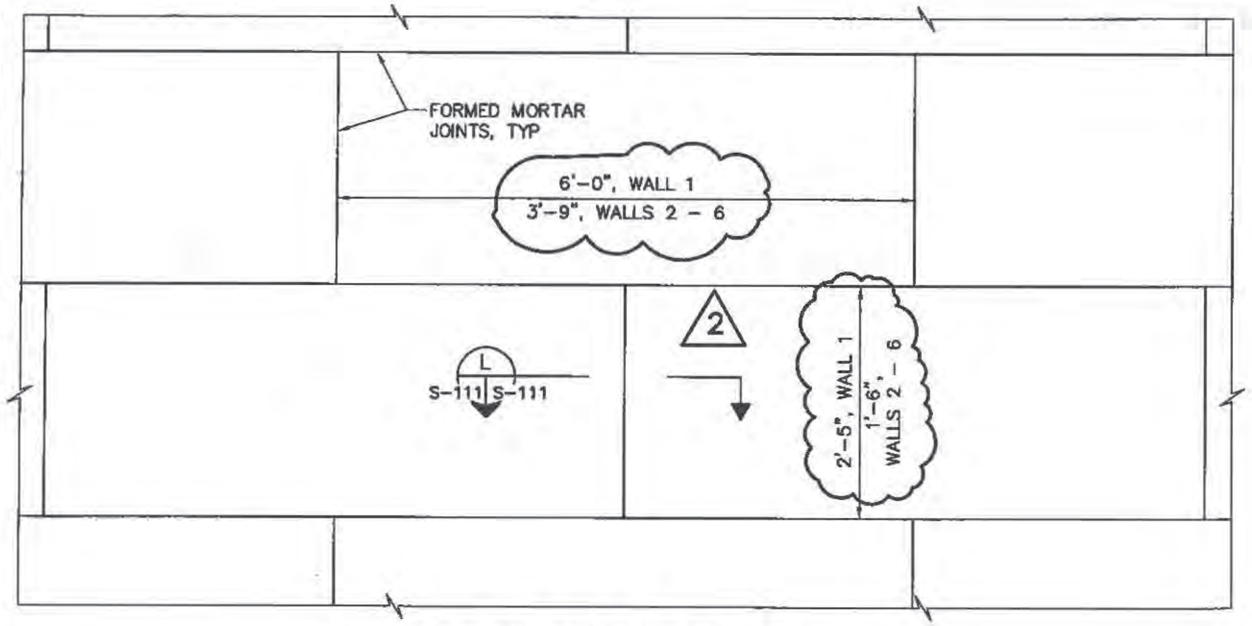
END WALL REINFORCEMENT DETAILS

SCALE: 1" = 1'-0"



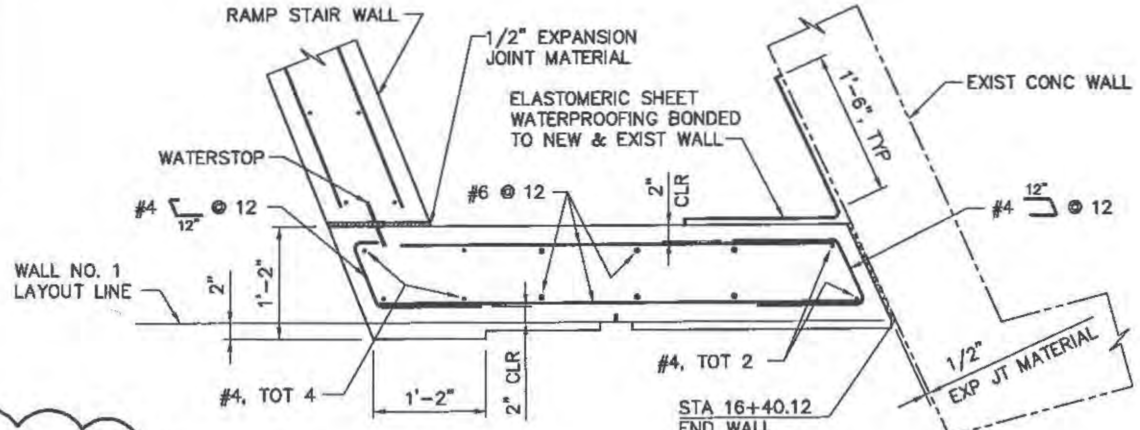
SECTION

SCALE: 1 1/2" = 1'-0" J S-111 S-111



SCORED BLOCK TEXTURE DETAILS

SCALE: NONE

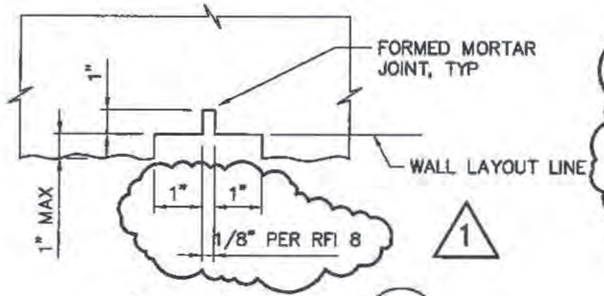


SECTION

SCALE: 1" = 1'-0" K S-111 S-111

2

NOTE 1: PROVIDE 2 DISTINCT MOLDS FOR BOTH SIZES OF BLOCKS. AVOID REPETITIVE PATTERNS BY INVERTING & ROTATING THE MOLDS.

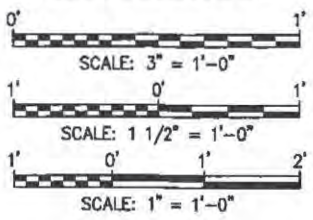


SECTION

SCALE: 3" = 1'-0" L S-111 S-111

REVISED AS-BUILT

GRAPHIC SCALE



US Army Corps
of Engineers
Sacramento District

Symbol	Description	Date	Approved
2	CHANGES PER RFI 4 – ARCHITECTURAL BLOCK PATTERNS	[illegible]	
1	CHANGES PER RFI 8 – FORM MORTOR JT DIMENSION	[illegible]	

Designed by: D. AN		Date: 04/27/05	Rev.
Dwn by: X. SUN	Spec No.: 1407	Design file no: NA-25-030	
Reviewed by: R. SENNETT		Drawing Code:	
Submitted by: /s/ R. SENNETT		File name: Plot date: Plot scale: 1:1	

DEPARTMENT OF THE ARMY
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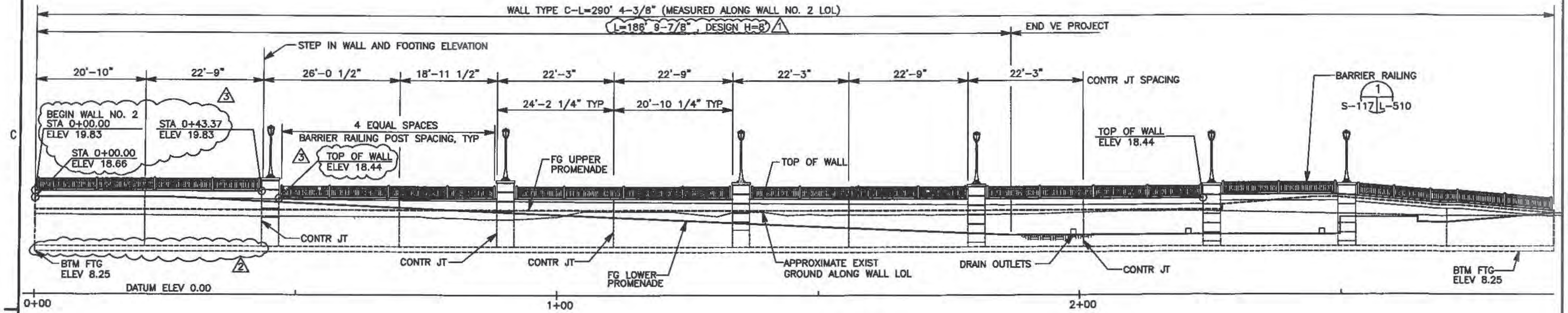
MGE ENGINEERING INC.
7415 Greenhaven Dr. Suite 100
Sacramento, California 95831
916.421.1000

NAPA CALIFORNIA
NAPA RIVER / NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST
(HATT BUILDING TO FIRST STREET)

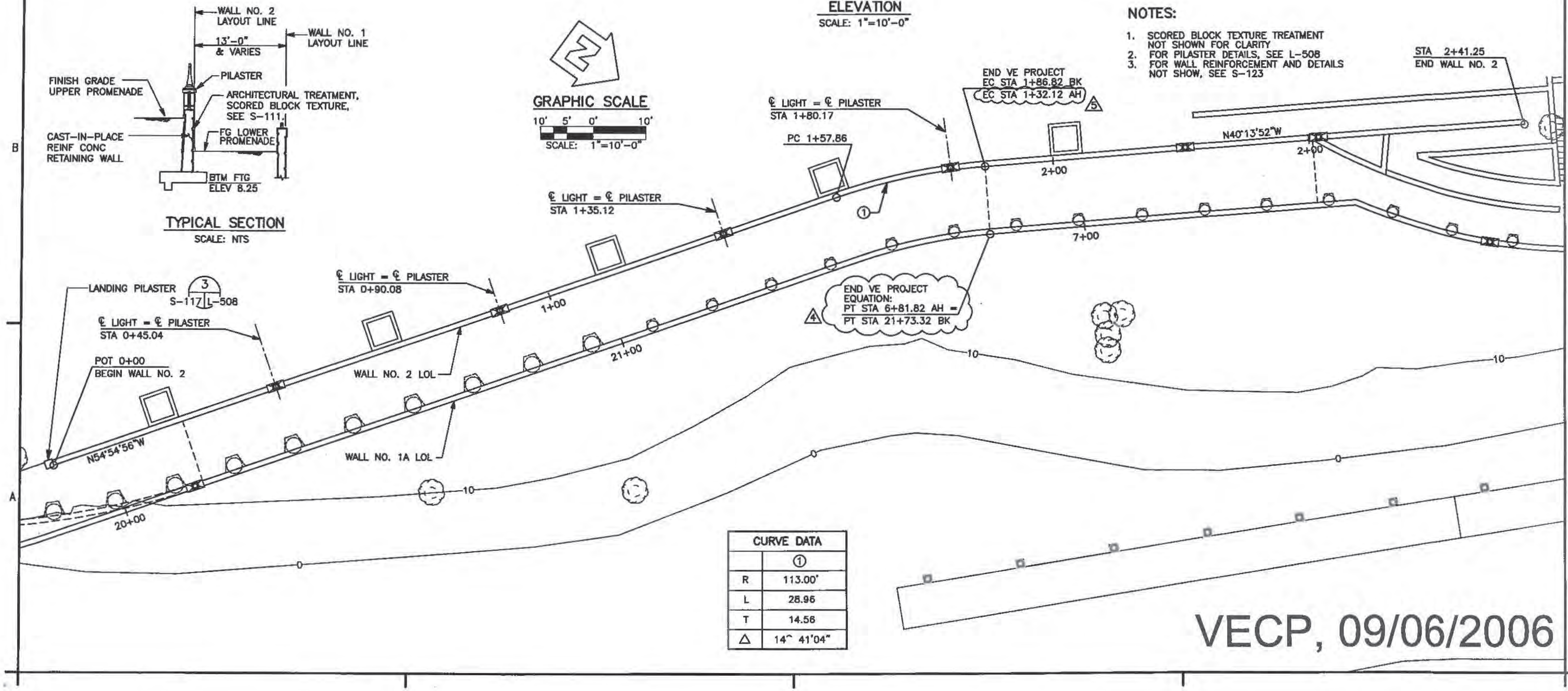
WALL NO. 1
TYPE C WALL DETAILS NO. 2

Sheet reference number:
S-111
Sheet 83 of 166



ELEVATION

SCALE: 1"=10'-0"



TYPICAL SECTION

SCALE: NTS

NOTES:

1. SCORED BLOCK TEXTURE TREATMENT NOT SHOWN FOR CLARITY
2. FOR PILASTER DETAILS, SEE L-508
3. FOR WALL REINFORCEMENT AND DETAILS NOT SHOW, SEE S-123

CURVE DATA	
	①
R	113.00'
L	28.96
T	14.56
Δ	14° 41'04"

VECP, 09/06/2006

US Army Corps of Engineers
Sacramento District

REVISIONS

Symbol	Description	Date	Approved
5	REVISED STATION		
4	MOVED STATION EQUATION TO VE LIMITS		
3	REVISED ELEVATIONS		
2	EXTENDED FOOTING		
1	REVISED WALL LENGTH		

Designed by:	Date:		Rev.
Dwn by:	Spec No.:	Design file no:	
Reviewed by:		Drawing Code:	
Submitted by:		File name: Plot date: Reg width:	

DEPARTMENT OF THE ARMY
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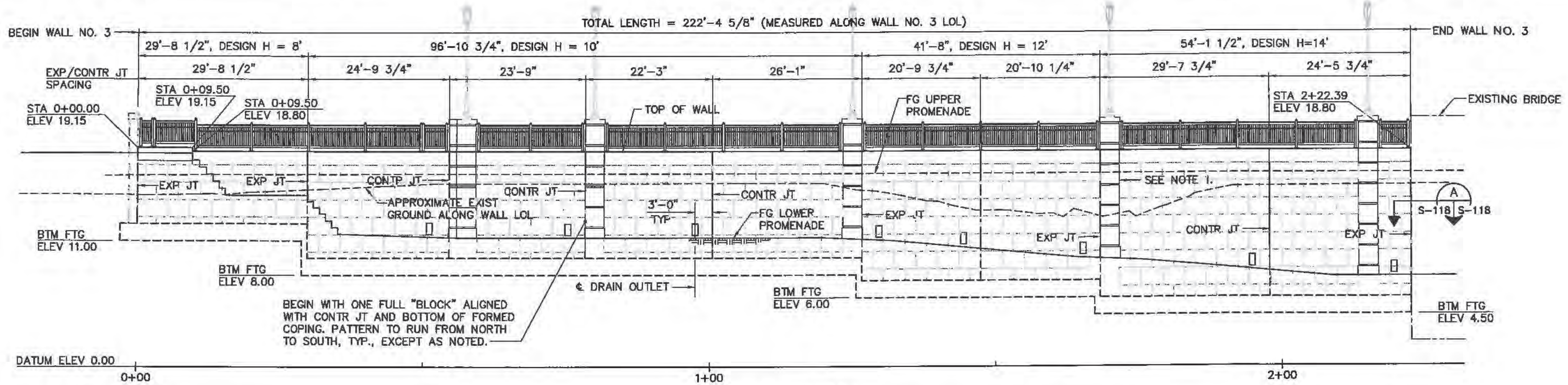
SACRAMENTO, CALIFORNIA

BE ENGINEERING INC.
900 East Main Street, Suite 100
Grass Valley, California 95945
530.477.9960

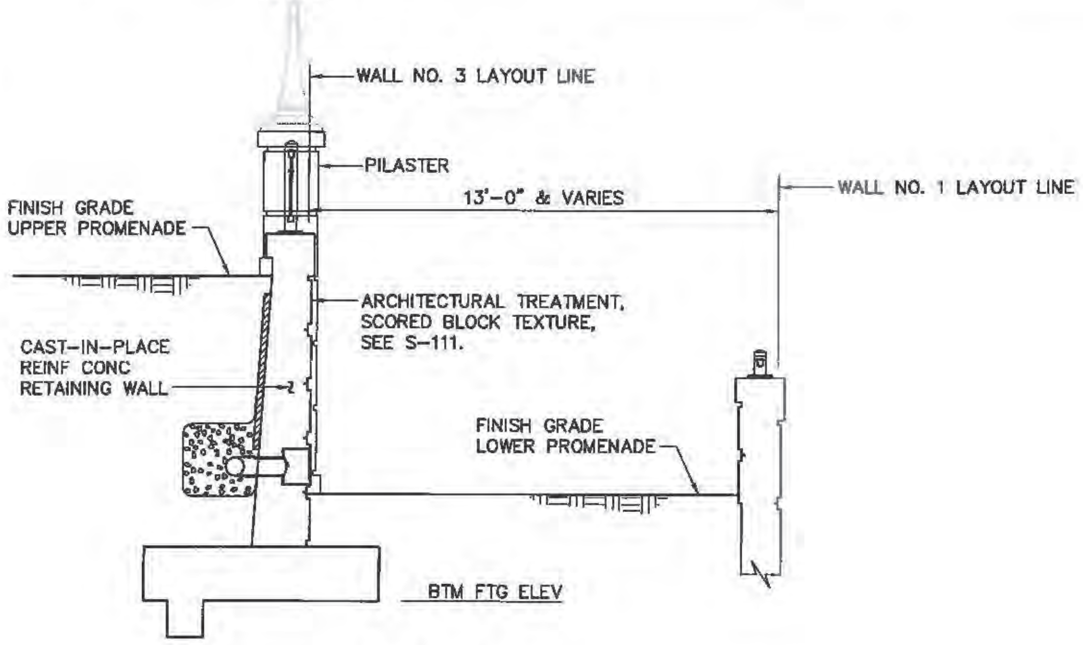
NAPA CALIFORNIA
NAPA RIVER/NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (BROWN STREET
TO FIFTH STREET)
(VALUE ENGINEERING PLANS)
WALL NO. 2
GENERAL PLAN

Sheet reference number:
S-117
Sheet of 166
VE



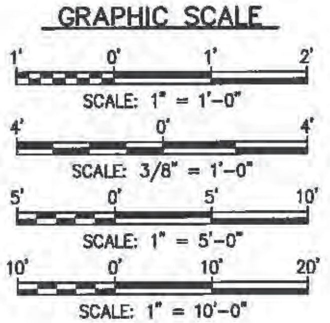
DEVELOPED ELEVATION

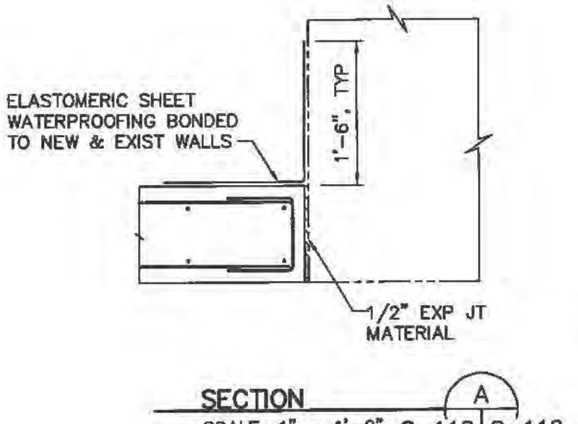
SCALE: 1" = 10'-0" (H)
1" = 5'-0" (V)



TYPICAL SECTION

SCALE: 3/8" = 1'-0"

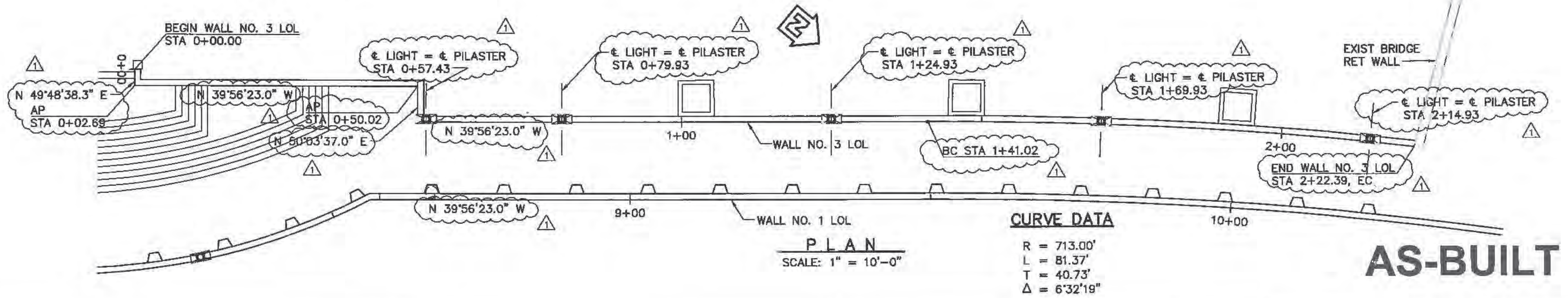




SECTION (A) S-118 | S-118

SCALE: 1" = 1'-0"

NOTES:
1. FOR PILASTER DETAILS, SEE L-508.
2. FOR WALL REINFORCEMENT AND DETAILS NOT SHOWN, SEE S-123.



P L A N

SCALE: 1" = 10'-0"

CURVE DATA

R = 713.00'
L = 81.37'
T = 40.73'
Δ = 6°32'19"

AS-BUILT

US Army Corps of Engineers Sacramento District

03/10/06 R. S.
Symbol | Description | Date | Approved
DATA UPDATED

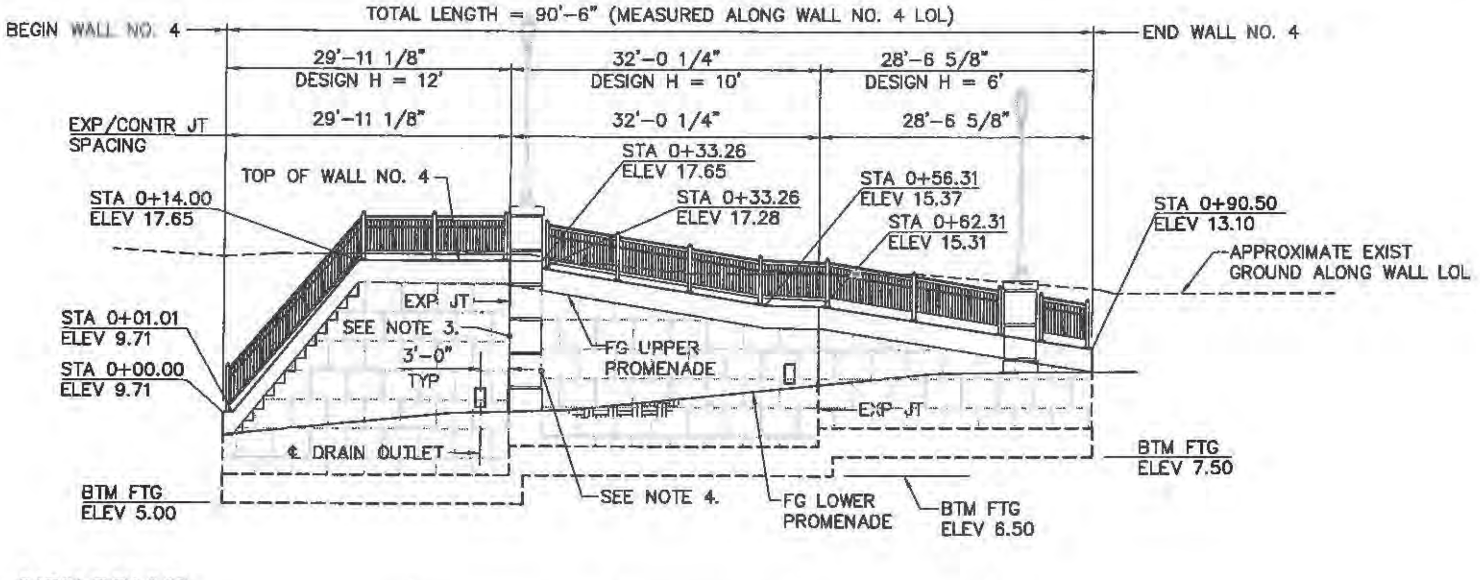
NAPA CALIFORNIA
NAPA RIVER / NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST
(HATT BUILDING TO FIRST STREET)
WALL NO. 3
GENERAL PLAN

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

MGE ENGINEERING INC.
7415 Greenhaven Dr. Suite 100
Sacramento, California 95831
916.421.1000

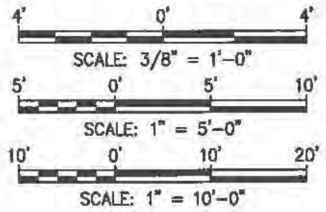
Designed by: D. AN
Dwn by: X. SUN
Spec No.: 1407
Reviewed by: R. SENNETT
Submitted by: /s/ R. SENNETT
Date: 04/271/05
Design file no: NA-25-030
Drawing Code:
File name:
Plot date:
Plot scale: 1:1
Rev.

Sheet reference number:
S-118
Sheet 90 of 166

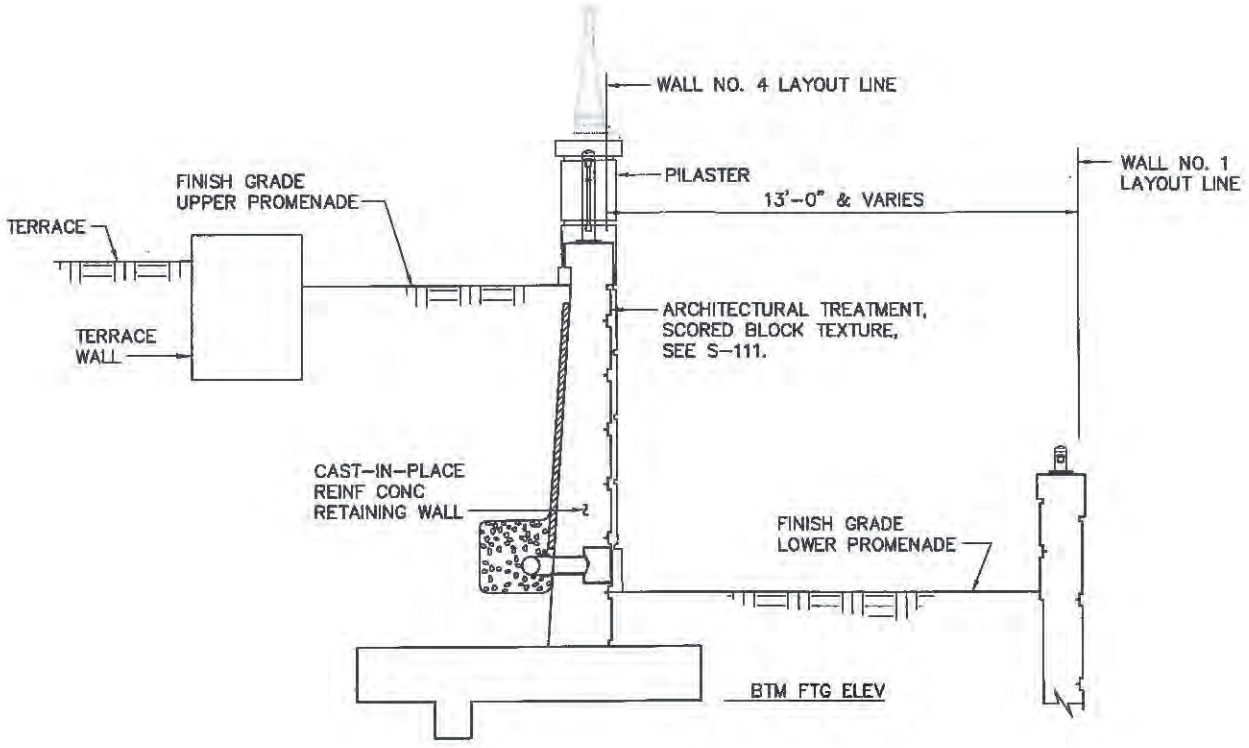


DEVELOPED ELEVATION
SCALE: 1" = 10'-0" (H)
1" = 5'-0" (V)

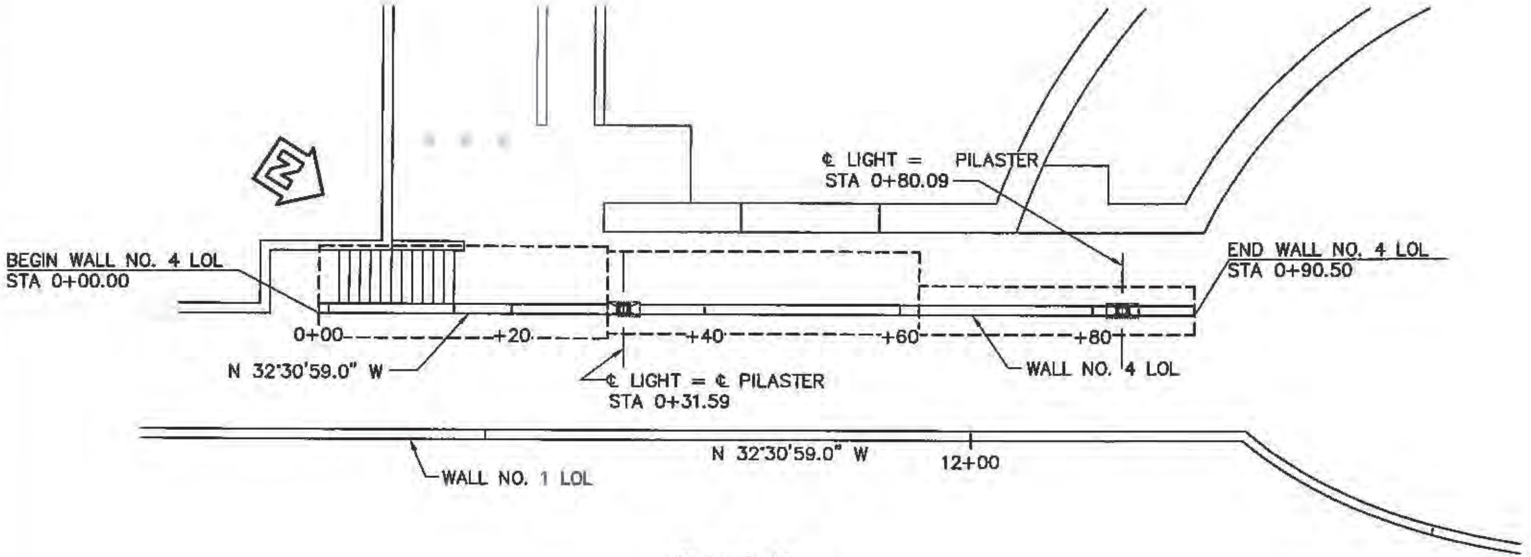
GRAPHIC SCALE



US Army Corps of Engineers Sacramento District



TYPICAL SECTION
SCALE: 3/8" = 1'-0"



PLAN
SCALE: 1" = 10'-0"

AS-BUILT

NOTES:
1. FOR PILASTER DETAILS, SEE L-508.
2. FOR WALL REINFORCEMENT & DETAILS NOT SHOWN, SEE S-123.
3. BEGIN WITH ONE FULL "BLOCK" ALIGNED WITH CONTR JT AND BOTTOM OF FORMED COPING. PATTERN TO RUN FROM NORTH TO SOUTH, TYP., EXCEPT AS NOTED.
4. BLOCK PATTERN TO RUN FROM SOUTH TO NORTH TO END OF WALL.

Designed by: D. AN	Date: 04/27/05	Rev.
Dwn by: X. SUN	Spec No.: 1407	Design file no: NA-25-030
Reviewed by: R. SENNETT		Drawing Code:
Submitted by: /s/ R. SENNETT		File name: Plot date: Plot scale: 1:1

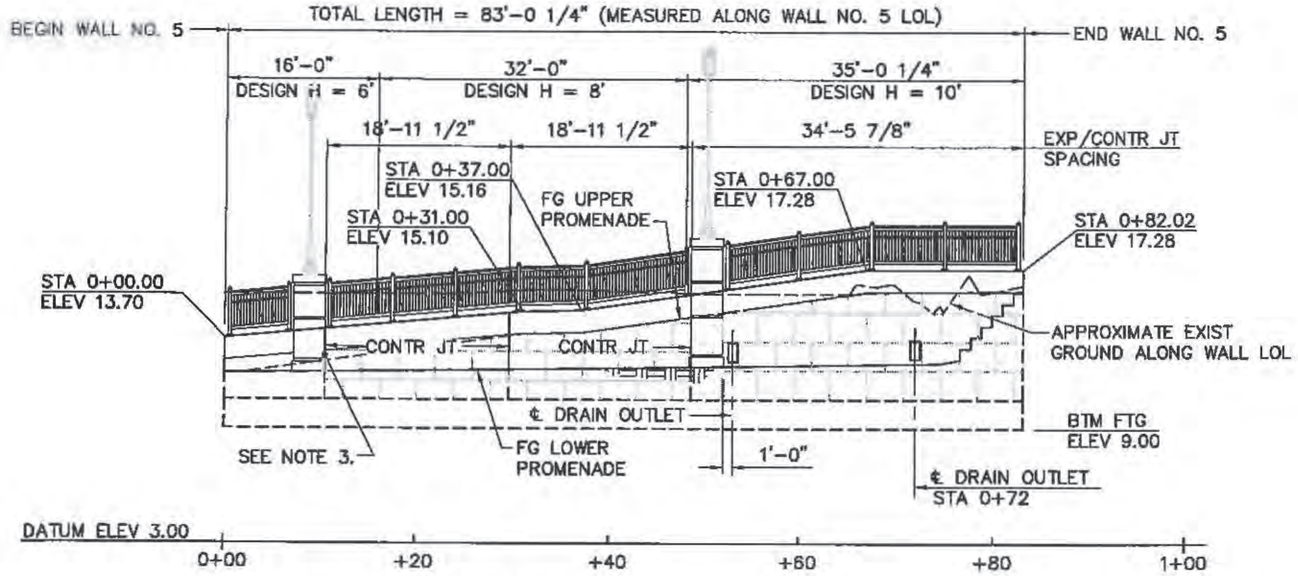
DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

MGE ENGINEERING INC.
7415 Greenhaven Dr. Suite 100
Sacramento, California 95831
916.421.1000

NAPA CALIFORNIA
NAPA RIVER / NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST
(HATT BUILDING TO FIRST STREET)

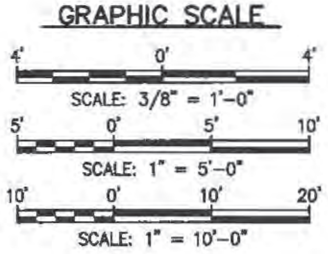
WALL NO. 4
GENERAL PLAN

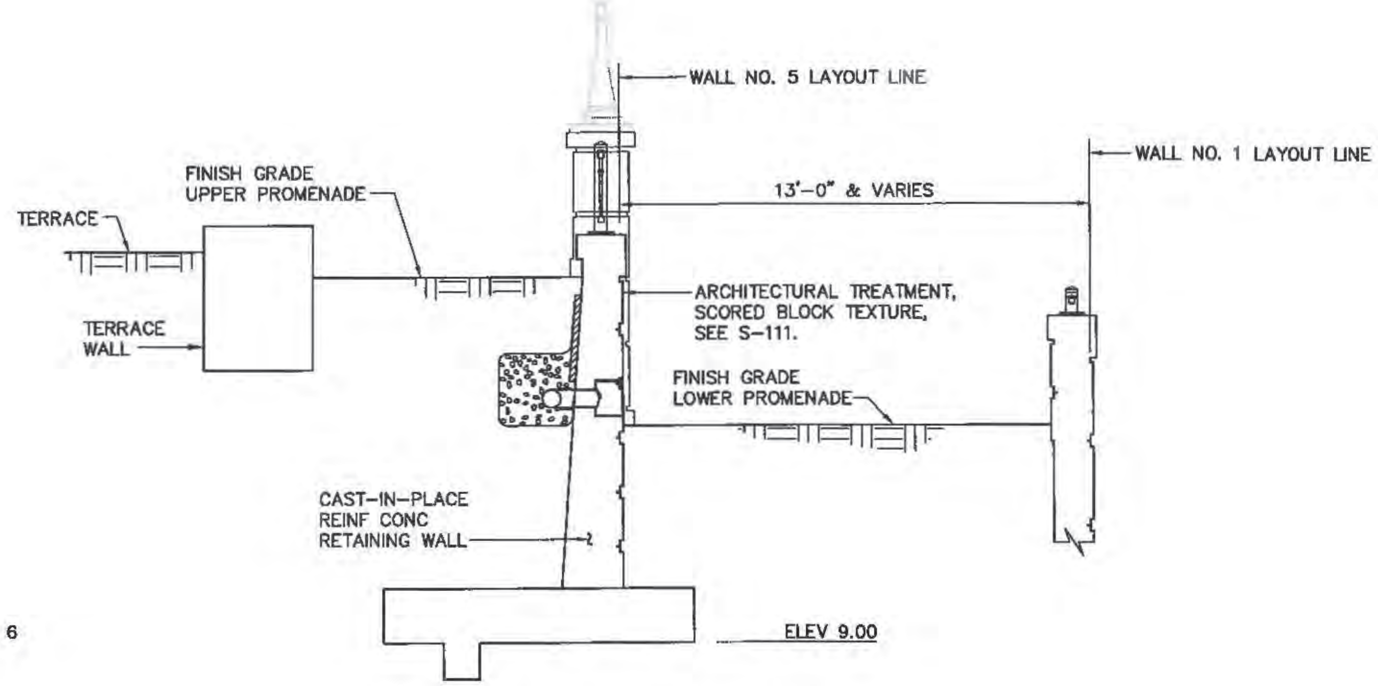
Sheet reference number:
S-119
Sheet 91 of 166



DEVELOPED ELEVATION

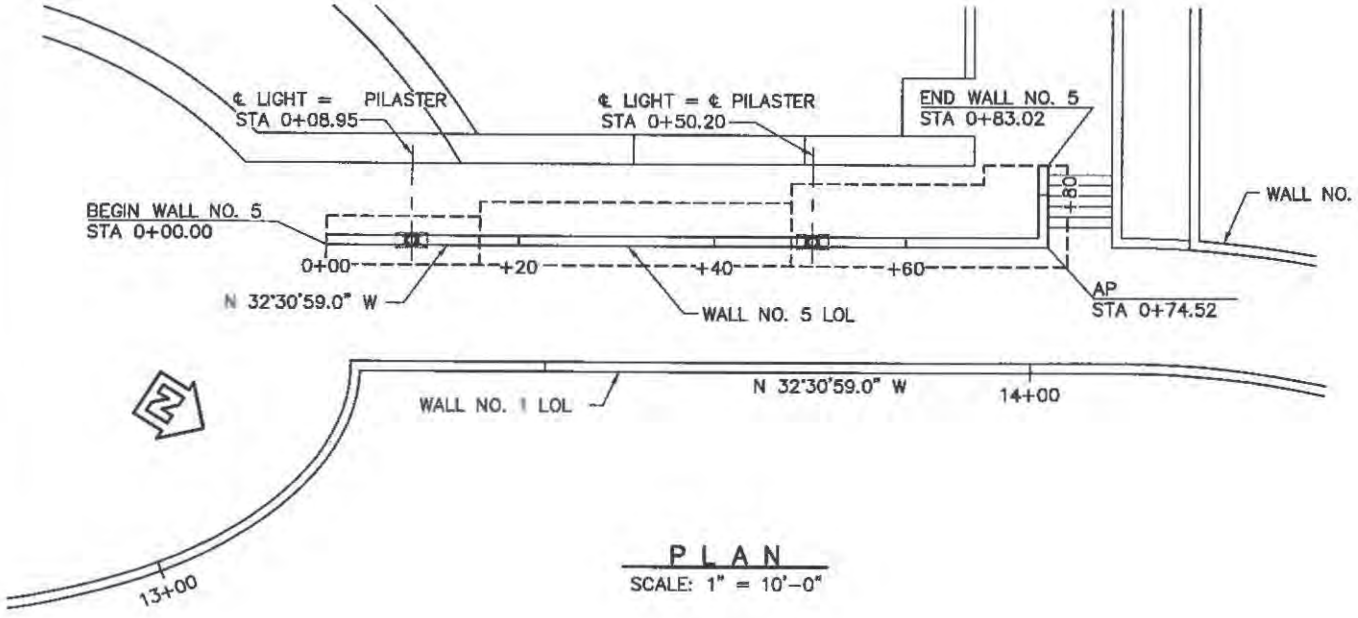
SCALE: 1" = 10'-0" (H)
1" = 5'-0" (V)





TYPICAL SECTION

SCALE: 3/8" = 1'-0"



PLAN

SCALE: 1" = 10'-0"

NOTES:
1. FOR PILASTER DETAILS, SEE L-508.
2. FOR WALL REINFORCEMENT & DETAILS NOT SHOWN, SEE S-123.
3. BLOCK PATTERN TO RUN FROM SOUTH TO NORTH TO END OF WALL.

AS-BUILT

US Army Corps
of Engineers
Sacramento District

Symbol	Description	Date	Approved

Designed by: D. AN
Dwn by: X. SUN
Spec No.: 1407
Reviewed by: R. SENNETT
Submitted by: /s/ R. SENNETT
Date: 04/27/05
Rev.
Design file no: NA-25-030
Drawing Code:
File name:
Plot date:
Plot scale: 1:1

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

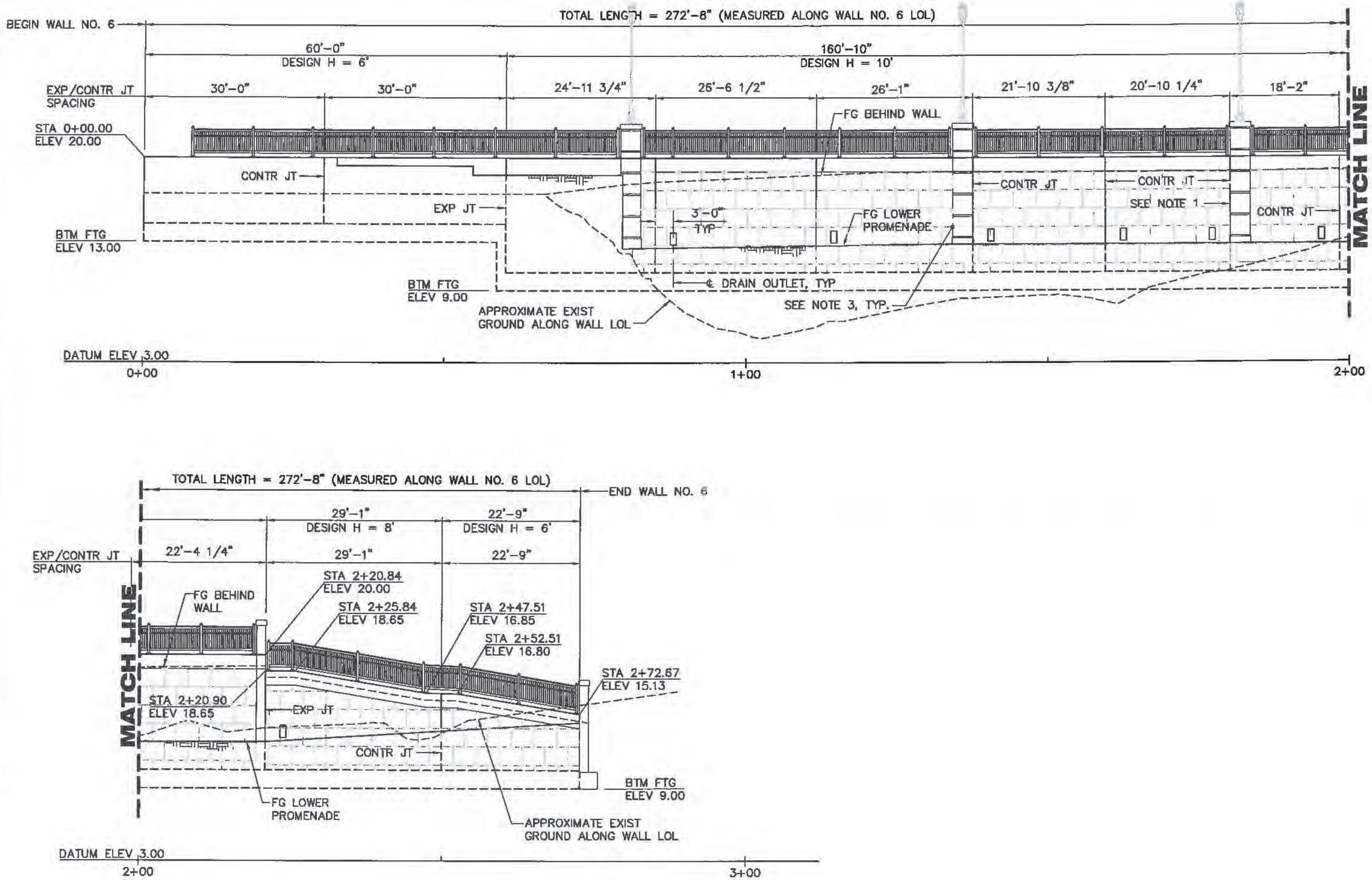
MGE ENGINEERING INC.
7415 Greenhaven Dr., Suite 100
Sacramento, California 95831
916.421.1000

NAPA CALIFORNIA
NAPA RIVER / NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST
(HATT BUILDING TO FIRST STREET)

WALL NO. 5
GENERAL PLAN

Sheet reference number:
S-120

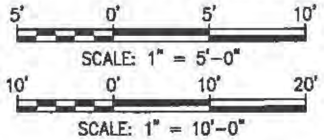
Sheet 92 of 166



DEVELOPED ELEVATION
SCALE: 1" = 10'-0" (H)
1" = 5'-0" (V)

NOTES:
1. FOR PILASTER DETAILS, SEE L-508.
2. FOR WALL REINFORCEMENT DETAILS NOT SHOWN, SEE S-123.
3. BEGIN WITH ONE FULL "BLOCK" ALIGNED WITH CONTR JT AND BOTTOM OF FORMED COPING. PATTERN TO RUN FROM NORTH TO SOUTH, TYP., EXCEPT AS NOTED.

GRAPHIC SCALE



AS-BUILT

US Army Corps
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DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

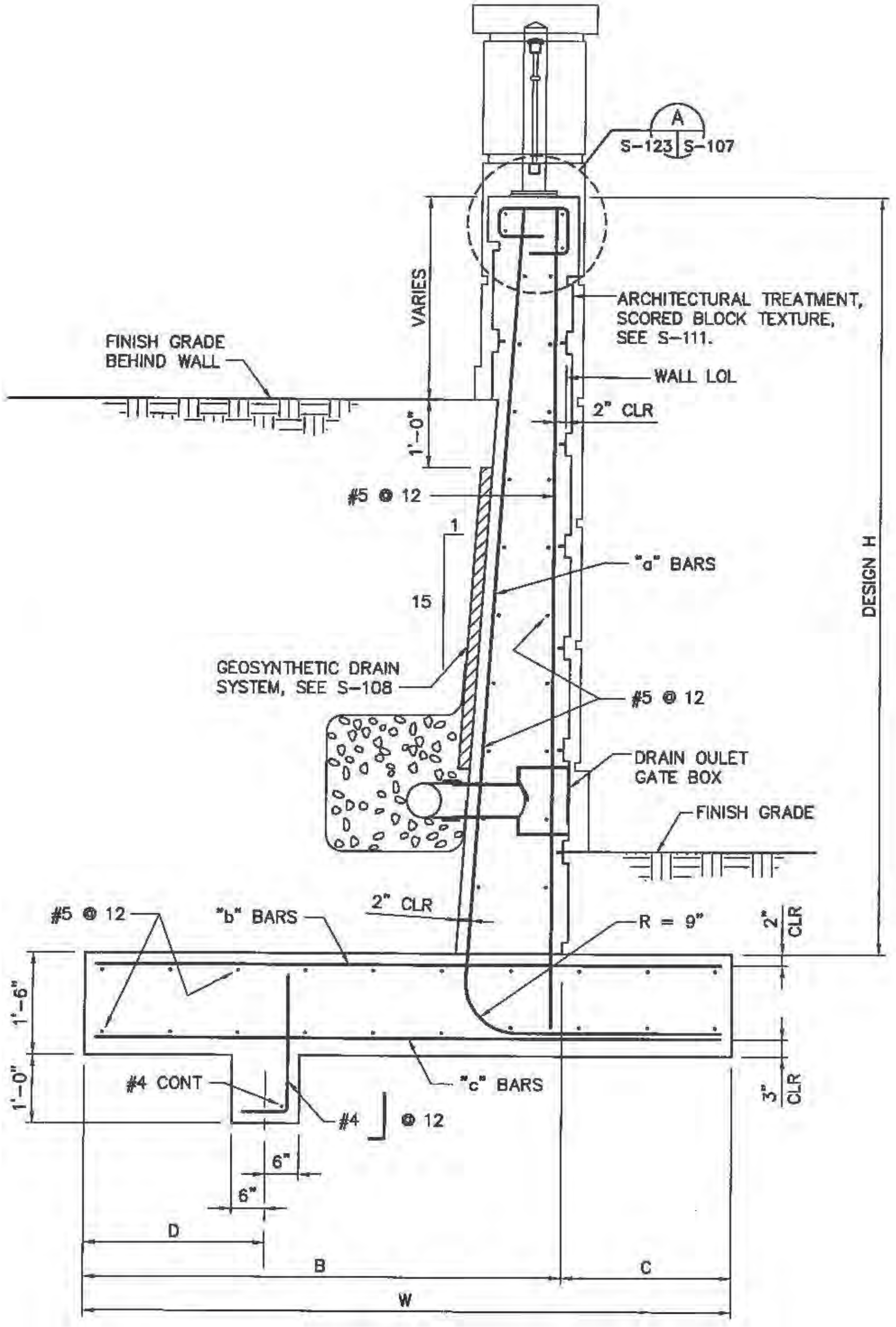
MGE ENGINEERING INC.
7415 Greenhaven Dr. Suite 100
Sacramento, California 95831
916.421.1000

Designed by: D. AN		Date: 04/27/05
Dwn by: X. SUN	Spec No.: 1407	Design file no: NA-25-030
Reviewed by: R. SENNETT		Drawing Code:
Submitted by: /s/ R. SENNETT		File name: Plot date: Plot scale: 1:1

NAPA
NAPA RIVER / NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST
(HATT BUILDING TO FIRST STREET)
CALIFORNIA

WALL NO. 6
DEVELOPED ELEVATION

Sheet
reference
number:
S-122
Sheet 94 of 166



TYPICAL SECTION

SCALE: 3/4" = 1'-0"

TABLE OF WALL DIMENSIONS AND REINFORCING DATA

DESIGN H (FT)	6	8	10	12	14
W (Ft)	5.00	6.50	8.50	9.50	11.00
C (Ft)	2.00	2.00	2.00	2.50	2.75
B (Ft)	3.00	4.50	6.50	7.00	8.25
D (Ft)	1.17	1.17	2.17	2.17	2.17
"a" BARS	#5 @ 12	#6 @ 12	#6 @ 9	#7 @ 9	#8 @ 9
"b" BARS	#4 @ 12	#5 @ 12	#5 @ 12	#6 @ 12	#6 @ 10
"c" BARS	#4 @ 12	#4 @ 12	#5 @ 12	#7 @ 12	#7 @ 10

GRAPHIC SCALE

2' 0' 2'

SCALE: 3/4" = 1'-0"

NOTE:
1. FOR OUTLET GATE BOX DETAILS NOT SHOWN, SEE S-108.

AS-BUILT

US Army Corps of Engineers
Sacramento District

Symbol | Description | Date | Approved

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

MBE ENGINEERING INC.
7415 Greenhaven Dr. Suite 100
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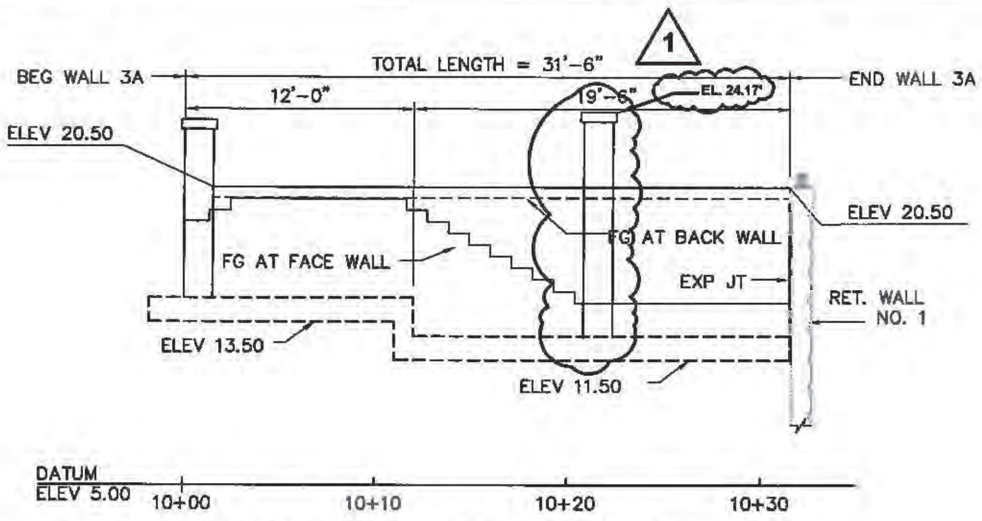
916.421.1000

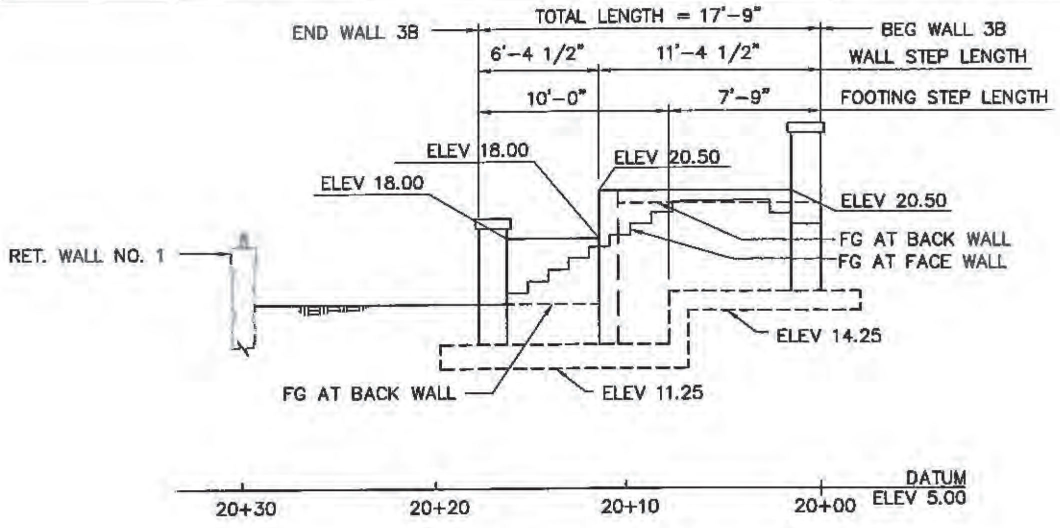
Designed by: D. AN
Dwn by: X. SUN
Spec No.: 1407
Reviewed by: R. SENNETT
Submitted by: /s/ R. SENNETT
Date: 04/27/05
Rev.
Design file no: NA-25-030
Drawing Code:
File name:
Plot date:
Plot scale: 1:1

NAPA CALIFORNIA
NAPA RIVER / NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST
(HATT BUILDING TO FIRST STREET)
RETAINING WALL NO. 2, 3, 4, 5, & 6
TYPICAL DETAILS

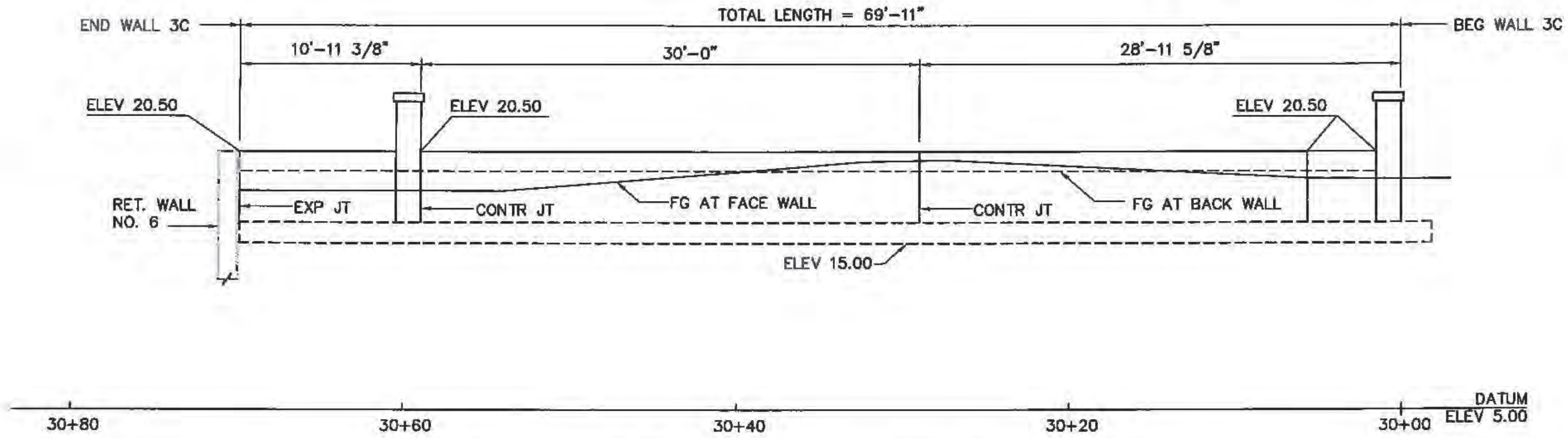
Sheet reference number:
S-123



DEVELOPED ELEVATION – WALL 3A
SCALE: 1" = 5'-0"

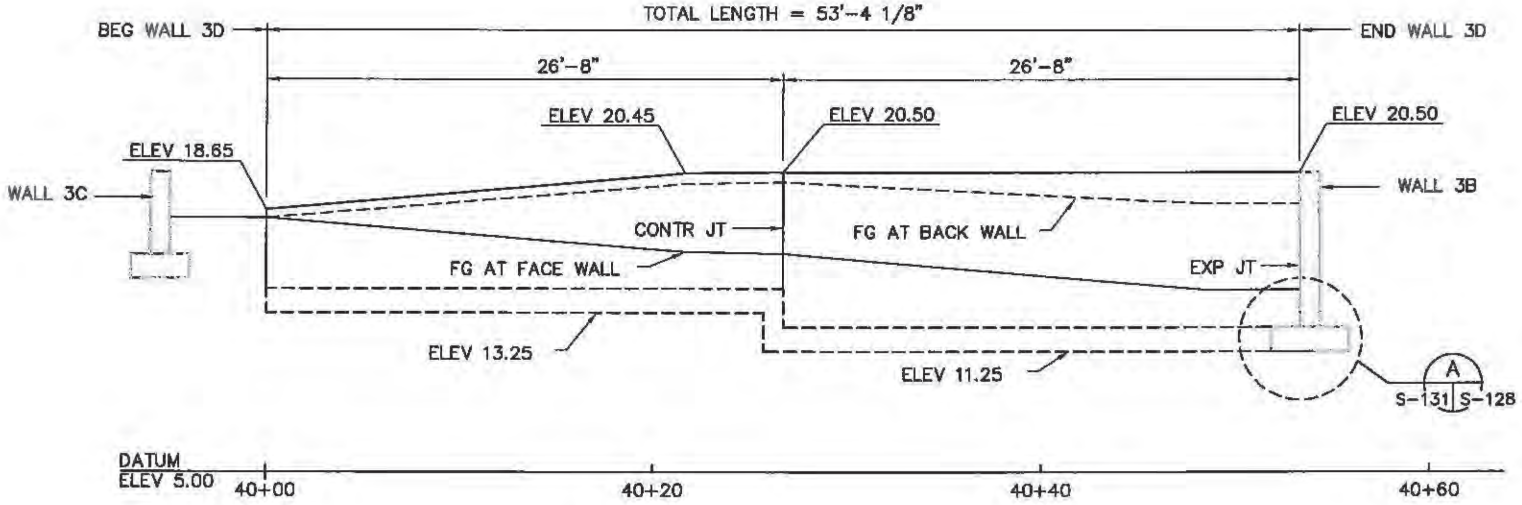


DEVELOPED ELEVATION – WALL 3B
SCALE: 1" = 5'-0"



DEVELOPED ELEVATION – WALL 3C
SCALE: 1" = 5'-0"

GRAPHIC SCALE
5' 0' 5' 10'
SCALE: 1" = 5'-0"



DEVELOPED ELEVATION – WALL 3D
SCALE: 1" = 5'-0"

REVISED AS-BUILT

NOTES:
1. FOR DIMENSIONS AND ELEVATIONS NOT SHOWN, SEE SHEET L-516.
2. RAILING NOT SHOWN FOR CLARITY.

US Army Corps
of Engineers
Sacramento District

Symbol	Description	Date	Approved
1	ADD LANDING PILASTER PER RFI 86	10/17/07	AN

Designed by: D. AN
Date: 04/27/05
Rev.
Dwn by: X. SUN
Spec No.: 1407
Design file no: NA-25-030
Reviewed by: R. SENNETT
Drawing Code:
Submitted by: /s/ R. SENNETT
File name:
Plot date:
Plot scale: 1:1

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

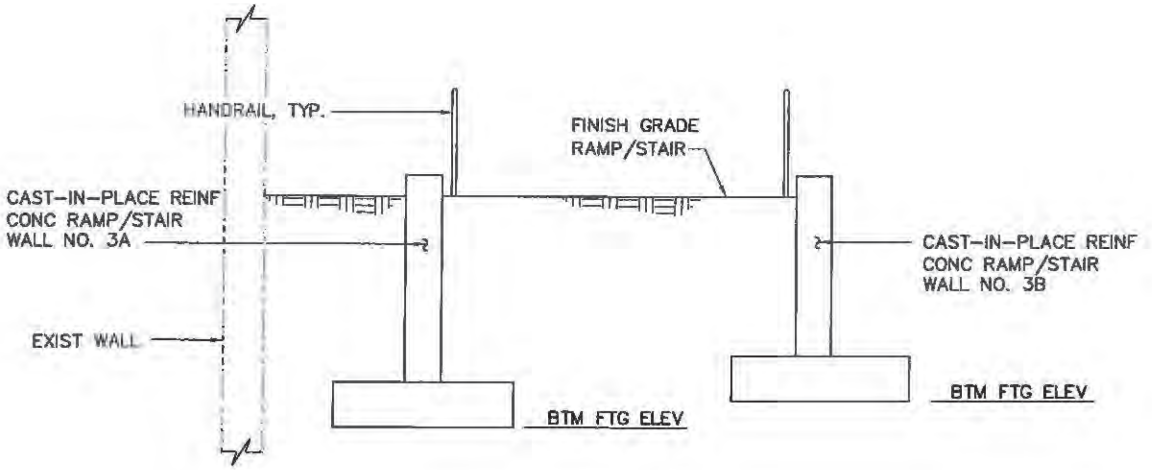
MGE ENGINEERING INC.
7415 Greenhaven Dr. Suite 100
Sacramento, California 95831
916.421.1000

Soil Design Section Sheet No.:

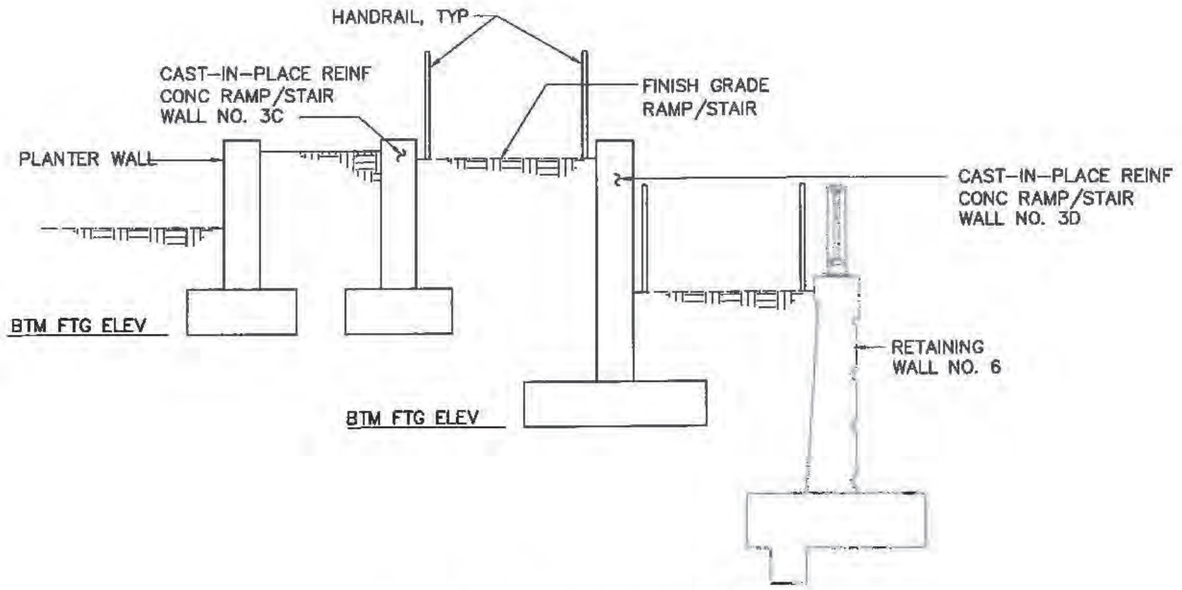
NAPA CALIFORNIA
NAPA RIVER / NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST
(HATT BUILDING TO FIRST STREET)
RAMP/STAIR ACCESS NO. 3
DEVELOPED ELEVATIONS

Reference – Exploration Log file no.:

Sheet reference number:
S-131
Sheet 103 of 166



SECTION A S-130 | S-132
SCALE: 3/8" = 1'-0"



SECTION B S-130 | S-132
SCALE: 3/8" = 1'-0"

NOTES:
1. FOR REINFORCEMENT DETAILS NOT SHOWN, SEE SHEETS S-133 & S-134.
2. FOR HANDRAIL DETAILS NOT SHOWN, SEE SHEET L-516.
3. RAILING NOT SHOWN FOR CLARITY.

AS-BUILT

GRAPHIC SCALE
4' 0' 4'
SCALE: 3/8" = 1'-0"

US Army Corps
of Engineers
Sacramento District

Symbol	Description	Date	Approved

Designed by: D. AN
Dwn by: X. SUN
Spec No.: 1407
Reviewed by: R. SENNETT
Submitted by: /s/ R. SENNETT
Date: 04/27/05
Rev.
Design file no: NA-25-030
Drawing Code:
File name:
Plot date:
Plot scale: 1:1

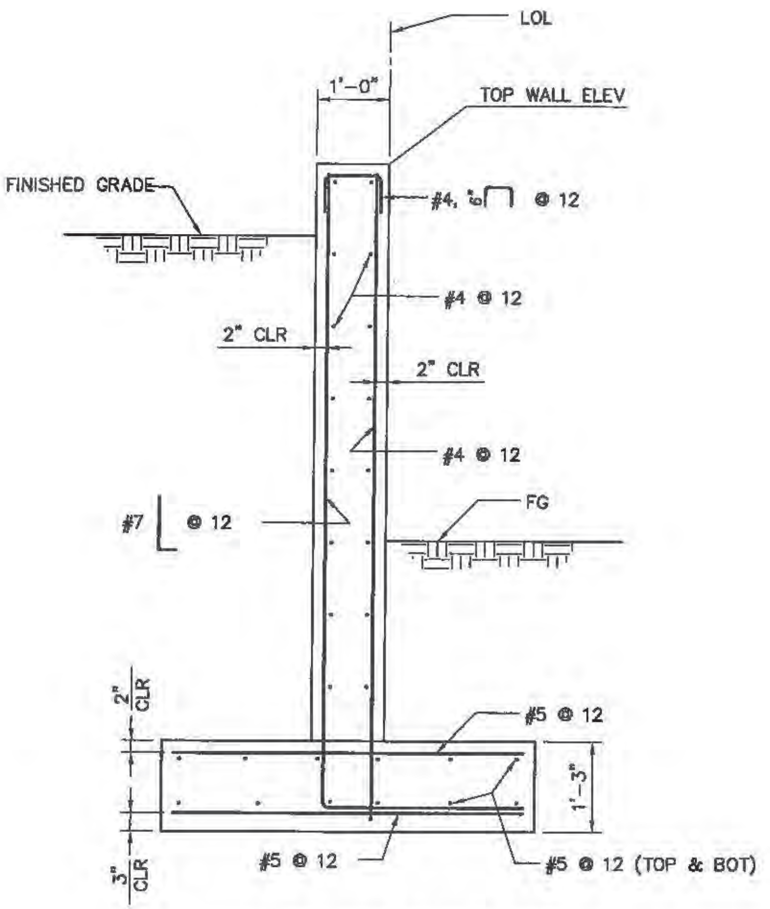
DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

MGE ENGINEERING INC.
7415 Greenhaven Dr. Suite 100
Sacramento, California 95831
916.421.1000

NAPA CALIFORNIA
NAPA RIVER / NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST
(HATT BUILDING TO FIRST STREET)
RAMP/STAIR ACCESS NO. 3
SECTIONS AND DETAILS

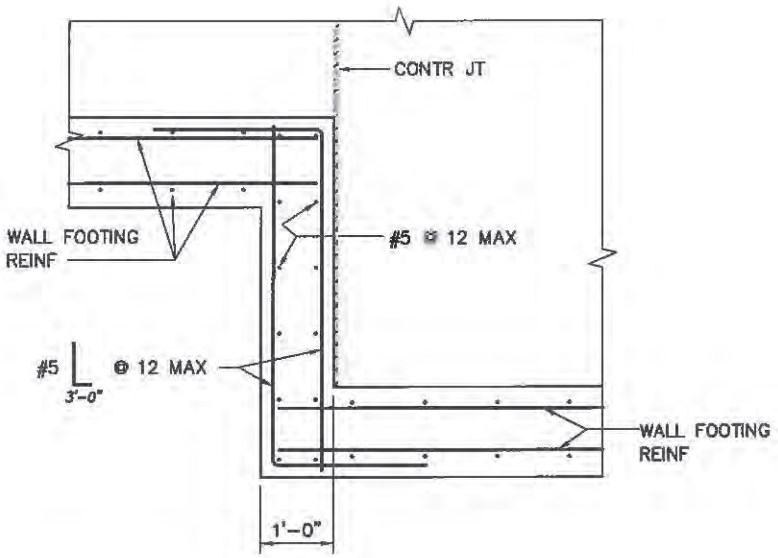
Sheet reference number:
S-132
Sheet 104 of 166

Soil Design Section Sheet No.:
Reference - Exploration Log file no.:



WALL REINFORCEMENT DETAILS

SCALE: 3/4" = 1'-0"



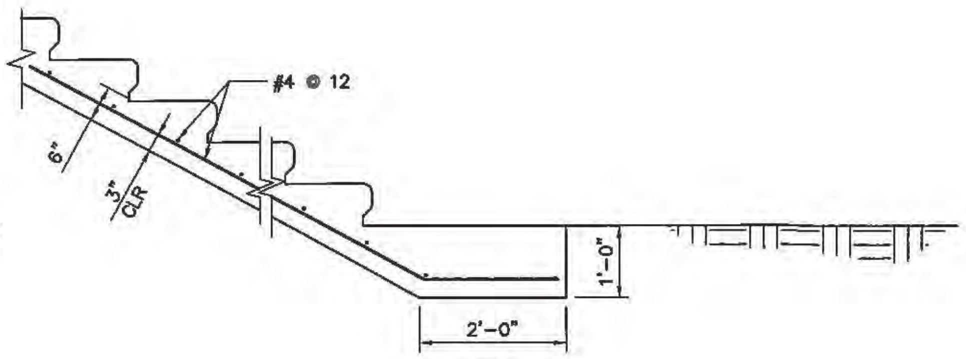
FOOTING STEP DETAILS

SCALE: 3/4" = 1'-0"

GRAPHIC SCALE

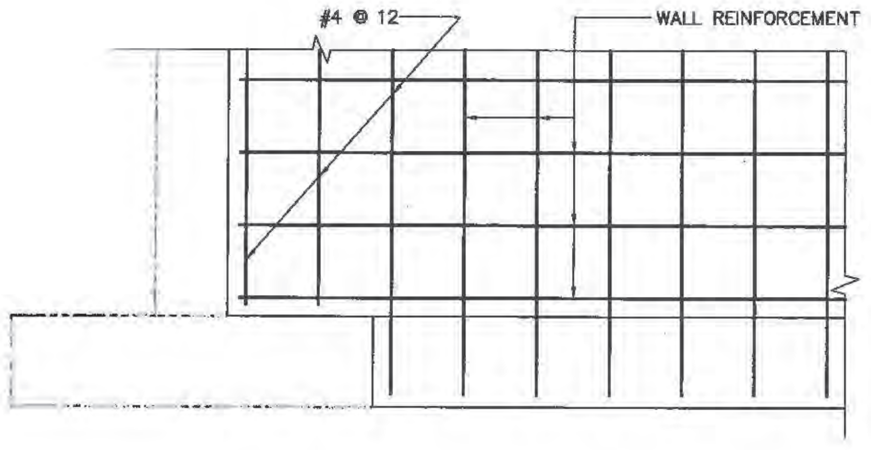
2' 0' 2'

SCALE: 3/4" = 1'-0"



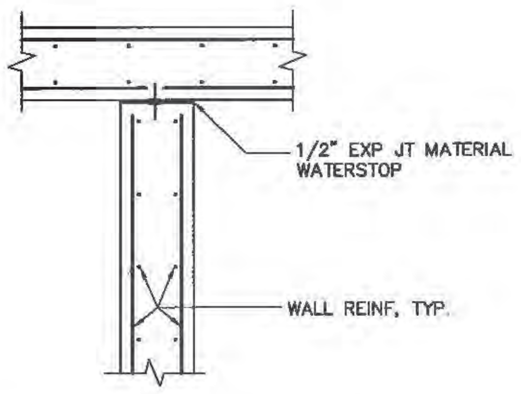
STAIR REINFORCEMENT DETAILS

SCALE: 3/4" = 1'-0"



WALL REINFORCEMENT AT INTERSECTIONS

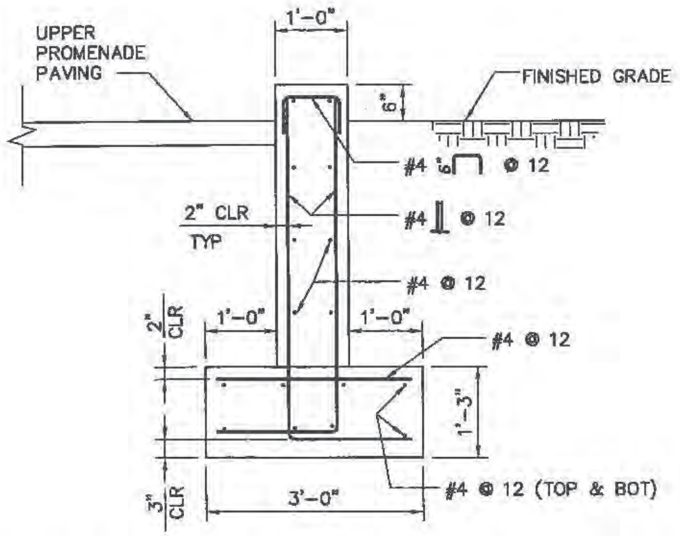
SCALE: 3/4" = 1'-0"



EXPANSION JOINT DETAILS

SCALE: 3/4" = 1'-0"

AS-BUILT



PLANTER WALL REINFORCEMENT DETAILS

SCALE: 3/4" = 1'-0"

US Army Corps
of Engineers
Sacramento District

Date | Approved | Description | Symbol

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

MGE ENGINEERING INC.
7415 Greenhaven Dr. Suite 100
Sacramento, California 95831
916.421.1000

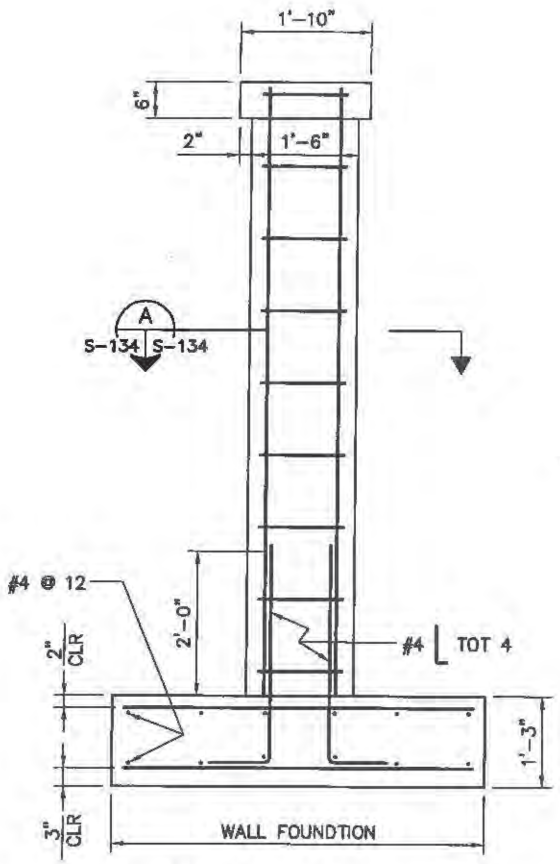
Designed by: D. AN
Dwn by: X. SUN
Spec No.: 1407
Reviewed by: R. SENNETT
Submitted by: /s/ R. SENNETT
Date: 04/27/05
Rev.
Design file no: NA-25-030
Drawing Code:
File name:
Plot date:
Plot scale: 1:1

NAPA CALIFORNIA
NAPA RIVER / NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST
(HATT BUILDING TO FIRST STREET)

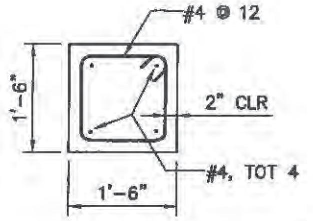
RAMP/STAIR ACCESS
TYPICAL DETAILS NO. 1

Sheet reference number:
S-133
Sheet 105 of 166

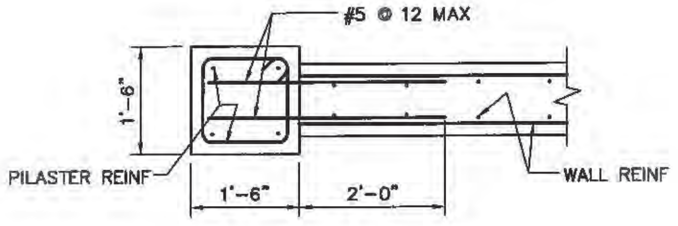
Soil Design Section Sheet No.:
Reference – Exploration Log file no.:



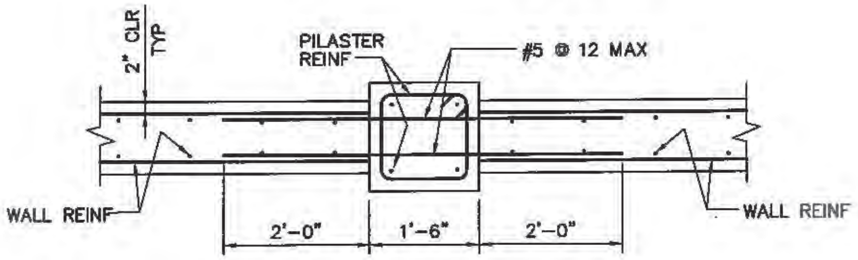
PILASTER REINFORCEMENT
SCALE: 3/4" = 1'-0"



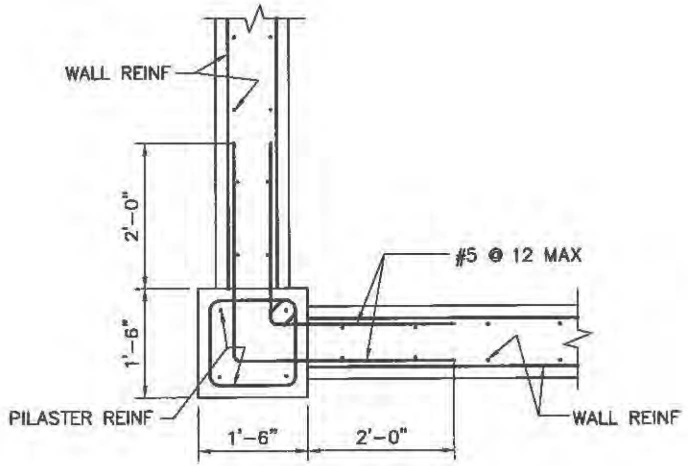
SECTION
SCALE: 3/4" = 1'-0" S-134 A S-134



END PILASTER REINFORCEMENT
SCALE: 3/4" = 1'-0"



MIDDLE PILASTER REINFORCEMENT
SCALE: 3/4" = 1'-0"



CORNER PILASTER REINFORCEMENT
SCALE: 3/4" = 1'-0"

AS-BUILT

GRAPHIC SCALE

SCALE: 3/4" = 1'-0"

US Army Corps
of Engineers
Sacramento District

Symbol
Description
Date
Approved

Designed by:
D. AN
Dwn by:
X. SUN
Spec No.:
1407
Reviewed by:
R. SENNETT
Submitted by:
/s/ R. SENNETT

Date:
04/27/05
Rev.
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Plot scale: 1:1

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

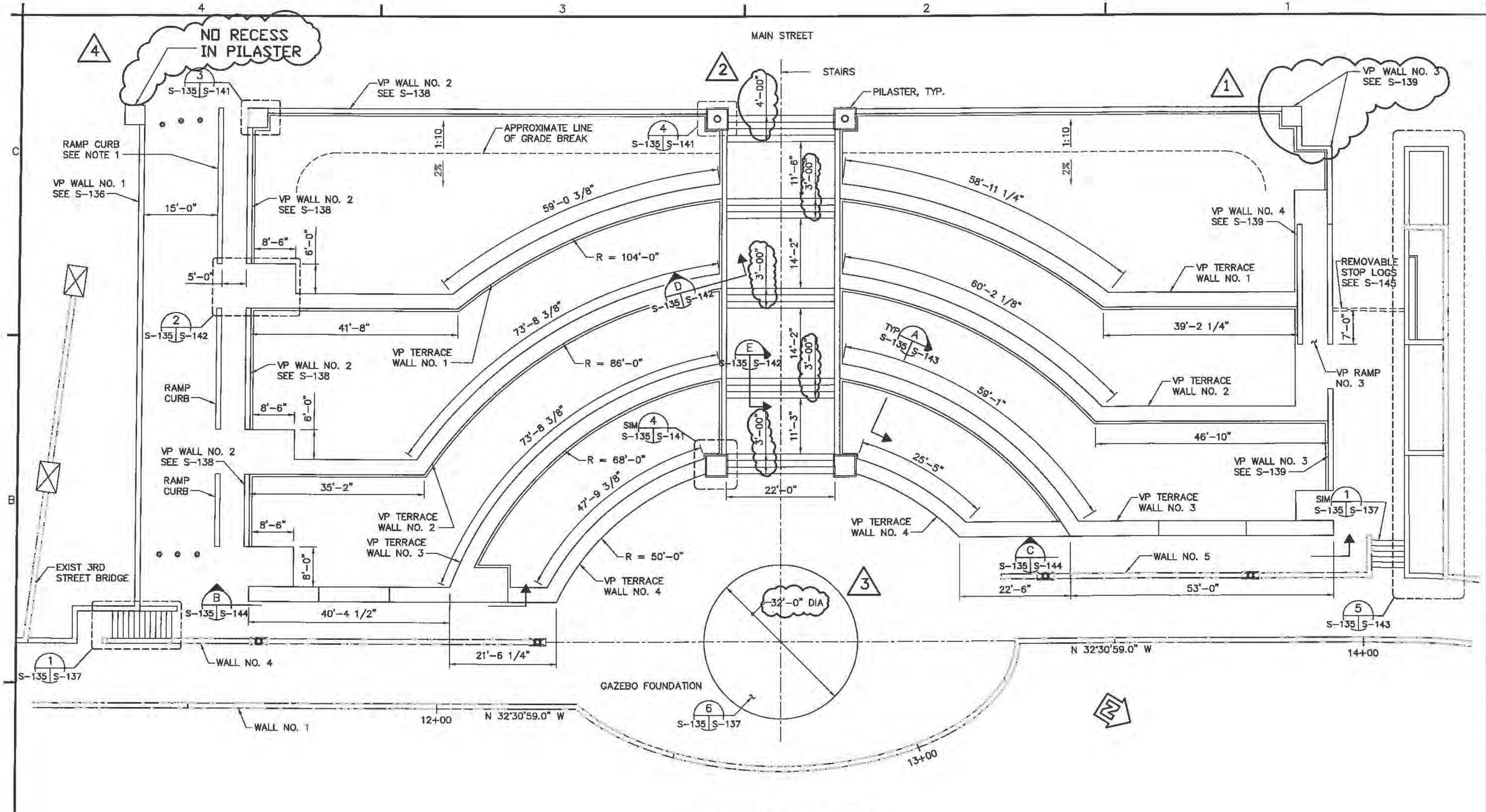
MGE ENGINEERING INC.
7415 Greenhaven Dr. Suite 100
Sacramento, California 95831
916.421.1000

NAPA
CALIFORNIA
NAPA RIVER / NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST
(HATT BUILDING TO FIRST STREET)
RAMP/STAIR ACCESS
TYPICAL DETAILS NO. 2

Sheet
reference
number:
S-134
Sheet 106 of 166

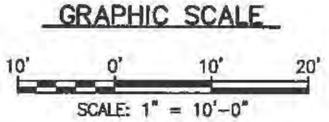
Soil Design Section Sheet No.:

Reference - Exploration Log file no.:



PLAN – VETERANS PARK

SCALE: 1" = 10'-0"



NOTES:
1. REFER TO C-SHEETS FOR DETAILS OF RAMPS & CURBS.
2. FOR ELEVATIONS, DIMENSIONS AND DETAILS NOT SHOWN, SEE SHEETS C-432, C-433, L-509 & L-515.

REVISED AS-BUILT

US Army Corps
of Engineers
Sacramento District

Symbol	Description		Date	Approved
4	CLARIFICATION PER RFI 90	AN	[illegible]	
3	CHANGES PER RFI 95 – GAZEBO CLARIFICATION	AN	[illegible]	
2	CHANGES PER RFI 96 – STEP LIGHTS @ STAIRWAY	AN	[illegible]	
1	CHANGES PER RFI 97 – UTILITIES CONFLICT @VETS PARK	AN	[illegible]	

Designed by: D. AN		Date: 04/27/05	Rev.
Dwn by: X. SUN	Spec No.: 1407	Design file no: NA-25-030	
Reviewed by: R. SENNETT		Drawing Code:	
Submitted by: /s/ R. SENNETT		File name: Plot date: Plot scale: 1:1	

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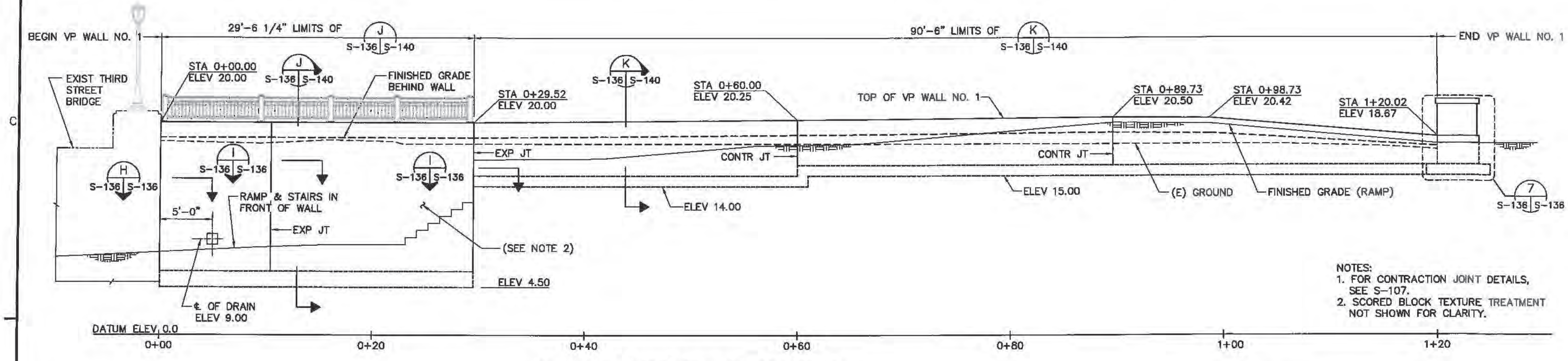
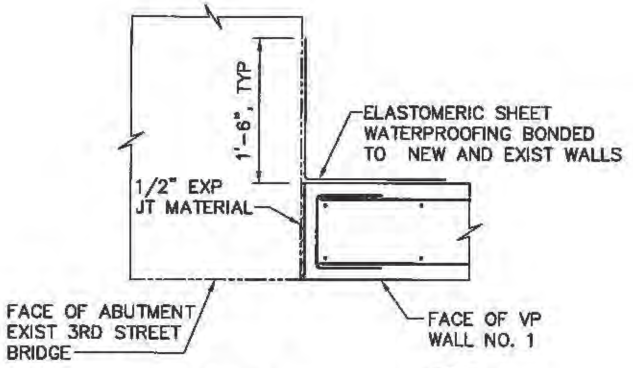
NAPA CALIFORNIA
NAPA RIVER / NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST
(HATT BUILDING TO FIRST STREET)
VETERANS PARK
PLAN

Sheet reference number:
S-135
Sheet 107 of 166

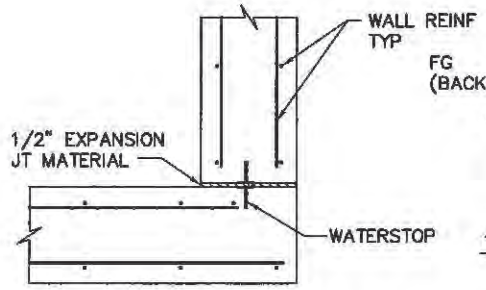


NOTES:
1. FOR CONTRACTION JOINT DETAILS, SEE S-107.
2. SCORED BLOCK TEXTURE TREATMENT NOT SHOWN FOR CLARITY.

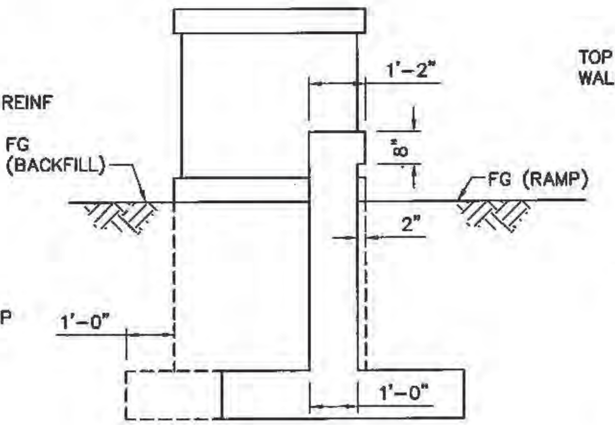
DEVELOPED ELEVATION – VP WALL NO. 1
SCALE: 1" = 5'-0"



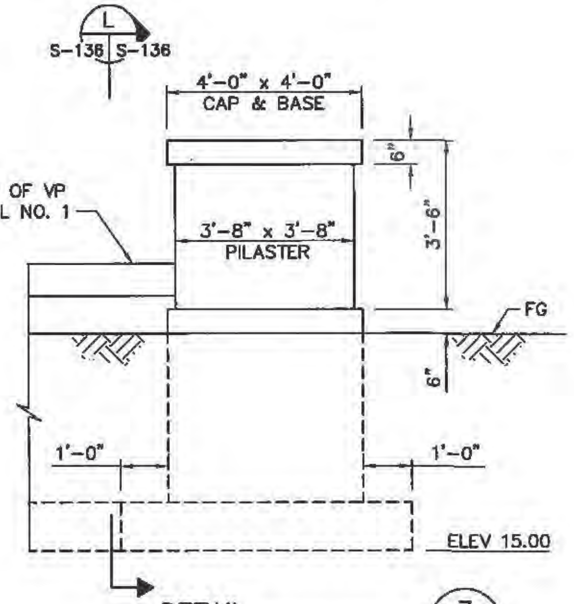
SECTION H
SCALE: 1" = 1'-0" S-136 S-136



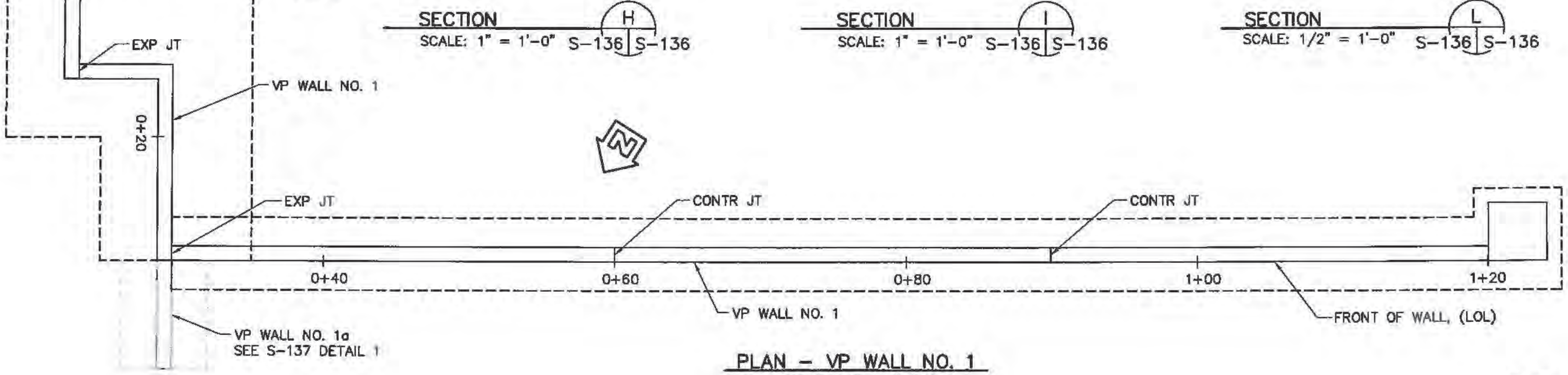
SECTION I
SCALE: 1" = 1'-0" S-136 S-136



SECTION L
SCALE: 1/2" = 1'-0" S-136 S-136



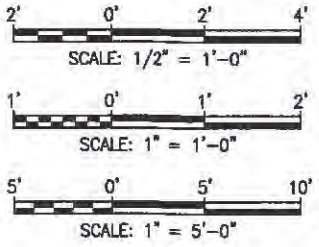
DETAIL 7
SCALE: 1/2" = 1'-0" S-136 S-136



PLAN – VP WALL NO. 1
SCALE: 1" = 5'-0"

AS-BUILT

GRAPHIC SCALE



US Army Corps of Engineers Sacramento District

Designed by: D. AN
Dwn by: X. SUN
Spec No.: 1407
Reviewed by: R. SENNETT
Submitted by: /s/ R. SENNETT
Date: 04/27/05
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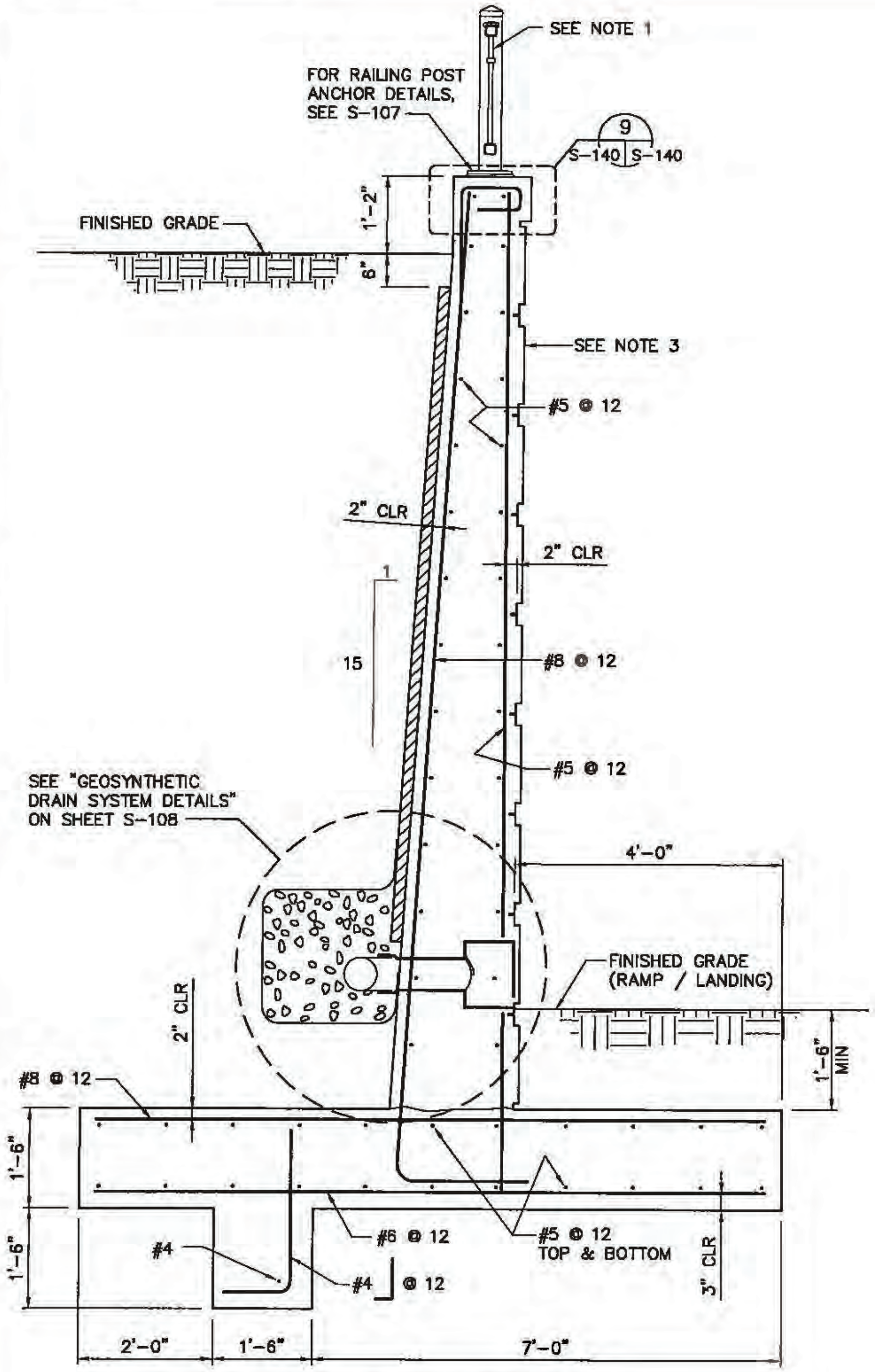
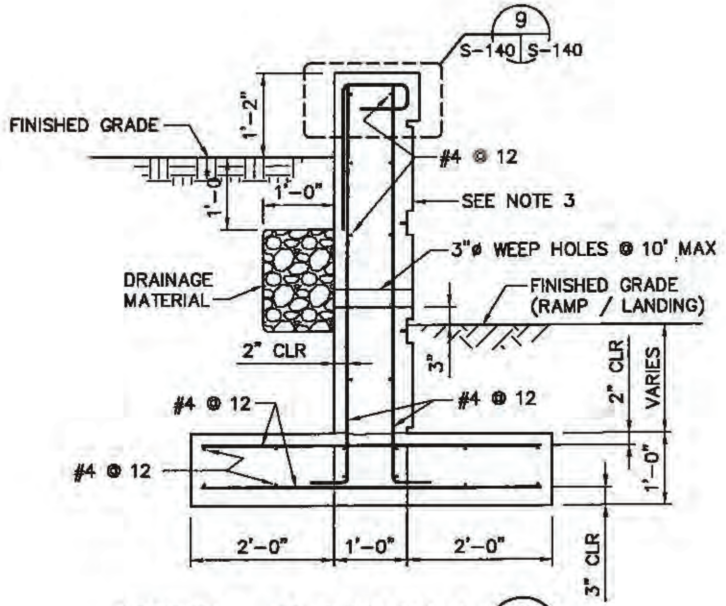
NAPA RIVER / NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST
(HATT BUILDING TO FIRST STREET)

VETERANS PARK
VP WALL NO. 1 DETAILS

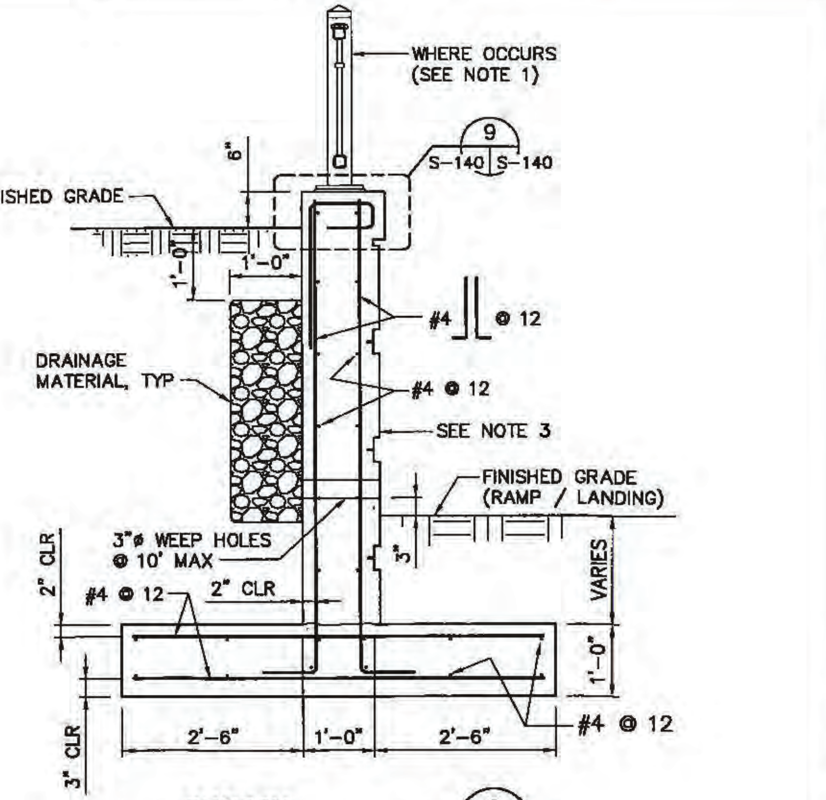
Sheet reference number:
S-136
Sheet 108 of 156



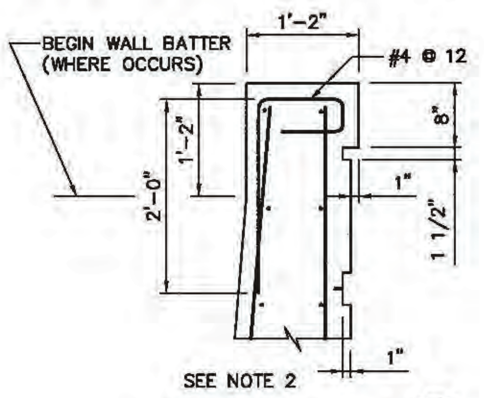
SECTION – VP WALL NO. 1 (J)
SCALE: 3/4" = 1'-0" S-136 | S-140



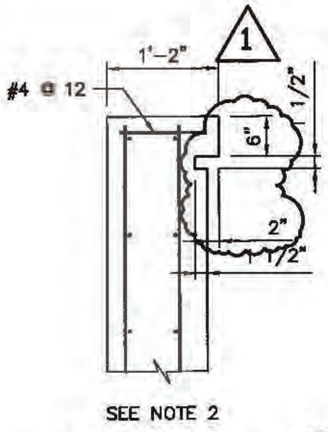
SECTION – VP WALL NO. 1 (K)
SCALE: 3/4" = 1'-0" S-136 | S-140



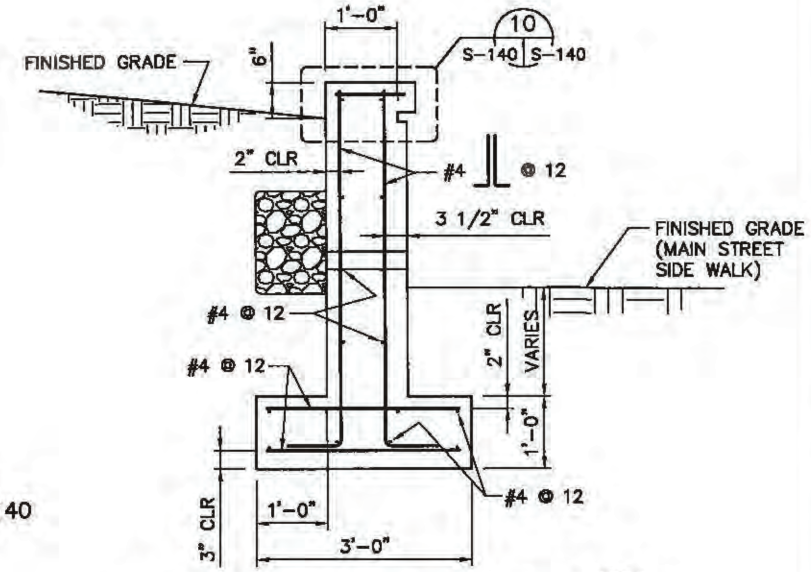
SECTION (O)
SCALE: 3/4" = 1'-0" S-137 | S-140
S-143



DETAIL (9)
SCALE: 1" = 1'-0" S-140 | S-140
S-143

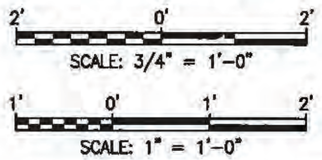


DETAIL (10)
SCALE: 1" = 1'-0" S-140 | S-140



SECTION – VP WALL NO. 2 & 3 (P)
SCALE: 3/4" = 1'-0" S-138 | S-140
S-139

GRAPHIC SCALE



REVISED AS-BUILT

NOTES:
1. SEE L-515 FOR RAILING LAYOUT & DETAILS.
2. DETAIL 9 APPLIES TO VP WALL NO. 1 – 3 EXCEPT FOR WALL FACING MAIN STREET WHERE DETAIL 10 APPLIES.
3. SAME TEXTURE DETAILS AS WALLS 2-6 APPLY, SEE S-107.

US Army Corps
of Engineers
Sacramento District

Symbol	Description	Date	Approved
1	CHANGES PER RFI 73 – VP WALL CAP DIMENSION DISCREPANCY AN	[illegible]	

Designed by: D. AN		Date: 04/27/05
Dwn by: X. SUN	Spec No.: 1407	Design file no: NA-25-030
Reviewed by: R. SENNETT		Drawing Code:
Submitted by: /s/ R. SENNETT		File name: Plot date: Plot scale: 1:1

Rev.

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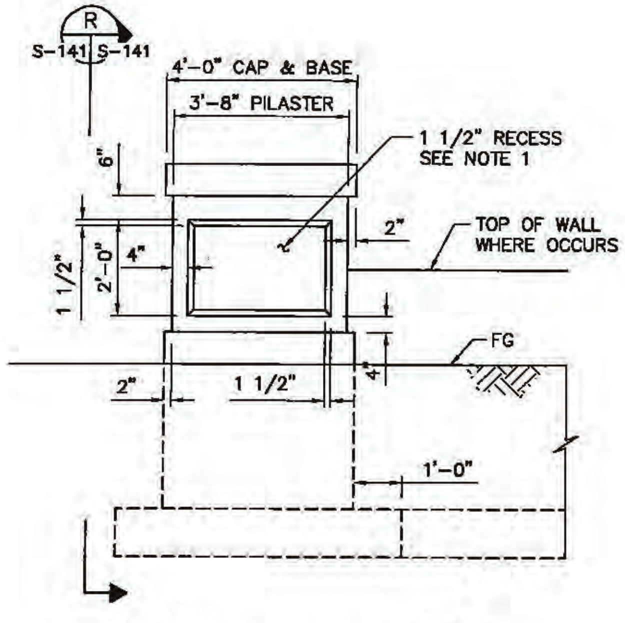
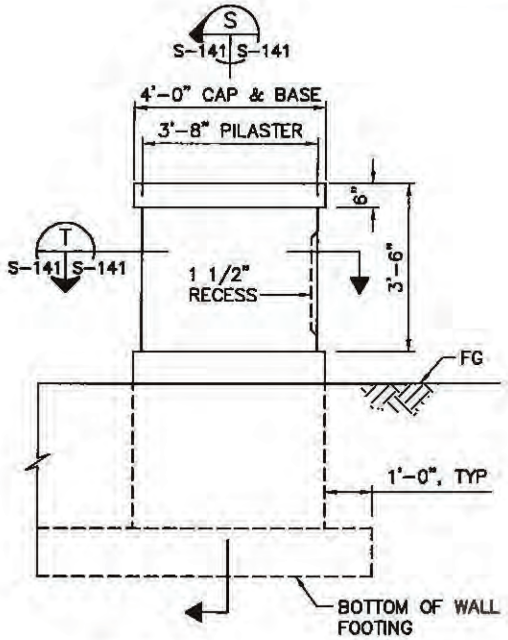
NAPA CALIFORNIA
NAPA RIVER / NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST
(HATT BUILDING TO FIRST STREET)

VETERANS PARK
VP WALLS SECTIONS AND DETAILS

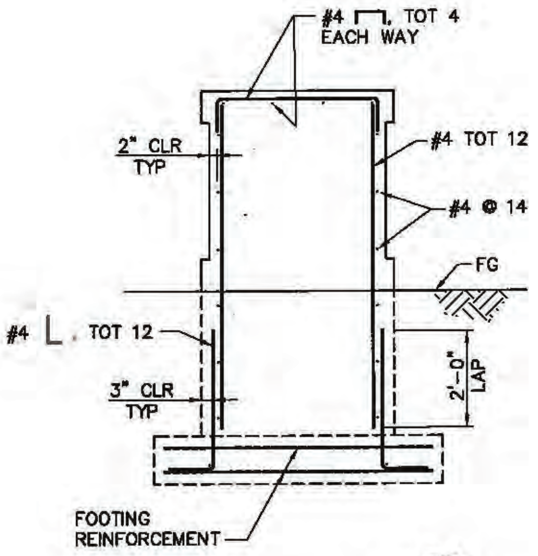
Sheet reference number:
S-140
Sheet 112 of 166



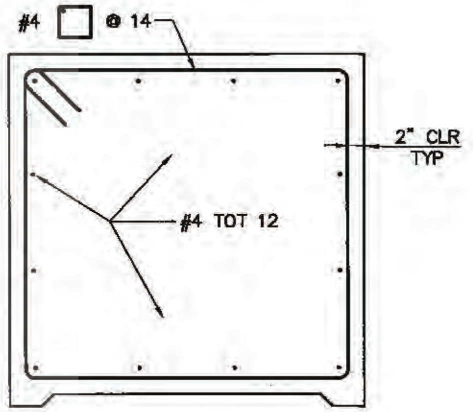
DETAIL 8
SCALE: 1/2" = 1'-0" S-138 S-141
S-139



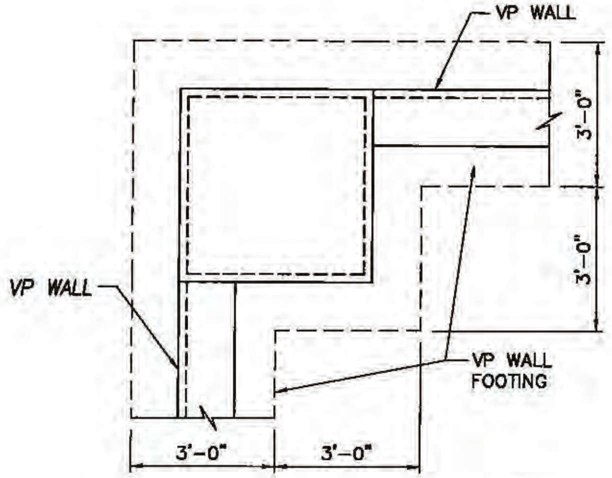
SECTION R
SCALE: 1/2" = 1'-0" S-141 S-141



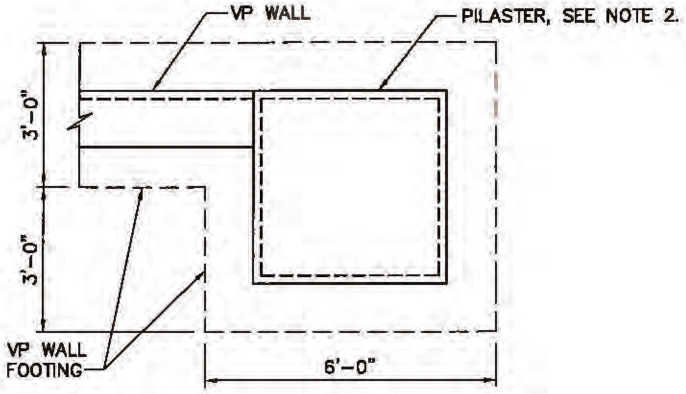
SECTION S
SCALE: 1/2" = 1'-0" S-141 S-141



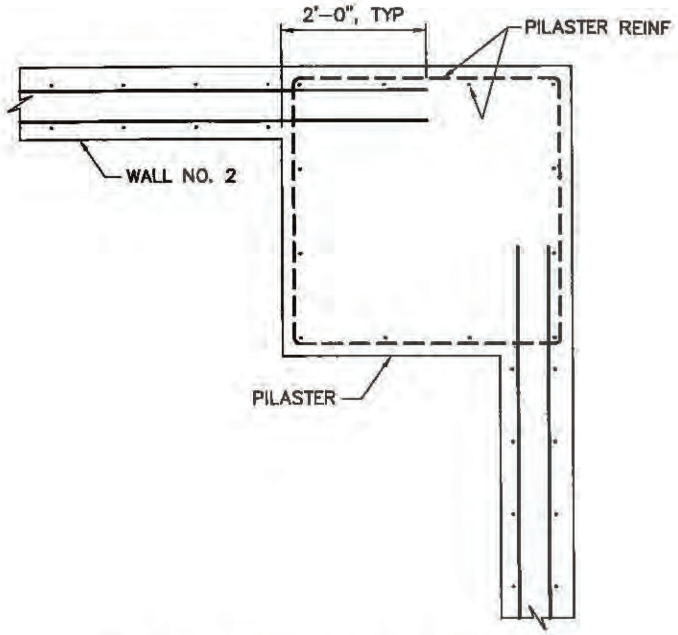
SECTION T
SCALE: 1" = 1'-0" S-141 S-141



DETAIL – VP WALL CORNER 3
SCALE: 1/2" = 1'-0" S-135 S-141

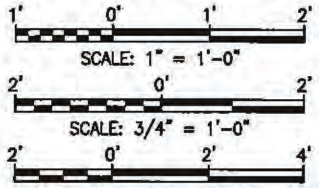


DETAIL – END OF VP WALL 4
SCALE: 1/2" = 1'-0" S-135 S-141



VP WALL CORNER REINF DETAIL
SCALE: 3/4" = 1'-0"

GRAPHIC SCALE



AS-BUILT

NOTES:
1. RECESS TO FACE MAIN STREET AND TO FACE NAPA RIVER AS INDICATED ON ARCHITECTURAL PLANS.
2. FOR LOCATION OF PILASTERS, SEE S-135. FOR ANCHOR DETAILS OF LIGHT POST AND FLAG POLE WHERE OCCURS, SEE SHEETS L-509, C-544 AND ELECTRICAL PLANS.

US Army Corps
of Engineers
Sacramento District

Symbol | Description | Date | Approved

Designed by: D. AN
Dwn by: X. SUN
Spec No.: 1407
Reviewed by: R. SENNETT
Submitted by: /s/ R. SENNETT
Date: 04/27/05
Design file no: NA-25-030
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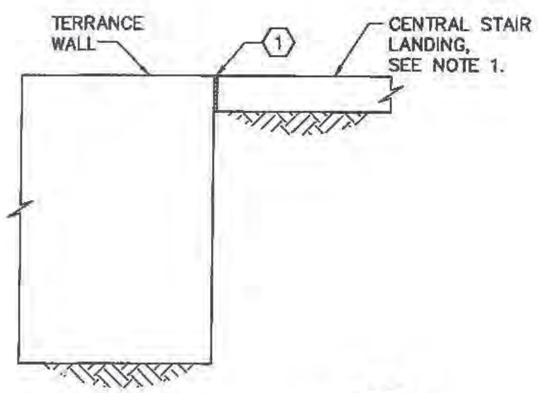
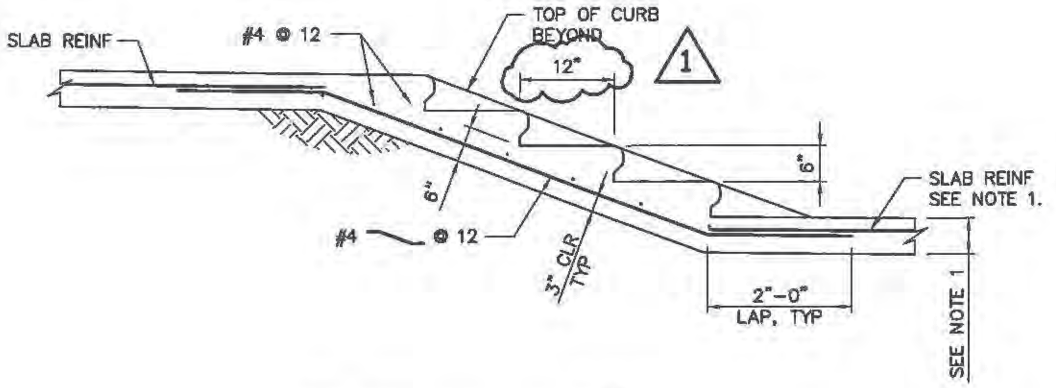
NAPA CALIFORNIA
NAPA RIVER / NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST
(HATT BUILDING TO FIRST STREET)
VETERANS PARK
VP WALL MISCELLANEOUS DETAILS NO. 1

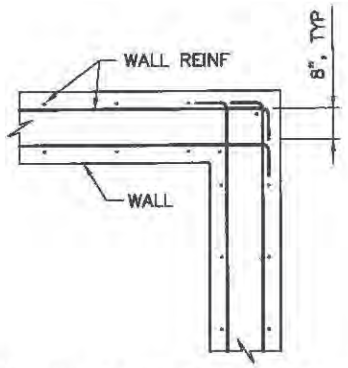
Sheet
reference
number:
S-141
Sheet 113 of 166



SECTION D
SCALE: 3/4" = 1'-0" S-135 | S-142

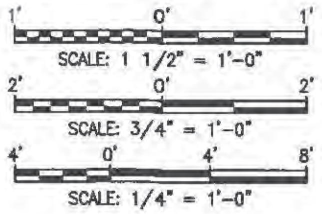


SECTION E
SCALE: 3/4" = 1'-0" S-135 | S-142



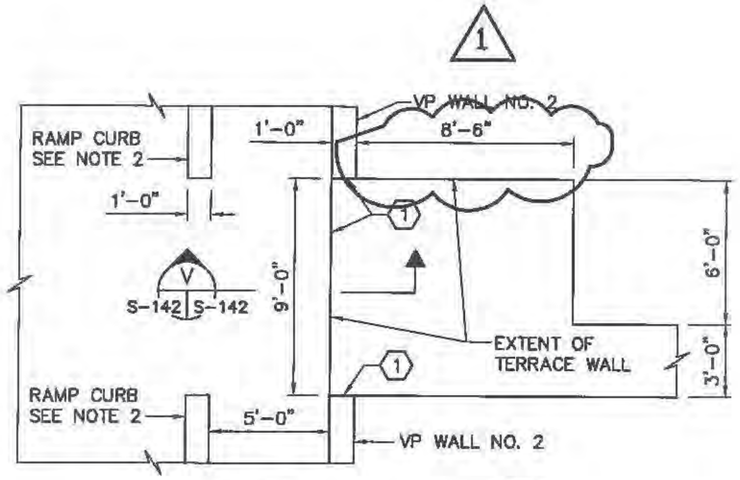
WALL CORNER REINF DETAIL – TYPICAL
SCALE: 3/4" = 1'-0"

GRAPHIC SCALE

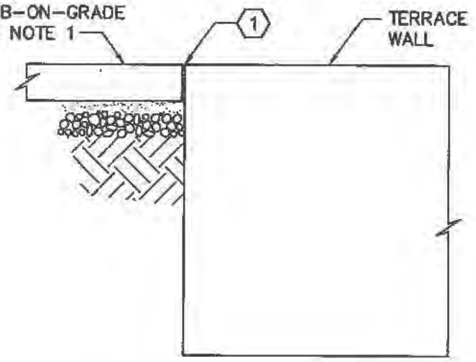


LEGEND

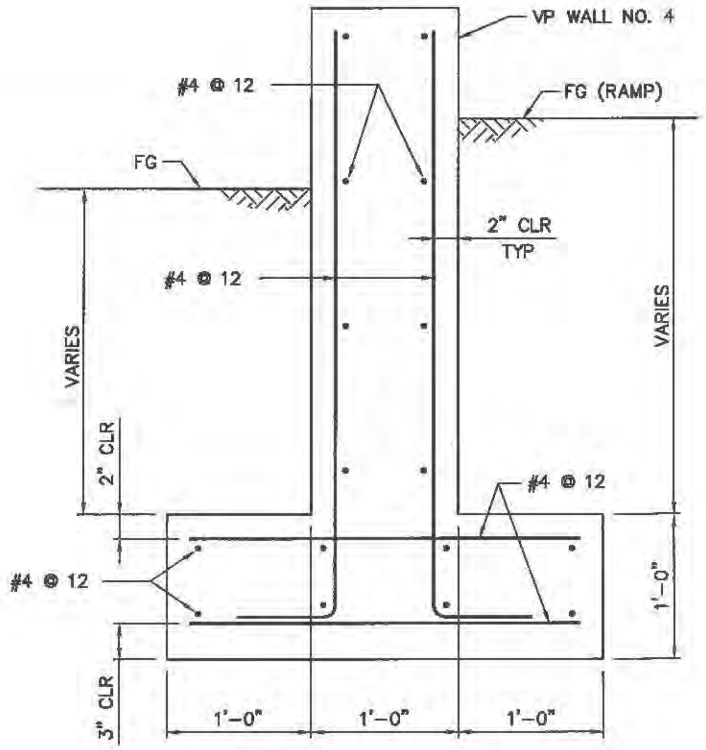
① – TYPICAL JOINT DETAIL, SEE S-137



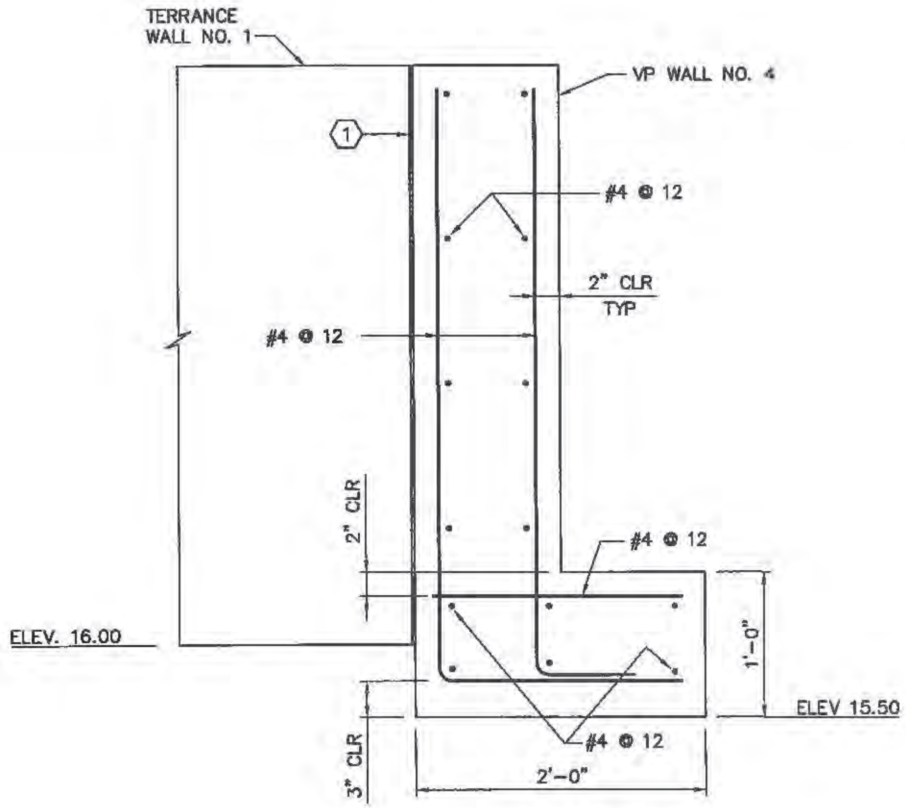
DETAIL 2
SCALE: 1/4" = 1'-0" S-135 | S-142



SECTION V
SCALE: 3/4" = 1'-0" S-142 | S-142



SECTION GG
SCALE: 1 1/2" = 1'-0" S-139 | S-142



SECTION HH
SCALE: 1 1/2" = 1'-0" S-139 | S-142

REVISED AS-BUILT

NOTES:
1. FOR DETAILS OF SLAB-ON-GRADE, SEE SHEET G-008.
2. FOR CURB DETAILS, SEE SHEET C-543.

US Army Corps
of Engineers
Sacramento District

Symbol	Description		Date	Approved
1	CHANGES FOR RFI 68 – STEP LIGHTS @ VP STAIRWAY	AN	[illegible]	

Designed by: D. AN		Date: 04/27/05
Dwn by: X. SUN	Spec No.: 1407	Design file no: NA-25-030
Reviewed by: R. SENNETT		Drawing Code:
Submitted by: /s/ R.SENNETT		File name: Plot date: Plot scale: 1:1

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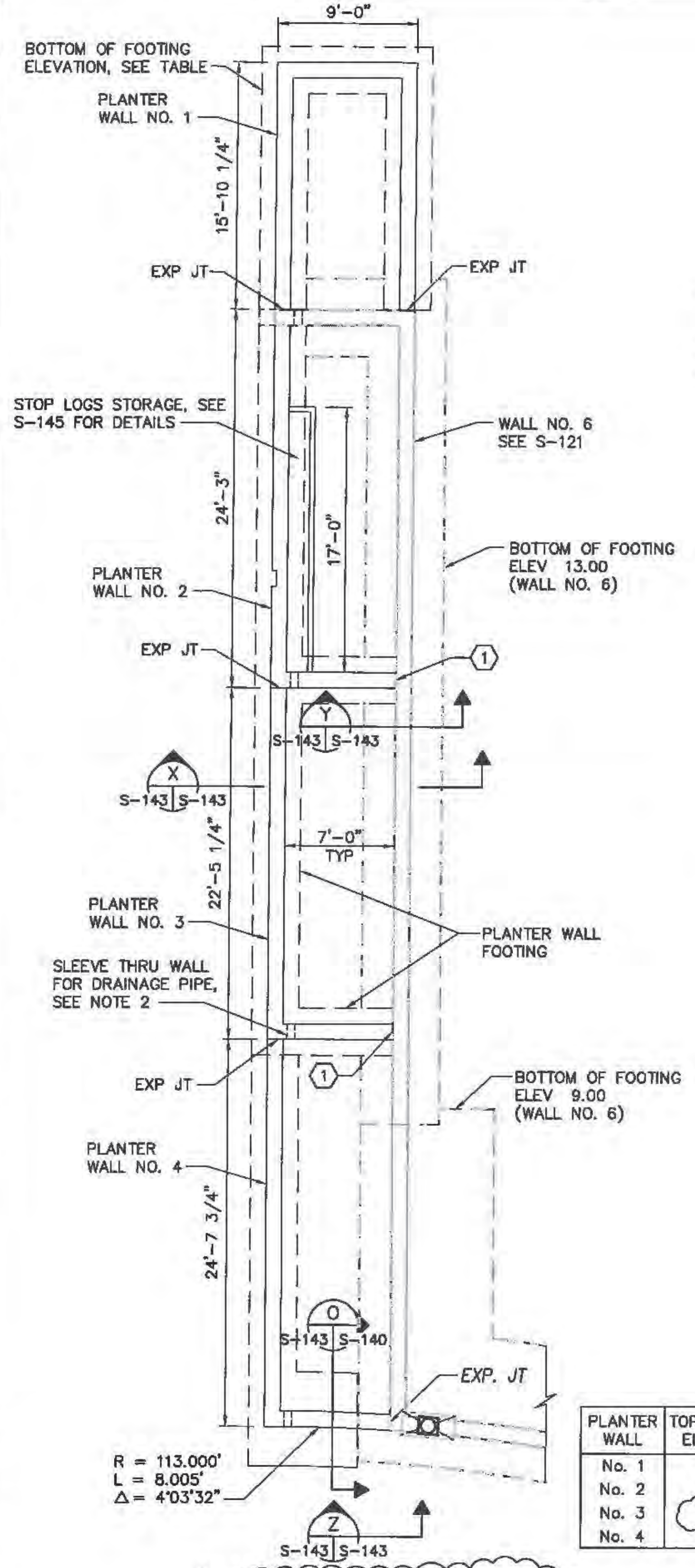
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST
(HATT BUILDING TO FIRST STREET)
VETERANS PARK
VP WALL MISCELLANEOUS DETAILS NO. 2

Sheet reference number:
S-142

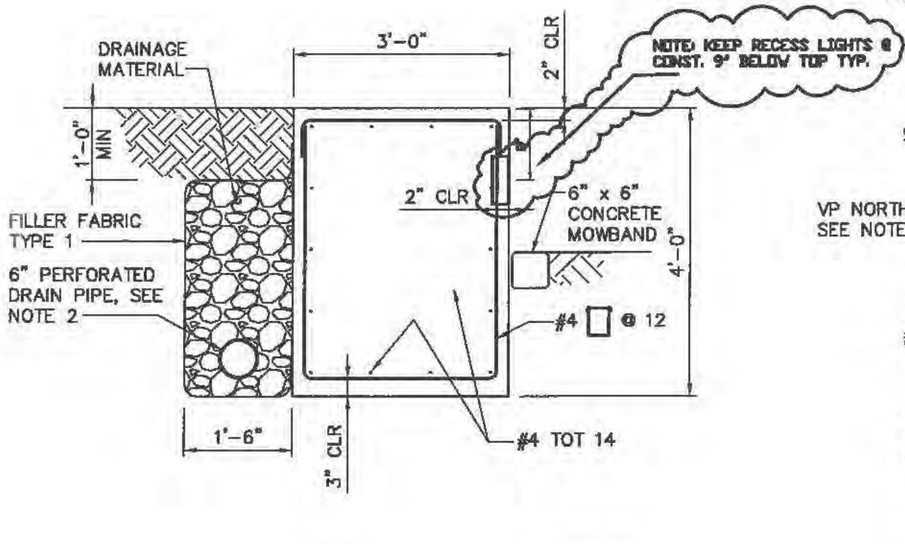
Sheet 114 of 166



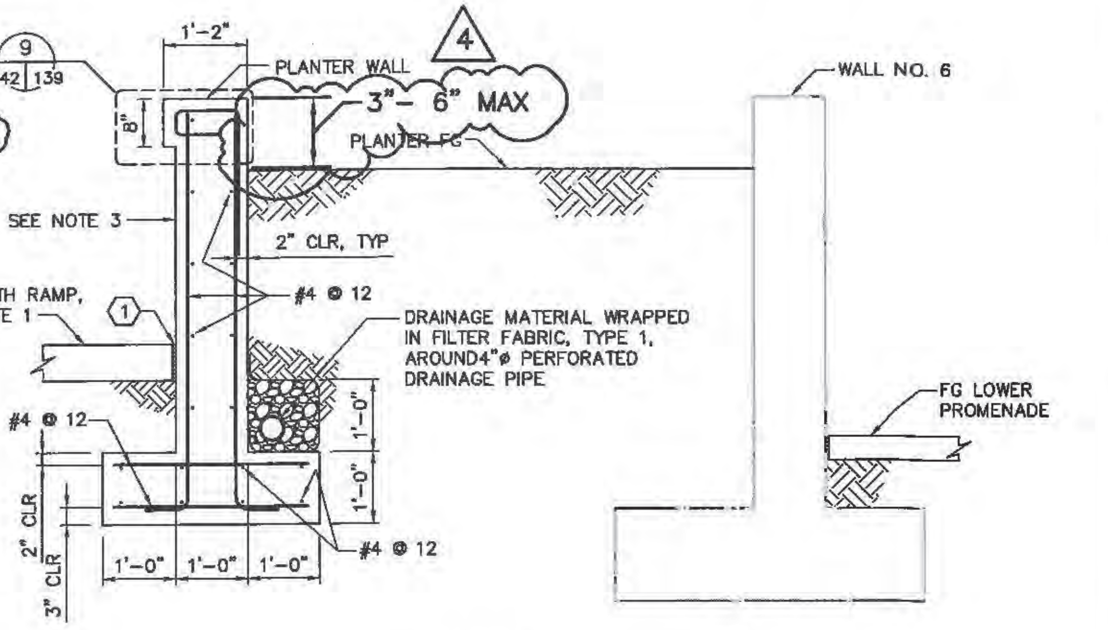
DETAIL 5 S-135 | S-143
SCALE: 3/16" = 1'-0"

PLANTER WALL	TOP OF WALL ELEVATION	BOTTOM OF FTG ELEV
No. 1	19.33	14.50
No. 2	20.00	14.50
No. 3	19.33	14.50
No. 4	18.59	9.50

1



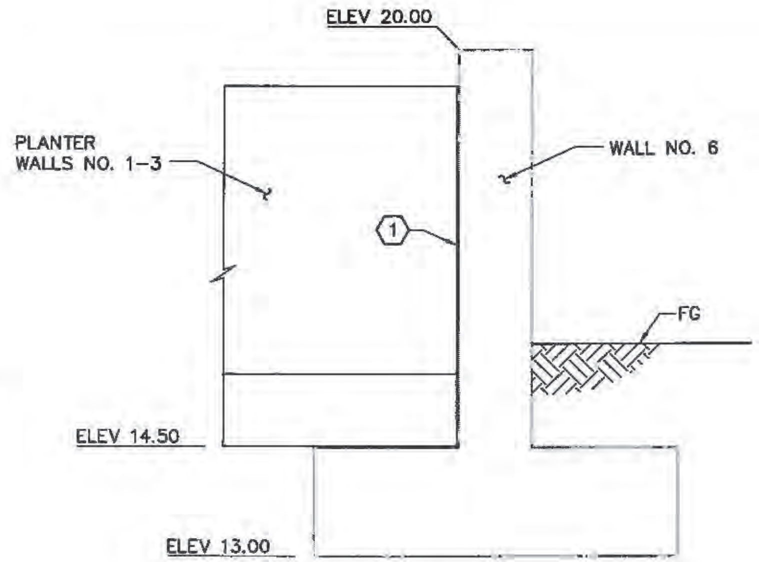
SECTION A S-135 | S-143
SCALE: 3/4" = 1'-0"



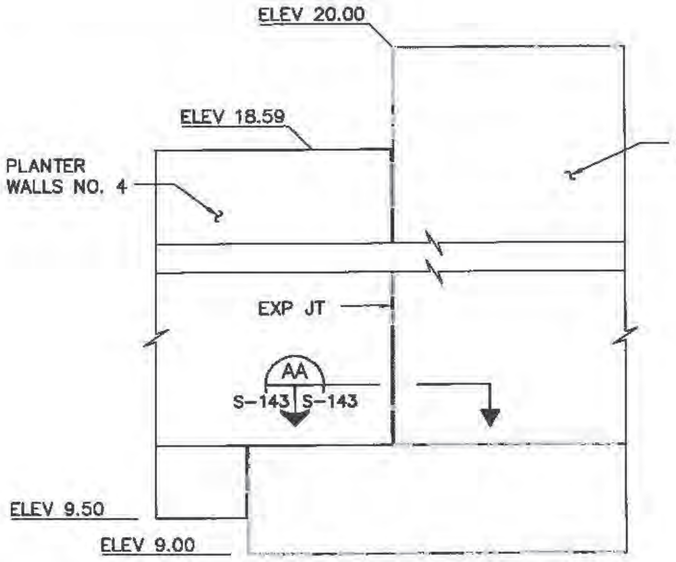
SECTION X S-143 | S-143
SCALE: 3/4" = 1'-0"

LEGEND

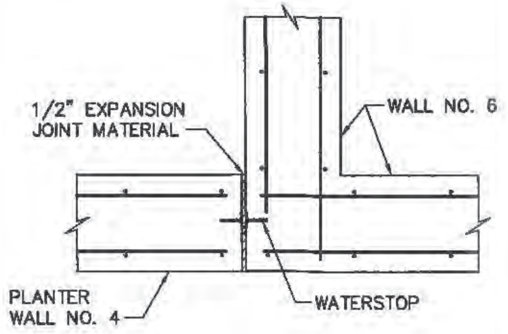
1 – TYPICAL JOINT DETAIL, SEE S-137



SECTION Y S-143 | S-143
SCALE: 3/4" = 1'-0"

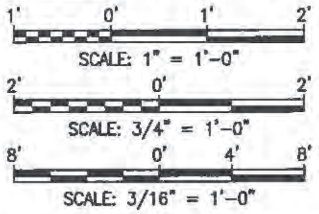


VIEW Z S-143 | S-143
SCALE: 3/4" = 1'-0"



SECTION AA S-143 | S-143
SCALE: 1" = 1'-0"

GRAPHIC SCALE



AS-BUILT

NOTES:
1. FOR DETAILS OF SLAB-ON-GRADE, SEE SHEET G-007.
2. FOR DRAINAGE LAYOUT & DETAILS, SEE SHEET C-106
3. SAME TEXTURE DETAILS AS WALLS 2-6 APPLY, SEE SHEET S-107.

US Army Corps
of Engineers
Sacramento District

Symbol	Description	Date	Approved
4	CLARIFICATION PER RFI 70 – TOPSOIL IN PLANTER BOXES	[illegible]	
3	CLARIFICATION PER RFI 19 – STEP LIGHT ELEVATIONS	[illegible]	
2	ADDED DIMENSIONS	06/22/05	RES
1	REVISED ELEVATION	06/22/05	RES

Designed by: D. AN		Date: 04/27/05	Rev.
Dwn by: X. SUN	Spec No.: 1407	Design file no: NA-25-030	
Reviewed by: R. SENNETT		Drawing Code:	
Submitted by: /s/ R. SENNETT		File name: Plot date: Plot scale: 1:1	

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

MGE ENGINEERING INC.
7415 Greenhaven Dr. Suite 100
Sacramento, California 95831
916.421.1000

NAPA CALIFORNIA
NAPA RIVER / NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST
(HATT BUILDING TO FIRST STREET)
VETERANS PARK
MISCELLANEOUS DETAILS NO. 1

Sheet reference number:
S-143
Sheet 115 of 166
AM-003

Appendix B

Flood Damage Reduction Segment/System Inspection Report

&

Inspection Map

US Army Corps
of Engineers®

Flood Damage Reduction Segment / System Inspection Report

Name of Segment / System: Napa River, Hatt to 1st Street floodwall

Public Sponsor(s): Napa County Flood Control and Water Conservation District

Public Sponsor Representative: Jeremy Sarrow

Sponsor Phone: 707-259-8204

Sponsor Email: jeremy.sarrow@countyofnapa.org

Corps of Engineers Inspector: Micheal Franssen PE and Nathan DeLannoy

Inspection Start Date: 07/22/2020

Inspection End Date: 07/22/2020

Inspection Report Prepared By: Nathan DeLannoy

Date Report Prepared: 08/05/2020

Internal Technical Review (for Periodic Inspections) By:

Date of ITR:

Final Approved By: Marcus Palmer, PE, Levee Safety Officer

Date Approved:

Type of Inspection:
☐ **Initial Eligibility Inspection**
☒ **Continuing Eligibility Inspection (Routine)**
☐ **Continuing Eligibility Inspection (Periodic)**

Overall Segment / System Rating:
☒ **Acceptable**
☐ **Minimally Acceptable**
☐ **Unacceptable**

Contents of Report:
☒ **Instructions**
☐ **Initial Eligibility Inspection**
☒ **General Items for All Flood Control Works**
☐ **Levee Embankment**
☒ **Concrete Floodwalls**
☐ **Sheet Pile and Concrete I-walls**
☒ **Interior Drainage System**
☐ **Pump Stations**
☐ **FDR System Channels**

Note: In addition to the report contents indicated here, a plan view drawing of the system, with stationing, should be included with this report to reference locations of items rated less than acceptable. Photos of general system condition and any noted deficiencies should also be attached.

Note: This inspection rating represents the Corps evaluation of operations and maintenance of the flood damage reduction system and may be used in conjunction with other information for a levee certification determination for National Flood Insurance Program (NFIP) purposes if applicable. An Acceptable Corps inspection rating, alone, does not equate to a certifiable levee for the NFIP. It is recommended for levee systems currently accredited by the Federal Emergency Management Agency (FEMA) for NFIP purposes receiving a Corps Minimally Acceptable or Unacceptable rating, be evaluated by the levee owner to determine the potential impacts to the certification for FEMA.

US Army Corps
of Engineers®

Flood Damage Reduction Segment / System
Public Sponsor Pre-Inspection Form

The following information is to be provided by the levee district sponsor prior to an inspection. This information will be used to help evaluate the organizational capability of the levee district to manage the levee segment / system maintenance program.

1. Levee segment / system and district: (name of the segment / system and levee district)

Napa River, Hatt to 1st Street floodwall for CESPN

2. Reporting period: (month/day/year to month/day/year)

3. Summary of maintenance required by last inspection report:

None

4. Summary of maintenance performed this reporting period:

Excerising Flap Gate

5. Summary of maintenance planned next reporting period:

Excerising Flap Gate

6. Summary of changes to segment / system since last inspection:

None

7. Problems/ issues requiring the assistance of the US Army Corps of Engineers:

None

Public Sponsor Pre-Inspection Report

The following information is to be provided by the levee district sponsor prior to an inspection

8. Levee district organization: (elected or appointed levee district officials and key employees)

Name	Position	Mailing Address	Phone Number	Email Address
Jeremy Sorrow	Resources Specialist	804 First Street, Napa, CA 94559	707-259-8204	jeremy.sarrow@countyofnapa.org

General Instructions for the Inspection of Flood Damage Reduction Segments / Systems

A. Purpose of USACE Inspections:

The primary purpose of these inspections is to prevent loss of life and catastrophic damages; preserve the value of Federal investments, and to encourage non-Federal sponsors to bear responsibility for their own protection. Inspections should assure that Flood Damage Reduction structures and facilities are continually maintained and operated as necessary to obtain the maximum benefits. Inspections are also conducted to determine eligibility for Rehabilitation Assistance under authority of PL 84-99 for Federal and non-Federal systems. (ER 1130-2-530, ER 500-1-1)

B. Types of Inspections:

The Corps conducts several types of inspections of Flood Damage Reduction systems, as outlined below:

Initial Eligibility Inspections	**Continuing Eligibility Inspections**	
	Routine Inspections	**Periodic Inspections**
IEIs are conducted to determine whether a non-Federally constructed Flood Damage Reduction system meets the minimum criteria and standards set forth by the Corps for initial inclusion into the Rehabilitation and Inspection Program.	RIs are intended to verify proper maintenance, owner preparedness, and component operation.	PIs are intended to verify proper maintenance and component operation and to evaluate operational adequacy, structural stability, and safety of the system. Periodic Inspections evaluate the system's original design criteria vs. current design criteria to determine potential performance impacts, evaluate the current conditions, and compare the design loads and design analysis used against current design standards. This is to be done to identify components and features for the sponsor that need to be monitored more closely over time or corrected as needed. (Periodic Inspections are used as the basis of risk assessments.)

C. Inspection Boundaries:

Inspections should be conducted so as to rate each Flood Damage Reduction "Segment" of the system. The overall system rating will be the lowest segment rating in the system.

Project	System	Segment
A flood damage reduction project is made up of one or more flood damage reduction systems which were under the same authorization.	A flood damage reduction system is made up of one or more flood damage reduction segments which collectively provide flood damage reduction to a defined area. Failure of one segment within a system constitutes failure of the entire system. Failure of one system does not affect another system.	A flood damage reduction segment is defined as a discrete portion of a flood damage reduction system that is operated and maintained by a single entity. A flood damage reduction segment can be made up of one or more features (levee, floodwall, pump stations, etc).

D. Land Use Definitions:

The following three definitions are intended for use in determining minimum required inspection intervals and initial requirements for inclusion into the Rehabilitation and Inspection Program. Inspections should be considered for all systems that would result in significant environmental or economic impact upon failure regardless of specific land use.

Agricultural	Rural	Urban
Protected population in the range of zero to 5 households per square mile protected.	Protected population in the range of 6 to 20 households per square mile protected.	Greater than 20 households per square mile; major industrial areas with significant infrastructure investment. Some protected urban areas have no permanent population but may be industrial areas with high value infrastructure with no overnight population.

E. Use of the Inspection Report Template:

The report template is intended for use in all Army Corps of Engineers inspections of levee and floodwall systems and flood damage reduction channels. The section of the template labeled “Initial Eligibility" only needs to be completed during Initial Eligibility Inspections of Non-Federally constructed Flood Damage Reduction Systems. The section labeled "General Items" needs to be completed with every inspection, along with all other sections that correspond to features in the system. The section labeled "Public Sponsor Pre-Inspection Report" is intended for completion before the inspection, if possible.

F. Individual Item / Component Ratings:

Assessment of individual components rated during the inspection should be based on the criteria provided in the inspection report template, though inspectors may incorporate additional items into the report based on the characteristics of the system. The assessment of individual components should be based on the following definitions.

Acceptable Item	Minimally Acceptable Item	Unacceptable Item
The inspected item is in satisfactory condition, with no deficiencies, and will function as intended during the next flood event.	The inspected item has one or more minor deficiencies that need to be corrected. The minor deficiency or deficiencies will not seriously impair the functioning of the item as intended during the next flood event.	The inspected item has one or more serious deficiencies that need to be corrected. The serious deficiency or deficiencies will seriously impair the functioning of the item as intended during the next flood event.

G. Overall Segment / System Ratings:

Determination of the overall system rating is based on the definitions below. Note that an Unacceptable System Rating may be either based on an engineering determination that concluded that noted deficiencies would prevent the system from functioning as intended during the next flood event, or based on the sponsor's demonstrated lack of commitment or inability to correct serious deficiencies in a timely manner.

Acceptable System	Minimally Acceptable System	Unacceptable System
All items or components are rated as Acceptable.	One or more items are rated as Minimally Acceptable or one or more items are rated as Unacceptable and an engineering determination concludes that the Unacceptable items would not prevent the segment / system from performing as intended during the next flood event.	One or more items are rated as Unacceptable and would prevent the segment / system from performing as intended, or a serious deficiency noted in past inspections (which had previously resulted in a minimally acceptable system rating) has not been corrected within the established timeframe, not to exceed two years.

H. Eligibility for PL84-99 Rehabilitation Assistance:

Inspected systems that are not operated and maintained by the Federal government may be Active in the Corps' Rehabilitation and Inspection Program (RIP) and eligible for rehabilitation assistance from the Corps as defined below:

If the Overall System Rating is Acceptable	If the Overall System Rating is Minimally Acceptable	If the Overall System Rating is Unacceptable
The system is active in the RIP and eligible for PL84-99 rehabilitation assistance.	The system is Active in the RIP during the time that it takes to make needed corrections. Active systems are eligible for rehabilitation assistance. However, if the sponsor does not present USACE with proof that serious deficiencies (which had previously resulted in a minimally acceptable system rating) were corrected within the established timeframe, then the system will become Inactive in the RIP.	The system is Inactive in the RIP, and the status will remain Inactive until the sponsor presents USACE with proof that all items rated Unacceptable have been corrected. Inactive systems are ineligible for rehabilitation assistance.

I. Reporting:

After the inspection, the Corps is responsible for assembling an inspection report (or a summary report if it was a Periodic Inspection) including the following information:

a. All sections of the report template used during the inspection, including the cover and pre-inspection materials. (Supplemental data collected, and any sections of the template that weren't used during the inspection do not need to be included with the report.)

b. Photos of the general system condition and noted deficiencies.

c. A plan view drawing of the system, with stationing, to reference locations of items rated less than acceptable.

d. The relative importance of the identified maintenance issues should be specified in the transmittal letter.

e. If the Overall System Rating is Minimally Acceptable, the report needs to establish a timeframe for correction of serious deficiencies noted (not to exceed two years) and indicate that if these items are not corrected within the required timeframe, the system will be rated as Unacceptable and made Inactive in the Rehabilitation Inspection Program.

J. Notification:

Reports are to be disseminated as follows within 30 days of the inspection date.

If the Overall System Rating is Acceptable	If the Overall System Rating is Minimally Acceptable	If the Overall System Rating is Unacceptable
Reports need to be provided to the local sponsor and the county emergency management agency.	Reports need to be provided to the local sponsor, state emergency management agency, county emergency management agency, and to the FEMA region.	Reports need to be provided to the local sponsor, state emergency management agency, county emergency management agency, FEMA region, and to the Congressional delegation within 30 days of the inspection.

General Items for All Flood Damage Reduction Segments / Systems

For use during all inspections of all Flood Damage Reduction Segments / Systems

Rated Item	Rating	Rating Guidelines		Location/Remarks/Recommendations
1. Operations and Maintenance Manuals	**A**	**A**	Levee Owner's Manual, O&M Manuals, and/or manufacturer's operating instructions are present.	Our current Operations and Maintenance Manual is kept in sponsor's office along with a digit copy kept on their server.
		M	Sponsor manuals are lost or missing or out of date; however, sponsor will obtain manuals prior to next scheduled inspection.	
		U	Sponsor has not obtained lost or missing manuals identified during previous inspection.	
2. Emergency Supplies and Equipment (A or M only)	**A**	**A**	The sponsor maintains a stockpile of sandbags, shovels, and other flood fight supplies which will adequately supply all needs for the initial days of a flood fight. Sponsor determines required quantity of supplies after consulting with inspector.	The District's Emergency Supplies and Equipment are located at 933 Water St. Supplies consist of sand bags, shovels, sand for the sand bags, chain saws, flash lights, barriers, a grip hoist, and other various flood fighting supplies.
		M	The sponsor does not maintain an adequate supply of flood fighting materials as part of their preparedness activities.	
3. Flood Preparedness and Training (A or M only)	**A**	**A**	Sponsor has a written system-specific flood response plan and a solid understanding of how to operate, maintain, and staff the FDR system during a flood. Sponsor maintains a list of emergency contact information for appropriate personnel and other emergency response agencies.	Annual flood fighting training program conducted by the CA Department of Water Resources at the Napa Sheriff's Department each fall.
		M	The sponsor maintains a good working knowledge of flood response activities, but documentation of system-specific emergency procedures and emergency contact personnel is insufficient or out of date.	

Key: A = Acceptable. M = Minimally Acceptable; Maintenance is required. U = Unacceptable. N/A = Not Applicable. FDR = Flood Damage Reduction

Floodwalls

For use during Initial and Continuing Eligibility Inspections of all floodwalls

Rated Item	Rating	Rating Guidelines		Location/Remarks/Recommendations
1. Unwanted Vegetation Growth[1]	**A**	**A**	A grass-only or paved zone is maintained on both sides of the floodwall, free of all trees, brush, and undesirable weeds. The vegetation-free zone extends 15 feet from both the land and riverside of the floodwall, at ground-level, to the centerline of the tree. Additionally, an 8-foot root-free zone is maintained around the entire structure, including the floodwall toe, heel, and any toe-drains. If the floodwall access easement doesn't extend to the described limits, then the vegetation-free zone must be maintained to the easement limits. Reference EM 1110-2-301 and/or Corps policy for regional vegetation variance.	NRN1_2020_a_0001: Station_1 NA: Upstream tie-in.: No action required at this time. (A) NRN1_2020_a_0008: Station_1 NA: Station_2 NA: Revetment: Monitor. (A)
		M	Minimal vegetation growth (brush, weeds, or trees 2 inches in diameter or smaller) is present within the zones described above. This vegetation must be removed but does not currently threaten the operation or integrity of the floodwall.	
		U	Significant vegetation growth (brush, weeds, or any trees greater than 2 inches in diameter) is present within the zones described above. This vegetation threatens the operation or integrity of the floodwall and must be removed.	
2. Encroachments	**A**	**A**	No trash, debris, unauthorized structures, excavations, or other obstructions present within the easement area. Encroachments have been previously reviewed by the Corps, and it was determined that they do not diminish proper functioning of the floodwall.	NRN1_2020_a_0005: Station_1 NA: Station_2 NA: landscaping anchors.: Monitor. (A) NRN1_2020_a_0006: Station_1 NA: City dock access.: No action required at this time. (A)
		M	Trash, debris, unauthorized structures, excavations, or other obstructions present, or inappropriate activities noted that should be corrected but will not inhibit operations and maintenance or emergency operations. Encroachments have not been reviewed by the Corps.	
		U	Unauthorized encroachments or inappropriate activities noted are likely to inhibit operations and maintenance, emergency operations, or negatively impact the integrity of the floodwall.	
3. Closure Structures (Stop Log Closures and Gates) (A or U only)	**A**	**A**	Closure structure in good repair. Placing equipment, stoplogs, and other materials are readily available at all times. Components are clearly marked and installation instructions/ procedures readily available. Trial erections have been accomplished in accordance with the O&M Manual.	NRN1_2020_a_0003: Station_1 NA: Log closure area.: No action required at this time. (A)
		U	Any of the following issues is cause for this rating: Closure structure in poor condition. Parts missing or corroded. Placing equipment may not be available within the anticipated warning time. The storage vaults cannot be opened during the time of inspection. Components of closure are not clearly marked and installation instructions/ procedures are not readily available. Trial erections have not been accomplished in accordance with the O&M Manual.	
		N/A	There are no closure structures along this component of the FDR segment / system.	
4. Concrete Surfaces	**A**	**A**	Negligible spalling, scaling or cracking. If the concrete surface is weathered or holds moisture, it is still satisfactory but should be seal coated to prevent freeze/ thaw damage.	NRN1_2020_a_0007: Station_1 NA: Spalling was observed on concrete floor. Minor spall has no bearing on the integrity of the floodwall.: No action required at this time. (A)
		M	Spalling, scaling, and open cracking present, but the immediate integrity or performance of the structure is not threatened. Reinforcing steel may be exposed. Repairs/ sealing is necessary to prevent additional damage during periods of thawing and freezing.	

Key: A = Acceptable. M = Minimally Acceptable; Maintenance is required. U = Unacceptable. N/A = Not Applicable. FDR = Flood Damage Reduction

Floodwalls

For use during Initial and Continuing Eligibility Inspections of all floodwalls

Rated Item	Rating	Rating Guidelines		Location/Remarks/Recommendations
		U	Surface deterioration or deep cracks present that may result in an unreliable structure. Any surface deterioration that exposes the sheet piling or lies adjacent to monolith joints may indicate underlying reinforcement corrosion and is unacceptable.	
5. Tilting, Sliding or Settlement of Concrete Structures[2]	**A**	**A**	There are no significant areas of tilting, sliding, or settlement that would endanger the integrity of the structure.	No tilting, sliding or settlement of concrete floodwall was observed during PI.
		M	There are areas of tilting, sliding, or settlement (either active or inactive) that need to be repaired. The maximum offset, either laterally or vertically, does not exceed 2 inches unless the movement can be shown to be no longer actively occurring. The integrity of the structure is not in danger.	
		U	There are areas of tilting, sliding, or settlement (either active or inactive) that threaten the structure's integrity and performance. Any movement that has resulted in failure of the waterstop (possibly identified by daylight visible through the joint) is unacceptable. Differential movement of greater than 2 inches between any two adjacent monoliths, either laterally or vertically, is unacceptable unless it can be shown that the movement is no longer active. Also, if the floodwall is of I-wall construction, then any visible or measurable tilting of the wall toward the protected side that has created an open horizontal crack on the riverside base of a monolith is unacceptable.	
6. Foundation of Concrete Structures[1]	**A**	**A**	No active erosion, scouring, or bank caving that might endanger the structure's stability.	No foundation concerns were observed during PI.
		M	There are areas where the ground is eroding towards the base of the structure. Efforts need to be taken to slow and repair this erosion, but it is not judged to be close enough to the structure or to be progressing rapidly enough to affect structural stability before the next inspection. For the purposes of inspection, the erosion or scour is not closer to the riverside face of the wall than twice the floodwall's underground base width if the wall is of L-wall or T-wall construction; or if the wall is of sheetpile or I-wall construction, the erosion is not closer than twice the wall's visible height. Additionally, rate of erosion is such that the wall is expected to remain stabile until the next inspection.	
		U	Erosion or bank caving observed that is closer to the wall than the limits described above, or is outside these limits but may lead to structural instabilities before the next inspection. Additionally, if the floodwall is of I-wall or sheetpile construction, the foundation is unacceptable if any turf, soil or pavement material got washed away from the landside of the I-wall as the result of a previous overtopping event.	
7. Monolith Joints	**A**	**A**	The joint material is in good condition. The exterior joint sealant is intact and cracking/ desiccation is minimal. Joint filler material and/or waterstop is not visible at any point.	Expansion and construction joints were in good condition.
		M	The joint material has appreciable deterioration to the point where joint filler material and/or waterstop is visible in some locations. This needs to be repaired or replaced to prevent spalling and cracking during freeze/ thaw cycles, and to ensure water tightness of the joint.	

Key: A = Acceptable. M = Minimally Acceptable; Maintenance is required. U = Unacceptable. N/A = Not Applicable. FDR = Flood Damage Reduction

Floodwalls

For use during Initial and Continuing Eligibility Inspections of all floodwalls

Rated Item	Rating	Rating Guidelines		Location/Remarks/Recommendations
		U	The joint material is severely deteriorated or the concrete adjacent to the monolith joints has spalled and cracked, damaging the waterstop; in either case damage has occurred to the point where it is apparent that the joint is no longer watertight and will not provide the intended level of protection during a flood.	
		N/A	There are no monolith joints in the floodwall.	
8. Underseepage Relief Wells/ Toe Drainage Systems	**A**	**A**	Toe drainage systems and pressure relief wells necessary for maintaining FDR segment / system stability during high water functioned properly during the last flood event and no sediment is observed in horizontal system (if applicable). Nothing is observed which would indicate that the drainage systems won't function properly during the next flood, and maintenance records indicate regular cleaning. Wells have been pumped tested within the past 5 years and documentation is provided.	The drain system was in good condition with no signs of corrosion, deterioration or any blockages to prevent water from landside floodwall to Napa River.
		M	Toe drainage systems or pressure relief wells are damaged and may become clogged if they are not repaired. Maintenance records are incomplete or indicate irregular cleaning and pump testing.	
		U	Toe drainage systems or pressure relief wells necessary for maintaining FDR segment / system stability during flood events have fallen into disrepair or have become clogged. No maintenance records. No documentation of the required pump testing.	
		N/A	There are no relief wells/ toe drainage systems along this component of the FDR segment / system.	
9. Seepage	**A**	**A**	No evidence or history of unrepaired seepage, saturated areas, or boils.	No seepage concerns were observed douring PI
		M	Evidence or history of minor unrepaired seepage or small saturated areas at or beyond the landside toe but not on the landward slope of levee. No evidence of soil transport.	
		U	Evidence or history of active seepage, extensive saturated areas, or boils.	

[1] Inspectors must have as-built drawings available during the inspection so that the lateral distance to the heel and toe of the floodwalls can be determined in the field.
[2] The sponsor should be monitoring any observed movement to verify whether the movement is active or inactive.

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Inspect ID: NRN1_2020_a_0001 **Title:** USACE_CESPN_NRN1_2020_a_0001_1.jpg
Rated Item: 1. Unwanted Vegetation Growth **Caption:** Rating: Acceptable; Remarks: Upstream tie-in.; Action: No action required at this time.

Inspect ID: NRN1_2020_a_0008 **Title:** USACE_CESPN_NRN1_2020_a_0008_1.jpg
Rated Item: 1. Unwanted Vegetation Growth **Caption:** Rating: Acceptable; Remarks: Revetment; Action: Monitor.

Inspect ID: NRN1_2020_a_0008 **Title:** USACE_CESPN_NRN1_2020_a_0008_2.jpg **Rated Item:** 1. Unwanted Vegetation Growth **Caption:** Rating: Acceptable; Remarks: Revetment; Action: Monitor.

Inspect ID: NRN1_2020_a_0005 **Title:** USACE_CESPN_NRN1_2020_a_0005_1.jpg **Rated Item:** 2. Encroachments **Caption:** Rating: Acceptable; Remarks: landscaping anchors.; Action: Monitor.

Floodwalls

For use during Initial and Continuing Eligibility Inspections of all floodwalls

Inspect ID: NRN1_2020_a_0006 **Title:** USACE_CESPN_NRN1_2020_a_0006_1.jpg **Rated Item:** 2. Encroachments **Caption:** Rating: Acceptable; Remarks: City dock access.; Action: No action required at this time.

Inspect ID: NRN1_2020_a_0003 **Title:** USACE_CESPN_NRN1_2020_a_0003_1.jpg **Rated Item:** 3. Closure Structures (Stop Log Closures and Gates) (A or U only) **Caption:** Rating: Acceptable; Remarks: Log closure area.; Action: No action required at this time.

Floodwalls

For use during Initial and Continuing Eligibility Inspections of all floodwalls

	**Inspect ID:** NRN1_2020_a_0003 **Title:** USACE_CESPN_NRN1_2020_a_0003_2.jpg **Rated Item:** 3. Closure Structures (Stop Log Closures and Gates) (A or U only) **Caption:** Rating: Acceptable; Remarks: Log closure area.; Action: No action required at this time.
	**Inspect ID:** NRN1_2020_a_0003 **Title:** USACE_CESPN_NRN1_2020_a_0003_3.jpg **Rated Item:** 3. Closure Structures (Stop Log Closures and Gates) (A or U only) **Caption:** Rating: Acceptable; Remarks: Log closure area.; Action: No action required at this time.

Floodwalls

For use during Initial and Continuing Eligibility Inspections of all floodwalls

	**Inspect ID:** NRN1_2020_a_0007 **Title:** USACE_CESPN_NRN1_2020_a_0007_1.jpg **Rated Item:** 4. Concrete Surfaces **Caption:** Rating: Acceptable; Remarks: Spalling was observed on concrete floor. Minor spall has no bearing on the integrity of the floodwall.; Action: No action required at this time.

Interior Drainage System

For use during Initial and Continuing Eligibility Inspections of interior drainage systems

Rated Item	Rating	Rating Guidelines		Location/Remarks/Recommendations
1. Vegetation and Obstructions	**M**	**A**	No obstructions, vegetation, debris, or sediment accumulation noted within interior drainage channels or blocking the culverts, inlets, or discharge areas. Concrete joints and weep holes are free of grass and weeds.	Plantings that were observed on the PI were part of the original construction contract of the levee and have minimal risk the integrity of the levee.
		M	Obstructions, vegetation, debris, or sediment are minor and have not impaired channel flow capacity or blocked more than 10% of any culvert openings, but should be removed. A limited volume of grass and weeds may be present in concrete channel joints and weep holes.	
		U	Obstructions, vegetation, debris, or sediment have impaired the channel flow capacity or blocked more than 10% of a culvert opening. Sediment and debris removal required to re-establish flow capacity.	
2. Encroachments	**A**	**A**	No trash, debris, unauthorized structures, excavations, or other obstructions present within the easement area. Encroachments have been previously reviewed by the Corps, and it was determined that they do not diminish proper functioning of the interior drainage system.	All landside structures have been approved and pose no threat to the floodwall.
		M	Trash, debris, unauthorized structures, excavations, or other obstructions present, or inappropriate activities noted that should be corrected but will not inhibit operations and maintenance or emergency operations. Encroachments have not been reviewed by the Corps.	
		U	Unauthorized encroachments or inappropriate activities noted are likely to inhibit operations and maintenance, emergency operations, or negatively impact the integrity of this component of the interior drainage system.	
3. Ponding Areas	**NA**	**A**	No trash, debris, structures, or other obstructions present within the ponding areas. Sediment deposits do not exceed 10% of capacity.	
		M	Trash, debris, excavations, structures, or other obstructions present, or inappropriate activities that will not inhibit operations and maintenance. Sediment deposits do not exceed 30% of capacity.	
		U	Trash, debris, excavations, structures, or other obstructions, or other encroachments or activities noted that will inhibit operations, maintenance, or emergency work. Sediment deposits exceeds 30% of capacity.	
		N/A	There are no ponding areas associated with the interior drainage system.	
4. Fencing and Gates[1]	**NA**	**A**	Fencing is in good condition and provides protection against falling or unauthorized access. Gates open and close freely, locks are in place, and there is little corrosion on metal parts.	
		M	Fencing or gates are damaged or corroded but appear to be maintainable. Locks may be missing or damaged.	
		U	Fencing and gates are damaged or corroded to the point that replacement is required, or potentially dangerous features are not secured.	
		N/A	There are no features noted that require safety fencing.	
5. Concrete Surfaces (Such as gate	**A**	**A**	Negligible spalling, scaling or cracking. If the concrete surface is weathered or holds moisture, it is still satisfactory but should be seal coated to prevent freeze/ thaw damage.	

Key: A = Acceptable. M = Minimally Acceptable; Maintenance is required. U = Unacceptable. N/A = Not Applicable. FDR = Flood Damage Reduction

Interior Drainage System

For use during Initial and Continuing Eligibility Inspections of interior drainage systems

Rated Item	Rating	Rating Guidelines		Location/Remarks/Recommendations
wells, outfalls, intakes, or culverts)		**M**	Spalling, scaling, and open cracking present, but the immediate integrity or performance of the structure is not threatened. Reinforcing steel may be exposed. Repairs/ sealing is necessary to prevent additional damage during periods of thawing and freezing.	
		U	Surface deterioration or deep cracks present that may result in an unreliable structure. Any surface deterioration that exposes the sheet piling or lies adjacent to monolith joints may indicate underlying reinforcement corrosion and is unacceptable.	
		N/A	There are no concrete items in the interior drainage system.	
6. Tilting, Sliding or Settlement of Concrete and Sheet Pile Structures[2] (Such as gate wells, outfalls, intakes, or culverts)	**A**	**A**	There are no significant areas of tilting, sliding, or settlement that would endanger the integrity of the structure.	No tilting, sliding or settlement of concrete floodwall was observed during PI.
		M	There are areas of tilting, sliding, or settlement (either active or inactive) that need to be repaired. The maximum offset, either laterally or vertically, does not exceed 2 inches unless the movement can be shown to be no longer actively occurring. The integrity of the structure is not in danger.	
		U	There are areas of tilting, sliding, or settlement (either active or inactive) that threaten the structure's integrity and performance. Any movement that has resulted in failure of the waterstop (possibly identified by daylight visible through the joint) is unacceptable. Differential movement of greater than 2 inches between any two adjacent monoliths, either laterally or vertically, is unacceptable unless it can be shown that the movement is no longer active. Also, if the floodwall is of I-wall construction, then any visible or measurable tilting of the wall toward the protected side that has created an open horizontal crack on the riverside base of a monolith is unacceptable.	
		N/A	There are no concrete items in the interior drainage system.	
7. Foundation of Concrete Structures[3] (Such as culverts, inlet and discharge structures, or gatewells.)	**A**	**A**	No active erosion, scouring, or bank caving that might endanger the structure's stability.	No foundation concerns were observed during PI.
		M	There are areas where the ground is eroding towards the base of the structure. Efforts need to be taken to slow and repair this erosion, but it is not judged to be close enough to the structure or to be progressing rapidly enough to affect structural stability before the next inspection. The rate of erosion is such that the structure is expected to remain stabile until the next inspection.	
		U	Erosion or bank caving observed that may lead to structural instabilities before the next inspection.	
		N/A	There are no concrete items in the interior drainage system.	
8. Monolith Joints	**A**	**A**	The joint material is in good condition. The exterior joint sealant is intact and cracking/ desiccation is minimal. Joint filler material and/or waterstop is not visible at any point.	No monolith concerns were observed during PI.
		M	The joint material has appreciable deterioration to the point where joint filler material and/or waterstop is visible in some locations. This needs to be repaired or replaced to prevent spalling and cracking during freeze/ thaw cycles, and to ensure water tightness of the joint.	

Key: A = Acceptable. M = Minimally Acceptable; Maintenance is required. U = Unacceptable. N/A = Not Applicable. FDR = Flood Damage Reduction

Interior Drainage System

For use during Initial and Continuing Eligibility Inspections of interior drainage systems

Rated Item	Rating	Rating Guidelines		Location/Remarks/Recommendations
		U	The joint material is severely deteriorated or the concrete adjacent to the monolith joints has spalled and cracked, damaging the waterstop; in either case damage has occurred to the point where it is apparent that the joint is no longer watertight and will not provide the intended level of protection during a flood.	
		N/A	There are no monolith joints in the interior drainage system.	
9. Culverts/ Discharge Pipes[4]	**A**	**A**	There are no breaks, holes, cracks in the discharge pipes/ culverts that would result in significant water leakage. The pipe shape is still essentially circular. All joints appear to be closed and the soil tight. Corrugated metal pipes, if present, are in good condition with 100% of the original coating still in place (either asphalt or galvanizing) or have been relined with appropriate material, which is still in good condition. Condition of pipes has been verified using television camera video taping or visual inspection methods within the past five years, and the report for every pipe is available for review by the inspector.	No culvert obstructions, breaks or cracks were observed during PI.
		M	There are a small number of corrosion pinholes or cracks that could leak water and need to be repaired, but the entire length of pipe is still structurally sound and is not in danger of collapsing. Pipe shape may be ovalized in some locations but does not appear to be approaching a curvature reversal. A limited number of joints may have opened and soil loss may be beginning. Any open joints should be repaired prior to the next inspection. Corrugated metal pipes, if present, may be showing corrosion and pinholes but there are no areas with total section loss. Condition of pipes has been verified using television camera video taping or visual inspection methods within the past five years, and the report for every pipe is available for review by the inspector.	
		U	Culvert has deterioration and/or has significant leakage; it is in danger of collapsing or as already begun to collapse. Corrugated metal pipes have suffered 100% section loss in the invert. HOWEVER: Even if pipes appear to be in good condition, as judged by an external visual inspection, an Unacceptable Rating will be assigned if the condition of pipes has not been verified using television camera video taping or visual inspection methods within the past five years, and reports for all pipes are not available for review by the inspector.	
		N/A	There are no discharge pipes/ culverts.	
10. Sluice / Slide Gates[5]	**NA**	**A**	Gates open and close freely to a tight seal or minor leakage. Gate operators are in good working condition and are properly maintained. Sill is free of sediment and other obstructions. Gates and lifters have been maintained and are free of corrosion. Documentation provided during the inspection.	
		M	Gates and/or operators have been damaged or have minor corrosion, and open and close with resistance or binding. Leakage quantity is controllable, but maintenance is required. Sill is free of sediment and other obstructions.	
		U	Gates do not open or close and/or operators do not function. Gate, stem, lifter and/or guides may be damaged or have major corrosion.	
		N/A	There are no sluice/ slide gates.	

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Interior Drainage System

For use during Initial and Continuing Eligibility Inspections of interior drainage systems

Rated Item	Rating	Rating Guidelines		Location/Remarks/Recommendations
11. Flap Gates/ Flap Valves/ Pinch Valves[1]	**A**	**A**	Gates/ valves open and close easily with minimal leakage, have no corrosion damage, and have been exercised and lubricated as required.	NRN1_2020_a_0002: Station_1 NA: Flap gate, sponsor relays that is exercised twice a year.: Monitor. (A) NRN1_2020_a_0004: Station_1 NA: Flap gate in good working order. Exercised twice a year.: No action required at this time. (A)
		M	Gates/ valves will not fully open or close because of obstructions that can be easily removed, or have minor corrosion damage that requires maintenance.	
		U	Gates/ valves are missing, have been damaged, or have deteriorated to the point that they need to be replaced.	
		N/A	There are no flap gates.	
12. Trash Racks (non-mechanical)	**NA**	**A**	Trash racks are fastened in place and properly maintained.	
		M	Trash racks are in place but are unfastened or have bent bars that allow debris to enter into the pipe or pump station, bars are corroded to the point that up to 10% of the sectional area may be lost. Repair or replacement is required.	
		U	Trash racks are missing or damaged to the extent that they are no longer functional and must be replaced. (For example, more than 10% of the sectional area may be lost.)	
		N/A	There are no trash racks, or they are covered in the pump stations section of the report.	
13. Other Metallic Items	**NA**	**A**	All metal parts are protected from corrosion damage and show no rust, damage, or deterioration that would cause a safety concern.	
		M	Corrosion seen on metallic parts appears to be maintainable.	
		U	Metallic parts are severely corroded and require replacement to prevent failure, equipment damage, or safety issues.	
		N/A	There are no other significant metallic items.	
14. Riprap Revetments of Inlet/ Discharge Areas	**NA**	**A**	No riprap displacement or stone degradation that could pose an immediate threat to the integrity of channel bank. Riprap intact with no woody vegetation present.	
		M	Minor riprap displacement or stone degradation that could pose an immediate threat to the integrity of the channel bank. Unwanted vegetation must be cleared or sprayed with an appropriate herbicide.	
		U	Significant riprap displacement, exposure of bedding, or stone degradation observed. Scour activity is undercutting banks, eroding embankments, or impairing channel flows by causing turbulence or shoaling. Rock protection is hidden by dense brush, trees, or grasses.	
		N/A	There is no riprap protecting this feature of the segment / system, or riprap is discussed in another section.	
15. Revetments other than Riprap	**NA**	**A**	No riprap displacement or stone degradation that could pose an immediate threat to the integrity of channel bank. Riprap intact with no woody vegetation present.	

Key: A = Acceptable. M = Minimally Acceptable; Maintenance is required. U = Unacceptable. N/A = Not Applicable. FDR = Flood Damage Reduction

Interior Drainage System

For use during Initial and Continuing Eligibility Inspections of interior drainage systems

Rated Item	Rating	Rating Guidelines		Location/Remarks/Recommendations
		M	Minor riprap displacement or stone degradation that could pose an immediate threat to the integrity of the channel bank. Unwanted vegetation must be cleared or sprayed with an appropriate herbicide.	
		U	Significant riprap displacement, exposure of bedding, or stone degradation observed. Scour activity is undercutting banks, eroding embankments, or impairing channel flows by causing turbulence or shoaling. Rock protection is hidden by dense brush, trees, or grasses.	
		N/A	There are no such revetments protecting this feature of the segment / system.	

[1] Proper operation of this item must be demonstrated during the inspection.

[2] The sponsor should be monitoring any observed movement to verify whether the movement is active or inactive.

[3] Inspectors must have as-built drawings available during the inspection so that the lateral distance to the heel and toe of the floodwalls can be determined in the field.

[4] The decision on whether or not USACE inspectors should enter a pipe to perform a detailed inspection must be made at the USACE District level. This decision should be made in conjunction with the District Safety Office, as pipes may be considered confined spaces. This decision should consider the age of the pipe, the diameter of the pipe, the apparent condition of the pipe, and the length of the pipe. If a pipe is entered for the purposes of inspection, the inspector should record observations with a video camera in order that the condition of the entire pipe, including all joints, can later be assessed. Additionally, the video record provides a baseline to which future inspections can be compared.

[5] Proper operation of the gates (full open and closed) must be demonstrated during the inspection if no documentation is available. Be aware of both manual and electrical operators.

Interior Drainage System

For use during Initial and Continuing Eligibility Inspections of interior drainage systems


2020/07/22 12:59

Inspect ID: NRN1_2020_a_0002 **Title:** USACE_CESPN_NRN1_2020_a_0002_1.jpg **Rated Item:** 11. Flap Gates/ Flap Valves/ Pinch Valves **Caption:** Rating: Acceptable; Remarks: Flap gate, sponsor relays that is exercised twice a year.; Action: Monitor.


2020/07/22 13:00

Inspect ID: NRN1_2020_a_0002 **Title:** USACE_CESPN_NRN1_2020_a_0002_2.jpg **Rated Item:** 11. Flap Gates/ Flap Valves/ Pinch Valves **Caption:** Rating: Acceptable; Remarks: Flap gate, sponsor relays that is exercised twice a year.; Action: Monitor.

	**Inspect ID:** NRN1_2020_a_0004 **Title:** USACE_CESPN_NRN1_2020_a_0004_1.jpg **Rated Item:** 11. Flap Gates/ Flap Valves/ Pinch Valves **Caption:** Rating: Acceptable; Remarks: Flap gate in good working order. Exercised twice a year.; Action: No action required at this time.

Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community

2020 Levee Inspection

Napa River,
below Napa Creek, California
Pg. 1 of 1
Bank: Right

Legend

Point Features

Rating:

- 🔴 Unacceptable
- 🟡 Minimally Acceptable
- 🟢 Acceptable
- 🔵 N/A

Line Features

Rating:

- 🟥 Unacceptable
- 🟨 Minimally Acceptable
- 🟩 Acceptable
- 🟪 Centerline

0 30 60 120 180 240 Feet

N

US Army Corps
of Engineers
Walla Walla District

CREATED BY: Nathan DeLannoy
LAST UPDATED BY: g4ecdnld
MAP ID: MXD_NRN1_DDP.mxd
DATE: 09/02/20
COORDINATE SYSTEM: GCS North American 1983
Datum: North American 1983

DISCLAIMER

Appendix C

2005 Hatt Bldg to First Street Floodwall

Geotechnical Design Documentation Report

NAPA RIVER/NAPA CREEK FLOOD PROTECTION PROJECT
NAPA, CALIFORNIA
CONTRACT 2 WEST
HATT BUILDING TO FIRST STREET

GEOTECHNICAL DESIGN DOCUMENT REPORT

Prepared by: Soil Design Section, Sacramento District
U.S. Army Corps of Engineers

May 2005

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Napa River/Napa Creek Flood Protection Project
Contract 2 West
Hatt Building to First Street

Geotechnical Design Document Report

1. Introduction. The purpose of this report is to document the design process for preparing plans and specifications for the Napa River/Napa Creek Flood Protection Project, Contract 2 West, Hatt Building to First Street (also known as "Hatt to First"). This report is intended as a supplement to the Napa River Geotechnical Basis of Design, prepared in February 1998 as an appendix to the Final Supplemental General Design Memorandum (SGDM), dated October 1998 (Reference 1). This report presents information obtained and analyses performed since the SGDM, and discusses this portion of the flood control project in greater detail.

1.1. Project Delivery Team. The PDT for this contract was comprised of both Corps of Engineers and A/E personnel. The geotechnical design was performed by Corps of Engineers personnel. Structural and Civil design was performed by MGE Engineering, Inc., Sacramento, with oversight by Corps personnel. Landscape architecture and electrical design was performed by The HLA Group, Sacramento, with oversight by Corps personnel.

1.2. Area Description. The contract area, with the major project features, is shown on Figures 1 through 3. The contract area is on the west side of the Napa River in downtown Napa, extending from the Hatt Building (also known as the Napa Mill) on the south (downstream) to just south of First Street on the north (upstream.) The Napa Mill was originally constructed in the late 1800's to early 1900's and served as a grain mill. After being vacant for a number of years, it is being renovated and refurbished as a tourist destination with hotel rooms, restaurants, a general store, and patios (Photos 1 through 5). Fifth Street is immediately north of the Napa Mill. North of Fifth Street are vacant lots which are slated for development (the Channel Development) at a later date (Photos 6 and 7). The recently constructed Third Street Bridge is immediately north of the vacant lots (Photo 7). North of Third Street are Veterans Park (Photo 8), Downtown Joe's restaurant (Photo 9), a parking lot, and an existing concrete counterfort wall near the Semorile Building (location of the Bounty Hunter wine bar) at the north end of the project (Photo 10). The soldier pile wall in this contract will tie into the concrete counterfort wall.

1.3. Major Contract Features. The main contract feature is a vertical soldier pile retaining wall which extends from the Napa Mill to the existing concrete counterfort wall just south of First Street. At the Napa Mill, this wall functions as the 100-year floodwall. North of the Napa Mill, the top of the soldier pile wall lowers in elevation to allow for a pedestrian walkway (called the lower promenade), and a second, shorter wall (called the upper wall), which provides the 100-year flood protection north of the Napa

Mill. Another pedestrian walkway (called the upper promenade) is on the landside of the upper wall, at the finish grade elevation (slightly above the existing ground elevation in most places). The soldier pile wall and lower promenade dip underneath the Third Street Bridge, then rise in elevation and continue northward to tie into the existing concrete counterfort wall. The soldier pile wall (also referred to as the lower wall) provides 12-year flood protection at it's lowest point, underneath the Third Street Bridge. The lower wall provides greater flood protection over the remaining project area. Stairs and ramps provide access to the lower promenade at Fifth Street, Fourth Street, Third Street, Veterans Park, and the Semorile Building. As part of the contract, Veterans Park will be completely rebuilt into a small amphitheater.

2.0. Geotechnical Explorations. At the time of the SGDM preparation, Soil Design section had the following explorations in the Hatt to First contract area, from south to north: 2F-90-29, 2F-30 (just south of the Napa Mill); 2F-94-14 (just north of the Napa Mill); 2F-29, CPT-94-2, and 2F-94-15 (near the Third Street bridge). For plans and specifications, more subsurface information was needed, so the following deep explorations were conducted by the Corps: 2F-03-3, 2F-03-4, 2F-04-51 (from a barge in the Napa River near the Napa Mill); 2F-03-5, 2F-03-6, 2F-03-7 (between Fifth Street and Third Street); and 2F-03-8 (in the parking lot north of Downtown Joes). Numerous shallow exploration logs at the Napa Mill, many conducted for an environmental assessment, were obtained from Raney Geotechnical. Two boring logs for the construction of the Third Street Bridge (B-3 and B-4) were obtained from AGS, Inc. Locations of explorations are shown on Figure 4. Soil boring logs from the Corps of Engineers and AGS Inc. are shown on Figures 5 through 16. Soil boring logs from Raney Geotechnical are in Appendix 1.

2.1. Subsurface Conditions – Napa Mill. The land-based borings in the vicinity of the Napa Mill indicate a soil profile of silts and clays to a depth of about 20 feet, underlain by a dense, 20 to 25-foot thick clayey sand and gravel, underlain by 12 feet of clay, underlain by another dense clayey sand and gravel layer approximately 10 feet thick, underlain by a lean clay. The groundwater level varies on the borings logs, but is generally about 13 feet below ground surface, or 7 feet above the top of the upper dense sand and gravel layer. Borings 2F-03-3 and 2F-03-4, into the riverbed, were intended to be 70 feet in depth, but problems during drilling restricted the depths to 35 and 24 feet respectively. Drilling was very slow because the hollow-stem augers could not drill through the upper dense sand and gravel layer and extensive wood debris (believed to be remnants of boat docks constructed by the mill), and one hole was terminated due to hard material (most likely boulders or concrete rubble) which caused refusal. Due to the drilling problems in 2003, boring 2F-04-51 was drilled by mud rotary in the riverbed in 2004, successfully reaching full depth of 75 feet. The riverbed explorations indicate a subsurface profile of 5 to 10 feet of very soft silts, silty sands, and clays (river sediments), underlain by a 25-foot thick dense clayey sand and gravel, underlain by 25 feet of lean clay, underlain by 10 feet of dense clayey sand and gravel, underlain by lean clay.

2.2. Subsurface Conditions – Fifth Street to First Street. Soil borings between Fifth and First Streets indicate a soil profile of 20 to 22 feet of sandy clay, underlain by a dense clayey sand and gravel, underlain by sandy lean clay, underlain by another dense clayey sand and gravel layer, underlain by lean clay. The upper dense clayey sand and gravel layer is about 30 feet thick from Fifth Street to just south of Third Street, where it decreases to 8 to 10 feet thick. The underlying clay layer is 12 feet thick at Fifth Street, and increases to 36 feet thick just south of Third Street. Upstream of Third Street, this middle clay layer consists of about 19 feet of fat clay overlying about 17 feet of lean clay. The lower clayey sand and gravel layer is about 8 to 10 feet thick over the entire area. The groundwater level on the boring logs varies but is about 14 feet below ground surface between Fifth and Third Streets, and about 20 feet below ground surface upstream of Third Street. The groundwater level is about 2 to 6 feet above the top of the upper dense sand and gravel layer. Comparison of boring logs on land and in the river at both the Third Street Bridge and the Napa Mill indicate that the elevation of the top of the upper dense sand and gravel layer is several feet lower in the river than it is in the upland areas.

3. Foundation Conditions of Existing Structures. Foundation information was obtained for some of the existing structures in the contract area. The Napa River Inn Suites building at the Napa Mill has two rows of 30-foot deep piles on the east side of the building (closest to the soldier pile wall in this contract) and individual spread footings over the rest of the building. Foundation conditions of the main Napa River Inn building are not known. A portion of the building which overhangs the riverbank is founded on piles, but it is not known if the entire building has a pile foundation. The Third Street bridge abutment is founded on piles. Downtown Joe's has a shallow foundation. The counterfort concrete wall at the north end of the project has a shallow foundation. No foundation information is available on the Semorile Building. Because the Semorile building is a relatively light structure and the soils in the area have good bearing capacity, it is possible the building has a shallow foundation.

4. Channel Development. The Channel Development is a planned, private-sector development for the (currently) empty fields between Fifth and Third Streets. The project is in design concurrently with the Hatt to First project. Preliminary plans show two 3-story buildings with basement parking garages. The first story will be retail shops, and the upper stories will be offices and condominiums. A meeting was held in early 2004 between the Corps, MGE, HLA, the Channel landowners, and the Channel design A/E. Among other items, the location and elevations of a match line between the two projects was determined. Both the Corps and Channel design teams will design up to the match line. The Channel buildings will be no less than 37 feet from the lower wall of the Hatt to First project. Thirty-seven feet was chosen in 2003 after a preliminary wall design by the Corps of Engineers. The preliminary design assumed tiebacks would be used for the lower soldier pile wall, and 37 feet was chosen because it was beyond the anticipated tieback length, to avoid conflict between tiebacks and any below-ground foundation or parking garage that the Channel team would design. The Channel Development is not anticipated to impact the design of the lower and upper wall in this contract. While final plans for the Channel Development have not been produced, the first floor of the Channel Development will likely be somewhere between 16 and 19 feet NGVD in elevation. The bottom of the lower wall is at elevation 1 foot NGVD between Fifth and Third Streets. It is anticipated that the below-ground parking garage will be at least 12 to 15 feet tall. Assuming the “worst-case” situation of a shallow foundation, the foundation will likely be several feet thick, so the bottom of the Channel Development will be at or below the bottom of the lower wall. The Channel Development is also outside the active failure wedges of both the upper and lower walls in this contract (see Appendix 7, first page). The schedules at this time indicate the Hatt to First contract will be constructed prior to the Channel Development. Assuming this is the case, the Channel contractor will have to vertical cut and shore the excavation for their foundation/parking garage construction. Vertical cut and temporary shoring causes a horizontal stress release in the surrounding soil, which can lead to deflection of the shoring and settlement of the soil behind the shoring. The lower wall of this contract is not anticipated to be affected due to the distance away and the 40-foot deep piles that the wall is founded upon. The upper wall of this contract will be a minimum of 24 feet from the Channel Development. The upper wall might be impacted as this wall is closer to the development and it has a shallow foundation. The upper wall should be monitored during Channel Development construction.

5. Liquefaction Evaluation. Most of the SPT N-values obtained in the sand and gravel layers are above 30, indicating the soils are extremely unlikely to liquefy during an earthquake. A few zones of lower SPT N-values do exist. A liquefaction analysis using the simplified procedure of Seed and Idriss (reference 11) was conducted. Results are given in Appendix 2. This analysis showed there is no potential for liquefaction in the sand and gravel layers in the project area.

6. Floodwall Design. In the SGDM, the vertical wall was identified as a soldier pile wall with tiebacks, and the shorter upper wall (north of Fifth Street) was identified as a standard, T-shaped cantilever floodwall.

6.1. Subsurface Profiles and Material Properties. Three soil profiles were provided to the structural engineers for the design of the floodwalls. These profiles were developed by examining the soil borings in the project area. One profile covers the Napa Mill area, another profile covers the area from Fifth Street to just south of Third Street, and the third profile covers the remaining area. The profiles are shown in Appendix 3. The sampling/laboratory testing plans for the deep soil borings drilled in 2003 included undisturbed sampling and triaxial shear strength testing of clay soils. Not all the planned undisturbed samples were actually collected, and some of the triaxial test results were not believeable (for example, drained cohesion of 1200 pounds/square foot). Therefore most of the properties of the clay soils shown in Appendix 3 are values developed in the SGDM. Unconfined compression, triaxial, and consolidation test results are given in Appendix 4. The SPT N-values were not used to determine the phi angles of the clayey sand and gravel layers because the presence of the gravels produces artificially high N-values. References 16 and 17 were used to determine the phi angles. Both references recommend using a phi angle greater than 34 degrees for a silty gravel (USCS classification GM) and a phi angle greater than 31 degrees for a clayey gravel (USCS classification GC). References 16 and 17 also recommend phi angles of 33 and 31 degrees respectively for clayey sands (USCS classification SC). A phi angle of 33 degrees was used for this project. The shallow fat clay layer shown between 10 and 16 feet below ground surface in the Napa Mill profile does not appear to be continuous over the entire Mill area; it was logged in some explorations but not in others. The layer was included in the soil profile used for wall design for conservatism; fat clays in general have lower shear strengths than lean clays.

6.2. Soldier Pile Wall Design.

6.2.1. Tiebacks. Early in the design process, the feasibility of using tiebacks was examined. Since the preparation of the SGDM, a new building (Napa River Inn Suites) has been constructed at the Napa Mill. The soldier pile wall will be 10 feet from the eastern side of that building. As stated previously, the building has two rows of pile foundations closest to the soldier pile wall. It was decided that tiebacks could not be used at the Napa Mill and at Downtown Joes due to interference with the existing foundations (the decision was made long before foundation drawings of Downtown Joes were obtained). MGE designed the soldier pile so that tiebacks are not used anywhere along it's length.

6.2.2. Pile Installation. The method of pile installation was examined. Due to the presence of the hard sand and gravel layer, the potential presence of subsurface obstructions, and the closeness of operating private businesses (noise complaints and possible vibration damage), driving piles using either a drop or vibration hammer is not feasible. Jetting is also not feasible because jetting can cause unacceptable settlements in nearby structures. Cast-in-drilled-hole (CIDH) piles will be used over the entire contract area.

6.2.3. Preliminary Design. MGE's preliminary analysis, documented in their Wall Type Selection report (Reference 5) showed that the soldier pile wall from station 0+00 to station 2+48 (where the wall height is greater than 20 feet) will need to be on a footing with two, 2-foot diameter CIDH piles (called "Wall Type A"). Where the wall height is between 17 and 20 feet, a standard soldier pile design with 40-foot deep, 3-foot diameter CIDH soldier piles (called "Wall Type B") is adequate . Where the wall height is less than 17 feet, a standard soldier pile design with 40-foot deep, 2-foot diameter CIDH soldier piles (called "Wall Type C") is adequate.

6.2.4. Wall Loadings/Design. MGE submitted calculations of the wall loadings, design values, and deflections in each of their submittals. The final values are in the Structural Design Calculations (100% Submittal) report (reference 6). For hydraulic structures, EM 1110-2-2502 (Reference 3) recommends the use of the coefficient of earth pressure at rest (K_0) rather than the active earth pressure coefficient (K_A) for calculating horizontal soil pressures on retaining and flood walls. This is because hydraulic structures are often critical features, and since K_0 is greater than K_A, the calculated loadings will be higher, resulting in a more conservative design. For each wall type, the station with the greatest free wall height was chosen for design. The soil and water loadings were calculated for four different cases: end-of-construction, long-term with no flood, long-term with a flood, and long-term with an earthquake and no flood. The case which produced the highest loadings was selected for structural design purposes. The small passive wedge above the bottom of the soldier pile wall was ignored in all the calculations, simulating erosion at the toe of the wall. A rapid drawdown case was not examined because rapid drawdown conditions are highly unlikely to develop in this project. The 100-year hydrograph for the Napa River indicates the river level rises and falls relatively quickly (2 days). The vertical concrete wall faces, the pavements on the upper and lower promenade, and the trench drains will reduce water infiltration into the soils behind the retaining walls. The lower wall has a drainage system consisting of a geocomposite drainage net, gravelly sand structural backfill, and a collector pipe surrounded in gravel with weepholes about 1 foot above the mean high tide water level. Any excess water that infiltrates the backfill material will drain relatively quickly.

6.2.5. Pile Design. Four references (EM 1110-1-1905, Bearing Capacity of Soils, Reference 2; NAVFAC 7.2, Foundations and Earth Structures, Reference 7; Engineering Manual for Drilled Shafts, Virginia Tech, Reference 9; and the FHWA Drilled Shafts Manual, Reference 15) were used to determine the pile depth for the (originally planned) 2-pile foundation system between stations 0+00 and 2+48 using the maximum compression and tension loadings supplied by MGE at the 65% design submittal stage (327 kips and 149 kips respectively). The end bearing and skin friction were calculated using all four of the references, and those values were averaged for the final design value. Preliminary calculations are on file in the Soil Design Section; the final design calculation spreadsheet is given in Appendix 5. Once it was determined that seating the piles within the upper dense sand and gravel layer would not produce the needed design loads, it was desired to seat the piles in the upper 1 or 2 feet of the lower dense sand and gravel layer to take advantage of the increased end bearing value of that layer. However, the exact elevation of the top of that layer is not known because only one deep boring exists in the river, and the exact location of that boring is not known. (GPS coordinates taken at the time of drilling place the boring in the middle of the river, which according to the field geologist was not the actual location. The boring was located on the site map by the geologist from memory.) Therefore, to be conservative, the end bearing was calculated using the methods for clay soils, which produce a lower end bearing than the methods for granular soils. The construction specifications require the Contractor to drill small-diameter pilot holes every 24 lineal feet over the critical Wall Type A foundation area prior to production CIDH pile installation. The pilot holes will provide additional subsurface information prior to pile installation. Installing piles often causes changes in the density of the surrounding soils. The drilling process to be used in this project causes the density of granular soils to decrease. This decrease in density causes a decrease in skin friction. The calculated skin friction of the granular layers was multiplied by 0.7 (resulting in a decreased skin friction value) as per the references. Drilling does not significantly effect the density of cohesionless (clay) soils, so no multiplier was used to reduce the skin friction of the clay soils. Calculations showed the preliminary design produced loadings that were too high for the initial pile geometry. Difficult site conditions (nearby building, no equipment access by land, river water, and soft river sediments) make performing a pile load test almost logistically impossible at this site. EM 1110-2-2906, Design of Pile Foundations (Reference 4) states minimum factors of safety (F.S.) for pile design are 2.0 if a pile load test will be conducted and 3.0 if a pile load test will not be conducted. After discussion with MGE, a new pile geometry was designed for Wall Type A. The new pile geometry consists of rows of three 2-foot diameter piles located 8 feet apart. A pile tip elevation of –70 feet NGVD was provided to MGE for inclusion on the plans and in quantities for the cost estimate. Sixty-foot deep piles will provide the required compression and tension loadings utilizing an F.S. of 3. The pile loadings are given in Table 1. MGE performed final design, including lateral seismic loading and deflection, using the LPILE computer program. The LPILE output is included in the Final Design Calculations (Reference 6).

Table 1. Pile Design Loadings.

Load Condition	Maximum Load (kips) (F.S. = 3)
Compression	86.87
Tension	60

6.2.6. Wall Deflection/Settlement. The force of the active soil wedge behind a retaining wall will cause the wall to deflect outward over time. The Corps of Engineers does not have a set requirement for retaining wall deflection (such as the maximum allowable deflection is x% of the wall height). Obviously, deflection of the retaining walls and the pile foundations must not be large enough to negatively impact the structural capacity of those elements. The maximum deflection which will not negatively impact the structural capacity will be determined by MGE. As the retaining wall deflects outward, it causes settlement of the soil behind the retaining wall, as the soil fills in the “gap” between the as-constructed wall and the deflected wall. The maximum settlement will be immediately behind the wall, and the settlement will taper off to zero at some distance away from the wall. Viewed in cross section, the area between the as-constructed and the deflected wall shapes is typically assumed to be equal to the area between the end-of-construction ground surface and the settled ground surface behind the deflected retaining wall. An extensive literature search showed that no recent data has been published on the deflection of cantilever retaining walls; all of the published data is for retaining walls with tiebacks or braced cuts. To estimate deflection, a chart published by Ralph Peck (Reference 14) was used. The chart showed, for the soil types at this site and assuming average workmanship, the settlement will taper to zero where the (distance from excavation divided by the depth of excavation) is about 2. For a free wall height of 24 feet (Wall Type A at the Napa Mill, the maximum for this project), that distance is 48 feet. Also according to the chart, the (settlement immediately behind the wall divided by the depth of excavation) will be about 1%. For a free wall height of 24 feet, that is 0.24 feet (2.9 inches) of settlement. The chart showed a slight curvature between the 2 endpoints, but it is almost a straight line and it is common in practice to use a straight line. MGE used the LPILE computer program to determine the deflection of the pile foundation for each wall type in this contract, and they also calculated the deflection of the top of the wall for each wall type. Those calculations are documented in their final Structural Design Calculations report (reference 6). The total deflection at the top of the retaining wall is calculated as 0.89 inches for Wall Type A at the Napa Mill; 1.81 inches for Wall Type B, 1.75 inches for Wall Type C downstream of Third Street, and 3.41 inches for Wall Type C upstream of Third Street. Calculations conducted for Wall Type A (Appendix 6) indicate the deflected area behind the retaining wall is considerably less than the settlement area calculated using the Peck chart. This indicates that the retaining walls in this contract are very stiff and the anticipated settlement will likely be less than that estimated using the Peck chart. Except for Downtown Joes and possibly the Semorile Building, all of the structures adjacent to the new retaining walls in this contract are founded on piles. Given the low calculated deflections and the pile foundations, settlement of those structures is anticipated to be less than one inch. Downtown Joes is approximately 15 feet from Wall Type B, and the elevation of the bottom of their shallow foundation footings is several feet below the top of Wall Type B. Given the low deflection, the distance away from the new retaining wall, and the footing depth, settlement of Downtown Joes is also expected to be less than an inch. The deflection of the top of Wall Type C near the Semorile Building has been calculated at 3.41 inches. Foundation conditions of the Semorile Building are not known. Given the larger deflection of the retaining wall at this location and the unknown foundation conditions, this building may experience more settlement than the others, but an exact amount can not be accurately predicted. During construction, all existing structures in the project area will be monitored daily for settlement (see Geotechnical Instrumentation section). If the settlement of any structure exceeds 1 inch (or 0.75 inches for the old historic building at the Napa Mill), the Government will be notified and the Contractor must stop work and adjust his methods, equipment, and/or operations to prevent additional settlement.

6.2.7. Drainage and Excavation/Backfill. A geosynthetic wall drain with a collector pipe and weep holes located about 1 foot above the mean high tide water surface elevation will provide drainage to the retained soil behind the soldier pile wall. Granular material with 35 to 100% finer than the No. 4 sieve and no more than 5% finer than the #200 sieve is specified as structural backfill material. The sand backfill will assist in drainage behind the walls. Except at the Napa Mill and possibly at Downtown Joes, the excavation required to construct the walls will be cut back no steeper than a 1H:1V slope on the landside. After construction of the walls, the excavation will be backfilled with structural backfill material to ensure that the entire active wedge failure zone behind the walls is composed of the same soil type. At the Napa Mill, existing buildings located close to the wall do not permit sloping the excavation. Between stations 0+00 and approximate station 2+40, the soldier pile wall is a fill wall. Upstream of station 2+40, the soldier pile wall is a cut wall. For the fill wall segment, minor excavation is needed at the base of the wall, and the area between the constructed wall and the existing ground surface will be filled with structural backfill material. In the cut wall segment, temporary shoring, approximately 2 or 3 feet behind the back of the completed soldier pile wall, will be used to stabilize the excavation. The area between the completed soldier pile wall and the temporary shoring will be backfilled with structural backfill material. Design of the temporary shoring is the responsibility of the Contractor. The contract specifications require the Contractor to submit a temporary shoring plan for Government approval prior to construction. Two possibilities for temporary shoring are a soldier pile wall with H-piles in drilled holes and wood lagging, or a soil nailed wall.

6.2.8. Pile Specification. Neither the Sacramento District nor the Unified Facilities Guide Specification (UFGS) databases include a guide specification for CIDH piles. After a review of all the concrete pile specifications, it was decided to modify the UFGS Drilled Foundation Caisson guide specification for this project. Based on the soil boring logs, groundwater is expected to infiltrate the pile borings, and caving sands were encountered in a few of the borings. The specification will require the Contractor to use temporary steel casings, concrete seal courses, and/or pumping (or any combination thereof) in the pile boreholes to prevent groundwater infiltration and sidewall caving. The specification will also require the drilling of small-diameter pilot holes every 24 lineal feet prior to the production pile drilling for Wall Type A. The purpose of the pilot holes is to obtain additional information about the subsurface soil conditions and the presence of any subsurface drilling obstructions prior to production CIDH pile drilling in this critical area.

6.3. Upper Wall Design. EM 1110-2-1905 (Reference 2) and EM 1110-2-2502 (Reference 3) were used to calculate the bearing capacity of the soils for the shallow foundation of the upper wall. Details are shown in Appendix 7. The drawing on the first page of Appendix 7 is a to-scale depiction of the anticipated construction conditions of the dual-wall system, with 1 foot of structural backfill below the shallow footing. Depending on the reference, the zone of influence for shallow foundation bearing capacity extends below the footing to a depth of 2 to 3 times the width of the footing. Therefore, structural backfill (granular), insitu sandy clay, and insitu dense sand and gravel will all exist within the bearing capacity zone of influence. Because of differing cohesion and phi values of the various soils, the soil types will have different bearing capacities, and the actual value will be somewhere in the middle. Calculations were conducted for the sandy clay and structural backfill soils. A value of 2,000 pounds/square foot was selected for the design of the upper wall. A bearing failure would likely “concentrate” in the sandy clay soil, as it is the weakest soil type. A value of 2,000 pounds/square foot is likely conservative, but not excessively so. A settlement analysis of the upper wall was not conducted because the settlement will be negligible. The concrete wall is replacing an equivalent volume of soil. While concrete has a higher unit weight than soil (150 pounds per cubic foot as opposed to 119 pounds per cubic foot), the resulting stress increase will be very small. The clay soils in Napa are slightly overconsolidated. The stress increase caused by the upper wall will produce a stress lower than the preconsolidation pressure. Below the preconsolidation pressure, the recompression coefficient (C_r) is used instead of the coefficient of consolidation (C_c) when calculating settlements. Since C_r is always at least one order of magnitude less than C_c and the stress increase is very small, settlement will be negligible.

6.4. Global Stability. The computer program UTEXAS4, developed by Dr. Stephen Wright, was used to evaluate the global stability of the dual-wall system upstream of Fifth Street. A “composite section”, consisting of all the worst-case conditions, was used in the analysis. The soil profile upstream of Third Street was used, as this profile contains only 8 feet of the upper dense (strong) clayey sand and gravel layer, in addition to two (weak) fat clay layers that are not present south of Third Street. The free wall height of the upper wall is relatively constant (6 to 6.5 feet) throughout the project area. The maximum free wall height of the lower wall (about 10 feet) occurs at the northern end of the project area, and the two maximum free wall heights were used. The river bottom elevations at Third Street were used, as the river bottom elevation at the northern end of the project is shallow (Napa Creek instead of the Napa River). Analyses were conducted for end-of-construction, long term with no flood, long term with a flood, and long term with an earthquake and no flood conditions. Rapid drawdown analysis was not conducted because rapid drawdown conditions will not develop in this project as stated in paragraph 6.2.4, Wall Loadings/Design. For the long term with earthquake analysis, a seismic coefficient of 0.15 was used as per the SGDM. Failure surfaces are shown in Appendix 8. Calculated factors of safety are given in Table 3. No Corps minimum requirements exist for global slope stability of retaining walls, but Table 2 lists the Corps minimum factors of safety for sliding stability at the base of inland floodwalls and for flood-control levees for comparison. For global stability, long term with an earthquake is the most critical situation.

Table 2. Results of Slope Stability Analysis

Condition	F.S. (Calculated)	Minimum F.S. (Base Sliding)	Minimum F.S. (Flood Control Levee)
End of Construction	1.89	1.33	1.3
Long Term	2.65	1.5	1.4
Long Term w/Flood	4.80	1.5	1.4
Long Term w/Earthquake	1.22	1.1	None Listed (1.1 Typically Used)

Because the long term with earthquake is the most critical condition, that condition was used to evaluate the effect of the shallow-foundation surcharge of Downtown Joes on global stability. The long term with earthquake analysis was repeated to determine the maximum building surcharge that would result in a factor of safety of 1.1. A surcharge of 2,200 pounds per square foot produced a factor of safety of 1.10. According to MGE, a building of the size and type of Downtown Joe’s would typically have a surcharge load of about 1,000 pounds per square foot (Appendix 9). The long-term analysis was also conducted with a 2,200 pounds per square foot building surcharge, resulting in a factor of safety of 2.11. Therefore the global stability of the dual-wall system is not a concern.

7. Terrace Excavation. A marsh plain terrace, with a slope varying between 6H:1V and 4H:1V, will be excavated on the riverside of the soldier pile wall. The terrace will be excavated out into the river until the excavation line intersects with the existing river bottom. The purpose of the terrace is to provide additional channel capacity for flood flows. Some excavated material will be used as fill during construction. The remaining material will be placed in the Ghisletta disposal site. To protect the toe of the soldier pile wall from scour, riprap will be placed over the entire wall length.

8. Dewatering. Dewatering is a major concern for this project. Dewatering system design is the responsibility of the Contractor. The contract specifications require the Contractor to submit a dewatering plan for Government approval prior to construction. Based on the geotechnical explorations, groundwater will likely be encountered somewhere between elevations +1 and –2 feet NGVD. The water elevation of the Napa River varies from about +3.75 feet NGVD at high tide to about –2.84 feet NGVD at low tide. Upstream of approximate station 2+80 along the soldier pile wall layout line, the bottom of the lower wall is at elevation +1 foot NGVD. The only exception is at Downtown Joes, where the bottom of the lower wall is at elevation –4 feet NGVD for a distance of approximately 116 lineal feet due a depression in the existing ground surface in that area. It is anticipated limited dewatering measures will be required upstream of station 2+80. The marsh plane terrace is about 25 to 30 feet wide over most of this area. Dewatering in this area may be accomplished by temporarily piling excavated soil on the waterside end of the marshplane terrace to keep the river flows out, supplemented by the use of pumps and/or a seal course (a thin layer of concrete as per Caltrans standard specification 51-1.10) in the base of the excavation if necessary to control groundwater infiltration. Groundwater control for the CIDH pile boreholes is discussed in paragraph 6.2.8, Pile Specification. Downstream of station 2+80, dewatering will require a significant effort. A temporary cofferdam will likely be required to keep the river water out of the lower wall excavation. Possible methods for cofferdam construction are a sheet pile wall, a soldier pile wall with excavated H-piles and wooden lagging, or a deep soil mixed wall below ground with H-piles sticking above ground and wooden lagging between the piles. For construction of both the Third and First Street bridges, sheet pile cofferdams were used to construct the bridge piers. According to City of Napa Public Works personnel (Appendix 9), the contractors used vibratory hammers to install the sheet piles, except for a few piles at the First Street bridge where a diesel drop hammer had to be used after refusal with the vibratory hammer. It is well documented that vibratory hammers produce lower vibrations than drop hammers (References 12 and 13). According to several references (Figure 11 in Reference 8, Chapter 8 in Reference 12, and Figure 7 in Reference 13), cosmetic cracking in buildings typically will not occur if the peak particle velocities in the soil at the building site are less than 3 inches per second. Reference 12 discusses a British study where both sheet and H-piles were installed using both drop and vibratory hammers through a moderately dense sand layer only 1.6 feet away from a brick wall. The maximum particle velocity was measured as 2.6 inches per second at the brick wall and the wall was not damaged in any way. For this project, a temporary cofferdam on the waterside of the soldier pile wall will be about 25 feet away from the Napa Mill buildings. According to Reference 12, pile driving vibrations dissipate fairly quickly. The dewatering specifications will allow the use of vibratory hammers only to install sheet pile walls, and the particle velocities at the buildings must be kept below 1 inch per second. Unfortunately, vibrations typically become bothersome to humans at a velocity of about 0.3 inches per second, so the perception of people within the Napa Mill buildings of excessive vibrations will occur prior to any building damage occurring. While a sheetpile temporary cofferdam will keep out water from the riverside, some groundwater infiltration from the landside will occur. Because the upper sand and gravel layer is so dense and has a high fines content , it's permeability will likely be low for that soil type, reducing groundwater infiltration. Pumps and/or seal courses may be sufficient to dewater the excavation from the landside. If not, a short sheetpile wall or shallow wellpoints may be required.

9. Constructability. Numerous constructability concerns exist for this contract. A major concern is the presence of debris and possible boulders in the subsurface. Some of the borings drilled in 2003 encountered refusal and had to be terminated or moved over 5 feet due to the obstacles. When drilling for the CIDH piles, a high-powered drill rig and a strong drill bit must be used. If refusal is encountered, a small-diameter test or pilot hole should be drilled through the object to determine exactly what the object is and how deep it extends. Very shallow obstacles can be excavated out and replaced with aggregate base course or concrete. Deeper obstacles, if encountered, will be assessed on a case-by-case basis. Objects can often be broken up with special equipment, such as down-the-hole hammers and churn drills (Reference 10). The Contractor must have equipment to perform these operations on-site or readily obtainable (within 36 hours to avoid long and costly construction delays). If an obstacle cannot be broken up and drilled through, the pile will have to be relocated along the wall alignment. This could entail replacing one pile with two piles, with the new piles on either side of the planned pile. Changing pile locations and/or number of piles would necessitate a redesign of the reinforcing bars that connect the pile to the structural concrete of the wall. The contract bid sheet contains one bid item for 24-inch CIDH piles and another bid item for 36-inch CIDH piles with the quantities (measured in length) as shown on the contract drawings. There are also optional bid items for additional length of 24-inch CIDH piles and additional length of 36-inch CIDH piles. These optional items can be exercised if unexpected conditions during construction necessitate a redesign of a portion of the foundation. Access is another major concern. Construction equipment cannot access the area behind the Napa Mill, so construction will have to be from a barge or a temporary platform constructed over the Napa River. Access is also limited at Downtown Joes. Heavy construction close to existing structures could cause excessive movements or vibrations that could lead to damage. Geotechnical instrumentation will be installed and monitored during construction to ensure that adjacent buildings and structures are not damaged (see Geotechnical Instrumentation section). The contract specifications require the Contractor to have a geotechnical engineer to monitor the geotechnical site conditions during construction. The Corps geotechnical designer will also make frequent site visits to monitor conditions. The Corps geotechnical designer will be on site all the time during the installation of the CIDH piles for Wall Type A, and will visit the site once or twice a week during installation of the remaining CIDH piles. The Corps will also have the structural designer under contract during construction to handle structural issues which arise during construction.

10. Geotechnical Instrumentation. Because heavy construction activities will occur very close to existing structures, geotechnical instrumentation is required to monitor the structures to prevent damage. The primary concern is settlement/tilting of the structures, with vibrations from construction equipment as a secondary concern. The following structures will be monitored during construction: all the buildings and patios at the Napa Mill, the Third Street Bridge abutment, Downtown Joes, the Semorile building, and the counterfort concrete retaining wall. Due to access limitations and the fact that some of the structures are on pile foundations, monitoring for settlement/tilting will be conducted by the use of surveyed settlement monuments and/or beam tiltmeters. Some monuments will be installed in the ground near the structures, and some will be attached to the structures themselves. All instrumentation must be installed and an initial set of readings taken prior to the beginning of all other construction activities. Vibrations will be monitored daily while construction is occurring near a specific structure. Settlement/tilting will be monitored daily when construction is occurring near a specific structure and for a week afterwards, and once a week thereafter for 2 months. The contract specifications require the Contractor to submit an instrumentation plan for Government approval prior to construction.

10.1. Inclinometers. Due to limited space and access at most buildings inclinometers will most likely not be used to monitor ground movement due to excavation for the wall. Recommend that at least two inclinometers be installed along the slope between the Oberon Building and the River. Inclinometers should indicate any slope movement caused by excavation.

10.2. Observation Wells. Observation Wells should be installed adjacent to buildings at the Napa Mill where dewatering is anticipated. Dewatering can lower the groundwater table and induce settlement. Recommend that at least three observation wells be placed around each building in the dewatering area. At least two observation wells should be located on the slope between the building and river or one on each side of the building. At least one observation well should be installed along the other end of the building. This should provide a picture of groundwater elevations, and potential settlement, under the entire building.

10.3. Survey Monuments. Recommend survey monuments be placed on the abutment of the Third Street Bridge, Fourth Street, and the patio connecting the Angele Building and Napa River Inn Suites. The abutment should have at least one monument, Forth Street should have at least one monument, and the patio should have at least three monuments. In addition survey monuments can be placed on buildings to monitor structural movement. Recommend at least two monuments on each side of the building facing the river or one on each side the building adjacent to the wall of the building facing the river.

10.4. Beam Tiltmeters. Beam Tiltmeters may be used in addition or in place of survey monuments for structural rotation (vertical beam tiltmeters) or observation wells for settlement (horizontal tiltmeters). Recommend that two vertical beam tiltmeters be placed on each side the face of the building and one horizontal beam tiltmeter be placed at the side of the building facing the river.

10.5. Vibration Monitors. Recommend at least one vibration monitor be placed on each building.

11. References.

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PHOTOS

Photo 1. View of the southern end of the Napa Mill complex from across the Napa River.

Photo 2. View of the Napa Mill complex from across the Napa River.

Photo 3. View of the southern end of the Napa Mill complex. The soldier pile wall will tie into existing ground (station 0+00) in this area.

Photo 4. View of the patio behind the Napa Mill complex.

Photo 5. View of cantilevered portion of the historic Napa Mill building. The soldier pile wall will be 10 feet to the left of this building wall.

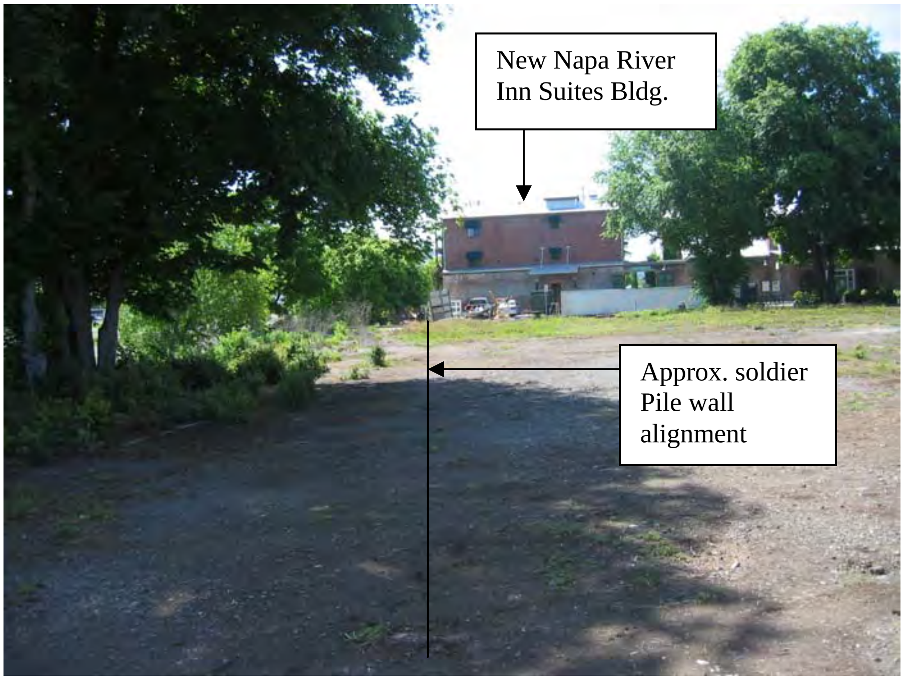


Photo 6. View of vacant lot from Fourth Street to the Napa Mill, looking south (downstream)

Photo 7. View of parking lot at Fourth Street, looking north (upstream) towards the Third Street Bridge (in background).

Photo 8. View of Veterans Park, looking north (upstream) towards Downtown Joes (in background).

Photo 9. View of east wall of Downtown Joes, looking south. Soldier pile wall alignment is about 25 feet to the left of the building wall.

Photo 10. View of parking lot north of Downtown Joes, looking northeast. Upstream end of soldier pile wall ties into the corner near the center of the photograph.

FIGURES

APPENDIX 1: RANEY GEOTECHNICAL BORING LOGS AND CPT DATA

GRAPH	SYMBOL	DESCRIPTION	MAJOR DIVISIONS		
◌	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES	CLEAN GRAVELS WITH LESS THAN 5% FINES	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	COARSE GRAINED SOILS MORE THAN 50% LARGER THAN NO. 200 SIEVE
◌	GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES			
▥	GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES	GRAVELS WITH MORE THAN 12% FINES		
▨	GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES			
░	SW	WELL GRADED SANDS, GRAVELLY SANDS	CLEAN SANDS WITH LESS THAN 5% FINES	SANDS AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	
░	SP	POORLY GRADED SANDS, GRAVELLY SANDS			
▥	SM	SILTY SANDS, SAND-SILT MIXTURES	SANDS WITH MORE THAN 12% FINES		
▨	SC	CLAYEY SANDS, SAND-CLAY MIXTURES			
▥	ML	INORGANIC SILTS, ROCK FLOUR, OR CLAYEY SILTS WITH SLIGHT PLASTICITY	LIQUID LIMIT LESS THAN 50	SILTS AND CLAYS	FINE GRAINED SOILS MORE THAN 50% SMALLER THAN NO. 200 SIEVE
▨	CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS			
▥	OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY			
▥	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS SILTS, ELASTIC SILTS	LIQUID LIMIT GREATER THAN 50	SILTS AND CLAYS	
▧	CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS			
▧	OH	ORGANIC CLAYS AND ORGANIC SILTS OF MEDIUM TO HIGH PLASTICITY			
▦	PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENT	HIGHLY ORGANIC SOILS		

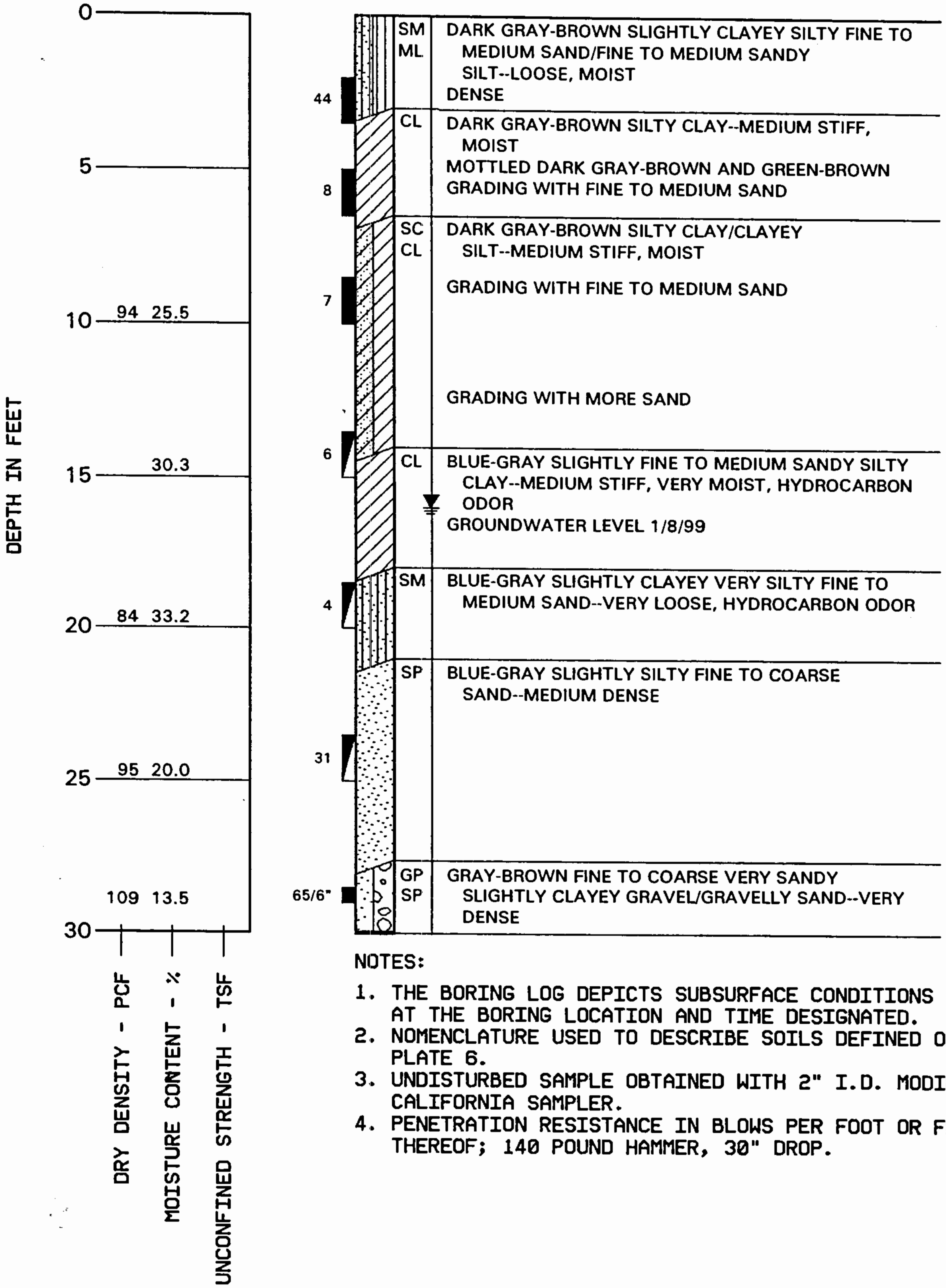
PROJECT NUMBER: 424-025
PLATE NUMBER: 6

UNIFIED SOIL CLASSIFICATION SYSTEM

Raney Geotechnical Inc

BORING 1

DRILLED: 1/8/99



SM ML	DARK GRAY-BROWN SLIGHTLY CLAYEY SILTY FINE TO MEDIUM SAND/FINE TO MEDIUM SANDY SILT--LOOSE, MOIST DENSE
CL	DARK GRAY-BROWN SILTY CLAY--MEDIUM STIFF, MOIST MOTTLED DARK GRAY-BROWN AND GREEN-BROWN GRADING WITH FINE TO MEDIUM SAND
SC CL	DARK GRAY-BROWN SILTY CLAY/CLAYEY SILT--MEDIUM STIFF, MOIST GRADING WITH FINE TO MEDIUM SAND GRADING WITH MORE SAND
CL	BLUE-GRAY SLIGHTLY FINE TO MEDIUM SANDY SILTY CLAY--MEDIUM STIFF, VERY MOIST, HYDROCARBON ODOR GROUNDWATER LEVEL 1/8/99
SM	BLUE-GRAY SLIGHTLY CLAYEY VERY SILTY FINE TO MEDIUM SAND--VERY LOOSE, HYDROCARBON ODOR
SP	BLUE-GRAY SLIGHTLY SILTY FINE TO COARSE SAND--MEDIUM DENSE
GP SP	GRAY-BROWN FINE TO COARSE VERY SANDY SLIGHTLY CLAYEY GRAVEL/GRAVELLY SAND--VERY DENSE

NOTES:

1. THE BORING LOG DEPICTS SUBSURFACE CONDITIONS ONLY AT THE BORING LOCATION AND TIME DESIGNATED.
2. NOMENCLATURE USED TO DESCRIBE SOILS DEFINED ON PLATE 6.
3. UNDISTURBED SAMPLE OBTAINED WITH 2" I.D. MODIFIED CALIFORNIA SAMPLER.
4. PENETRATION RESISTANCE IN BLOWS PER FOOT OR FRACTION THEREOF; 140 POUND HAMMER, 30" DROP.

LOG OF BORING

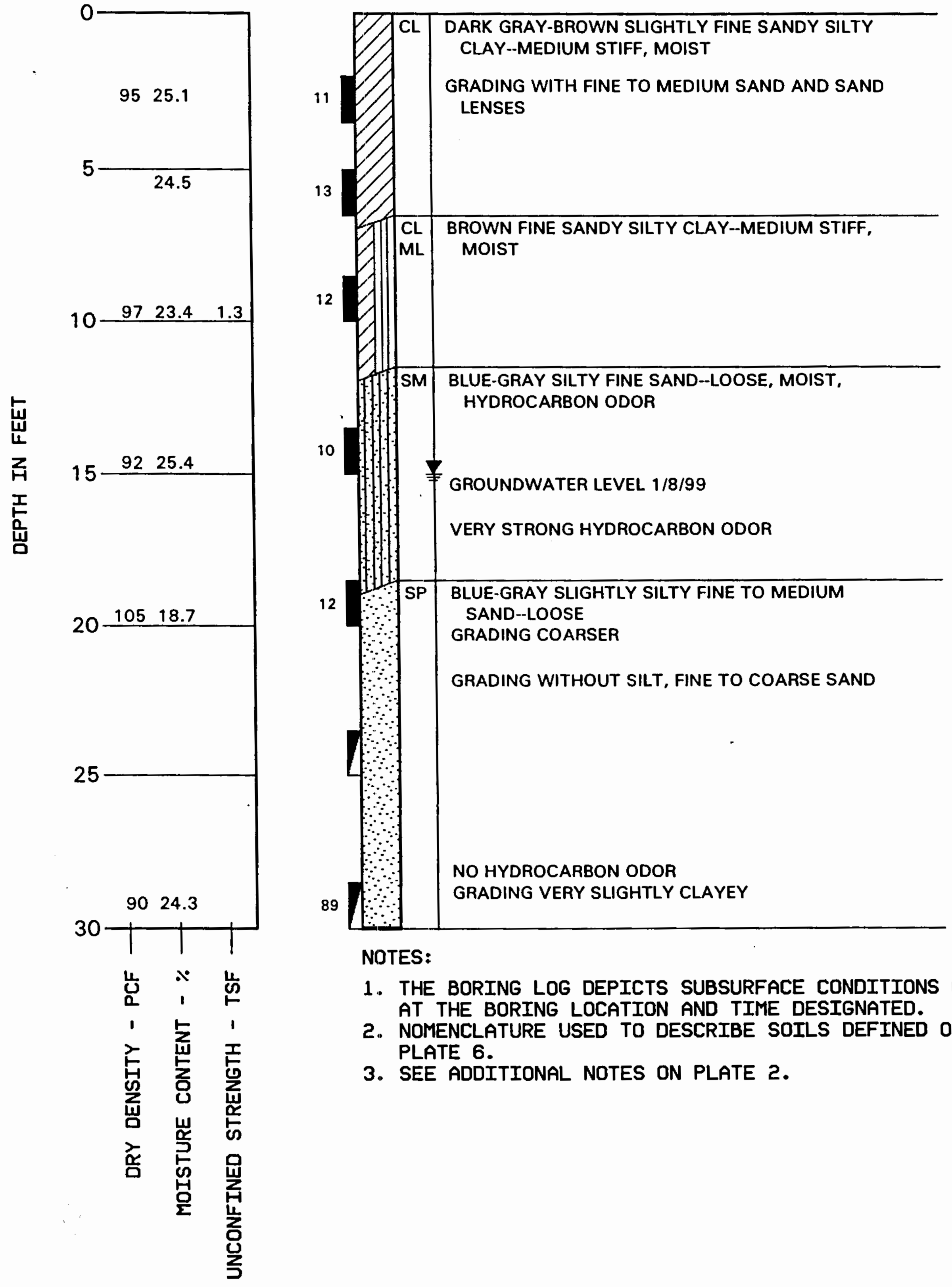
Raney Geotechnical Inc

PROJECT NUMBER: 424-025
PLATE NUMBER: 2
DRAWN BY: GC DATE: 2/1/99
CHECKED BY: DATE: 6/22/99

PROJECT NUMBER: 424-025
PLATE NUMBER: 3
DRAWN BY: GC DATE: 2/1/99
CHECKED BY: DATE: 6/22/99

BORING 2

DRILLED: 1/8/99



DARK GRAY-BROWN SLIGHTLY FINE SANDY SILTY CLAY--MEDIUM STIFF, MOIST

GRADING WITH FINE TO MEDIUM SAND AND SAND LENSES

BROWN FINE SANDY SILTY CLAY--MEDIUM STIFF, MOIST

BLUE-GRAY SILTY FINE SAND--LOOSE, MOIST, HYDROCARBON ODOR

GROUNDWATER LEVEL 1/8/99

VERY STRONG HYDROCARBON ODOR

BLUE-GRAY SLIGHTLY SILTY FINE TO MEDIUM SAND--LOOSE
GRADING COARSER

GRADING WITHOUT SILT, FINE TO COARSE SAND

NO HYDROCARBON ODOR
GRADING VERY SLIGHTLY CLAYEY

NOTES:

1. THE BORING LOG DEPICTS SUBSURFACE CONDITIONS ONLY AT THE BORING LOCATION AND TIME DESIGNATED.
2. NOMENCLATURE USED TO DESCRIBE SOILS DEFINED ON PLATE 6.
3. SEE ADDITIONAL NOTES ON PLATE 2.

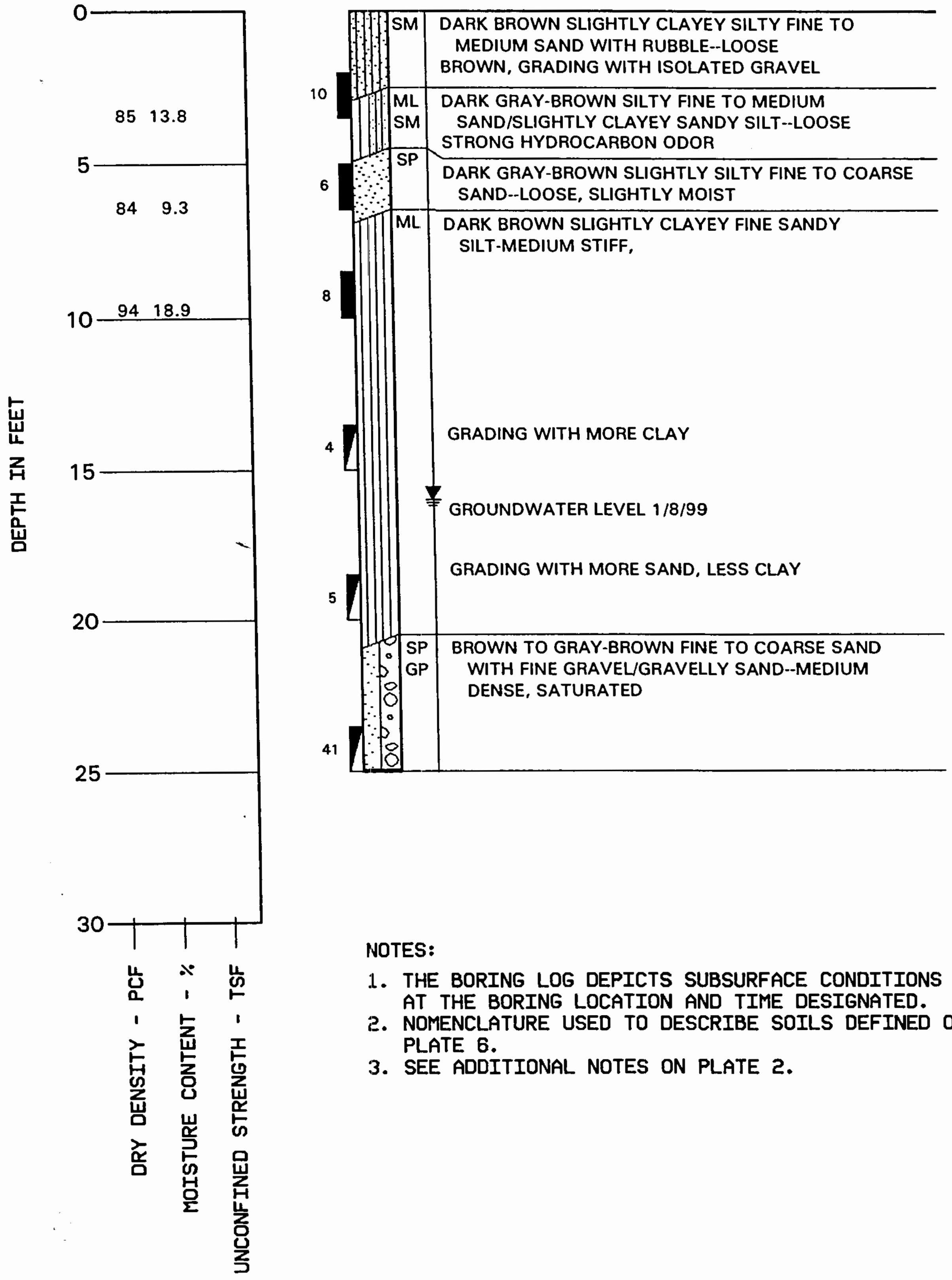
LOG OF BORING

Raney Geotechnical Inc

PROJECT NUMBER: 424-025
PLATE NUMBER: 4
DRAWN BY: GC DATE: 2/1/99
CHECKED BY: DATE: 6/22/M.

BORING 3

DRILLED: 1/8/99



SM DARK BROWN SLIGHTLY CLAYEY SILTY FINE TO MEDIUM SAND WITH RUBBLE--LOOSE
BROWN, GRADING WITH ISOLATED GRAVEL

ML/SM DARK GRAY-BROWN SILTY FINE TO MEDIUM SAND/SLIGHTLY CLAYEY SANDY SILT--LOOSE
STRONG HYDROCARBON ODOR

SP DARK GRAY-BROWN SLIGHTLY SILTY FINE TO COARSE SAND--LOOSE, SLIGHTLY MOIST

ML DARK BROWN SLIGHTLY CLAYEY FINE SANDY SILT-MEDIUM STIFF,

GRADING WITH MORE CLAY

GROUNDWATER LEVEL 1/8/99

GRADING WITH MORE SAND, LESS CLAY

SP/GP BROWN TO GRAY-BROWN FINE TO COARSE SAND WITH FINE GRAVEL/GRAVELLY SAND--MEDIUM DENSE, SATURATED

NOTES:

1. THE BORING LOG DEPICTS SUBSURFACE CONDITIONS ONLY AT THE BORING LOCATION AND TIME DESIGNATED.
2. NOMENCLATURE USED TO DESCRIBE SOILS DEFINED ON PLATE 6.
3. SEE ADDITIONAL NOTES ON PLATE 2.

LOG OF BORING

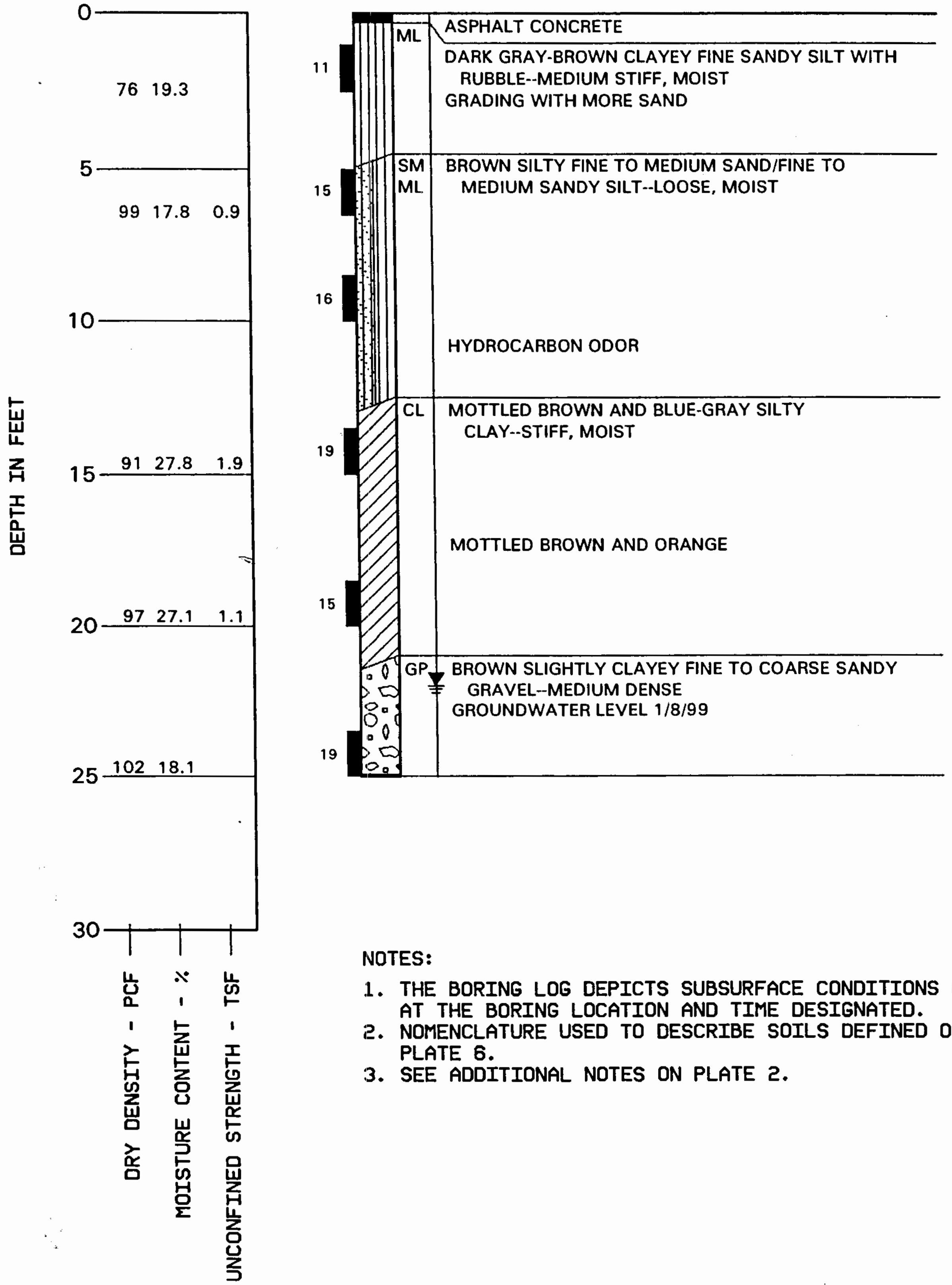
Raney Geotechnical Inc

PLATE 4

PROJECT NUMBER: 424-025
PLATE NUMBER: 5
DRAWN BY: GC DATE: 2/1/99
CHECKED BY: DATE: 6/22/99

BORING 4

DRILLED: 1/8/99

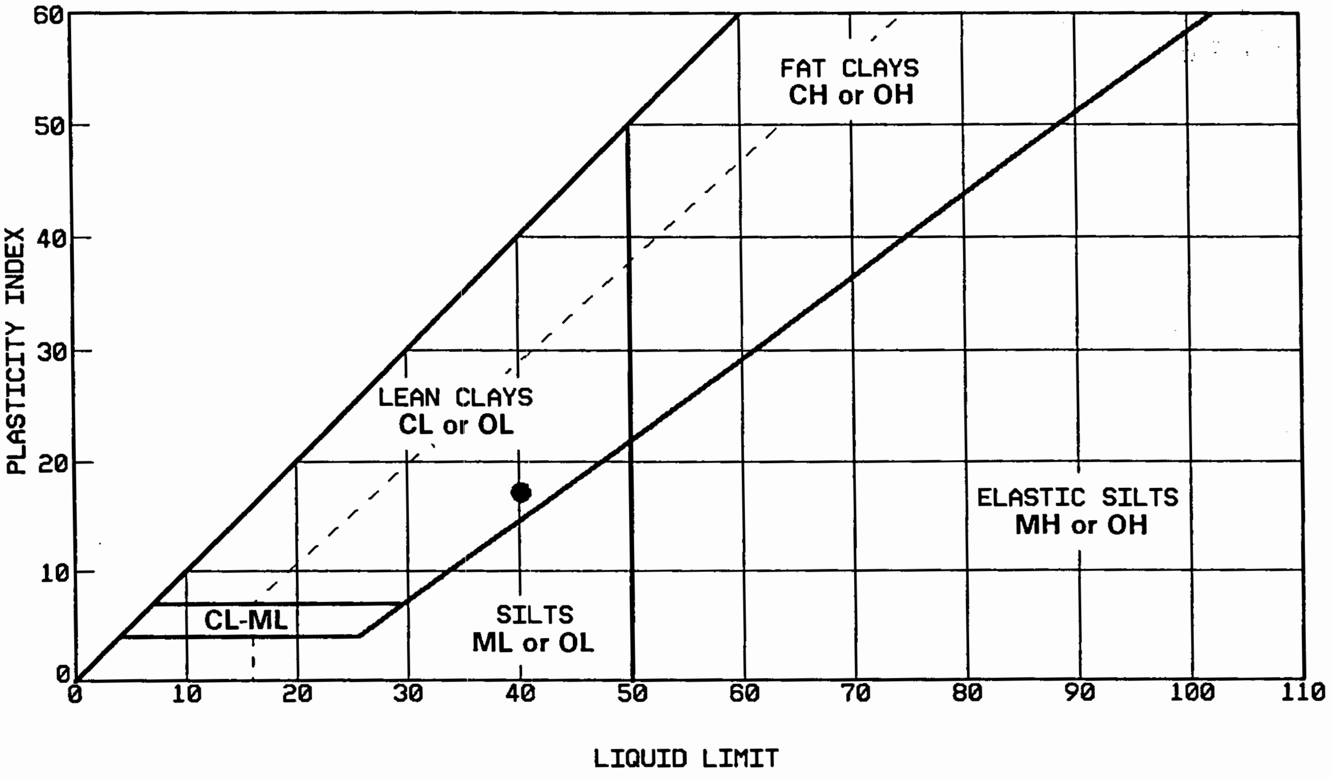


NOTES:

1. THE BORING LOG DEPICTS SUBSURFACE CONDITIONS ONLY AT THE BORING LOCATION AND TIME DESIGNATED.
2. NOMENCLATURE USED TO DESCRIBE SOILS DEFINED ON PLATE 6.
3. SEE ADDITIONAL NOTES ON PLATE 2.

LOG OF BORING

Raney Geotechnical Inc



CLASSIFICATION TEST RESULTS						
SYMBOL	SAMPLE LOCATION	DEPTH FEET	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	SOIL CLASSIFICATION
●	BORING 2	6.0	40	23	17	CL

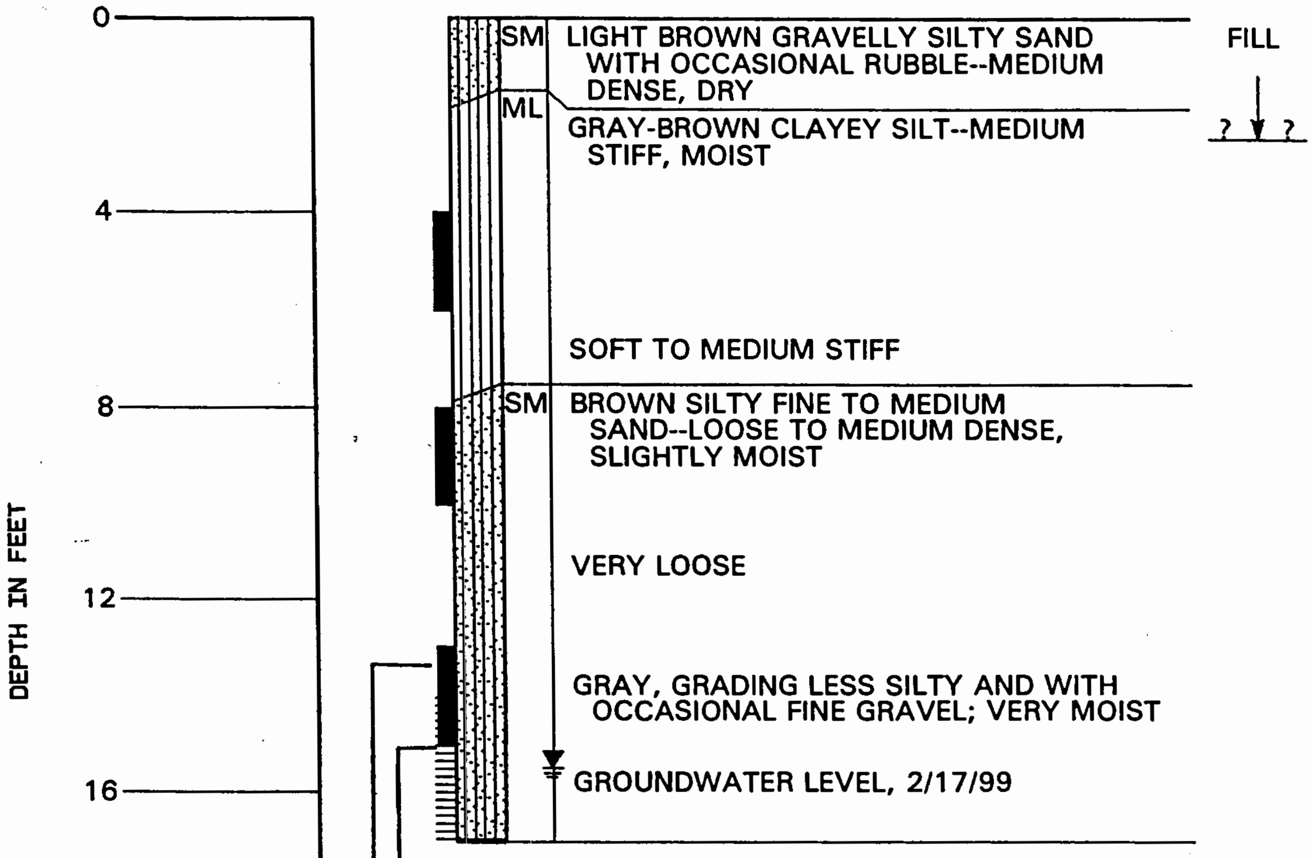
ATTERBERG LIMIT DATA

Raney Geotechnical Inc

PROJECT NUMBER: 424-005 DRAWN BY: GC DATE: 4/23/99
PLATE NUMBER: 6 CHECKED BY: DATE: 6/12/99.

BORING P1

DRILLED: 2/17/99



NOTES:

1. THE BORING LOG DEPICTS SUBSURFACE CONDITIONS ONLY AT THE BORING LOCATION AND TIME DESIGNATED.
2. NOMENCLATURE USED TO DESCRIBE SOILS DEFINED ON PLATE 19.
3. UNDISTURBED SOIL SAMPLE OBTAINED WITH DIRECT PUSH EQUIPMENT.
4. FREE GROUNDWATER MEASURED DURING DRILLING.
5. GRAB GROUNDWATER SAMPLE OBTAINED.

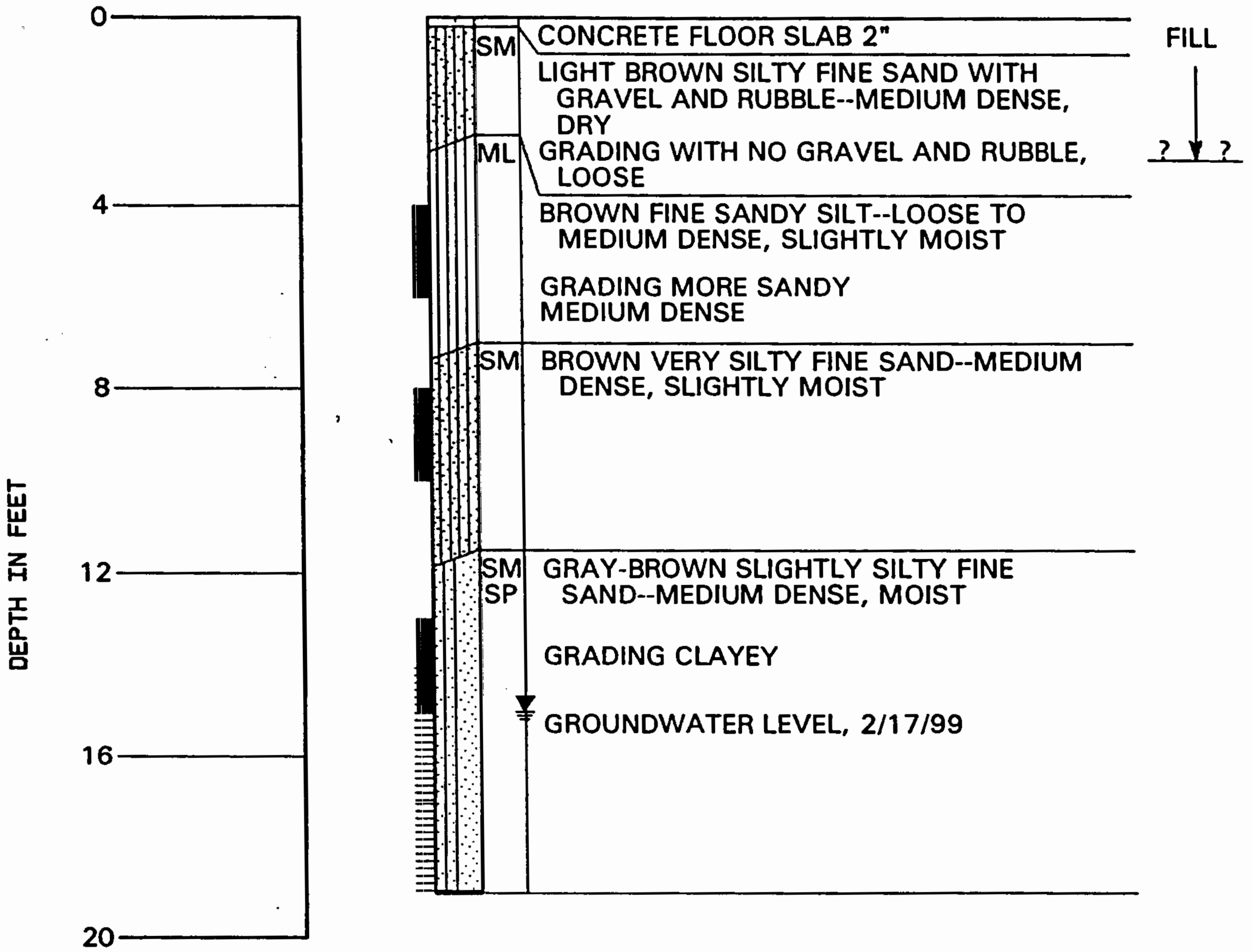
LOG OF BORING

Raney Geotechnical Inc

PLATE NUMBER: 7
CHECKED BY: DATE: 6/22/99

BORING P2

DRILLED: 2/17/99



CONCRETE FLOOR SLAB 2"

LIGHT BROWN SILTY FINE SAND WITH GRAVEL AND RUBBLE--MEDIUM DENSE, DRY

GRADING WITH NO GRAVEL AND RUBBLE, LOOSE

BROWN FINE SANDY SILT--LOOSE TO MEDIUM DENSE, SLIGHTLY MOIST

GRADING MORE SANDY MEDIUM DENSE

BROWN VERY SILTY FINE SAND--MEDIUM DENSE, SLIGHTLY MOIST

GRAY-BROWN SLIGHTLY SILTY FINE SAND--MEDIUM DENSE, MOIST

GRADING CLAYEY

GROUNDWATER LEVEL, 2/17/99

NOTES:

1. THE BORING LOG DEPICTS SUBSURFACE CONDITIONS ONLY AT THE BORING LOCATION AND TIME DESIGNATED.
2. NOMENCLATURE USED TO DESCRIBE SOILS DEFINED ON PLATE 19.
3. SEE ADDITIONAL NOTES ON PLATE 6.

LOG OF BORING

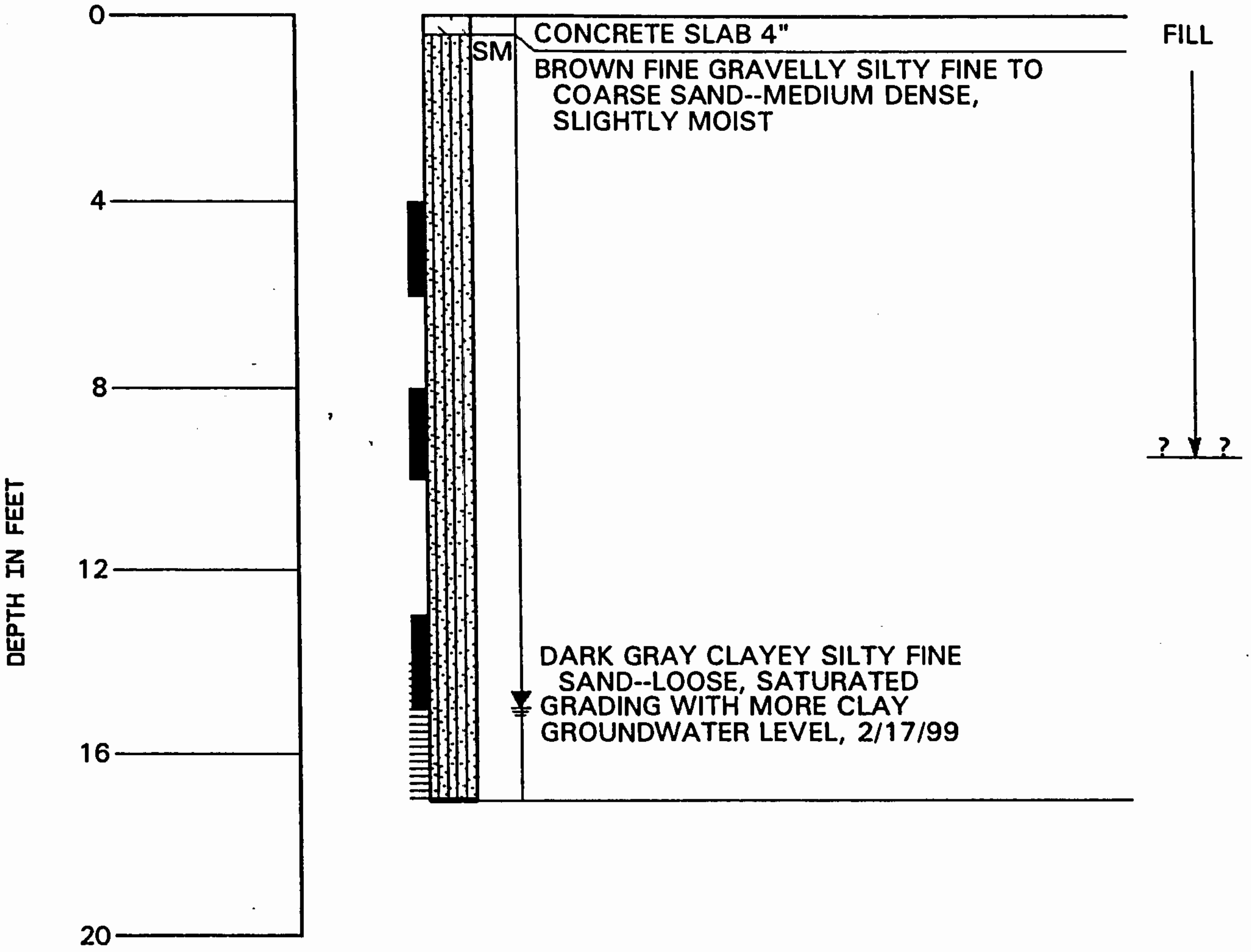
Raney Geotechnical Inc

PLATE 7

PROJECT NUMBER: 424-005
PLATE NUMBER: 9
DRAWN BY: GC DATE: 4/23/99
CHECKED BY: DATE: 6/22/99

BORING P4

DRILLED: 2/17/99



NOTES:

1. THE BORING LOG DEPICTS SUBSURFACE CONDITIONS ONLY AT THE BORING LOCATION AND TIME DESIGNATED.
2. NOMENCLATURE USED TO DESCRIBE SOILS DEFINED ON PLATE 19.
3. SEE ADDITIONAL NOTES ON PLATE 6.

LOG OF BORING

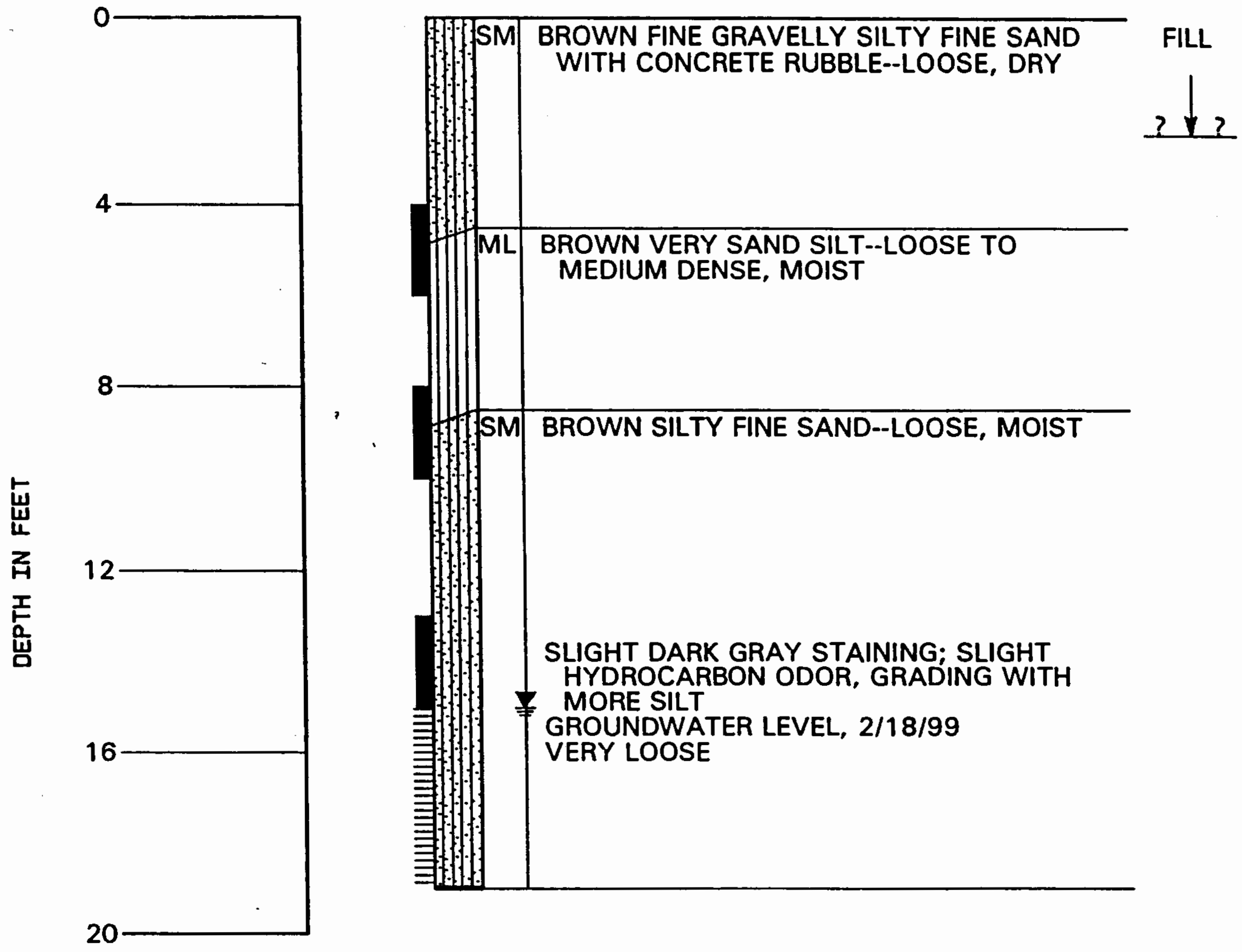
Raney Geotechnical Inc

PLATE 9

PROJECT NUMBER: 424-005
PLATE NUMBER: 11
DRAWN BY: GC DATE: 4/23/99
CHECKED BY: DATE: 4/23/99.

BORING P6

DRILLED: 2/18/99



SM BROWN FINE GRAVELLY SILTY FINE SAND WITH CONCRETE RUBBLE--LOOSE, DRY

ML BROWN VERY SAND SILT--LOOSE TO MEDIUM DENSE, MOIST

SM BROWN SILTY FINE SAND--LOOSE, MOIST

SLIGHT DARK GRAY STAINING; SLIGHT HYDROCARBON ODOR, GRADING WITH MORE SILT
GROUNDWATER LEVEL, 2/18/99
VERY LOOSE

NOTES:

1. THE BORING LOG DEPICTS SUBSURFACE CONDITIONS ONLY AT THE BORING LOCATION AND TIME DESIGNATED.
2. NOMENCLATURE USED TO DESCRIBE SOILS DEFINED ON PLATE 19.
3. SEE ADDITIONAL NOTES ON PLATE 6.

LOG OF BORING

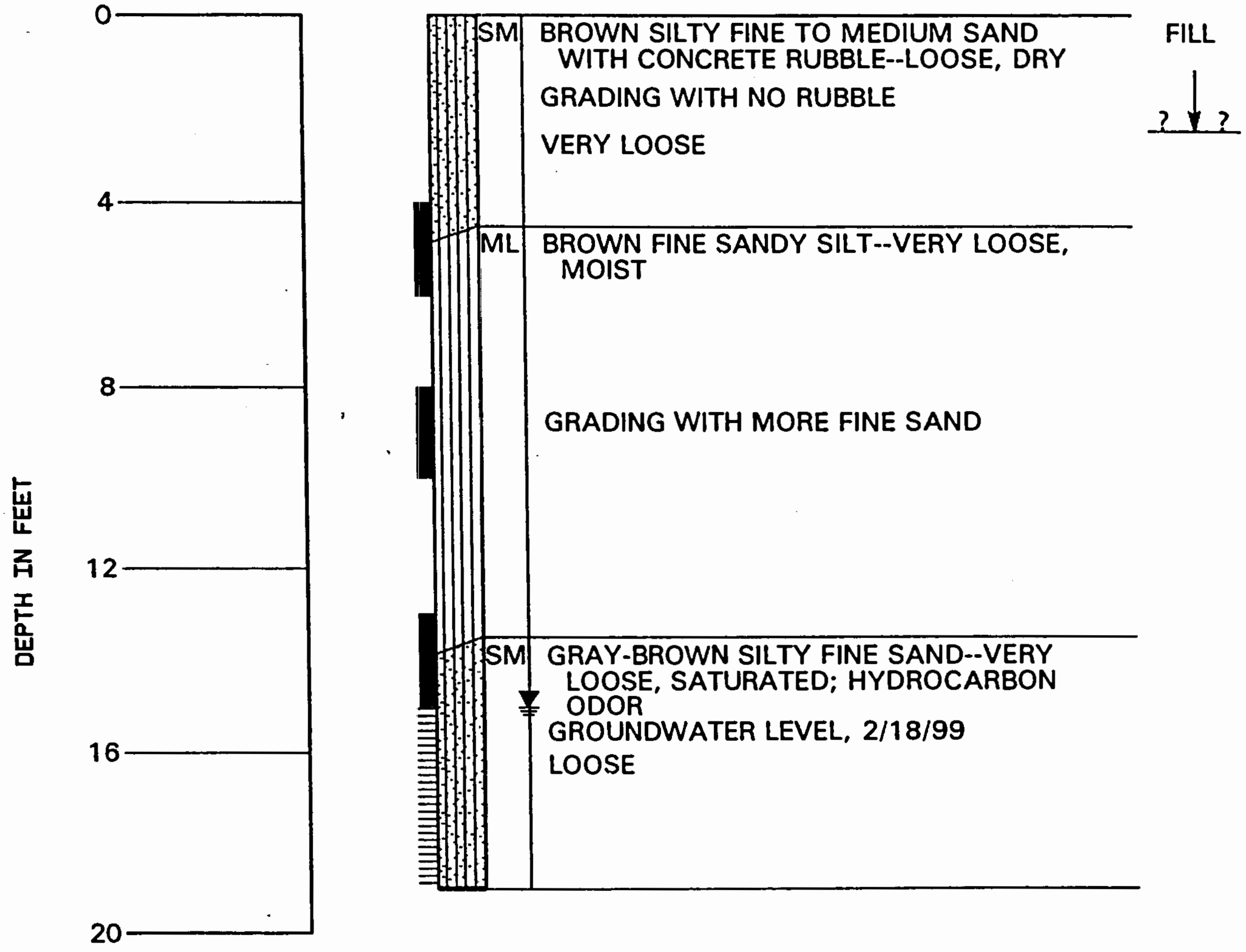
Raney Geotechnical Inc

PLATE 11

PROJECT NUMBER: 424-005 DRAWN BY: GC DATE: 4/23/99
PLATE NUMBER: 12 CHECKED BY: DATE: 4/23/99

BORING P7

DRILLED: 2/18/99



BROWN SILTY FINE TO MEDIUM SAND WITH CONCRETE RUBBLE--LOOSE, DRY

GRADING WITH NO RUBBLE

VERY LOOSE

BROWN FINE SANDY SILT--VERY LOOSE, MOIST

GRADING WITH MORE FINE SAND

GRAY-BROWN SILTY FINE SAND--VERY LOOSE, SATURATED; HYDROCARBON ODOR

GROUNDWATER LEVEL, 2/18/99

LOOSE

NOTES:

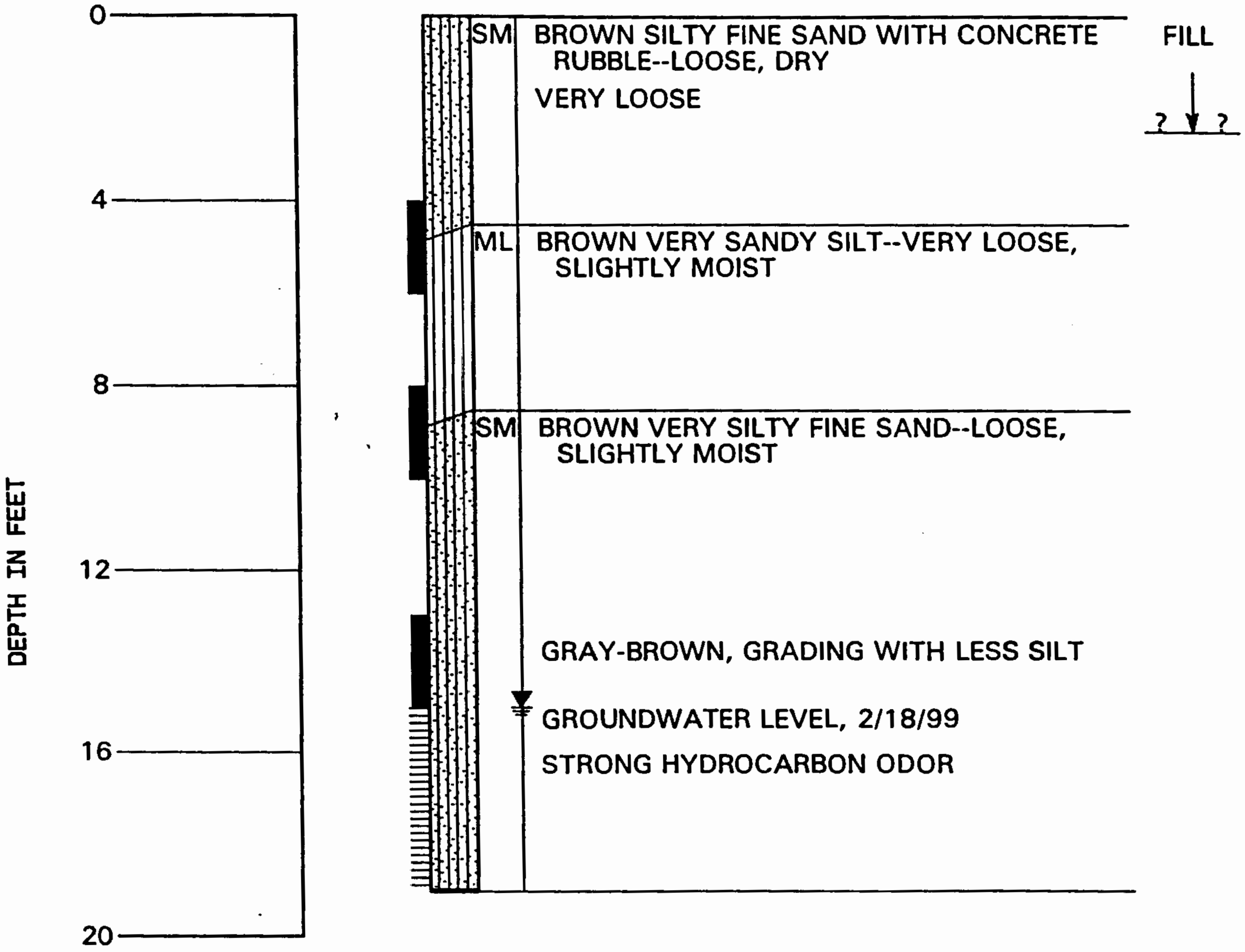
1. THE BORING LOG DEPICTS SUBSURFACE CONDITIONS ONLY AT THE BORING LOCATION AND TIME DESIGNATED.
2. NOMENCLATURE USED TO DESCRIBE SOILS DEFINED ON PLATE 19.
3. SEE ADDITIONAL NOTES ON PLATE 6.

LOG OF BORING

Raney Geotechnical Inc

BORING P8

DRILLED: 2/18/99



BROWN SILTY FINE SAND WITH CONCRETE RUBBLE--LOOSE, DRY

VERY LOOSE

BROWN VERY SANDY SILT--VERY LOOSE, SLIGHTLY MOIST

BROWN VERY SILTY FINE SAND--LOOSE, SLIGHTLY MOIST

GRAY-BROWN, GRADING WITH LESS SILT

GROUNDWATER LEVEL, 2/18/99

STRONG HYDROCARBON ODOR

NOTES:

1. THE BORING LOG DEPICTS SUBSURFACE CONDITIONS ONLY AT THE BORING LOCATION AND TIME DESIGNATED.
2. NOMENCLATURE USED TO DESCRIBE SOILS DEFINED ON PLATE 19.
3. SEE ADDITIONAL NOTES ON PLATE 6.

PROJECT NUMBER: 424-005
DRAWN BY: GC DATE: 4/23/99
CHECKED BY: DATE:
PLATE NUMBER: 13

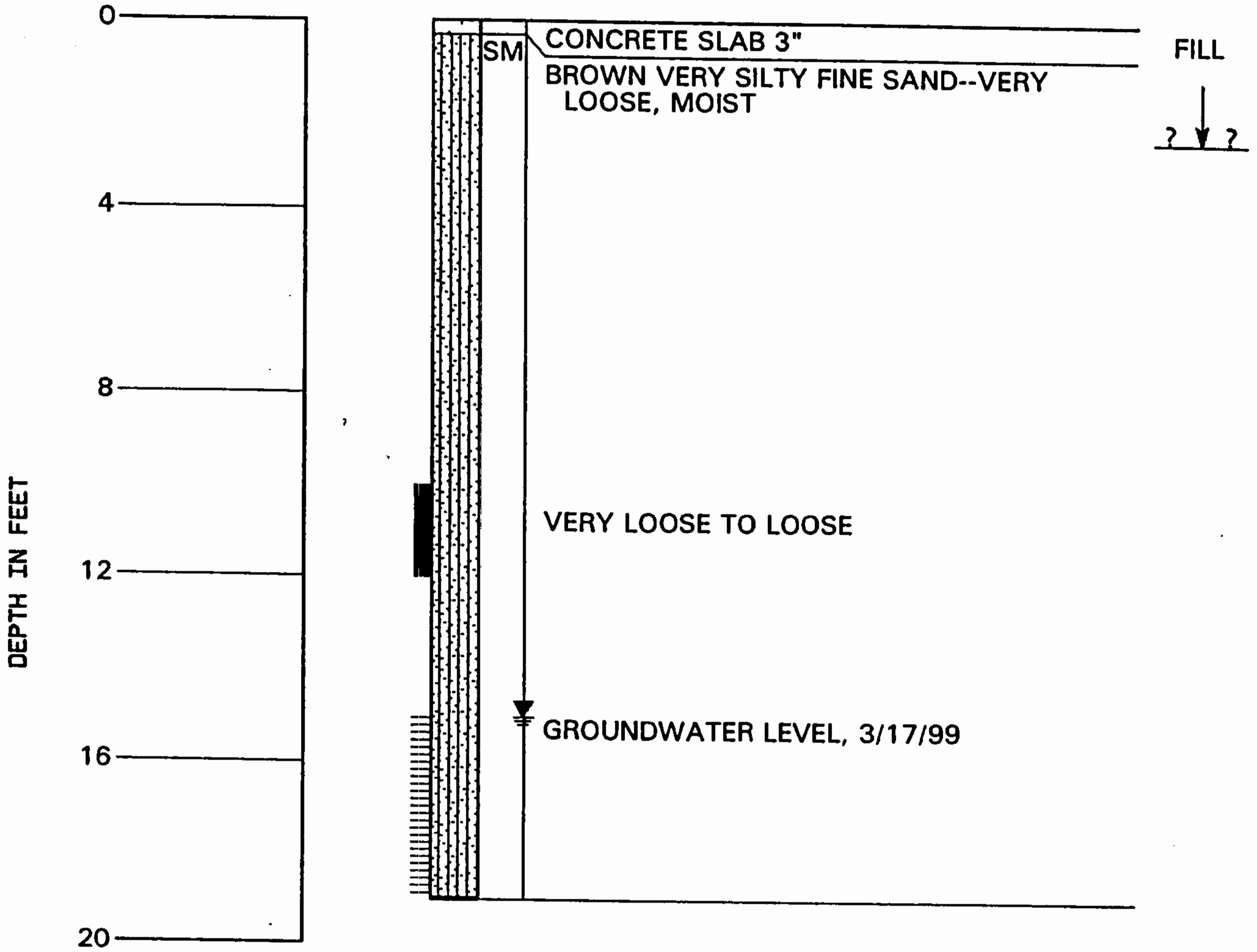
LOG OF BORING

Raney Geotechnical Inc

PLATE 13

BORING P10

DRILLED: 3/17/99



NOTES:

1. THE BORING LOG DEPICTS SUBSURFACE CONDITIONS ONLY AT THE BORING LOCATION AND TIME DESIGNATED.
2. NOMENCLATURE USED TO DESCRIBE SOILS DEFINED ON PLATE 19.
3. SEE ADDITIONAL NOTES ON PLATE 6.

LOG OF BORING

Raney Geotechnical Inc

PLATE 15

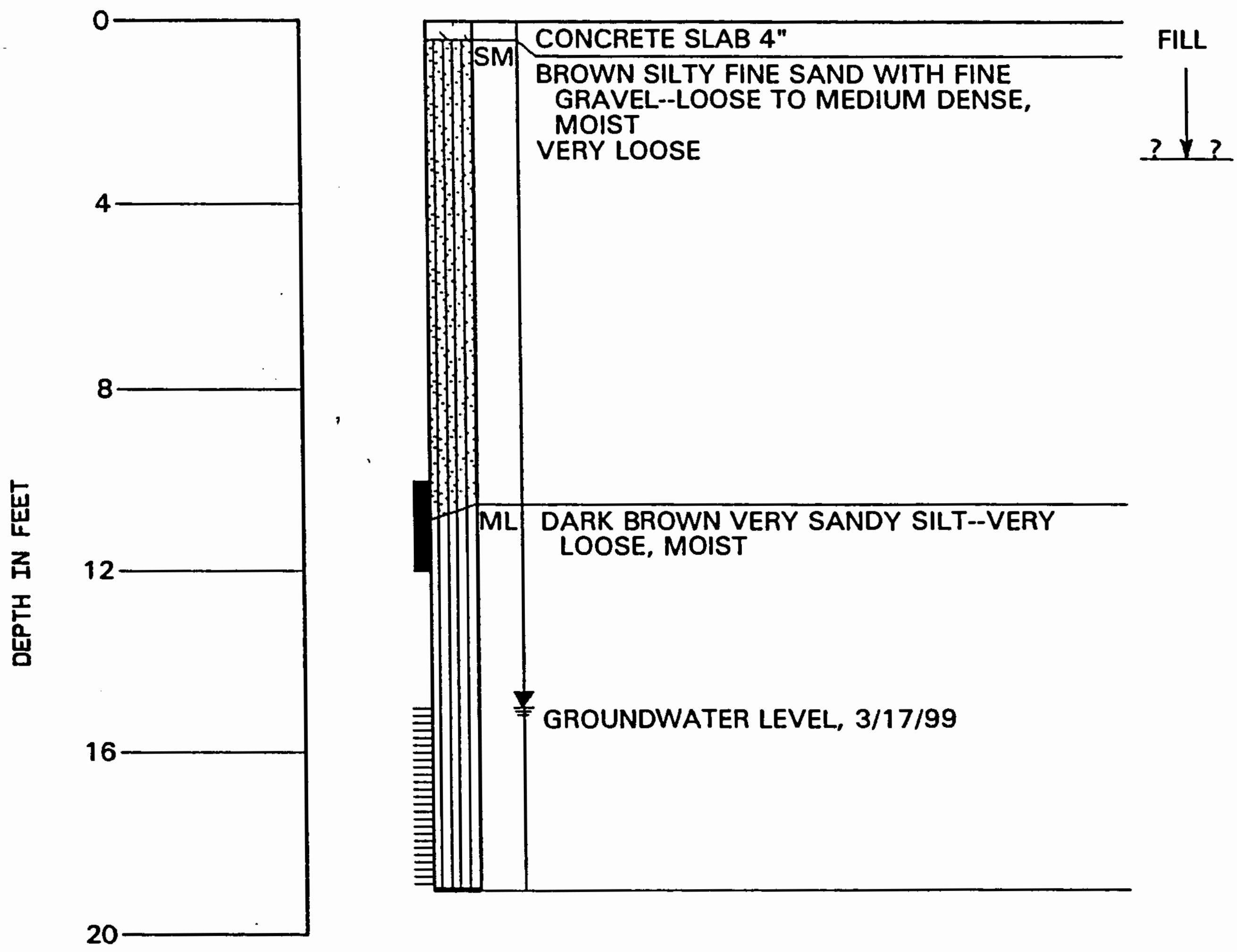
PROJECT NUMBER: 424-005 DRAWN BY: GC DATE: 4/23/99

PLATE NUMBER: 15 CHECKED BY: DATE: 6/22/99.

PROJECT NUMBER: 424-005
PLATE NUMBER: 16
DRAWN BY: GC DATE: 4/23/99
CHECKED BY: DATE: 4/22/99

BORING P11

DRILLED: 3/17/99



CONCRETE SLAB 4"

BROWN SILTY FINE SAND WITH FINE GRAVEL--LOOSE TO MEDIUM DENSE, MOIST
VERY LOOSE

DARK BROWN VERY SANDY SILT--VERY LOOSE, MOIST

GROUNDWATER LEVEL, 3/17/99

NOTES:

1. THE BORING LOG DEPICTS SUBSURFACE CONDITIONS ONLY AT THE BORING LOCATION AND TIME DESIGNATED.
2. NOMENCLATURE USED TO DESCRIBE SOILS DEFINED ON PLATE 19.
3. SEE ADDITIONAL NOTES ON PLATE 6.

LOG OF BORING

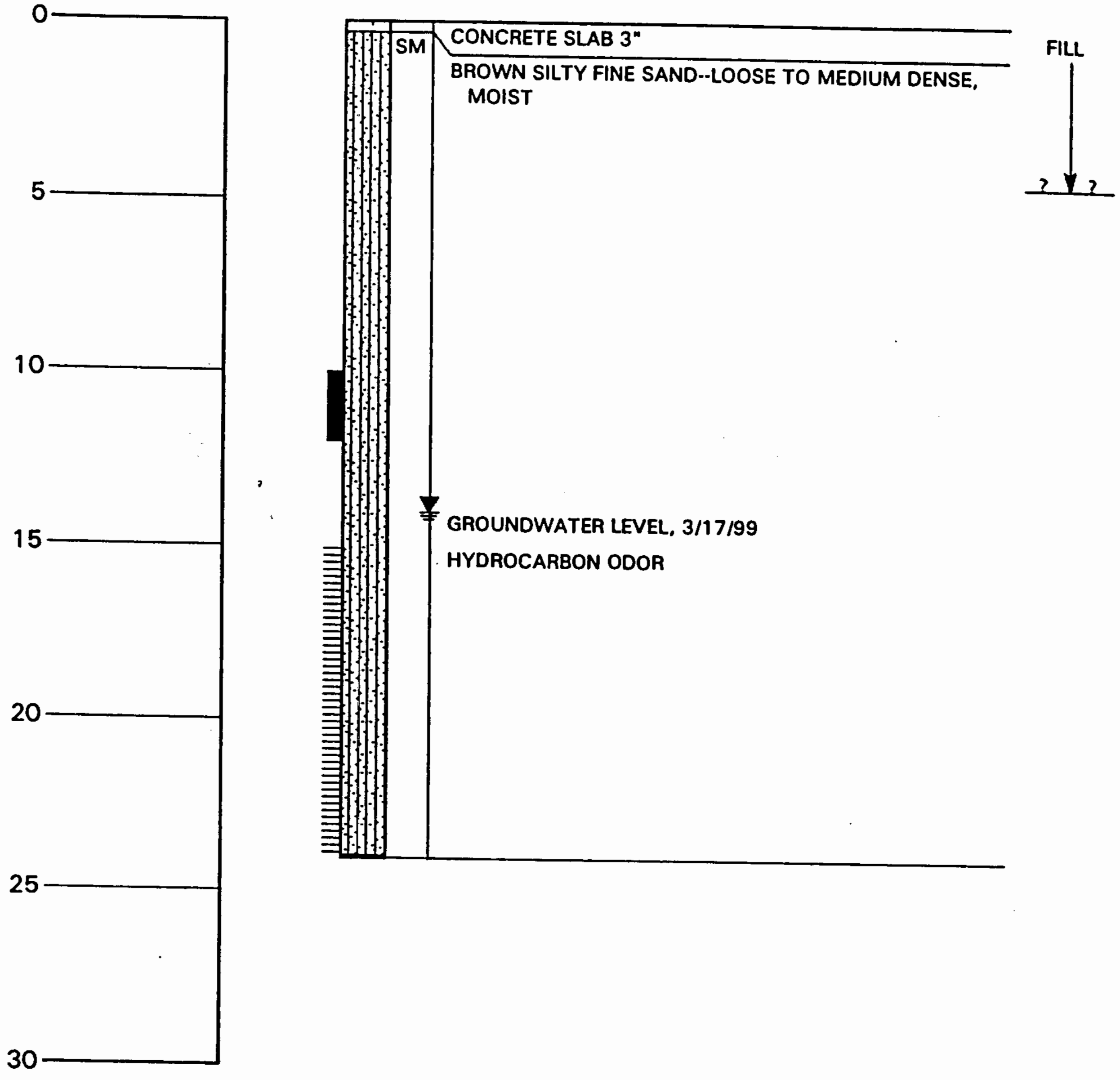
Raney Geotechnical Inc

PLATE 16

PROJECT NUMBER: 424-005 DRAWN BY: GC DATE: 4/23/99
PLATE NUMBER: 17 CHECKED BY: DATE: 6/2/99.

BORING P12

DRILLED: 3/17/99



NOTES:

1. THE BORING LOG DEPICTS SUBSURFACE CONDITIONS ONLY AT THE BORING LOCATION AND TIME DESIGNATED.
2. NOMENCLATURE USED TO DESCRIBE SOILS DEFINED ON PLATE 19.
3. SEE ADDITIONAL NOTES ON PLATE 6.

LOG OF BORING

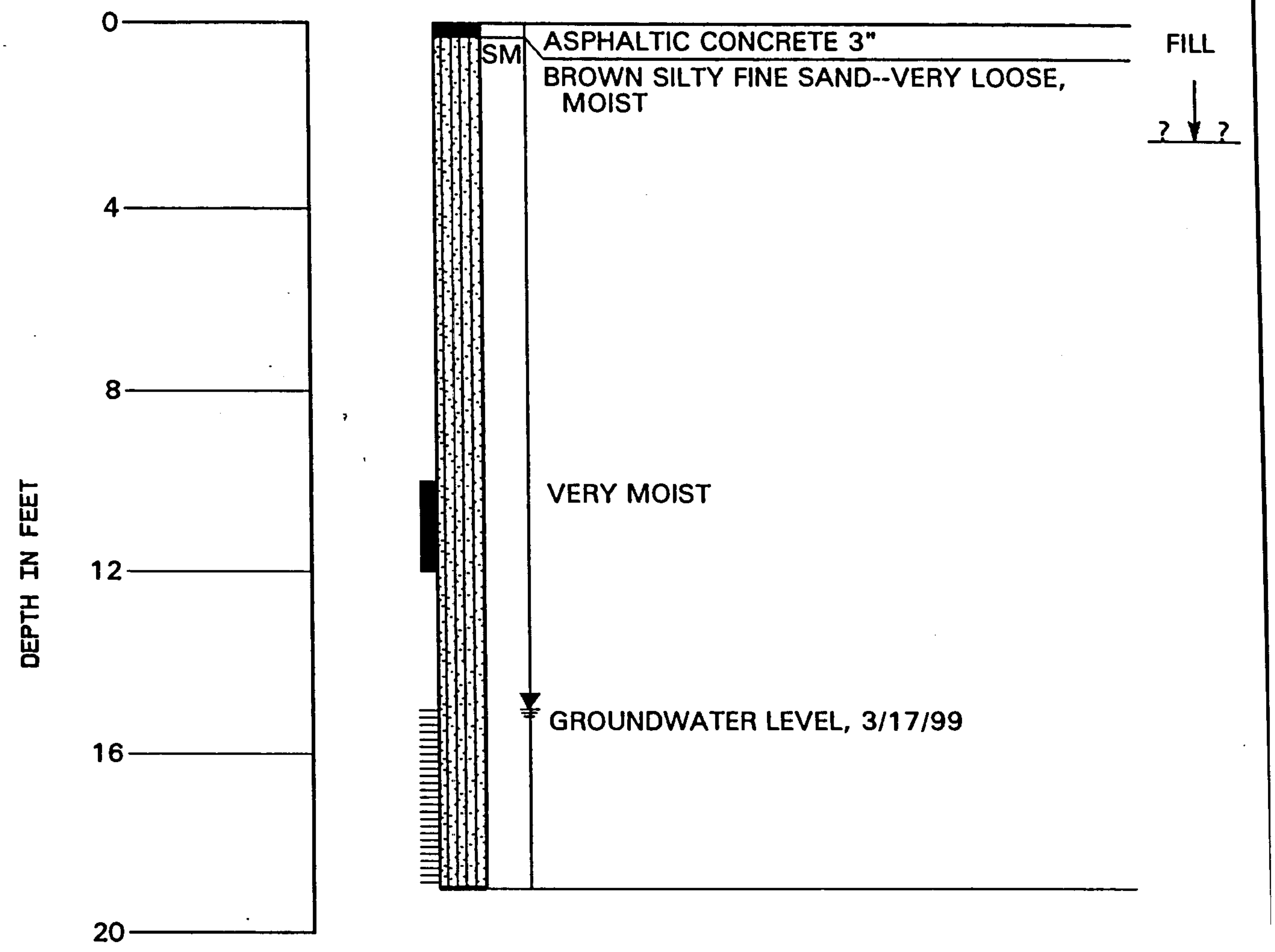
Raney Geotechnical Inc

PLATE 17

PROJECT NUMBER: 424-005 DRAWN BY: GC DATE: 4/23/99
PLATE NUMBER: 18 CHECKED BY: DATE:

BORING P13

DRILLED: 3/17/99



NOTES:

1. THE BORING LOG DEPICTS SUBSURFACE CONDITIONS ONLY AT THE BORING LOCATION AND TIME DESIGNATED.
2. NOMENCLATURE USED TO DESCRIBE SOILS DEFINED ON PLATE 19.
3. SEE ADDITIONAL NOTES ON PLATE 6.

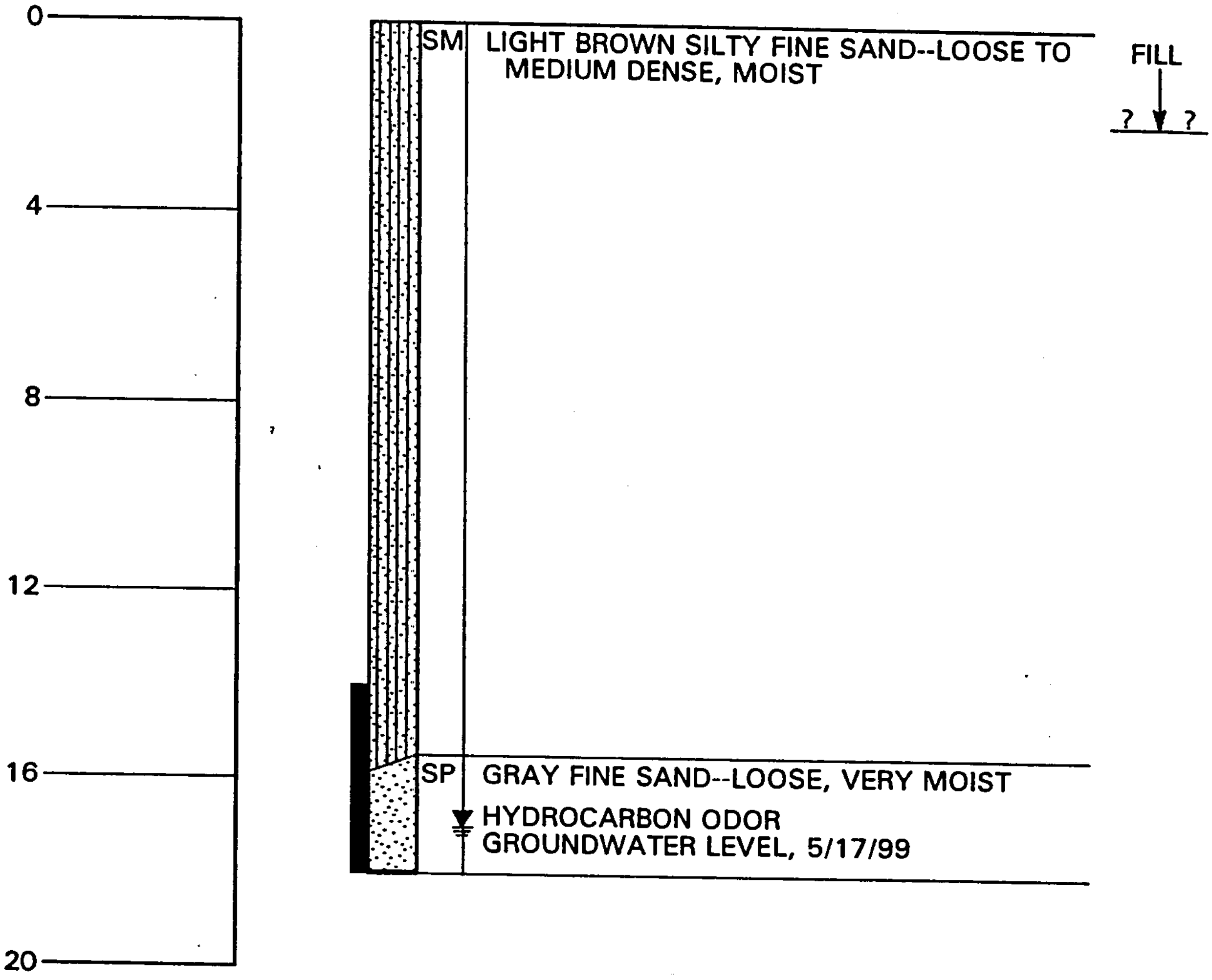
LOG OF BORING

Raney Geotechnical Inc

PLATE 18

BORING P14

DRILLED: 5/17/99



NOTES:

1. THE BORING LOG DEPICTS SUBSURFACE CONDITIONS ONLY AT THE BORING LOCATION AND TIME DESIGNATED.
2. NOMENCLATURE USED TO DESCRIBE SOILS DEFINED ON PLATE 25.
3. SEE ADDITIONAL NOTES ON PLATE 6.

PROJECT NUMBER: 424-005 DRAWN BY: GC DATE: 5/19/99

PLATE NUMBER: 19 CHECKED BY: DATE: 5/22/99.

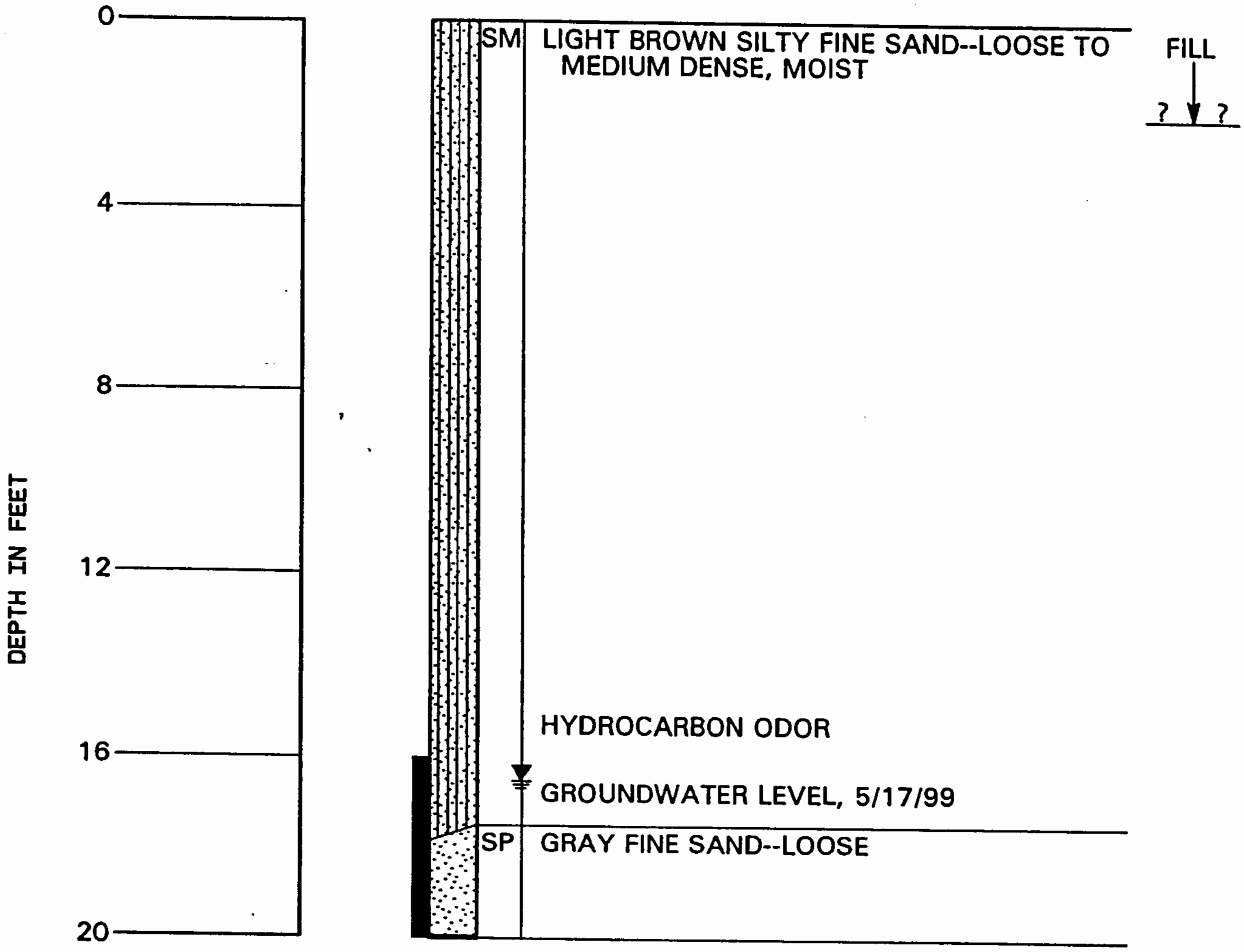
LOG OF BORING

Raney Geotechnical Inc

PROJECT NUMBER: 424-005 DRAWN BY: GC DATE: 5/19/99
PLATE NUMBER: 20 CHECKED BY: DATE: 6/22/99.

BORING P15

DRILLED: 5/17/99



NOTES:

1. THE BORING LOG DEPICTS SUBSURFACE CONDITIONS ONLY AT THE BORING LOCATION AND TIME DESIGNATED.
2. NOMENCLATURE USED TO DESCRIBE SOILS DEFINED ON PLATE 25.
3. SEE ADDITIONAL NOTES ON PLATE 6.

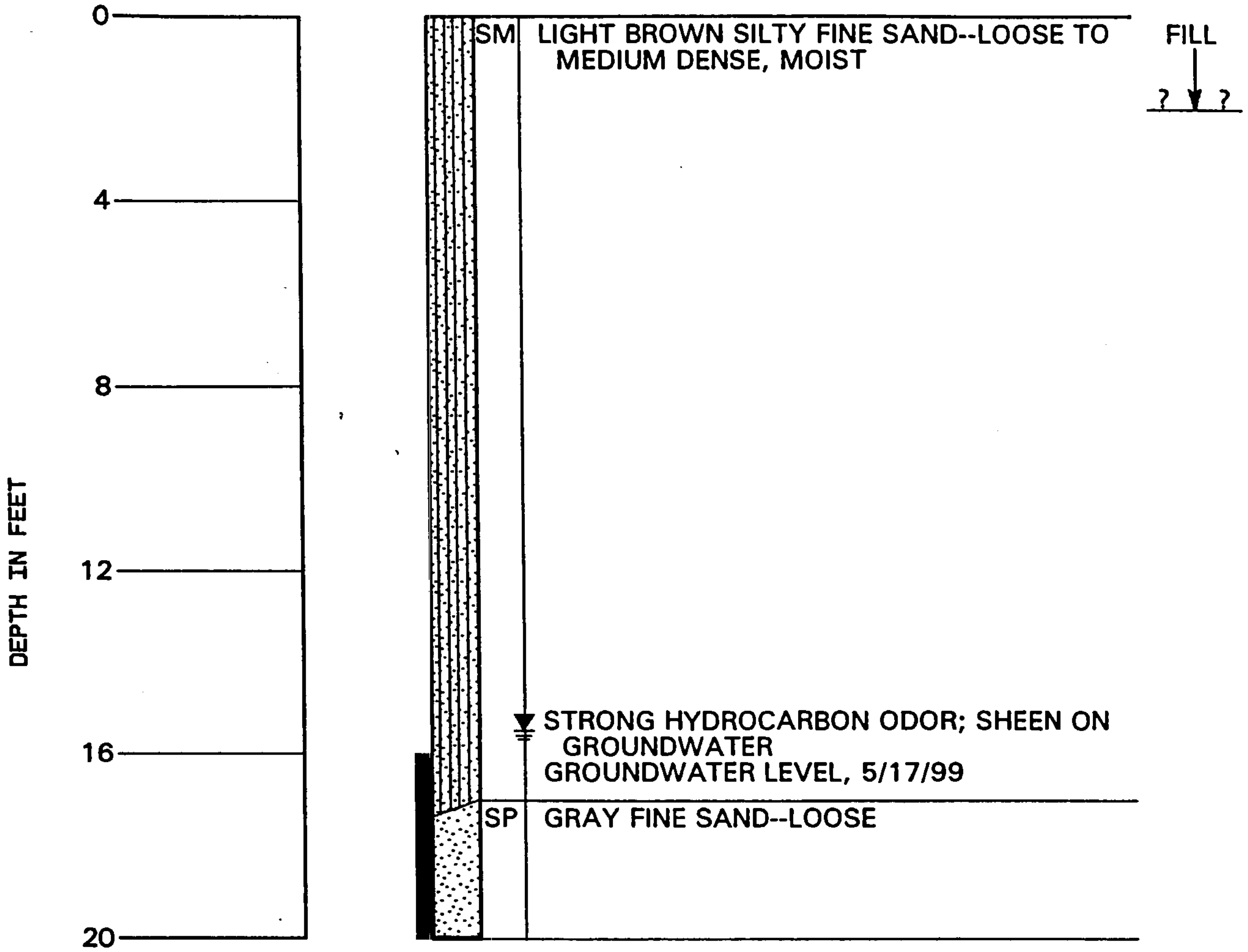
LOG OF BORING

Raney Geotechnical Inc

PROJECT NUMBER: 424-005
DRAWN BY: GC DATE: 5/13/99
PLATE NUMBER: 21
CHECKED BY: DATE: 6/12/99

BORING P16

DRILLED: 5/17/99



NOTES:

1. THE BORING LOG DEPICTS SUBSURFACE CONDITIONS ONLY AT THE BORING LOCATION AND TIME DESIGNATED.
2. NOMENCLATURE USED TO DESCRIBE SOILS DEFINED ON PLATE 25.
3. SEE ADDITIONAL NOTES ON PLATE 6.

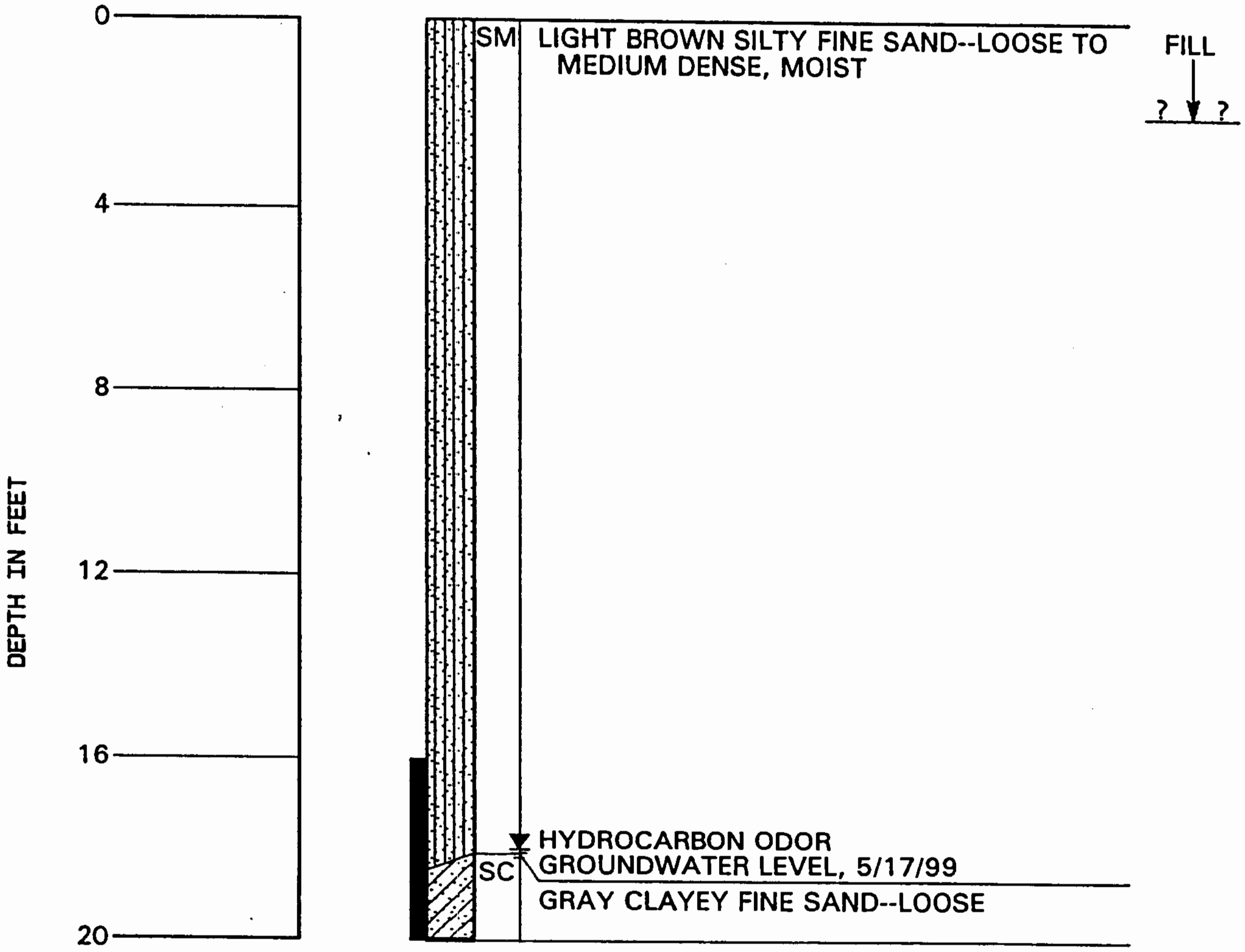
LOG OF BORING

Raney Geotechnical Inc

PLATE 21

BORING P17

DRILLED: 5/17/99



NOTES:

1. THE BORING LOG DEPICTS SUBSURFACE CONDITIONS ONLY AT THE BORING LOCATION AND TIME DESIGNATED.
2. NOMENCLATURE USED TO DESCRIBE SOILS DEFINED ON PLATE 25.
3. SEE ADDITIONAL NOTES ON PLATE 6.

LOG OF BORING

Raney Geotechnical Inc

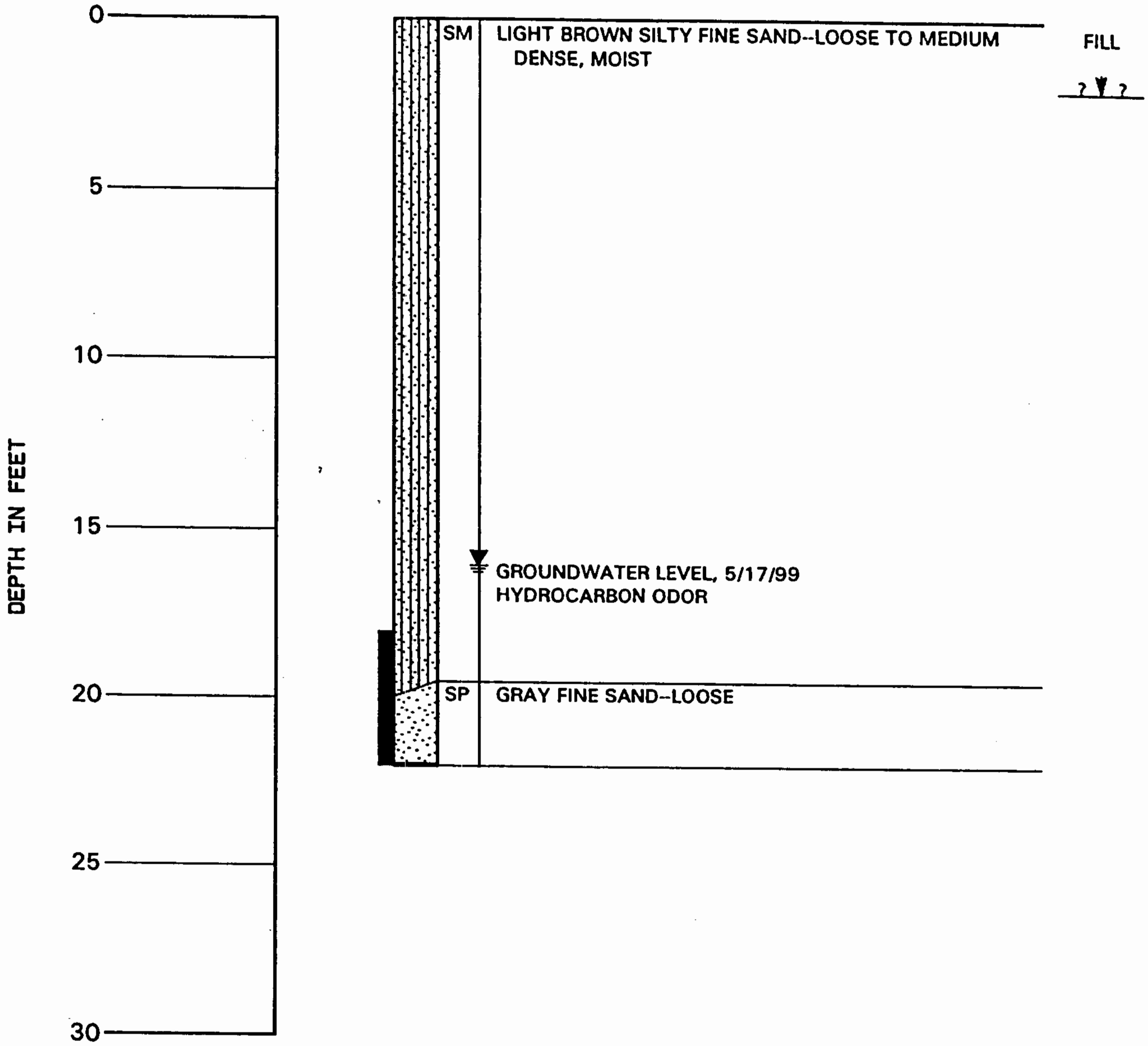
PLATE 22

PLATE NUMBER: 22
CHECKED BY:
DATE: 6/22/99

PLATE NUMBER: 23 CHECKED BY: DATE: 6/22/99

BORING P18

DRILLED: 5/17/99



NOTES:

1. THE BORING LOG DEPICTS SUBSURFACE CONDITIONS ONLY AT THE BORING LOCATION AND TIME DESIGNATED.
2. NOMENCLATURE USED TO DESCRIBE SOILS DEFINED ON PLATE 25.
3. SEE ADDITIONAL NOTES ON PLATE 6.

LOG OF BORING

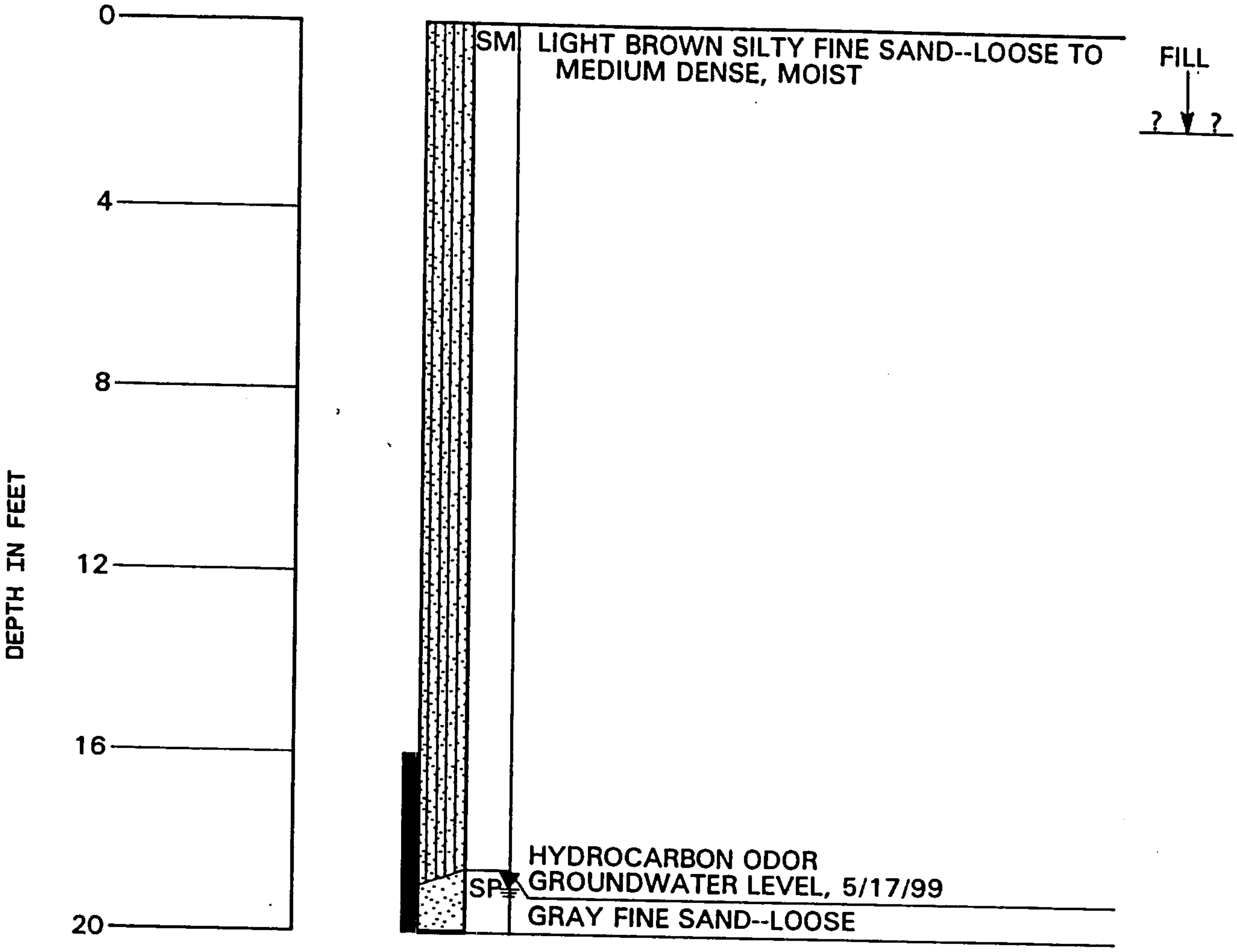
Raney Geotechnical Inc

PLATE NUMBER: 24

CHECKED BY: DATE: 6/22/9

BORING P19

DRILLED: 5/17/99



NOTES:

1. THE BORING LOG DEPICTS SUBSURFACE CONDITIONS ONLY AT THE BORING LOCATION AND TIME DESIGNATED.
2. NOMENCLATURE USED TO DESCRIBE SOILS DEFINED ON PLATE 25.
3. SEE ADDITIONAL NOTES ON PLATE 6.

LOG OF BORING

Raney Geotechnical Inc

PROJECT: Napa River Flood Control Project
LOCATION: Napa CA
PROJ. NO.: COE-01

CPT NO.: CPT-94-2
DATE : 10-14-1994
ESTIMATED WT DEPTH: 20.17 feet

DEPTH (feet)	Qc (tsf)	Fs (tsf)	Rf (%)	SPT (N)	TotHzStr (ksf)	PHI (deg.)	SU (ksf)	SOIL BEHAVIOR TYPE	DENSITY RANGE (pcf)
1.00	24.70	0.67	2.7	12	0.12	----	2.90	Clayey SILT to Silty CLAY	''
1.50	28.90	0.42	1.5	12	0.18	----	3.39	Sandy SILT to Clayey SILT	''
2.00	45.39	0.76	1.7	18	0.25	----	5.33	''	130-140
2.50	41.39	0.82	2.0	17	0.32	----	4.85	''	''
3.00	45.09	0.75	1.7	15	0.38	45	----	Silty SAND to Sandy SILT	''
3.50	34.79	0.52	1.5	12	0.45	44	----	''	120-130
4.00	38.19	0.61	1.6	15	0.51	----	4.46	Sandy SILT to Clayey SILT	''
4.50	41.79	0.66	1.6	14	0.57	43	----	Silty SAND to Sandy SILT	''
5.00	54.89	0.86	1.6	18	0.63	44	----	''	''
5.50	63.49	0.96	1.5	21	0.70	44	----	''	130-140
6.00	63.39	0.89	1.4	21	0.76	44	----	''	120-130
6.50	78.39	1.11	1.4	26	0.83	44	----	''	''
7.00	70.99	1.27	1.8	24	0.89	44	----	''	130-140
7.50	67.09	1.29	1.9	22	0.96	43	----	''	''
8.00	67.69	1.85	2.7	27	1.03	----	7.90	Sandy SILT to Clayey SILT	''
8.50	61.49	1.39	2.3	20	1.10	42	----	Silty SAND to Sandy SILT	''
9.00	65.49	1.72	2.6	26	1.16	----	7.64	Sandy SILT to Clayey SILT	''
9.50	65.59	1.25	1.9	22	1.23	42	----	Silty SAND to Sandy SILT	''
10.00	37.79	0.84	2.2	13	1.30	38	----	''	''
10.50	18.70	0.63	3.3	9	1.36	----	2.12	Clayey SILT to Silty CLAY	120-130
11.00	16.30	0.42	2.6	8	1.42	----	1.83	''	''
11.50	16.20	0.37	2.3	8	1.49	----	1.82	''	''
12.00	11.90	0.26	2.2	6	1.54	----	1.31	''	110-120
12.50	12.40	0.20	1.6	6	1.60	----	1.36	''	100-110
13:00	10.40	0.14	1.4	5	1.64	----	1.13	''	90-100
13.50	9.90	0.13	1.3	5	1.69	----	1.07	''	''
14.00	13.50	0.34	2.5	9	1.75	----	1.49	Silty CLAY to CLAY	110-120
14.50	7.50	0.13	1.8	4	1.80	----	0.78	Clayey SILT to Silty CLAY	90-100
15.00	9.70	0.15	1.6	5	1.85	----	1.03	''	100-110
15.50	8.60	0.15	1.8	4	1.90	----	0.90	''	''
16.00	6.90	0.10	1.5	3	1.95	----	0.70	''	90-100
16.50	4.70	0.08	1.7	2	2.00	----	0.44	''	''
17.00	5.50	0.07	1.3	3	2.05	----	0.50	Sensitive Fine Grained	''
17.50	6.10	0.09	1.5	3	2.09	----	0.56	''	''
18.00	5.30	0.07	1.3	3	2.14	----	0.47	''	''
18.50	5.01	0.10	2.0	3	2.19	----	0.46	Silty CLAY to CLAY	''
19.00	8.50	0.29	3.4	9	2.24	----	0.82	CLAY	110-120
19.50	9.50	0.36	3.8	10	2.30	----	0.93	''	''
20.00	10.20	0.39	3.8	10	2.36	----	1.00	''	''
20.50	41.20	1.39	3.4	27	2.43	----	4.70	Silty CLAY to CLAY	130-140
21.00	135.58	3.02	2.2	54	2.49	----	15.80	Sandy SILT to Clayey SILT	''
21.50	66.10	2.39	3.6	26	2.56	----	7.63	''	''
22.00	105.89	2.74	2.6	42	2.63	----	12.30	''	''
22.50	83.69	2.37	2.8	33	2.70	----	9.69	''	''
23.00	138.18	2.34	1.7	46	2.77	41	----	Silty SAND to Sandy SILT	''
23.50	252.05	3.68	1.5	63	2.83	44	----	SAND to Silty SAND	''
24.00	255.05	5.03	2.0	51	2.90	44	----	SAND	''
24.50	316.23	2.80	0.9	63	2.96	45	----	''	120-130
25.00	185.37	2.34	1.3	37	3.03	42	----	''	130-140
25.50	304.34	3.33	1.1	61	3.09	44	----	''	120-130
26.00	321.73	6.46	2.0	64	3.16	45	----	''	130-140
26.50	69.40	2.78	4.0	35	3.23	37	----	SAND to Clayey SAND *	''
27.00	53.30	0.81	1.5	18	3.29	35	----	Silty SAND to Sandy SILT	120-130
27.50	43.10	2.14	5.0	29	3.36	----	4.87	Silty CLAY to CLAY	130-140
28.00	44.50	2.68	6.0	45	3.43	----	4.75	CLAY	''
28.50	27.31	1.01	3.7	14	3.49	----	3.01	Clayey SILT to Silty CLAY	''
29.00	28.41	1.36	4.8	28	3.56	----	2.96	CLAY	''
29.50	179.17	2.24	1.3	60	3.63	41	----	Silty SAND to Sandy SILT	''
30.00	118.78	2.48	2.1	40	3.70	39	----	''	''
30.50	120.48	2.19	1.8	40	3.76	39	----	''	''
31.00	47.00	1.52	3.2	19	3.83	----	5.30	Sandy SILT to Clayey SILT	''
31.50	13.41	0.39	2.9	5	3.89	----	1.35	''	120-130
32.00	98.19	2.80	2.9	49	3.96	----	11.32	Clayey SILT to Silty CLAY	130-140

PROJECT: Napa River Flood Control Project
LOCATION: Napa CA
PROJ. NO.: COE-01

CPT NO.: CPT-94-2
DATE : 10-14-1994
ESTIMATED WT DEPTH: 20.17 feet

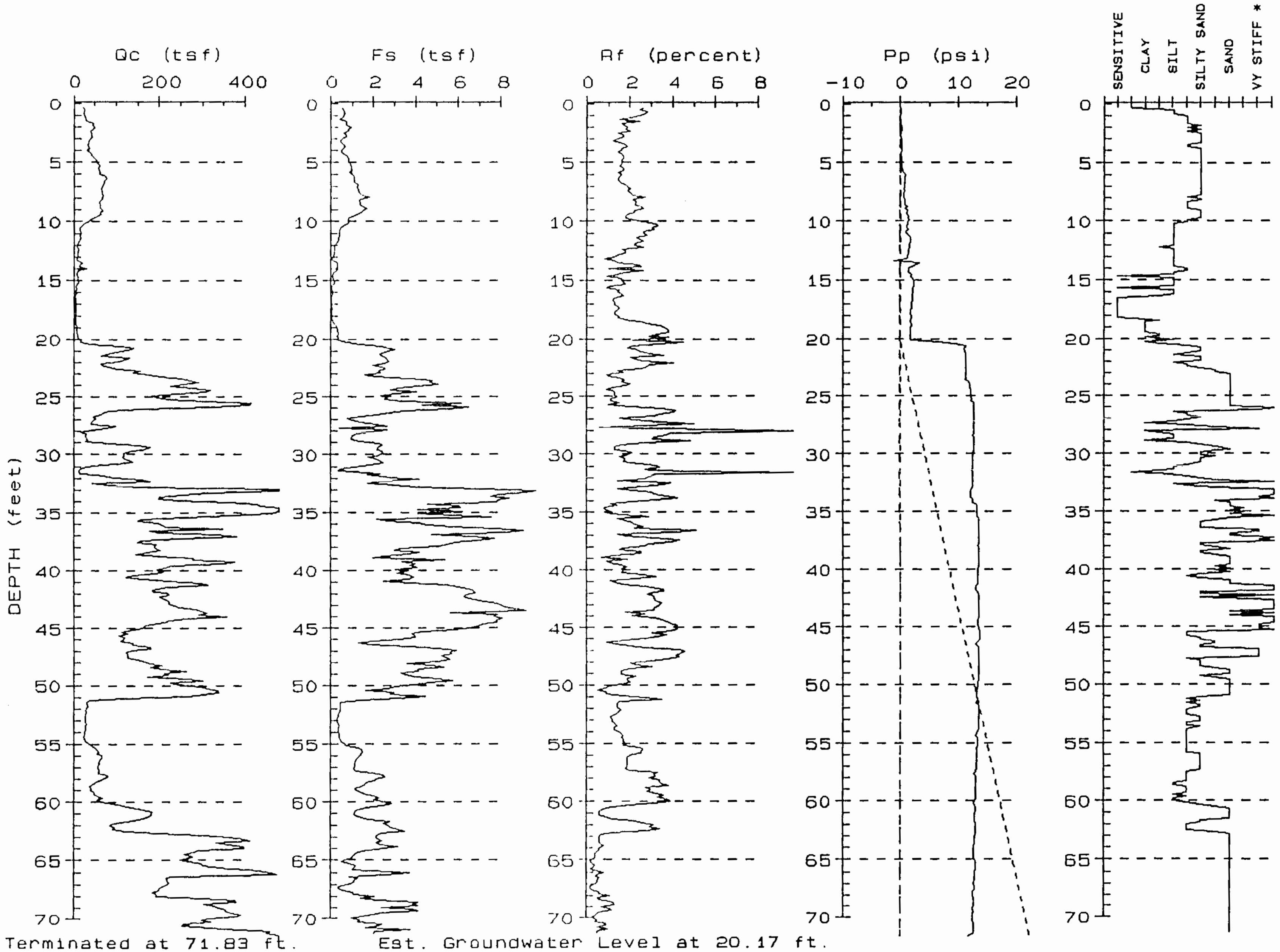
DEPTH (feet)	Qc (tsf)	Fs (tsf)	Rf (%)	SPT (N)	TotHzStr (ksf)	PHI (deg.)	SU (ksf)	SOIL BEHAVIOR TYPE	DENSITY RANGE (pcf)
32.50	44.90	1.74	3.9	22	4.03	----	5.05	''	''
33.00	515.48	7.04	1.4	172	4.10	46	----	Silty SAND to Sandy SILT	''
33.50	255.25	7.89	3.1	128	4.17	42	----	SAND to Clayey SAND *	>140
34.00	242.15	7.82	3.2	121	4.24	42	----	''	''
34.50	433.40	6.00	1.4	87	4.30	45	----	SAND	130-140
35.00	556.53	6.19	1.1	111	4.37	46	----	''	''
35.50	259.03	7.50	2.9	130	4.44	42	----	SAND to Clayey SAND *	''
36.00	195.76	4.14	2.1	49	4.51	41	----	SAND to Silty SAND	''
36.50	345.68	7.64	2.2	115	4.57	43	----	Silty SAND to Sandy SILT	''
37.00	329.81	4.75	1.4	66	4.64	43	----	SAND	''
37.50	159.87	6.90	4.3	80	4.71	39	----	SAND to Clayey SAND *	>140
38.00	189.26	5.06	2.7	63	4.78	40	----	Silty SAND to Sandy SILT	130-140
38.50	169.76	4.17	2.5	57	4.85	39	----	''	''
39.00	278.63	1.96	0.7	56	4.91	42	----	SAND	120-130
39.50	348.58	3.99	1.1	70	4.98	43	----	''	130-140
40.00	193.16	3.02	1.6	39	5.04	40	----	''	''
40.50	138.47	3.65	2.6	46	5.11	38	----	Silty SAND to Sandy SILT	''
41.00	223.45	2.47	1.1	45	5.17	41	----	SAND	120-130
41.50	218.65	5.95	2.7	73	5.24	40	----	Silty SAND to Sandy SILT	130-140
42.00	219.05	6.79	3.1	110	5.31	40	----	SAND to Clayey SAND *	>140
42.50	210.75	6.52	3.1	105	5.38	40	----	''	130-140
43.00	234.44	7.93	3.4	117	5.45	41	----	''	>140
43.50	296.82	9.15	3.1	148	5.52	42	----	''	''
44.00	356.68	7.63	2.1	71	5.59	43	----	SAND	130-140
44.50	211.65	7.79	3.7	106	5.66	40	----	SAND to Clayey SAND *	>140
45.00	161.97	6.55	4.0	81	5.73	38	----	''	''
45.50	126.58	3.81	3.0	42	5.79	37	----	Silty SAND to Sandy SILT	130-140
46.00	118.68	3.23	2.7	40	5.86	36	----	''	''
46.50	139.17	1.85	1.3	35	5.93	37	----	SAND to Silty SAND	''
47.00	137.97	5.92	4.3	138	6.00	----	15.88	Very Stiff Fine Grained *	>140
47.50	125.98	5.17	4.1	126	6.07	----	14.46	''	''
48.00	164.86	5.00	3.0	66	6.14	----	19.03	Sandy SILT to Clayey SILT	130-140
48.50	179.55	5.31	3.0	60	6.20	38	----	Silty SAND to Sandy SILT	''
49.00	211.84	3.29	1.6	53	6.27	39	----	SAND to Silty SAND	''
49.50	266.90	5.07	1.9	67	6.34	40	----	''	''
50.00	243.53	3.92	1.6	49	6.41	40	----	SAND	''
50.50	334.59	1.64	0.5	67	6.47	41	----	''	120-130
51.00	267.32	4.45	1.7	53	6.54	40	----	''	130-140
51.50	31.21	0.47	1.5	6	6.60	30	----	''	120-130
52.00	31.82	0.45	1.4	13	6.66	----	3.35	Sandy SILT to Clayey SILT	''
52.50	27.92	0.39	1.4	9	6.72	30	----	Silty SAND to Sandy SILT	''
53.00	27.72	0.32	1.2	9	6.78	30	----	''	110-120
53.50	29.12	0.40	1.4	12	6.84	----	3.02	Sandy SILT to Clayey SILT	120-130
54.00	26.92	0.39	1.4	11	6.91	----	2.76	''	''
54.50	26.02	0.43	1.6	10	6.97	----	2.65	''	''
55.00	36.72	0.65	1.8	15	7.03	----	3.91	''	''
55.50	57.31	1.48	2.6	23	7.10	----	6.32	''	130-140
56.00	67.20	1.42	2.1	22	7.17	31	----	Silty SAND to Sandy SILT	''
56.50	61.50	1.17	1.9	21	7.23	31	----	''	''
57.00	59.41	1.11	1.9	20	7.30	30	----	''	''
57.50	58.31	1.93	3.3	23	7.37	----	6.43	Sandy SILT to Clayey SILT	''
58.00	72.60	2.13	2.9	29	7.44	----	8.10	''	''
58.50	45.51	1.61	3.5	18	7.50	----	4.91	''	''
59.00	39.82	1.20	3.0	16	7.57	----	4.24	''	''
59.50	56.81	2.03	3.6	28	7.64	----	6.23	Clayey SILT to Silty CLAY	''
60.00	64.00	2.48	3.9	32	7.71	----	7.08	''	''
60.50	134.77	1.65	1.2	45	7.77	35	----	Silty SAND to Sandy SILT	120-130
61.00	181.84	1.13	0.6	36	7.83	37	----	SAND	''
61.50	154.66	1.80	1.2	31	7.89	36	----	''	''
62.00	86.89	2.70	3.1	35	7.96	----	9.75	Sandy SILT to Clayey SILT	130-140
62.50	123.77	3.49	2.8	50	8.03	----	14.09	''	''
63.00	363.25	2.14	0.6	91	8.09	41	----	SAND to Silty SAND	120-130
63.50	331.37	2.11	0.6	66	8.15	40	----	SAND	''
64.00	398.04	2.56	0.6	80	8.22	41	----	''	''

PROJECT: Napa River Flood Control Project   CPT NO.: CPT-94-2
LOCATION: Napa CA   DATE : 10-14-1994
PROJ. NO.: COE-01   ESTIMATED WT DEPTH: 20.17 feet

DEPTH (feet)	Qc (tsf)	Fs (tsf)	Rf (%)	SPT (N)	TotHzStr (ksf)	PHI (deg.)	SU (ksf)	SOIL BEHAVIOR TYPE	DENSITY RANGE (pcf)
64.50	258.11	1.14	0.4	52	8.28	39	----	''	''
65.00	248.21	0.71	0.3	50	8.34	38	----	''	''
65.50	301.29	1.21	0.4	60	8.40	39	----	''	''
66.00	460.01	3.73	0.8	92	8.47	42	----	''	''
66.50	217.53	1.31	0.6	44	8.53	38	----	''	''
67.00	219.52	0.78	0.4	44	8.59	38	----	''	''
67.50	200.94	0.62	0.3	40	8.65	37	----	''	''
68.00	194.34	2.07	1.1	39	8.72	37	----	''	''
68.50	379.35	4.12	1.1	76	8.78	41	----	''	''
69.00	335.37	2.41	0.7	67	8.84	40	----	''	''
69.50	367.35	2.35	0.6	73	8.90	40	----	''	''
70.00	314.88	0.98	0.3	63	8.97	39	----	''	''
70.50	256.41	2.42	0.9	51	9.03	38	----	''	''
71.00	388.34	3.79	1.0	78	9.09	41	----	''	''
71.50	514.44	6.84	1.3	103	9.16	42	----	''	130-140

DEPTH = Sampling interval (2 inches)
Qc = Tip bearing resistance
Fs = Sleeve friction resistance
Rf = Tip/Sleeve ratio
SPT = Equivalent Standard Penetration Test*
TotStr = Total Stress using est. density**
Phi = Soil friction angle*
Su = Undrained Soil Strength*

References: * Robertson and Campanella, 1988
** Olsen, 1989



Terminated at 71.83 ft. Est. Groundwater Level at 20.17 ft.

PROJECT: Napa River Flood Control Project
LOCATION: Napa CA
PROJ. NO.: COE-01

CPT NO.: CPT-94-2
DATE : 10-14-1994

John Sarmiento & Associates
Cone Penetration Testing Service

SOUNDING DATA IN FILE 04w216   12-09-04 10:26

OPERATOR : Alberto De Leon   LOCATION : CPT-1

CONE ID : H0836TC   JOB No. : CPT-04-01

N 38° 12.808'
W 122° 16.956'

N 1,870,036
E 6,480,560

VBI IN-SITU TESTING
3911 W. Capitol Ave., W.Sacramento, CA 95691

DEPTH meters	DEPTH feet	TIP Qc tsf	CORR TIP Qt tsf	FRICTION Fs tsf	FR RATIO Fs/Qc %	PORE PR Pw psi	DIFF P P RATIO (Pw-Ph)/Qc %	INC i deg	INTERPRETED SOIL TYPE	N SPT
Baseline		-7.2		0.32		-24.3		-1.3		
0.05	0.2	0.7	0.7	-0.02	-2.0	-0.1	-1.4	-1.3	?	?
0.10	0.3	1.0	1.0	0.07	6.7	-2.3	-1.8	-1.3	sandy silt to clayey silt	4
0.15	0.5	28.1	28.1	0.04	0.1	0.2	0.1	-1.3	silty sand to sandy silt	9
0.20	0.7	59.0	59.0	0.10	0.2	-0.2	-0.0	-1.3	sand to silty sand	11
0.25	0.8	56.8	56.8	0.11	0.2	-0.3	-0.0	-1.3	sand to silty sand	14
0.30	1.0	61.0	61.0	0.10	0.2	-0.2	-0.0	-1.3	sand to silty sand	12
0.35	1.1	37.3	37.3	0.10	0.3	-0.4	-0.1	-1.3	sand to silty sand	11
0.40	1.3	37.6	37.6	0.07	0.2	-0.1	-0.0	-1.3	silty sand to sandy silt	11
0.45	1.5	33.0	33.0	0.09	0.3	-0.2	-0.0	-1.3	silty sand to sandy silt	11
0.50	1.6	29.6	29.6	0.11	0.4	-0.1	-0.0	-1.3	silty sand to sandy silt	10
0.55	1.8	31.8	31.8	0.09	0.3	-0.0	-0.0	-1.3	silty sand to sandy silt	12
0.60	2.0	51.4	51.4	0.42	0.8	0.0	0.0	-1.3	sand to silty sand	16
0.65	2.1	111.6	111.6	0.61	0.5	0.1	0.0	-1.3	sand to silty sand	21
0.70	2.3	95.8	96.8	0.46	0.5	-0.0	-0.0	-1.3	sand to silty sand	22
0.75	2.5	68.5	68.5	0.42	0.6	0.1	0.0	-1.3	sand to silty sand	18
0.80	2.6	59.0	59.0	0.31	0.5	0.0	0.0	-1.3	sand to silty sand	14
0.85	2.8	50.2	50.3	0.42	0.8	0.2	0.0	-1.3	silty sand to sandy silt	17
0.90	3.0	50.7	50.7	0.43	0.9	0.1	0.0	-1.3	silty sand to sandy silt	14
0.95	3.1	30.2	30.2	0.43	1.4	-0.4	-0.1	-1.3	silty sand to sandy silt	12
1.00	3.3	30.3	30.3	0.43	1.4	-0.3	-0.1	-1.3	sandy silt to clayey silt	12
1.05	3.4	30.6	30.6	0.43	1.4	-0.5	-0.1	-1.3	sandy silt to clayey silt	12
1.10	3.6	30.3	30.3	0.43	1.4	-0.4	-0.1	-1.3	sandy silt to clayey silt	12
1.15	3.8	30.2	30.1	0.44	1.4	-0.4	-0.1	-1.3	sandy silt to clayey silt	12
1.20	3.9	30.5	30.5	0.44	1.5	-0.4	-0.1	-1.3	sandy silt to clayey silt	12
1.25	4.1	30.3	30.3	0.55	1.8	-0.5	-0.1	-1.3	sandy silt to clayey silt	12
1.30	4.3	30.1	30.1	0.74	2.5	-0.5	-0.1	-1.3	sandy silt to clayey silt	13
1.35	4.4	40.5	40.5	0.29	0.7	-0.5	-0.1	-1.3	silty sand to sandy silt	11
1.40	4.6	36.4	36.4	0.26	0.7	-0.6	-0.1	-1.3	silty sand to sandy silt	12
1.45	4.8	37.8	37.8	0.17	0.4	-0.6	-0.1	-1.3	silty sand to sandy silt	12
1.50	4.9	34.7	34.6	0.40	1.2	-0.5	-0.1	-1.3	silty sand to sandy silt	11
1.55	5.1	31.8	31.8	0.49	1.6	-0.5	-0.1	-1.3	sandy silt to clayey silt	12
1.60	5.2	30.6	30.6	0.90	3.0	-0.4	-0.1	-1.3	sandy silt to clayey silt	13
1.65	5.4	37.6	37.6	0.73	1.9	-0.3	-0.1	-1.3	sandy silt to clayey silt	12
1.70	5.6	27.0	27.0	0.88	3.3	-0.4	-0.1	-1.3	sandy silt to clayey silt	12
1.75	5.7	27.9	27.8	0.92	3.3	-2.5	-0.6	-1.3	clayey silt to silty clay	12
1.80	5.9	21.5	21.5	0.96	4.5	-2.5	-0.8	-1.3	silty clay to clay	14
1.85	6.1	14.7	14.6	0.83	5.7	-2.8	-1.4	-1.3	clay	15
1.90	6.2	11.9	11.8	0.60	5.1	-1.8	-1.1	-1.3	clay	12
1.95	6.4	12.5	12.5	0.57	4.6	-0.2	-0.1	-1.3	clay	12
2.00	6.6	12.4	12.4	0.55	4.4	0.7	0.4	-1.3	clay	11

Soil interpretation reference: Robertson & Campanella-1983, based on 60% hammer efficiency and .15 m sliding data average

DEPTH meters	DEPTH feet	TIP Qc tsf	CORR TIP Qt tsf	FRICTION Fs tsf	FR RATIO Fs/Qc %	PORE PR Pw psi	DIFF P P RATIO (Pw-Pn)/Qc %	INC I deg	INTERPRETED SOIL TYPE	N SPT
2.05	6.7	9.9	9.9	0.47	4.8	0.7	0.5	-1.3	clay	10
2.10	6.9	9.9	9.9	0.47	4.7	0.6	0.5	-1.3	clay	10
2.15	7.1	12.1	12.1	0.62	5.2	-0.2	-0.1	-1.3	clay	13
2.20	7.2	17.9	17.8	0.90	5.1	-1.7	-0.7	-1.3	clay	17
2.25	7.4	23.2	23.2	1.11	4.8	-4.2	-1.3	-1.3	clay	21
2.30	7.5	24.2	24.2	1.05	4.4	-2.8	-0.8	-1.3	clay	23
2.35	7.7	23.3	23.2	1.07	4.6	-1.8	-0.6	-1.3	clay	22
2.40	7.9	20.6	20.8	1.04	5.0	-3.0	-1.0	-1.3	clay	20
2.45	8.0	19.8	19.8	0.98	5.0	-4.4	-1.6	-1.3	clay	19
2.50	8.2	19.3	19.3	0.83	4.3	-4.3	-1.6	-1.3	clay	18
2.55	8.4	17.5	17.4	0.75	4.3	-4.5	-1.9	-1.3	clay	17
2.60	8.5	17.2	17.1	0.73	4.3	-1.6	-0.7	-1.3	clay	16
2.65	8.7	15.5	15.5	0.74	4.8	-0.4	-0.2	-1.3	clay	15
2.70	8.9	14.8	14.8	0.64	4.3	-2.4	-1.2	-1.3	clay	13
2.75	9.0	11.8	11.8	0.60	5.1	-0.7	-0.4	-1.3	clay	12
2.80	9.2	10.4	10.4	0.48	4.6	-3.2	-2.2	-1.3	clay	10
2.85	9.4	9.3	9.3	0.38	4.1	-2.0	-1.5	-1.3	clay	10
2.90	9.5	11.0	10.9	0.35	3.2	-1.3	-0.8	-1.3	silty clay to clay	8
2.95	9.7	15.5	15.5	0.48	3.1	-0.2	-0.1	-1.3	clayey silt to silty clay	7
3.00	9.8	19.2	19.2	0.54	2.8	-0.4	-0.1	-1.3	clayey silt to silty clay	9
3.05	10.0	19.6	19.6	0.45	2.3	-0.4	-0.1	-1.3	clayey silt to silty clay	9
3.10	10.2	19.7	19.7	0.33	1.7	-0.1	-0.0	-1.3	sandy silt to clayey silt	7
3.15	10.3	18.7	18.7	0.32	1.7	-0.2	-0.1	-1.3	sandy silt to clayey silt	7
3.20	10.5	17.0	17.0	0.30	1.8	-0.3	-0.1	-1.3	clayey silt to silty clay	8
3.25	10.7	14.2	14.2	0.29	2.0	-0.3	-0.2	-1.3	clayey silt to silty clay	7
3.30	10.8	12.2	12.2	0.29	2.4	-0.3	-0.2	-1.3	clayey silt to silty clay	7
3.35	11.0	14.9	14.9	0.35	2.4	-0.2	-0.1	-1.3	clayey silt to silty clay	7
3.40	11.2	13.9	13.9	0.43	3.1	-0.2	-0.1	-1.3	silty clay to clay	9
3.45	11.3	11.4	11.4	0.45	3.9	-0.4	-0.2	-1.3	silty clay to clay	7
3.50	11.5	9.1	9.1	0.35	3.9	-0.4	-0.3	-1.3	clay	9
3.55	11.6	7.2	7.2	0.20	2.7	-0.4	-0.4	-1.3	clay	7
3.60	11.8	4.9	5.0	0.12	2.3	0.8	1.2	-1.3	silty clay to clay	4
3.65	12.0	4.7	4.7	0.10	2.0	3.2	4.9	-1.2	silty clay to clay	3
3.70	12.1	4.6	4.7	0.08	1.8	3.3	5.2	-1.2	sensitive fine grained	2
3.75	12.3	4.0	4.0	0.06	1.6	2.5	4.5	-1.2	sensitive fine grained	2
3.80	12.5	4.0	4.1	0.05	1.3	3.0	5.4	-1.2	sensitive fine grained	2
3.85	12.6	4.4	4.5	0.05	1.1	3.1	5.1	-1.2	sensitive fine grained	2
3.90	12.8	4.5	4.6	0.05	1.1	3.5	5.5	-1.2	sensitive fine grained	2
3.95	13.0	3.9	4.0	0.04	0.9	3.9	7.1	-1.2	sensitive fine grained	2
4.00	13.1	3.3	3.4	0.03	0.8	4.2	9.2	-1.2	sensitive fine grained	2
4.05	13.3	3.1	3.2	0.03	1.0	4.4	10.2	-1.2	sensitive fine grained	2
4.10	13.5	4.2	4.3	0.02	0.5	4.5	7.8	-1.2	sensitive fine grained	2
4.15	13.6	3.8	3.8	0.02	0.6	4.7	9.0	-1.2	sensitive fine grained	2
4.20	13.8	3.8	3.9	0.02	0.5	5.0	9.4	-1.2	sensitive fine grained	2
4.25	13.9	3.5	3.5	0.06	1.6	5.3	11.1	-1.2	sensitive fine grained	2
4.30	14.1	3.7	3.8	0.06	1.5	5.5	10.8	-1.2	sensitive fine grained	2
4.35	14.3	4.1	4.2	0.05	1.1	7.9	14.0	-1.2	sensitive fine grained	2
4.40	14.4	3.5	3.6	0.03	1.0	9.3	19.0	-1.1	sensitive fine grained	2
4.45	14.6	3.9	4.1	0.03	0.7	9.8	18.1	-1.1	sensitive fine grained	2
4.50	14.8	4.0	4.2	0.03	0.8	10.7	19.2	-1.1	sensitive fine grained	2

Soil interpretation reference: Robertson & Campanella-1983, based on 60% hammer efficiency and .15 m sliding data average

DEPTH meters	DEPTH feet	TIP Qc tsf	CORR TIP Qt tsf	FRICTION Fs tsf	FR RATIO Fs/Qc %	PORE PR Pw psi	DIFF P P RATIO (Pw-Ph)/Qc %	INC I deg	INTERPRETED SOIL TYPE	N SPT
4.55	14.9	4.2	4.4	0.03	0.8	11.0	18.7	-1.1	sensitive fine grained	2
4.60	15.1	4.0	4.1	0.03	0.8	11.2	20.4	-1.1	sensitive fine grained	2
4.65	15.3	4.3	4.5	0.05	1.2	11.6	19.2	-1.1	sensitive fine grained	2
4.70	15.4	4.6	4.8	0.05	1.0	11.8	18.6	-1.1	sensitive fine grained	2
4.75	15.6	4.3	4.5	0.04	1.0	12.2	20.4	-1.1	sensitive fine grained	2
4.80	15.7	4.5	4.7	0.03	0.8	12.4	20.0	-1.1	sensitive fine grained	2
4.85	15.9	4.9	5.1	0.04	0.8	12.8	18.9	-1.1	sensitive fine grained	2
4.90	16.1	5.3	5.5	0.05	0.9	13.3	18.1	-1.1	sensitive fine grained	2
4.95	16.2	5.1	5.3	0.06	1.1	13.6	19.0	-1.1	sensitive fine grained	3
5.00	16.4	6.0	6.2	0.07	1.1	14.0	16.7	-1.1	sensitive fine grained	3
5.05	16.6	7.3	7.5	0.08	1.1	14.4	14.2	-1.0	sensitive fine grained	3
5.10	16.7	8.1	8.3	0.10	1.2	14.6	13.0	-1.0	clayey silt to silty clay	4
5.15	16.9	8.6	8.8	0.11	1.3	14.5	12.2	-1.0	clayey silt to silty clay	4
5.20	17.1	8.2	8.4	0.07	0.9	14.3	12.6	-1.0	clayey silt to silty clay	4
5.25	17.2	8.0	8.2	0.14	1.7	14.7	13.3	-1.0	clayey silt to silty clay	4
5.30	17.4	7.5	7.8	0.18	2.3	15.1	14.4	-1.0	silty clay to clay	5
5.35	17.6	8.7	9.0	0.18	2.1	21.3	17.6	-0.9	silty clay to clay	5
5.40	17.7	6.5	6.8	0.14	2.2	21.9	24.3	-0.9	clayey silt to silty clay	4
5.45	17.9	8.6	9.0	0.13	1.5	22.5	18.7	-0.9	clayey silt to silty clay	4
5.50	18.0	8.9	9.2	0.14	1.6	23.3	18.9	-0.9	clayey silt to silty clay	4
5.55	18.2	8.9	9.2	0.13	1.4	23.6	19.1	-0.9	clayey silt to silty clay	4
5.60	18.4	9.0	9.3	0.11	1.2	24.6	19.8	-0.9	clayey silt to silty clay	4
5.65	18.5	9.4	9.8	0.12	1.3	25.4	19.4	-0.9	clayey silt to silty clay	5
5.70	18.7	10.1	10.5	0.11	1.1	26.2	18.7	-0.9	clayey silt to silty clay	5
5.75	18.9	11.7	12.1	0.11	1.0	27.2	16.7	-0.8	sandy silt to clayey silt	4
5.80	19.0	12.2	12.6	0.14	1.2	28.4	16.8	-0.8	clayey silt to silty clay	6
5.85	19.2	12.1	12.5	0.25	2.0	27.6	16.3	-0.8	clayey silt to silty clay	7
5.90	19.4	22.2	22.5	0.43	1.9	22.6	7.4	-0.9	sandy silt to clayey silt	8
5.95	19.5	26.2	26.2	0.48	1.8	2.8	0.8	-1.2	sandy silt to clayey silt	9
6.00	19.7	22.7	22.7	0.27	1.2	-4.7	-1.5	-1.3	sandy silt to clayey silt	8
6.05	19.8	14.2	14.1	0.22	1.6	-5.9	-3.0	-1.4	sandy silt to clayey silt	6
6.10	20.0	12.9	12.8	0.13	1.0	-3.3	-1.8	-1.3	sandy silt to clayey silt	5
6.15	20.2	11.9	11.8	0.13	1.1	-3.4	-2.1	-1.3	clayey silt to silty clay	6
6.20	20.3	10.7	10.6	0.36	3.3	-3.2	-2.1	-1.3	silty clay to clay	7
6.25	20.5	10.4	10.4	0.36	3.5	-2.9	-2.0	-1.3	silty clay to clay	8
6.30	20.7	16.2	16.2	0.62	3.8	0.6	0.3	-1.3	clayey silt to silty clay	9
6.35	20.8	31.7	31.7	0.64	2.0	-1.2	-0.3	-1.3	clayey silt to silty clay	12
6.40	21.0	27.0	26.9	0.78	2.9	-6.9	-1.8	-1.4	sandy silt to clayey silt	11
6.45	21.2	24.0	23.9	0.54	2.2	-7.5	-2.2	-1.4	clayey silt to silty clay	10
6.50	21.3	14.3	14.2	0.36	2.5	-7.6	-3.8	-1.4	clayey silt to silty clay	8
6.55	21.5	11.2	11.1	0.22	2.0	-7.5	-4.8	-1.4	clayey silt to silty clay	6
6.60	21.7	9.9	9.8	0.13	1.3	-7.4	-5.4	-1.4	clayey silt to silty clay	5
6.65	21.8	8.6	8.5	0.08	1.0	-7.2	-6.0	-1.4	clayey silt to silty clay	4
6.70	22.0	7.4	7.3	0.05	0.7	-7.1	-7.0	-1.4	sensitive fine grained	4
6.75	22.1	7.4	7.3	0.04	0.5	-6.9	-6.7	-1.4	sensitive fine grained	4
6.80	22.3	7.9	7.8	0.12	1.5	-6.8	-6.2	-1.4	clayey silt to silty clay	4
6.85	22.5	10.9	10.8	0.27	2.5	-6.7	-4.4	-1.4	clayey silt to silty clay	6
6.90	22.6	16.5	16.4	0.19	1.2	-6.6	-2.9	-1.4	clayey silt to silty clay	7
6.95	22.8	16.1	16.0	0.42	2.6	-6.7	-3.0	-1.4	clayey silt to silty clay	8
7.00	23.0	18.5	18.4	0.49	2.6	-6.5	-2.5	-1.4	clayey silt to silty clay	10

Soil interpretation reference: Robertson & Campanella-1983, based on 60% hammer efficiency and .15 m sliding data average

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 DEPTH  DEPTH    TIP   CORR TIP  FRICTION  FR RATIO  PORE PR  DIFF P P RATIO     INC              INTERPRETED      N
 meters  feet  Qc tsf    Qt tsf    Fs tsf   Fs/Qc %   Pw psi   (Pw-Pn)/Qc %   I deg                SOIL TYPE     SPT

   7.05   23.1   25.4      25.3      0.49       1.9     -6.8           -1.9    -1.4  sandy silt to clayey silt     9
   7.10   23.3   25.4      25.3      0.32       1.3     -7.3           -2.1    -1.4  sandy silt to clayey silt     9

QUIT for Inc
   7.25          10.0               -0.00               -7.1                   -1.5

   7.15   23.5   16.5      16.4      0.23       1.4     -7.8           -3.4    -1.4  clayey silt to silty clay     8

QUIT for Inc
   7.30          33.2                0.56               -7.3                   -1.5

   7.20   23.6   10.6      10.4      0.57       5.4     -7.9           -5.4    -1.4  clayey silt to silty clay     8
   7.25   23.8   20.9      20.8      0.66       3.2     -7.6           -2.6    -1.4  clayey silt to silty clay    11
   7.30   23.9   34.8      34.7      0.57       1.6     -6.9           -1.4    -1.4  sandy silt to clayey silt    14
   7.35   24.1   51.6      51.5      0.50       1.0     -8.3           -1.2    -1.4  silty sand to sandy silt     17
   7.40   24.3   73.3      73.2      0.45       0.6     -8.8           -0.9    -1.4     sand to silty sand        18
   7.45   24.4   96.9      96.8      0.56       0.6     -9.1           -0.7    -1.4     sand to silty sand        24
   7.50   24.6  135.5     135.3      0.69       0.5     -9.4           -0.5    -1.4           sand                24
   7.55   24.8  139.5     139.4      0.46       0.3     -9.4           -0.5    -1.4           sand                26
   7.60   24.9  130.1     129.9      0.45       0.3     -9.2           -0.5    -1.4           sand                24
   7.65   25.1  113.0     112.9      0.40       0.4     -9.3           -0.6    -1.4           sand                23
   7.70   25.3  117.4     117.3      0.56       0.5     -9.3           -0.6    -1.4           sand                24
   7.75   25.4  140.3     140.2      0.62       0.4     -9.8           -0.5    -1.4           sand                26
   7.80   25.6  154.5     154.3      1.13       0.7     -9.7           -0.5    -1.4           sand                30
   7.85   25.8  168.0     167.9      0.95       0.6     -5.2           -0.4    -1.4           sand                32
   7.90   25.9  186.3     186.2      1.34       0.7     -4.4           -0.2    -1.3           sand                35
   7.95   26.1  189.4     189.4      2.23       1.2     -1.8           -0.1    -1.3           sand                36
   8.00   26.2  186.0     186.0      1.60       0.9      2.0            0.1    -1.2           sand                35
   8.05   26.4  179.6     179.7      1.36       0.8      1.7            0.1    -1.2           sand                35
   8.10   26.6  189.3     189.4      1.38       0.7      4.1            0.2    -1.2           sand                36
   8.15   26.7  195.8     195.9      1.12       0.6      4.4            0.2    -1.2           sand                37
   8.20   26.9  196.5     196.6      1.16       0.6      5.7            0.2    -1.2           sand                38
   8.25   27.1  206.7     206.7      1.81       0.9     -0.5           -0.0    -1.3           sand                37
   8.30   27.2  182.7     182.7      1.94       1.1     -3.9           -0.2    -1.3           sand                37
   8.35   27.4  186.4     186.3      1.43       0.8     -5.6           -0.2    -1.4           sand                35
   8.40   27.6  178.1     178.2      0.68       0.4      2.8            0.1    -1.3           sand                35
   8.45   27.7  184.0     184.0      0.92       0.5     -4.5           -0.2    -1.3           sand                35
   8.50   27.9  182.5     182.4      1.36       0.7     -3.7           -0.1    -1.3           sand                35
   8.55   28.1  185.7     185.6      1.21       0.7     -2.0           -0.1    -1.3           sand                34
   8.60   28.2  167.8     167.9      1.20       0.7      2.0            0.1    -1.2           sand                33
   8.65   28.4  165.8     165.8      1.04       0.6      0.3            0.0    -1.3           sand                31
   8.70   28.5  145.3     145.2      0.60       0.4     -4.7           -0.2    -1.3           sand                29
   8.75   28.7  141.8     141.8      0.49       0.3     -1.8           -0.1    -1.3           sand                27
   8.80   28.9  132.4     132.4      0.30       0.2     -0.7           -0.0    -1.3           sand                25
   8.85   29.0  122.9     122.9      0.48       0.4     -1.9           -0.1    -1.3           sand                23
   8.90   29.2  109.1     109.2      0.40       0.4      0.5            0.0    -1.3           sand                21
   8.95   29.4   94.7      94.7      0.50       0.5      2.8            0.2    -1.2     sand to silty sand        23
   9.00   29.5   86.0      86.0      0.46       0.6      3.6            0.3    -1.2     sand to silty sand        21
   9.05   29.7   76.5      76.6      0.52       0.7      4.6            0.4    -1.2     sand to silty sand        18
   9.10   29.9   67.7      67.7      0.48       0.7      6.3            0.7    -1.2     sand to silty sand        16

Soil interpretation reference: Robertson & Campanella-1983, based on 60% hammer efficiency and .15 m sliding data average
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 DEPTH  DEPTH    TIP   CORR TIP  FRICTION  FR RATIO  PORE PR  DIFF P P RATIO     INC              INTERPRETED      N
 meters  feet  Qc tsf   Qt tsf    Fs tsf   Fs/Qc %   Pw psi    (Pw-Ph)/Qc %    I deg               SOIL TYPE      SPT

   9.15   30.0   56.9     57.0      0.18       0.3      7.2              0.9    -1.2     sand to silty sand         12
   9.20   30.2   21.0     21.1      0.18       0.8      6.0              2.1    -1.2  silty sand to sandy silt      10
   9.25   30.3   13.6     13.7      0.15       1.1      7.0              3.7    -1.2  sandy silt to clayey silt      5
   9.30   30.5    7.0      7.0      0.07       0.9      3.4              3.5    -1.2  sandy silt to clayey silt      4
   9.35   30.7   13.4     13.4      0.10       0.8      4.8              2.6    -1.2  sandy silt to clayey silt      5
   9.40   30.8   16.4     16.5      0.08       0.5      7.2              3.2    -1.2  sandy silt to clayey silt      5
   9.45   31.0   11.8     11.9     -0.02      -0.2      7.4              4.5    -1.1  sandy silt to clayey silt      6
   9.50   31.2   17.8     17.9     -0.00      -0.0      7.5              3.0    -1.1  sandy silt to clayey silt      7

QUIT for  Qc Rate
   9.65         174.7               0.62                5.7                     -1.2

   9.55   31.3   21.8     21.9      0.41       1.9      7.5              2.5    -1.1  sandy silt to clayey silt      9
   9.60   31.5   29.9     30.0      0.66       2.2      7.6              1.8    -1.1  clayey silt to silty clay     21
   9.65   31.7   73.0     73.1      3.37       4.3      2.2              0.2    -1.2     sand to silty sand         25
   9.70   31.8  202.7    202.8      0.57       0.3      3.9              0.1    -1.2                         ?      ?
   9.75   32.0  337.7    337.8         ?         ?      2.9              0.1    -1.2                         ?      ?
   9.80   32.2  123.6    123.7         ?         ?      5.5              0.3    -1.2                         ?      ?

WRITE NUMBER OF RODS USED __

Soil interpretation reference: Robertson & Campanella-1983, based on
 60% hammer efficiency and .15 m sliding data average
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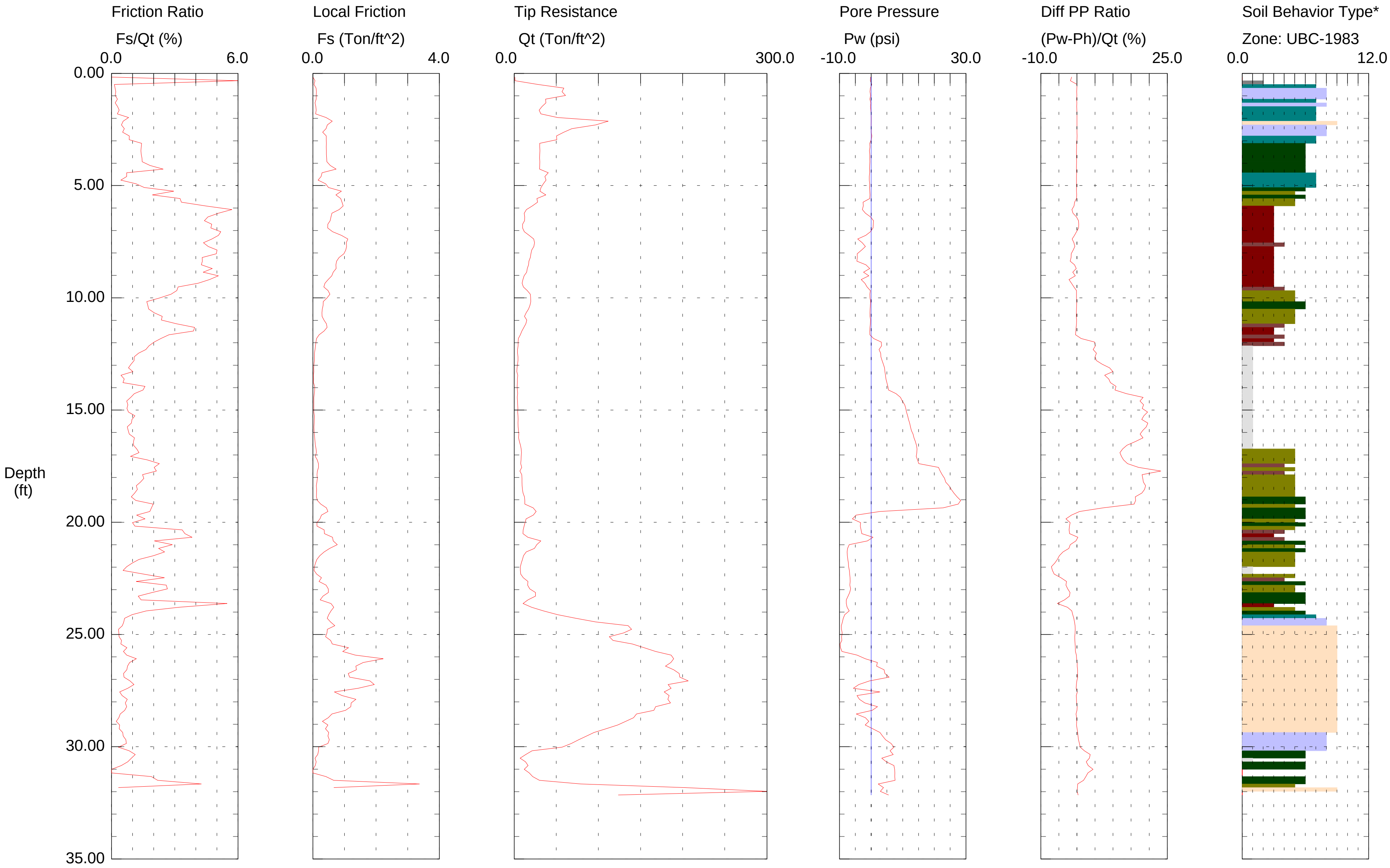
*Soil behavior type and SPT based on data from UBC-1983

VBI In-Situ Testing

Operator: Alberto De Leon
Sounding: 04w216
Cone Used: HO836TC

CPT Date/Time: 12-09-04 10:26
Location: CPT-1
Job Number: CPT-04-01



Maximum Depth = 32.15 feet

Depth Increment = 0.16 feet

■ 1 sensitive fine grained
■ 2 organic material
■ 3 clay
■ 4 silty clay to clay
■ 5 clayey silt to silty clay
■ 6 sandy silt to clayey silt
■ 7 silty sand to sandy silt
■ 8 sand to silty sand
■ 9 sand
■ 10 gravelly sand to sand
■ 11 very stiff fine grained (*)
■ 12 sand to clayey sand (*)

APPENDIX 2: LIQUEFACTION EVALUATION

Napa Contract 2 West Vertical Wall

Liquefaction Potential - Napa Mill Magnitude 6.5 Earthquake

Location	Depth (feet)	Depth (meters)	N_{60}	σ'_{vo} (psf)	σ'_{vo} (kPa)	C_N	C_B	C_R	C_S	$(N_1)_{60}$	% Fines
2F-90-29	24	7.32	28	1996	95.57	1.02	1.05	0.95	1.2	34	
2F-90-29	29	8.84	30	2364	113.19	0.94	1.05	1	1.2	36	12
2F-90-29	39	11.89	66	3100	148.43	0.82	1.05	1	1.2	68	
2F-94-14	19	5.79	61	1922.4	92.04	1.04	1.05	0.95	1.2	76	
2F-94-14	21	6.40	17	2069.6	99.09	1.00	1.05	0.95	1.2	20	7
2F-94-14	24	7.32	7	2290.4	109.66	0.95	1.05	0.95	1.2	8	
2F-94-14	31	9.45	35	2805.6	134.33	0.86	1.05	1	1.2	38	21
2F-94-14	36	10.97	53	3173.6	151.95	0.81	1.05	1	1.2	54	
2F-94-14	38	11.58	55	3320.8	159.00	0.79	1.05	1	1.2	55	
2F-94-14	40	12.19	48	3468	166.05	0.78	1.05	1	1.2	47	7

1 psf = 0.04788 kPa

C_N maximum = 1.7

C_B = 1.0 for borehole diameter less than 6"; 1.05 for 6"; 1.15 for 8"

C_R = 0.75 for rod length < 10'; 0.8 for 10-13'; 0.85 for 13-19.7'; 0.95 for 19.7-32.8'; 1.0 for 32.8-98.4'

C_S = 1.1-1.3 for samplers without liners

Boring	Depth (feet)	Depth (meters)	a_{max} (g)	$0.65(a_{max}/g)$	σ'_{vo} (kPa)	σ_{vo} (psf)	σ_{vo} (kPa)	r_d	MSF	CSR
2F-90-29	24	7.32	0.42	0.2730	95.57	2819	134.97	0.944039	1.44	0.253
2F-90-29	29	8.84	0.42	0.2730	113.19	3499	167.53	0.93238	1.44	0.262
2F-90-29	39	11.89	0.42	0.2730	148.43	4859	232.65	0.856612	1.44	0.255
2F-94-14	19	5.79	0.42	0.2730	92.04	2266	108.50	0.955697	1.44	0.214
2F-94-14	21	6.40	0.42	0.2730	99.09	2538	121.52	0.951034	1.44	0.221
2F-94-14	24	7.32	0.42	0.2730	109.66	2946	141.05	0.944039	1.44	0.230
2F-94-14	31	9.45	0.42	0.2730	134.33	3898	186.64	0.921717	1.44	0.243
2F-94-14	36	10.97	0.42	0.2730	151.95	4578	219.19	0.881026	1.44	0.241
2F-94-14	38	11.58	0.42	0.2730	159.00	4850	232.22	0.86475	1.44	0.239
2F-94-14	40	12.19	0.42	0.2730	166.05	5122	245.24	0.848474	1.44	0.238

$r_d = 1.0 - 0.00765z$ for $z <= 9.15m$

$r_d = 1.174 - 0.0267z$ for $9.15m > z <= 23$ m

Napa Contract 2 West Vertical Wall

Liquefaction Potential - Boring 2F-03-05 Magnitude 6.5 Earthquake

Boring	Depth (feet)	Depth (meters)	N_{60}	σ'_{vo} (psf)	σ'_{vo} (kPa)	C_N	C_B	C_R	C_S	$(N_1)_{60}$	% Fines
2F-03-05	17	5.18	11	2009.6	96.22	1.02	1	0.95	1.2	13	
2F-03-05	19	5.79	16	2156.8	103.27	0.98	1	0.95	1.2	18	
2F-03-05	21	6.40	25	2304	110.32	0.95	1	0.95	1.2	27	
2F-03-05	23	7.01	20	2451.2	117.36	0.92	1	0.95	1.2	21	
2F-03-05	25	7.62	23	2598.4	124.41	0.90	1	0.95	1.2	24	4
2F-03-05	27	8.23	29	2745.6	131.46	0.87	1	0.95	1.2	29	
2F-03-05	31	9.45	23	3040	145.56	0.83	1	1	1.2	23	9
2F-03-05	33	10.06	29	3187.2	152.60	0.81	1	1	1.2	28	
2F-03-05	35	10.67	39	3334.4	159.65	0.79	1	1	1.2	37	
2F-03-05	37	11.28	45	3481.6	166.70	0.77	1	1	1.2	42	8
2F-03-05	39	11.89	45	3628.8	173.75	0.76	1	1	1.2	41	
2F-03-05	43	13.11	71	3923.2	187.84	0.73	1	1	1.2	62	
2F-03-05	45	13.72	65	4070.4	194.89	0.72	1	1	1.2	56	9
2F-03-05	47	14.33	61	4217.6	201.94	0.70	1	1	1.2	52	
2F-03-05	49	14.94	72	4364.8	208.99	0.69	1	1	1.2	60	

1 psf = 0.04788 kPa

C_N maximum = 1.7

C_B = 1.0 for borehole diameter less than 6"; 1.05 for 6"; 1.15 for 8"

C_R = 0.75 for rod length < 10'; 0.8 for 10-13'; 0.85 for 13-19.7'; 0.95 for 19.7-32.8'; 1.0 for 32.8-98.4'

C_S = 1.1-1.3 for samplers without liners

Boring	Depth (feet)	Depth (meters)	a_{max} (g)	$0.65(a_{max}/g)$	σ'_{vo} (kPa)	σ_{vo} (psf)	σ_{vo} (kPa)	r_d	MSF	CSR
2F-03-05	17	5.18	0.42	0.2730	96.22	2072	99.21	0.960361	1.44	0.188
2F-03-05	19	5.79	0.42	0.2730	103.27	2344	112.23	0.955697	1.44	0.197
2F-03-05	21	6.40	0.42	0.2730	110.32	2616	125.25	0.951034	1.44	0.205
2F-03-05	23	7.01	0.42	0.2730	117.36	2888	138.28	0.94637	1.44	0.211
2F-03-05	25	7.62	0.42	0.2730	124.41	3160	151.30	0.941707	1.44	0.217
2F-03-05	27	8.23	0.42	0.2730	131.46	3432	164.32	0.937044	1.44	0.222
2F-03-05	31	9.45	0.42	0.2730	145.56	3976	190.37	0.921717	1.44	0.229
2F-03-05	33	10.06	0.42	0.2730	152.60	4248	203.39	0.905441	1.44	0.229
2F-03-05	35	10.67	0.42	0.2730	159.65	4520	216.42	0.889164	1.44	0.229
2F-03-05	37	11.28	0.42	0.2730	166.70	4792	229.44	0.872888	1.44	0.228
2F-03-05	39	11.89	0.42	0.2730	173.75	5064	242.46	0.856612	1.44	0.227
2F-03-05	43	13.11	0.42	0.2730	187.84	5608	268.51	0.824059	1.44	0.223
2F-03-05	45	13.72	0.42	0.2730	194.89	5880	281.53	0.807783	1.44	0.221
2F-03-05	47	14.33	0.42	0.2730	201.94	6152	294.56	0.791506	1.44	0.219
2F-03-05	49	14.94	0.42	0.2730	208.99	6424	307.58	0.77523	1.44	0.216

$r_d = 1.0 - 0.00765z$ for $z <= 9.15m$

$r_d = 1.174 - 0.0267z$ for $9.15m > z <= 23$ m

Napa Contract 2 West Vertical Wall

Liquefaction Potential - Boring 2F-03-06 Magnitude 6.5 Earthquake

Boring	Depth (feet)	Depth (meters)	N_{60}	σ'_{vo} (psf)	σ'_{vo} (kPa)	C_N	C_B	C_R	C_S	$(N_1)_{60}$	% Fines
2F-03-06	13	3.96	3	1513.6	72.47	1.17	1	0.85	1.2	4	31
2F-03-06	15	4.57	1	1636.8	78.37	1.13	1	0.85	1.2	1	
2F-03-06	25	7.62	21	2264.6	108.43	0.96	1	0.95	1.2	23	11
2F-03-06	27	8.23	36	2411.8	115.48	0.93	1	0.95	1.2	38	
2F-03-06	29	8.84	39	2559	122.52	0.90	1	1	1.2	42	
2F-03-06	31	9.45	59	2706.2	129.57	0.88	1	1	1.2	62	
2F-03-06	33	10.06	27	2853.4	136.62	0.86	1	1	1.2	28	
2F-03-06	35	10.67	37	3000.6	143.67	0.83	1	1	1.2	37	
2F-03-06	37	11.28	45	3147.8	150.72	0.81	1	1	1.2	44	10
2F-03-06	39	11.89	31	3295	157.76	0.80	1	1	1.2	30	
2F-03-06	41	12.50	60	3442.2	164.81	0.78	1	1	1.2	56	
2F-03-06	43	13.11	45	3589.4	171.86	0.76	1	1	1.2	41	
2F-03-06	45	13.72	51	3736.6	178.91	0.75	1	1	1.2	46	9
2F-03-06	47	14.33	72	3883.8	185.96	0.73	1	1	1.2	63	
2F-03-06	49	14.94	48	4031	193.00	0.72	1	1	1.2	41	

1 psf = 0.04788 kPa

C_N maximum = 1.7

C_B = 1.0 for borehole diameter less than 6"; 1.05 for 6"; 1.15 for 8"

C_R = 0.75 for rod length < 10'; 0.8 for 10-13'; 0.85 for 13-19.7'; 0.95 for 19.7-32.8'; 1.0 for 32.8-98.4'

C_S = 1.1-1.3 for samplers without liners

Boring	Depth (feet)	Depth (meters)	a_{max} (g)	$0.65(a_{max}/g)$	σ'_{vo} (kPa)	σ_{vo} (psf)	σ_{vo} (kPa)	r_d	MSF	CSR
2F-03-06	13	3.96	0.42	0.2730	72.47	1576	75.46	0.969688	1.44	0.191
2F-03-06	15	4.57	0.42	0.2730	78.37	1824	87.33	0.965024	1.44	0.204
2F-03-06	25	7.62	0.42	0.2730	108.43	3076	147.28	0.941707	1.44	0.242
2F-03-06	27	8.23	0.42	0.2730	115.48	3348	160.30	0.937044	1.44	0.247
2F-03-06	29	8.84	0.42	0.2730	122.52	3620	173.33	0.93238	1.44	0.250
2F-03-06	31	9.45	0.42	0.2730	129.57	3892	186.35	0.921717	1.44	0.251
2F-03-06	33	10.06	0.42	0.2730	136.62	4164	199.37	0.905441	1.44	0.251
2F-03-06	35	10.67	0.42	0.2730	143.67	4436	212.40	0.889164	1.44	0.249
2F-03-06	37	11.28	0.42	0.2730	150.72	4708	225.42	0.872888	1.44	0.248
2F-03-06	39	11.89	0.42	0.2730	157.76	4980	238.44	0.856612	1.44	0.245
2F-03-06	41	12.50	0.42	0.2730	164.81	5252	251.47	0.840335	1.44	0.243
2F-03-06	43	13.11	0.42	0.2730	171.86	5524	264.49	0.824059	1.44	0.240
2F-03-06	45	13.72	0.42	0.2730	178.91	5796	277.51	0.807783	1.44	0.238
2F-03-06	47	14.33	0.42	0.2730	185.96	6068	290.54	0.791506	1.44	0.234
2F-03-06	49	14.94	0.42	0.2730	193.00	6340	303.56	0.77523	1.44	0.231

$r_d = 1.0 - 0.00765z$ for $z <= 9.15m$

$r_d = 1.174 - 0.0267z$ for $9.15m > z <= 23 m$

Napa Contract 2 West Vertical Wall

Liquefaction Potential - Boring 2F-03-07 Magnitude 6.5 Earthquake

Boring	Depth (feet)	Depth (meters)	N_{60}	σ'_{vo} (psf)	σ'_{vo} (kPa)	C_N	C_B	C_R	C_S	$(N_1)_{60}$	% Fines
2F-03-07	25	7.62	63	2383.6	114.13	0.94	1	0.95	1.2	67	
2F-03-07	27	8.23	33	2530.8	121.17	0.91	1	0.95	1.2	34	9
2F-03-07	29	8.84	37	2678	128.22	0.88	1	1	1.2	39	
2F-03-07	31	9.45	28	2825.2	135.27	0.86	1	1	1.2	29	
2F-03-07	33	10.06	23	2972.4	142.32	0.84	1	1	1.2	23	8
2F-03-07	37	11.28	49	3266.8	156.41	0.80	1	1	1.2	47	
2F-03-07	39	11.89	47	3414	163.46	0.78	1	1	1.2	44	10
2F-03-07	41	12.50	24	3561.2	170.51	0.77	1	1	1.2	22	
2F-03-07	43	13.11	45	3708.4	177.56	0.75	1	1	1.2	41	
2F-03-07	45	13.72	23	3855.6	184.61	0.74	1	1	1.2	20	19
2F-03-07	47	14.33	36	4002.8	191.65	0.72	1	1	1.2	31	
2F-03-07	49	14.94	61	4150	198.70	0.71	1	1	1.2	52	10
2F-03-07	51	15.54	55	4297.2	205.75	0.70	1	1	1.2	46	

1 psf = 0.04788 kPa

C_N maximum = 1.7

C_B = 1.0 for borehole diameter less than 6"; 1.05 for 6"; 1.15 for 8"

C_R = 0.75 for rod length < 10'; 0.8 for 10-13'; 0.85 for 13-19.7'; 0.95 for 19.7-32.8'; 1.0 for 32.8-98.4'

C_S = 1.1-1.3 for samplers without liners

Boring	Depth (feet)	Depth (meters)	a_{max} (g)	$0.65(a_{max}/g)$	σ'_{vo} (kPa)	σ_{vo} (psf)	σ_{vo} (kPa)	r_d	MSF	CSR
2F-03-07	25	7.62	0.42	0.2730	114.13	3070	146.99	0.941707	1.44	0.230
2F-03-07	27	8.23	0.42	0.2730	121.17	3342	160.01	0.937044	1.44	0.235
2F-03-07	29	8.84	0.42	0.2730	128.22	3614	173.04	0.93238	1.44	0.239
2F-03-07	31	9.45	0.42	0.2730	135.27	3886	186.06	0.921717	1.44	0.240
2F-03-07	33	10.06	0.42	0.2730	142.32	4158	199.09	0.905441	1.44	0.240
2F-03-07	37	11.28	0.42	0.2730	156.41	4702	225.13	0.872888	1.44	0.238
2F-03-07	39	11.89	0.42	0.2730	163.46	4974	238.16	0.856612	1.44	0.237
2F-03-07	41	12.50	0.42	0.2730	170.51	5246	251.18	0.840335	1.44	0.235
2F-03-07	43	13.11	0.42	0.2730	177.56	5518	264.20	0.824059	1.44	0.232
2F-03-07	45	13.72	0.42	0.2730	184.61	5790	277.23	0.807783	1.44	0.230
2F-03-07	47	14.33	0.42	0.2730	191.65	6062	290.25	0.791506	1.44	0.227
2F-03-07	49	14.94	0.42	0.2730	198.70	6334	303.27	0.77523	1.44	0.224
2F-03-07	51	15.54	0.42	0.2730	205.75	6606	316.30	0.758954	1.44	0.221

$r_d = 1.0 - 0.00765z$ for $z <= 9.15m$

$r_d = 1.174 - 0.0267z$ for $9.15m > z <= 23$ m

Napa Contract 2 West Vertical Wall

Liquefaction Potential - Third Street to First Street Magnitude 6.5 Earthquake

Location	Depth (feet)	Depth (meters)	N_{60}	σ'_{vo} (psf)	σ'_{vo} (kPa)	C_N	C_B	C_R	C_S	$(N_1)_{60}$	% Fines
2F-94-15	22	6.71	23	2384	114.15	0.94	1.15	0.95	1.2	28	16
2F-94-15	24	7.32	38	2531	121.19	0.91	1.15	0.95	1.2	45	15
2F-94-15	26	7.92	50	2678	128.24	0.88	1.15	0.95	1.2	58	15
		0.00				#DIV/0!				#DIV/0!	
2F-03-08	25	7.62	28	2929.6	140.27	0.84	1	0.95	1.2	27	10 to 15
2F-03-08	27	8.23	33	3076.8	147.32	0.82	1	0.95	1.2	31	10
2F-03-08	29	8.84	39	3224	154.37	0.80	1	1	1.2	38	10 to 15
2F-03-08	31	9.45	24	3371.2	161.41	0.79	1	1	1.2	23	10 to 15
		0.00			0.00	#DIV/0!				#DIV/0!	
		0.00			0.00	#DIV/0!				#DIV/0!	
		0.00			0.00	#DIV/0!				#DIV/0!	
		0.00			0.00	#DIV/0!				#DIV/0!	

1 psf = 0.04788 kPa

C_N maximum = 1.7

C_B = 1.0 for borehole diameter less than 6"; 1.05 for 6"; 1.15 for 8"

C_R = 0.75 for rod length < 10'; 0.8 for 10-13'; 0.85 for 13-19.7'; 0.95 for 19.7-32.8'; 1.0 for 32.8-98.4'

C_S = 1.1-1.3 for samplers without liners

Boring	Depth (feet)	Depth (meters)	a_{max} (g)	$0.65(a_{max}/g)$	σ'_{vo} (kPa)	σ_{vo} (psf)	σ_{vo} (kPa)	r_d	MSF	CSR
2F-94-15	22	6.71	0.42	0.2730	114.15	2634	126.12	0.948702	1.44	0.199
2F-94-15	24	7.32	0.42	0.2730	121.19	2906	139.14	0.944039	1.44	0.205
2F-94-15	26	7.92	0.42	0.2730	128.24	3178	152.16	0.939375	1.44	0.211
0	0	0.00	0.42	0.2730	0.00		0.00		#DIV/0!	#DIV/0!
2F-03-08	25	7.62	0.42	0.2730	140.27	2992	143.26	0.941707	1.44	0.182
2F-03-08	27	8.23	0.42	0.2730	147.32	3264	156.28	0.937044	1.44	0.188
2F-03-08	29	8.84	0.42	0.2730	154.37	3536	169.30	0.93238	1.44	0.194
2F-03-08	31	9.45	0.42	0.2730	161.41	3808	182.33	0.921717	1.44	0.197
0	0	0.00	0.42	0.2730	0.00		0.00		#DIV/0!	#DIV/0!
0	0	0.00	0.42	0.2730	0.00		0.00		#DIV/0!	#DIV/0!
0	0	0.00	0.42	0.2730	0.00		0.00		#DIV/0!	#DIV/0!
0	0	0.00	0.42	0.2730	0.00		0.00		#DIV/0!	#DIV/0!
0	0	0.00	0.42	0.2730	0.00		0.00		#DIV/0!	#DIV/0!

$r_d = 1.0 - 0.00765z$ for $z <= 9.15m$

$r_d = 1.174 - 0.0267z$ for $9.15m > z <= 23$ m

APPENDIX 3: SOIL AND MATERIAL PROPERTY PROFILES

Napa Contract 2 West Vertical Wall

Station 2+00 to 4 +75 Soil Profile (Napa Mill Cut Wall Section)

Depth	Thickness	Soil type	Unit Weights	Undrained Shear	Drained Shear R Strength	Drained Shear S strength	
0'	11'	Sandy Clay	Moist = 115 pcf Sat = 120 pcf	C = 1400 psf Phi = 0	C = 250 psf Phi = 15	C = 0 psf Phi = 32	EL. 15
10'	6'	Fat Clay GWT 13'	Moist = 112 pcf Sat = 120 pcf	C = 500 psf Phi = 10	C = 500 psf Phi = 10	C = 0 psf Phi = 27	EL. 4
16'	4'	Sandy Clay	Moist = 115 Sat = 120 pcf	C = 1200 Phi = 0	C = 250 psf Phi = 15	C = 0 psf Phi = 32	EL. -2
20'	20'	Clayey Sand & Gravel	Moist = 128 pcf Sat = 136 pcf	C = 0 Phi = 33	C = 0 Phi = 33	C = 0 Phi = 33	EL. -6
40'	12'	Fat Clay	See above	See above	See above	See above	EL. -26
52'	12'	Clayey Sand & gravel	Moist = 128 pcf Sat = 136 pcf	C = 0 Phi = 33	Phi = 33	Phi = 33	EL. -38
64' 70'	6'	Lean Clay	Moist = 115 Sat = 120 pcf	C = 1200 Phi = 0	C = 250 psf Phi = 15	C = 0 psf Phi = 32	EL. -50 EL. -56

Napa Contract 2 West Vertical Wall

Station 4+75 to 9+30 Soil Profile (Fifth Street to Third Street)

Depth	Thickness	Soil Type	Unit Weights	Undrained Shear	Drained Shear R strength	Drained Shear S Strength	
0'	20'	Sandy Clay GWT 14'	Moist = 121 pcf Sat = 124 pcf	C = 1400 psf Phi = 0	C = 220 psf Phi = 18	C = 0 psf Phi = 32	EL. 16
20'	30'	Clayey Sand & Gravel	Moist = 128 pcf Sat = 136 pcf	C = 0 Phi = 33	C = 0 Phi = 33	C = 0 Phi = 33	EL. -4
50'	12'	Sandy Clay	Moist = 115 pcf Sat = 120 pcf	C = 1200 psf Phi = 0	C = 250 psf Phi = 15	C = 0 psf Phi = 32	EL. -34
62'	13'	Clayey Sand & Gravel	Moist = 128 pcf Sat = 136 pcf	C = 0 Phi = 33	C = 0 Phi = 33	C = 0 Phi = 33	EL. -46
75' 80'	5'	Lean Clay	Moist = 115 pcf Sat = 120 pcf	C = 1200 psf Phi = 0	C = 500 psf Phi = 15	C = 0 psf Phi = 28	EL. -59 EL. -64

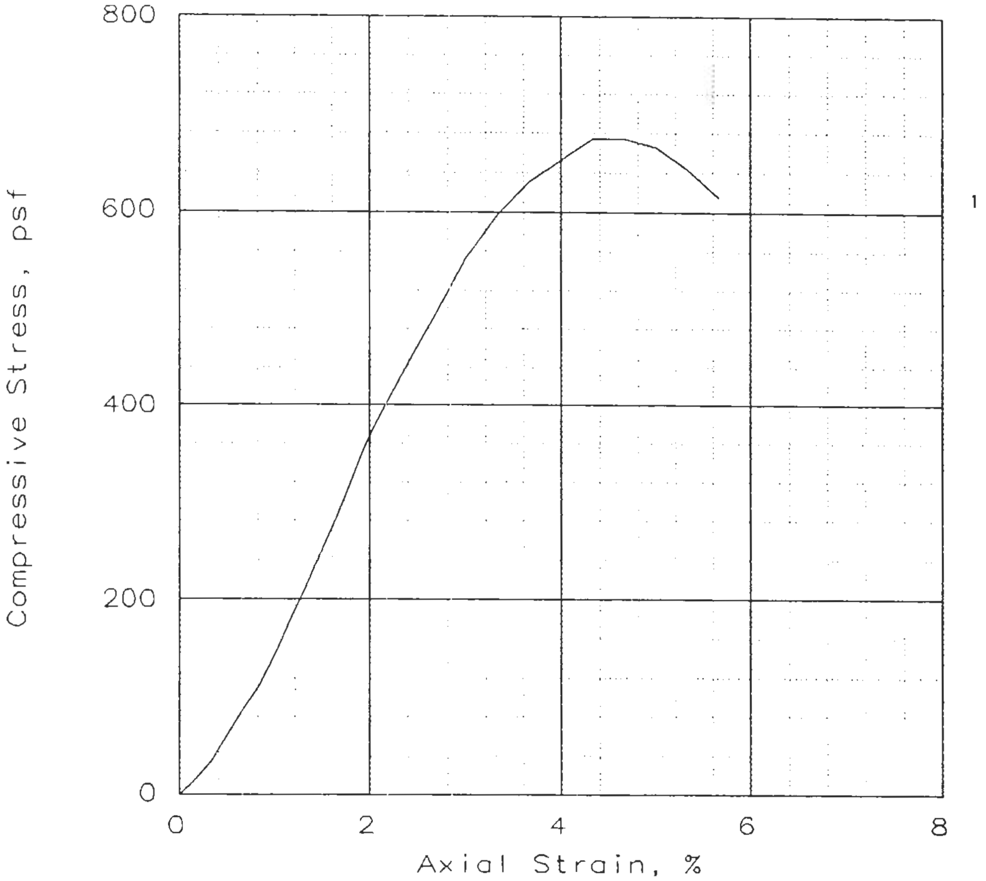
Napa 2 West Vertical Wall

Station 9+30 to U/S End of Wall (Third Street to First Street)

Depth	Thickness	Soil Type	Unit Weights	Undrained Shear	Drained Shear R Strength	Drained Shear S Strength	
0'	22'	Sandy Clay *** GWT 20'	Moist = 119 pcf Saturated = 123 pcf	C = 800 psf Phi = 0	C = 100 psf Phi = 15	C = 0 psf Phi = 30	EL.17
22'	8'	Clayey Gravel & Sand	Moist = 128 pcf Sat = 136 pcf	C = 0 Phi = 33	C = 0 Phi = 33	C = 0 Phi = 33	EL. -5
30'	19'	Fat Clay	Moist = 121 pcf Saturated = 125 pcf	C = 600 psf Phi = 0	C = 500 psf Phi = 10	C = 0 psf Phi = 27	EL. -13
49'	17'	Lean Clay	Moist = 122 pcf Saturated = 125 pcf	C = 1200 pcf Phi = 0	C = 500 psf Phi = 15	C = 0 psf Phi = 28	EL. -32
66'	9'	Clayey gravel & Sand	Moist = 128 pcf Sat = 136 pcf	C = 0 Phi = 33	C = 0 Phi = 33	C = 0 Phi = 33	EL. -49
75' 80'	5'	Fat Clay	See above	See above	See above	See above	EL. -58 EL. -63

APPENDIX 4: UNCONFINED COMPRESSION, TRIAXIAL, AND CONSOLIDATION TEST RESULTS

UNCONFINED COMPRESSION TEST



SAMPLE NO.:	1			
Unconfined strength, psf	675			
Undrained shear strength, psf	337			
Failure strain, %	4.3			
Strain rate, %/min	0.30			
Water content, %	26.2			
Wet density, pcf	124.3			
Dry density, pcf	98.5			
Saturation, %	99.4			
Void ratio	0.7111			
Specimen diameter, in	2.83			
Specimen height, in	6.00			
Height/diameter ratio	2.12			

Description: silty SAND

LL = 22	PL = 19	PI = 3	GS= 2.7	Type: Undisturbed

Project No.: 03-153
Date: 3-19-03
Remarks:

Fig. No.: ____

Client: USACE

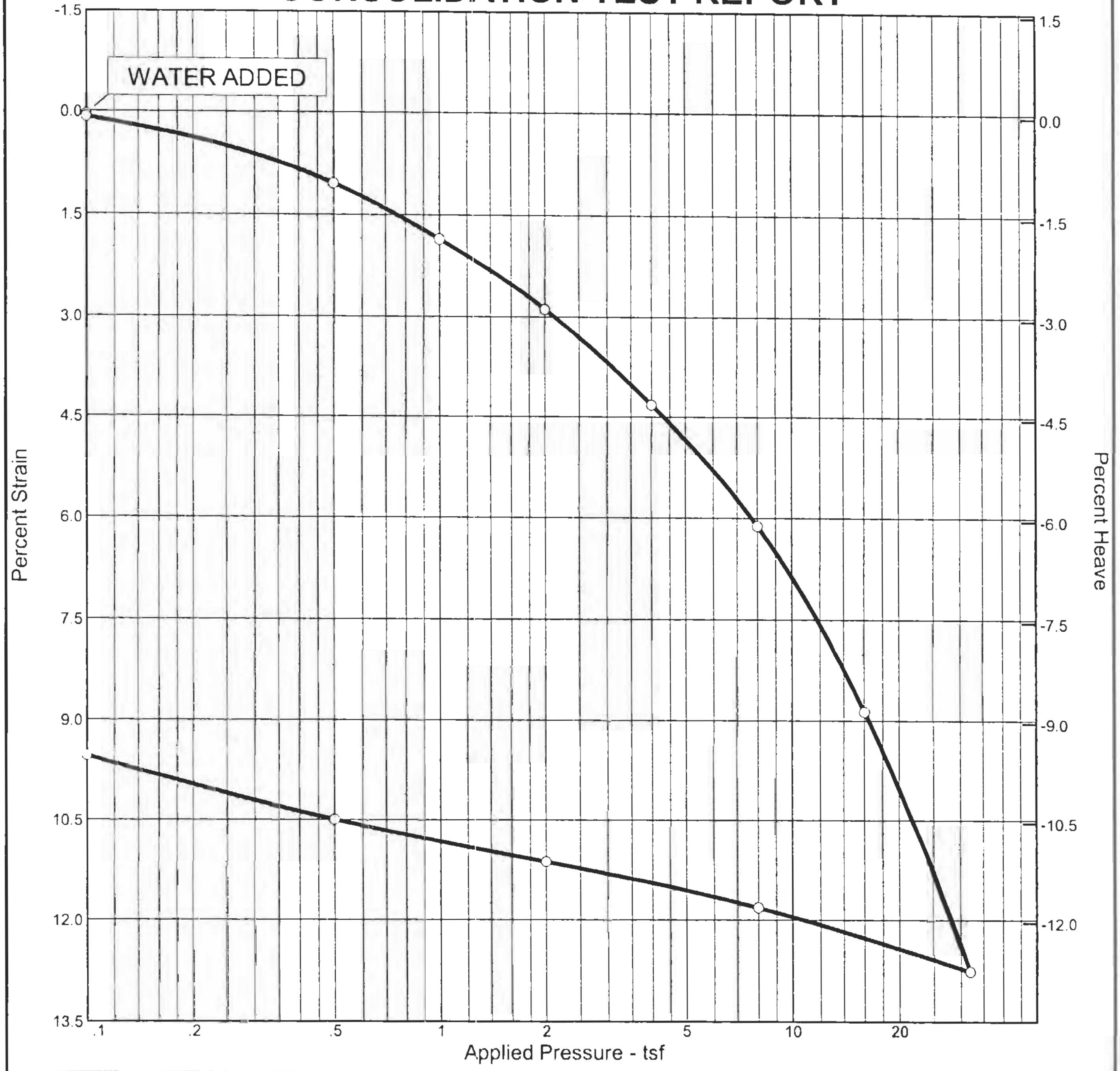
Project: Napa River Flood Protection

Location: 2F-03-05 Tube 1
@ 13.0'-15.5'

UNCONFINED COMPRESSION TEST

SIERRA TESTING LABORATORIES, INC.

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_s	Swell Press. (tsf)	Heave %	e_o
Sat.	Moist.											
102.5 %	30.0 %	93.1	22	3	2.65		12.01	0.23	0.02		-0.0	0.777

MATERIAL DESCRIPTION	USCS	AASHTO
Silty sand	SM	A-2-4(0)

Project No. 03-153 **Client:** U.S. Army Corps of Engineers, Sacramento District

Project: Napa River Flood Protection, Contract 2 West

Location: 2F-03-05

Remarks:

CONSOLIDATION TEST REPORT

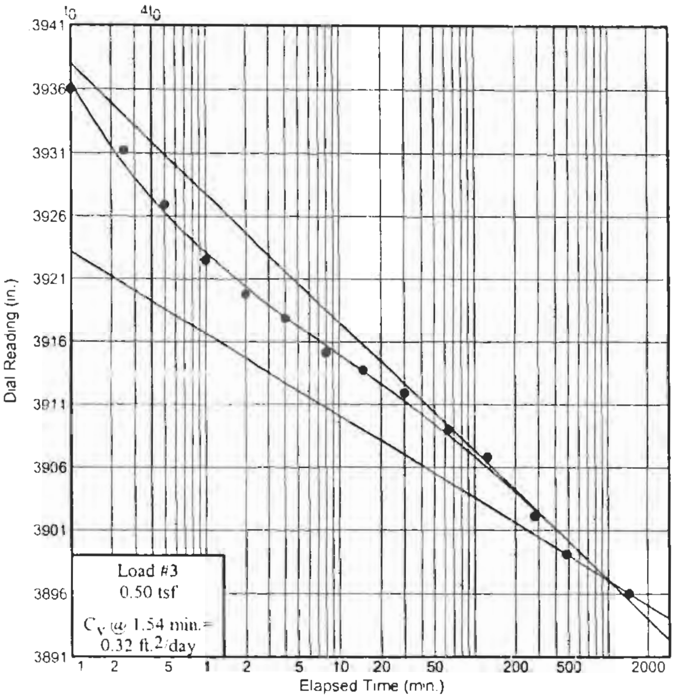
SIERRA TESTING LABS, INC.

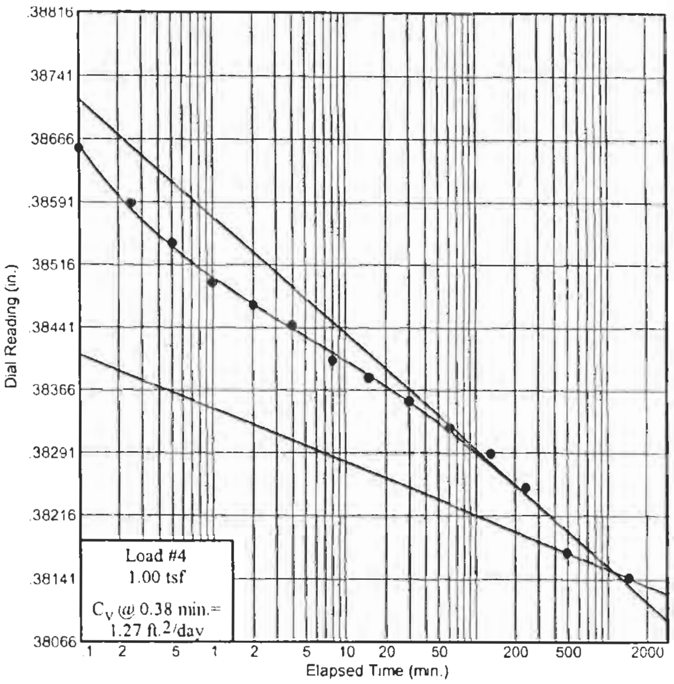
Figure

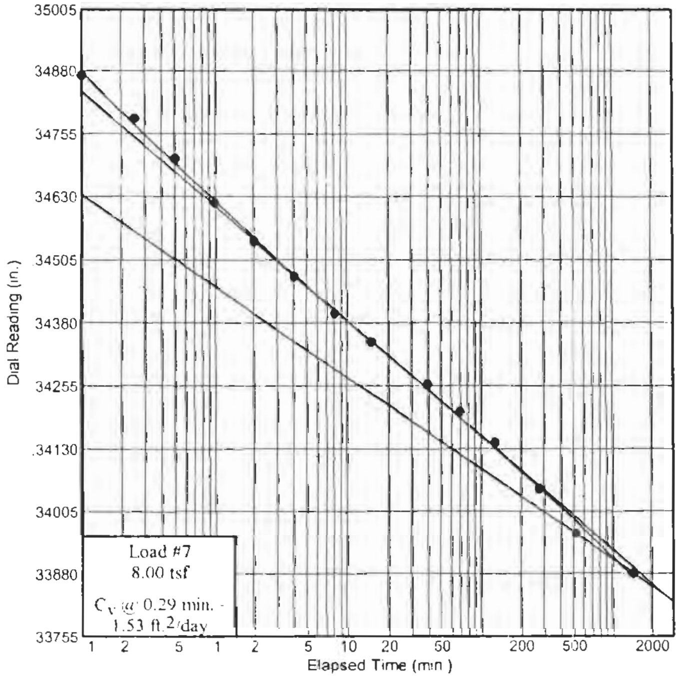
Dial Reading vs. Time

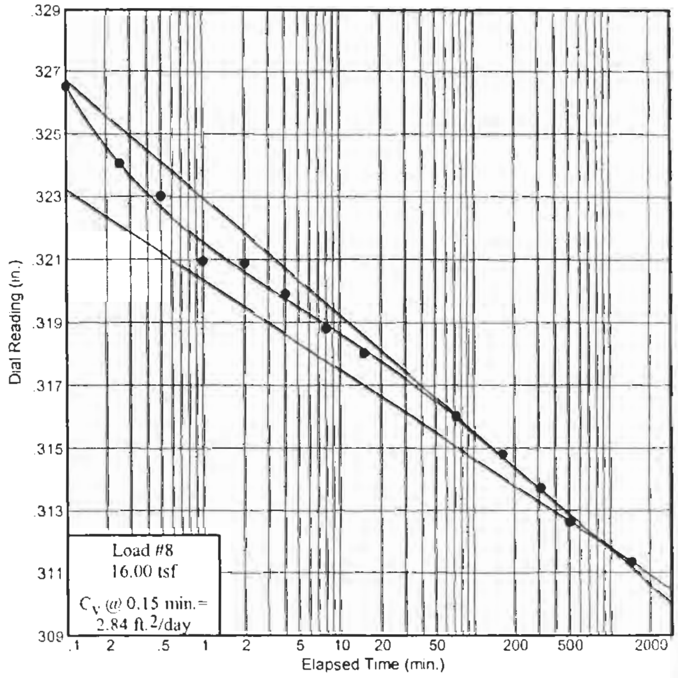
Project No.: 03-153
Project: Napa River Flood Protection, Contract 2 West

Location: 2F-03-05









Dial Reading vs. Time

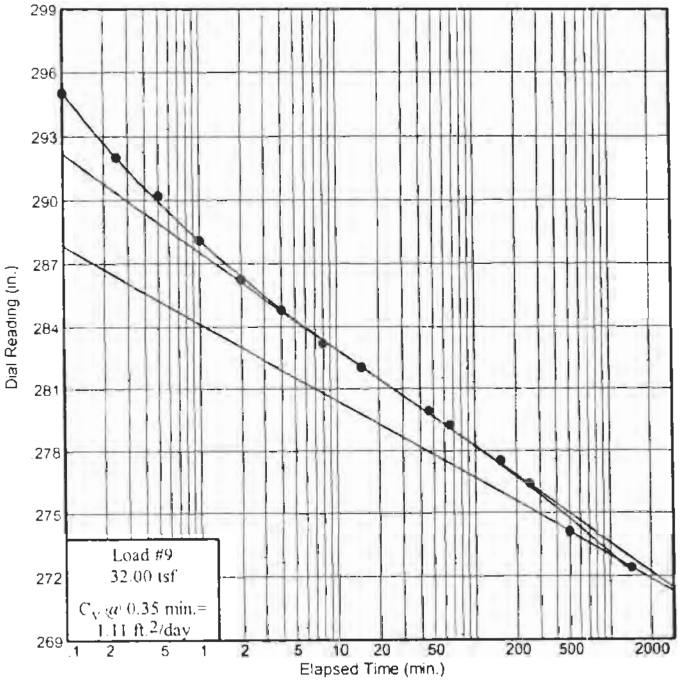
SIERRA TESTING LABS, INC.

Figure

Dial Reading vs. Time

Project No.: 03-153
Project: Napa River Flood Protection, Contract 2 West

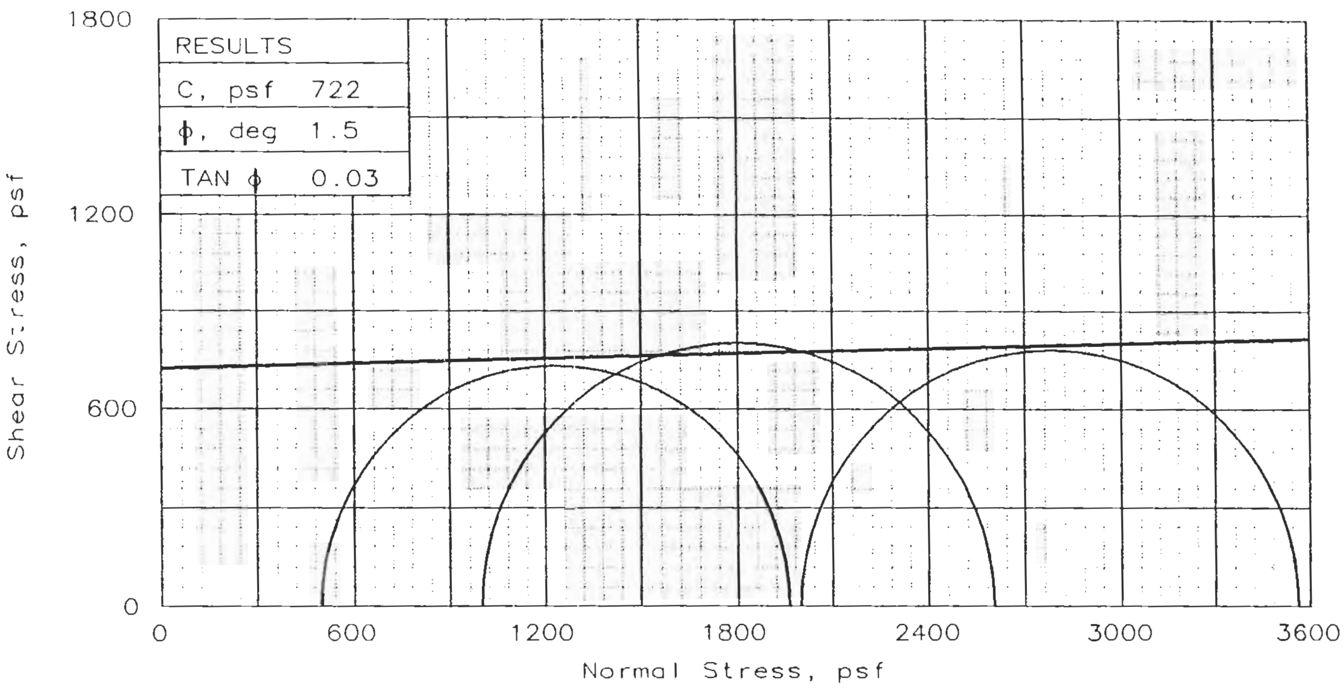
Location: 2F-03-05

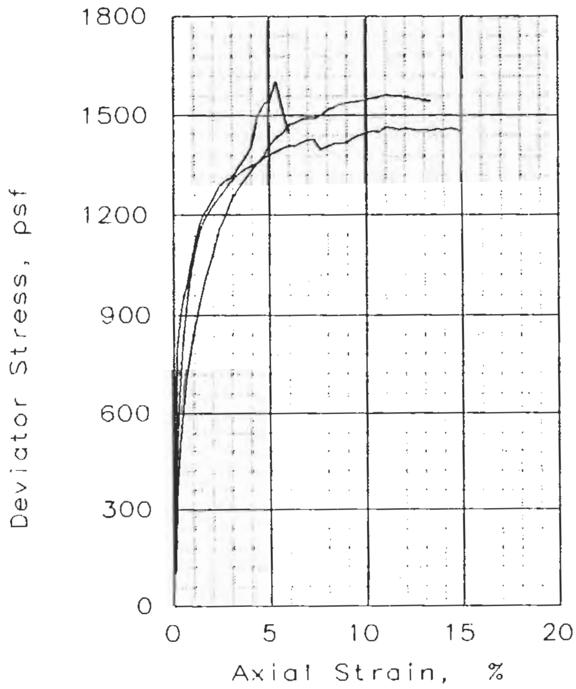


Dial Reading vs. Time

SIERRA TESTING LABS, INC.

Figure





	SAMPLE NO.:	1	2	3	
INITIAL	WATER CONTENT, % 27.03	27.5	27.1	26.5	AVG. 27
	DRY DENSITY, pcf 95.83	95.0	95.3	97.2	95.8
	SATURATION, %	96.0	95.3	97.4	
	VOID RATIO	0.773	0.768	0.734	
	DIAMETER, in	2.87	2.87	2.87	
	HEIGHT, in	6.00	6.00	6.00	
AT TEST	WATER CONTENT, %	28.3	27.8	26.9	
	DRY DENSITY, pcf	95.5	96.3	97.7	
	SATURATION, %	100.0	100.0	100.0	
	VOID RATIO	0.765	0.750	0.725	
	DIAMETER, in	2.87	2.86	2.87	
	HEIGHT, in	5.99	5.98	5.99	
	Strain rate, %/min	0.30	0.30	0.30	
	BACK PRESSURE, psf	0	0	0	
	CELL PRESSURE, psf	500	999	2000	
	FAIL. STRESS, psf	1464	1604	1562	
	ULT. STRESS, psf				
	σ_1 FAILURE, psf	1963	2603	3562	
	σ_3 FAILURE, psf	500	999	2000	

TYPE OF TEST:
Unconsolidated Undrained
SAMPLE TYPE: Undisturbed
DESCRIPTION: Pending

LL= 27 PL= 19 PI= 8
SPECIFIC GRAVITY= 2.7
REMARKS: Samples @ 8.2'-8.7', 8.7'-9.2', 9.2'-9.7'

Fig. No.: ______

CLIENT: USACE

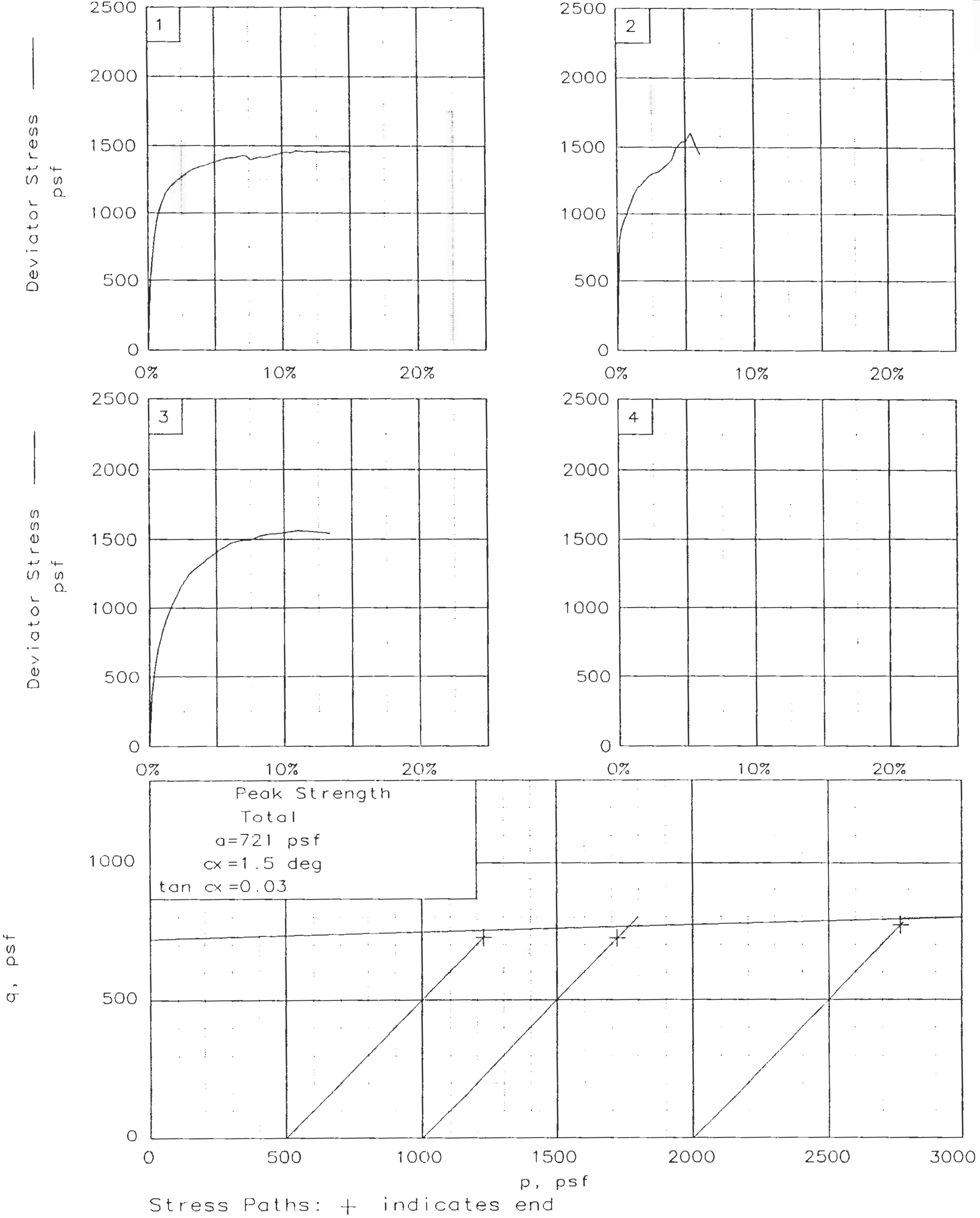
PROJECT: Napa River Flood Protection

SAMPLE LOCATION: 2F-03-06

PROJ. NO.: 03-153 DATE: 3-19-03

TRIAXIAL SHEAR TEST REPORT

SIERRA TESTING LABORATORIES, INC.

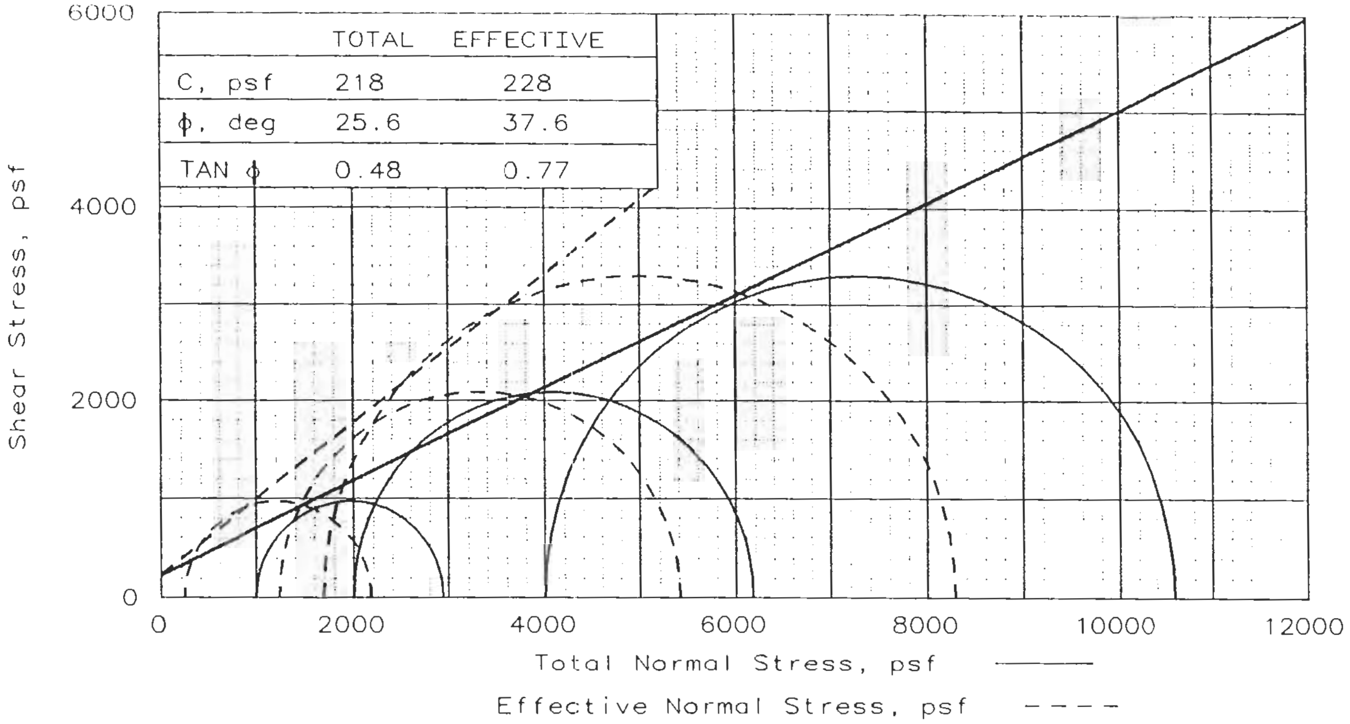


Client: USACE

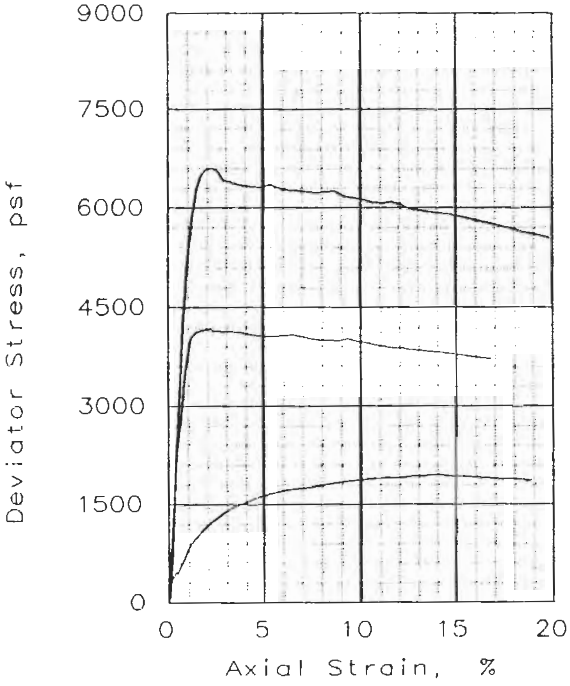
Project: Napa River Flood Protection

Location: 2F-03-06

File: 03-153B Project No.: 03-153 Fig. No.: ____



	TOTAL	EFFECTIVE
C, psf	218	228
ϕ, deg	25.6	37.6
TAN ϕ	0.48	0.77



	SAMPLE NO.:	1	2	3	
INITIAL	WATER CONTENT, % 26.33	26.8	26.2	26.0	AVG. 26
	DRY DENSITY, pcf 96.13	94.1	96.7	97.6	96.1
	SATURATION, %	91.4	95.3	96.6	
	VOID RATIO	0.791	0.742	0.726	
	DIAMETER, in	2.85	2.85	2.85	
	HEIGHT, in	5.33	5.42	5.25	
AT TEST	WATER CONTENT, %	28.5	25.4	24.0	
	DRY DENSITY, pcf	95.2	100.0	102.3	
	SATURATION, %	100.0	100.0	100.0	
	VOID RATIO	0.770	0.685	0.648	
	DIAMETER, in	2.84	2.82	2.81	
	HEIGHT, in	5.31	5.36	5.17	
	Strain rate, %/min	0.03	0.03	0.03	
	BACK PRESSURE, psf	8640	9202	7733	
	CELL PRESSURE, psf	9639	11202	11733	
	FAIL. STRESS, psf	1944	4179	6598	
	TOTAL PORE PR., psf	9403	9965	10037	
	ULT. STRESS, psf				
	TOTAL PORE PR., psf				
	$\bar{\sigma}_1$ FAILURE, psf	2180	5416	8294	
	$\bar{\sigma}_3$ FAILURE, psf	236	1237	1696	

TYPE OF TEST:
CU with Pore Pressures
SAMPLE TYPE: Undisturbed
DESCRIPTION: Pending

LL= 31 PL= 17 PI= 14
SPECIFIC GRAVITY= 2.7
REMARKS: Depths @ 20.0-20.7', 20.7-21.2', 21.2-21.7'

Fig. No.: ______

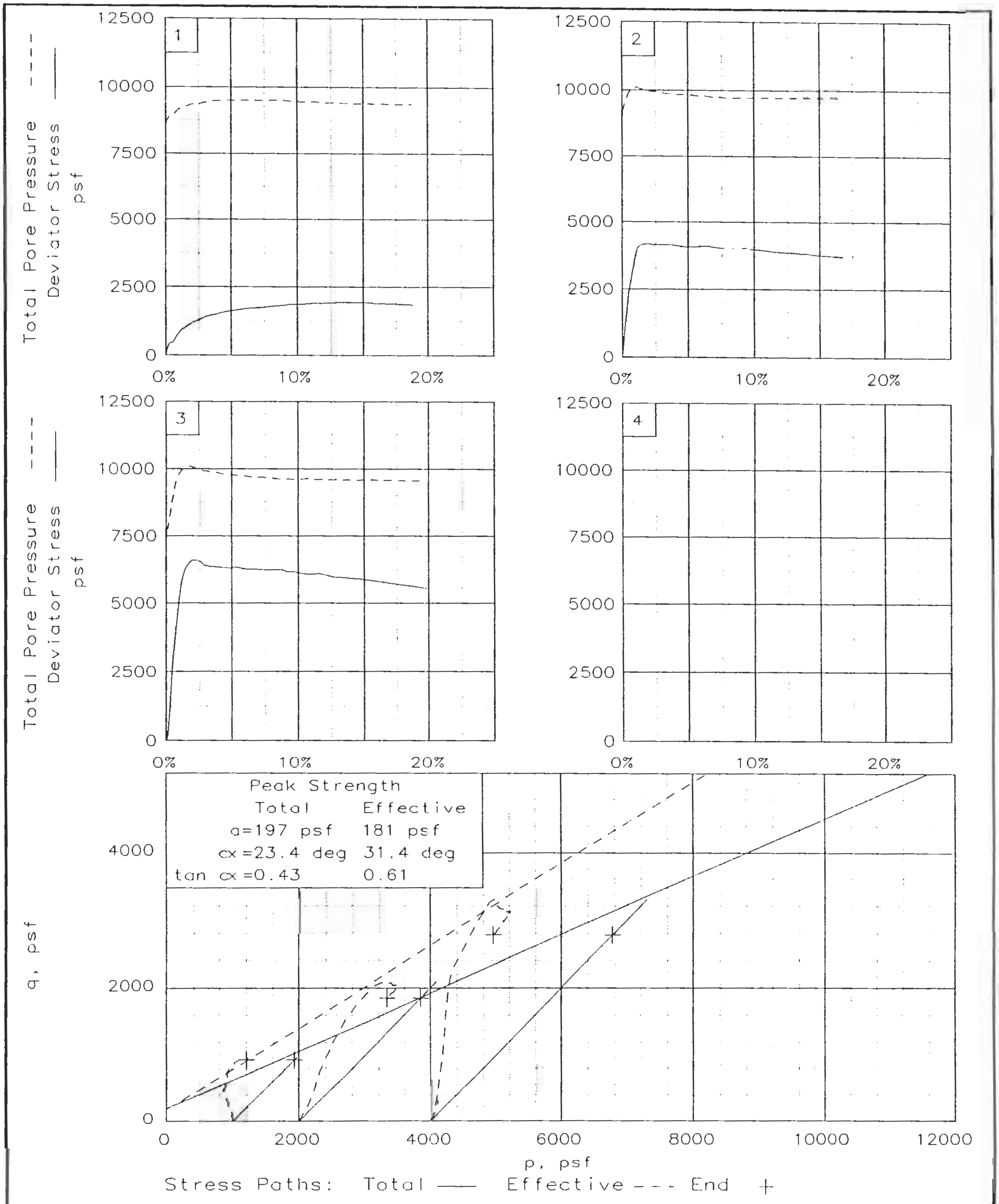
CLIENT: USACE

PROJECT: Napa River Flood Protection
Contract 2 West
SAMPLE LOCATION: 2F-03-06 Tube 2

PROJ. NO.: 03-153 DATE: 3-19-03

TRIAXIAL SHEAR TEST REPORT

SIERRA TESTING LABORATORIES, INC.

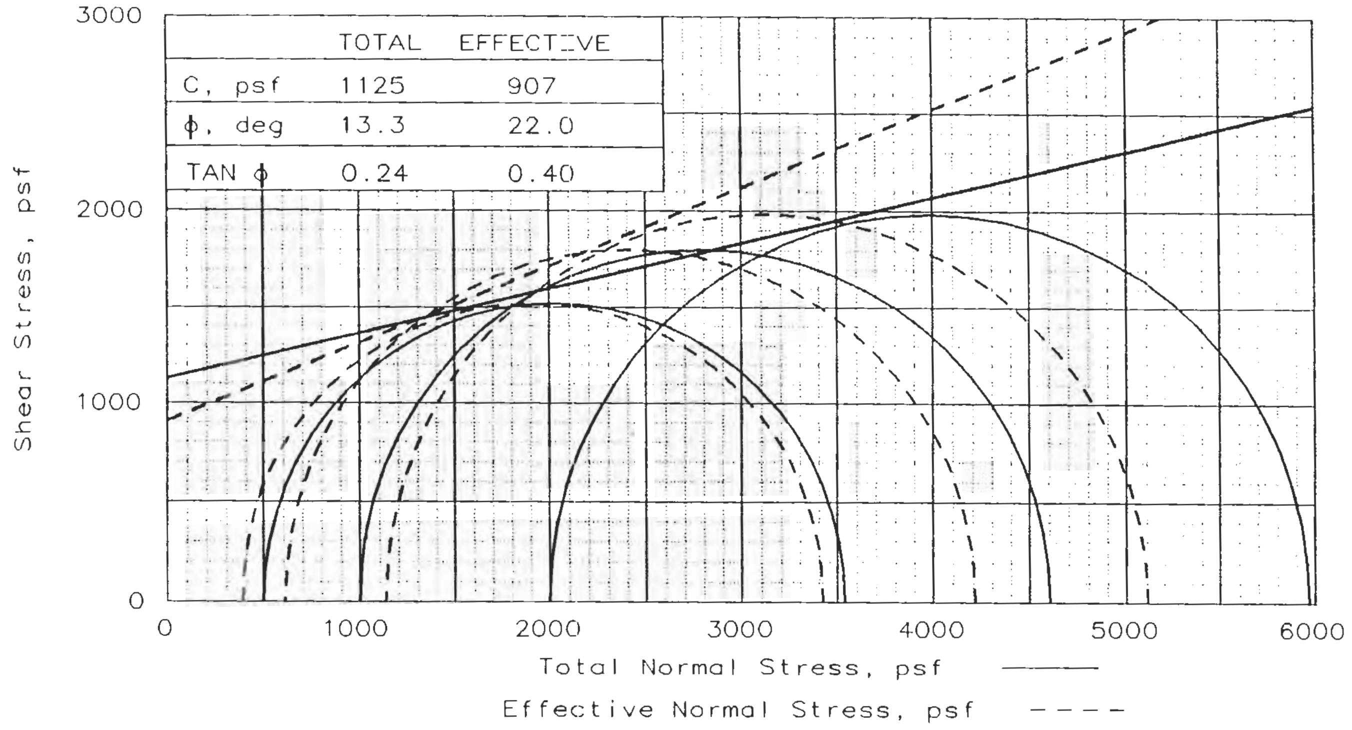


Client: USACE

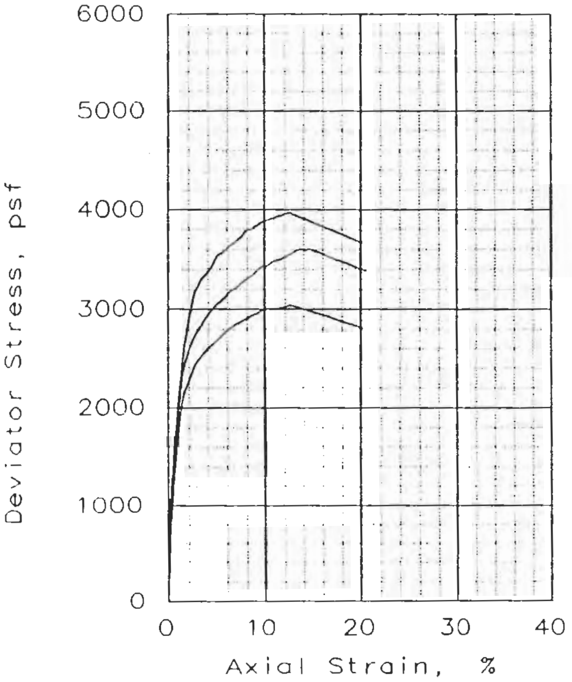
Project: Napa River Flood Protection Contract 2 West

Location: 2F-03-06 Tube 2

File: 03-153E Project No.: 03-153 Fig. No.: ____



	TOTAL	EFFECTIVE
C, psf	1125	907
ϕ, deg	13.3	22.0
TAN ϕ	0.24	0.40



SAMPLE NO.:		1	2	3	AVG
INITIAL	WATER CONTENT, % 22.3	22.0	22.4	22.5	22.~~1~~3
	DRY DENSITY, pcf 104.233	105.4	103.9	103.4	104.2
	SATURATION, %	99.4	97.1	96.3	
	VOID RATIO	0.599	0.623	0.630	
	DIAMETER, in	2.80	2.83	2.83	
	HEIGHT, in	6.00	5.15	6.00	
AT TEST	WATER CONTENT, %	21.0	22.0	22.4	
	DRY DENSITY, pcf	107.6	105.7	105.0	
	SATURATION, %	100.0	100.0	100.0	
	VOID RATIO	0.567	0.595	0.606	
	DIAMETER, in	2.78	2.81	2.82	
	HEIGHT, in	5.96	5.12	5.97	
Strain rate, %/min		0.03	0.03	0.03	
BACK PRESSURE, psf		9317	9302	9331	
CELL PRESSURE, psf		9816	10302	11331	
FAIL. STRESS, psf		3040	3606	3969	
TOTAL PORE PR., psf		9432	9691	10195	
ULT. STRESS, psf					
TOTAL PORE PR., psf					
$\bar{\sigma}_1$ FAILURE, psf		3424	4217	5105	
$\bar{\sigma}_3$ FAILURE, psf		384	611	1136	

TYPE OF TEST:
CU with Pore Pressures
SAMPLE TYPE: Undisturbed
DESCRIPTION: Pending

LL= 42 PL= 37 PI= 5
SPECIFIC GRAVITY= 2.7
REMARKS: Depths @ 8.8-9.3', 8.3-8.8', 9.3-9.8'

Fig. No.: ______

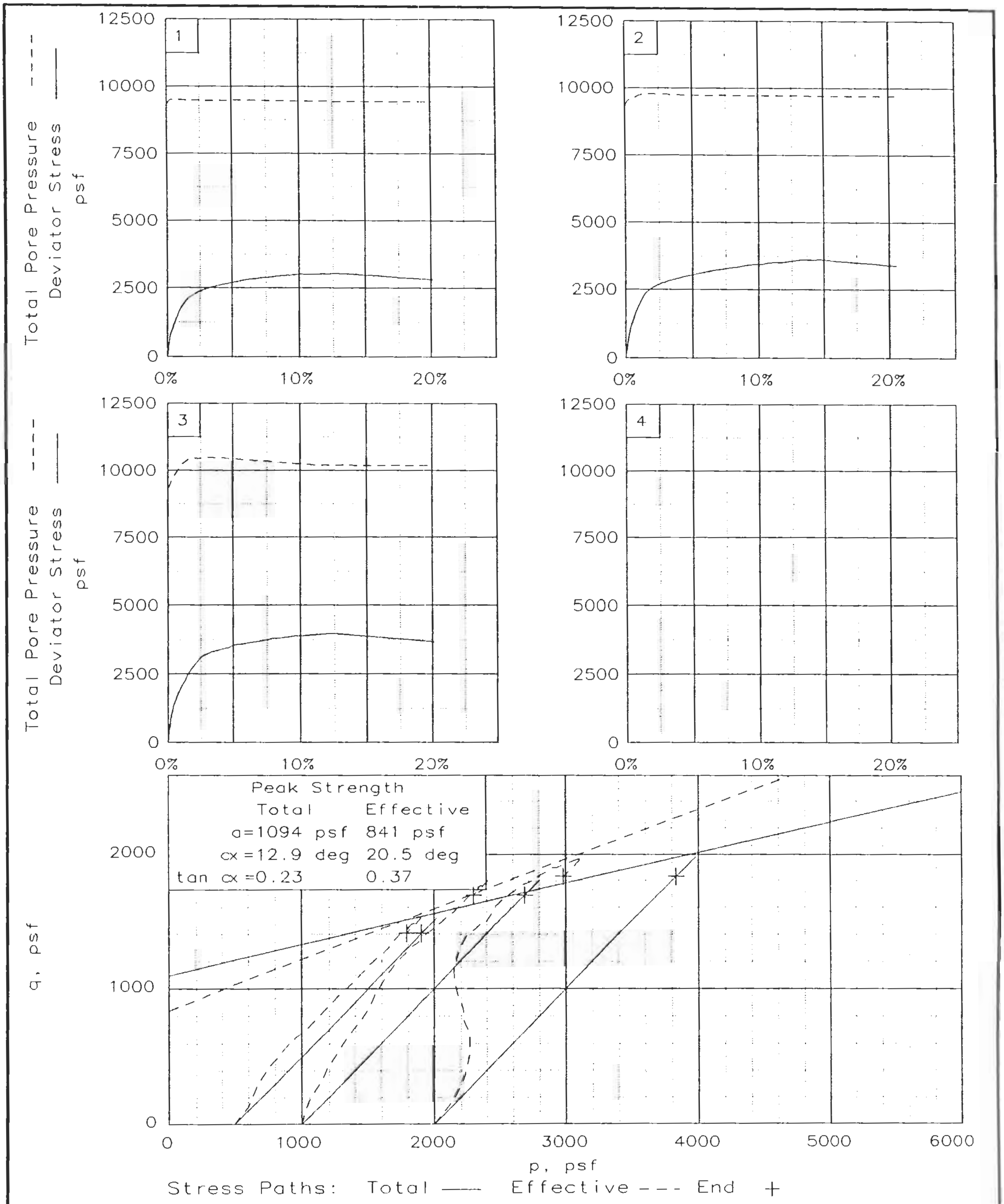
CLIENT: USACE

PROJECT: Napa River Flood Protection
Contract 2 West
SAMPLE LOCATION: 2F-03-07 T1

PROJ. NO.: 03-153 DATE: 3-19-02

TRIAXIAL SHEAR TEST REPORT

SIERRA TESTING LABORATORIES, INC.

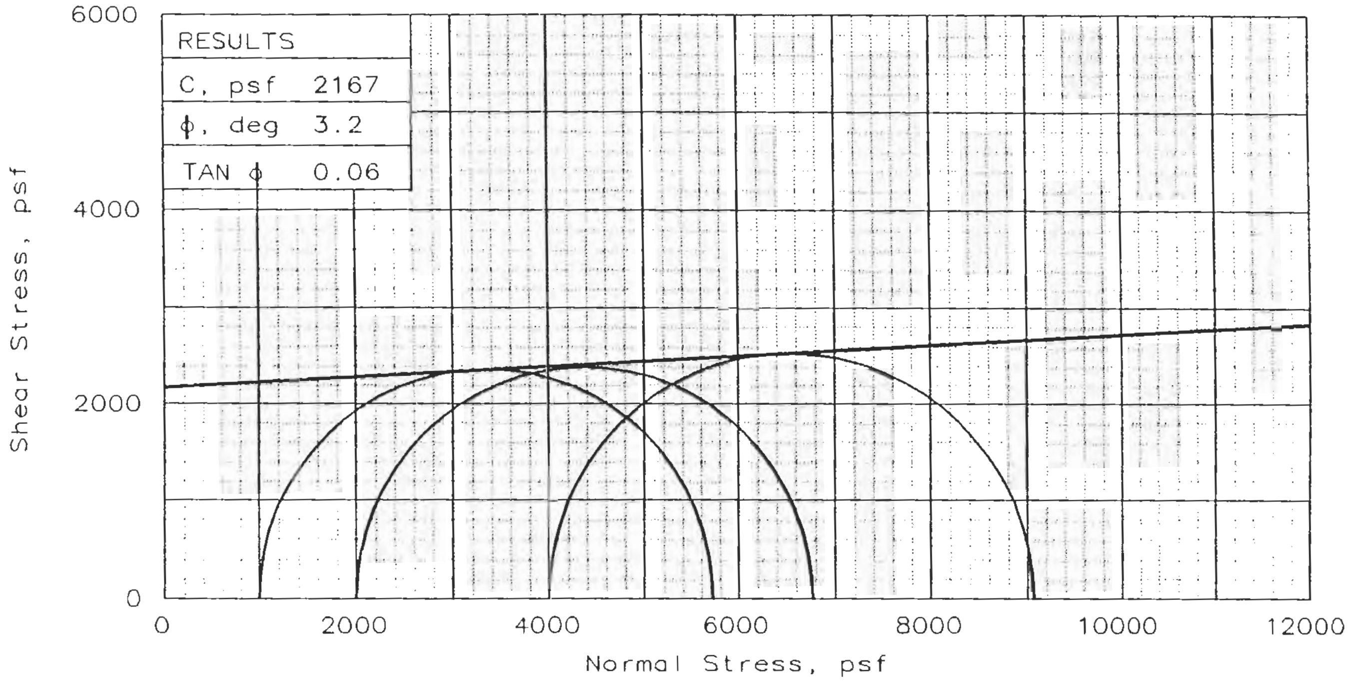


Client: USACE

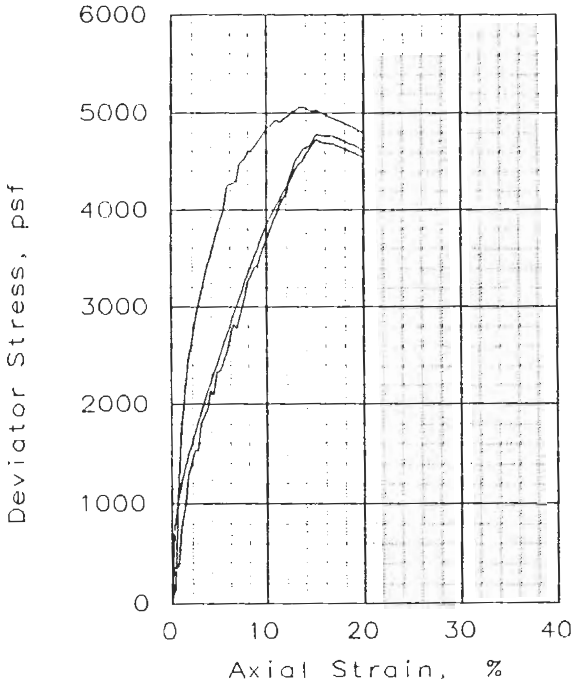
Project: Napa River Flood Protection Contract 2 West

Location: 2F-03-07 T1

File: 03-153F Project No.: 03-153 Fig. No.: ____



RESULTS	
C, psf	2167
ϕ, deg	3.2
TAN ϕ	0.06



	SAMPLE NO.:	1	2	3	AVG
INITIAL	WATER CONTENT, % 20.43	20.8	20.8	19.7	20
	DRY DENSITY, pcf 106.93	105.2	107.5	108.1	106.9
	SATURATION, %	93.3	98.9	95.0	
	VOID RATIO	0.602	0.568	0.559	
	DIAMETER, in	2.62	2.62	2.65	
	HEIGHT, in	6.00	6.00	6.00	
AT TEST	WATER CONTENT, %	22.0	20.4	20.1	
	DRY DENSITY, pcf	105.7	108.6	109.2	
	SATURATION, %	100.0	100.0	100.0	
	VOID RATIO	0.594	0.552	0.544	
	DIAMETER, in	2.62	2.61	2.64	
	HEIGHT, in	5.99	5.98	5.98	
	Strain rate, %/min	0.30	0.30	0.30	
	BACK PRESSURE, psf	0	0	0	
	CELL PRESSURE, psf	999	2000	4000	
	FAIL. STRESS, psf	4726	4773	5062	
	ULT. STRESS, psf				
	σ_1 FAILURE, psf	5725	6773	9062	
	σ_3 FAILURE, psf	999	2000	4000	

TYPE OF TEST:
Unconsolidated Undrained
SAMPLE TYPE: Undisturbed
DESCRIPTION: Pending

LL= 45 PL= 22 PI= 23
SPECIFIC GRAVITY= 2.7
REMARKS: Depths @ 20.3-20.8', 20.8-21.3', 21.3-21.8'

Fig. No.: ______

CLIENT: USACE

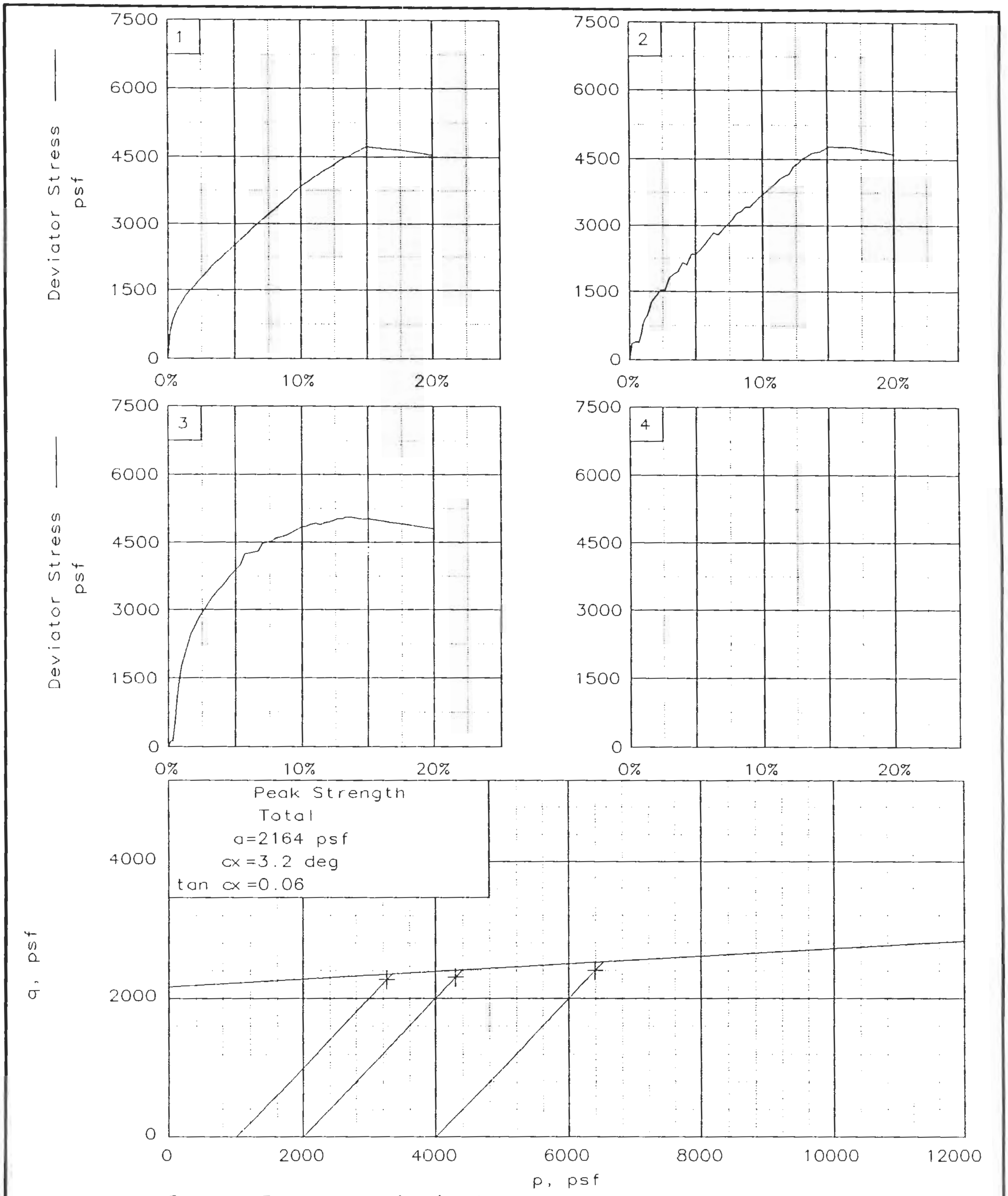
PROJECT: Napa River Flood Protection

SAMPLE LOCATION: 2F-03-07 Tube 2

PROJ. NO.: 03-153 DATE: 3-19-03

TRIAXIAL SHEAR TEST REPORT

SIERRA TESTING LABORATORIES, INC.



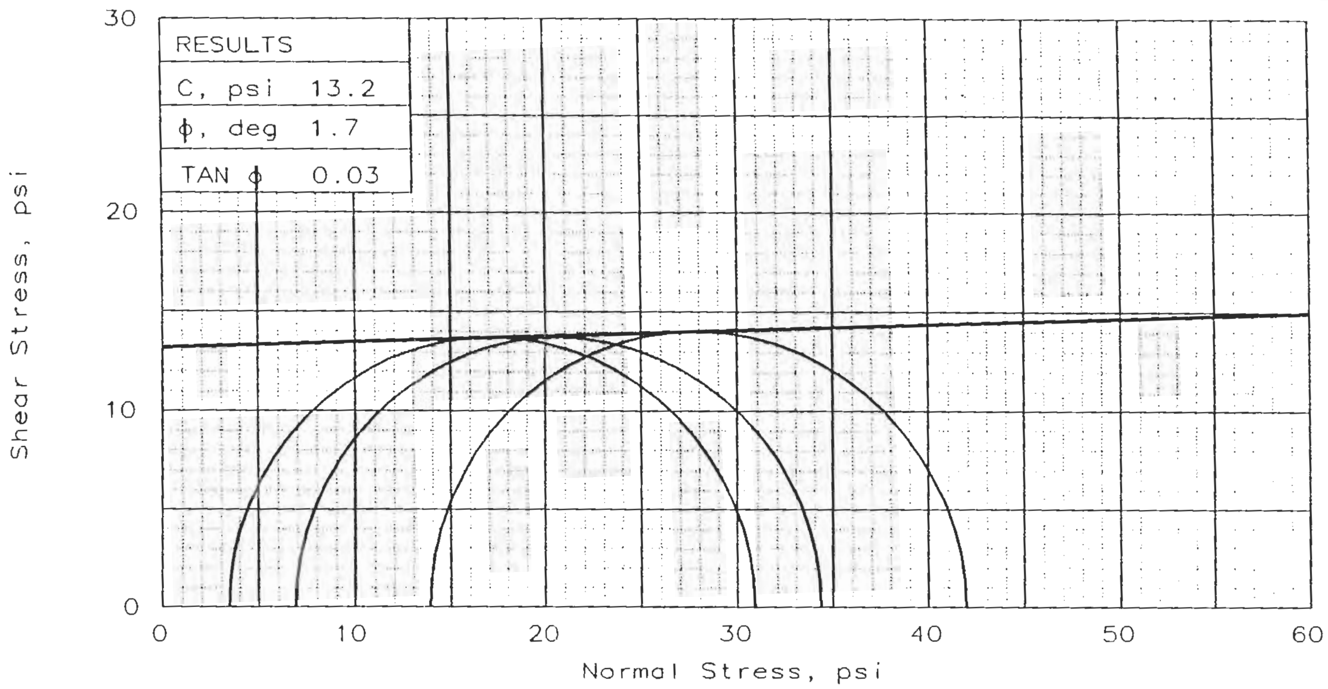
Stress Paths: + indicates end

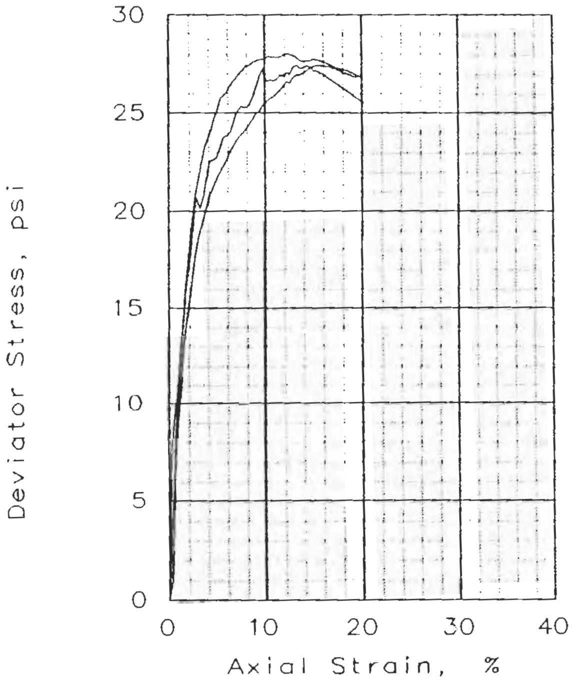
Client: USACE

Project: Napa River Flood Protection

Location: 2F-03-07 Tube 2

File: 03-153D Project No.: 03-153 Fig. No.: ____





	SAMPLE NO.:	1	2	3	AVG
INITIAL	WATER CONTENT, % 27.967	28.6	27.9	27.4	28
	DRY DENSITY, pcf 95.067	93.4	95.0	96.8	95.1
	SATURATION, %	96.0	97.3	99.7	
	VOID RATIO	0.804	0.774	0.741	
	DIAMETER, in	2.85	2.85	2.85	
	HEIGHT, in	5.72	5.75	6.00	
AT TEST	WATER CONTENT, %	29.4	28.0	27.1	
	DRY DENSITY, pcf	93.9	96.0	97.3	
	SATURATION, %	100.0	100.0	100.0	
	VOID RATIO	0.795	0.755	0.733	
	DIAMETER, in	2.85	2.84	2.85	
	HEIGHT, in	5.71	5.73	5.99	
	Strain rate, %/min	0.30	0.30	0.30	
	BACK PRESSURE, psi	0.0	0.0	0.0	
	CELL PRESSURE, psi	3.5	6.9	13.9	
	FAIL. STRESS, psi	27.4	27.4	28.0	
	ULT. STRESS, psi				
	σ_1 FAILURE, psi	30.9	34.4	41.9	
	σ_3 FAILURE, psi	3.5	6.9	13.9	

TYPE OF TEST:
Unconsolidated Undrained

SAMPLE TYPE: Undisturbed

DESCRIPTION: Pending

LL= 31 PL= 18 PI= 13

SPECIFIC GRAVITY= 2.77

REMARKS: Depths @ 12.3-12.8', 12.8-13.3', 13.3-13.8'

Fig. No.: ______

CLIENT: USACE

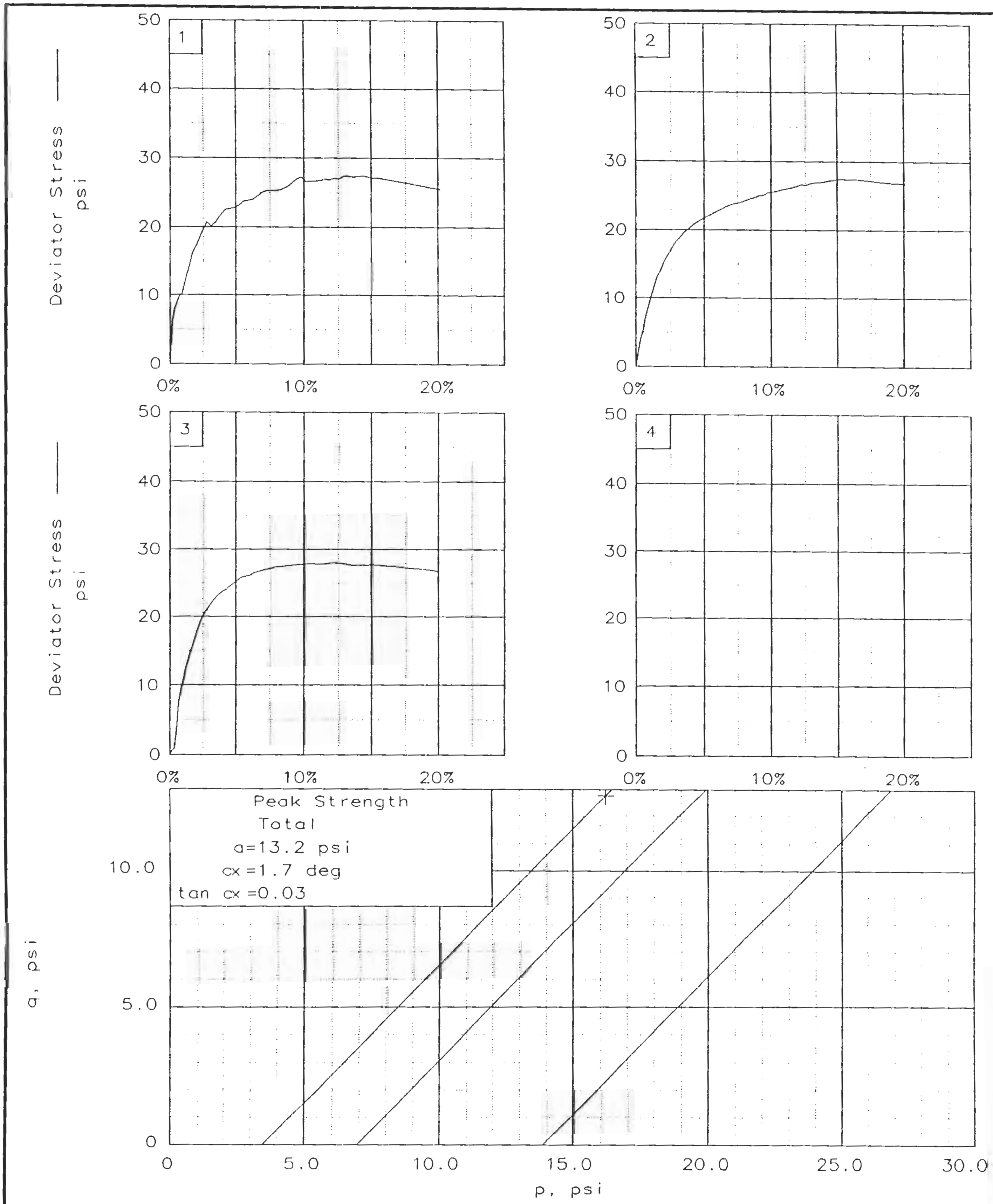
PROJECT: Napa River Flood Protection

SAMPLE LOCATION: 2F-03-08 Tube 1

PROJ. NO.: 03-153 DATE: 3-19-03

TRIAXIAL SHEAR TEST REPORT

SIERRA TESTING LABORATORIES, INC.



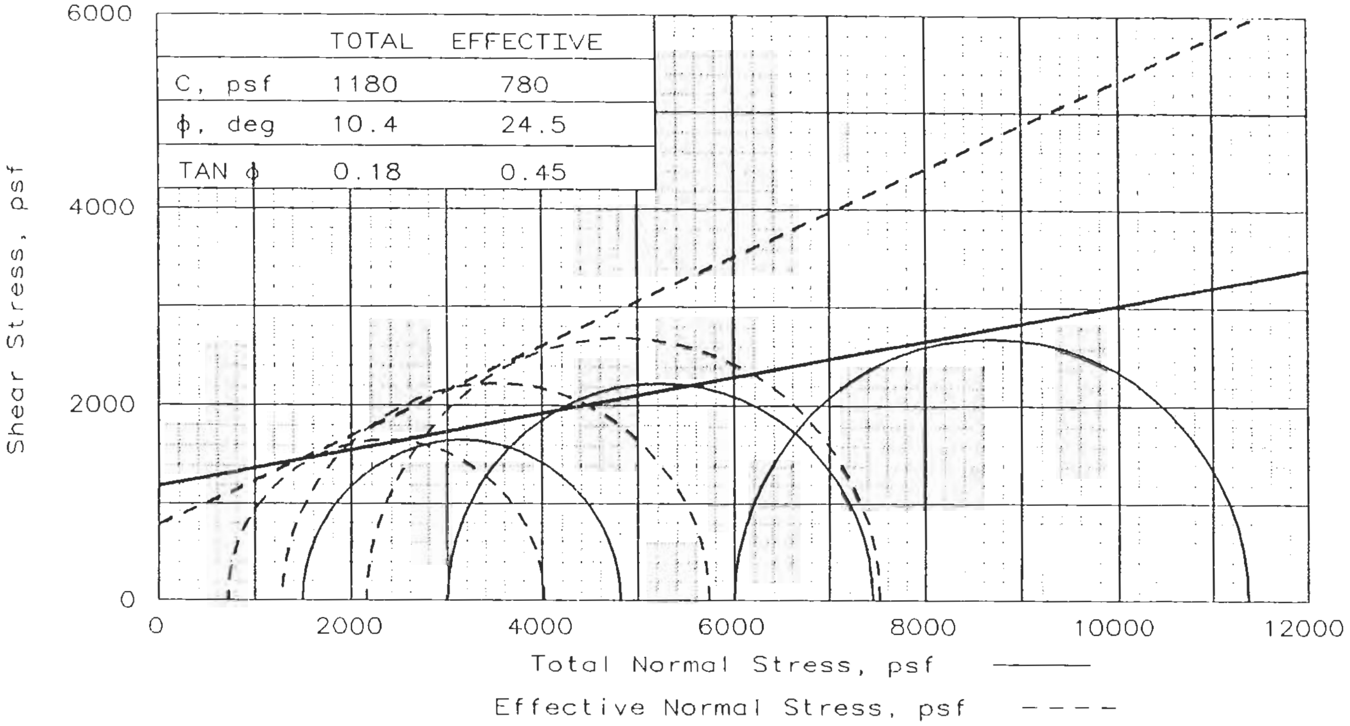
Stress Paths: + indicates end

Client: USACE

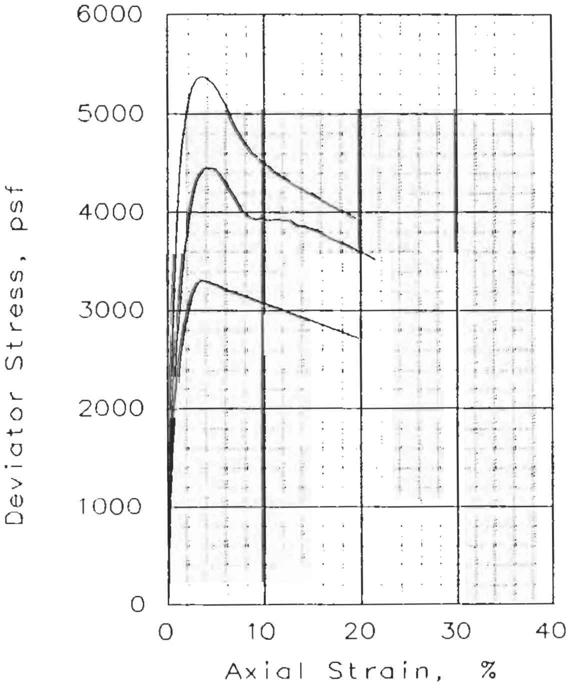
Project: Napa River Flood Protection

Location: 2F-03-08 Tube 1

File: 03-153C Project No.: 03-153 Fig. No.: ____



	TOTAL	EFFECTIVE
C, psf	1180	780
ϕ, deg	10.4	24.5
TAN ϕ	0.18	0.45



	SAMPLE NO.:	1	2	3	AVG
INITIAL	WATER CONTENT, % 25.03	25.1	24.5	25.5	25
	DRY DENSITY, pcf 97.567	99.4	96.4	96.9	97.6
	SATURATION, %	97.4	88.3	93.3	
	VOID RATIO	0.696	0.748	0.739	
	DIAMETER, in	2.80	2.75	2.83	
	HEIGHT, in	5.61	5.20	6.00	
AT TEST	WATER CONTENT, %	24.4	24.7	24.2	
	DRY DENSITY, pcf	101.6	101.1	102.0	
	SATURATION, %	100.0	100.0	100.0	
	VOID RATIO	0.659	0.668	0.653	
	DIAMETER, in	2.78	2.71	2.78	
	HEIGHT, in	5.57	5.12	5.90	
	Strain rate, %/min	0.03	0.03	0.03	
	BACK PRESSURE, psf	9317	9331	6278	
	CELL PRESSURE, psf	10817	12331	12279	
	FAIL. STRESS, psf	3310	4453	5371	
	TOTAL PORE PR., psf	10109	11045	10123	
	ULT. STRESS, psf				
	TOTAL PORE PR., psf				
	$\bar{\sigma}_1$ FAILURE, psf	4019	5739	7527	
	$\bar{\sigma}_3$ FAILURE, psf	708	1286	2156	

TYPE OF TEST:
CU with Pore Pressures

SAMPLE TYPE: Undisturbed

DESCRIPTION: Pending

LL= 51 PL= 26 PI= 25

SPECIFIC GRAVITY= 2.7

REMARKS: Depths @ 36.3-36.8', 36.8-37.3', 37.3-37.8'

Fig. No.: ______

CLIENT: USACE-Sacramento District

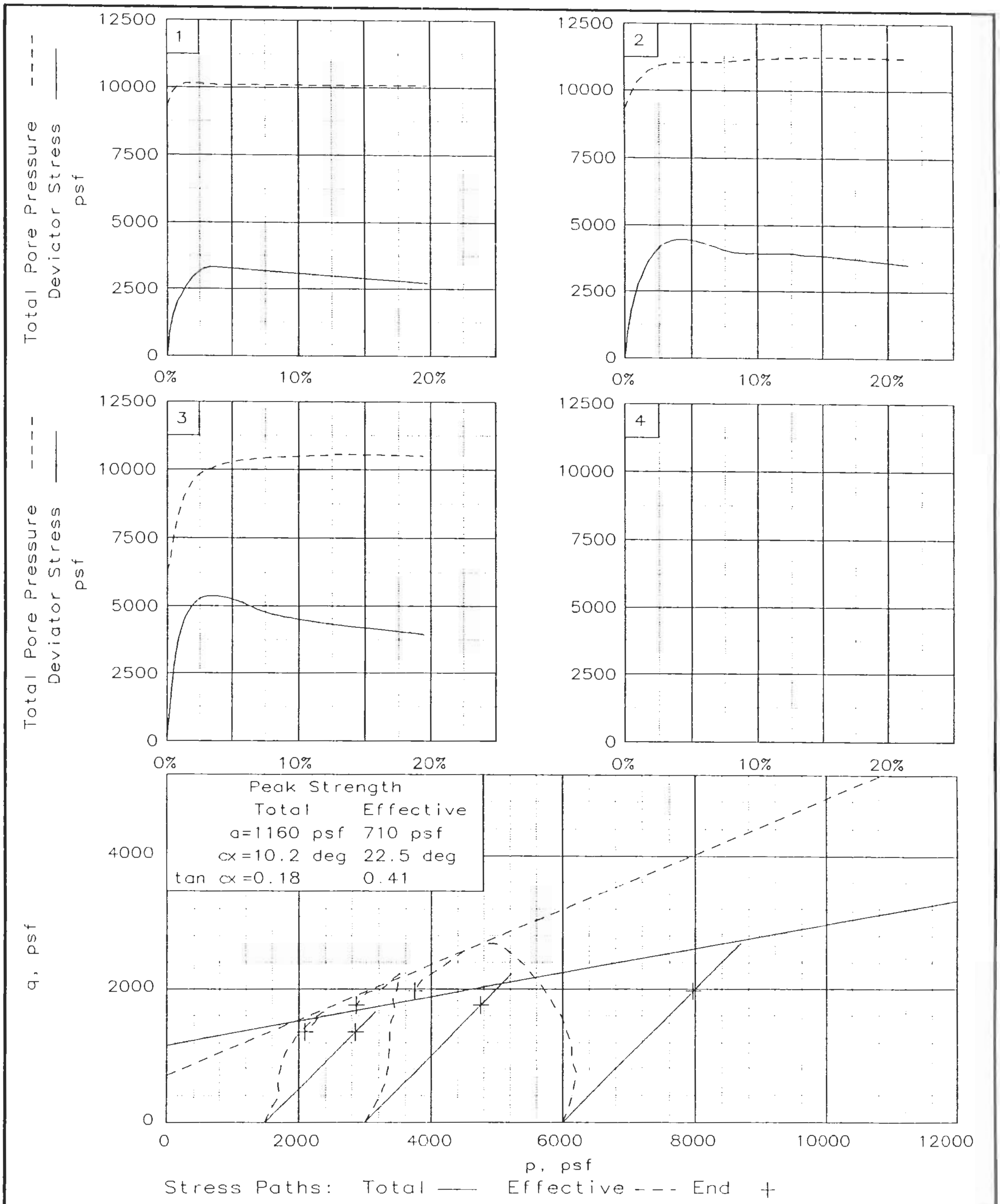
PROJECT: Napa River Flood Protection

SAMPLE LOCATION: 2F-03-08 Tube 2

PROJ. NO.: 03-153 DATE: 5-7-03

TRIAXIAL SHEAR TEST REPORT

SIERRA TESTING LABORATORIES, INC.



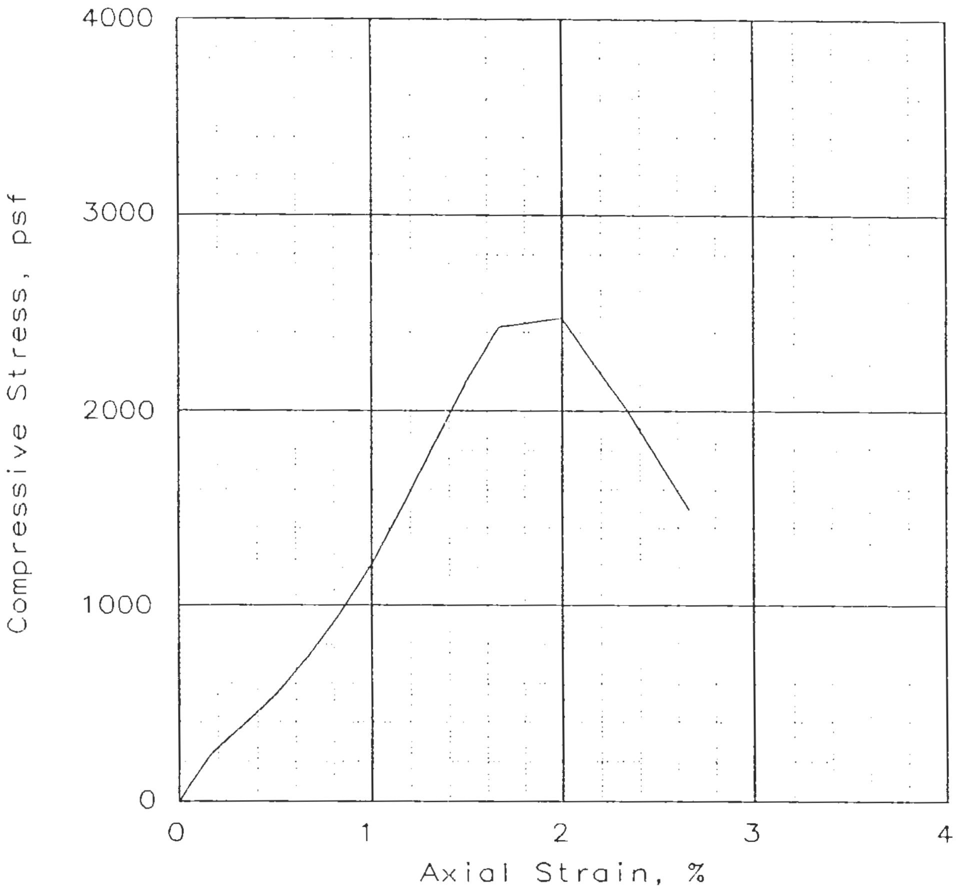
Client: USACE-Sacramento District

Project: Napa River Flood Protection

Location: 2F-03-08 Tube 2

File: 03-153G Project No.: 03-153 Fig. No.: ____

UNCONFINED COMPRESSION TEST



SAMPLE NO.:	1			
Unconfined strength, psf	2475			
Undrained shear strength, psf	1237			
Failure strain, %	2.0			
Strain rate, %/min	0.30			
Water content, %	25.8			
Wet density, pcf	123.3			
Dry density, pcf	98.0			
Saturation, %	96.7			
Void ratio	0.7201			
Specimen diameter, in	2.83			
Specimen height, in	6.00			
Height/diameter ratio	2.12			

Description: Fat CLAY

LL = 51	PL = 11	PI = 40	GS= 2.7	Type: Undisturbed

Project No.: 03-153

Date: 3-19-03

Remarks:

Fig. No.: ——

Client: USACE

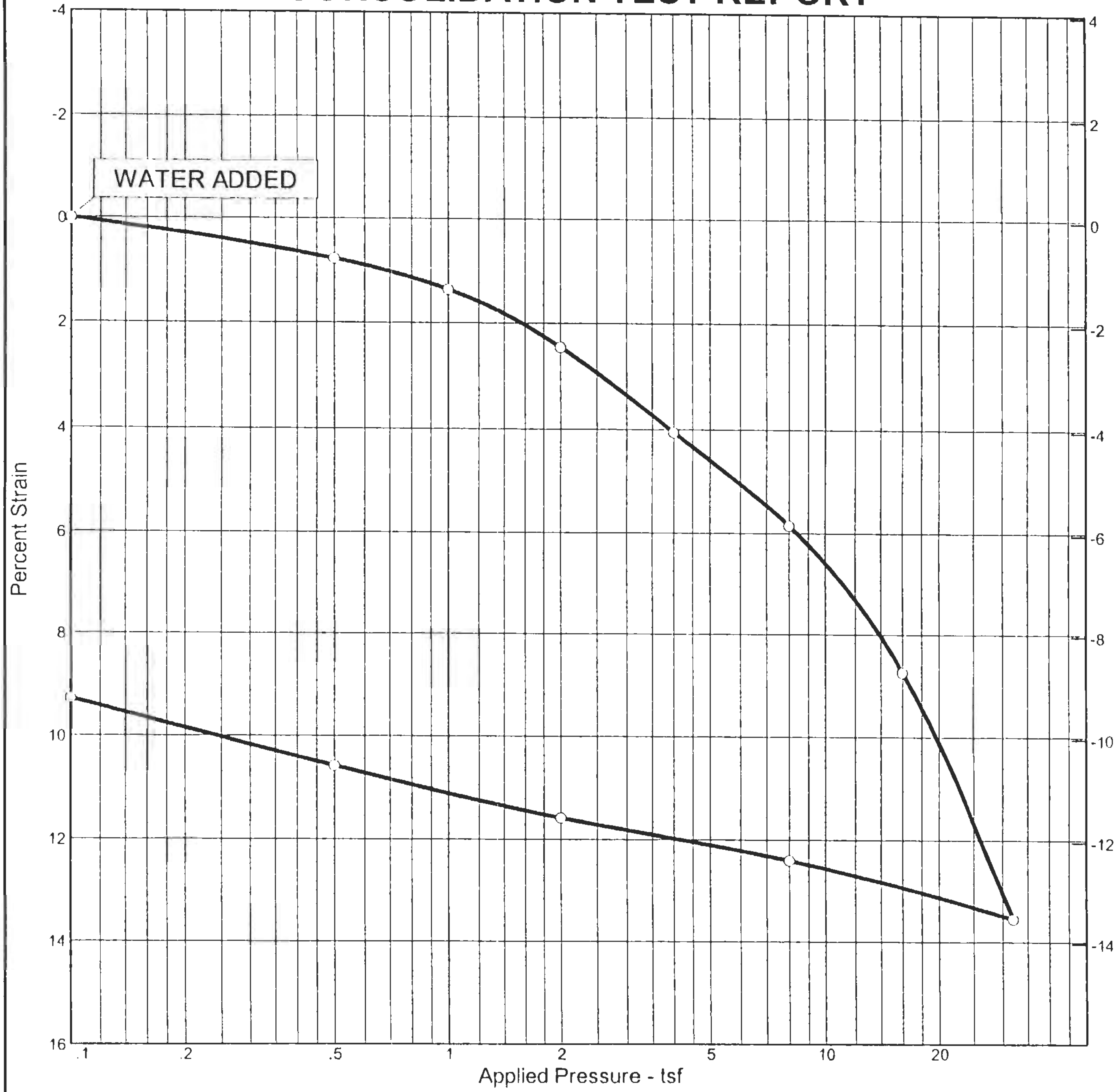
Project: Napa River Flood Protection

Location: 2F-03-08 Tube 3 @ 52'-54'

UNCONFINED COMPRESSION TEST

SIERRA TESTING LABORATORIES, INC.

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P$_c$ (tsf)	C$_c$	C$_s$	Swell Press. (tsf)	Heave %	e$_o$
Sat.	Moist.											
98.4 %	29.3 %	92.4	51	40	2.65		14.77	0.29	0.03	0.11		0.790

MATERIAL DESCRIPTION	USCS	AASHTO
Fat clay	CH	A-7-6(36)

Project No. 03-153 **Client:** U.S. Army Corps of Engineers, Sacramento District

Project: Napa River Flood Protection, Contract 2 West

Location: 2F-03-08

Remarks:

CONSOLIDATION TEST REPORT

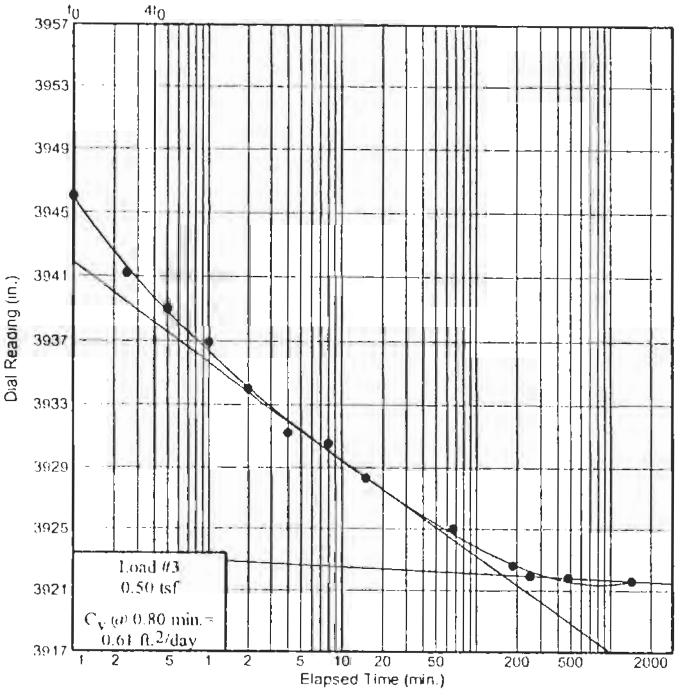
SIERRA TESTING LABS, INC.

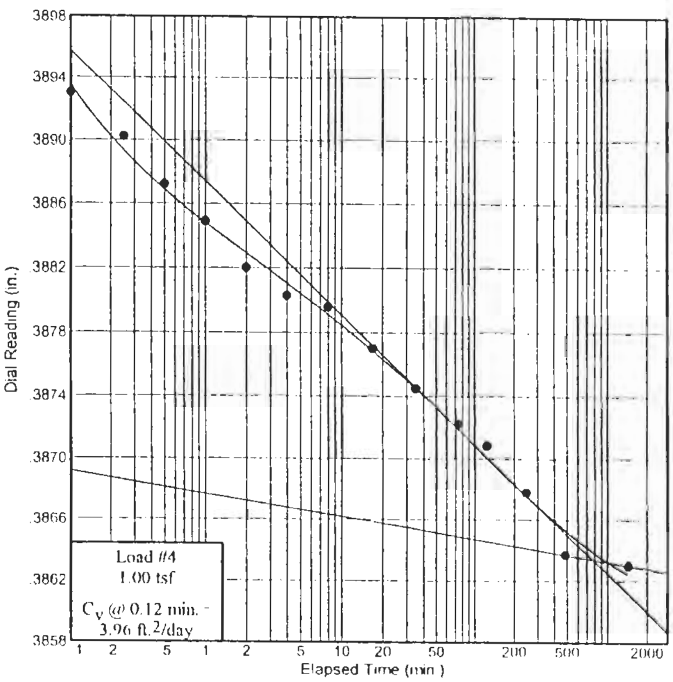
Figure

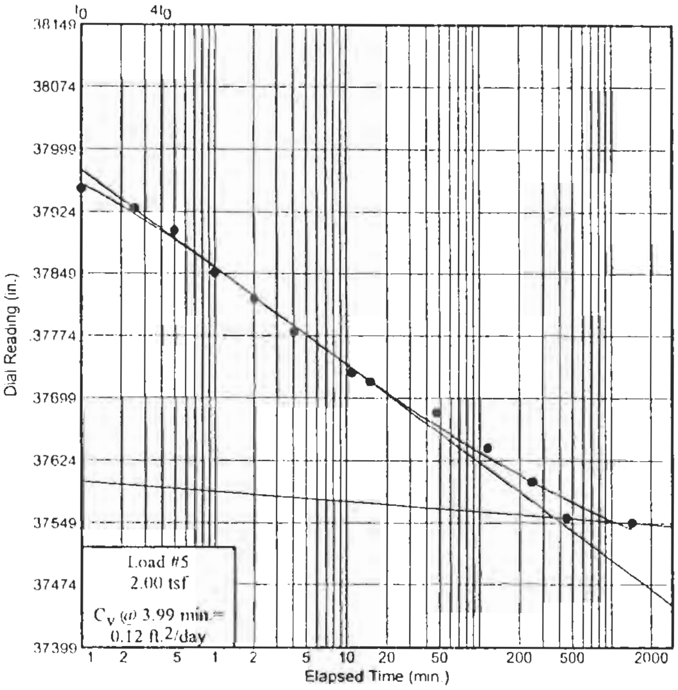
Dial Reading vs. Time

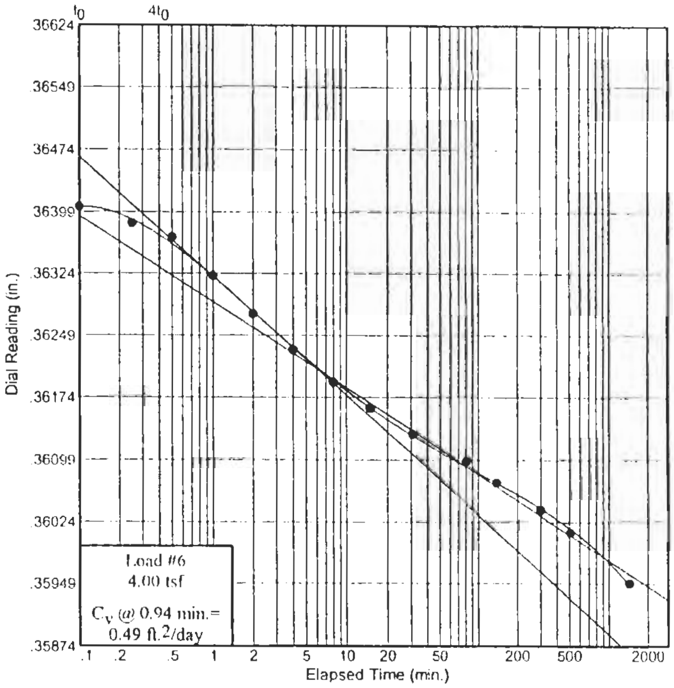
Project No.: 03-153
Project: Napa River Flood Protection, Contract 2 West

Location: 2F-03-08









Dial Reading vs. Time

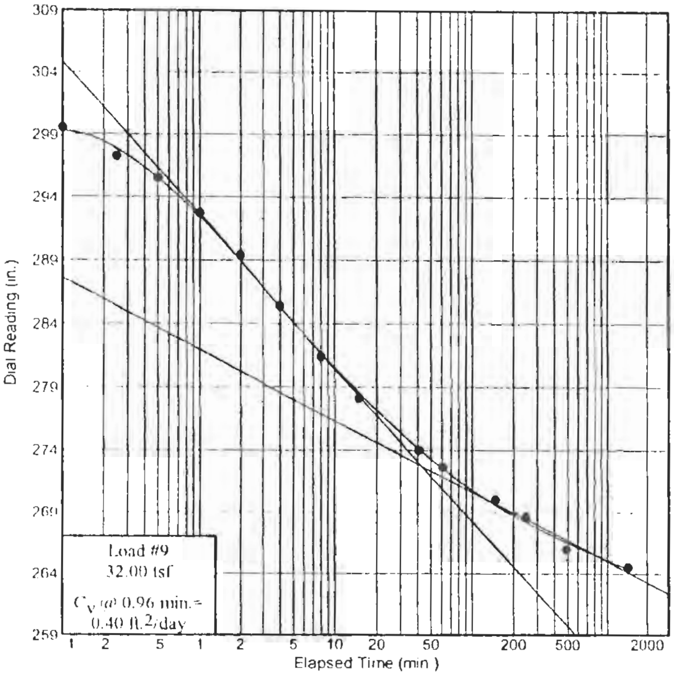
SIERRA TESTING LABS, INC.

Figure

Dial Reading vs. Time

Project No.: 03-153
Project: Napa River Flood Protection, Contract 2 West

Location: 2F-03-08



Dial Reading vs. Time

SIERRA TESTING LABS, INC.

Figure

APPENDIX 5: PILE DESIGN CALCULATIONS

Napa Contract 2 West Soldier Pile Wall

Pile Foundation Depth for Vertical Wall at Hatt Building

Information (From MGE Engineering, structural designers)
- 2' diameter CIDH piles
- 8' spacing between footings, 6' spacing in footing
- Compression Load 88.2 kips/pile
- Tension Load 39.2 kips/pile

Factors of Safety (EM 1110-2-2906)
- Verified by Pile Load Test 2.0
- Not verified by Pile Load Test 3.0

Stratigraphy (Soil borings 2F-03-3, 2F-03-4, 2F-04-51, B-4)
- 25 feet dense sand and gravel (phi = 33 degrees)
- 25 feet very stiff/hard lean clay (c =1200 psf)
- 10 feet dense sand and gravel (phi = 33 degrees)
- 6 feet hard lean clay (c=1200 psf)

There are many references and many methods for performing pile design. Use several methods and average the results for the design value.

References are: EM 1110-1-1905 (Bearing Capacity of Soils)
NAVFAC 7.2 (Foundations and Earth Structures)
Virginia Tech (Engineering Manual for Drilled Shafts)
FHwA-HI-88-042 (Drilled Shafts: Construction Procedures & Design Methods)

Case A. Assume pile depth = 51', 2 ' diameter, 3 piles

Pile founded 1 foot into lower sand and gravel layer. BUT, due to unknowns associated with the top elevation of that layer, assume "worst-case" condition and use clay equations for end bearing

Compression capacity

End Bearing - use clay equations

Method	Critical Depth L_c (ft)	Pile Depth (ft)	Pile Diam (ft)	Cohesion (psf)	N_{cp}	F_r	Unit End Bear (ksf)	As (ft2)	End Bear kips
Bear Cap EM	N.A.	51	2	1200	9.000	1.000	10.80	3.142	33.93
NAVFAC 7.2	N.A.	51	2	1200	9		10.8	3.142	33.93
VA Tech	N.A.	51	2	1200	9		10.8	3.142	33.93
FHwA	N.A.	51	2	1200	9		10.8	3.1416	33.93
								Avg	33.93

Skin Friction *sand portion*

Method	Pile Diam (ft)	Pile Depth (ft)	A_s (ft^2)	Midpt. Eff.Str. σ'_L (ksf)	β_f	δ (deg)	tan δ	K	K_{HC}	ϕ (deg)	tan ϕ	fsi (ksf)	Qsui (kips)
Bear Cap EM	2	25	157	1.1581	0.45							0.52	81.82
NAVFAC 7.2	2	25	157	1.1581		24.75	0.461		0.70			0.37	58.64
Touma&Reese (VA Tech)	2	25	157	1.1581				0.7		33.00	0.649	0.53	82.60
FHwA	2	25	157	1.1581	1.02							1.18	185.95

*Avg. 74.35
(ignore FHwA - seems too hig

VA Tech and FHwA refs say drilled shafts in clay do not require a strength reduction for group effects. Drilled shafts in sand require a strength reduction multiplier of 0.7 for spacing of 3 diameters

Pile Group Multiplier 0.7

Group Avg. 52.05

Skin Friction *(clay portion)*

Method	Clay Depth (ft)	Pile Diam (ft)	A_s clay (ft)	Cohesion (ksf)	α_a	C_A	f_{si} (ksf)	A_{si} (ft^2)	Qsui (kips)
Bear Cap EM	26	2	163.28	1.2	0.55		0.66	157	103.62
NAVFAC 7.2	26	2	163.28	1.2		0.6	0.72	157	113.04
VA Tech	26	2	163.28	1.2	0.55		0.66	157	103.62
FHwA	26	2	163.28	1.2	0.55		0.66	157	103.62
								Avg.	105.98

Total Skin Friction	158.02
End bearing	33.93
Total	**191.95**
Allowable F.S. = 2	**95.98**
Allowable F.S. = 3	**63.98**

CHECK TENSION CAPACITY

Case A. Pile Depth = 51 feet, diameter = 2', pile founded in clay, 3 piles

Method	Pile Depth (ft)	Pile Diam (ft)	Wt Pile (kips)	Skin Frict (kips)	Multiplier	Qni (kips)	Pni (kips)
Bear Cap EM	51	2	14.03	185.44	0.667	123.69	137.72
NAVFAC 7.2	51	2	14.03	171.68	0.4	68.67	82.70
Va Tech	51	2	14.03	186.22	0.55	102.42	116.45
FHwA	51	2	14.03	not used - too high			
				Average			**112.29 (for one pile)**

The References say the uplift capacity for a group of drilled shafts is the lesser of
a. The sum of the individual uplift capacities of the drilled shafts
b. The effective weight of the block of soil and the piles within the group

a. 112.29 x 3 = 337 Kips

b. Soil Wt. 105 Kips Pile wt = 42 Total 147 controls

Allowable F.S. = 2 74

Allowable F.S. = 3 49

Case B. Assume pile depth = 51', 3' diameter pile

Pile founded 1 foot into lower sand and gravel layer. BUT, due to unknowns associated with the top of that layer, assume "worst-case" condition and use clay equations for end bearing

Compression Capacity

End Bearing - use clay equations

Method	Critical Depth L_c (ft)	Pile Depth (ft)	Pile Diam (ft)	Cohesion (psf)	N_{cp}	F_r	Unit End Bear (ksf)	As (ft2)	End Bear kips
Bear Cap EM	N.A.	51	3	1200	9.000	1.000	10.80	7.069	76.34
NAVFAC 7.2	N.A.	51	3	1200	9		10.8	7.069	76.34
VA Tech	N.A.	51	3	1200	9		10.8	7.069	76.34
FHwA	N.A.	51	3	1200	9		10.8	7.0686	76.34
								Avg	76.34

Skin Friction   *sand portion*

Method	Pile Diam (ft)	Pile Depth (ft)	A_s (ft^2)	Midpt. Eff.Str. σ'_L (ksf)	β_f	δ (deg)	tan δ	K	K_{HC}	ϕ (deg)	tan ϕ	fsi (ksf)	Qsui (kips)
Bear Cap EM	3	25	235.5	1.1581	0.45							0.52	122.73
NAVFAC 7.2	3	25	235.5	1.1581		24.75	0.461		0.70			0.37	87.96
Touma&Reese (VA Tech)	3	25	235.5	1.1581				0.7		33.00	0.649	0.53	123.90
FHwA	3	25	235.5	1.1581	1.02							1.18	278.92
												*Avg.	111.53

(ignore FHwA - seems too hig

VA Tech and FHwA refs say drilled shafts in clay do not require a strength reduction for group effects. Drilled shafts in sand require a strength reduction multiplier of 0.7 for spacing of 3 diameters

Pile Group Multiplier 0.7

Group Avg. 78.07

Skin Friction   *(clay portion)*

Method	Clay Depth (ft)	Pile Diam (ft)	A_s clay (ft)	Cohesion (ksf)	α_a	C_A	f_{si} (ksf)	A_{si} (ft^2)	Qsui (kips)
Bear Cap EM	26	3	244.92	1.2	0.55		0.66	235.5	155.43
NAVFAC 7.2	26	3	244.92	1.2		0.6	0.72	235.5	169.56
VA Tech	26	3	244.92	1.2	0.55		0.66	235.5	155.43
FHwA	26	3	244.92	1.2	0.55		0.66	235.5	155.43
								Avg.	158.96

Total Skin Friction	237.03
End bearing	76.34
Total	**313.37**
Allowable F.S. = 2	**156.69**
Allowable F.S. = 3	**104.46**

CHECK TENSION CAPACITY

Case B. Pile Depth = 51 feet, diameter = 3', pile founded in clay, assume 2 piles

Method	Pile Depth (ft)	Pile Diam (ft)	Wt Pile (kips)	Skin Frict (kips)	Multiplier	Qni (kips)	Pni (kips)
Bear Cap EM	51	3	31.56	278.16	0.667	185.53	217.10
NAVFAC 7.2	51	3	31.56	257.52	0.4	103.01	134.57
Va Tech	51	3	31.56	279.33	0.55	153.63	185.20
FHwA	51	3	31.56	not used - too high			

Average **178.95** **(for one pile)**

The References say the uplift capacity for a group of drilled shafts is the lesser of

a. The sum of the individual uplift capacities of the drilled shafts
b. The effective weight of the block of soil and the piles within the group

a. 178.95 x 2 = 358 Kips

b. Soil Wt. 110 Kips Pile wt = 63 Total 173 controls

Allowable F.S. = 2 87

Allowable F.S. = 3 58

Case C. 2 ' diameter, 3 piles, Check capacity at bottom of explorations (depth = 66').

Pile founded in clay

Compressin Capacity

End Bearing - use clay equations

Method	Critical Depth L_c (ft)	Pile Depth (ft)	Pile Diam (ft)	Cohesion (psf)	N_{cp}	F_r	Unit End Bear (ksf)	As (ft2)	End Bear kips
Bear Cap EM	N.A.	66	2	1200	9.000	1.000	10.80	3.142	33.93
NAVFAC 7.2	N.A.	66	2	1200	9		10.8	3.142	33.93
VA Tech	N.A.	66	2	1200	9		10.8	3.142	33.93
FHwA	N.A.	66	2	1200	9		10.8	3.1416	33.93
								Avg	**33.93**

Skin Friction *upper sand*

Method	Pile Diam (ft)	Pile Depth (ft)	A_s (ft^2)	Midpt. Eff.Str. σ'_L (ksf)	β_f	δ (deg)	tan δ	K	K_{HC}	ϕ (deg)	tan ϕ	fsi (ksf)	Qsui (kips)
Bear Cap EM	2	25	157	1.1581	0.45							0.52	81.82
NAVFAC 7.2	2	25	157	1.1581		24.75	0.461		0.70			0.37	58.64
Touma&Reese (VA Tech)	2	25	157	1.1581				0.7		33.00	0.649	0.53	82.60
FHwA	2	25	157	1.1581	1.02							1.18	185.95

*Avg. 74.35
(ignore FHwA - seems too hig

VA Tech and FHwA refs say drilled shafts in clay do not require a strength reduction for group effects. Drilled shafts in sand require a strength reduction multiplier of 0.7 for spacing of 3 diameters

Pile Group Multiplier 0.7

Group Avg. 52.05

Skin Friction *lower sand*

Method	Pile Diam (ft)	Pile Depth (ft)	A_s (ft^2)	Midpt. Eff.Str. σ'_L (ksf)	β_f	δ (deg)	tan δ	K	K_{HC}	ϕ (deg)	tan ϕ	fsi (ksf)	Qsui (kips)
Bear Cap EM	2	10	62.8	3.8186	0.45							1.72	107.91
NAVFAC 7.2	2	10	62.8	3.8186		24.75	0.461		0.70			1.23	77.34
Touma&Reese (VA Tech)	2	10	62.8	3.8186				0.7		33.00	0.649	1.73	108.94
FHwA	2	10	62.8	3.8186	1.20							4.58	287.32

	*Avg.	98.07
	(ignore FHwA - seems too hig	
Pile Group	Multiplier	0.7
	Group Avg.	68.65
Total Sand Skin Friction		**120.69**

VA Tech and FHwA refs say drilled shafts in clay do not require a strength reduction for group effects. Drilled shafts in sand require a strength reduction multiplier of 0.7 for spacing of 3 diameters

Skin Friction *upper clay*

Method	Clay Depth (ft)	Pile Diam (ft)	A_s clay (ft)	Cohesion (ksf)	α_a	C_A	f_{si} (ksf)	A_{si} (ft^2)	Qsui (kips)
Bear Cap EM	26	2	163.28	1.2	0.55		0.66	157	103.62
NAVFAC 7.2	26	2	163.28	1.2		0.6	0.72	157	113.04
VA Tech	26	2	163.28	1.2	0.55		0.66	157	103.62
FHwA	26	2	163.28	1.2	0.55		0.66	157	103.62
								Avg.	105.98

Skin Friction *lower clay*

Method	Clay Depth (ft)	Pile Diam (ft)	A_s clay (ft)	Cohesion (ksf)	α_a	C_A	f_{si} (ksf)	A_{si} (ft^2)	Qsui (kips)
Bear Cap EM	6	2	37.68	1.2	0.55		0.66	31.4	20.72
NAVFAC 7.2	6	2	37.68	1.2		0.6	0.72	31.4	22.61
VA Tech	6	2	37.68	1.2	0.55		0.66	31.4	20.72
FHwA	6	2	37.68	1.2	0.55		0.66	31.4	20.72
								Avg.	21.20

Total clay skin friction **127.17**

TOTAL SKIN FRICTION **247.86**

END BEARING **33.93**

TOTAL CAPACITY **281.79**

Allowable F.S. = 2 **140.90**
Allowable F.S. = 3 **93.93**

Don't need to check tension capacity as 51' pile is OK in tension

Case D. Assume 60 foot pile depth, 2' diameter, 3 piles

Everything the same as previous case except don't have the skin friction of the lower clay layer.

Total skin friction	**226.67**
End Bearing	**33.93**
TOTAL CAPACITY	**260.60**
Allowable F.S. = 2	**130.30**
Allowable F.S. = 3	**86.87**

CHECK TENSION CAPACITY

Case D. Pile Depth = 60 feet, diameter = 2', pile founded in clay, 3 piles

Method	Pile Depth (ft)	Pile Diam (ft)	Wt Pile (kips)	Skin Frict (kips)	Multiplier	Qni (kips)	Pni (kips)
Bear Cap EM	60	2	16.50	236.43	0.667	157.70	174.20
NAVFAC 7.2	60	2	16.50	208.23	0.4	83.29	99.80
Va Tech	60	2	16.50	215.58	0.55	118.57	135.07
FHwA	60	2	16.50	not used - too high			
				Average			**136.36** **(for one pile)**

The References say the uplift capacity for a group of drilled shafts is the lesser of

a. The sum of the individual uplift capacities of the drilled shafts
b. The effective weight of the block of soil and the piles within the group

a. 136.36 x 3 = 409 Kips

b. Soil Wt. 131 Kips Pile wt = 50 Total 181 controls

Allowable F.S. = 2 90

Allowable F.S. = 3 60

Case E. All soil borings extending deep enough have the lower sand & gravel layer. If it does not exist over all of Wall Type A area, will 10' of clay instead of sand at bottom give sufficient capacity

Minimum required total capacity (88.2 x 3) 264.6 Kips

End bearing and skin friction in upper sand layer will not change

End Bearing	33.93
Upper Sand Skin Friction	74.35
Sum	108.28

Required clay skin friction 156.32

Skin Friction *clay*

Method	Clay Depth (ft)	Pile Diam (ft)	A_s clay (ft)	Cohesion (ksf)	α_a	C_A	f_{si} (ksf)	A_{si} (ft^2)	Qsui (kips)
Bear Cap EM	35	2	219.8	1.2	0.55		0.66	213.52	140.92
NAVFAC 7.2	35	2	219.8	1.2		0.6	0.72	213.52	153.73
VA Tech	35	2	219.8	1.2	0.55		0.66	213.52	140.92
FHwA	35	2	219.8	1.2	0.55		0.66	213.52	140.92
								Avg.	144.13

Total capacity	252.41	Kips
F.S.	2.86	

Don't need to check tension capacity as 51' pile is OK in tension

CHECK BLOCK FAILURE CAPACITY FOR CASES D AND E

Case D

Use worst case block situation, 90 feet along wall LOL, 9 feet perpendicular to wall LOL, 36 total piles

Capacity of 36 piles = 36 x 277.10 9,381.55 Kips

Block Capacity

End Bearing

X (ft)	Y (ft)	N_c	C (ksf)	Q_{ui} kips
9	90	7.65	1.2	7,435.80

Skin Friction

Sand layers - assume calculated the same as above

Upper sand

Method	Width (ft)	Pile Depth (ft)	A_s (ft^2)	Midpt. Eff.Str. σ'_L (ksf)	β_f	δ (deg)	$\tan\delta$	K	K_{HC}	ϕ (deg)	$\tan\phi$	fsi (ksf)	Qsui (kips)
Bear Cap EM	9	25	4950	1.1581	0.45							0.52	2579.67
NAVFAC 7.2	9	25	4950	1.1581		24.75	0.461		0.70			0.37	1848.87
Touma&Reese (VA Tech)	9	25	4950	1.1581				0.7		33.00	0.649	0.53	2604.29
FHwA	9	25	4950	1.1581	1.02							1.18	5862.74

*Avg. 2344.27
(ignore FHwA - seems too hig

VA Tech and FHwA refs say drilled shafts in clay do not require a strength reduction for group effects. Drilled shafts in sand require a strength reduction multiplier of 0.7 for spacing of 3 diameters

Pile Group Multiplier 0.7

Group Avg. 1640.99

Skin Friction lower sand

Method	Width (ft)	Pile Depth (ft)	A_s (ft^2)	Midpt. Eff.Str. σ'_L (ksf)	β_f	δ (deg)	$\tan\delta$	K	K_{HC}	ϕ (deg)	$\tan\phi$	fsi (ksf)	Qsui (kips)
Bear Cap EM	9	10	1980	3.8186	0.45							1.72	3402.37
NAVFAC 7.2	9	10	1980	3.8186		24.75	0.461		0.70			1.23	2438.51
Touma&Reese (VA Tech)	9	10	1980	3.8186				0.7		33.00	0.649	1.73	3434.84
FHwA	9	10	1980	3.8186	1.20							4.58	9058.86

*Avg. 3091.91
(ignore FHwA - seems too higl

Pile Group	Multiplier	0.7
	Group Avg.	2164.34
Total Sand Skin Friction		**3805.33**

VA Tech and FHwA refs say drilled shafts in clay do not require a strength reduction for group effects. Drilled shafts in sand require a strength reduction multiplier of 0.7 for spacing of 3 diameters

Skin Friction clay

According to VA Tech and FHwA refs, do not use the reduction multiplier (usually called α) when doing block analysis

X (ft)	Y (ft)	Z (ft)	C (ksf)	Q_{ui} kips
9	90	25	1.2	5,940.00

Total Capacity **17,181.13 Kips** So individual pile capacity controls

Case E.

End bearing same as Case D	7,435.80	Kips
Skin friction sand - same as upper sand in Case D	1,640.99	Kips

Skin friction clay

X (ft)	Y (ft)	Z (ft)	C (ksf)	Q_{ui} kips
9	90	35	1.2	8,316.00

Total Capacity 17,392.79 So individual pile capacity controls

h)

h)

h)

h)

h)

h)

APPENDIX 6: DEFLECTION/SETTLEMENT CALCULATIONS

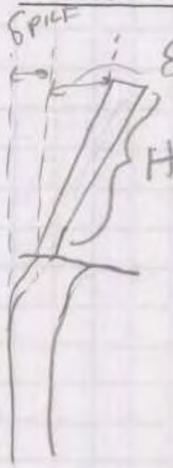
COMPUTATION SHEET

PROJECT NAPA 2 WEST HATT 2 FIRST SHEET NO 1 OF 2 SHEETS

ITEM DEFLECTION / SETTLEMENT DATE

WALL TYPE A FILE

COMPUTED BY JMB CHECKED BY REF. DWG. NO.



PILE DEFLECTION FROM LPILE

DEPTH = 11' NO DEFLECTION

DEPTH = 0' $\delta_{PILE} = 0.28''$

$\delta_{WALL} = 0.61''$

$H = 24.0'$ (MAX)

FIND DEFLECTED AREA

$A_{PILE} = \frac{1}{2}\left(\frac{0.28''}{12}\right)(11') = 0.13 \text{ FT}^2$

$A_{WALL} = \left(\frac{0.28''}{12}\right)(24') + \frac{1}{2}\left(\frac{0.61''}{12}\right)(24') = 0.56 + 0.61$

$A_{WALL} = 1.17 \text{ FT}^2$

$A_{DEFLECTED} = 0.13 + 1.17 = 1.30 \text{ FT}^2 = 187.2 \text{ IN}^2$

SETTLEMENT

$A_{SETTLEMENT} = A_{DEFLECTED}$

RALPH PECK. CHART

$\frac{\text{MAX SETTLEMENT}}{H} = 1\%$

MAX SETTLEMENT = 0.24 FT

$\approx 2.9''$

COMPUTATION SHEET

PROJECT NAPA 2 WEST HATT 2 FIRST SHEET NO 2 OF 2 SHEETS

ITEM DEFLECTION / SETTLEMENT
WALL TYPE A
DATE

FILE

COMPUTED BY JMB CHECKED BY REF. DWG. NO.

FIND x

RALPH PECK CHART

$$\frac{x}{H} = 2$$

$$x = 2(24) = 48'$$

CHECK $A_{SETTLEMENT}$

$$A_{SETTLEMENT} = \frac{1}{2}(0.24')(48') = 5.76 \text{ Ft}^2$$

$$5.76 \text{ Ft}^2 >> 1.30 \text{ Ft}^2 \; (A_{DEFLECTED})$$

SO, THE SETTLEMENT WILL BE << THAN PREDICTED BY PECK CHART.

OLD BUILDING AT NAPA MILL IS 9' FROM WALL LOL

CALC SETTLEMENT USING VARIOUS VALUES FOR x

x	δ_{max}	δ_{bldg}
[illegible]	1.37"	1.59
[illegible]	0.88"	0.41
30'	0.59"	0.53"
40'	0.44"	0.39

$$106.6 \text{ in}^2 = \frac{1}{2}(\delta_{max})(x \cdot 12)$$

$$\delta_{max} = \frac{(106.6) \cdot 2}{x \cdot 12}$$

$$\frac{\delta_{bldg}}{9 \times 12} = \frac{\delta_{max}}{x \times 12}$$

$$\delta_{bldg} = \frac{\delta_{max}(108)}{x \times 12}$$

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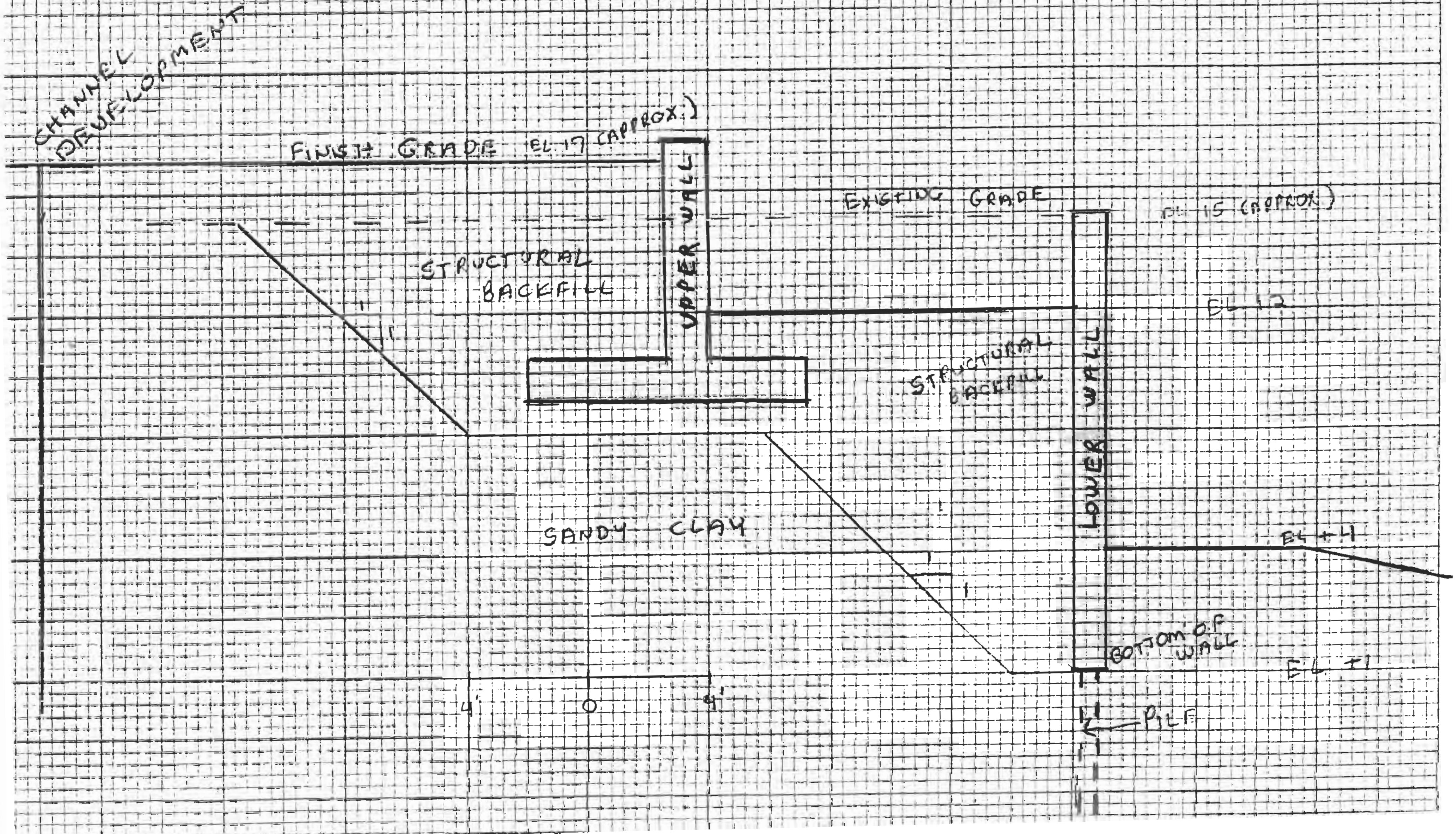
APPENDIX 7: BEARING CAPACITY CALCULATIONS – UPPER WALL


CHANNEL DEVELOPMENT
FINISH GRADE EL 17 (APPROX.)
EXISTING GRADE
EL 15 (APPROX.)
UPPER WALL
STRUCTURAL BACKFILL
1
1
EL 12
STRUCTURAL BACKFILL
LOWER WALL
SANDY CLAY
EL +4
1
1
BOTTOM OF WALL
EL +1
4'
0
4'
PILE

COMPUTATION SHEET

PROJECT NAPA CT. 2 WEST

SHEET NO 1 OF 5 SHEETS

ITEM HATT TO FIRST

UPPER WALL BEAR. CAP.

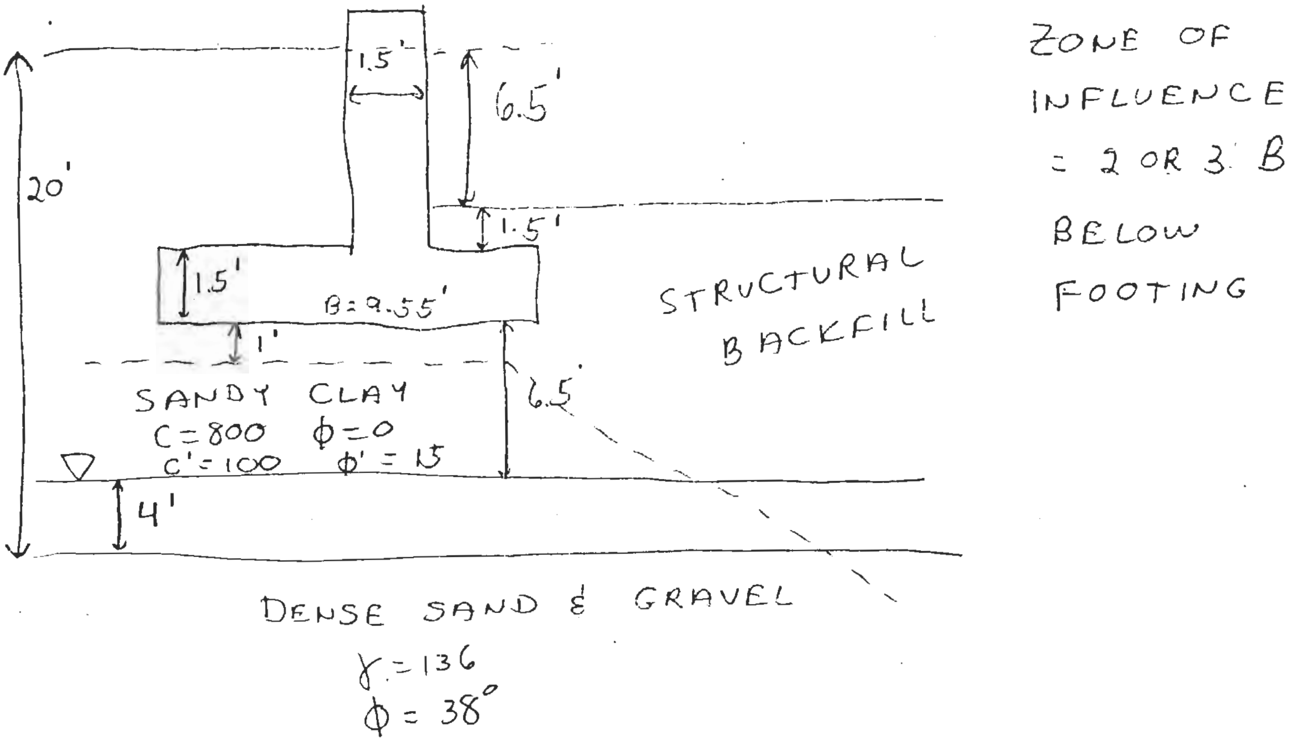
DATE 6/16/04

FILE

COMPUTED BY JMB

CHECKED BY

REF. DWG. NO.



ZONE OF INFLUENCE BELOW FOOTING CONTAINS STRUCTURAL BACKFILL AND INSITU CLAY & SAND. ∴ DUE TO LOWER ϕ ANGLE, BEAR. CAP. OF SANDY CLAY WILL BE LOWER THAN GRANULAR SOILS. ACTUAL BEAR. CAP. WILL BE BETWEEN THE VALUES. DO CALCS FOR ALL SOIL TYPES.

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COMPUTATION SHEET

PROJECT ______ SHEET NO 2 OF 5 SHEETS

ITEM ______ DATE 6/16/04

UPPER WALL BEAR. CAP. FILE ______

COMPUTED BY ______ CHECKED BY ______ REF. DWG. NO. ______

EM 1110-2-2502 (RETAINING & FLOOD WALLS)

$$q_{ULT} = \bar{B}\left(C N_c + q_o N_q + \tfrac{1}{2}\bar{B}\,\gamma N_\gamma\right)$$

$$q_o = 120(3) = 360 \ \text{lb}/\text{FT}^2$$

FOR ECCENTRIC LOADING $\bar{B} = B - 2e$

$$e = \frac{9.5}{2} - 3.25 = 1.50$$

$$\bar{B} = 9.5 - 2(1.5) = 6.5'$$

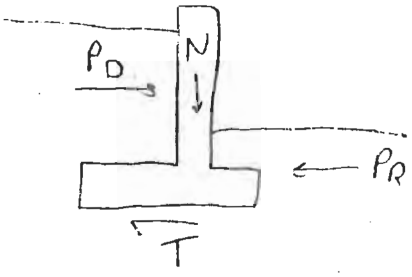
SANDY CLAY

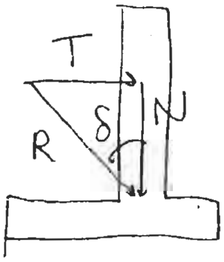
LOWER ϕ = LOWER N_c, N_q, N_γ SO EOC CONTROLS

FOR $\phi = 0$ $N_c = 5.14$ $N_q = 1$ $N_\gamma = 0$

$$q_{ULT} = \bar{B}\,(800 \cdot 5.14 + 360 \cdot 1) = \bar{B}\,(4112 + 360)$$

BUT INCLINED LOADING SO NEED CORRECTION FACTOR





$$\delta = \tan^{-1}\frac{T}{N}$$

CESPD FORM 284, 1 SEP 89

COMPUTATION SHEET

PROJECT ______ SHEET NO 3 OF 5 SHEETS

ITEM ______ DATE 6/16/04

UPPER WALL BEAR. CAP. FILE ______

COMPUTED BY JMB CHECKED BY ______ REF. DWG. NO. ______

N = WT WALL + WT. SOIL ABOVE FOOTING

WT. WALL = $[(9.5 \times 1.5) + (1.5 \times 11)] \times 150 = 4,613$

WT SOIL ABOVE FOOTING = $[(2.5 \times 1.5) + (4.5 \times 6.5)] \times 120$

$= 3,960$

$N = 4,613 + 3,960 = 8,573$

$T = P_D - P_R$

$P_D = K_o \gamma Z_1 \cdot \frac{Z_1}{2} = (0.4)(120)(9.5)\left(\frac{9.5}{2}\right) = 2,166$

$P_R = K_p \gamma Z_2 \cdot \frac{Z_2}{2} = (3)(120)(3)\left(\frac{3}{2}\right) = 1,620$

$T = 2,166 - 1,620 = 546$

$\delta = \text{TAN}^{-1} \frac{546}{8,573} = 3.6°$

$\xi_{qi} = \xi_{ci} = \left(1 - \frac{\delta}{90}\right)^2 = 0.92$

EMBEDMENT FACTOR

$\xi_{cd} = 1 + 0.2\left(\frac{D}{B}\right) \text{TAN}\left(45 + \frac{\phi}{2}\right)$

$= 1 + 0.2\left(\frac{3}{6.5}\right) \text{TAN}(45 + 15) = 1.16$

$\xi_{qd} = 1$

SO $q_{ULT} = \overline{B}\left(4,112 \times 0.92 \times 1.16 + 360 \times 0.92 \times 1\right)$

$= \overline{B}\ (4,391 + 331) = \overline{B}\ (4,722)$

(DON'T MULTIPLY BY $\overline{B}$ TO GET VALUE IN PSF)

CESPD FORM 284, 1 SEP 89

COMPUTATION SHEET

PROJECT

SHEET NO 4 OF 5 SHEETS

ITEM

DATE 6/16/04

UPPER WALL BEAR CAP.

FILE

COMPUTED BY JMB

CHECKED BY

REF. DWG. NO.

$$q_{ALL} = \frac{q_{ULT}}{F.S.} = \frac{4{,}722}{3} = 1{,}574 \text{ PSF}$$

STRUCTURAL BACKFILL

FOR $\phi = 30$ $N_c = 30.14$ $N_q = 18.4$ $N_\gamma = 15.67$

$c = 0$ SO $cN_c = 0$

$$q_{ULT} = q_o N_q + \frac{1}{2} \bar{B} \gamma N_\gamma$$

$$= (360 \times 18.4) + (\tfrac{1}{2} \times 6.5 \times 120 \times 15.67)$$

$$q_{ULT} = 6{,}624 + 6{,}111$$

CORRECTION FACTORS

INCLINATION $\xi_{qi} = \left(1 - \frac{\delta}{90}\right)^2 = 0.92$

$\xi_{\gamma i} = \left(1 - \frac{\delta}{\phi}\right)^2 = 0.77$

EMBEDMENT

$$\xi_{qd} = \xi_{\gamma d} = 1 + 0.1\left(\frac{D}{B}\right) \tan\left(45 + \frac{d}{2}\right)$$

$$= 1.08$$

$$q_{ULT} = (6{,}624 \times 0.92 \times 1.08 + 6{,}111 \times 0.77 \times 1.08)$$

$$= 6{,}582 + 5{,}111 = 11{,}693$$

$$q_{ALL} = \frac{11{,}693}{3} = 3{,}897 \text{ PSF}$$

DENSE SAND & GRAVEL WILL HAVE A HIGHER q_{ALL} DUE TO HIGHER ϕ.

FAILURE WILL LIKELY BE "CONCENTRATED" IN SANDY CLAY LAYER.

CESPD FORM 284, 1 SEP 89

COMPUTATION SHEET

PROJECT

SHEET NO 5 OF 5 SHEETS

ITEM

DATE 6/16/04

UPPER WALL BEAR. CAP.

FILE

COMPUTED BY JMB

CHECKED BY

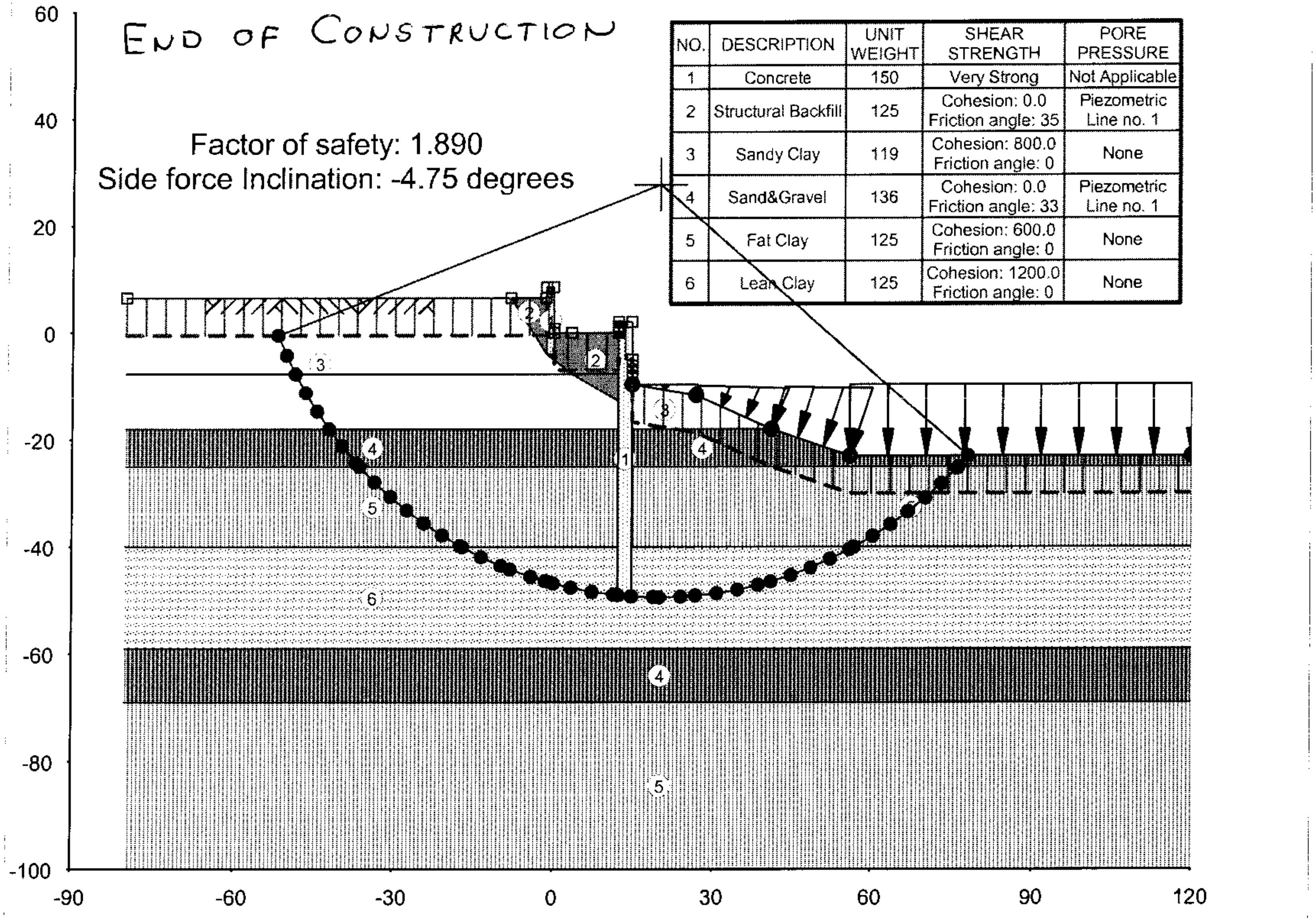
REF. DWG. NO.

USE q_{ALL} = 2,000 PSF. TAKES INTO ACCOUNT BOTH STRUCTURAL BACKFILL AND SANDY CLAY. PROBABLY A LITTLE ON THE CONSERVATIVE SIDE WITHOUT BEING HYPERCONSERVATIVE.

CESPD FORM 284, 1 SEP 89

APPENDIX 8: GLOBAL STABILITY FAILURE SURFACES

Napa Contract 2 West Hatt to First



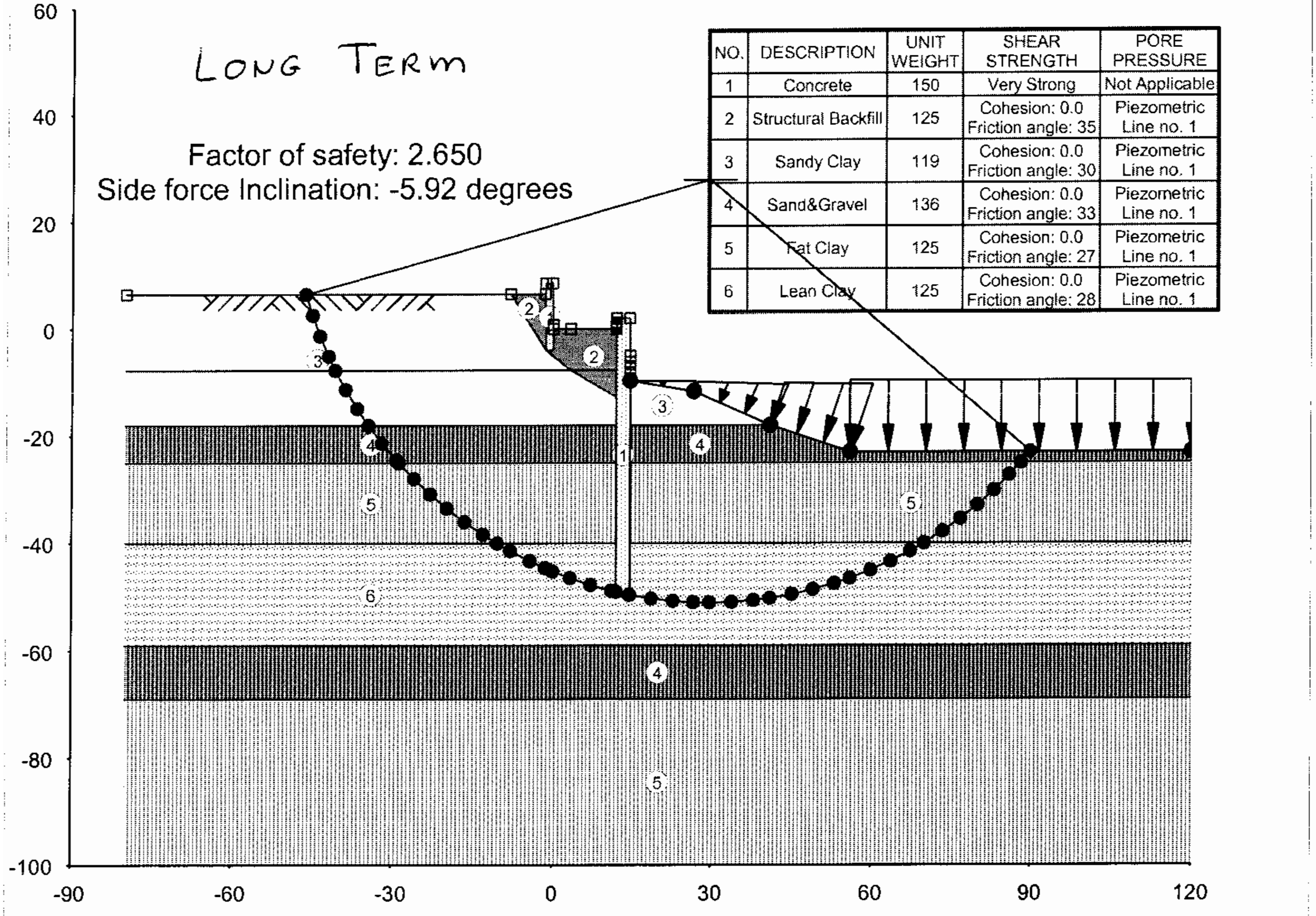
END OF CONSTRUCTION

Factor of safety: 1.890
Side force Inclination: -4.75 degrees

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Concrete	150	Very Strong	Not Applicable
2	Structural Backfill	125	Cohesion: 0.0 Friction angle: 35	Piezometric Line no. 1
3	Sandy Clay	119	Cohesion: 800.0 Friction angle: 0	None
4	Sand&Gravel	136	Cohesion: 0.0 Friction angle: 33	Piezometric Line no. 1
5	Fat Clay	125	Cohesion: 600.0 Friction angle: 0	None
6	Lean Clay	125	Cohesion: 1200.0 Friction angle: 0	None

Date: Wed May 04 2005 Filename: C:\WINUT4\UTexas4 Samples\Napa2West\hatt2first\dualwalleoc.UT4 Time: 13:17:27

Napa Contract 2 West Hatt to First



LONG TERM

Factor of safety: 2.650
Side force Inclination: -5.92 degrees

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Concrete	150	Very Strong	Not Applicable
2	Structural Backfill	125	Cohesion: 0.0 Friction angle: 35	Piezometric Line no. 1
3	Sandy Clay	119	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 1
4	Sand&Gravel	136	Cohesion: 0.0 Friction angle: 33	Piezometric Line no. 1
5	Fat Clay	125	Cohesion: 0.0 Friction angle: 27	Piezometric Line no. 1
6	Lean Clay	125	Cohesion: 0.0 Friction angle: 28	Piezometric Line no. 1

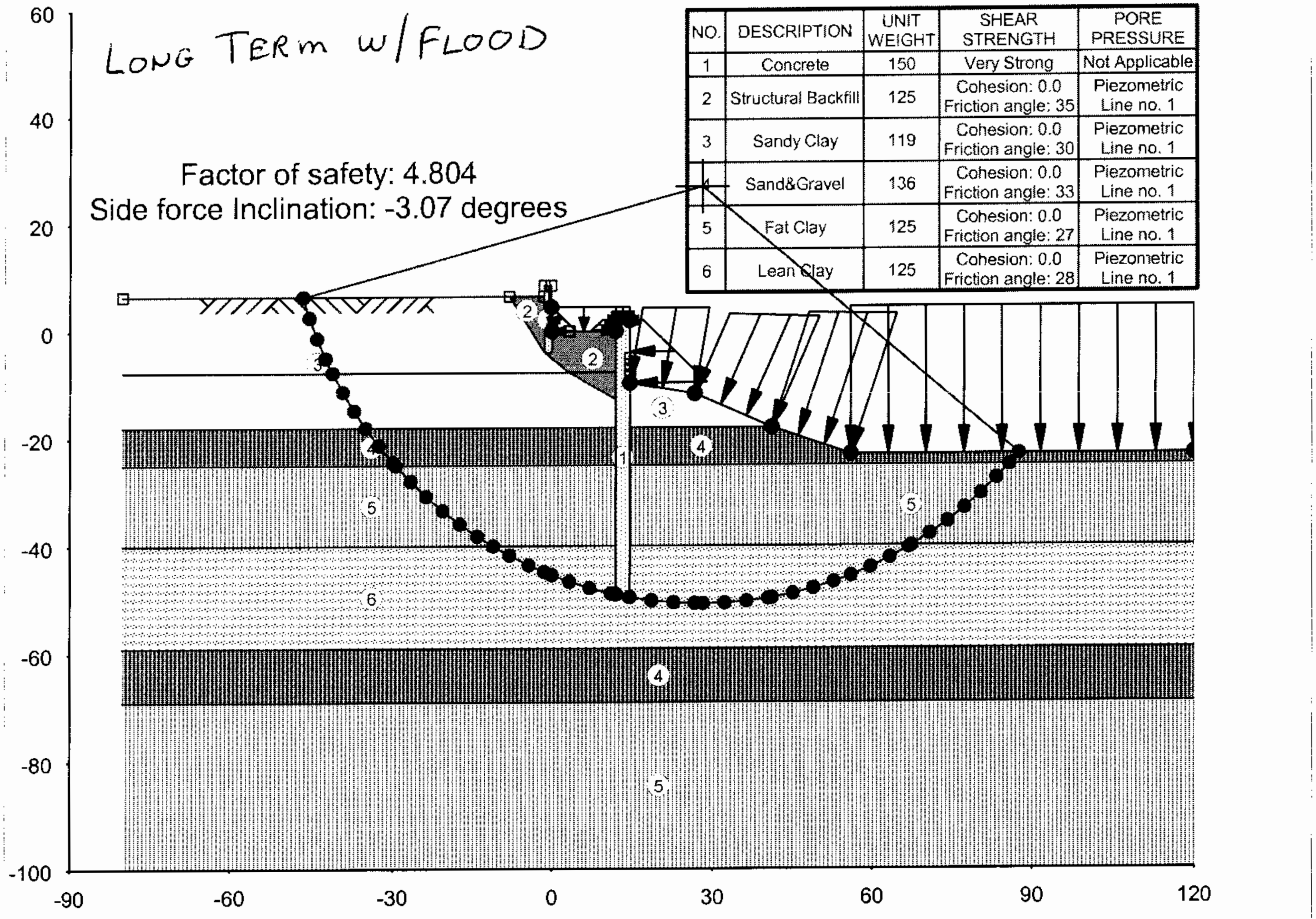
Date: Wed May 04 2005 Filename: C:\WINUT4\UTexas4 Samples\Napa2West\hatt2first\dualwallss.UT4 Time: 13:19:20

Napa Contract 2 West Hatt to First

LONG TERM w/FLOOD

Factor of safety: 4.804
Side force Inclination: -3.07 degrees

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Concrete	150	Very Strong	Not Applicable
2	Structural Backfill	125	Cohesion: 0.0 Friction angle: 35	Piezometric Line no. 1
3	Sandy Clay	119	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 1
4	Sand&Gravel	136	Cohesion: 0.0 Friction angle: 33	Piezometric Line no. 1
5	Fat Clay	125	Cohesion: 0.0 Friction angle: 27	Piezometric Line no. 1
6	Lean Clay	125	Cohesion: 0.0 Friction angle: 28	Piezometric Line no. 1



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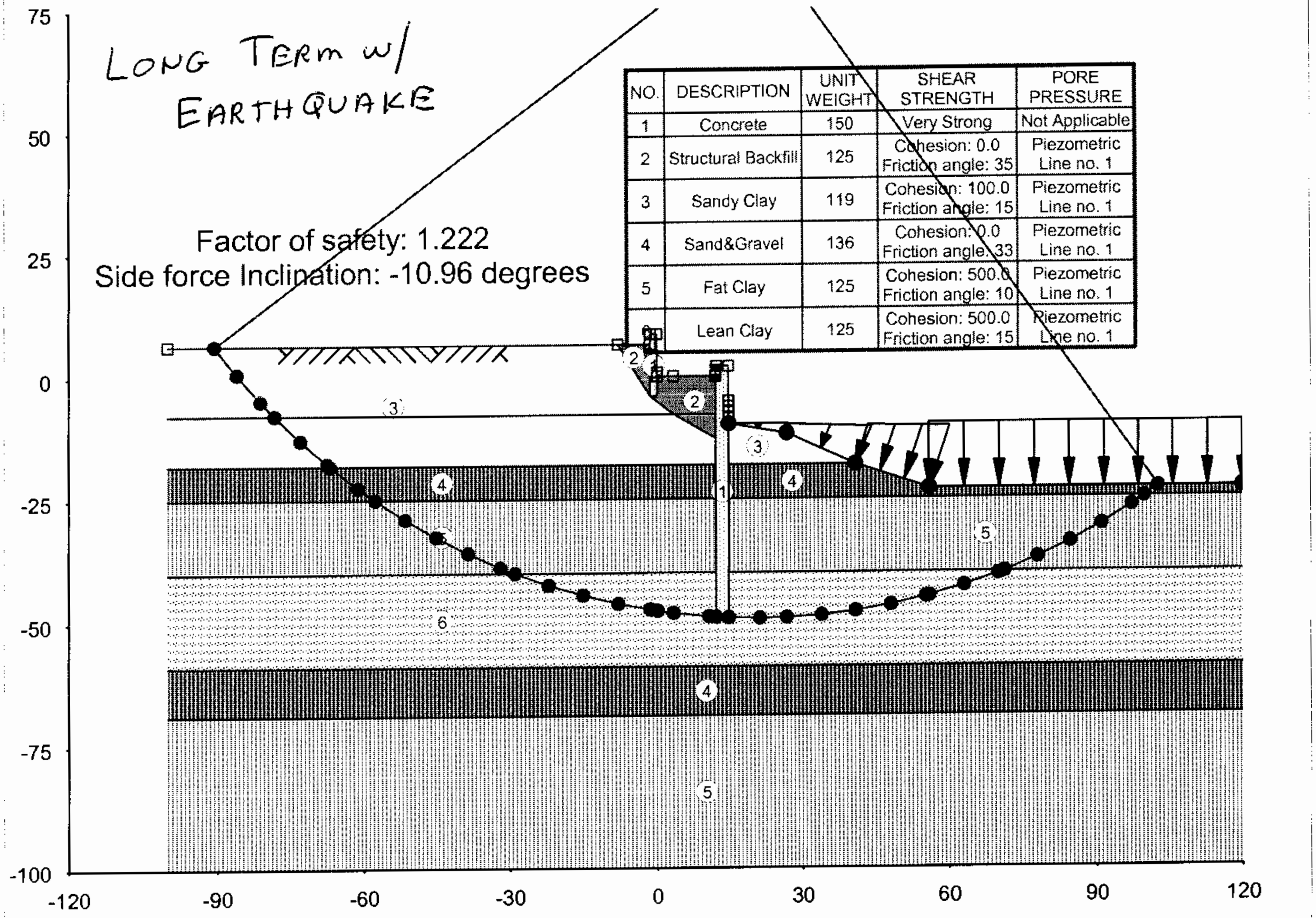
Napa Contract 2 West Hatt to First



LONG TERM w/
EARTHQUAKE

Factor of safety: 1.222
Side force Inclination: -10.96 degrees

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Concrete	150	Very Strong	Not Applicable
2	Structural Backfill	125	Cohesion: 0.0 Friction angle: 35	Piezometric Line no. 1
3	Sandy Clay	119	Cohesion: 100.0 Friction angle: 15	Piezometric Line no. 1
4	Sand&Gravel	136	Cohesion: 0.0 Friction angle: 33	Piezometric Line no. 1
5	Fat Clay	125	Cohesion: 500.0 Friction angle: 10	Piezometric Line no. 1
6	Lean Clay	125	Cohesion: 500.0 Friction angle: 15	Piezometric Line no. 1

Date: Wed May 04 2005 Filename: C:\WINUT4\UTexas4 Samples\Napa2West\hatt2first\dualwalleq.UT4 Time: 13:22:55

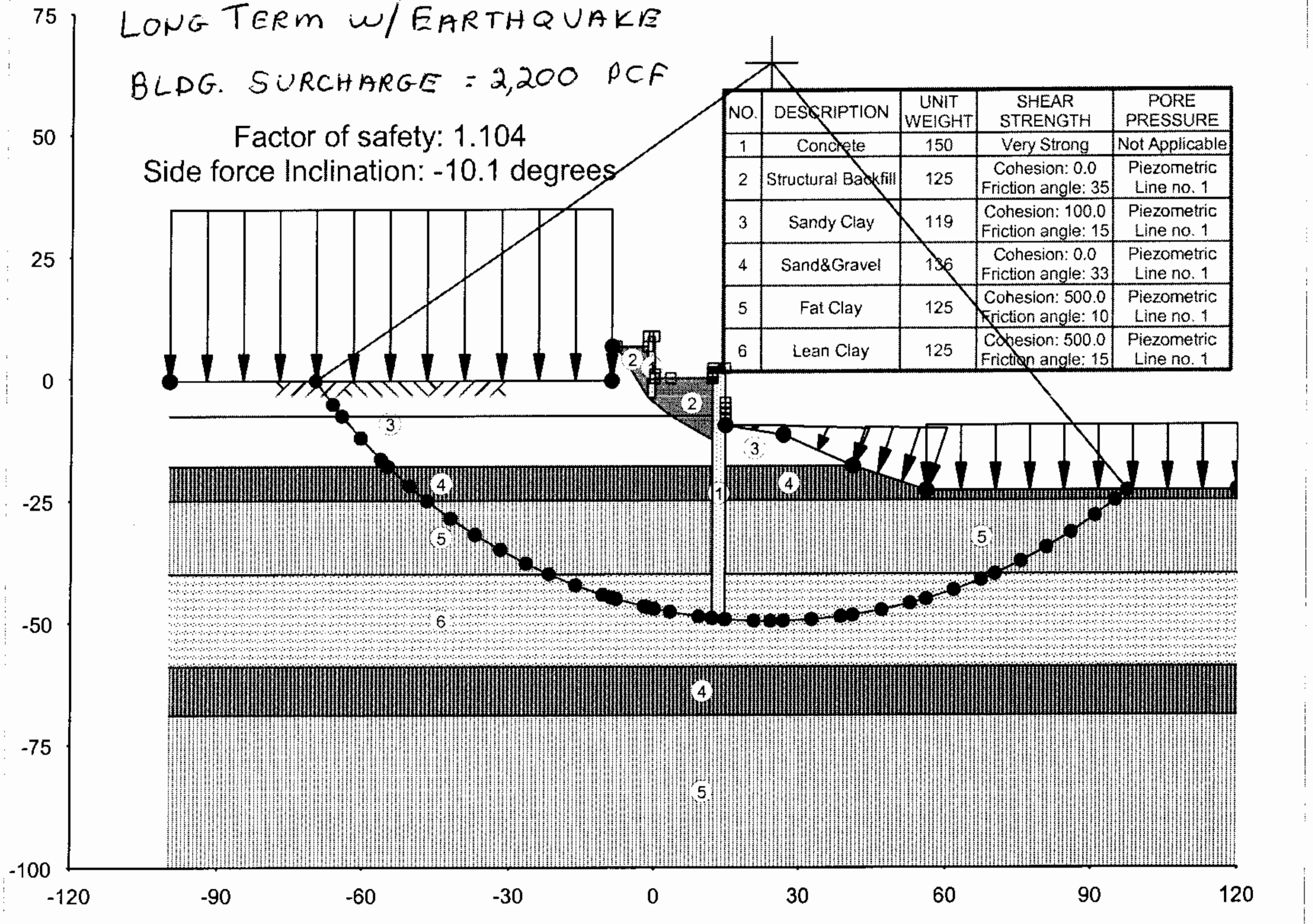
Napa Contract 2 West Hatt to First

LONG TERM w/ EARTHQUAKE

BLDG. SURCHARGE = 2,200 PCF

Factor of safety: 1.104

Side force Inclination: -10.1 degrees



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Concrete	150	Very Strong	Not Applicable
2	Structural Backfill	125	Cohesion: 0.0 Friction angle: 35	Piezometric Line no. 1
3	Sandy Clay	119	Cohesion: 100.0 Friction angle: 15	Piezometric Line no. 1
4	Sand&Gravel	136	Cohesion: 0.0 Friction angle: 33	Piezometric Line no. 1
5	Fat Clay	125	Cohesion: 500.0 Friction angle: 10	Piezometric Line no. 1
6	Lean Clay	125	Cohesion: 500.0 Friction angle: 15	Piezometric Line no. 1

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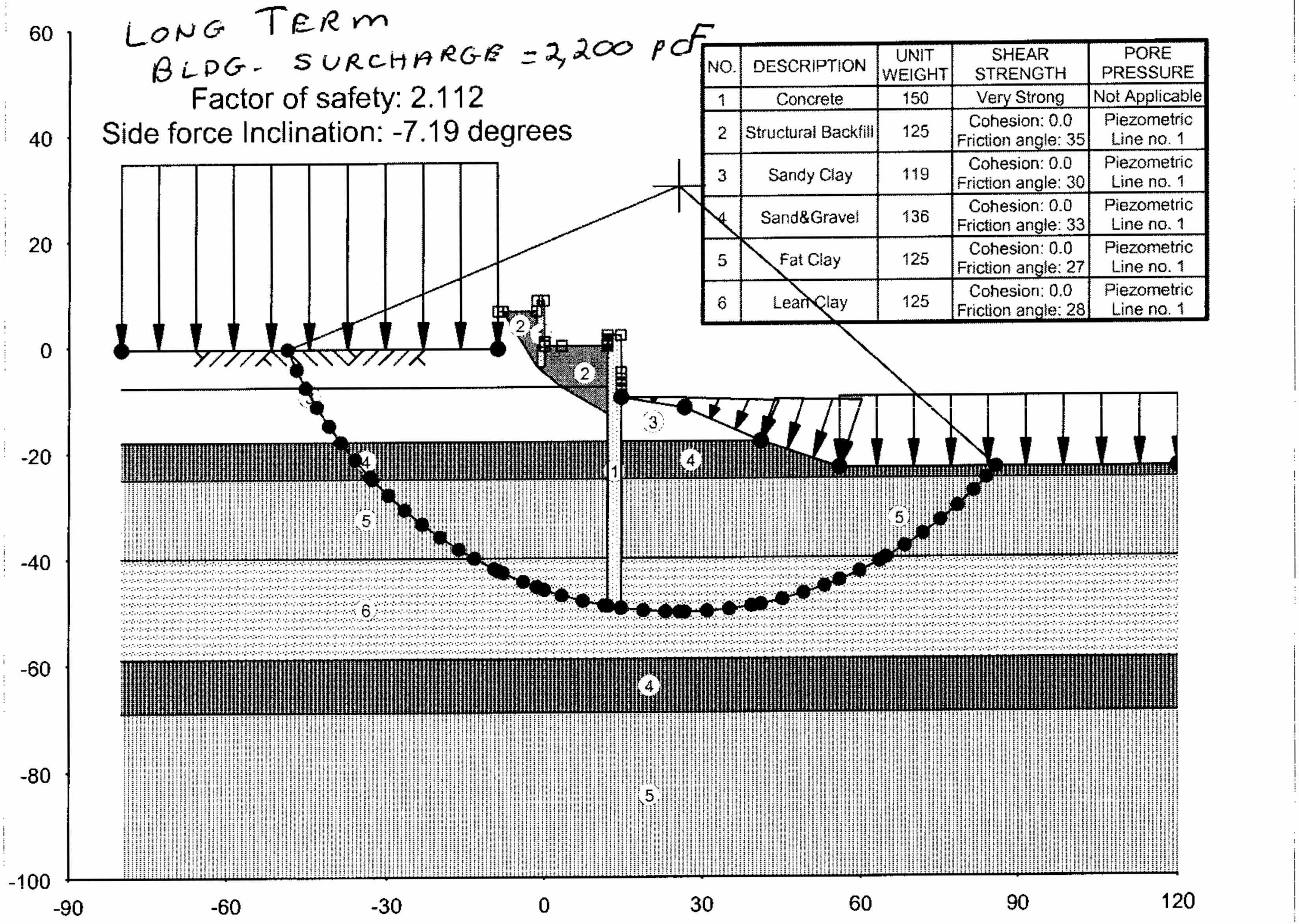
Napa Contract 2 West Hatt to First



LONG TERM
BLDG. SURCHARGE = 2,200 pcf

Factor of safety: 2.112
Side force Inclination: -7.19 degrees

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Concrete	150	Very Strong	Not Applicable
2	Structural Backfill	125	Cohesion: 0.0 Friction angle: 35	Piezometric Line no. 1
3	Sandy Clay	119	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 1
4	Sand&Gravel	136	Cohesion: 0.0 Friction angle: 33	Piezometric Line no. 1
5	Fat Clay	125	Cohesion: 0.0 Friction angle: 27	Piezometric Line no. 1
6	Lean Clay	125	Cohesion: 0.0 Friction angle: 28	Piezometric Line no. 1

Date: Wed May 04 2005Filename: C:\WINUT4\UTexas4 Samples\Napa2West\hatt2first\surchargess.UT4 Time: 13:26:02

APPENDIX 9: TELEPHONE CONVERSATION RECORDS

CONVERSATION RECORD

Person Called: Mark Andrilla, City of Napa Public Works, (707) 257-9520 x7423

Person Calling: Jane Bolton, CESPK-ED-GS, (916) 557-7637

Date of Call: January 12, 2005

I asked Mark if he could direct me to someone who had any knowledge about the experiences of the Third and/or First Street bridge contractors with installing sheet piles in the Napa River. Mark said he had been on site off and on throughout the construction of the pier cofferdams for both bridges. He said the sheet piles were installed with vibratory hammers. In some cases the piles went in easy, in some cases the piles were difficult to install. Only for the easterly pier of the First Street bridge did they encounter refusal with the vibratory hammer on some of the piles. In those cases, they switched to a diesel drop hammer and were able to install the piles with the drop hammer.

File: C:\Data\Napa\Cont2West2Hatt2First\bridgepiles

CONVERSATION RECORD

Person Called: Jane Bolton, CESPK-ED-GS, (916) 806-0239 (cell)

Person Calling: Bob Sennett, MGE Engineering, (916) 421-1000

Date of Call: February 1, 2005

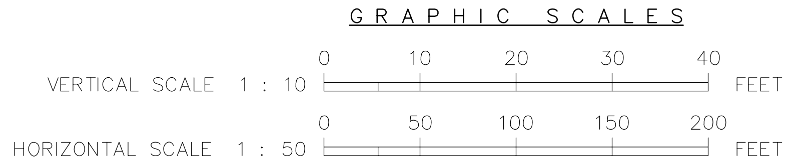
Bob asked me about the tip elevation of the piles for the first 248 lineal feet of the soldier pile wall. I told him I had given him a tip elevation of –60 ft. NGVD in an earlier email and he said he would look for it. We also talked about the possible impact of a building surcharge for Downtown Joes on the lower wall design. Bob said MGE will look at whether temporary shoring will be required during construction. I said it was possible the building surcharge would add to the loading on the lower wall, but given the elevations, the distance away from the wall, and the relatively light loading of Downtown Joes that an impact would likely be small, but MGE will look at it. I described the global stability analysis I did, that it took a building surcharge of 3,000 pounds per square foot to develop a critical global stability situation, and Bob said a building of that size and type would likely have a surcharge of about 1,000 pounds per square foot.

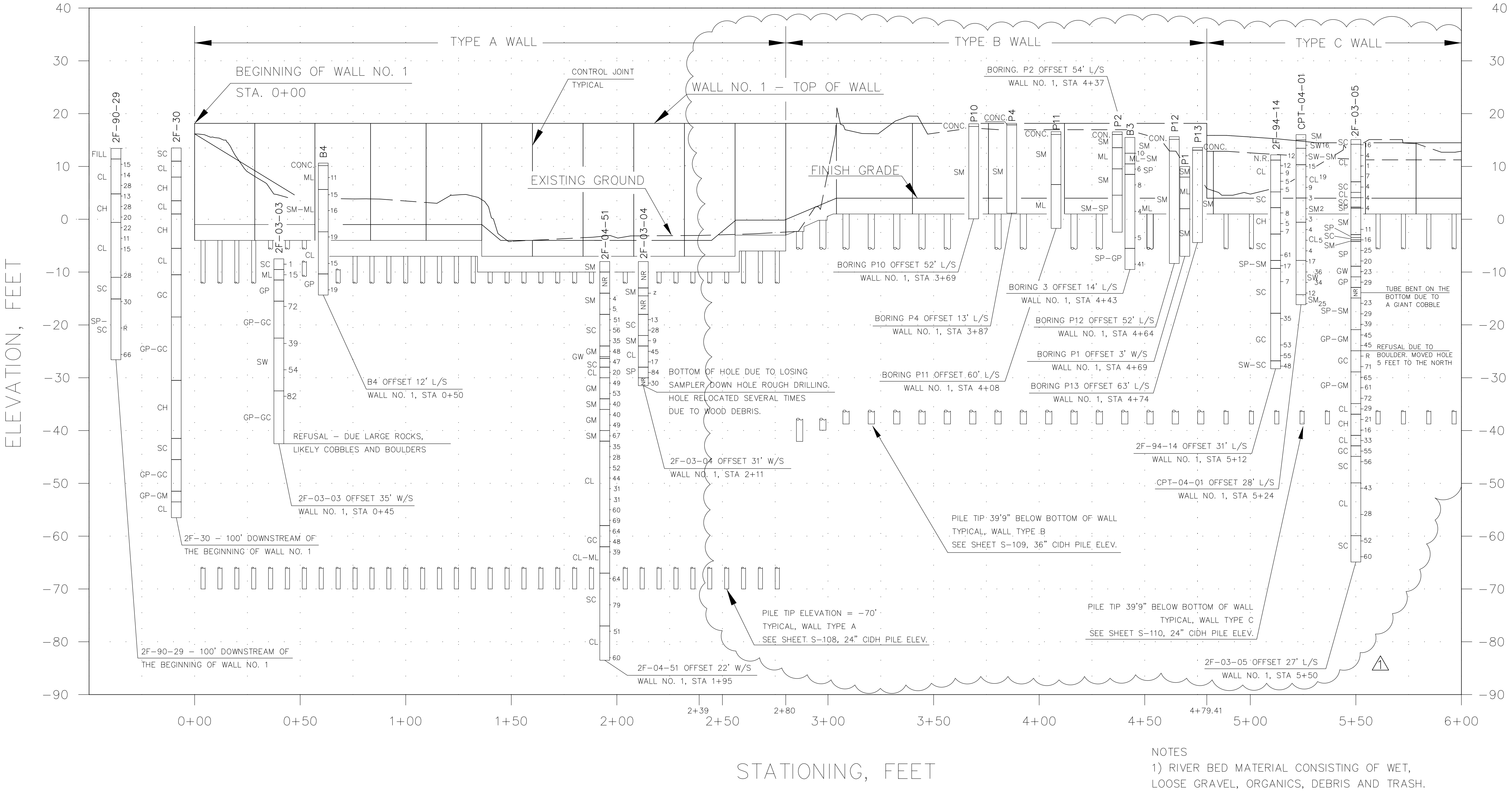
File: C:\Data\Napa\Cont2West2Hatt2First\bridgepiles

NAPA RIVER

GEOTECHNICAL PROFILE OF WALL NUMBER ONE

STATIONS 0+00 TO 6+00





NOTES

1) RIVER BED MATERIAL CONSISTING OF WET, LOOSE GRAVEL, ORGANICS, DEBRIS AND TRASH.

2) REFER TO SECTION 02020 SUBSURFACE DATA FOR ADDITIONAL INFORMATION.

US Army Corps
of Engineers
Sacramento District

Symbol	Description	Date	Approved
1	WALL NO.1 ALIGNMENT CHANGE	3/10/06	

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

SACRAMENTO DISTRICT
IN-HOUSE DESIGN
1325 'J' STREET
SACRAMENTO, CA 95814-2922

Designed by: J. BOLTON		Date: April 27, 2005	Rev.
Dwn by: P. FORTUNE	Spec No.:	Design file no: NA-25-030	
Reviewed by: MICHAEL D. RAMSBOTHAM		Drawing Code:	
Submitted by: /s/ BEN GOMPERS Chief, Soil Design. Sec.		File name: NA-25-030B-313.dw Plot date: Dwg scale: AS_NOTED	

NAPA CALIFORNIA

NAPA RIVER / NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (HATT BUILDING TO FIRST STREET)

STATIONS 0+00 TO 6+00
GEOTECHNICAL PROFILE UNDER WALL NO. 1

Sheet
reference
number:
B-313
21 of 166

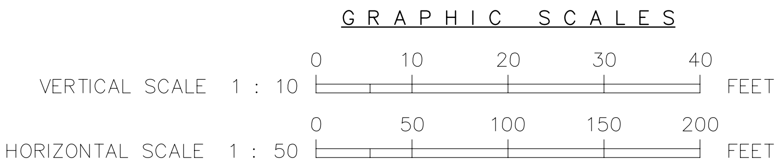
Reference – Exploration Log file no.: NA-01-034

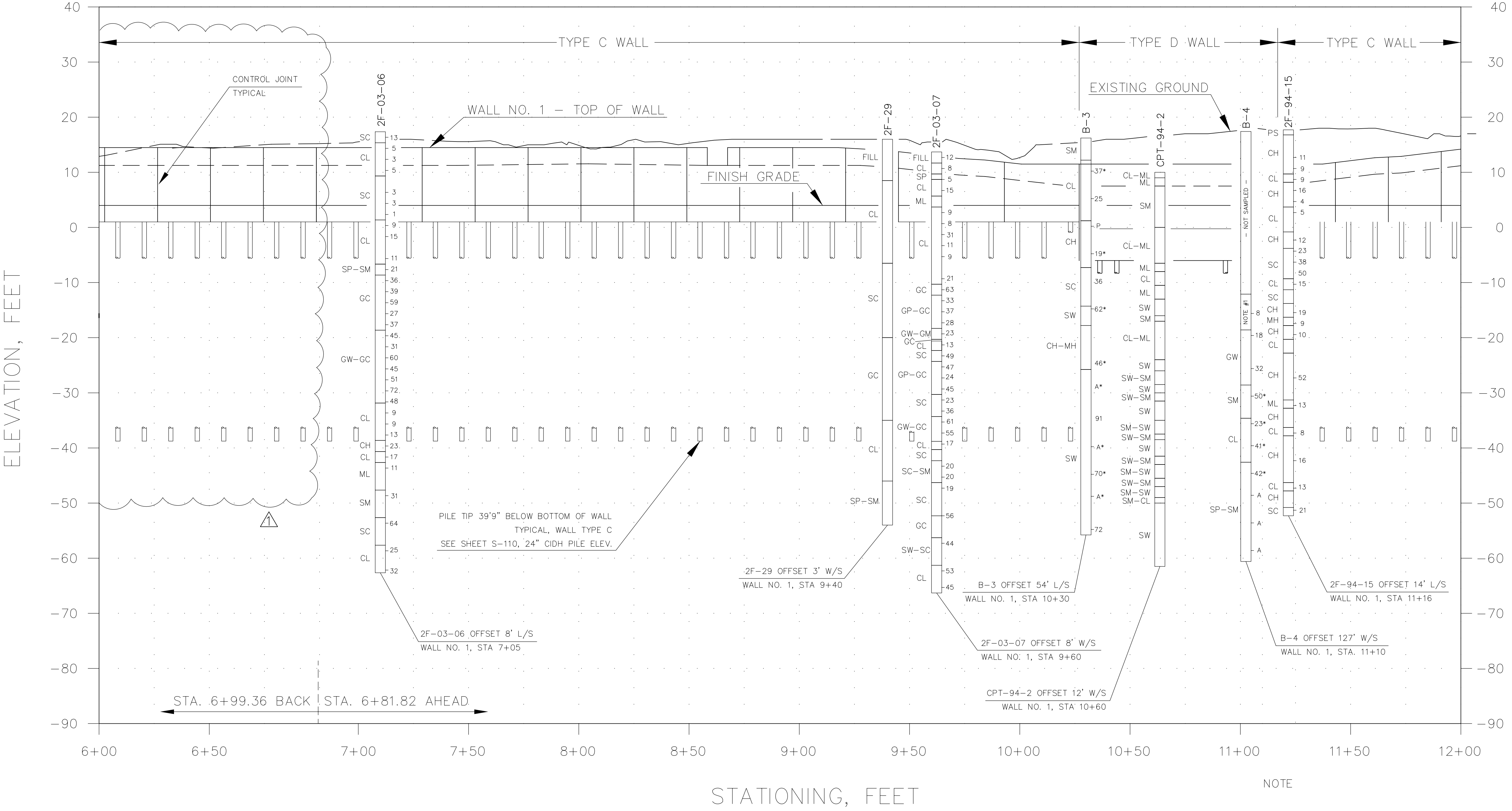
Soil Design Section Sheet No.: 13 of 15

NAPA RIVER

GEOTECHNICAL PROFILE OF WALL NUMBER ONE

STATIONS 6+00 to 12+00





NOTE

1) REFER TO SECTION 02020 SUBSURFACE DATA FOR ADDITIONAL INFORMATION.

US Army Corps
of Engineers
Sacramento District

Symbol	Description	Date	Approved
1	WALL NO. 1 ALIGNMENT CHANGE	3/10/06	

Designed by: J. BOLTON
Dwn by: P. FORTUNE
Spec No.:
Reviewed by: MICHAEL D. RAMSBOTHAM
Submitted by: /s/ BEN GOMPERS
Chief, Soil Design. Sec.
Date: April 27, 2005
Rev.
Design file no: NA-25-030
Drawing Code:
File name: NA-25-030B-314.dwg
Plot date:
Dwg scale: AS_NOTED

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

SACRAMENTO DISTRICT
IN-HOUSE DESIGN
1325 'J' STREET
SACRAMENTO, CA 95814-2922

NAPA CALIFORNIA
NAPA RIVER / NAPA CREEK
FLOOD PROTECTION PROJECT
CONTRACT 2 WEST (HATT BUILDING TO FIRST STREET)
STATIONS 6+00 TO 12+00
GEOTECHNICAL PROFILE UNDER WALL NO. 1

Sheet reference number:
B-314
22 of 166

Soil Design Section Sheet No.: 14 of 15

Reference – Exploration Log file no.: NA-01-034

Border Revised 08/30/04, COESHT02.DWG

Appendix D

2005 Hatt Bldg to First Street Floodwall

Design Documentation Report Supplemental

NAPA RIVER/NAPA CREEK FLOOD PROTECTION PROJECT

CONTRACT 2 WEST (HATT BUILDING TO FIRST STREET)

DESIGN DOCUMENTATION REPORT
(SUPPLEMENTAL TO THE USACE SGDM)

PREPARED FOR
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

CONTRACT NUMBER DACW05-01-D-0011

PREPARED BY

MGE ENGINEERING, INC.
7415 GREENHAVEN DRIVE, SUITE 100
SACRAMENTO, CA 95831

MARCH 2005

Table of Contents

1.0 Executive Summary

This report discusses the results of final level designs for the primary floodwall between the Napa Mill complex (Hatt Building) and the 1st Street Bridge as well as secondary walls retaining the upper promenade walkway for the Napa River/Napa Creek Flood Protection Project within the City of Napa, California. The purpose of this report is to document design features that differ from the concepts as shown in the Final Supplemental General Design Memorandum (FSGDM) prepared by the USACE and dated October 1998.

Project aesthetics and features utilized the recommendations documented in the *City of Napa, Downtown Riverfront Urban Design Plan,* February, 2003 and includes features such as river walk pedestrian access, surface finish aesthetics, lighting, planting, and redevelopment of Veterans Park. Generally, the recommendations outlined in the *Urban Design Plan* were incorporated into the Flood Protection project through direct improvements, or through design incorporating provision for future installation.

The primary floodwall can be separated into two distinct portions: 1) Napa Mill Complex (from the beginning of the wall to 5th Street, and 2) 5th Street to the 1st Street Bridge. The portion of the wall at the Napa Mill Complex extends into the river channel initially before entering the existing patio area, paralleling the hotel addition, and ending at 5th Street. The design height of the wall varies from approximately 16-feet to 24-feet, transitioning to a design height of approximately 12-feet at 5th Street at the beginning of the lower promenade. The remaining portion of the wall terminates at an existing concrete wall adjacent to Riverside Auto and just southeast of the existing 1st Street Bridge. The design height of this portion of the wall varies from approximately 10-feet for the majority of the wall limits to approximately 7-feet where the lower promenade passes below the existing 3rd Street Bridge.

The FSGDM indicated the use of a soldier pile wall with precast concrete panels and tieback anchors for the primary lower floodwall, and conventional cast-in-place, reinforced concrete cantilever walls with spread footings for the upper setback walls.

Additional investigations regarding appropriate wall types to be utilized were performed. Wall type selection considerations included:

1. Constructability
2. Cost
3. Aesthetics

The following wall types were considered:

1. Cast-in-place (CIP), reinforced concrete cantilever soldier pile supported on 24-inch and 36-inch cast-in-drilled-hole (CIDH) concrete piles,

2. CIP, reinforced concrete cantilever wall supported on 24-inch CIDH concrete pile footings,

3. Soldier pile wall with precast concrete panels, and
4. Soil nail tieback wall with CIP concrete facing.

Recommended wall types for the various portions of the project were as follows:

1. Napa Mill Complex:
    a) Beginning of wall (Station 0+00) to Station 2+56 - Cast-in-place, reinforced concrete cantilever wall supported on 24-inch diameter CIDH concrete pile footings.
    b) Station 2+56 to Station 4+68 - Cast-in-place, reinforced concrete soldier pile wall supported on 36-inch diameter CIDH concrete piles.
2. 5th Street to Existing Wall at Riverside Auto: Station 4+68 to 16+40 - Cast-in-place, reinforced concrete soldier pile wall supported on 24-inch diameter CIDH concrete piles.
3. Upper Level Promenade Walls: From 5th Street to the Main Street Landing, from the Main Street Landing to the south side of the 3rd Street Bridge, from the north side of the 3rd Street Bridge to Veterans Park, and from Veterans Park to the First Street Bridge - Cast-in-place, reinforced concrete, cantilever walls supported on spread footings.

In addition to the construction of the new primary floodwall, the existing floodwall below the 3rd Street Bridge will be modified to facilitate the raising of the lower promenade as well as to complete the connection of the new primary floodwall at each side of the bridge.

2.0 Design Criteria

2.1 Floodwalls and Retaining Walls

Floodwalls and retaining walls will be designed in accordance with the design criteria in Section 18.2 of the FSGDM with the following exceptions:

1. Minimum concrete strength of 4,000 psi specified.
2. Vehicle loadings considered only at wall locations where vehicle access is feasible.

2.2 General Design Criteria

2.2.1 Primary Flood Wall Alignment

Final Design for this construction contract began with the finalization of the primary floodwall alignments. In general, the alignments as shown on the *Urban Design Plan* were compared with the alignments used for hydraulic analysis prepared by the USACE. Minor modifications to the USACE alignment were proposed to meet the intent of the *Urban Design Plan*, and affects of this realignment were adopted after analysis determined that desired flood protection was achieved and the alignments were acceptable to the City of Napa. Specific alignment changes that were adopted include:

1. Bulb radius at the 4th Street Boat Dock – Radius and location of the curve were modified to better center the access stairs and provide promenade width.

2. Radius and tangent alignments at Veteran's Park Bandstand – Radius and tangent alignments were modified to highlight the bandstand area, and provide desired promenade widths. Additionally, the center of the ellipse was shifted to center on the park and provide adequate ramp length to comply with accessibility requirements.

2.2.2 Location of Flood Protection Elevation (FPE)

The flood protection elevation (with freeboard) was defined during finalization of the floodwall alignments. In general, the flood protection elevation from south to north is defined as follows:

1. Begin Floodwall #1 – FPE at 17.50' located at beginning of promenade, Station 0+00. Protection is maintained at or above this elevation to Station 4+68 where FPE turns west to top of Ramp/Stairs Access No. 1 at 5th Street.

2. Begin Floodwall #2 – FPE from beginning of Wall #2 to top of Ramp/Stairs Access #2 at 4th Street.

3. Begin Floodwall #3 – FPE from beginning of Wall #3 to end of Wall #3 at the south side of the Third Street Bridge abutment. FPE is along the face of the existing Third Street Bridge abutment to the beginning of VP Wall #1.

4. Begin VP Wall #1 – FPE from beginning of VP Wall #1 to approximate station 0+90 where the FPE travels perpendicular to the VP South Ramp to Terrace Wall No. 1.

5. VP Terrace Wall # 1 – FPE travels along Terrace Wall #1 from beginning to end, then follows VP Wall #4 to the top of VP Ramp #3.

6. Flood protection across VP North Ramp is provided by a temporary flood wall (Stop Log) assembled prior to high water.

7. Wall #6 – FPE from beginning of Wall #6 to End of Wall # 6 at Ramp/Stair Access No. 3.

8. Top of Ramp/Stair Access No. 3 northern wall to terminus of Wall #1.

2.2.3 Structural Concrete Pavement Design

The structural concrete pavement for the lower promenade is intended to accommodate pedestrian loading only, and was therefore designed as a reinforced concrete slab 4 inches thick. The upper promenade is intended to accommodate pedestrian, maintenance truck and emergency vehicle loading and was designed as a reinforced concrete slab 6 inches thick.

Because of the difficulty of pavement replacement after construction and the phasing of construction of the upper promenade, the design team determined that a geotextile would be utilized to separate the pavement structural section from the pavement subgrade. This geotextile will prevent intrusion of subgrade materials into the structural section and provide extended service life. Additionally, the geotextile will serve to uniformly distribute pavement loading to the subgrade and prevent localized failure. It is believed that utilization of the geotextile to reinforce the structural pavement section is justified to extend the useful life of the pavement and reduce the long-term maintenance burden on the owner.

2.2.4 Ramps and Stairs

The three sets of ramps and stairs at 5th, 4th and 1St street have been designed to meet ADA and pedestrian loading requirements.

2.2.5 Veterans Park Amphitheater

The Veteran's Park Amphitheater was designed in accordance with the concepts outlined in the *Urban Design Plan.* There are three points of access from Main Street and access to the lower promenade pedestrian path. Access between Main Street and the lower promenade meets ADA requirements. Maintenance vehicle access is limited to the bottom of the main north and south ramps.

The Bandstand Plaza was located above the 10-year flood elevation, with a series of turf-planted elliptical terraces forming the amphitheater. All terraces provide wheelchair access and wheelchair parking. The park is contained on the north and south by landscaped planters.

Electrical power for lighting, water for a drinking fountain and landscape irrigation and subsurface drainage has been provided. A foundation and electrical conduits for future power to the bandstand have also been provided.

Many of the existing improvements within the park will be salvaged. Four light fixtures will be relocated from within the park to the park's Main Street frontage. The pedestrian sidewalk along Main Street will be replaced.

2.3 Design Modifications from the *Urban Design Plan*

Several design modifications to the *Urban Design Plan* have occurred to the flood protection project. In general, these modifications have been dictated by schedule, adjacent property owner requirements or compliance with state and local codes. These modifications are summarized below:

2.3.1 Hatt to 5th Street

The design alignment from the beginning of the project to 5th Street primarily follows the alignment shown in the Urban Design Plan. A short section of the existing pedestrian walkway required removal and replacement to meet the required project flood control elevations. Initial discussion contemplated access to the promenade from the corridor between the Angele Restaurant and the outdoor covered dining area fronting the Main Street parking lot. This access was discounted after discussion with the property Owner.

Discussions concerning continuous pedestrian access between the beginning of the project and the 5th Street access required several iterations of design in an attempt to provide continuous pedestrian access and provide continued use of the patio for Hatt building customers. Due to the alignment of the floodwall in proximity to the newer Hatt Residential Suites, it was determined that sufficient Right-of-Way would not be available in a post project condition to provide sufficient walkway width without significant impact to the suites. As a result, the design team was directed to eliminate the walkway between the floodwall and the Hatt Suites building in favor of an unimproved area to be landscaped after completion of the flood control project.

Construction setback requirements dictated a significant impact to the patio/dining area between the primary floodwall and the existing Hatt Building. The design team assembled design documents for the reconstruction of the Hatt patio as part of the flood control project.

2.3.2 5th Street to 3rd Street

The project design between 5th Street and 3rd Street matches the *Urban Design Plan*. The access ramp/stairs were mirrored to eliminate conflicts between stair and ramp users.

As a result of pending commercial improvements (Channel Development) planned for the undeveloped area bounded by Main Street and the flood control project and by 5th and 3rd Streets, the design team determined that portions of the flood control project could not be constructed prior to the Channel Development construction without damage to flood project improvements. This led to the development of a Matchline between the projects to clearly define limits of construction and to define finish elevations between the projects to be used for drainage and other improvements.

Conceptual design improvements for the terminus of 5th Street were completed by the design team, however, responsibility for final design and construction will be left to the Channel Development Project. A similar division of design and construction responsibility was developed for the area south of the Third Street Bridge and Main Street.

At the request of the City of Napa, the ramp/stair access at 4th Street was redesigned by the author of the *Urban Design Plan* after the 50% plan submittal to revise the aesthetics and operations of the stairs and access ramp. The redesign replaced the symmetrical concept with the asymmetrical design presented in the final plans.

Accommodation for the 4th Street boat dock and ramp was included in the design. Meetings and shared design files with Charles Rauw, the City of Napa dock and ramp designer, were utilized to provide future dock and ramp accommodations without compromise to the goals of the flood control project.

2.3.3 Below the 3rd Street Bridge

The existing solider-pile wall and walkway below the 3rd Street Bridge was constructed several feet below the required elevation. As a result, the design team prepared a design to raise the surface elevation of this portion of the lower promenade. Limitation to the finish height of the walkway was determined by the soffit elevation of the bridge and the height of an existing waterline hung from the underside of the bridge.

The knowledge that this portion of the lower promenade will be subject to flooding and flood debris led the design team to provide longitudinal and transverse surface drainage for this portion of the walkway. In addition, a removable grate leading to a river drain protected by a flap gate will also aid in the post-flood recovery.

Accommodation was also provided for existing bridge deck and abutment drain-line extensions.

2.3.4 Veterans Park

Design and layout of Veterans Park (VP) followed the concepts prescribed in the *Urban Design Plan* thorough 35% design. A slight modification to the floodwall alignment was approved, moving the floodwall closer the river centerline. This shift allowed the design to more closely reflect the concept of the *Urban Design Plan.*

Between the 35% and 50% design, analysis by the City of Napa Accessibility Consultant required that the northern VP stairs proposed in the *Urban Design Plan* be replaced by accessible ramps and that all terraces be provided with handicap accessibility and wheelchair parking. In order for the ramp and terrace access to meet accessibility requirements, finish surface elevations below the 100-year protection elevation were required on the VP north ramp. As a result, a flood control bulkhead has been provided to meet the project goals.

The bulkhead consists of self-sealing, 6” high by 14’ long flood panels. The flood panels have been provided with on-site, secure storage for easy deployment prior to high water. The installation and maintenance responsibility for this bulkhead will be provided by the Napa County Flood Control Agency.

Per the *Urban Design Plan* a drinking fountain has been provided at Veterans Park. Due to vandalism protection and to prevent potable water cross-contamination during flood events, the drinking fountain has been located outside the flooding zone.

2.3.5 Veterans Park to 1st Street

The northern boundary of Veterans Park is shared by the 'Downtown Joe's' restaurant (DJ). In order to reduce the impact to the DJ dining patio, the north and south VP ramps and the northern VP planter was slightly reduced in size between the 50% and 95% design submittals. This shift minimized the impact to the DJ dining patio, and located Wall #6 in the location of the current patio wall.

DJ operates a dining room in a building that has been constructed on cantilevered steel moment frames. The design team requested and obtained a shift in the primary flood wall at this location towards the river centerline in order to provide room to construct the upper wall near this dining room. In addition, DJ utilizes the area below the dining room for equipment and restaurant storage. To protect and allow continuation of this use, the design team provided a retaining wall and drainage below the dining room to allow for backfill and drainage of the upper floodwall without compromise to the DJ storage area.

The terminus of the reach of this contract is at an existing un-reinforced masonry wall located south of the bridge at 1st Street. The ramp and stairs providing access to the lower promenade at this location mirrors the ramp/stairs at 5th Street. The design of this area reflects the *Urban Design Plan*. Conformance to existing asphalt pavement and pedestrian walkways has been provided. The relocation of the existing trash enclosure removed by the project is the responsibility of the City of Napa.

A 6' wide walkway at the level of the upper promenade has been provided at the northern terminus of the project. This 'half-section' of the promenade is intended as a provision for future widening and river walk extension with development to the north.

3.0 Wall Type Recommendations

Selection of the various wall types for the limits of the project considered constructability, cost, and aesthetics. Constructability issues included: limited areas and lengths for soil nail tieback anchors, subsurface debris, construction below the mean lower low water elevation, areas of cut and fill configurations, and construction adjacent to existing buildings and public facilities. Desired wall aesthetics consist of a rusticated block pattern achieved through the use of form liners.

3.1 Napa Mill Complex (Beginning of Wall to 5th Street)

Within this portion of the wall limits, tieback anchors could not be considered due to the close proximity of the wall to the existing structures as well as the lack of complete "as-built" plans. Wall types considered consisted of a cast-in-place, reinforced concrete soldier pile wall supported on 36-inch diameter cast-in-drilled-hole concrete piles, and a conventional cast-in-place concrete cantilever wall supported on a concrete pile footing utilizing 24-inch diameter cast-in-drilled-hole concrete piles. Analysis results indicated that at the tallest regions of the wall, displacements were not acceptable considering a soldier pile type wall without tieback anchors. Thus, for wall heights greater than about 16 feet, the cantilever wall with a pile footing utilizing 24-inch CIDH concrete piles is required. Considering the higher cost of the cantilever wall on pile footings, a transition to the soldier pile wall is recommended as soon as the design height drops below about 16-feet.

3.2 5th Street to 1st Street (Riverside Auto)

Within this portion of the project, the wall limits between station 4+68 (5th Street) to station 14+07 (just north of Downtown Joe's restaurant), a soil nail type tieback anchor wall is a feasible alternative to a conventional cast-in-place concrete soldier pile wall utilizing 24-inch CIDH concrete piles. From station 14+07 to station 16+40 (end of wall), a CIP soldier pile wall supported on CIDH concrete piles is required due to the wall height and existing grade fill conditions. The use of a wall type utilizing precast concrete panels was also considered. However, the expected need to make slight location adjustments of soldier piles as a result of the presence of subsurface debris would result in the need to cast additional non-uniform panel widths. Considering the potential negative affects on the aesthetics in addition to likely construction change orders, the decision was made to not further evaluate a precast concrete wall type alternative.

Further investigations revealed that a cast-in-place reinforced concrete soldier pile wall with 24-inch CIDH concrete piling spaced at 12-feet would be adequate to satisfy the design requirements for the majority of the limits of the wall. Cost of the wall was estimated to be $74/SF. Costs for a soil nail type tieback wall were estimated to be $66/SF. Potential cost savings considering construction of a soil nail wall between stations 4+68 and 14+07 was estimated to be $96,000 (2.6% of total wall cost).

Considering the relatively small potential cost savings of constructing a soil nail wall, and considering the advantages of utilizing one wall type for the lower promenade, the cast-in-place soldier pile wall was recommended.

4.0 Upper Promenade Walls

At upper promenade wall locations, conventional cast-in-place, reinforced concrete cantilever walls supported on spread footings were recommended due to the advantages of cost, constructability, and aesthetics over other wall types considered.

5.0 Wall Aesthetics

Desired wall aesthetic treatment consists of a rusticated block pattern achieved through the use of form liners, and matching those used on the new Third Street Bridge retaining/wing walls. Walls will incorporate pilasters, coping, and metal tube hand and picket railings and other features as described in the Downtown Riverfront Urban Design Plan prepared by the City of Napa and dated February 2003.

6.0 Project Utilities Design

Project utilities design consists primarily of storm drainage, landscape irrigation supply, domestic and fire water supply at Veteran's Park, power for project site lighting, and a sanitary sewer connection for the Veteran's Park drinking fountain.

6.1.1 Storm Drainage

The City of Napa, who will be responsible for storm drainage maintenance, requested that primary storm drainage pipe be a minimum of 12" diameter and utilize Class 3 Reinforced Concrete pipe (RCP).

In general, the project directs surface drainage within the boundary of the project into subsurface drainage systems that outlet through the lower floodwall. Where the lower promenade has little or no slope, the surface pavement has been graded away from the lower wall towards the upper wall and into longitudinal trench drains. These trench drains are piped under the lower promenade and through the lower wall. All wall penetrations have been provided with gate boxes and flap gates to prevent debris and flood water from entering the storm drain system.

From the beginning of the project to the Hatt patio, the promenade has been sloped to drain into inlets that are vented through the wall to the river. The Hatt patio has surface drainage directed into drainage inlets that vent thorough the wall to the river. These inlets have been provided with additional connections to accept drainage from the rest of the Hatt patio. The landscaped area between the Hatt Suites and the flood wall is drained by area inlets that vent through the wall to the river.

Between 5th and 3rd Street the upper promenade has been sloped away from the upper wall towards Main Street where storm water is collected in longitudinal trench drains. Drainage between 5th and 4th Street is collected by trench drains, conveyed in RCP pipe to the existing 5th Street drainage system. Upper promenade drainage between 4th and 3rd Streets is collected by trench drains and conveyed to the existing storm drain system in Main Street near 3rd Street.

The lower promenade between 5th and 4th Street is drained by longitudinal trench drains at the base of the upper wall. The lower promenade platform at 4th Street has been graded to surface drain north and south. The southern drainage is directed into the trench drain system, and the northern portion is directed down the promenade grade to the crossing below the 3rd Street Bridge.

Surface drainage below the 3rd Street Bridge has been provided by longitudinal trench drains that are collected into an inlet structure near the centerline below 3rd Street. This inlet has been provided with a removable decorative grate to facilitate removal of flood debris. This inlet structure is vented through the lower wall into the river.

At Veteran's Park the sidewalk along Main Street is graded to drain into the existing Main Street storm drain system. The north and south ramps are graded to direct surface drainage to the bandstand platform. The bandstand platform is graded to drain into two decorative, grated inlets on either side of the center stairs. The bandstand platform inlets are vented through the lower flood wall via an 18" RCP. The Veteran's Park terraces are graded to drain into numerous area drains that are directed into the bandstand platform drainage inlets. At the request of the City of Napa, this terrace and terrace wall drain system has been provided with clean-outs to facilitate maintenance access.

The promenade between Veteran's Park and 1st Street is graded towards the park bandstand platform. The platform above the 1st Street ramp/stairs is graded to the 1st Street parking lot, where drainage is directed via curb and gutter to a subsurface system through the lower wall to the river.

6.1.2 Water Supply

Water supply for the project is provided at four locations. Irrigation water supply for planting from the beginning of the project to 5th Street will be provided by the Owner of the Hatt building and utilizes an existing supply system.

Water supply for irrigation between 5th and 3rd Street will be provided from a new supply developed from a City of Napa Water main that terminates at the end of 5th Street. This 1-1/2" supply is provided with an irrigation meter and reduced pressure backflow prevention device meeting City of Napa Water Division Standards.

Water supply for Veterans Park and the landscaping needs to 1st Street will be provided via a new water supply at 3rd and Main tapped from the City of Napa water main within Main Street. This 1-1/2" water supply will provide both potable and non-potable supply for the project north of the 3rd Street Bridge. The water meter is relocated from its pre-project location in the Main Street sidewalk.

The existing water supply tap for Veteran's Park will be utilized as the required fire service supply and is piped to the bandstand location.

6.1.3 Sanitary Sewer

A single 4" sanitary sewer connection is required to service the drinking fountain located in the northwest corner of Veteran's Park. The sewer service will be connected to the existing City of Napa collection system near the intersection of Main and 2nd Street.

6.1.4 Electrical Power Supply

Two electrical service sources are required for this project. Electrical power for the project south of the 3rd Street Bridge will be supplied from a shared transformer (with Channel Properties) located at the end of 5th Street. The electrical power for the project north of the 3rd Street Bridge will be supplied from a new transformer to be located in Veterans Park. The location of this service will be determined after Application for Electric Service is completed by the City of Napa.

7.0 Project Construction Staging and Phasing

7.1.1 Construction Staging

The Owner of the Hatt building and the Hatt Suites requested that construction be staged so that simultaneous construction will not occur below the Hatt Building and the Hatt Suites. Contract documents require that wall from the beginning of the project to approximate Station 2+50 not be constructed at the same time as the wall from Station 2+50 to 5th Street, thereby complying with the Hatt building Owner's request.

To accommodate the Napa County Flood Protection District and the Channel Development project, the walls between 5th and 3rd Street are required to be completed prior to May of 2006. The contract documents require that this area be completed prior to the above date.

7.1.2 Construction Phasing

To accommodate the construction of the Channel Development properties between 5th and 3rd Streets, the design team has made provision to defer the construction of the upper promenade and the trench drains until after completion of the Channel Development project. This deferral allows the Channel Development to complete their construction and not damage flood control project improvements and provide them with construction staging areas. The access stairs and ramps, the lighting and trees and the underground utilities will be installed as part of the flood control project.

8.0 Storm Water Pollution Prevention

Standard Best Management Practices (BMP's) have been included in the contract documents for construction storm water treatment. BMP's include inlet protection, temporary erosion control planting, stabilized construction entrances and concrete wash down areas.

To protect the water quality of the Napa River, the project requires that a continuous Turbidity Curtain be placed in the water for the length of the project. The Contractor is given direction concerning material, installation, repair, maintenance and removal of the Turbidity Curtain.

Permanent storm water quality is protected by the paving, landscaping and drainage systems that will be part of the constructed improvements. These temporary and permanent measures guarantee compliance with the requirements of the project Environmental Document as well as local, state and federal regulations.

9.0 References

1. MGE Engineering, *Napa River/Napa Creek Flood Protection – Wall Type Selection, Contract 2West (Hatt Building to First Street), Technical Memorandum*, June, 2004.
2. USACE Sacramento District, *Napa River/Napa Creek Flood Protection Project – Final Supplemental General Design Memorandum, Main Report,* October, 1998.
3. City of Napa, *Downtown Riverfront Urban Design Plan*, February, 2003.

Napa River / Napa Creek Flood Protection Project

Contract 2West (Hatt Building to First Street)

STRUCTURAL DESIGN CALCULATIONS
(100% SUBMITTAL)

Prepared for
U.S. Army Corps of Engineers
Sacramento District

Contract Number DACW05-01-D-0011

REGISTERED PROFESSIONAL ENGINEER
Ding An
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March 2005

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MGE ENGINEERING, INC.

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Subject: Design Criteria Summary
By: David An
Date: Mar-05

DESIGN CRITERIA SUMMARY

Analyses and design of flood wall reinforced concrete structures are based on "Final Supplemental General Design Memorandum" dated October 1998, using the following parameters:

Materials:

Concrete: f'c = 4,000 psi
n = 8
Reinforcement: Fy = 60,000 psi

Loading:

"DEPARTMENT OF THE ARMY, Sacramento District, Corps of Engineers"
"Soldier Pile and Sheet Pile Wall Load Conditions & Load Diagrams"
H-15: AASHTO
D-4 Dozer: V = 2.5 k/ft
Live Load Considerations:

Wall #1: Type A: No Vehicles Considered Due to Access
Type B: D-4 Dozer Only
Type C: D-4 Dozer Only

Wall #2: H-15 OR D-4 Dozer
Wall #3: H-15 OR D-4 Dozer
Wall #4: D-4 Dozer Only
Wall #5: D-4 Dozer Only
Wall #6: No Vehicles Considered Due to Access
Ramp/Access Walls: No Vehicles Considered Due to Access
Veterans Park Walls: No Vehicles Considered Due to Access

Backfill Materials Properties

Backfill Unit Weight = 125 pcf
Φ = 37 degree
C = 0 pcf
SMF= $\mathrm{Tan}(\Phi_d)$ / $\mathrm{Tan}\Phi = 2/3$ = 0.67
Φ_d = 27 degree
Ka = Tan^2 (45° - $\Phi/2$) = 0.25
Ko = Tan^2 (45° - $\Phi_d/2$) = 0.38
Kp = Tan^2 (45° + $\Phi/2$) = 4.02

Water Property

Water Unit Weight = 62.5 pcf

Soil Profile (from US Army Corps of Engineers)

1. Station 0+00 to 2+00

Elevation	Depth	Thickness	Soil Type	Unit Wt	Shear Strength	Phi	
-8 -33	25'	25	Dense Sand & Gravel	Moist = 115 pct Sat = 120 pct		38	
-58	50'	25	Stiff, Slightly O.C. Clay	Moist = 128 pct Sat = 136 pct	c = 1200 psf		
-68	60'	10	Dense Sand & Gravel	Moist = 115 pct Sat = 120 pct		38	
-74	66'	6	Stiff, Slightly O.C. Clay	Moist = 128 pct Sat = 136 pct	c = 1200 psf		

2. Station 2+00 to 4+75

Elevation	Depth	Thickness	Soil Type	Unit Wt	Undrained Shear	Drained Shear R Strength	Drained Shear S Strength
15 4	11'	11	Sand Clay	Moist = 115 pct Sat = 120 pct	c = 1400 psf Phi = 0	c = 250 psf Phi = 15	c = 0 psf Phi = 32
-2	17'	6	Fat Clay, GWT 13'	Moist = 112 pct Sat = 120 pct	c = 500 psf Phi = 10	c = 500 psf Phi = 10	c = 0 psf Phi = 27
-6	21'	4	Sand Clay	Moist = 115 pct Sat = 120 pct	c = 1200 psf Phi = 0	c = 250 psf Phi = 15	c = 0 psf Phi = 32
-26	41'	20	Clayey Sand & Gravel	Moist = 128 pct Sat = 136 pct	c = 0 psf Phi = 39	c = 0 psf Phi = 39	c = 0 psf Phi = 39
-38	53'	12	Fat Clay	See Above	See Above	See Above	See Above
-50	65'	12	Clayey Sand & Gravel	See Above	See Above	See Above	See Above
-56	71'	6	Lean Clay	Moist = 115 pct Sat = 120 pct	c = 1200 psf Phi = 0	c = 250 psf Phi = 15	c = 0 psf Phi = 32

3. Station 4+75 to 9+30

Elevation	Depth	Thickness	Soil Type	Unit Wt	Undrained Shear	Drained Shear R Strength	Drained Shear S Strength
16 -4	20'	20	Sand Clay, GWT 14'	Moist = 121 pct Sat = 124 pct	c = 1400 psf Phi = 0	c = 220 psf Phi = 18	c = 0 psf Phi = 32
-34	50'	30	Clayey Sand & Gravel	Moist = 128 pct Sat = 136 pct	c = 0 psf Phi = 38	c = 0 psf Phi = 38	c = 0 psf Phi = 38
-46	62'	12	Sand Clay	Moist = 115 pct Sat = 120 pct	c = 1200 psf Phi = 0	c = 250 psf Phi = 15	c = 0 psf Phi = 32
-59	75'	13	Clayey Sand & Gravel	See Above	See Above	See Above	See Above
-64	80'	5	Lean Clay	Moist = 115 pct Sat = 120 pct	c = 1200 psf Phi = 0	c = 500 psf Phi = 15	c = 0 psf Phi = 28

4. Station 9+30 to U/S End of Wall

Elevation	Depth	Thickness	Soil Type	Unit Wt	Undrained Shear	Drained Shear R Strength	Drained Shear S Strength
17 -5	22'	22	Sand Clay, GWT 20'	Moist = 119 pct Sat = 123 pct	c = 800 psf Phi = 0	c = 100 psf Phi = 15	c = 0 psf Phi = 30
-13	8'	8	Clayey Sand & Gravel	Moist = 128 pct Sat = 136 pct	c = 0 psf Phi = 35	c = 0 psf Phi = 35	c = 0 psf Phi = 35

-32	19'	19	Fat Clay	Moist = 121 pct Sat = 125 pct	c = 600 psf Phi = 0	c = 500 psf Phi = 10	c = 0 psf Phi = 27
-49	17'	17	Lean Clay	Moist = 122 pct Sat = 125 pct	c = 1200 psf Phi = 0	c = 500 psf Phi = 10	c = 0 psf Phi = 28
-58	9'	9	Clayey Sand & Gravel	See Above	c = 0 psf Phi = 38	c = 0 psf Phi = 38	c = 0 psf Phi = 38
-63	5'	5	Fat Clay	See Above	See Above	See Above	See Above

Design and Analysis Description

Wall #1 includes Types A, B, C, and D. For each type wall in Wall #1 (except Type D which is the extension existing walls), a critical section is selected with a station identified to perform the analyses and design. The results are then applied to that type of walls.

The design and analyses for Upper Walls #2 through #6 and Ramp Walls are performed based on different design height. Results are summarized in the table. For Vertans Park (VP) walls 1 to 3, a critical section is selected for analysis and design and results are then applied to whole length of that wall. For VP wall No. 4, a minimum reinforcement is provided due to small wall height.

Pile capacities are calculated using Xsection software. Pile deflections and structure-soil interaction were analyzed by LPile software. Flood scouring is ignored.

p-y curves were generated in Lpile program by following soil types:

Soil Profiles	Lpile Soil Types	Lpile Input Data (from Lpile "user's manual"-table 3.2 thru table 3.4				
		Eff. Unit Wt. pci	undrained cohesion, c psi	p-y modulus,k pci	soil strain e50	friction angle (degree)
1. Station 0+00 to 2+200						
Dense Sand & Gravel	Sand(Reese)	0.03		125		38
Stiff, Slightly O.C. Clay	Stiff Clay	0.04	8.33	500	0.005	
2. Station 2+00 to 4+75						
Sand Clay	Stiff Clay	0.03	9.72	500	0.005	
Fat Clay	Soft Clay	0.03	3.47		0.02	
Clayey Sand & Gravel	Sand(Reese)	0.04		60		39
Lean Clay	Stiff Clay	0.03	8.33	500	0.005	
3. Station 4+75 to 9+30						
Sand Clay	Stiff Clay	0.04	9.72	500	0.005	
Clayey Sand & Gravel	Sand(Reese)	0.04		60		38
Sand Clay	Stiff Clay	0.03	8.33	500	0.005	
Lean Clay	Stiff Clay	0.03	8.33	500	0.005	
4. Station 9+30 to End of Wall						
Sand Clay	Soft Clay	0.04	5.56		0.02	
Clayey Sand & Gravel	Sand(Reese)	0.04		60		35
Fat Clay	Soft Clay	0.04	4.17		0.02	
Lean Clay	Stiff Clay	0.04	8.33	500	0.005	

For details of p-y curves, please see "Lpile analyses-Lpile output files" from the page 190

MGE ENGINEERING, INC.

Project Napa River Flood Ccontrol Project
Subject Flood Wall Design Summary
By David An
Date Mar-05

Summary of Flood Wall (Wall #1)

Wall Station		Type	Pile Station		No. of Piles	Pile Main Rebar	Spiral Required	Section Computed
Begin	0+00.00	A	Begin	0+04.00	32 x 3-24" CIDH	12#6	#4@6"	1+88.00
End	2+56.00		End	2+52.00				
Begin	2+56.00	B	Begin	2+61.00	18 x 1-36" CIDH	18#11	#5@5"	2+61.00 3+15.00
End	4+67.79		End	4+59.00				
Begin	4+67.79	C	Begin	4+71.00	47 x 1-24" CIDH	14#10	#5@5"	4+83.00
End	10+26.92		End	10+23.00				
Begin	10+26.92	D	Existing Wall					
End	11+11.70							
Begin	11+16.92	C	Begin	11+25.00	44 x 1-24" CIDH	14#10	#5@5"	4+83.00
End	16+40.12		End	14+01.00				

Upper Walls (#2 to #6) and Ramp Walls Dimensions and Reinforcing Steel

Max. Design H (ft)	Upper Walls (#2 to #6)					Ramp Walls		
	6.00	8.00	10.00	12.00	14.00	3.50	5.50	7.50
W (ft)	5.25	7.50	9.50	11.00	12.50	4.00	5.50	7.50
W1(ft)	2.00	2.75	3.50	4.75	5.00	1.50	2.25	3.25
W2 (ft)	2.00	3.00	4.00	3.75	4.50	1.50	2.25	3.25
W3 (ft)	1.25	1.75	2.00	2.50	3.00	1.00	1.00	1.00
B (ft)	1.30	1.43	1.57	1.70	1.83	1.00	1.00	1.00
Bk (ft)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
F (ft)	1.50	1.50	1.50	1.50	1.50	1.25	1.25	1.25
a bars	#5@12"	#6@12"	#6@9"	#7@9"	#8@9"	#4@12"	#5@12"	#7@12"
b bars	#4@12"	#4@12"	#5@12"	#7@12"	#7@10"	#4@12"	#4@12"	#5@12"
c bars	#4@12"	#5@12"	#5@12"	#6@12"	#6@10"	#4@12"	#4@12"	#5@12"
Max toe pressure (ksf)	1.67	1.80	1.84	1.73	1.99	1.34	1.41	1.32

Pile Summary (From Lpile Runs Under Max. Loads)

Flood Wall (Wall#1)Type	Pile Type	Force at Top of Pile			Max Bending Moment (k-ft)	Bending Cap. (k-ft)	D/C	Notes
		V (kips)	M (k-ft)	P (kips)				
A	Ftg-Pilesb 3 rows 24"CIDH	42	0	88	138	310	0.44	
B (9ft spacing)	Single row 36" CIDH	87	631	50	942	2,045	0.46	
C (From 4+75 to 9+30)	Single row 24" CIDH	52	265	27	432	810	0.53	
C (From 9+30 to End)	Single row 24" CIDH	52	265	27	562	810	0.69	

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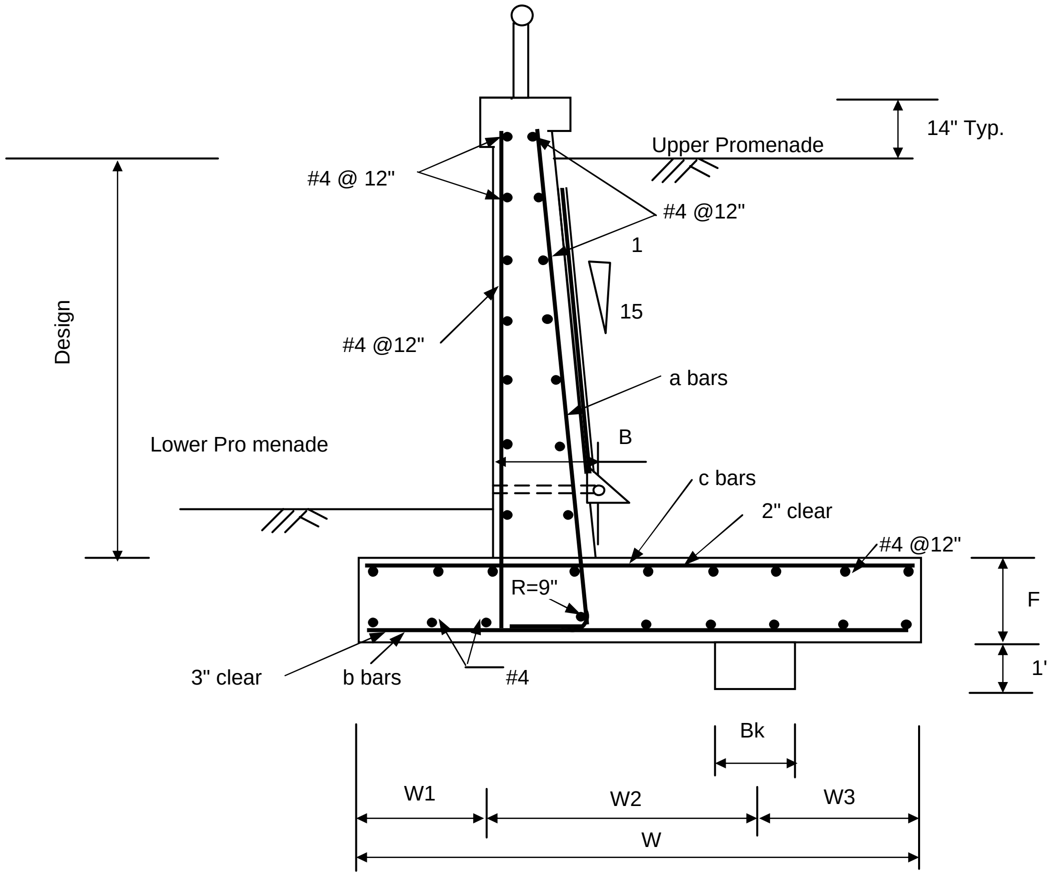
Project	Napa River Flood Ccontrol Project		
Subject	Flood Wall Design Summary		
By	David An	Date	Mar-05

Upper Wall and Ramp Wall Typical Section



WALL #1 DESIGN

WALL #1, TYPE A

Backfill Properties

Backfill Thickness = (17.00') - (-7.00') = 24.00 ft
Backfill Unit Weight = 125 pcf
Φ = 37 degree
C = 0 pcf
SMF= $\text{Tan}(\Phi_d)$ / $\text{Tan}\Phi$ = 2/3 = 0.67
Φ_d = 27 degree
Ka = Tan^2 (45° - Φ/2) = 0.25
Ko = Tan^2 (45° - Φ_d/2) = 0.38
Kp = Tan^2 (45° + Φ/2) = 4.02

Water Property

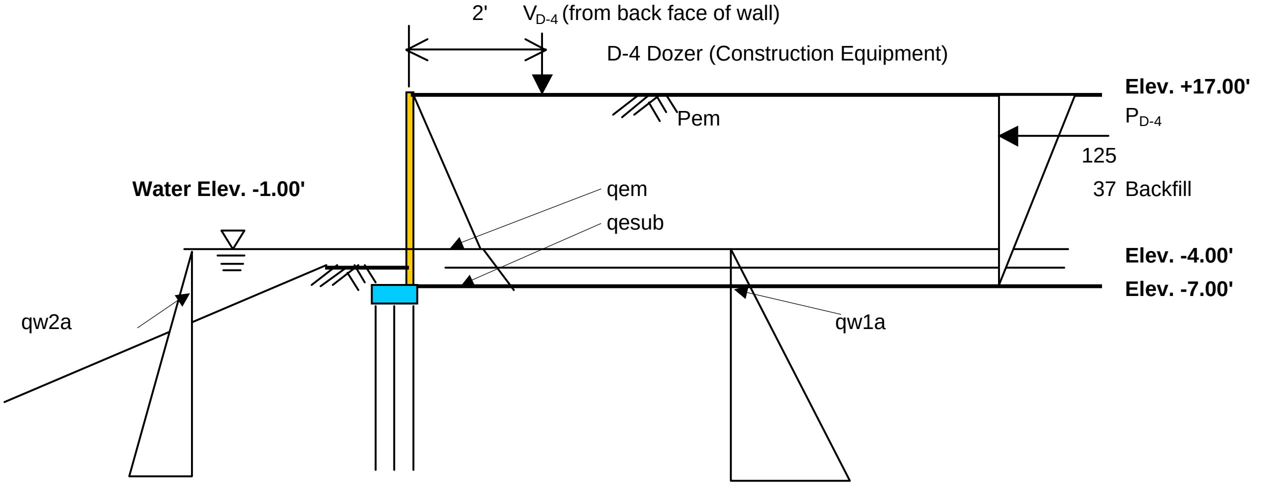
Water Unit Weight = 62.5 pcf

Pile and Wall Data

Station = **1+88**
Finish Grade Elevation(behind) = 17.00 ft
Finish Grade Elevation(front) = -4.00 ft
Top of Footing Elevation = -7.00 ft
Pile Spacing = 8.00 ft
Pile Diameter = 2.00 ft
100 Year Flood Level = 15.27 ft
Water Elevation (Mean higher) = 3.76 ft
Water Elevation (Mean lower) = -2.84 ft

MGE ENGINEERING, INC.		
Project	Napa River Flood Control Project	
Subject	Flood Wall Design (Wall #1, Type A)	
By	David An	Date Mar-05

Load Case 1 -- Short Term (Undrained) In Service Condition (Station 1+88)



Backfill Soil Pressure at Wall (Soil pressure = γ Ki hi)

Note: Passive soil resistance were ignored.

Name	Thickness(ft)	Pressure(ksf)
qem	18.00	0.856
qesub	6.00	0.999
qw1a=qw2a	6.00	0.375

qem - Moist soil pressure at rest wall
qesub - Submerged soil pressure at rest wall
qw - Water pressure

Backfill Resultant Forces Summary

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem	61.6	12.00	740
Pesub	44.5	2.92	130
Pw1a	9.0	2.00	18
Pd-4	0.0		0
Pw2a	-9.0	2.00	-18
At bot of wall	**106.1**	Safety Factor	**869.8**
	Σ V	**1.1**	Σ M

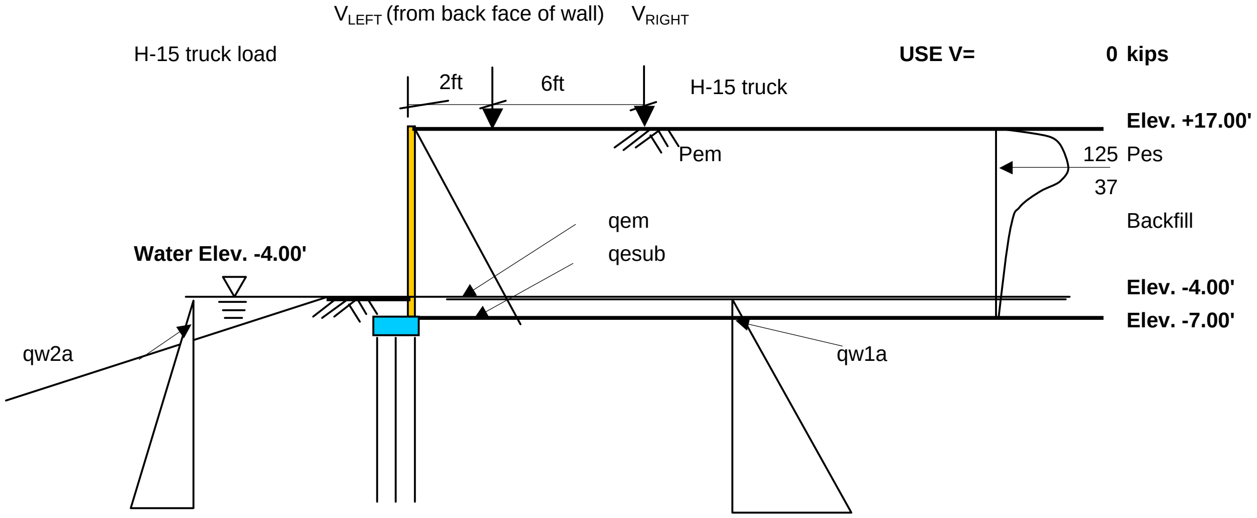
D-4 Dozer Loading Summary

b	Z	ΔP_{D-4}	Moment
0.0	0.00	0.000	0.000
0.1	2.40	0.000	0.000
0.2	4.80	0.000	0.000
0.3	7.20	0.000	0.000
0.4	9.60	0.000	0.000
0.5	12.00	0.000	0.000
0.6	14.40	0.000	0.000
0.7	16.80	0.000	0.000
0.8	19.20	0.000	0.000
0.9	21.60	0.000	0.000
1.0	24.00	0.000	0.000
Σ		0.000	0.000

h = 24.00 ft
a = 2' / 24.00' = 0.08 ≤ 0.4
V_{D-4} = 0 kips/ft
$\Delta P_{D-4} = (V_{D-4} /h)\ [(0.203b)/(0.16+b^2)^2]$
(EM 1110-2-2502 Page 3-49)

Demand at Top of Pile: Vd = 117 kips Md = 957 k-ft

Load Case 2 -- Long Term (Drained) In Service Condition (Station 1+88)



Backfill Soil Pressure at Wall (Soil pressure =γ Ki hi)

Note: Passive soil resistance were ignored.

Name	Thickness(ft)	Pressure(ksf)
qem	21.00	0.999
qesub	3.00	1.070
qw1a=qw2a	3.00	0.188

qem - Moist soil pressure at rest wall

qesub - Submerged soil pressure at rest wall

qw - Water pressure

Backfill Resultant Forces Summary

Name	Force	Arm to bot.	Moments
Pem	83.9	10.00	839
Pesub	24.8	1.48	37
Pw1a	2.3	1.00	2
Ph-15	0.0		0
Pw2a	-2.3	1.00	-2
At bot of wall	**108.7**	Safety Factor	**875.8**
	Σ V	**1.3**	Σ M

H-15 Truck Loading Summary (Left)

b (for V_{LEFT})	Z	$\Delta P_{PH\ (LEFT)}$	Moment
0.1	2.40	0.000	0.000
0.2	4.80	0.000	0.000
0.3	7.20	0.000	0.000
0.4	9.60	0.000	0.000
0.5	12.00	0.000	0.000
0.6	14.40	0.000	0.000
0.7	16.80	0.000	0.000
0.8	19.20	0.000	0.000
0.9	21.60	0.000	0.000
1.0	24.00	0.000	0.000
Σ		0.000	0.000

H-15 Truck Loading Summary (Right)

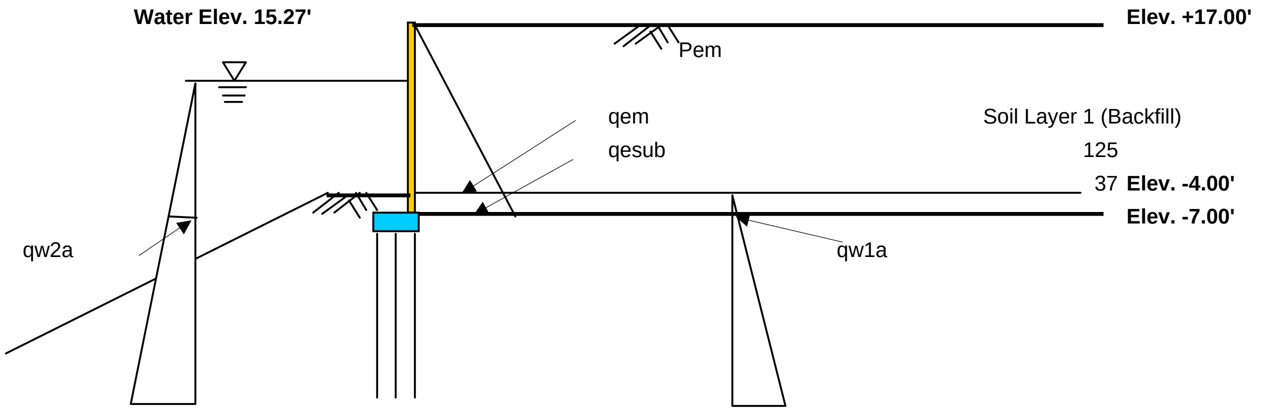
b (for V_{RIGHT})	Z	$\Delta P_{PH\ (RIGHT)}$	Moment
0.1	2.40	0.000	0.000
0.2	4.80	0.000	0.000
0.3	7.20	0.000	0.000
0.4	9.60	0.000	0.000
0.5	12.00	0.000	0.000
0.6	14.40	0.000	0.000
0.7	16.80	0.000	0.000
0.8	19.20	0.000	0.000
0.9	21.60	0.000	0.000
1.0	24.00	0.000	0.000
Σ		0.000	0.000

h = 24.00 ft

a=2'/24.00' = 0.08 ≤ 0.4 **For V_{LEFT}**

Demand at Top of Pile: Vd = 141 kips Md = 1,139 k-ft

Load Case 3 -- Long Term (Drained) In Service Condition With Flood (Station 1+88)



Backfill Soil Pressure at Wall (Soil pressure = γ Ki hi)

Name	Thickness(ft)	Pressure(ksf)
qem	21.00	0.999
qesub	3.00	1.070
qw1a	3.00	0.188
qw2a	22.27	1.392

Note: Passive soil resistance were ignored.

qem - Moist soil pressure at rest wall
qesub - Submerged soil pressure at rest wall
qw - Water pressure

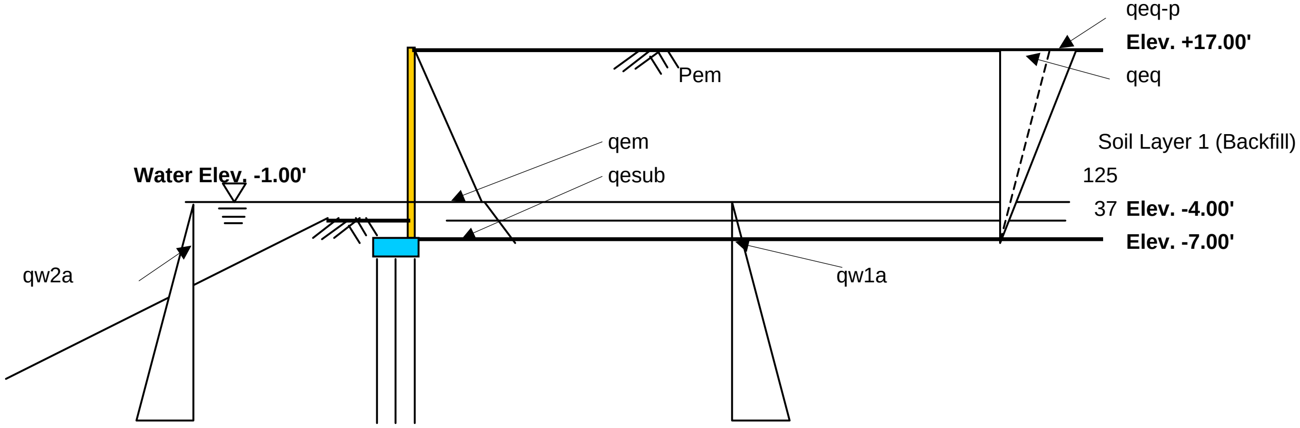
Backfill Resultant Forces Summary

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem	83.9	10.00	839
Pesub	24.8	1.48	37
Pw1a	2.3	1.00	2
Pw2a	-123.9	7.42	-920
At bot of wall	**-13.0**	Safety Factor	**-41.8**
	Σ V	**1.1**	Σ M

Demand at Top of Pile: Vd = -14 kips Md = -46 k-ft

MGE ENGINEERING, INC.	Project	Napa River Flood Control Project	
	Subject	Flood Wall Design (Wall #1, Type A)	
	By	David An	Date Mar-05

Load Case 4 -- Long Term (Drained) In Service Condition With Earthquqke (Station 1+88)



Backfill Soil Pressure at Wall (Soil pressure = γ Ki hi)

Name	Thickness(ft)	Pressure(ksf)
qem	18.00	0.856
qesub	3.00	0.927
qw1a	3.00	0.188
qw2a	3.00	0.188
qeq	24.00	0.344

Note: Passive soil resistance were ignored.

qem - Moist soil pressure at rest wall
qesub - Submerged soil pressure at rest wall
qw - Water pressure

qeq - Seismic components

$\alpha = \tan^{-1}[(C_1+\sqrt{(C_1^2+4C_2)})/2]$

(Equation 3-56 of EM 1110-2-2502, Page 3-67)

$C_1 = 2\ (\tan\Phi - K_h) / (1+K_h \tan\Phi)$

(Equation 3-57 of EM 1110-2-2502, Page 3-67)

$C_2 = [\tan\Phi(1-\tan\Phi\ \tan\beta)-(\tan\beta-K_h)] / [\tan\Phi(1+K_h \tan\Phi)]$

(Equation 3-58 of EM 1110-2-2502, Page 3-67)

Dynamic Components qeq $= \gamma\ K_h h^2 / [2(\tan\alpha - \tan\beta)]$

(Equation 3-62 of EM 1110-2-2502, Page 3-68)

K_h =	0.15	g
β =	0	
Φ =	30	degree
C_1 =	0.787	
C_2 =	0.681	
α =	52.6	degree

Backfill Resultant Forces Summary

Name	Force	Arm to bot.	Moments
Pem	61.6	9.00	555
Pesub	21.4	1.48	32
Pw1a	2.3	1.00	2
Peq	33.0	16.00	529
Pw2a	-2.3	1.00	-2
At bot of wall	**116.1**	Safety Factor	**1115.3**
	Σ V	**1.1**	Σ M

Demand at Top of Pile: Vd = 128 kips Md = 1,227 k-ft

Design Wall Section and Pile Footings (Station 1+88)

1. Loads

	Load Case	1	2	3	4
Forces w/o Safety Factor	Shear (k)	106.1	108.7	-13.0	0.3
	Moments(kft)	869.8	875.8	-41.8	9.8
	Safety Factor	1.1	1.3	1.1	125
Forces w/ Safety Factor	Shear (k)	116.8	141.3	-14.3	37.0
	Moments(kft)	956.8	1138.6	-46.0	1226.9

Demand at top of Footing:

Vd= **141** kips

Md= **1227** kft

Pd = [L x (Bt + Bb) / 2 x h) x 0.15 = **52** kips

Where

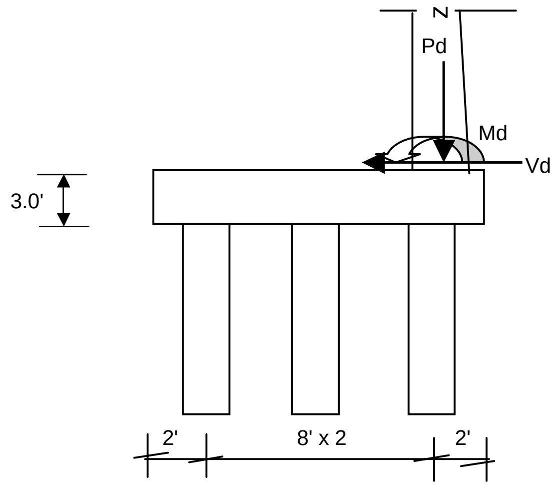
Wall thickness @ Top Bt = 1.00 ft

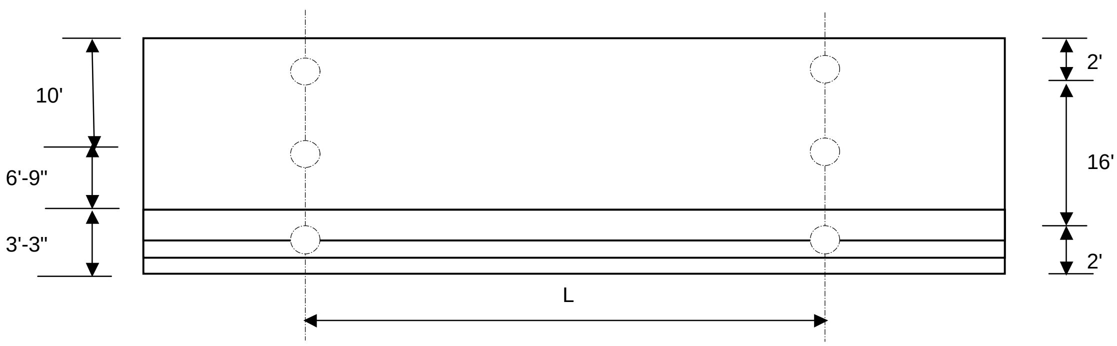
Wall thickness @ Bottom (1:15 batter) Bb = 2.60 ft

Front of Wall to Center of Footing (6'-9") = 6.75 ft

Wall Height h = 24.00 ft

Pile Spacing L = 8.00 ft





Plan View

2. Reinforced Concrete Wall Design (for Unit Width of 1')

Flexure reinforcement requirement (bending about horizontal axis at bottom of wall)

Mu ≤ ΦMn

Mu = Hf x 1.7 x Md / L = (1.30) (1.7) (153 k-ft/ft) = 339 k-ft/ft

Hf = Hydraulic factor 1.30

EM-1110-2-2104---P3-2, Equation3.3

Md / L = (1227 k-ft) / (8.00 ft) 153 k-ft/ft

Φ = 0.90

d = 2.60'x12 - 2.5" - 1.13"/2 = 28.1 in

b = 12 in

f'c = 4 ksi

fy = 60 ksi

Reinforcement Requirement

Mu ≤ ΦMn = Φ As x fy (d - a/2), where a = As x fy / (0.85 x f'c x b)

Therefore,

Mu ≤ ΦMn = Φ As x fy [d - As x fy / (2 x 0.85 x f'c x b)]

= Φ As x fy x d - Φ As^2 x fy^2 / (2 x 0.85 x f'c x b)

= (Φ fy x d) As - [Φ fy^2 / (2 x 0.85 x f'c x b)] As^2

[Φ fy^2 / (2 x 0.85 x f'c x b)] As^2 - (Φ fy x d) As + Mu = 0

{(0.90)(60 ksi)^2 / [2x0.85x(4 ksi)(12")]} As^2 - (0.90)(60 ksi)[(28.1") As + (339 k-ft/ft)(12 in/ft) = 0

39.71 As^2 -1519.43 As +4067.08=0

A = 39.71

B = -1519.43

C = 4067.08

Solve for As,

Required reinforcement As = 2.90 in^2

Try 2#8 bundle bars, 2 bundles, As = 0.79 in^2 x 2 x 2 = 3.16 in^2

Note that in each 2#8 bundle bars, only one extend to the top of wall.

a =Asfy / (0.85f'c*b) 4.65 in

ΦMn = Φ Asfy(d - a/2) 367 kft/ft

D/C = Mu / ΦMn = 339 / 367 = 0.92 **OK**

Use 2#8 bundle bars @6" Spacing

Check Shear

Vu = Hf x 1.7 x (Vd / L) = 39.0 kips/ft

ΦVn = Φ(Vc+Vs) 60.2 kips

where

Vc = 2 x √f'c x b x d 42.7 kips

Vs = As fy d / s 28.1 kips

use #4 @12" as shear reinforcement @ bottom of wall stem

Where Φ = 0.85

D/C = Vu / ΦVn 0.65 **OK**

Pile reinforcement development length, ld

ld = max{ ACI.R12..2, ACI.R12.2.3} 47.4 in

ACI R12.2.2 ld = [fy x αβλ/(20√f'c)]db = 47.4 in

ACI R12.2.3 ld = {3/40 x fy/√f'c x αβγλ/[(c+Ktr)/db]} = 28.5 in

where α = reinforcement location factor 1.0

β = coating factor 1.0

γ = reinforcement size factor 1.0

λ = lightweight aggregate concrete factor 1.0

c = cover 3.00 in

(clear cover c > db)

Ktr = Atr x fyt / (1500sn) = 0.592

where Atr = 0.79 in^2

fyt = fy 60.0 ksi

s = rebar spacing 5.3 in

(clear s > 2db)

n = number of bars 10.0

db = nominal diameter of bar 1.00 in

(c+Ktr)/db = 3.592

use (c+Ktr)/db = 2.5

3. Footing & Piles

Loads at Bottom of Footing

Mdmax=	Md - Pd x c - Σ(Wsoil x Arms) + Vd*D/2	392	k-ft
Vdmax =	Vd	109	kips
Pdmax=	Pd +Wfooting + Wsoil	191	kips

Where

Md = (Maximum of All Load Cases without Embedment Safety Factor	876	k-ft
Vd = (Same as Md)	109	kips
Pd =	51.9	kips
c = From Center of Wall to Center of Footing	8.05	ft
D = Depth of Footing	3.00	ft

Wfooting = 16 x D x L x 0.15	57.6	kips
Wsoil-1 (resisting side, RSP, Rec.) = 3' x (20'/2+6.75') x L x 0.12	48	kips
Moment Arm for Wsoil-1 = 20'/2 - (20'/2+6.75')/2	1.6	ft
Wsoil-2 (driving, Rec.) = (20'/2-6.75'-2.60') x (24.00') x L x 0.12	15	kips
Moment Arm for Wsoil-3 = (20'/2-6.75-2.60')/2 - 20'/2	-9.7	ft
Wsoil-3 (driving, Tri.) = (2.60'-1.00') x (24.00') x L / 2 x 0.12	18	kips
Moment Arm for Wsoil-3 = - (2.60-1.00') * 2 / 3 - 1.00' - 6.75'	-8.8	ft

Pile Force

I_{PILES} = 8^2*2 = (3 Rows 8ft x 2) 128 ft^2

Pile reaction

Tension		Rt=Pdmax/2-Mdmax*di / Ipiles =	**39.2**	kips
Compression		Rc=Pdmax/2+Mdmax*di / Ipiles =	**88.2**	kips
Laterial Force	Vpile = (Vd-Rsp)/2	(2 pile take lateral force)	**41.9**	kips

where

Laterial resistance of ftgs	@ Top of ftg	q_{pt} = Kp x γ' x h1	0.69	ksf
	Where	γ' = 120-62.5	57.50	pcf
		h1 =	3.00	ft
	@ Top of ftg	q_{pb} = Kp x γ' x h1	1.39	ksf
	Where	γ' = 120-62.5	57.50	pcf
		h2 =	6.00	ft
	R_{sp} = (q_{pt}+q_{pb})/2 x 2.5 x L		24.98	kips

Allowllable Pile Loading (without load test)

Compression =	105.00	**kips, OK**
Tension =	69.00	**kips, OK**

MGE ENGINEERING, INC.		
	Project	Napa River Flood Control Project
	Subject	Flood Wall Design (Wall #1, Type A)
	By	David An
	Date	Mar-05

Force at the Face of Wall

Mu_{MIN}= Hf x 1.7 x [Rt*3/12+(Pdf*2.25/10*2.25/2+Pds-inside*2.25/2)] = (include soil and footing			99.7	k-ft
Vu_{MIN} = Hf x 1.7 x[Rt+(Pdf*2.25/10+pds-inside)] = (include soil and footing			156.0	kips
Mu_{MAX}= Hf x 1.7 x [Rc*5.75-(0.12*3+0.15*2.5)*5.75^2/2] = (include soil and footing			1094.3	k-ft
Vu_{MAX} = Hf x 1.7 x [Rc - (0.12*3+0.15*2.5)*5.75] = (include soil and footing			185.6	kips
where	U = Hf x 1.7 x (D+L)			
		Hf = Hydraulic factor	1.30	
		EM-1110-2-2104---P3-2, Equation3.3		

Required flexure reinforcement at footing (@8ft space)

for Mumax	As = Mumax/[Φfy(d-a/2)]		10.95	in^2
	where Φ =		0.90	
	fy =		60	ksi
	d =2.5*12-6		24	in
	a = 0.15d	(assumed)	4	in
	f'c =		4	ksi
	b = L	Width of footing	8.00	ft
	Use #9@8", As = 1.0 x 8 x 12 / 8		12.00	in^2
Check	a=As*fy/(0.85*fc'*b)=		2.2	in
	ΦMn = Φ Asfy(d - a/2)		1236.4	kft/ft
		D/C = Mumax/ΦMn	0.89	**OK**
Shear Check				
		ΦVn = Φ (Vc + Vs)		kips
		ΦVc = Φ x 2 x √f'c x b x d	291.4	kips
		where Φ =	0.85	
		D/C = Vumax / ΦVc	0.64	**OK**
for Mumin	As = Mumin/[Φfy(d-a/2)]		0.89	in^2
	where Φ=		0.90	
	fy =		60	ksi
	d =2.5*12-3		27	in
	a = 0.15d	(assumed)	4	in
	f'c =		4	ksi
	b = L	Width of footing	8.00	ft
	Use #6@12", As = 0.44 x 8		3.52	in^2
Check	a=As*fy/(0.85*fc'*b)=		0.6	in
	ΦMn = Φ Asfy(d - a/2)		422.6	kft/ft
		D/C = Mumin/ΦMn	0.24	**OK**
Shear Check				
		D/C = Vumin / ΦVc	0.54	**OK**

4. Piles

Use 24" CIDH Piles at spacing 8' (3 rows)

Required rebars for compression piles

from BDS equation 8-31

		Ast = [(Rc/ Φ-0.85f'cAg)/(fy-0.85f'c)]/0.8	-31.4	in^2
	where	Φ =	0.75	
		Ag = $\pi D^2/4$	452.4	in^2
		(0.8--for zero eccentricity)		
	rebar no needed			**OK**

Required rebars for tension piles

As = Rt / (Φ x fy)/0.8 0.91 in^2
where Φ= 0.90
(0.8--for zero eccentricity)
Try 12#6, As = 0.44 x 12 5.28 in^2
Check Rtn = 0.8 x Φ x As x fy 228 kips
D/C = Rt / Rtn 0.17 **OK**
Rebar Ratio = (5.28 in^2) / (452.39 in^2) = 1.17%
Use 12#6 for Pile Longitudinal Reinforcement

Required Pile Shear Reinforcement

Shear demand at pile section, Vdp = Vd / 2 70.7 kips
Shear capacity of concrete
Φ Vc = Φ 2 x √f'c x Ae 41.3 kips
Φ = 0.85
Ae = 0.85Ag (assumed) 384.5 in^2
Required shear capacity of steel
Φ Vs ≥ Vsd ΦVn = Φ(Vc+Vs) = Φ x 2 x √f'c x A
where
Vsd = Vdp - Φ Vc 29.3 kips
Vs = π/2 Av fy d / s
Try #4 Spiral, Av = 0.2 in^2
d = 24-3 21 in
s ≤ Φ π/2 Av fy d / Vsd 11.5 in
Use #4 Spiral with Spacing s=6" **OK**

Pile reinforcement development length, ld

ld = max{ ACI.R12..2, ACI.R12.2.3} 35.6 in

ACI R12.2.2 ld = [fy x αβλ/(20√f'c)]db = 35.6 in
ACI R12.2.3 ld = {3/40 x fy/√f'c x αβγλ/[(c+Ktr)/db]} = 28.5 in

where α = reinforcement location factor 1.0
β = coating factor 1.0
γ = reinforcement size factor 1.0
λ = lightweight aggregate concrete factor 1.0
c = cover 2.69 in
(clear cover c > db)
Ktr = Atr x fyt / (1500sn) = 0.592
where Atr = 0.79 in^2
fyt = fy 60.0 ksi
s = rebar spacing 5.3 in
(clear s > 2db)
n = number of bars 10.0
db = nominal diameter of bar 0.75 in
(c+Ktr)/db = 4.376
use (c+Ktr)/db = 2.5

WALL #1, TYPE B (Pile Spacing 9')

Backfill Properties

Backfill Thickness = (17.00') - (-2.31') = 19.31 ft
Backfill Unit Weight = 125 pcf
Φ = 37 degree
C = 0 pcf
SMF= $\text{Tan}(\Phi_d)$ / $\text{Tan}\Phi$ = 2/3 = 0.67
Φ_d = 27 degree
Ka = Tan^2 (45° - Φ/2) = 0.25
Ko = Tan^2 (45° - Φ_d/2) = 0.38
Kp = Tan^2 (45° + Φ/2) = 4.02

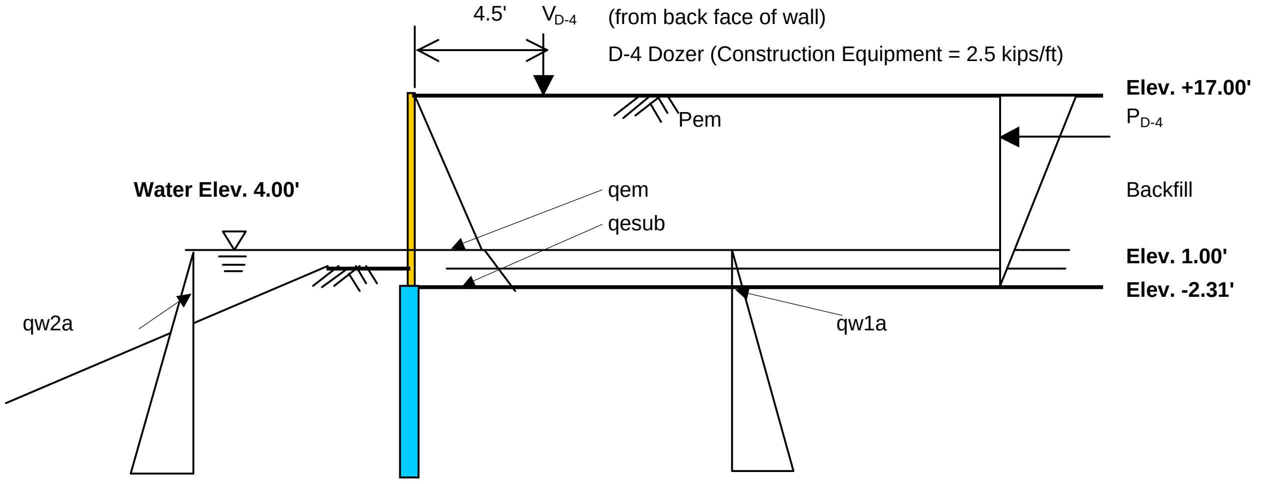
Water Property

Water Unit Weight = 62.5 pcf

Pile and Wall Data

Station = **2+61**
Finish Grade Elevation(behind) = 17.00 ft
Finish Grade Elevation(front) = 1.00 ft
Top of CIDH Pile Elevation = -2.31 ft
Pile Spacing = 9.00 ft
Pile Diameter = 3.00 ft
100 Year Flood Level = 15.30 ft
Water Elevation (Mean higher) = 3.76 ft
Water Elevation (Mean lower) = -2.84 ft

Load Case 1 -- Short Term (Undrained) In Service Condition (Station 2+61)



Backfill Soil Pressure at Wall (Soil pressure = γ Ki hi)

Note: Passive soil resistance were ignored.

Name	Thickness(ft)	Pressure(ksf)
qem	13.00	0.618
qesub	6.31	0.768
qw1a=qw2a	6.31	0.394

qem - Moist soil pressure at rest wall
qesub - Submerged soil pressure at rest wall
qw - Water pressure

Backfill Resultant Forces Summary

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem	36.2	10.64	385
Pesub	39.4	3.04	120
Pw1a	11.2	2.10	24
Pd-4	12.3		141
Pw2a	-11.2	2.10	-24
At bot of wall	**87.9**	Safety Factor	**646.3**
	Σ V	**1.1**	Σ M

D-4 Dozer Loading Summary

b	Z	ΔP_{D-4}	Moment
0.0	0.00	0.000	0.000
0.1	1.93	0.176	3.052
0.2	3.86	0.254	3.921
0.3	5.79	0.244	3.293
0.4	7.73	0.198	2.297
0.5	9.66	0.151	1.458
0.6	11.59	0.113	0.870
0.7	13.52	0.084	0.487
0.8	15.45	0.063	0.245
0.9	17.38	0.049	0.094
1.0	19.31	0.038	0.000
Σ		1.369	15.717

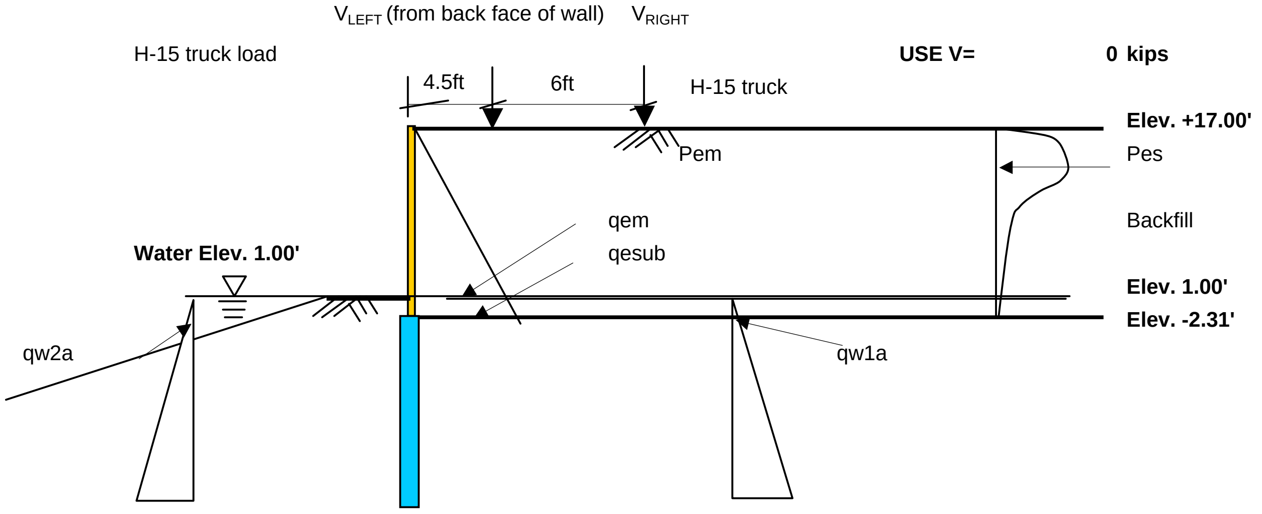
h = 19.31 ft
a = 4.5' / 19.31' = 0.23 ≤ 0.4
V_{D-4} = 2.5 kips/ft
$\Delta P_{D-4} = (V_{D-4} / h)$ [(0.203b)/(0.16+b^2)^2]
(EM 1110-2-2502 Page 3-49)

Demand at Top of Pile: Vd = 97 kips Md = 711 k-ft

MGE Engineering, Inc.

Project Napa River Flood Control Project
Subject Flood Wall Design (Wall #1, Type B)
By David An
Date Mar-05

Load Case 2 -- Long Term (Drained) In Service Condition (Station 2+61)



Backfill Soil Pressure at Wall (Soil pressure = γ Ki hi)

Note: Passive soil resistance were ignored.

Name	Thickness(ft)	Pressure(ksf)
qem	16.00	0.761
qesub	3.31	0.840
qw1a=qw2a	3.31	0.207

qem - Moist soil pressure at rest wall
qesub - Submerged soil pressure at rest wall
qw - Water pressure

Backfill Resultant Forces Summary

Name	Force	Arm to bot.	Moments
Pem	54.8	8.64	474
Pesub	23.8	1.63	39
Pw1a	3.1	1.10	3
Ph-15	0.0		0
Pw2a	-3.1	1.10	-3
At bot of wall	**78.6**	Safety Factor	**512.5**
	Σ V	**1.3**	Σ M

H-15 Truck Loading Summary (Left)

b (for V_{LEFT})	Z	$\Delta P_{PH\ (LEFT)}$	Moment
0.1	1.93	0.000	0.000
0.2	3.86	0.000	0.000
0.3	5.79	0.000	0.000
0.4	7.73	0.000	0.000
0.5	9.66	0.000	0.000
0.6	11.59	0.000	0.000
0.7	13.52	0.000	0.000
0.8	15.45	0.000	0.000
0.9	17.38	0.000	0.000
1.0	19.31	0.000	0.000
Σ		0.000	0.000

H-15 Truck Loading Summary (Right)

b (for V_{RIGHT})	Z	$\Delta P_{PH\ (RIGHT)}$	Moment
0.1	1.93	0.000	0.000
0.2	3.86	0.000	0.000
0.3	5.79	0.000	0.000
0.4	7.73	0.000	0.000
0.5	9.66	0.000	0.000
0.6	11.59	0.000	0.000
0.7	13.52	0.000	0.000
0.8	15.45	0.000	0.000
0.9	17.38	0.000	0.000
1.0	19.31	0.000	0.000
Σ		0.000	0.000

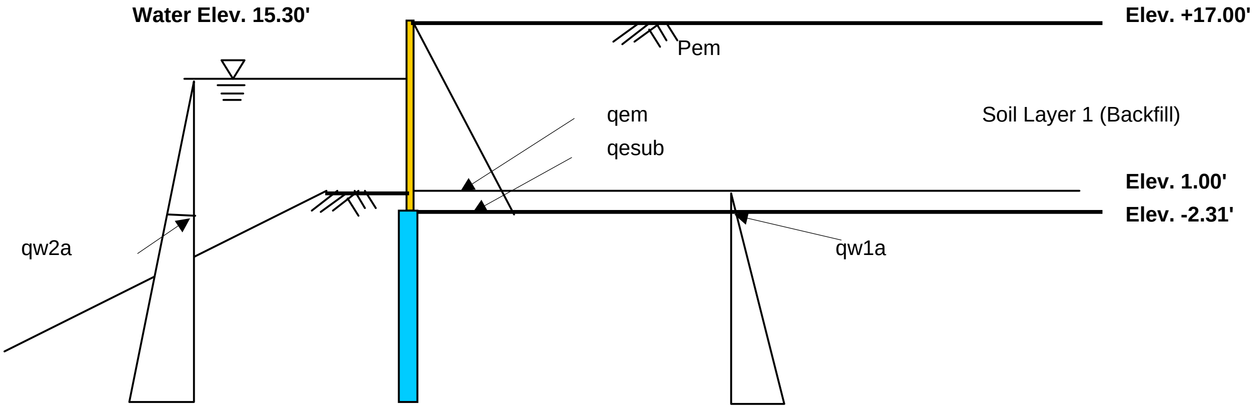
h = 19.31 ft

$\Delta P_{HZ} = (0.28V/h^2)\ [b^2 / (0.16+b^2)^3]$ (EM 1110-2-2502 Page 3-49)

a=4.5'/19.31' = 0.23 ≤ 0.4 **For V_{LEFT}**
a=10.5'/19.31' = 0.54 > 0.4 **For V_{RIGHT}**

Demand at Top of Pile: Vd = 102 kips Md = 666 k-ft

Load Case 3 -- Long Term (Drained) In Service Condition With Flood (Station 2+61)



Backfill Soil Pressure at Wall (Soil pressure = γ Ki hi)

Note: Passive soil resistance were ignored.

Name	Thickness(ft)	Pressure(ksf)
qem	16.00	0.761
qesub	3.31	0.840
qw1a	3.31	0.207
qw2a	17.61	1.101

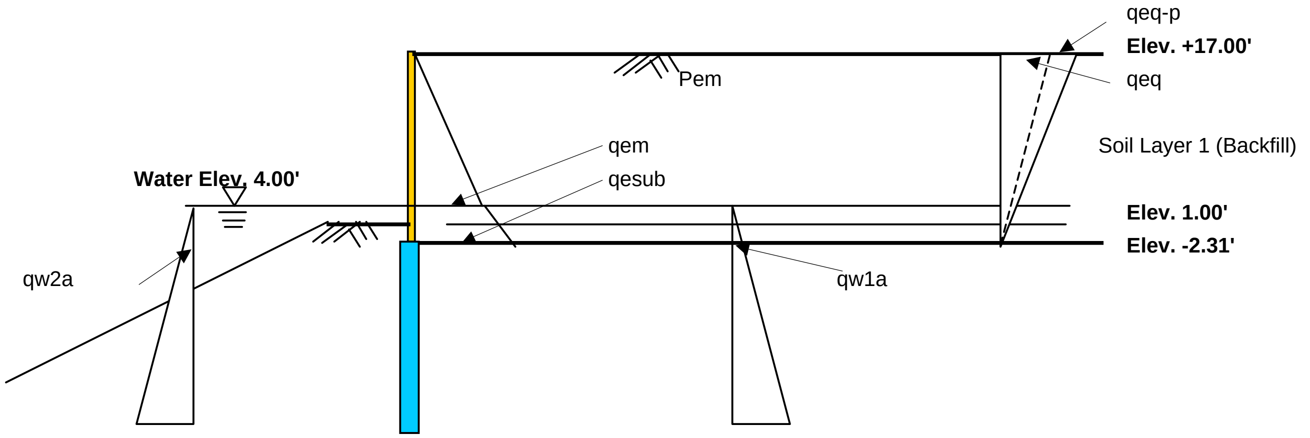
qem - Moist soil pressure at rest wall
qesub - Submerged soil pressure at rest wall
qw - Water pressure

Backfill Resultant Forces Summary

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem	54.8	8.64	474
Pesub	23.8	1.63	39
Pw1a	3.1	1.10	3
Pw2a	-87.3	5.87	-512
At bot of wall	**-5.6**	Safety Factor	**3.5**
	Σ V	**1.1**	Σ M

Demand at Top of Pile: Vd = -6 kips Md = 4 k-ft

Load Case 4 -- Long Term (Drained) In Service Condition With Earthquqke (Station 2+61)



Backfill Soil Pressure at Wall(Soil pressure =γ Ki hi)

Name	Thickness(ft)	Pressure(ksf)
qem	13.00	0.618
qesub	3.31	0.697
qw1a	3.31	0.207
qw2a	3.31	0.207
qeq	19.31	0.277

Note: Passive soil resistance were ignored.

qem - Moist soil pressure at rest wall
qesub - Submerged soil pressure at rest wall
qw - Water pressure

qeq - Seismic components

$\alpha = \tan^{-1}[(C_1+\sqrt{(C_1^2+4C_2)})/2]$

(Equation 3-56 of EM 1110-2-2502, Page 3-67)

$C_1 = 2\ (\tan\Phi - K_h)\ /\ (1+K_h \tan\Phi)$

(Equation 3-57 of EM 1110-2-2502, Page 3-67)

$C_2 = [\tan\Phi(1-\tan\Phi\ \ \mathrm{tar}\beta)-(\tan\beta-K_h)]\ /\ [\tan\Phi(1+K_h \tan\Phi)]$

(Equation 3-58 of EM 1110-2-2502, Page 3-67)

Dynamic Components qeq =$\gamma K_h h^2 / [2(\tan\alpha-\tan\beta)]$

(Equation 3-62 of EM 1110-2-2502, Page 3-68)

K_h =	0.15	g
β =	0	
Φ =	30	degree
C_1 =	0.787	
C_2 =	0.681	
α =	52.6	degree

Backfill Resultant Forces Summary

Name	Force	Arm to bot.	Moments
Pem	36.2	7.64	277
Pesub	19.6	1.62	32
Pw1a	3.1	1.10	3
Peq	24.1	12.88	310
Pw2a	-3.1	1.10	-3
At bot of wall	**79.8**	Safety Factor	**618.2**
	Σ V	**1.1**	Σ M

Demand at Top of Pile: Vd = 88 kips Md = 680 k-ft

MGE ENGINEERING, INC.

Project	Napa River Flood Control Project		
Subject	Flood Wall Design (Wall #1, Type B)		
By	David An	Date	Mar-05

Design Wall Section and Pile Reinforcement (Station 2+61)

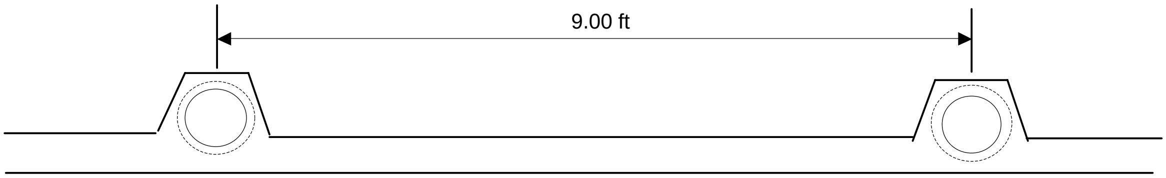
1. Loads

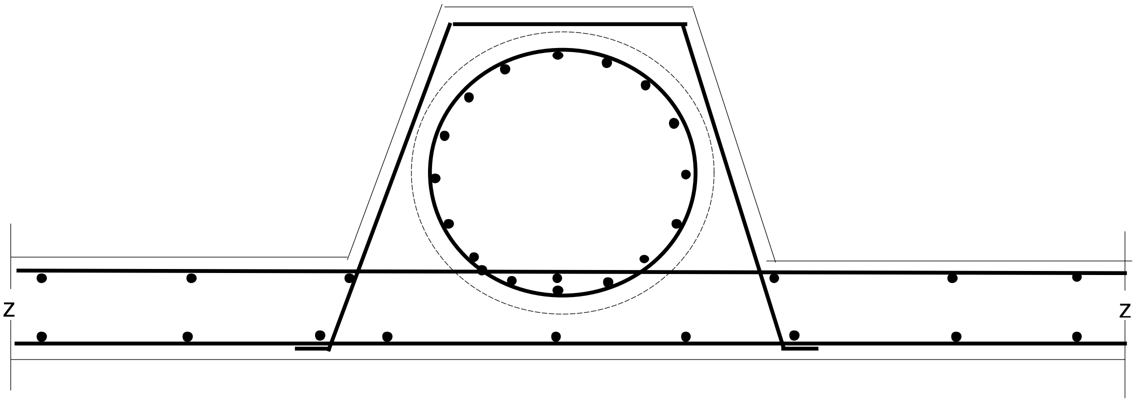
	Load Case	1	2	3	4
Forces w/o Safety Factor	Shear (k)	87.9	78.6	-5.6	79.8
	Moments(kft)	646.3	512.5	3.5	618.2
	Safety Factor	1.1	1.3	1.1	1.1
Forces w/ Safety Factor	Shear (k)	96.6	102.2	-6.1	87.8
	Moments(kft)	710.9	666.2	3.8	680.0

Demand at top of Pile:

Vd= **102** kips

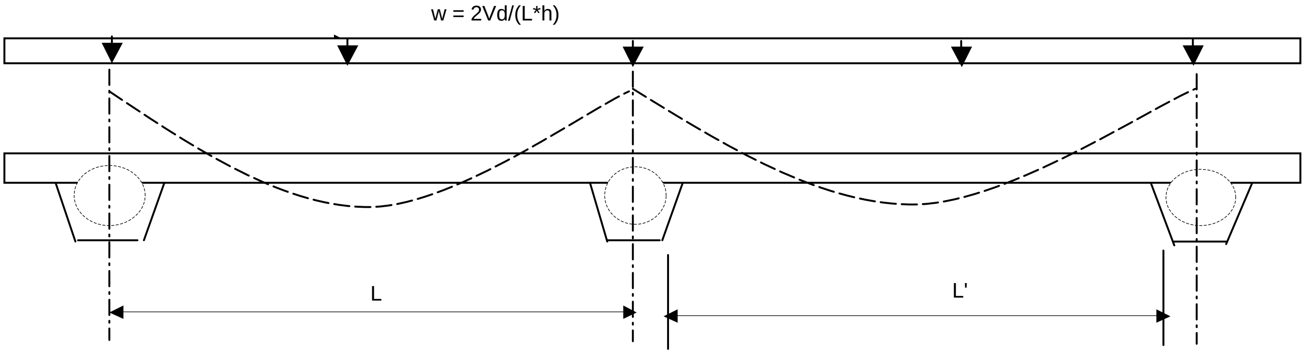
Md= **711** k-ft





2. Reinforced Concrete Wall (unit width)

Flexure reinforcement requirement (bending about vertical axis)



Plan View

	Mu ≤ ΦMn				
	Mu = Hf x 1.7 x 0.1 x w x l^2		(used 3 equal span continuous beam)	5.7	k-ft/ft
	where				
		w = Vd/(L'*h)		1.10	kip/ft
		L' = L-25/12x 2		4.8	ft
		h =		19.3	ft
		Hf =	Hydraulic factor	1.30	
		EM-1110-2-2104---P3-2, Equation3.3			
	ΦMn = Φ As x fy (d - a/2)				
	where				
		d = 12in -2.5in	(wall thick = 12 in)	9.5	in
		a = 0.15d		1.4	in
Required reinforcement					
		As = Mu/[Φfy(d-a/2)] (Required)		0.14	in^2
		where Φ=		0.90	
		fy =		60	ksi
	use 1#5, As = 0.31			0.31	in^2
Check	a =Asfy/(0.85f'c*b)			0.29	in
	where	f'c =		4	ksi
		b =	unit width of wall	12	in
	ΦMn = Φ Asfy(d - a/2)			13.0	kft/ft
			D/C = Mu / ΦMn	0.43	**OK**
Shear check					
		Vu = Hf x 1.7 x Vd / h /2 =		5.8	kips/ft
		ΦVc = Φ x 2 x √f'c x b x d		12.3	kips
		where Φ =		0.85	
			D/C = Vu / ΦVc	0.48	**OK**

3. Connections between wall and piles

Use 36" CIDH Piles at spacing of 9.00'

	Rebar Size (#):			11	
	Number of Rebar:			18	
	Spiral Spacing:			6	in
	Moment capacity Mn (Mp) =		(from Xsection)	2,071	k-ft
			Mu = Hf x 1.7 x Md	1,571	k-ft
			D/C = Mu/ΦMn = Hf x 1.7 x Md/ΦMn	0.84	**OK**
Use shear-friction design method					
		Vn = Avf x fy x μ	BDS p8-26	1011	kips
	where	Avf =		28.08	in^2
		fy =		60	ksi
		μ = 0.6 λ		0.6	
		λ =	normal concrete	1.0	
			D/C = Vu/ΦVn = Hf x 1.7 x Vd/ΦVn	0.26	**OK**

4. Piles

Shear capacity of pile

ΦVn = Φ(Vc+Vs) 273 kips
where Φ = 0.85
Vc = 2 x √f'c x Ae 129 kips
Vs = π/2 Av fy d / s 193 kips
Av Rebar Size (#) 5
Av = 0.31 in^2
d = 36" -3" 33 in
s = 5 in
Ae = 0.85Ag 1018 in^2
D/C = Vu/ΦVn = Hf x 1.7 x Vd/ΦVn 0.83 **OK**

Use #5@5" for Spirals.

Pile reinforcement development length, ld

ld = max{ ACI.R12..2, ACI.R12.2.3} 74.0 in

ACI R12.2.2 ld = [fy x αβλ/(20√f'c)]db = 74.0 in
ACI R12.2.3 ld = {3/40 x fy/√f'c x αβγλ/[(c+Ktr)/db]} = 36.5 in

where
α = reinforcement location factor 1.0
β = coating factor 1.0
γ = reinforcement size factor 1.0
λ = lightweight aggregate concrete factor 1.0
c = cover 2.69 in
(clear cover c > db)
Ktr = Atr x fyt / (1500sn) = 0.354
where Atr = 0.79 in^2
fyt = fy 60.0 ksi
s = rebar spacing 5.0 in
(clear s > 2db)
n = number of bars 18.0
db = nominal diameter of bar 1.56 in
(c+Ktr)/db = 1.951

Bar #	Area (SI)
3	0.11
4	0.20
5	0.31
6	0.44
7	0.60
8	0.79
9	1.00
10	1.27
11	1.56
14	2.25
18	4.00

WALL #1, TYPE B (Typical, Pile Spacing 12')

Backfill Properties

Backfill Thickness = (17.00') - (1.00') =	16.00	ft
Backfill Unit Weight =	125	pcf
Φ =	37	degree
C =	0	pcf
SMF= Tan(Φ_d) / TanΦ = 2/3 =	0.67	
Φ_d =	27	degree
Ka = Tan^2 (45° - Φ/2) =	0.25	
Ko = Tan^2 (45° - Φ_d/2) =	0.38	
Kp = Tan^2 (45° + Φ/2) =	4.02	

Water Property

Water Unit Weight =	62.5	pcf

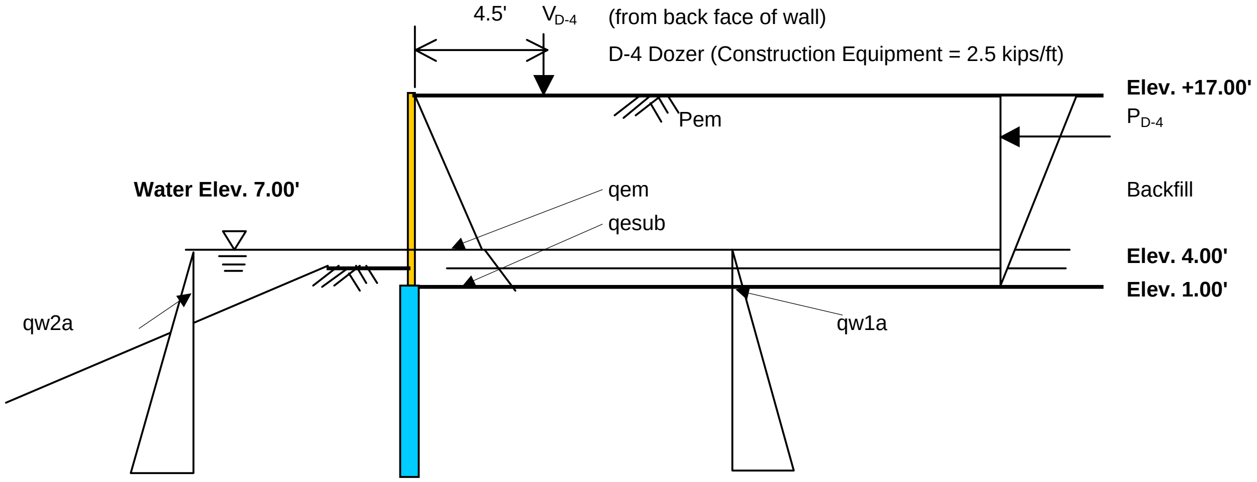
Pile and Wall Data

Station =	**3+15**	
Finish Grade Elevation(behind) =	17.00	ft
Finish Grade Elevation(front) =	4.00	ft
Top of CIDH Pile Elevation =	1.00	ft
Pile Spacing =	12.00	ft
Pile Diameter =	3.00	ft
100 Year Flood Level =	15.37	ft
Water Elevation (Mean higher) =	3.76	ft
Water Elevation (Mean lower) =	-2.84	ft

MGE ENGINEERING, INC.

Project	Napa River Flood Control Project		
Subject	Flood Wall Design (Wall #1, Type B)		
By	David An	Date	Mar-05

Load Case 1 -- Short Term (Undrained) In Service Condition (Station 3+15)



Backfill Soil Pressure at Wall (Soil pressure = γ Ki hi)

Note: Passive soil resistance were ignored.

Name	Thickness(ft)	Pressure(ksf)
qem	10.00	0.476
qesub	6.00	0.618
qw1a=qw2a	6.00	0.375

qem - Moist soil pressure at rest wall

qesub - Submerged soil pressure at rest wall

qw - Water pressure

Backfill Resultant Forces Summary

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem	28.5	9.33	266
Pesub	39.4	2.87	113
Pw1a	13.5	2.00	27
Pd-4	16.4		156
Pw2a	-13.5	2.00	-27
At bot of wall	**84.3**	Safety Factor	**535.7**
	Σ V	**1.1**	Σ M

D-4 Dozer Loading Summary

b	Z	ΔP_{D-4}	Moment
0.0	0.00	0.000	0.000
0.1	1.60	0.176	2.529
0.2	3.20	0.254	3.249
0.3	4.80	0.244	2.729
0.4	6.40	0.198	1.904
0.5	8.00	0.151	1.208
0.6	9.60	0.113	0.721
0.7	11.20	0.084	0.404
0.8	12.80	0.063	0.203
0.9	14.40	0.049	0.078
1.0	16.00	0.038	0.000
Σ		1.369	13.023

h = 16.00 ft

a = 4.5' / 16.00' = 0.28 ≤ 0.4

V_{D-4} = 2.5 kips/ft

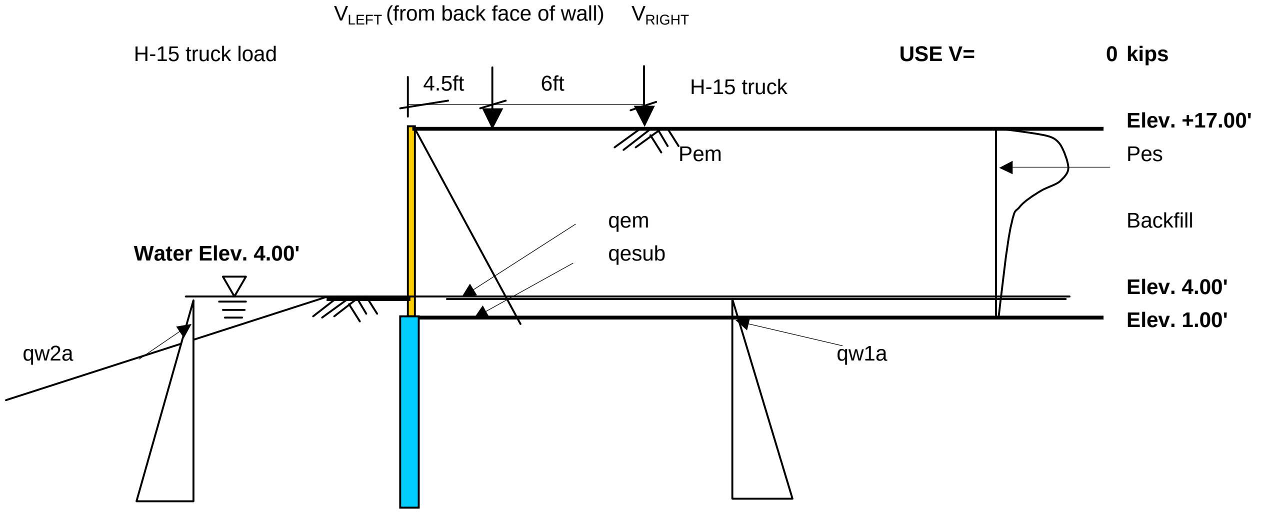
$\Delta P_{D-4} = (V_{D-4} / h)\ [(0.203b)/(0.16+b^2)^2]$

(EM 1110-2-2502 Page 3-49)

Demand at Top of Pile: Vd = 93 kips Md = 589 k-ft

Project	Napa River Flood Control Project
Subject	Flood Wall Design (Wall #1, Type B)
By	David An
Date	Mar-05

Load Case 2 -- Long Term (Drained) In Service Condition (Station 3+15)



Backfill Soil Pressure at Wall (Soil pressure = γ Ki hi)

Note: Passive soil resistance were ignored.

Name	Thickness(ft)	Pressure(ksf)
qem	13.00	0.618
qesub	3.00	0.690
qw1a=qw2a	3.00	0.188

qem - Moist soil pressure at rest wall

qesub - Submerged soil pressure at rest wall

qw - Water pressure

Backfill Resultant Forces Summary

Name	Force	Arm to bot.	Moments
Pem	48.2	7.33	354
Pesub	23.5	1.47	35
Pw1a	3.4	1.00	3
Ph-15	0.0		0
Pw2a	-3.4	1.00	-3
At bot of wall	**71.8**	Safety Factor	**388.4**
	Σ V	**1.3**	Σ M

H-15 Truck Loading Summary (Left)

b (for V_{LEFT})	Z	$\Delta P_{PH\ (LEFT)}$	Moment
0.1	1.60	0.000	0.000
0.2	3.20	0.000	0.000
0.3	4.80	0.000	0.000
0.4	6.40	0.000	0.000
0.5	8.00	0.000	0.000
0.6	9.60	0.000	0.000
0.7	11.20	0.000	0.000
0.8	12.80	0.000	0.000
0.9	14.40	0.000	0.000
1.0	16.00	0.000	0.000
Σ		0.000	0.000

H-15 Truck Loading Summary (Right)

b (for V_{RIGHT})	Z	$\Delta P_{PH\ (RIGHT)}$	Moment
0.1	1.60	0.000	0.000
0.2	3.20	0.000	0.000
0.3	4.80	0.000	0.000
0.4	6.40	0.000	0.000
0.5	8.00	0.000	0.000
0.6	9.60	0.000	0.000
0.7	11.20	0.000	0.000
0.8	12.80	0.000	0.000
0.9	14.40	0.000	0.000
1.0	16.00	0.000	0.000
Σ		0.000	0.000

h = 16.00 ft

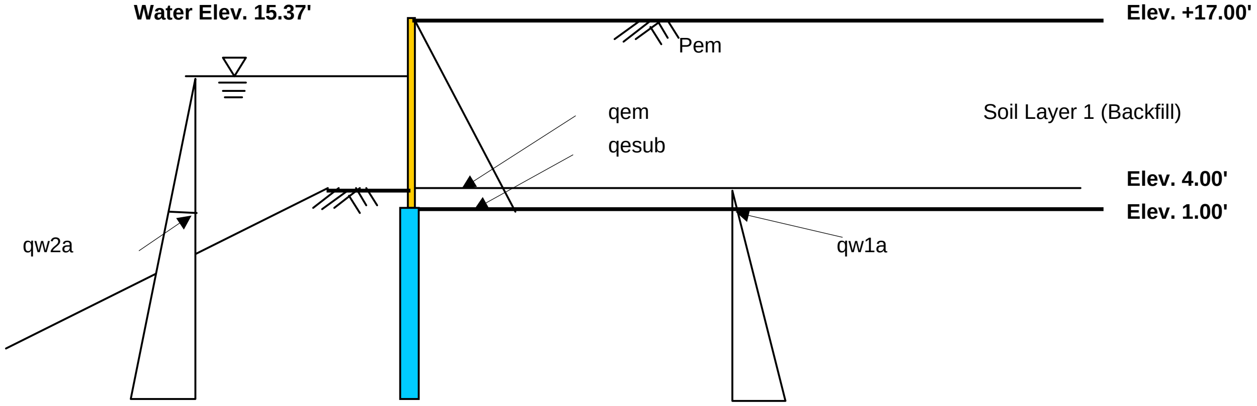
$\Delta P_{HZ} = (0.28V/h^2)\ [b^2 / (0.16+b^2)^3]$ (EM 1110-2-2502 Page 3-49)

a=4.5'/16.00' = 0.28 ≤ 0.4 **For V_{LEFT}**

a=10.5'/16.00' = 0.66 > 0.4 **For V_{RIGHT}**

Demand at Top of Pile: Vd = 93 kips Md = 505 k-ft

Load Case 3 -- Long Term (Drained) In Service Condition With Flood (Station 3+15)



Backfill Soil Pressure at Wall (Soil pressure = γ Ki hi)

Note: Passive soil resistance were ignored.

Name	Thickness(ft)	Pressure(ksf)
qem	13.00	0.618
qesub	3.00	0.690
qw1a	3.00	0.188
qw2a	14.37	0.898

qem - Moist soil pressure at rest wall

qesub - Submerged soil pressure at rest wall

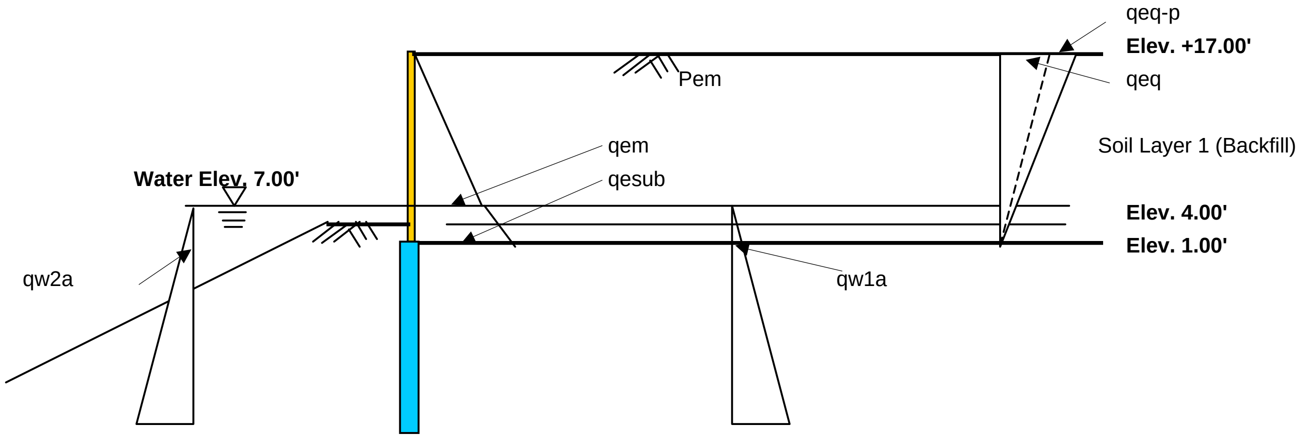
qw - Water pressure

Backfill Resultant Forces Summary

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem	48.2	7.33	354
Pesub	23.5	1.47	35
Pw1a	3.4	1.00	3
Pw2a	-77.4	4.79	-371
At bot of wall	**-2.3**	Safety Factor	**20.9**
	Σ V	**1.1**	Σ M

Demand at Top of Pile: Vd = -3 kips Md = 23 k-ft

Load Case 4 -- Long Term (Drained) In Service Condition With Earthquqke (Station 3+15)



Backfill Soil Pressure at Wall (Soil pressure = γ Ki hi)

Note: Passive soil resistance were ignored.

Name	Thickness(ft)	Pressure(ksf)
qem	10.00	0.476
qesub	3.00	0.547
qw1a	3.00	0.188
qw2a	3.00	0.188
qeq	16.00	0.229

qem - Moist soil pressure at rest wall
qesub - Submerged soil pressure at rest wall
qw - Water pressure

qeq - Seismic components

$\alpha = \tan^{-1}[(C_1+\sqrt{}(C_1^2+4C_2))/2]$

(Equation 3-56 of EM 1110-2-2502, Page 3-67)

$C_1 = 2\ (\tan\Phi - K_h)\ /\ (1+K_h \tan\Phi)$

(Equation 3-57 of EM 1110-2-2502, Page 3-67)

$C_2 = [\tan\Phi(1-\tan\Phi\ \tan\beta)-(\tan\beta-K_h)]\ /\ [\tan\Phi(1+K_h \tan\Phi)]$

(Equation 3-58 of EM 1110-2-2502, Page 3-67)

Dynamic Components $qeq = \gamma\ K_h h^2\ /\ [2(\tan\alpha-\tan\beta)]$

(Equation 3-62 of EM 1110-2-2502, Page 3-68)

$K_h =$ 0.15 g
$\beta =$ 0
$\Phi =$ 30 degree
$C_1 =$ 0.787
$C_2 =$ 0.681
$\alpha =$ 52.6 degree

Backfill Resultant Forces Summary

Name	Force	Arm to bot.	Moments
Pem	28.5	6.33	181
Pesub	18.4	1.47	27
Pw1a	3.4	1.00	3
Peq	22.0	10.67	235
Pw2a	-3.4	1.00	-3
At bot of wall	**69.0**	Safety Factor	**442.9**
	Σ V	**1.1**	Σ M

Demand at Top of Pile: Vd = 76 kips Md = 487 k-ft

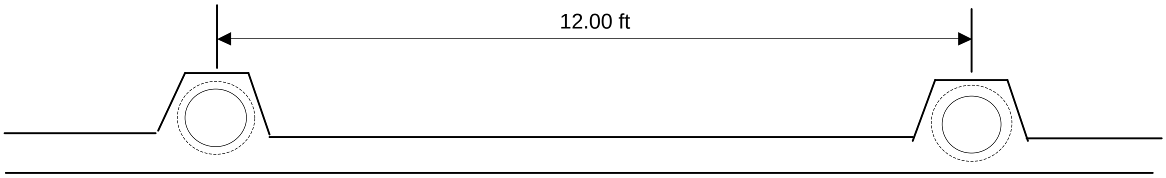
Design Wall Section and Pile Reinforcement (Station 3+15)

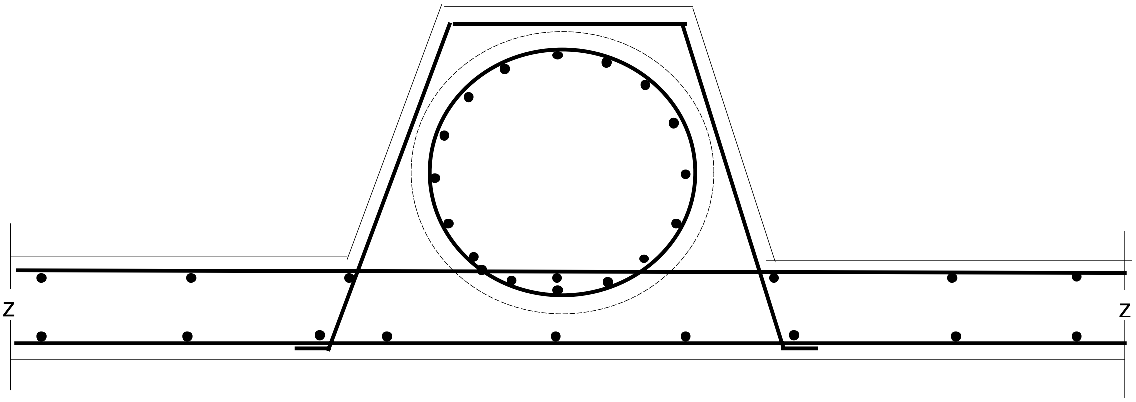
1. Loads

	Load Case	1	2	3	4
Forces w/o Safety Factor	Shear (k)	84.3	71.8	-2.3	69.0
	Moments(kft)	535.7	388.4	20.9	442.9
	Safety Factor	1.1	1.3	1.1	1.1
Forces w/ Safety Factor	Shear (k)	92.8	93.3	-2.5	75.9
	Moments(kft)	589.3	505.0	23.0	487.2

Demand at top of Pile: Vd= **93** kips

Md= **589** k-ft

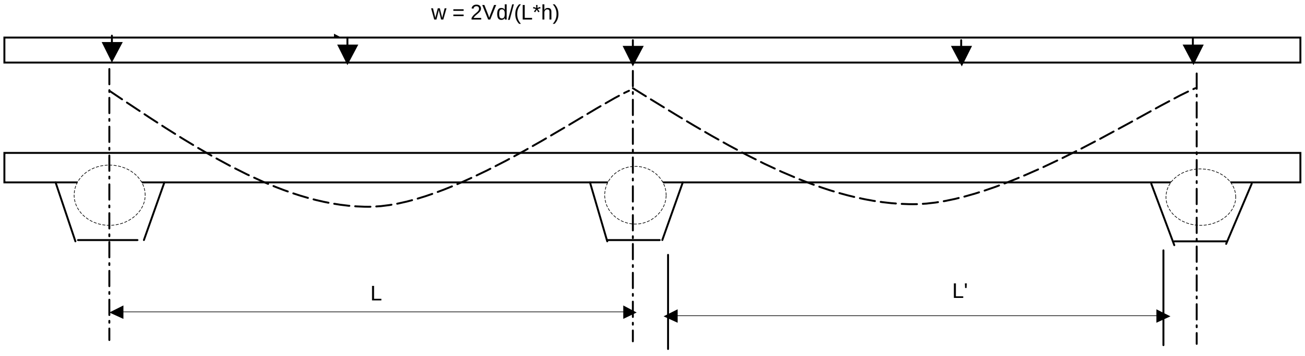




2. Reinforced Concrete Wall (unit width)

2a. As continuous beam

Flexure reinforcement requirement (bending about vertical axis)



Plan View

Mu ≤ ΦMn

Mu = Hf x 1.7 x 0.1 x w x l^2 (used 3 equal span continuous beam) 10.1 k-ft/ft

where

w = Vd/(L'*h) 0.74 kip/ft

L' = L-25/12x 2 7.8 ft

h = 16.0 ft

Hf = Hydraulic factor 1.30

EM-1110-2-2104---P3-2, Equation3.3

ΦMn = Φ As x fy (d - a/2)

where

d = 12in -2.5in (wall thick = 12 in) 9.5 in

a = 0.15d 1.4 in

Required reinforcement

As = Mu/[Φfy(d-a/2)] (Required) 0.26 in^2

where Φ= 0.90

fy = 60 ksi

use 1#5, As = 0.31 0.31 in^2

Check a =Asfy/(0.85f'c*b) 0.46 in

where f'c = 4 ksi

b = unit width of wall 12 in

ΦMn = Φ Asfy(d - a/2) 12.9 kft/ft

D/C = Mu / ΦMn 0.78 **OK**

Shear check

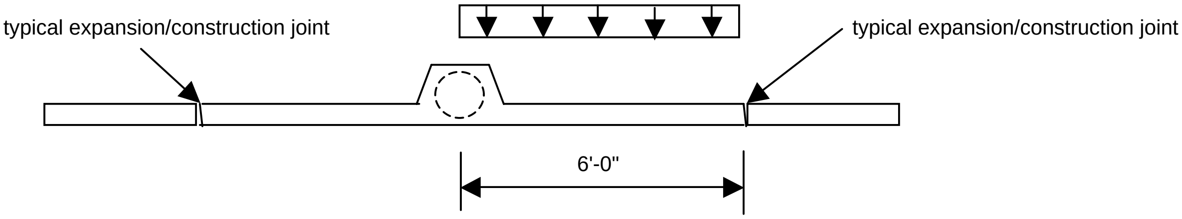
Vu = Hf x 1.7 x Vd / h /2 = 6.4 kips/ft

ΦVc = Φ x 2 x √f'c x b x d 12.3 kips

where Φ = 0.85

D/C = Vu / ΦVc 0.53 **OK**

2b. As cantilever beam



Mu = Hf x 1.7 x Mmax = 12.62 kft/ft

where Mmax = wl^2/2 5.71 k/ft

w = 0.74 k/ft/ft

l = 6 - 25/12 3.9 ft

Mn = Φ x As x fy (d-a/2) =

Φ = 0.9

d = 12in -2.5in (wall thick = 12 in) 9.5 in

assume a = 0.15d 1.4 in

Required reinforcement

As = Mu/[Φfy(d-a/2)] 0.32 in^2

fy = 60 ksi

use 1#6, As = 0.44 in^2 0.44 in^2

Check a =Asfy/(0.85f'c*b) 0.65 in
where f'c = 4 ksi
b = unit width of wall 12 in
ΦMn = Φ Asfy(d - a/2) 18.2 kft/ft
D/C = Mu / ΦMn 0.77 **OK**

Shear check

Vu = Hf x 1.7 x w x l = 6.4 kips/ft

ΦVc = Φ x 2 x √f'c x b x d 12.3 kips
where Φ = 0.85
D/C = Vu / ΦVc 0.53 **OK**

Consider 2a. And 2b., Use #6@12" for both faces.

Deflection check

Δmax =wl^4/(8EI) 0.01 in
where E = 57√f'c = 3605.00 ksi
I = 12 x 12^3 / 12 = 1728.00 in^4
OK

3. Connections between wall and piles

Use 36" CIDH Piles at spacing of 12.00'

Rebar Size (#): 11
Number of Rebar: 18
Spiral Spacing: 5 in
Moment capacity Mn (Mp) = (from Xsection) 2,071 k-ft
Mu = Hf x 1.7 x Md 1,302 k-ft
D/C = Mu/ΦMn = Hf x 1.7 x Md/ΦMn 0.70 **OK**

Use shear-friction design method

Vn = Avf x fy x μ BDS p8-26 1011 kips
where Avf = 28.08 in^2
fy = 60 ksi
μ = 0.6 λ 0.6
λ = normal concrete 1.0
D/C = Vu/ΦVn = Hf x 1.7 x Vd/ΦVn 0.24 **OK**

4. Piles

Shear capacity of pile

ΦVn = Φ(Vc+Vs) 246 kips
where Φ = 0.85
Vc = 2 x √f'c x Ae 129 kips
Vs = π/2 Av fy d / s 161 kips
Av Rebar Size (#) 5
Av = 0.31 in^2
d = 36" -3" 33 in
s = 6 in
Ae = 0.85Ag 1018 in^2
D/C = Vu/ΦVn = Hf x 1.7 x Vd/ΦVn 0.84 **OK**

Use #5@6" for Spirals.

Pile reinforcement development length, ld

ld = max{ ACI.R12..2, ACI.R12.2.3} 74.0 in

ACI R12.2.2 ld = [fy x αβλ/(20√f'c)]db = 74.0 in

ACI R12.2.3 ld = {3/40 x fy/√f'c x αβγλ/[(c+Ktr)/db]} = 36.5 in

where α = reinforcement location factor 1.0

β = coating factor 1.0

γ = reinforcement size factor 1.0

λ = lightweight aggregate concrete factor 1.0

c = cover 2.69 in

(clear cover c > db)

Ktr = Atr x fyt / (1500sn) = 0.354

where Atr = 0.79 in^2

fyt = fy 60.0 ksi

s = rebar spacing 5.0 in

(clear s > 2db)

n = number of bars 18

db = nominal diameter of bar 1.56 in

(c+Ktr)/db = 1.951

Bar #	Area (SI)
3	0.11
4	0.20
5	0.31
6	0.44
7	0.60
8	0.79
9	1.00
10	1.27
11	1.56
14	2.25
18	4.00

WALL #1, TYPE C

Backfill Properties

Backfill Thickness = (12.88') - (1.00') = 11.88 ft

Backfill Unit Weight = 125 pcf

Φ = 37 degree

C = 0 pcf

SMF= $\tan(\Phi_d) / \tan\Phi$ = 2/3 = 0.67

Φ_d = 27 degree

Ka = $\tan^2 (45° - \Phi/2)$ = 0.25

Ko = $\tan^2 (45° - \Phi_d/2)$ = 0.38

Kp = $\tan^2 (45° + \Phi/2)$ = 4.02

Water Property

Water Unit Weight = 62.5 pcf

Pile and Wall Data

Station = **4+83**

Finish Grade Elevation (behind) = 12.88 ft

Finish Grade Elevation(front) = 4.00 ft

Top of CIDH Pile Elevation = 1.00 ft

Pile Spacing = 12.00 ft

Pile Diameter = 2.00 ft

100 Year Flood Level = 15.55 ft

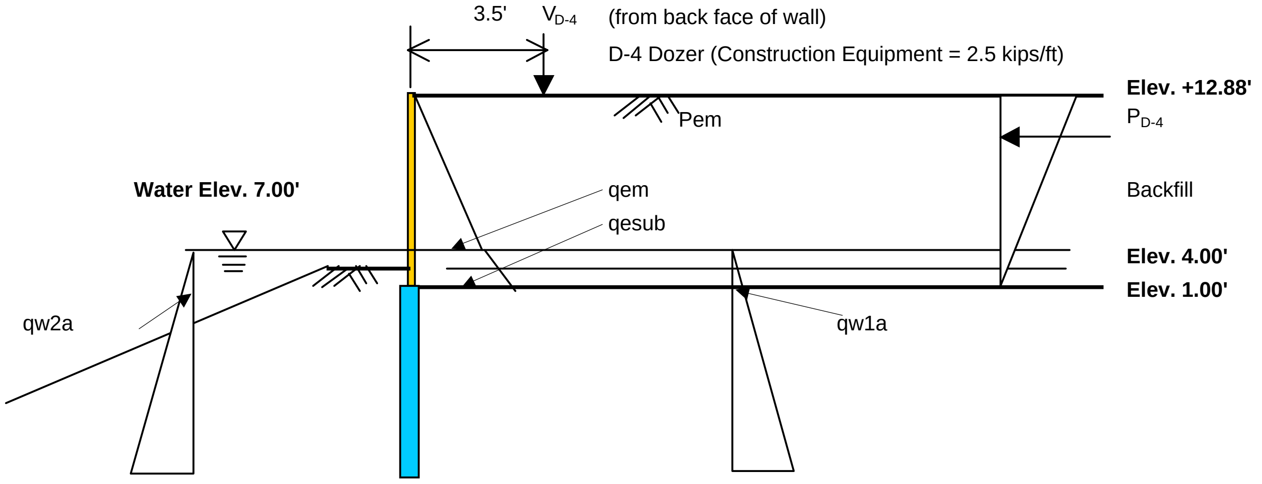
Water Elevation (Mean higher) = 3.76 ft

Water Elevation (Mean lower) = -2.84 ft

MGE Engineering, Inc.

Project: Napa River Flood Control Project
Subject: Flood Wall Design (Wall #1, Type C)
By: David An
Date: Mar-05

Load Case 1 -- Short Term (Undrained) In Service Condition (Station 4+83)



Backfill Soil Pressure at Wall (Soil pressure = γ Ki hi)

Note: Passive soil resistance were ignored.

Name	Thickness(ft)	Pressure(ksf)
qem	5.88	0.280
qesub	6.00	0.422
qw1a=qw2a	6.00	0.375

qem - Moist soil pressure at rest wall
qesub - Submerged soil pressure at rest wall
qw - Water pressure

Backfill Resultant Forces Summary

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem	9.9	7.96	79
Pesub	25.3	2.80	71
Pw1a	13.5	2.00	27
Pd-4	16.4		116
Pw2a	-13.5	2.00	-27
At bot of wall	**51.5**	Safety Factor	**265.2**
	Σ V	**1.1**	Σ M

D-4 Dozer Loading Summary

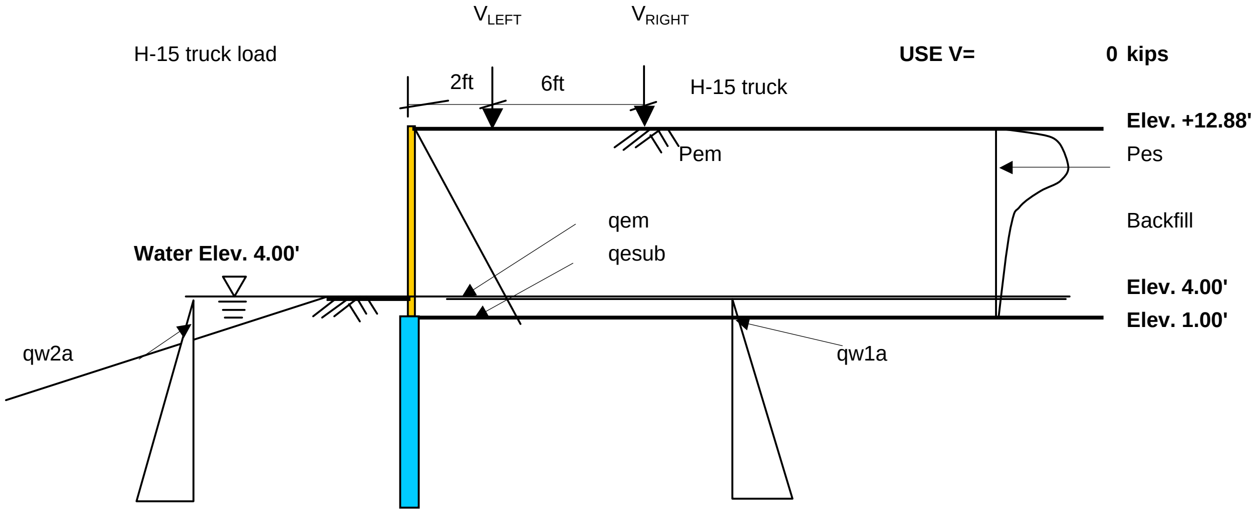
b	Z	ΔP_{D-4}	Moment
0.0	0.00	0.000	0.000
0.1	1.19	0.176	1.878
0.2	2.38	0.254	2.412
0.3	3.56	0.244	2.026
0.4	4.75	0.198	1.413
0.5	5.94	0.151	0.897
0.6	7.13	0.113	0.535
0.7	8.32	0.084	0.300
0.8	9.50	0.063	0.151
0.9	10.69	0.049	0.058
1.0	11.88	0.038	0.000
Σ		1.369	9.668

h = 11.88 ft
a = 3.5' / 11.88' = 0.29 ≤ 0.4
V_{D-4} = 2.5 kips/ft
$\Delta P_{D-4} = (V_{D-4} / h)\ [(0.203b)/(0.16+b\text{^}2)\text{^}2]$
(EM 1110-2-2502 Page 3-49)

Demand at Top of Pile: Vd = 57 kips Md = 292 k-ft

Load Case 2 -- Long Term (Drained) In Service Condition (Station 4+83)

USE V= 0 kips



Backfill Soil Pressure at Wall (Soil pressure = γ Ki hi)

Note: Passive soil resistance were ignored.

Name	Thickness(ft)	Pressure(ksf)
qem	8.88	0.422
qesub	3.00	0.494
qw1a=qw2a	3.00	0.188

qem - Moist soil pressure at rest wall
qesub - Submerged soil pressure at rest wall
qw - Water pressure

Backfill Resultant Forces Summary

Name	Force	Arm to bot.	Moments
Pem	22.5	5.96	134
Pesub	16.5	1.46	24
Pw1a	3.4	1.00	3
Ph-15	0.0		0
Pw2a	-3.4	1.00	-3
At bot of wall	**39.0**	Safety Factor	**158.1**
	Σ V	**1.3**	Σ M

H-15 Truck Loading Summary (Left)

b (for V_{LEFT})	Z	$\Delta P_{PH\ (LEFT)}$	Moment
0.1	1.19	0.000	0.000
0.2	2.38	0.000	0.000
0.3	3.56	0.000	0.000
0.4	4.75	0.000	0.000
0.5	5.94	0.000	0.000
0.6	7.13	0.000	0.000
0.7	8.32	0.000	0.000
0.8	9.50	0.000	0.000
0.9	10.69	0.000	0.000
1.0	11.88	0.000	0.000
Σ		0.000	0.000

H-15 Truck Loading Summary (Right)

b (for V_{RIGHT})	Z	$\Delta P_{PH\ (RIGHT)}$	Moment
0.1	1.19	0.000	0.000
0.2	2.38	0.000	0.000
0.3	3.56	0.000	0.000
0.4	4.75	0.000	0.000
0.5	5.94	0.000	0.000
0.6	7.13	0.000	0.000
0.7	8.32	0.000	0.000
0.8	9.50	0.000	0.000
0.9	10.69	0.000	0.000
1.0	11.88	0.000	0.000
Σ		0.000	0.000

h = 11.88 ft

$\Delta P_{HZ} = (0.28V/h^2)\ [b^2 / (0.16+b^2)^3]$ (EM 1110-2-2502 Page 3-49)

a=2'/11.88' = 0.17 ≤ 0.4 **For V_{LEFT}**

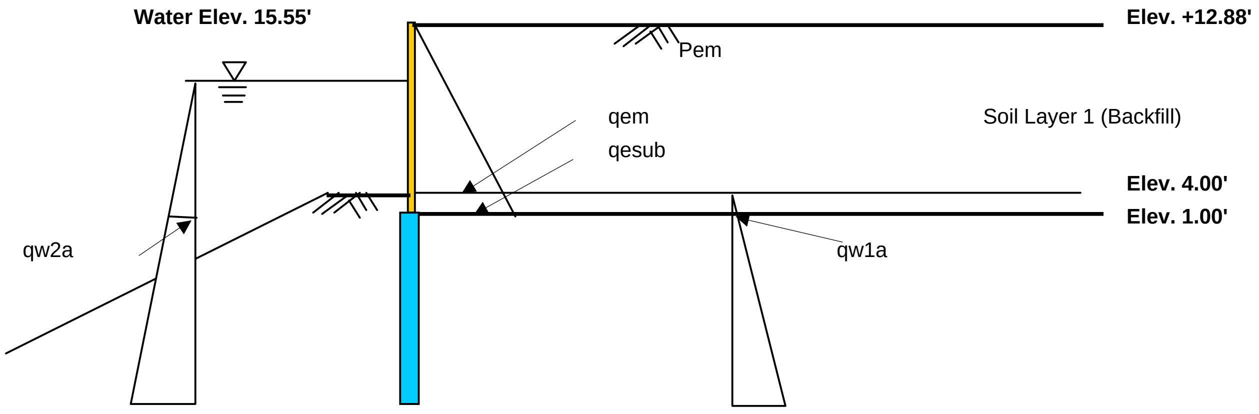
a=8'/11.88' = 0.67 > 0.4 **For V_{RIGHT}**

Demand at Top of Pile: Vd = 51 kips Md = 206 k-ft

MGE ENGINEERING, INC.

Project	Napa River Flood Control Project		
Subject	Flood Wall Design (Wall #1, Type C)		
By	David An	Date	Mar-05

Load Case 3 -- Long Term (Drained) In Service Condition With Flood (Station 4+83)



Backfill Soil Pressure at Wall(Soil pressure =γ Ki hi)

Note: Passive soil resistance were ignored.

Name	Thickness(ft)	Pressure(ksf)
qem	8.88	0.422
qesub	3.00	0.494
qw1a	3.00	0.188
qw2a	14.55	0.909

qem - Moist soil pressure at rest wall
qesub - Submerged soil pressure at rest wall
qw - Water pressure

Backfill Resultant Forces Summary

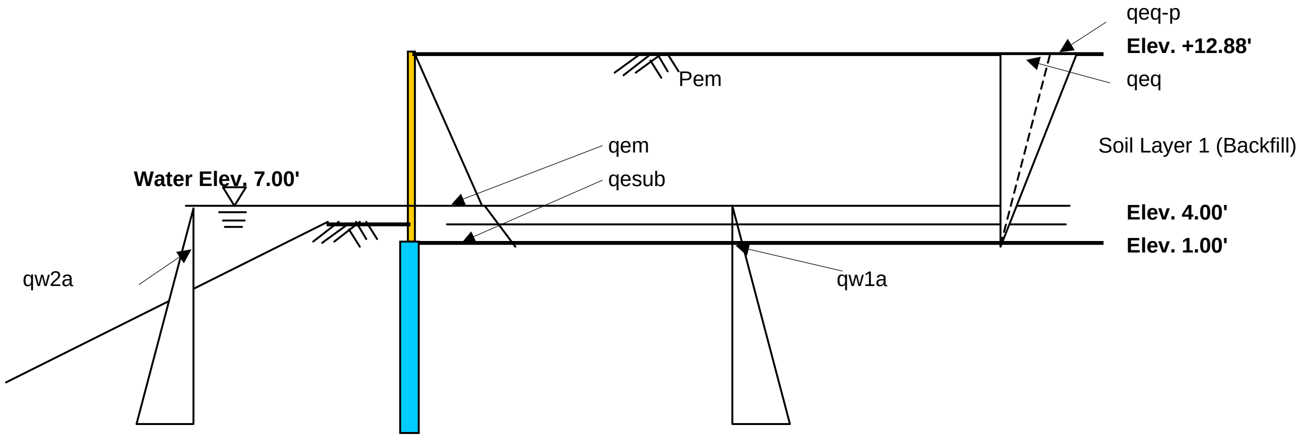
Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem	22.5	5.96	134
Pesub	16.5	1.46	24
Pw1a	3.4	1.00	3
Pw2a	-79.4	4.85	-385
At bot of wall	**-37.0**	Safety Factor	**-223.5**
	Σ V	**1.1**	Σ M

Demand at Top of Pile: Vd = -41 kips Md = -246 k-ft

MGE ENGINEERING, INC.

Project	Napa River Flood Control Project		
Subject	Flood Wall Design (Wall #1, Type C)		
By	David An	Date	Mar-05

Load Case 4 -- Long Term (Drained) In Service Condition With Earthquqke (Station 4+83)



Backfill Soil Pressure at Wall (Soil pressure = γ Ki hi)

Note: Passive soil resistance were ignored.

Name	Thickness(ft)	Pressure(ksf)
qem	5.88	0.280
qesub	3.00	0.351
qw1a	3.00	0.188
qw2a	3.00	0.188
qeq	11.88	0.170

qem - Moist soil pressure at rest wall
qesub - Submerged soil pressure at rest wall
qw - Water pressure

qeq - Seismic components

$\alpha = \tan^{-1}[(C_1+\sqrt{(C_1^2+4C_2)})/2]$

(Equation 3-56 of EM 1110-2-2502, Page 3-67)

$C_1 = 2\ (\tan\Phi - K_h) / (1+K_h \tan\Phi)$

(Equation 3-57 of EM 1110-2-2502, Page 3-67)

$C_2 = [\tan\Phi(1-\tan\Phi\ \ \mathrm{tar}\beta)-(\tan\beta-K_h)] / [\tan\Phi(1+K_h \tan\Phi)]$

(Equation 3-58 of EM 1110-2-2502, Page 3-67)

Dynamic Components qeq $= \gamma\ K_h\, h^2 / [2(\tan\alpha - \tan\beta)]$

(Equation 3-62 of EM 1110-2-2502, Page 3-68)

K_h =	0.15	g
β =	0	
Φ =	30	degree
C_1 =	0.787	
C_2 =	0.681	
α =	52.6	degree

Backfill Resultant Forces Summary

Name	Force	Arm to bot.	Moments
Pem	9.9	4.96	49
Pesub	11.3	1.44	16
Pw1a	3.4	1.00	3
Peq	12.1	7.92	96
Pw2a	-3.4	1.00	-3
At bot of wall	**33.4**	Safety Factor	**161.5**
	Σ V	**1.1**	Σ M

Demand at Top of Pile: Vd = 37 kips Md = 178 k-ft

MGE ENGINEERING, INC.

Project	Napa River Flood Control Project
Subject	Flood Wall Design (Wall #1, Type C)
By	David An
Date	Mar-05

Design Wall Section and Pile Reinforcement (Station 4+83)

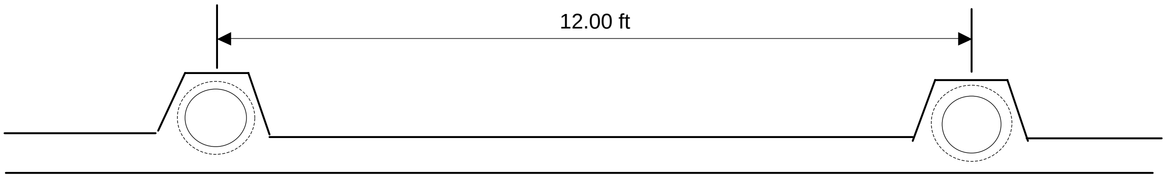
1. Loads

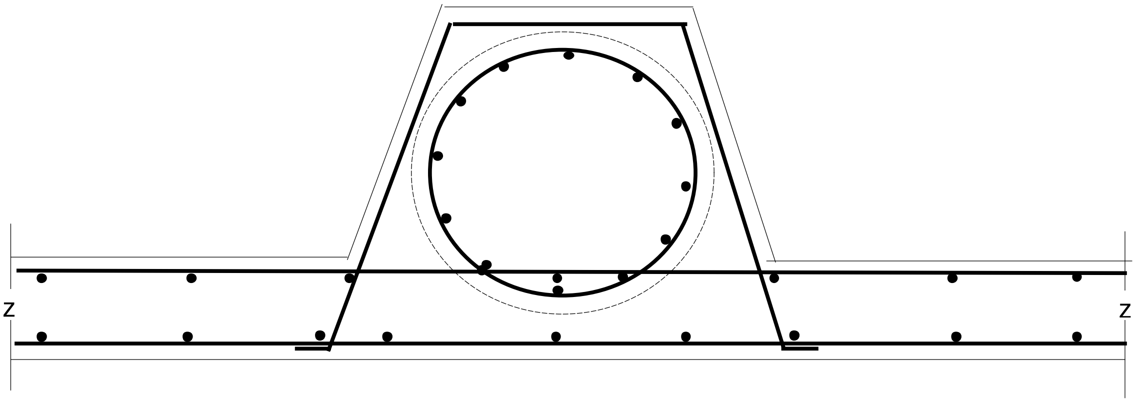
	Load Case	1	2	3	4
Forces w/o Safety Factor	Shear (k)	51.5	39.0	-37.0	33.4
	Moments(kft)	265.2	158.1	-223.5	161.5
	Safety Factor	1.1	1.3	1.1	1.1
Forces w/ Safety Factor	Shear (k)	56.7	50.7	-40.7	36.7
	Moments(kft)	291.7	205.6	-245.9	177.6

Demand at Top of Pile:

Vd= **57** kips

Md= **292** k-ft

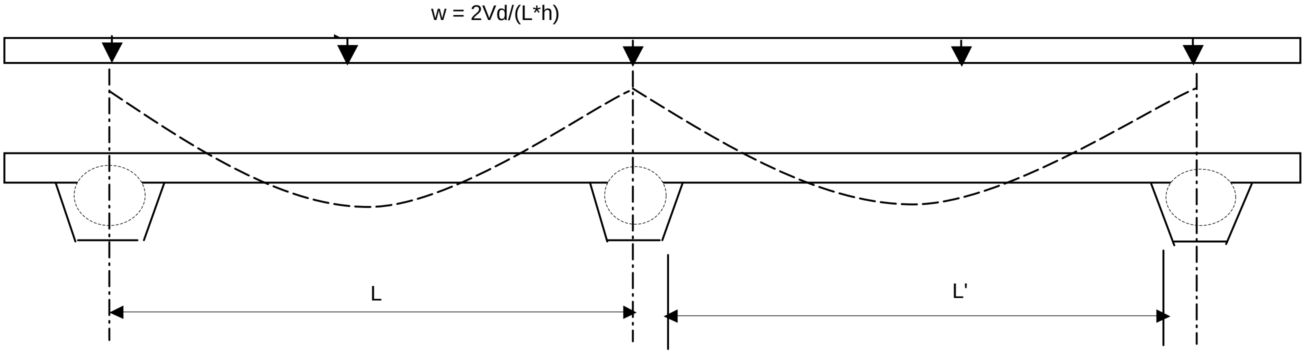




2. Reinforced Concrete Wall (unit width)

2a. As continuous beam

Flexure reinforcement requirement (bending about vertical axis)



Plan View

$Mu \le \Phi Mn$

$Mu = Hf \times 1.7 \times 0.1 \times w \times l^2$ (used 3 equal span continuous beam) 8.3 k-ft/ft

where

w = Vd/(L'*h) 0.61 kip/ft

L' = L-25/12x 2 7.8 ft

h = 11.9 ft

Hf = Hydraulic factor 1.30

EM-1110-2-2104---P3-2, Equation3.3

ΦMn = Φ As x fy (d - a/2)

where

d = 12in -2.5in (wall thick = 12 in) 9.5 in

a = 0.15d 1.4 in

Required reinforcement

As = Mu/[Φfy(d-a/2)] (Required) 0.21 in^2

where Φ= 0.90

fy = 60 ksi

use 1#5, As = 0.31 0.31 in^2

Check a =Asfy/(0.85f'c*b) 0.29 in

where f'c = 4 ksi

b = unit width of wall 12 in

ΦMn = Φ Asfy(d - a/2) 13.0 kft/ft

D/C = Mu / ΦMn 0.63 **OK**

Shear check

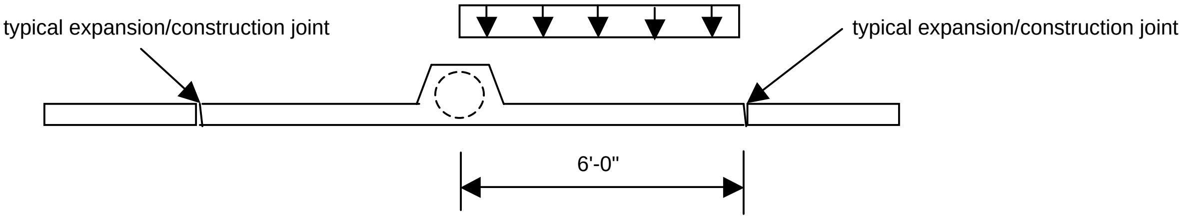
Vu = Hf x 1.7 x Vd / h /2 = 5.3 kips/ft

ΦVc = Φ x 2 x √f'c x b x d 12.3 kips

where Φ = 0.85

D/C = Vu / ΦVc 0.43 **OK**

2b. As cantilever beam



Mu = Hf x 1.7 x Mmax = 15.19 kft/ft

where Mmax = wl^2/2 6.87 k/ft

w = 0.61 k/ft/ft

l = 6 - 15/12 4.8 ft

Mn = Φ x As x fy (d-a/2) =

Φ = 0.9

d = 12in -2.5in (wall thick = 12 in) 9.5 in

assume a = 0.15d 1.4 in

Required reinforcement

As = Mu/[Φfy(d-a/2)] 0.38 in^2

fy = 60 ksi

use 1#6, As = 0.44 in^2 0.44 in^2

Check a =Asfy/(0.85f'c*b) 0.65 in

where f'c = 4 ksi

b = unit width of wall 12 in

ΦMn = Φ Asfy(d - a/2) 18.2 kft/ft

D/C = Mu / ΦMn 0.93 **OK**

Shear check **Say OK**

Vu = Hf x 1.7 x w x l = 6.4 kips/ft

ΦVc = Φ x 2 x √f'c x b x d 12.3 kips

where Φ = 0.85

D/C = Vu / ΦVc 0.52 **OK**

Consider 2a. And 2b., Use #6@12" for both faces.

Deflection check

Δmax =wl^4/(8EI) 0.01 in

where E = 57√f'c = 3605.00 ksi

I = 12 x 12^3 / 12 = 1728.00 in^4

OK

3. Connections between wall and piles

Use 24" CIDH Piles at spacing of 12.00'

Rebar Size (#): 10

Number of Rebar: 14

Spiral Spacing: 5 in

Moment capacity Mn (Mp) = (from Xsection) 824 k-ft

Mu = Hf x 1.7 x Md 645 k-ft

D/C = Mu/ΦMn = Hf x 1.7 x Md/ΦMn 0.87 **OK**

Use shear-friction design method

Vn = Avf x fy x μ BDS p8-26 640 kips

where Avf = 17.78 in^2

fy = 60 ksi

μ = 0.6 λ 0.6

λ = normal concrete 1.0

D/C = Vu/ΦVn = Hf x 1.7 x Vd/ΦVn 0.23 **OK**

4. Piles

Shear capacity of pile

ΦVn = Φ(Vc+Vs) 153 kips

where Φ = 0.85

Vc = 2 x √f'c x Ae 57 kips

Vs = π/2 Av fy d / s 123 kips

Av Rebar Size (#) 5

Av = 0.31 in^2

d = 24" -3" 21 in

s = 5 in

Ae = 0.85Ag 452 in^2

D/C = Vu/ΦVn = Hf x 1.7 x Vd/ΦVn 0.82 **OK**

Use #5@5" for Spirals.

Pile reinforcement development length, ld

ld = max{ ACI.R12..2, ACI.R12.2.3} 60.2 in

ACI R12.2.2 ld = [fy x αβλ/(20√f'c)]db = 60.2 in
ACI R12.2.3 ld = {3/40 x fy/√f'c x αβγλ/[(c+Ktr)/db]} = 28.5 in

where
α = reinforcement location factor 1.0
β = coating factor 1.0
γ = reinforcement size factor 1.0
λ = lightweight aggregate concrete factor 1.0
c = cover 2.69 in
(clear cover c > db)
Ktr = Atr x fyt / (1500sn) = 0.601
where Atr = 0.79 in^2
fyt = fy 60.0 ksi
s = rebar spacing 3.8 in
(clear s > 2db)
n = number of bars 14
db = nominal diameter of bar 1.27 in
(c+Ktr)/db = 2.592
use (c+Ktr)/db = 2.5

Bar #	Area (SI)
3	0.11
4	0.20
5	0.31
6	0.44
7	0.60
8	0.79
9	1.00
10	1.27
11	1.56
14	2.25
18	4.00

WALL #2 TO #6 DESIGN

Upper Wall Design

Design Height H= 6.00 ft

Backfill Properties

Backfill Thickness =	6.00	ft
Backfill Unit Weight =	125	pcf
Φ =	37	degree
C =	0	pcf
SMF= Tan(Φ_d) / TanΦ = 2/3 =	0.67	
Φ_d =	27	degree
Ka = Tan^2 (45° - Φ/2) =	0.25	
Ko = Tan^2 (45° - Φ_d/2) =	0.38	
Kp = Tan^2 (45° + Φ/2) =	4.02	

Water Property

Water Unit Weight =	62.5	pcf

Wall and Footing Data

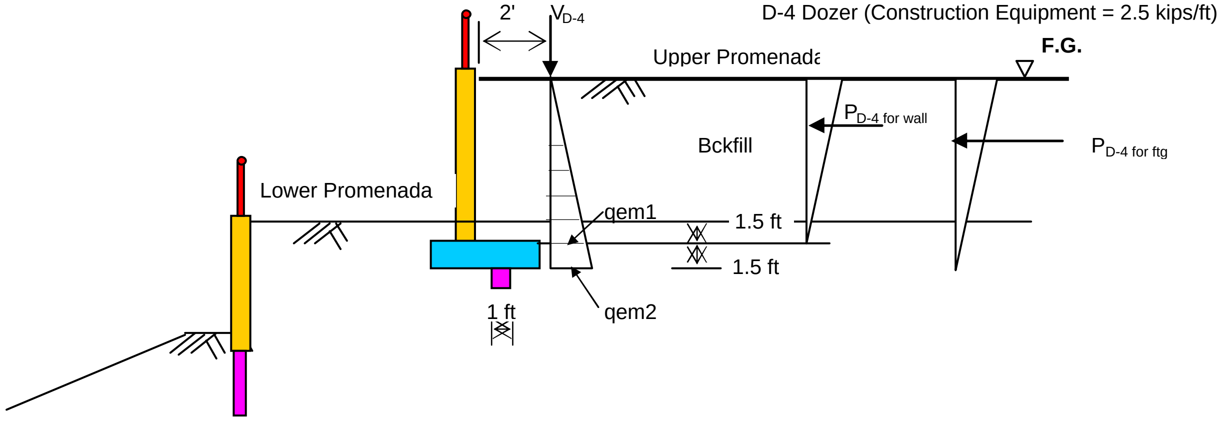
Design Height (ft)	6
Toe Cover (ft)	1.5
Top Wall Thick (ft)	1.0
100 Year Flood Level to Top of Wall (ft)	2.00

Upper Wall Design (H=6.00 ft)

Load case based on DOA--Flood Walls and Channel Retaining Walls, Load Conditions & Load Diagrams

Load Case 1 -- Construction Condition (Unusal Condition)

Wall Design Height, H= 6.00 ft



Soil Pressure at Wall (Soil pressure =γ Ki hi)

Note: Passive soil resistance were ignored.

Name	Layer	Thickness(ft)	Pressure(ksf)
qem1	Backfill	6.00	0.285
qem2	Backfill	1.50	0.357

qem - Moist soil pressure at rest wall

D-4 Load---- **For bottom of wall**

h = 6.00 ft

a = 2' / 6.0' = 0.33 ≤ 0.4

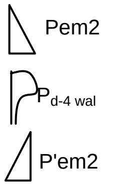
V_{D-4} = 2.5 kips/ft

$\Delta P_{D-4} = (V_{D-4} /h)\ [(0.203b)/(0.16+b^2)^2]$ (EM 1110-2-2502 Page 3-49)

b	Z = bh	$\Delta P_{D-4\ wal}$	Moment
0.0	0.00	0.000	0.000
0.1	0.60	0.176	0.948
0.2	1.20	0.254	1.218
0.3	1.80	0.244	1.023
0.4	2.40	0.198	0.714
0.5	3.00	0.151	0.453
0.6	3.60	0.113	0.270
0.7	4.20	0.084	0.151
0.8	4.80	0.063	0.076
0.9	5.40	0.049	0.029
1.0	6.00	0.038	0.000
Σ		1.369	4.883

Resultant Summary for Bottom of Wall

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem1	0.9	2.00	1.7
Pd-4 wall	1.4		4.9
At bot of wall	**2.2**		**6.6**
	Σ V		Σ M



Demand at Bottom of Wall: Vd = 2.2 kips Md = 6.6 k-ft

D-4 Load---- **For bottom of footing**

h = 7.50 ft

a = 2' / 7.5' = 0.27 ≤ 0.4

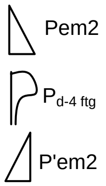
V_{D-4} = 2.5 kips/ft

$\Delta P_{D-4} = (V_{D-4} /h) [(0.203b)/(0.16+b^2)^2]$ (EM 1110-2-2502 Page 3-49)

b	Z = bh	$\Delta P_{D-4\ wal}$	Moment
0.0	0.00	0.000	0.000
0.1	0.75	0.176	1.185
0.2	1.50	0.254	1.523
0.3	2.25	0.244	1.279
0.4	3.00	0.198	0.892
0.5	3.75	0.151	0.566
0.6	4.50	0.113	0.338
0.7	5.25	0.084	0.189
0.8	6.00	0.063	0.095
0.9	6.75	0.049	0.036
1.0	7.50	0.038	0.000
Σ		1.369	6.103

Resultant Summary for Bottom of Footing

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem2	1.3	2.50	3.3
Pd-4 ftg	1.4		6.1
At bot of ftg	**2.7**		**9.4**
	Σ V		Σ M

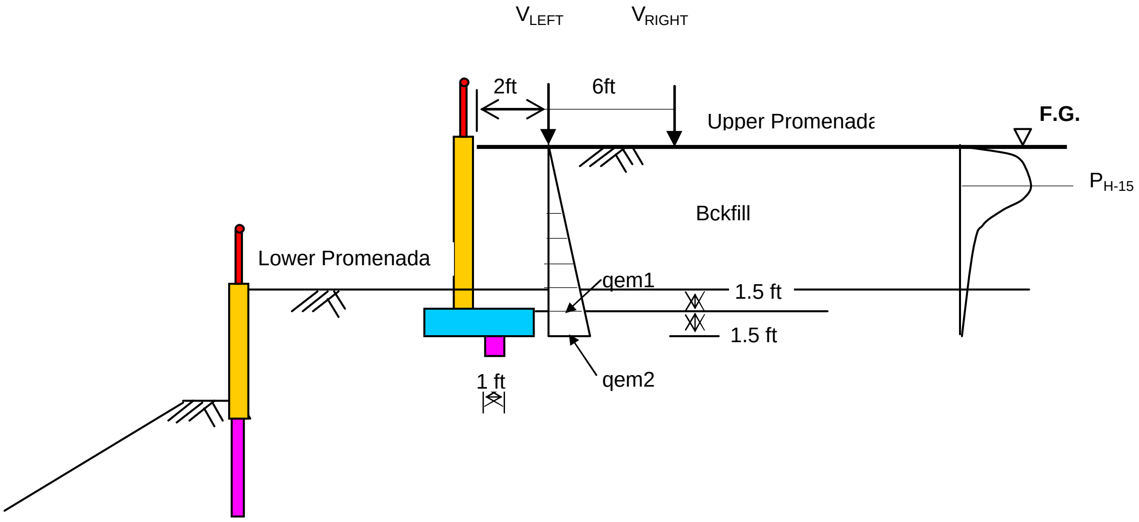


Demand at Bottom of Footing: Vd = 2.7 kips Md = 9.4 k-ft

Load Case 2 -- Normal Condition W/Maintenance Vehicle (Usual Condition)

Wall Design Height, H= 6.00 ft

H-15 truck load **USE V= 16 kips**



Soil Pressure at Wall (Soil pressure =γ Ki hi)

Note: Passive soil resistance were ignored.

Name	Layer	Thickness(ft)	Pressure(ksf)
qem1	Backfill	6.00	0.285
qem2	Backfill	1.50	0.357

qem - Moist soil pressure at rest wall

H-15 Load---- For bottom of footing

h = 6.00 ft

a=2'/6.00' = 0.33 ≤ 0.4 **For V_{LEFT}**

a=8'/6.00' = 1.33 > 0.4 **For V_{RIGHT}**

$\Delta P_{HZleft} = (0.28V/h^2)\ [b^2 / (0.16+b^2)^3]$ (EM 1110-2-2502 Page 3-48)

$\Delta P_{HZright} = (V/h^2)\ [a^2 b^2 / (a^2+b^2)^3]$ (EM 1110-2-2502 Page 3-48)

b (for V_{LEFT})	Z	$\Delta P_{PH\ (LEFT)}$	Moment
0.1	0.60	0.152	0.821
0.2	1.20	0.373	1.792
0.3	1.80	0.430	1.806
0.4	2.40	0.365	1.313
0.5	3.00	0.271	0.813
0.6	3.60	0.191	0.459
0.7	4.20	0.133	0.240
0.8	4.80	0.093	0.112
0.9	5.40	0.066	0.040
1.0	6.00	0.048	0.000
Σ		2.123	7.394

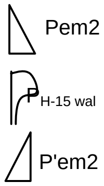
b (for V_{RIGHT})	Z	$\Delta P_{PH\ (RIGHT)}$	Moment
0.1	0.60	0.001	0.008
0.2	1.20	0.005	0.025
0.3	1.80	0.011	0.044
0.4	2.40	0.016	0.059
0.5	3.00	0.022	0.065
0.6	3.60	0.025	0.061
0.7	4.20	0.028	0.050
0.8	4.80	0.029	0.035
0.9	5.40	0.029	0.017
1.0	6.00	0.027	0.000
Σ		0.193	0.363

MGE ENGINEERING, INC.

Project	Napa River Flood Control Project		
Subject	Upper Wall Design (H=6')		
By	David An	Date	Feb-05

Resultant Summary for Bottom of Wall

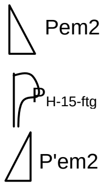
Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem1	0.9	2.00	1.7
$P_{H\text{-}15\ wall}$	2.3		
At bot of wall	**3.2**		**1.7**
	Σ V		Σ M



Demand at Bottom of Wall: Vd = 3.2 kips Md = 1.7 k-ft

Resultant Summary for Bottom of Footing

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem2	1.3	2.50	3.3
$P_{H\text{-}15\ ftg}$	2.3		
At bot of ftg	**3.7**		**3.3**
	Σ V		Σ M



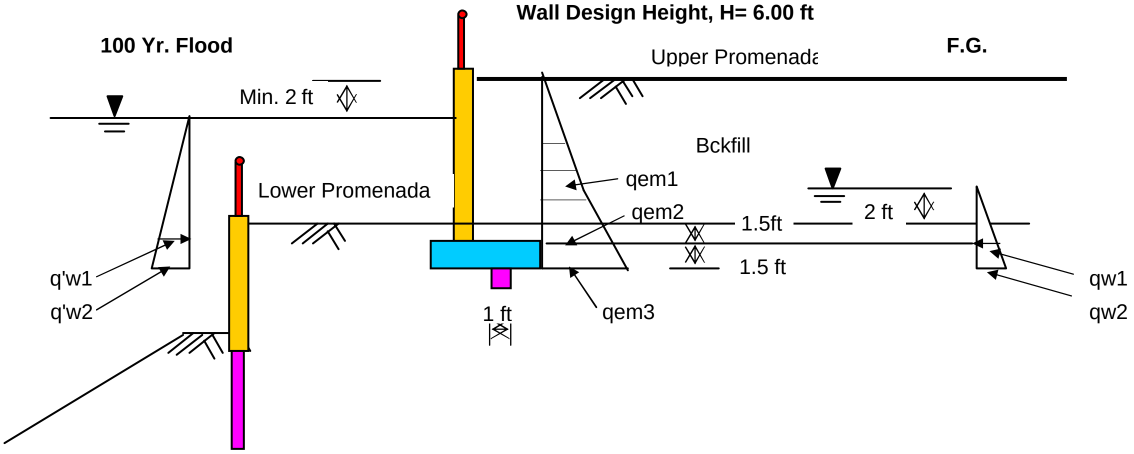
Demand at Bottom of Footing: Vd = 3.7 kips Md = 3.3 k-ft

MGE ENGINEERING, INC.

Project: Napa River Flood Control Project
Subject: Upper Wall Design (H=6')
By: David An
Date: Feb-05

Load Case 3a -- Design Flood Condition (Unusual Condition)

Wall Design Height, H= 6.00 ft



Soil Pressure at Wall (Soil pressure =γ Ki hi)

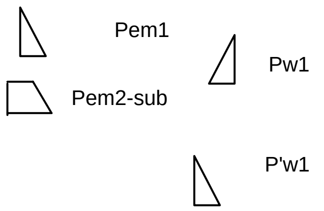
Note: Passive soil resistance were ignored.

Name	Layer	Thickness(ft)	Pressure(ksf)
qem1	Backfill	2.50	0.119
qem2	Backfill	3.50	0.284
qem3	Backfill	1.50	0.355
qw1	Backfill	3.50	0.002
qw2	Backfill	1.50	0.003
q'w1	Backfill	4.00	0.003
q'w2	Backfill	1.50	0.004

qem1 - Moist soil pressure at rest wall
qem2 & qem3 - Submerged soil pressure at rest wall
qw - Water pressure

Resultant Summary For Bottom Of Wall

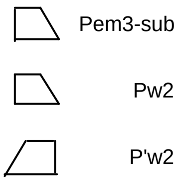
Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem1	0.1	4.33	0.64
Pem2	0.7	1.51	1.07
Pw1	0.004	1.17	0.005
P'w1	-0.01	0.50	0.00
At bot of wall	**0.85**		**1.7**
	Σ V		Σ M



Demand at Bottom of Wall: Vd = .85 kips Md = 1.71 k-ft

Resultant Summary For Bottom Of Footing

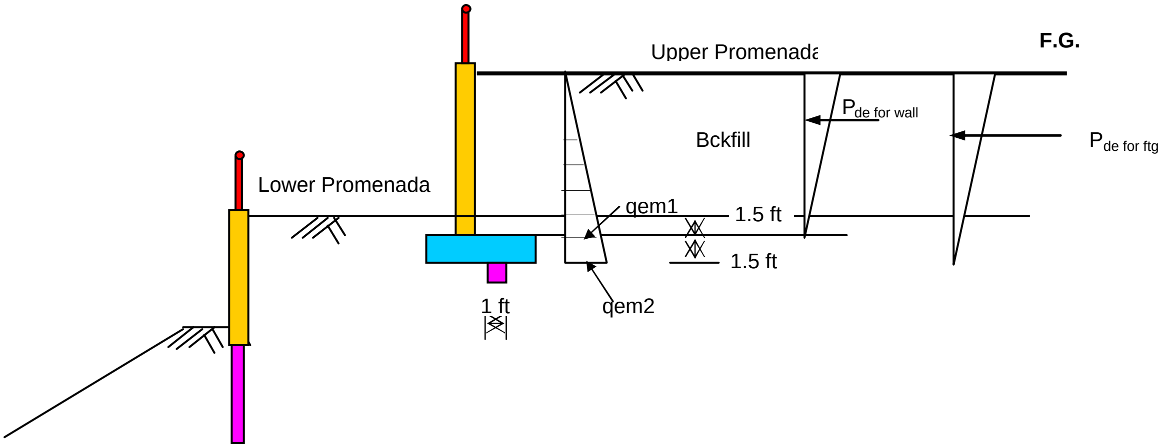
Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem1	0.1	5.83	0.87
Pem2	0.7	3.01	2.12
Pem3	0.5	0.72	0.35
Pw1	0.00	2.67	0.01
Pw2	0.0	0.71	0.00
P'w1	-0.01	1.33	0.0
P'w2	0.00	0.71	0.00
At bot of ftg	**1.34**		**3.3**
	Σ V		Σ M



Demand at Bottom of Wall: Vd = 1.34 kips Md = 3.34 k-ft

Load Case 4 -- Seismic Condition (Extreme condition)

Wall Design Height, H= 6.00 ft



Soil Pressure (Soil pressure =γ Ki hi)

Note: Passive soil resistance were ignored.

Name	Layer	Thickness(ft)	Pressure(ksf)
qem1	Backfill	6.00	0.285
qem2	Backfill	1.50	0.357
qde-wall	Backfill	6.00	0.086
qde-ftg	Backfill	1.50	0.022

qem - Moist soil pressure at rest wall

qde - Earthquake component

qw - Water pressure

Resultant Summary For bottom of wall

Name	Force	Arm to bot.	Moments
Pem1	0.9	2.00	1.7
Pde-wall	0.3	4.00	1.0
At bot of wall	**1.1**		**2.7**
	Σ V		Σ M

Demand at Top of Pile: Vd = 1.11 kips Md = 2.74 k-ft

Pem1

Pde-wall $\alpha = \tan^{-1}[(C_1+\sqrt{(C_1^2+4C_2)})/2]$

(Equation 3-56 of EM 1110-2-2502, Page 3-67)

P'em1 $C_1 = 2(\tan\Phi - K_h)/(1+K_h \tan\Phi)$

(Equation 3-57 of EM 1110-2-2502, Page 3-67)

$C_2 = [\tan\Phi(1-\tan\Phi\ \tan\beta)-(\tan\beta-K_h)] / [\tan\Phi(1+K_h \tan\Phi)]$

(Equation 3-58 of EM 1110-2-2502, Page 3-67)

Resultant Summary For Bottom Of Ftg

Name	Force	Arm to bot.	Moments
Pem2	1.3	2.50	3.3
Pde-ftg	0.02	1.00	0.02
At bot of ftg	**1.4**		**3.4**
	Σ V		Σ M

Demand at Top of Pile: Vd = 1.35 kips Md = 3.36 k-ft

Pem2

Pde-ftg

K_h =	0.15	g
β =	0	
Φ =	30	degree
C_1 =	0.787	
C_2 =	0.681	
α =	52.6	degree

Dynamic Components qeq =$\gamma\ K_h h^2 / [2(\tan\alpha - \tan\beta)]$

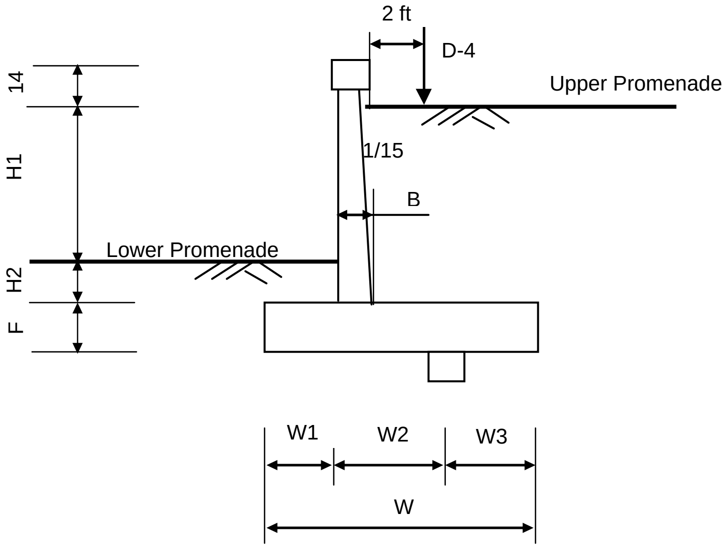
(Equation 3-62 of EM 1110-2-2502, Page 3-68)

Footing & Wall Design (Upper Wall)

Wall Design Height, H= 6.00 ft

1. Loads

Load Case	1		2		3		4	
	Bot. Of Wall	Bot. Of Ftg	Bot. Of Wall	Bot. Of Ftg	Bot. Of Wall	Bot. Of Ftg	Bot. Of Wall	Bot. Of Ftg
Hori. Force (k)	2.2	2.7	3.2	3.7	0.9	1.3	1.1	1.4
Moments(kft)	6.6	9.4	1.7	3.3	1.7	3.3	2.7	3.4



B = 1.30 ft
W1 = 2.00 ft
W2 = 2.00 ft
W3 = 1.25 ft
W = 5.25 ft

H1 = 4.50 ft
H2 = 1.50 ft
F = 1.5 ft

2. Resistances

Location	Dimension		Weight(k/ft)	Arm to toe (ft)	Mot (kft/ft)
	Thick/Depth	Width/Height			
Toe cover	1.50	2.00	0.375	1.00	0.38
Heel Soil	6.00	1.95	1.463	4.28	6.25
Footing	1.50	5.25	1.181	2.63	3.10
Concrete Key	1.00	1.00	0.15	4.00	0.60
Wall Stem	1.30	7.17	1.398	2.65	3.70
Total			**4.57**		**14.03**

Due to D-4
Vertical weight

Distributed weight, Pd-4 = 2.50 k/ft
Resistance of overturning due to D-4 = distributed weight x (W1 + 1 + 2ft) 12.50 kft/ft

MGE ENGINEERING, INC.			
Project	Napa River Flood Control Project		
Subject	Upper Wall Design (H=6')		
By	David An	Date	Feb-05

3. Overturning Check

Load Case	Resistance Moment	Overturning Moment	Safety Factor	
1	26.53	9.4	2.81	**Safety Factor > 1.5, OK**
2	14.03	3.3	4.20	**Safety Factor > 1.5, OK**
3	14.03	3.3	4.20	**Safety Factor > 1.5, OK**
4	14.03	3.4	4.18	**Safety Factor > 1.5, OK**

4. Sliding Check

Resistance Due to Soil Friction

μ = 0.3

Ffr = $P_d \times \mu$ 1.4 kips/ft

where, P_d -- weight of concrete & soil

Resistance Due to D-4

Fd-4 = $Pd_{-4} \times \mu$ 0.750 kips/ft

Pd_{-4}--distributed weight due to D-4

Resistance Due to Concrete Key

Fcr = 2√f'c x 1 ft 18.21 kips/ft

where, f'c = 4000 psi

Load Case	Resistance Due to Soil (kips)	Sliding Force (kips)	Safety Factor		Resistance Due to Concrete Key (kips)	Sliding Force (kips)	Safety Factor	
1	1.37	2.7	0.51	**< 1.33, NG**	18.21	2.7	6.73	**> 1.33, OK**
2	1.37	3.7	0.38	**< 1.5, NG**	18.21	3.7	4.99	**> 1.5, OK**
3	1.37	1.3	1.02	**< 1.5, NG**	18.21	1.3	13.62	**> 1.5, OK**
4	1.37	1.4	1.01	**< 1.1, NG**	18.21	1.4	13.46	**> 1.1, OK**

5. Soil Bearing Pressure Check

Moment about center of footing

M = Msmax(due to soil pressure) + Mt(due to toe soil) - Mk(due to key) - Mw(due to wall) - Mh(due to heel soil) - M_{d4}(Due to D-4)

M = 1.5 kft/ft

where, Max. M (due to lateral soil pressure), Msmax. = 9.4 kft/ft

Msmax = (case1, case2, case3, case4)

Mt =H2 x W1 x 0.12 x (W/2-W1/2) 0.6 kft/ft

Mk = - (1ft x 1ft x 0.15 x (W/2-W3)) -0.21 kft/ft

Mw = B x (H1 + H2 + 14/12) x 0.15 x (W/2-B/2-W1) -0.03 kft/ft

Mh =- (H1 + H2)x0.12x[W2 + W3 - (B+1)/2]x{W/2-[W2+W3-(B+1)/2]/2} -2.38 kft/ft

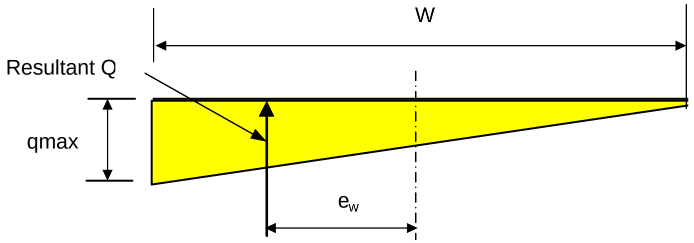
$M_{D-4} = - P_{D-4}$ x (W/2 - W1-1ft - 2) -5.94 kft/ft

Eccentricity e_W = M / (Pd+Pd-4) 0.21 ft

W / 6 = 0.88 ft

Therefor e_W < W / 6

Footing Contact Pressure



MGE Engineering, Inc.	Project	Napa River Flood Control Project	
	Subject	Upper Wall Design (H=6')	
	By	David An	Date Feb-05

Q = Pd + Pd-4 7.07 kips

1 + (6 e_w / W) = 1.24 ft

1 - (6 e_w / W) = 0.76 ft

qmax = Q [1 + (6ew / W)] / W 1.67 ksf

qmin = Q [1 - (6ew / W)] / W 1.03 ksf

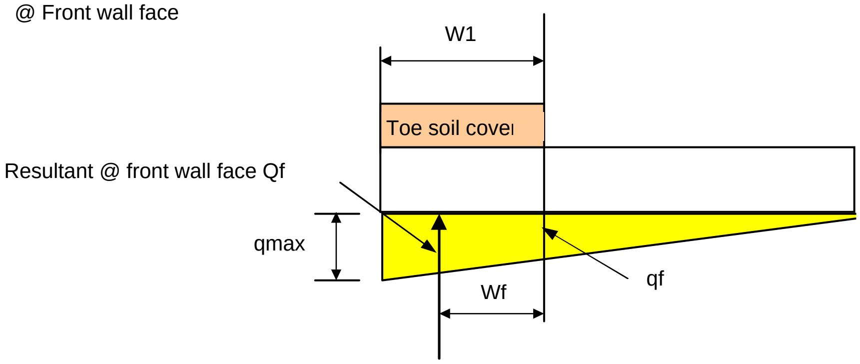
assumed allowable soil brearing pressure

qa = controlling 2.00 ksf

D / C = qmax / qa = 0.83 **<1, OK**

6. Footing Shear and Moment Check (@ face of wall)

@ Front wall face



qf = (W2+W3) / W x (qmax - qmin) + qmin 1.42 ksf

Qf =(qf+qmax)/2 x W1 3.09 k/ft

Arm to face of wall

Wf = W1 (2qmax+qf) / [3 (qmax+qf)] 1.03 ft

Mu = Hf{1.7 [Qf x Wf - (W1x F x 0.15 x W1/2 -W1 x h2 x 0.12 x W1/2)]} 5.22 kft/ft

Mu = Hf[1.7(D+L)]

Hf = Hydraulic factor 1.30

EM-1110-2-2104---P3-2, Equation3.3

Vu = Hf{1.7[Qf - (W1 x F x 0.15 - W1 x 1ft x 0.12)]} 5.04 k/ft

Required bar area for flexure

Mu ≤ ΦMn

ΦMn = Φ As x fy (d - a/2)

Assume a = 0.15d 2.25 in

where d = 15 in

As = Mu / [Φfy(d-a/2)] 0.08 in^2

Φ = 0.9

fy = 60 ksi

Use #4@12", As = 0.20 in^2 As = 0.20 in^2

Check

a =Asfy / (0.85f'c*b) 0.3 in

ΦMn = Φ Asfy(d - a/2) 13.37 kft/ft

D/C = Mu/ΦMn 0.39 **OK**

Shear Check

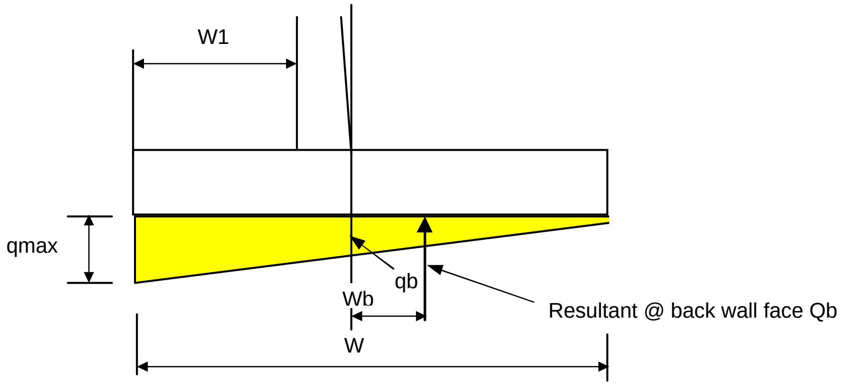
Shear capacity

ΦVn = Φ2√f'c x F 23.22 k/ft

where Φ = 0.85

D/C = Vu/ΦVn 0.22 **OK**

@ Back wall face



qb = (W-W1-B) / W x qmax + qmin 1.26 ksf

Qb =(qf+qmin)/2 x (W-W1-B) 2.23 k/ft

Arm to face of wall

Wb =(W-W1-B) - (W- W1-B) (2qb+qmin) / [3 (qb+qmin)] 0.94 ft

Mu = Hf{1.7[Qb x Wb - (F x 0.15 x (W-W1-B)^2/2 + (H1+H2) x 0.12 x (W-W1-B)/2)-(P_{h15} x 2ft)]

-10.38 kft/ft

Mu = Hf[1.7(D+L)]

Hf = Hydraulic factor 1.30

EM-1110-2-2104---P3-2, Equation3.3

Vu = Hf x 1.7[Qb - ((W-W1-B) x F x 0.15 + (W-W1-B) x H2 x 0.12) - Pd-4] -4.66 k/ft

Required bar area for flexure

Mu ≤ ΦMn

ΦMn = Φ As x fy (d - a/2)

Assume a = 0.15d 2.25 in

where d = 15 in

As = Mu / [Φfy(d-a/2)] 0.17 in^2

Φ = 0.9

fy = 60 ksi

Use #4@12", As = 0.20 in^2 As = 0.20 in^2

Check

a =Asfy / (0.85f'c*b) 0.3 in

ΦMn = Φ Asfy(d - a/2) 13.37 kft/ft

D/C = Mu/ΦMn 0.78 **OK**

Shear Check

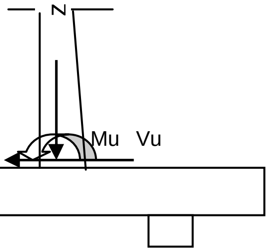
Shear capacity

ΦVn = Φ2√f'c x F 23.22 k/ft

where Φ = 0.85

D/C = Vu/ΦVn 0.20 **OK**

7. Wall Stem Shear and Moment Check (@ bottom of wall)



Mu = Hf x 1.7 x M			14.6	kft/ft
Where	M = max(case1, case2, case3, case4)			
	Mu = Hf[1.7(D+L)]			
	Hf =	Hydraulic factor	1.30	
	EM-1110-2-2104---P3-2, Equation3.3			
Vu = Hf x 1.7 x Vmax		Vmax = max(case1, case2, case3, case4)	7.0	k/ft
Required bar area for flexure				
	Mu ≤ ΦMn			
	ΦMn = Φ As x fy (d - a/2)			
	Assume a = 0.15d		1.89	in
	where	d =	12.6	in
	As = Mu / [Φfy(d-a/2)]		0.28	in^2
		Φ =	0.9	
		fy =	60	ksi
		f'c =	4000	psi
	Use #5@12", As = 0.31 in^2	As =	0.31	in^2
Check				
	a =Asfy / (0.85f'c*b)		0.5	in
	ΦMn = Φ Asfy(d - a/2)		17.26	kft/ft
	D/C = Mu/ΦMn		0.84	**OK**
Shear Check				
	Shear capacity			
	ΦVn = Φ2√f'c x B		20.13	k/ft
	where	Φ =	0.85	
	D/C = Vu/ΦVn		0.35	**OK**

Upper Wall Design

Design Height H= 8.00 ft

Backfill Properties

Backfill Thickness =	8.00	ft
Backfill Unit Weight =	125	pcf
Φ =	37	degree
C =	0	pcf
SMF= $\tan(\Phi_d)$ / $\tan\Phi$ = 2/3 =	0.67	
Φ_d =	27	degree
Ka = $\tan^2(45° - \Phi/2)$ =	0.25	
Ko = $\tan^2(45° - \Phi_d/2)$ =	0.38	
Kp = $\tan^2(45° + \Phi/2)$ =	4.02	

Water Property

Water Unit Weight =	62.5	pcf

Wall and Footing Data

Design Height (ft)	8
Toe Cover (ft)	1.5
Top Wall Thick (ft)	1.0
100 Year Flood Level to Top of Wall (ft)	2.00

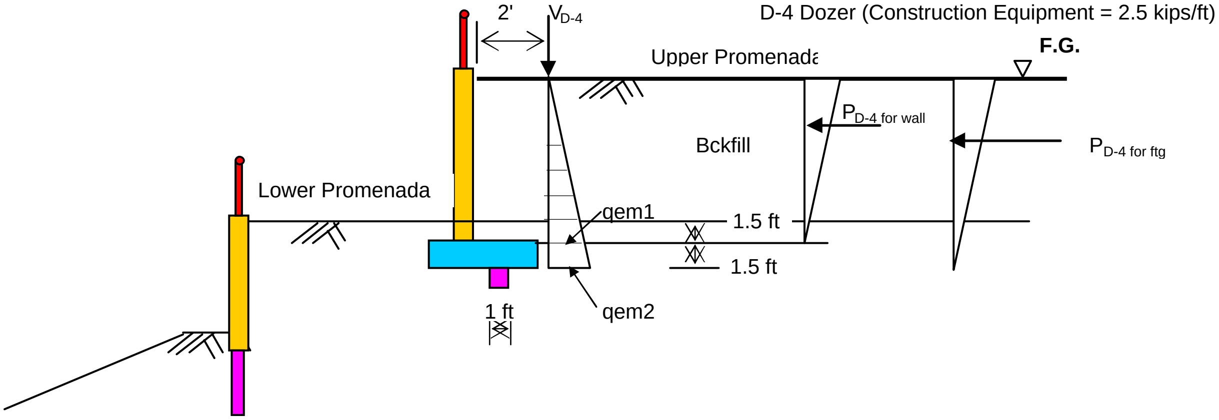
MGE ENGINEERING, INC.				
	Project	Napa River Flood Control Project		
	Subject	Upper Wall Design (H=8')		
	By	David An	Date	Feb-05

Upper Wall Design (H=8.00 ft)

Load case based on DOA--Flood Walls and Channel Retaining Walls, Load Conditions & Load Diagrams

Load Case 1 -- Construction Condition (Unusal Condition)

Wall Design Height, H= 8.00 ft



Soil Pressure at Wall (Soil pressure =γ Ki hi)

Note: Passive soil resistance were ignored.

Name	Layer	Thickness(ft)	Pressure(ksf)
qem1	Backfill	8.00	0.380
qem2	Backfill	1.50	0.452

qem - Moist soil pressure at rest wall

D-4 Load---- **For bottom of wall**

h = 8.00 ft

a = 2' / 8.0' = 0.25 ≤ 0.4

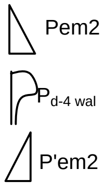
V_{D-4} = 2.5 kips/ft

$\Delta P_{D-4} = (V_{D-4} /h)\ [(0.203b)/(0.16+b^2)^2]$ (EM 1110-2-2502 Page 3-49)

b	Z = bh	$\Delta P_{D-4\ wal}$	Moment
0.0	0.00	0.000	0.000
0.1	0.80	0.176	1.264
0.2	1.60	0.254	1.624
0.3	2.40	0.244	1.364
0.4	3.20	0.198	0.952
0.5	4.00	0.151	0.604
0.6	4.80	0.113	0.360
0.7	5.60	0.084	0.202
0.8	6.40	0.063	0.102
0.9	7.20	0.049	0.039
1.0	8.00	0.038	0.000
Σ		1.369	6.510

Resultant Summary for Bottom of Wall

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem1	1.5	2.67	4.1
Pd-4 wall	1.4		6.5
At bot of wall	**2.9**		**10.6**
	Σ V		Σ M



Demand at Bottom of Wall: Vd = 2.9 kips Md = 10.6 k-ft

D-4 Load---- **For bottom of footing**

$h =$ 9.50 ft

$a = 2' / 9.5' =$ 0.21 ≤ 0.4

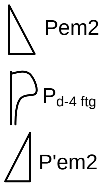
$V_{D-4} =$ 2.5 kips/ft

$\Delta P_{D-4} = (V_{D-4} /h) [(0.203b)/(0.16+b^2)^2]$ (EM 1110-2-2502 Page 3-49)

b	Z = bh	$\Delta P_{D-4\ wall}$	Moment
0.0	0.00	0.000	0.000
0.1	0.95	0.176	1.501
0.2	1.90	0.254	1.929
0.3	2.85	0.244	1.620
0.4	3.80	0.198	1.130
0.5	4.75	0.151	0.717
0.6	5.70	0.113	0.428
0.7	6.65	0.084	0.240
0.8	7.60	0.063	0.121
0.9	8.55	0.049	0.046
1.0	9.50	0.038	0.000
Σ		1.369	7.731

Resultant Summary for Bottom of Footing

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem2	2.1	3.17	6.8
Pd-4 ftg	1.4		7.7
At bot of ftg	**3.5**		**14.5**
	Σ V		Σ M



Demand at Bottom of Footing: Vd = 3.5 kips Md = 14.5 k-ft

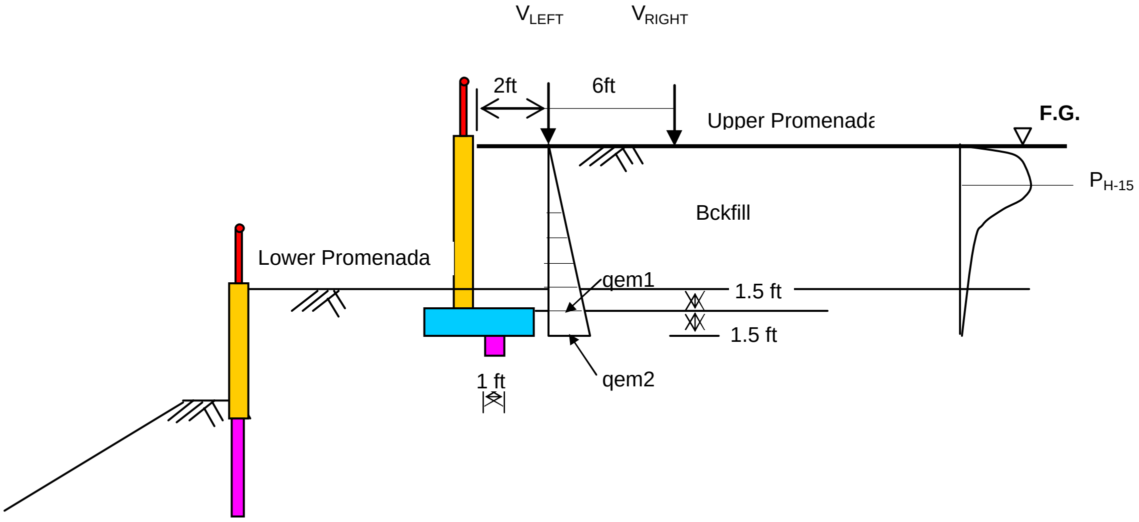
MGE ENGINEERING, INC.

Project Napa River Flood Control Project
Subject Upper Wall Design (H=8')
By David An
Date Feb-05

Load Case 2 -- Normal Condition W/Maintenance Vehicle (Usual Condition)

Wall Design Height, H= 8.00 ft

H-15 truck load **USE V= 16 kips**



Soil Pressure at Wall (Soil pressure =γ Ki hi)

Note: Passive soil resistance were ignored.

Name	Layer	Thickness(ft)	Pressure(ksf)
qem1	Backfill	8.00	0.380
qem2	Backfill	1.50	0.452

qem - Moist soil pressure at rest wall

H-15 Load---- **For bottom of footing**

h = 8.00 ft

a=2'/8.00' = 0.25 ≤ 0.4 **For V_{LEFT}**

a=8'/8.00' = 1.00 > 0.4 **For V_{RIGHT}**

$\Delta P_{HZleft} = (0.28V/h^2)\ [b^2 / (0.16+b^2)^3]$ (EM 1110-2-2502 Page 3-48)

$\Delta P_{HZright} = (V/h^2)\ [a^2 b^2 / (a^2+b^2)^3]$ (EM 1110-2-2502 Page 3-48)

b (for V_{LEFT})	Z	$\Delta P_{PH\ (LEFT)}$	Moment
0.1	0.80	0.114	0.821
0.2	1.60	0.280	1.792
0.3	2.40	0.323	1.806
0.4	3.20	0.273	1.313
0.5	4.00	0.203	0.813
0.6	4.80	0.143	0.459
0.7	5.60	0.100	0.240
0.8	6.40	0.070	0.112
0.9	7.20	0.050	0.040
1.0	8.00	0.036	0.000
Σ		1.592	7.394

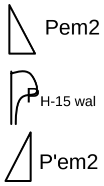
b (for V_{RIGHT})	Z	$\Delta P_{PH\ (RIGHT)}$	Moment
0.1	0.80	0.001	0.008
0.2	1.60	0.004	0.025
0.3	2.40	0.008	0.044
0.4	3.20	0.012	0.059
0.5	4.00	0.016	0.065
0.6	4.80	0.019	0.061
0.7	5.60	0.021	0.050
0.8	6.40	0.022	0.035
0.9	7.20	0.021	0.017
1.0	8.00	0.021	0.000
Σ		0.145	0.363

MGE ENGINEERING, INC.

Project	Napa River Flood Control Project		
Subject	Upper Wall Design (H=8')		
By	David An	Date	Feb-05

Resultant Summary for Bottom of Wall

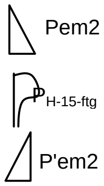
Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem1	1.5	2.67	4.1
$P_{H\text{-}15\ wall}$	1.7		
At bot of wall	**3.3**		**4.1**
	Σ V		Σ M



Demand at Bottom of Wall: Vd = 3.3 kips Md = 4.1 k-ft

Resultant Summary for Bottom of Footing

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem2	2.1	3.17	6.8
$P_{H\text{-}15\ ftg}$	1.7		
At bot of ftg	**3.9**		**6.8**
	Σ V		Σ M



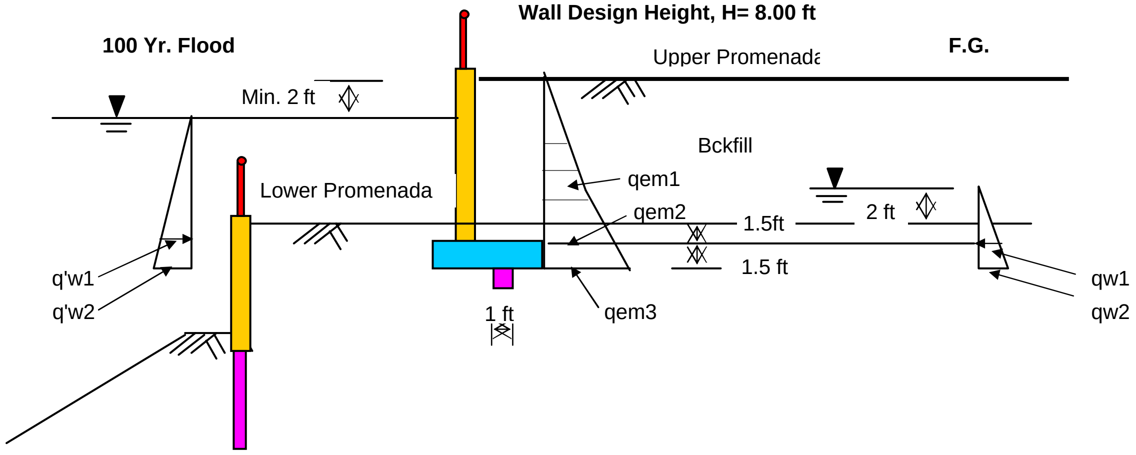
Demand at Bottom of Footing: Vd = 3.9 kips Md = 6.8 k-ft

MGE ENGINEERING, INC.

Project	Napa River Flood Control Project
Subject	Upper Wall Design (H=8')
By	David An
Date	Feb-05

Load Case 3a -- Design Flood Condition (Unusual Condition)

Wall Design Height, H= 8.00 ft



Soil Pressure at Wall (Soil pressure =γ Ki hi)

Note: Passive soil resistance were ignored.

Name	Layer	Thickness(ft)	Pressure(ksf)
qem1	Backfill	4.50	0.214
qem2	Backfill	3.50	0.379
qem3	Backfill	1.50	0.450
qw1	Backfill	3.50	0.002
qw2	Backfill	1.50	0.003
q'w1	Backfill	6.00	0.004
q'w2	Backfill	1.50	0.005

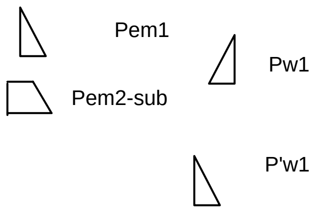
qem1 - Moist soil pressure at rest wall

qem2 & qem3 - Submerged soil pressure at rest wall

qw - Water pressure

Resultant Summary For Bottom Of Wall

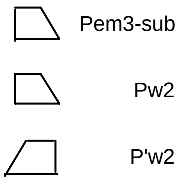
Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem1	0.5	5.00	2.41
Pem2	1.0	1.59	1.65
Pw1	0.004	1.17	0.005
P'w1	-0.01	0.50	-0.01
At bot of wall	**1.51**		**4.1**
	Σ V		Σ M



Demand at Bottom of Wall: Vd = 1.51 kips Md = 4.05 k-ft

Resultant Summary For Bottom Of Footing

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem1	0.5	6.50	3.13
Pem2	1.0	3.09	3.21
Pem3	0.6	0.73	0.45
Pw1	0.00	2.67	0.01
Pw2	0.0	0.71	0.00
P'w1	-0.01	2.00	0.0
P'w2	-0.01	0.72	0.00
At bot of ftg	**2.14**		**6.8**
	Σ V		Σ M



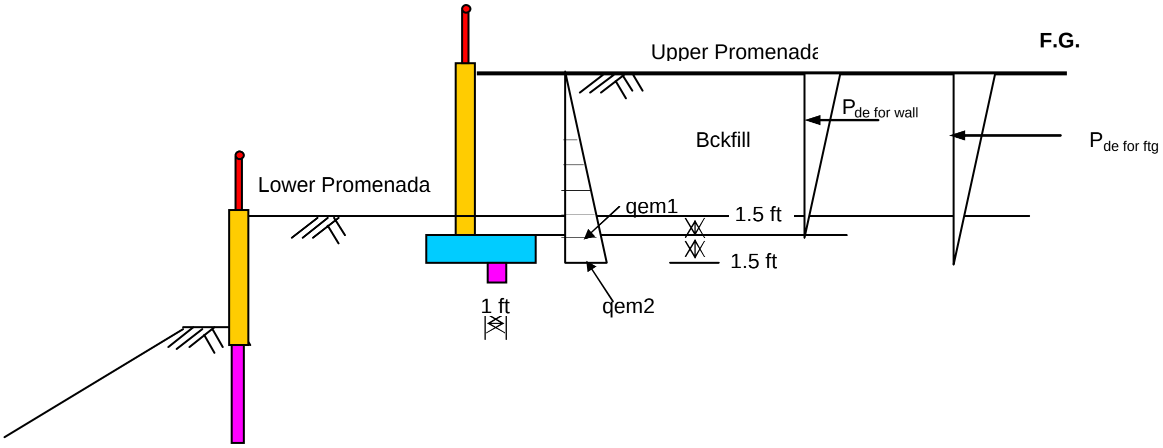
Demand at Bottom of Wall: Vd = 2.14 kips Md = 6.78 k-ft

MGE ENGINEERING, INC.

Project	Napa River Flood Control Project		
Subject	Upper Wall Design (H=8')		
By	David An	Date	Feb-05

Load Case 4 -- Seimic Condition (Extreme condition)

Wall Design Height, H= 8.00 ft



Soil Pressure (Soil pressure =γ Ki hi)

Name	Layer	Thickness(ft)	Pressure(ksf)
qem1	Backfill	8.00	0.380
qem2	Backfill	1.50	0.452
qde-wall	Backfill	8.00	0.115
qde-ftg	Backfill	1.50	0.022

Note: Passive soil resistance were ignored.

qem - Moist soil pressure at rest wall

qde - Earthquake component

qw - Water pressure

Resultant Summary For bottom of wall

Name	Force	Arm to bot.	Moments
Pem1	1.5	2.67	4.1
Pde-wall	0.5	5.33	2.4
At bot of wall	**2.0**		**6.5**
	Σ V		Σ M

Demand at Top of Pile: Vd = 1.98 kips Md = 6.50 k-ft

Pem1

Pde-wall $\alpha = \tan^{-1}[(C_1+\sqrt{}(C_1^2+4C_2))/2]$

(Equation 3-56 of EM 1110-2-2502, Page 3-67)

P'em1 $C_1 = 2\ (\tan\Phi - K_h)\ /\ (1+K_h \tan\Phi)$

(Equation 3-57 of EM 1110-2-2502, Page 3-67)

$C_2 = [\tan\Phi(1-\tan\Phi\ \ \tan\beta)-(\tan\beta-K_h)]\ /\ [\tan\Phi(1+K_h \tan\Phi)]$

(Equation 3-58 of EM 1110-2-2502, Page 3-67)

Resultant Summary For Bottom Of Ftg

Name	Force	Arm to bot.	Moments
Pem2	2.1	3.17	6.8
Pde-ftg	0.02	1.00	0.02
At bot of ftg	**2.2**		**6.8**
	Σ V		Σ M

Demand at Top of Pile: Vd = 2.16 kips Md = 6.81 k-ft

Pem2

Pde-ftg

K_h =	0.15	g
β =	0	
Φ =	30	degree
C_1 =	0.787	
C_2 =	0.681	
α =	52.6	degree

Dynamic Components $q_{eq} = \gamma K_h h^2 / [2(\tan\alpha - \tan\beta)]$

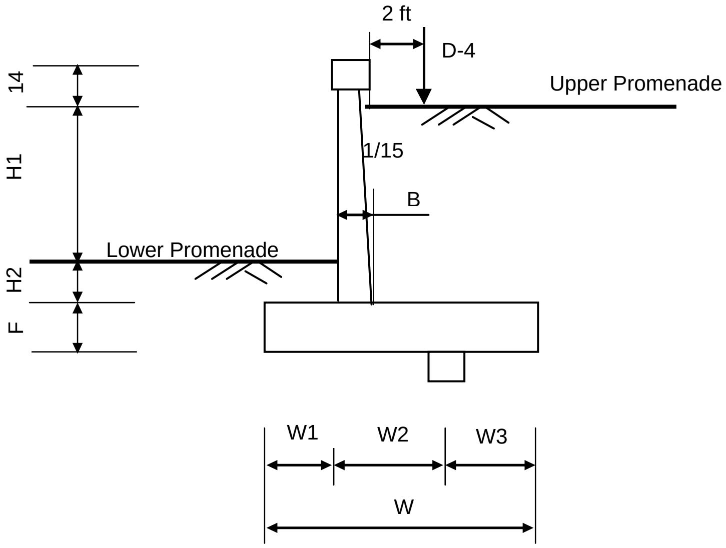
(Equation 3-62 of EM 1110-2-2502, Page 3-68)

Footing & Wall Design (Upper Wall)

Wall Design Height, H= 8.00 ft

1. Loads

Load Case	1		2		3		4	
	Bot. Of Wall	Bot. Of Ftg	Bot. Of Wall	Bot. Of Ftg	Bot. Of Wall	Bot. Of Ftg	Bot. Of Wall	Bot. Of Ftg
Hori. Force (k)	2.9	3.5	3.3	3.9	1.5	2.1	2.0	2.2
Moments(kft)	10.6	14.5	4.1	6.8	4.1	6.8	6.5	6.8



B =	1.43 ft		H1 =	6.50 ft
W1 =	2.75 ft		H2 =	1.50 ft
W2 =	3.00 ft		F =	1.5 ft
W3 =	1.75 ft			
W =	7.50 ft			

2. Resistances

Location	Dimension		Weight(k/ft)	Arm to toe (ft)	Mot (kft/ft)
	Thick/Depth	Width/Height			
Toe cover	1.50	2.75	0.515625	1.38	0.71
Heel Soil	8.00	3.32	3.317	5.84	19.37
Footing	1.50	7.50	1.688	3.75	6.33
Concrete Key	1.00	1.00	0.15	5.75	0.86
Wall Stem	1.43	9.17	1.971	3.47	6.83
Total			**7.64**		**34.11**

Due to D-4
Vertical weight

Distributed weight, Pd-4 = 2.50 k/ft
Resistance of overturning due to D-4 = distributed weight x (W1 + 1 + 2ft) 14.38 kft/ft

3. Overturning Check

Load Case	Resistance Moment	Overturning Moment	Safety Factor	
1	48.48	14.5	3.34	**Safety Factor > 1.5, OK**
2	34.11	6.8	5.02	**Safety Factor > 1.5, OK**
3	34.11	6.8	5.03	**Safety Factor > 1.5, OK**
4	34.11	6.8	5.01	**Safety Factor > 1.5, OK**

4. Sliding Check

Resistance Due to Soil Friction

μ = 0.3

Ffr = P_d x μ 2.3 kips/ft

where, P_d -- weight of concrete & soil

Resistance Due to D-4

Fd-4 = Pd_{-4} x μ 0.750 kips/ft

Pd_4--distributed weight due to D-4

Resistance Due to Concrete Key

Fcr = 2√f'c x 1 ft 18.21 kips/ft

where, f'c = 4000 psi

Load Case	Resistance Due to Soil (kips)	Sliding Force (kips)	Safety Factor		Resistance Due to Concrete Key (kips)	Sliding Force (kips)	Safety Factor	
1	2.29	3.5	0.65	**< 1.33, NG**	18.21	3.5	5.18	**> 1.33, OK**
2	2.29	3.9	0.59	**< 1.5, NG**	18.21	3.9	4.69	**> 1.5, OK**
3	2.29	2.1	1.07	**< 1.5, NG**	18.21	2.1	8.52	**> 1.5, OK**
4	2.29	2.2	1.06	**< 1.1, NG**	18.21	2.2	8.43	**> 1.1, OK**

5. Soil Bearing Pressure Check

Moment about center of footing

M = Msmax(due to soil pressure) + Mt(due to toe soil) - Mk(due to key) - Mw(due to wall) - Mh(due to heel soil) - M_{d4}(Due to D-4)

M = 4.2 kft/ft

where, Max. M (due to lateral soil pressure), Msmax. = 14.5 kft/ft

Msmax = (case1, case2, case3, case4)

Mt =H2 x W1 x 0.12 x (W/2-W1/2) 1.2 kft/ft

Mk = - (1ft x 1ft x 0.15 x (W/2-W3)) -0.30 kft/ft

Mw = B x (H1 + H2 + 14/12) x 0.15 x (W/2-B/2-W1) 0.56 kft/ft

Mh =- (H1 + H2)x0.12x[W2 + W3 - (B+1)/2]x{W/2-[W2+W3-(B+1)/2]/2} -6.73 kft/ft

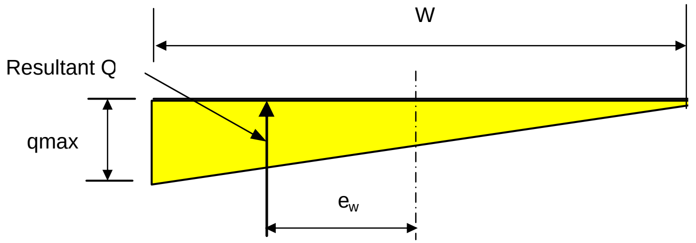
M_{D-4} = - P_{D-4} x (W/2 - W1-1ft - 2) -5.00 kft/ft

Eccentricity e_W = M / (Pd+Pd-4) 0.42 ft

W / 6 = 1.25 ft

Therefor e_W < W / 6

Footing Contact Pressure



Q = Pd + Pd-4 10.14 kips

1 + (6 e_w/ W) = 1.33 ft

1 - (6 e_w/ W) = 0.67 ft

qmax = Q [1 + (6ew / W)] / W 1.80 ksf

qmin = Q [1 - (6ew / W)] / W 0.90 ksf

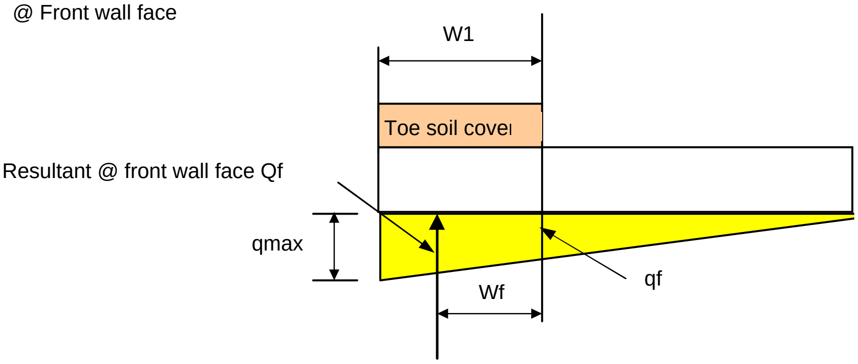
assumed allowable soil brearing pressure

qa = controlling 2.00 ksf

D / C = qmax / qa = 0.90 **<1, OK**

6. Footing Shear and Moment Check (@ face of wall)

@ Front wall face



qf = (W2+W3) / W x (qmax - qmin) + qmin 1.47 ksf

Qf =(qf+qmax)/2 x W1 4.50 k/ft

Arm to face of wall

Wf = W1 (2qmax+qf) / [3 (qmax+qf)] 1.42 ft

Mu = Hf{1.7 [Qf x Wf - (W1x F x 0.15 x W1/2 -W1 x h2 x 0.12 x W1/2)]} 10.76 kft/ft

Mu = Hf[1.7(D+L)]

Hf = Hydraulic factor 1.30

EM-1110-2-2104---P3-2, Equation3.3

Vu = Hf{1.7[Qf - (W1 x F x 0.15 - W1 x 1ft x 0.12)]} 7.49 k/ft

Required bar area for flexure

Mu ≤ ΦMn

ΦMn = Φ As x fy (d - a/2)

Assume a = 0.15d 2.25 in

where d = 15 in

As = Mu / [Φfy(d-a/2)] 0.17 in^2

Φ = 0.9

fy = 60 ksi

Use #4@12", As = 0.20 in^2 As = 0.20 in^2

Check

a =Asfy / (0.85f'c*b) 0.3 in

ΦMn = Φ Asfy(d - a/2) 13.37 kft/ft

D/C = Mu/ΦMn 0.81 **OK**

Shear Check

Shear capacity

ΦVn = Φ2√f'c x F 23.22 k/ft

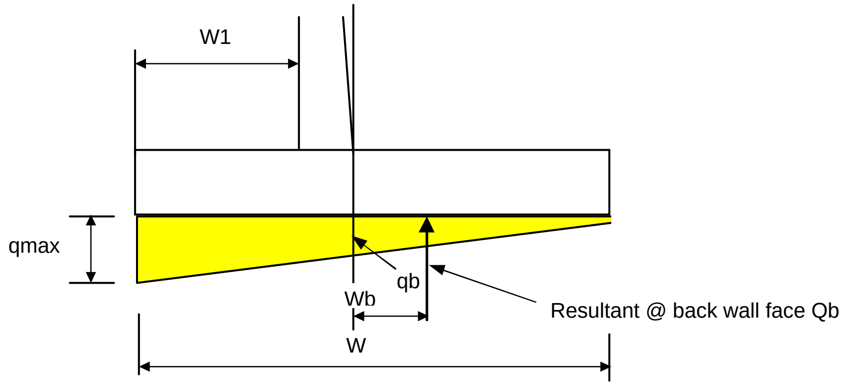
where Φ = 0.85

D/C = Vu/ΦVn 0.32 **OK**

MGE ENGINEERING, INC.

Project	Napa River Flood Control Project		
Subject	Upper Wall Design (H=8')		
By	David An	Date	Feb-05

@ Back wall face



qb = (W-W1-B) / W x qmax + qmin 1.30 ksf

Qb =(qf+qmin)/2 x (W-W1-B) 3.65 k/ft

Arm to face of wall

Wb =(W-W1-B) - (W- W1-B) (2qb+qmin) / [3 (qb+qmin)] 1.56 ft

Mu = Hf{1.7[Qb x Wb - (F x 0.15 x (W-W1-B)^2/2 + (H1+H2) x 0.12 x (W-W1-B)/2)-(Pd-4 x 2ft)]

-12.89 kft/ft

Mu = Hf[1.7(D+L)]

Hf = Hydraulic factor 1.30

EM-1110-2-2104---P3-2, Equation3.3

Vu = Hf x 1.7[Qb - ((W-W1-B) x F x 0.15 + (W-W1-B) x H2 x 0.12) - Ph15] -6.15 k/ft

Required bar area for flexure

Mu ≤ ΦMn

ΦMn = Φ As x fy (d - a/2)

Assume a = 0.15d 2.25 in

where d = 15 in

As = Mu / [Φfy(d-a/2)] 0.21 in^2

Φ = 0.9

fy = 60 ksi

Use #5@12", As = 0.31 in^2 As = 0.31 in^2

Check

a =Asfy / (0.85f'c*b) 0.5 in

ΦMn = Φ Asfy(d - a/2) 20.61 kft/ft

D/C = Mu/ΦMn 0.63 **OK**

Shear Check

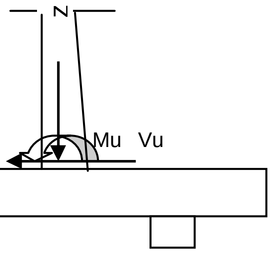
Shear capacity

ΦVn = Φ2√f'c x F 23.22 k/ft

where Φ = 0.85

D/C = Vu/ΦVn 0.26 **OK**

7. Wall Stem Shear and Moment Check (@ bottom of wall)



MGE ENGINEERING, INC.

Project: Napa River Flood Control Project
Subject: Upper Wall Design (H=8')
By: David An
Date: Feb-05

Mu = Hf x 1.7 x M			23.4	kft/ft
Where	M = max(case1, case2, case3, case4)			
	Mu = Hf[1.7(D+L)]			
	Hf = Hydraulic factor		1.30	
	EM-1110-2-2104---P3-2, Equation3.3			
Vu = Hf x 1.7 x Vmax	Vmax = max(case1, case2, case3, case4)		7.2	k/ft
Required bar area for flexure				
	Mu ≤ ΦMn			
	ΦMn = Φ As x fy (d - a/2)			
	Assume a = 0.15d		2.13	in
	where	d =	14.2	in
	As = Mu / [Φfy(d-a/2)]		0.40	in^2
		Φ =	0.9	
		fy =	60	ksi
		f'c =	4000	psi
	Use #6@12", As = 0.44 in^2	As =	0.44	in^2
Check				
	a =Asfy / (0.85f'c*b)		0.6	in
	ΦMn = Φ Asfy(d - a/2)		27.48	kft/ft
	D/C = Mu/ΦMn		0.85	**OK**
Shear Check				
	Shear capacity			
	ΦVn = Φ2√f'c x B		22.19	k/ft
	where	Φ =	0.85	
	D/C = Vu/ΦVn		0.32	**OK**

MGE ENGINEERING, INC.	Project	Napa River Flood Control Project	
	Subject	Upper Wall Design (H=10')	
	By David An	Date Feb-05	

Upper Wall Design

Design Height H= 10.00 ft

Backfill Properties

Backfill Thickness = 10.00 ft

Backfill Unit Weight = 125 pcf

Φ = 37 degree

C = 0 pcf

SMF= Tan(Φ_d) / TanΦ = 2/3 = 0.67

Φ_d = 27 degree

Ka = Tan^2 (45° - Φ/2) = 0.25

Ko = Tan^2 (45° - Φ_d/2) = 0.38

Kp = Tan^2 (45° + Φ/2) = 4.02

Water Property

Water Unit Weight = 62.5 pcf

Wall and Footing Data

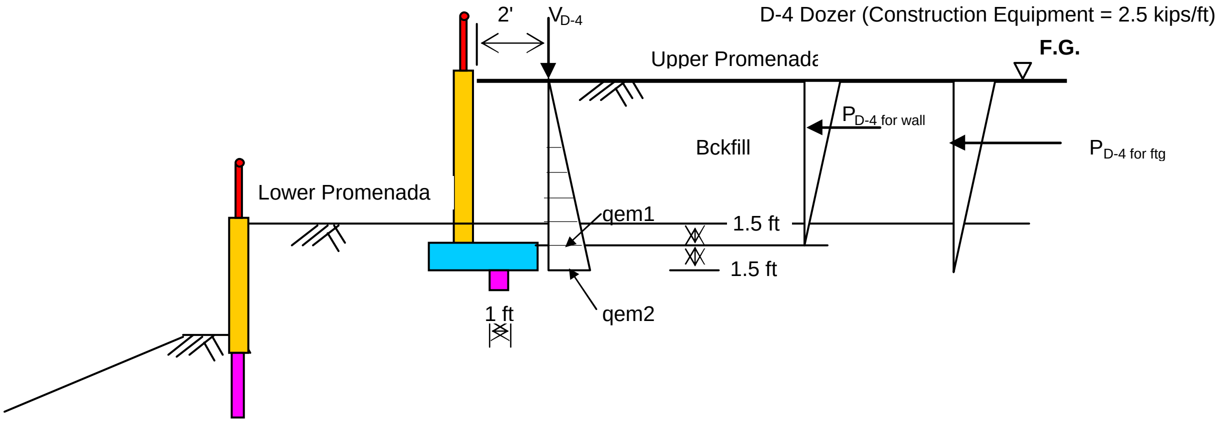
Design Height (ft)	10
Toe Cover (ft)	1.5
Wall Thick (ft)	1.0
100 Year Flood Level to Top of Wall (ft)	2.00

Upper Wall Design (H=10.00 ft)

Load case based on DOA--Flood Walls and Channel Retaining Walls, Load Conditions & Load Diagrams

Load Case 1 -- Construction Condition (Unusal Condition)

Wall Design Height, H= 10.00 ft



Soil Pressure at Wall (Soil pressure =γ Ki hi)

Note: Passive soil resistance were ignored.

Name	Layer	Thickness(ft)	Pressure(ksf)
qem1	Backfill	10.00	0.475
qem2	Backfill	1.50	0.547

qem - Moist soil pressure at rest wall

D-4 Load---- **For bottom of wall**

h = 10.00 ft

a = 2' / 10.0' = 0.20 ≤ 0.4

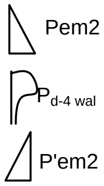
V_{D-4} = 2.5 kips/ft

$\Delta P_{D-4} = (V_{D-4} /h)$ [(0.203b)/(0.16+b^2)^2] (EM 1110-2-2502 Page 3-49)

b	Z = bh	$\Delta P_{D-4\ wal}$	Moment
0.0	0.00	0.000	0.000
0.1	1.00	0.176	1.580
0.2	2.00	0.254	2.030
0.3	3.00	0.244	1.705
0.4	4.00	0.198	1.189
0.5	5.00	0.151	0.755
0.6	6.00	0.113	0.450
0.7	7.00	0.084	0.252
0.8	8.00	0.063	0.127
0.9	9.00	0.049	0.049
1.0	10.00	0.038	0.000
Σ		1.369	8.138

Resultant Summary for Bottom of Wall

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem1	2.4	3.33	7.9
Pd-4 wall	1.4		8.1
At bot of wall	**3.7**		**16.1**
	Σ V		Σ M



Demand at Bottom of Wall: Vd = 3.7 kips Md = 16.1 k-ft

D-4 Load---- **For bottom of footing**

h = 11.50 ft

a = 2' / 11.5' = 0.17 ≤ 0.4

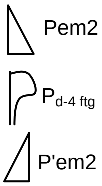
V_{D-4} = 2.5 kips/ft

$\Delta P_{D-4} = (V_{D-4} /h)$ [(0.203b)/(0.16+b^2)^2] (EM 1110-2-2502 Page 3-49)

b	Z = bh	$\Delta P_{D-4\ wall}$	Moment
0.0	0.00	0.000	0.000
0.1	1.15	0.176	1.818
0.2	2.30	0.254	2.335
0.3	3.45	0.244	1.961
0.4	4.60	0.198	1.368
0.5	5.75	0.151	0.868
0.6	6.90	0.113	0.518
0.7	8.05	0.084	0.290
0.8	9.20	0.063	0.146
0.9	10.35	0.049	0.056
1.0	11.50	0.038	0.000
Σ		1.369	9.359

Resultant Summary for Bottom of Footing

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem2	3.1	3.83	12.1
Pd-4 ftg	1.4		9.4
At bot of ftg	**4.5**		**21.4**
	Σ V		Σ M



Demand at Bottom of Footing: Vd = 4.5 kips Md = 21.4 k-ft

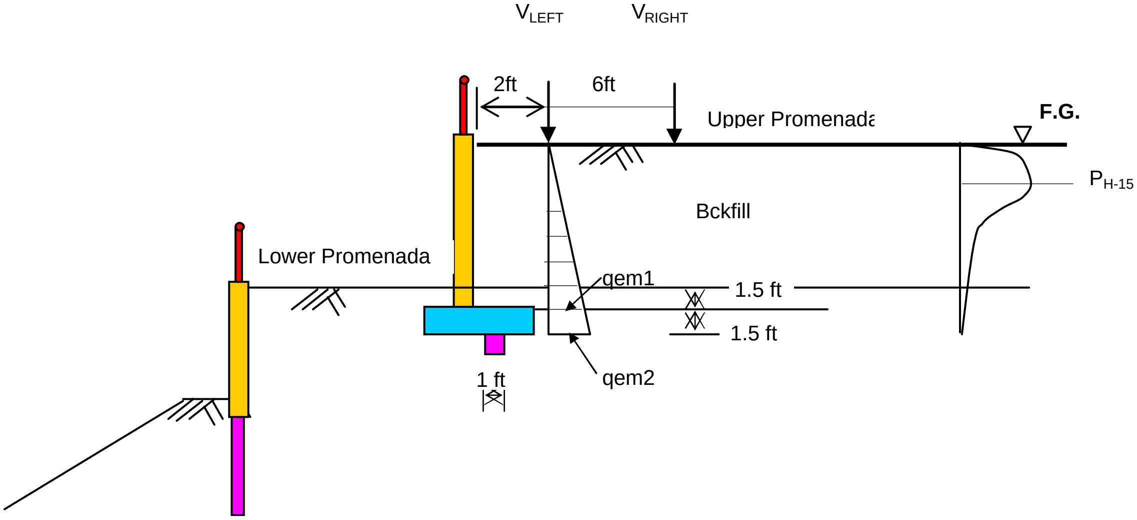
MGE ENGINEERING, INC.

Project	Napa River Flood Control Project
Subject	Upper Wall Design (H=10')
By	David An
Date	Feb-05

Load Case 2 -- Normal Condition W/Maintenance Vehicle (Usual Condition)

Wall Design Height, H= 10.00 ft

H-15 truck load **USE V= 16 kips**



Soil Pressure at Wall (Soil pressure =γ Ki hi) **Note:** Passive soil resistance were ignored.

Name	Layer	Thickness(ft)	Pressure(ksf)
qem1	Backfill	10.00	0.475
qem2	Backfill	1.50	0.547

qem - Moist soil pressure at rest wall

H-15 Load---- For bottom of footing

h = 10.00 ft

a=2'/10.00' = 0.20 ≤ 0.4 **For V_{LEFT}**

a=8'/10.00' = 0.80 > 0.4 **For V_{RIGHT}**

$\Delta P_{HZleft} = (0.28V/h^2)\ [b^2 / (0.16+b^2)^3]$ (EM 1110-2-2502 Page 3-48)

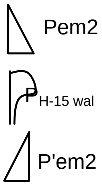
$\Delta P_{HZright} = (V/h^2)\ [a^2 b^2 / (a^2+b^2)^3]$ (EM 1110-2-2502 Page 3-48)

b (for V_{LEFT})	Z	$\Delta P_{PH\ (LEFT)}$	Moment
0.1	1.00	0.091	0.821
0.2	2.00	0.224	1.792
0.3	3.00	0.258	1.806
0.4	4.00	0.219	1.313
0.5	5.00	0.163	0.813
0.6	6.00	0.115	0.459
0.7	7.00	0.080	0.240
0.8	8.00	0.056	0.112
0.9	9.00	0.040	0.040
1.0	10.00	0.029	0.000
Σ		1.274	7.394

b (for V_{RIGHT})	Z	$\Delta P_{PH\ (RIGHT)}$	Moment
0.1	1.00	0.001	0.008
0.2	2.00	0.003	0.025
0.3	3.00	0.006	0.044
0.4	4.00	0.010	0.059
0.5	5.00	0.013	0.065
0.6	6.00	0.015	0.061
0.7	7.00	0.017	0.050
0.8	8.00	0.017	0.035
0.9	9.00	0.017	0.017
1.0	10.00	0.016	0.000
Σ		0.116	0.363

Resultant Summary for Bottom of Wall

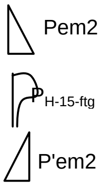
Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem1	2.4	3.33	7.9
$P_{H\text{-}15\ wall}$	1.4		
At bot of wall	**3.8**		**7.9**
	Σ V		Σ M



Demand at Bottom of Wall: Vd = 3.8 kips Md = 7.9 k-ft

Resultant Summary for Bottom of Footing

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem2	3.1	3.83	12.1
$P_{H\text{-}15\ ftg}$	1.4		
At bot of ftg	**4.5**		**12.1**
	Σ V		Σ M



Demand at Bottom of Footing: Vd = 4.5 kips Md = 12.1 k-ft

MGE ENGINEERING, INC.

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Subject Upper Wall Design (H=10')
By David An
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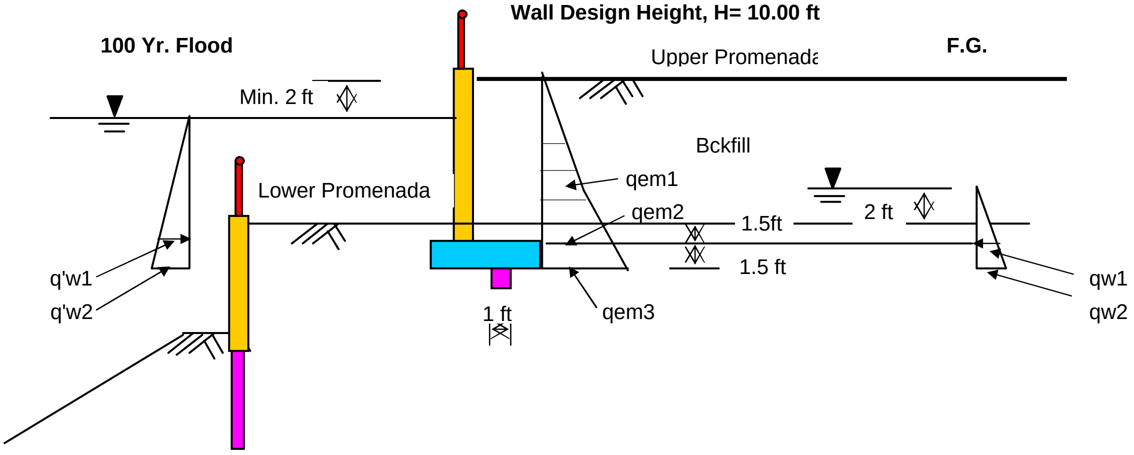
Load Case 3a -- Design Flood Condition (Unusual Condition)

Wall Design Height, H= 10.00 ft



Soil Pressure at Wall (Soil pressure =γ Ki hi)

Note: Passive soil resistance were ignored.

Name	Layer	Thickness(ft)	Pressure(ksf)
qem1	Backfill	6.50	0.309
qem2	Backfill	3.50	0.475
qem3	Backfill	1.50	0.545
qw1	Backfill	3.50	0.002
qw2	Backfill	1.50	0.003
q'w1	Backfill	8.00	0.005
q'w2	Backfill	1.50	0.006

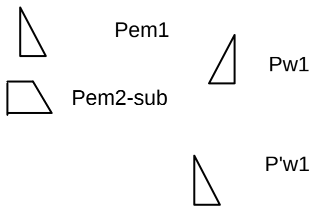
qem1 - Moist soil pressure at rest wall

qem2 & qem3 - Submerged soil pressure at rest wall

qw - Water pressure

Resultant Summary For Bottom Of Wall

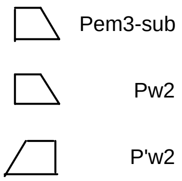
Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem1	1.0	5.67	5.69
Pem2	1.4	1.63	2.23
Pw1	0.004	1.17	0.005
P'w1	-0.02	0.50	-0.01
At bot of wall	**2.36**		**7.9**
	Σ V		Σ M



Demand at Bottom of Wall: Vd = 2.36 kips Md = 7.92 k-ft

Resultant Summary For Bottom Of Footing

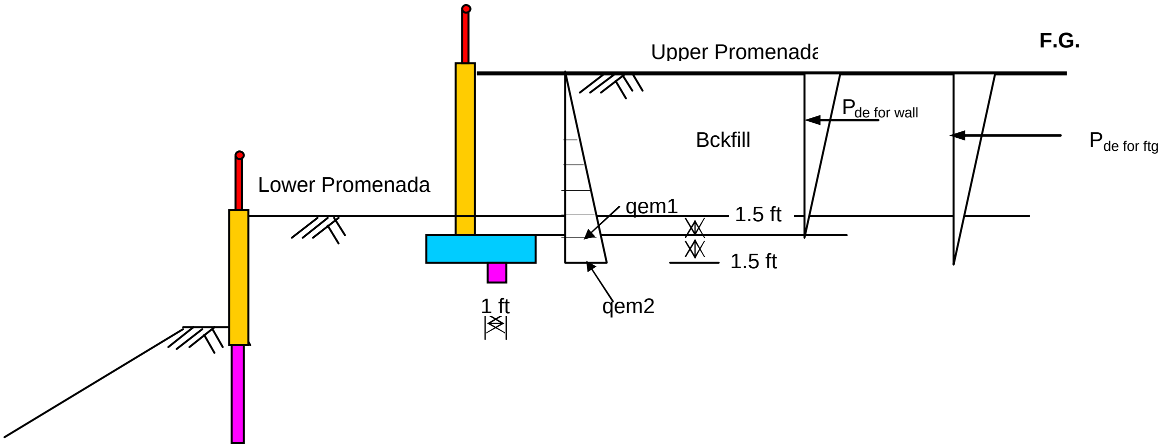
Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem1	1.0	7.17	7.20
Pem2	1.4	3.13	4.29
Pem3	0.8	0.73	0.56
Pw1	0.00	2.67	0.01
Pw2	0.0	0.71	0.00
P'w1	-0.02	2.67	-0.1
P'w2	-0.01	0.73	-0.01
At bot of ftg	**3.13**		**12.0**
	Σ V		Σ M



Demand at Bottom of Wall: Vd = 3.13 kips Md = 12. k-ft

Load Case 4 -- Seimic Condition (Extreme condition)

Wall Design Height, H= 10.00 ft



Soil Pressure (Soil pressure =γ Ki hi)

Name	Layer	Thickness(ft)	Pressure(ksf)
qem1	Backfill	10.00	0.475
qem2	Backfill	1.50	0.547
qde-wall	Backfill	10.00	0.143
qde-ftg	Backfill	1.50	0.022

Note: Passive soil resistance were ignored.

qem - Moist soil pressure at rest wall

qde - Earthquake component

qw - Water pressure

Resultant Summary For bottom of wall

Name	Force	Arm to bot.	Moments
Pem1	2.4	3.33	7.9
Pde-wall	0.7	6.67	4.8
At bot of wall	**3.1**		**12.7**
	Σ V		Σ M

Demand at Top of Pile: Vd = 3.09 kips Md = 12.70 k-ft

Pem1

Pde-wall $\alpha = \tan^{-1}[(C_1+\sqrt{}(C_1^2+4C_2))/2]$

(Equation 3-56 of EM 1110-2-2502, Page 3-67)

P'em1 $C_1 = 2\ (\tan\Phi - K_h)\ /\ (1+K_h \tan\Phi)$

(Equation 3-57 of EM 1110-2-2502, Page 3-67)

$C_2 = [\tan\Phi(1-\tan\Phi\ \ \tan\beta)-(\tan\beta-K_h)]\ /\ [\tan\Phi(1+K_h \tan\Phi)]$

(Equation 3-58 of EM 1110-2-2502, Page 3-67)

K_h = 0.15 g
β = 0
Φ = 30 degree
C_1 = 0.787
C_2 = 0.681
α = 52.6 degree

Resultant Summary For Bottom Of Ftg

Name	Force	Arm to bot.	Moments
Pem2	3.1	3.83	12.1
Pde-ftg	0.02	1.00	0.02
At bot of ftg	**3.2**		**12.1**
	Σ V		Σ M

Demand at Top of Pile: Vd = 3.16 kips Md = 12.07 k-ft

Pem2

Pde-ftg

Dynamic Components qeq =$\gamma\ K_h h^2\ /\ [2(\tan\alpha-\tan\beta)]$

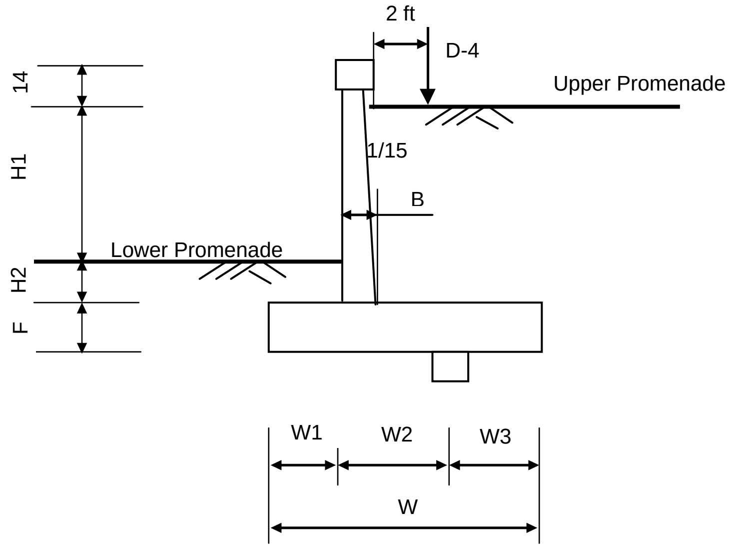
(Equation 3-62 of EM 1110-2-2502, Page 3-68)

Footing & Wall Design (Upper Wall)

Wall Design Height, H= 10.00 ft

1. Loads

Load Case	1		2		3		4	
	Bot. Of Wall	Bot. Of Ftg	Bot. Of Wall	Bot. Of Ftg	Bot. Of Wall	Bot. Of Ftg	Bot. Of Wall	Bot. Of Ftg
Hori. Force (k)	3.7	4.5	3.8	4.5	2.4	3.1	3.1	3.2
Moments(kft)	16.1	21.4	7.9	12.1	7.9	12.0	12.7	12.1



B =	1.57 ft		H1 =	8.50 ft
W1 =	3.50 ft		H2 =	1.50 ft
W2 =	4.00 ft		F =	1.5 ft
W3 =	2.00 ft			
W =	9.50 ft			

2. Resistances

Location	Dimension		Weight(k/ft)	Arm to toe (ft)	Mot (kft/ft)
	Thick/Depth	Width/Height			
Toe cover	1.50	3.50	0.65625	1.75	1.15
Heel Soil	10.00	4.43	5.542	7.28	40.36
Footing	1.50	9.50	2.138	4.75	10.15
Concrete Key	1.00	1.00	0.15	7.50	1.13
Wall Stem	1.57	11.17	2.624	4.28	11.24
Total			**11.11**		**64.03**

Due to D-4

Vertical weight

Distributed weight, Pd-4 = 2.50 k/ft

Resistance of overturning due to D-4 = distributed weight x (W1 + 1 + 2ft) 16.25 kft/ft

MGE ENGINEERING, INC.

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3. Overturning Check

Load Case	Resistance Moment	Overturning Moment	Safety Factor	
1	80.28	21.4	3.75	**Safety Factor > 1.5, OK**
2	64.03	12.1	5.31	**Safety Factor > 1.5, OK**
3	64.03	12.0	5.33	**Safety Factor > 1.5, OK**
4	64.03	12.1	5.31	**Safety Factor > 1.5, OK**

4. Sliding Check

Resistance Due to Soil Friction

μ = 0.3

Ffr = P_d x μ 3.3 kips/ft

where, P_d -- weight of concrete & soil

Resistance Due to D-4

Fd-4 = Pd_{-4} x μ 0.750 kips/ft

Pd_4--distributed weight due to D-4

Resistance Due to Concrete Key

Fcr = 2√f'c x 1 ft 18.21 kips/ft

where, f'c = 4000 psi

Load Case	Resistance Due to Soil (kips)	Sliding Force (kips)	Safety Factor		Resistance Due to Concrete Key (kips)	Sliding Force (kips)	Safety Factor	
1	3.33	4.5	0.74	**< 1.33, NG**	18.21	4.5	4.04	**> 1.33, OK**
2	3.33	4.5	0.74	**< 1.5, NG**	18.21	4.5	4.02	**> 1.5, OK**
3	3.33	3.1	1.07	**< 1.5, NG**	18.21	3.1	5.82	**> 1.5, OK**
4	3.33	3.2	1.05	**< 1.1, NG**	18.21	3.2	5.76	**> 1.1, OK**

5. Soil Bearing Pressure Check

Moment about center of footing

M = Msmax(due to soil pressure) + Mt(due to toe soil) - Mk(due to key) - Mw(due to wall) - Mh(due to heel soil) - M_{d4}(Due to D-4)

M = 6.2 kft/ft

where, Max. M (due to lateral soil pressure), Msmax. = 21.4 kft/ft

Msmax = (case1, case2, case3, case4)

Mt =H2 x W1 x 0.12 x (W/2-W1/2) 1.9 kft/ft

Mk = - (1ft x 1ft x 0.15 x (W/2-W3)) -0.41 kft/ft

Mw = B x (H1 + H2 + 14/12) x 0.15 x (W/2-B/2-W1) 1.22 kft/ft

Mh =- (H1 + H2)x0.12x[W2 + W3 - (B+1)/2]x{W/2-[W2+W3-(B+1)/2]/2} -13.54 kft/ft

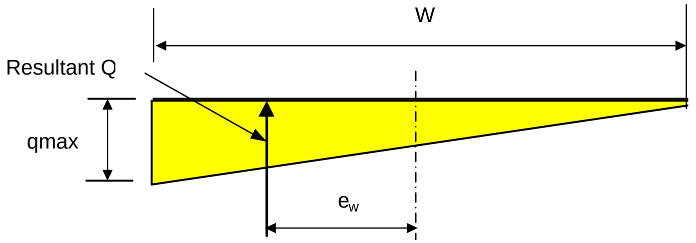
M_{D-4} = - P_{D-4} x (W/2 - W1-1ft - 2) -4.38 kft/ft

Eccentricity e_W = M / (Pd+Pd-4) 0.46 ft

W / 6 = 1.58 ft

Therefor e_W < W / 6

Footing Contact Pressure



MGE Engineering, Inc.

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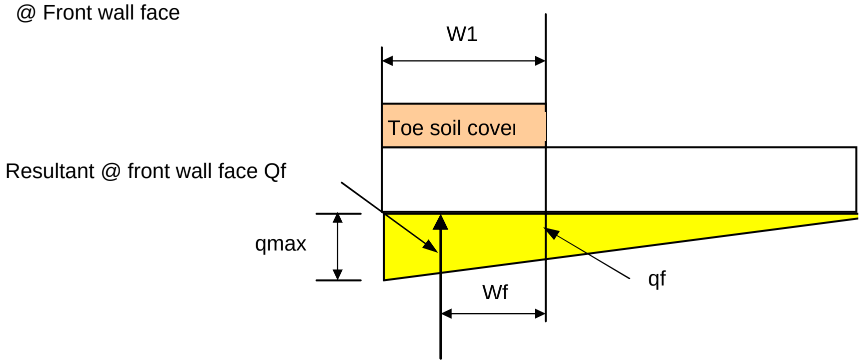
By	David An	Date	Feb-05

Q = Pd + Pd-4		13.61	kips
$1 + (6\ e_w / W) =$		1.29	ft
$1 - (6\ e_w / W) =$		0.71	ft
qmax = Q [1 + (6ew / W)] / W		1.84	ksf
qmin = Q [1 - (6ew / W)] / W		1.02	ksf
assumed allowable soil brearing pressure			
qa =	controlling	2.00	ksf
D / C = qmax / qa =	0.92	**<1, OK**	

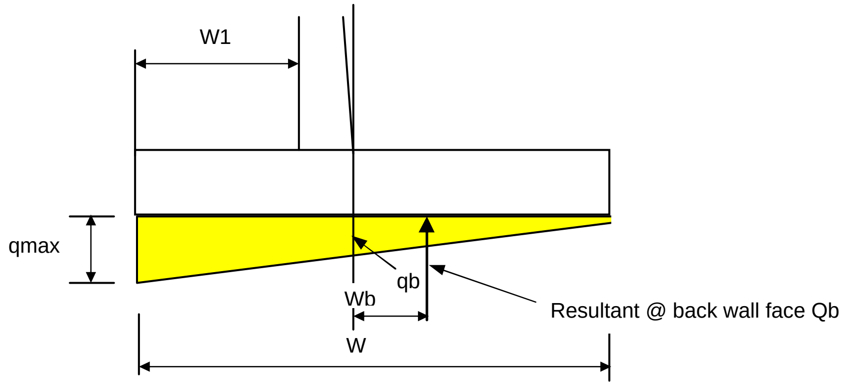
6. Footing Shear and Moment Check (@ face of wall)

@ Front wall face



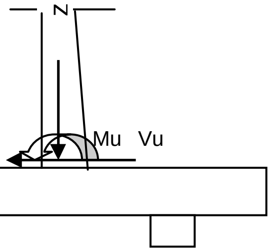
qf = (W2+W3) / W x (qmax - qmin) + qmin		1.54	ksf
Qf =(qf+qmax)/2 x W1		5.93	k/ft
Arm to face of wall			
Wf = W1 (2qmax+qf) / [3 (qmax+qf)]		1.80	ft
Mu = Hf{1.7 [Qf x Wf - (W1x F x 0.15 x W1/2 -W1 x h2 x 0.12 x W1/2)]}		18.12	kft/ft
Mu = Hf[1.7(D+L)]			
Hf = Hydraulic factor		1.30	
EM-1110-2-2104---P3-2, Equation3.3			
Vu = Hf{1.7[Qf - (W1 x F x 0.15 - W1 x 1ft x 0.12)]}		9.96	k/ft
Required bar area for flexure			
Mu ≤ ΦMn			
ΦMn = Φ As x fy (d - a/2)			
Assume a = 0.15d		2.25	in
where	d =	15	in
As = Mu / [Φfy(d-a/2)]		0.29	in^2
	Φ =	0.9	
	fy =	60	ksi
Use #5@12", As = 0.31 in^2	As =	0.31	in^2
Check			
a =Asfy / (0.85f'c*b)		0.5	in
ΦMn = Φ Asfy(d - a/2)		20.61	kft/ft
D/C = Mu/ΦMn		0.88	**OK**
Shear Check			
Shear capacity			
ΦVn = Φ2√f'c x F		23.22	k/ft
where	Φ =	0.85	
D/C = Vu/ΦVn		0.43	**OK**

@ Back wall face



qb = (W-W1-B) / W x qmax + qmin		1.41	ksf
Qb =(qf+qmin)/2 x (W-W1-B)		5.38	k/ft
Arm to face of wall			
Wb =(W-W1-B) - (W- W1-B) (2qb+qmin) / [3 (qb+qmin)]		2.10	ft
Mu = Hf{1.7[Qb x Wb - (F x 0.15 x (W-W1-B)^2/2 + (H1+H2) x 0.12 x (W-W1-B)/2)-(Pd-4 x 2ft)]		-17.05	kft/ft
Mu = Hf[1.7(D+L)]			
Hf = Hydraulic factor		1.30	
EM-1110-2-2104---P3-2, Equation3.3			
Vu = Hf x 1.7[Qb - ((W-W1-B) x F x 0.15 + (W-W1-B) x H2 x 0.12) - Ph15]		-7.60	k/ft
Required bar area for flexure			
Mu ≤ ΦMn			
ΦMn = Φ As x fy (d - a/2)			
Assume a = 0.15d		2.25	in
where	d =	15	in
As = Mu / [Φfy(d-a/2)]		0.27	in^2
	Φ =	0.9	
	fy =	60	ksi
Use #5@12", As = 0.31 in^2	As =	0.31	in^2
Check			
a =Asfy / (0.85f'c*b)		0.5	in
ΦMn = Φ Asfy(d - a/2)		20.61	kft/ft
D/C = Mu/ΦMn		0.83	**OK**
Shear Check			
Shear capacity			
ΦVn = Φ2√f'c x F		23.22	k/ft
where	Φ =	0.85	
D/C = Vu/ΦVn		0.33	**OK**

7. Wall Stem Shear and Moment Check (@ bottom of wall)



MGE ENGINEERING, INC.	Project	Napa River Flood Control Project	
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	By	David An	Date Feb-05

Mu = Hf x 1.7 x M 35.5 kft/ft

Where M = max(case1, case2, case3, case4)

Mu = Hf[1.7(D+L)]

Hf = Hydraulic factor 1.30

EM-1110-2-2104---P3-2, Equation3.3

Vu = Hf x 1.7 x Vmax Vmax = max(case1, case2, case3, case4) 8.3 k/ft

Required bar area for flexure

Mu ≤ ΦMn

ΦMn = Φ As x fy (d - a/2)

Assume a = 0.15d 2.37 in

where d = 15.8 in

As = Mu / [Φfy(d-a/2)] 0.54 in^2

Φ = 0.9

fy = 60 ksi

f'c = 4000 psi

Use #6@9", As = 0.44 in^2 x 12/9 As = 0.59 in^2

Check

a =Asfy / (0.85f'c*b) 0.9 in

ΦMn = Φ Asfy(d - a/2) 40.57 kft/ft

D/C = Mu/ΦMn 0.87 **OK**

Shear Check

Shear capacity

ΦVn = Φ2√f'c x B 24.26 k/ft

where Φ = 0.85

D/C = Vu/ΦVn 0.34 **OK**

MGE ENGINEERING, INC.

Project Napa River Flood Control Project

Subject Upper Wall Design (H=12')

By David An

Date Feb-05

Upper Wall Design

Design Height H= 12.00 ft

Backfill Properties

Backfill Thickness = 12.00 ft

Backfill Unit Weight = 125 pcf

Φ = 37 degree

C = 0 pcf

SMF= $\tan(\Phi_d) / \tan\Phi = 2/3$ = 0.67

Φ_d = 27 degree

Ka = $\tan^2 (45° - \Phi/2)$ = 0.25

Ko = $\tan^2 (45° - \Phi_d/2)$ = 0.38

Kp = $\tan^2 (45° + \Phi/2)$ = 4.02

Water Property

Water Unit Weight = 62.5 pcf

Wall and Footing Data

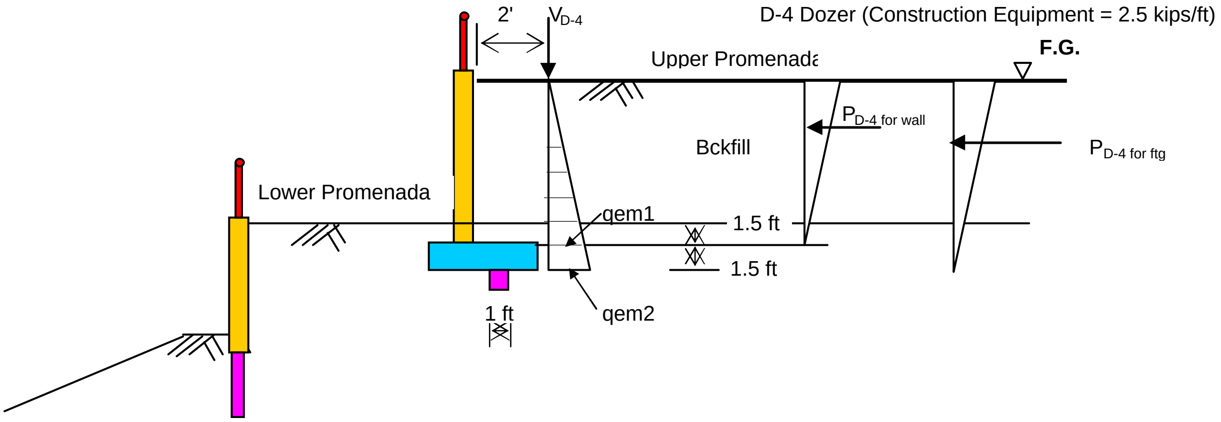
Design Height (ft)	12
Toe Cover (ft)	1.5
Top Wall Thick (ft)	1.0
100 Year Flood Level to Top of Wall (ft)	2.00

Upper Wall Design (H=12.00 ft)

Load case based on DOA--Flood Walls and Channel Retaining Walls, Load Conditions & Load Diagrams

Load Case 1 -- Construction Condition (Unusal Condition)

Wall Design Height, H= 12.00 ft



Soil Pressure at Wall (Soil pressure =γ Ki hi)

Note: Passive soil resistance were ignored.

Name	Layer	Thickness(ft)	Pressure(ksf)
qem1	Backfill	12.00	0.571
qem2	Backfill	1.50	0.642

qem - Moist soil pressure at rest wall

D-4 Load---- **For bottom of wall**

h = 12.00 ft

a = 2' / 12.0' = 0.17 ≤ 0.4

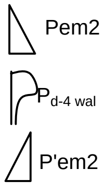
V_{D-4} = 2.5 kips/ft

$\Delta P_{D-4} = (V_{D-4} /h)\ [(0.203b)/(0.16+b^2)^2]$ (EM 1110-2-2502 Page 3-49)

b	Z = bh	$\Delta P_{D-4\ wal}$	Moment
0.0	0.00	0.000	0.000
0.1	1.20	0.176	1.897
0.2	2.40	0.254	2.436
0.3	3.60	0.244	2.046
0.4	4.80	0.198	1.427
0.5	6.00	0.151	0.906
0.6	7.20	0.113	0.541
0.7	8.40	0.084	0.303
0.8	9.60	0.063	0.152
0.9	10.80	0.049	0.058
1.0	12.00	0.038	0.000
Σ		1.369	9.766

Resultant Summary for Bottom of Wall

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem1	3.4	4.00	13.7
Pd-4 wall	1.4		9.8
At bot of wall	**4.8**		**23.5**
	Σ V		Σ M



Demand at Bottom of Wall: Vd = 4.8 kips Md = 23.5 k-ft

MGE Engineering, Inc.

Project: Napa River Flood Control Project
Subject: Upper Wall Design (H=12')
By: David An
Date: Feb-05

D-4 Load---- **For bottom of footing**

h = 13.50 ft

a = 2' / 13.5' = 0.15 ≤ 0.4

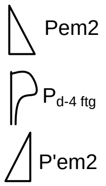
V_{D-4} = 2.5 kips/ft

$\Delta P_{D-4} = (V_{D-4}/h)\,[(0.203b)/(0.16+b^2)^2]$ (EM 1110-2-2502 Page 3-49)

b	Z = bh	$\Delta P_{D-4\ wall}$	Moment
0.0	0.00	0.000	0.000
0.1	1.35	0.176	2.134
0.2	2.70	0.254	2.741
0.3	4.05	0.244	2.302
0.4	5.40	0.198	1.606
0.5	6.75	0.151	1.019
0.6	8.10	0.113	0.608
0.7	9.45	0.084	0.341
0.8	10.80	0.063	0.171
0.9	12.15	0.049	0.066
1.0	13.50	0.038	0.000
Σ		1.369	10.986

Resultant Summary for Bottom of Footing

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem2	4.3	4.50	19.5
Pd-4 ftg	1.4		11.0
At bot of ftg	**5.7**		**30.5**
	Σ V		Σ M



Demand at Bottom of Footing: Vd = 5.7 kips Md = 30.5 k-ft

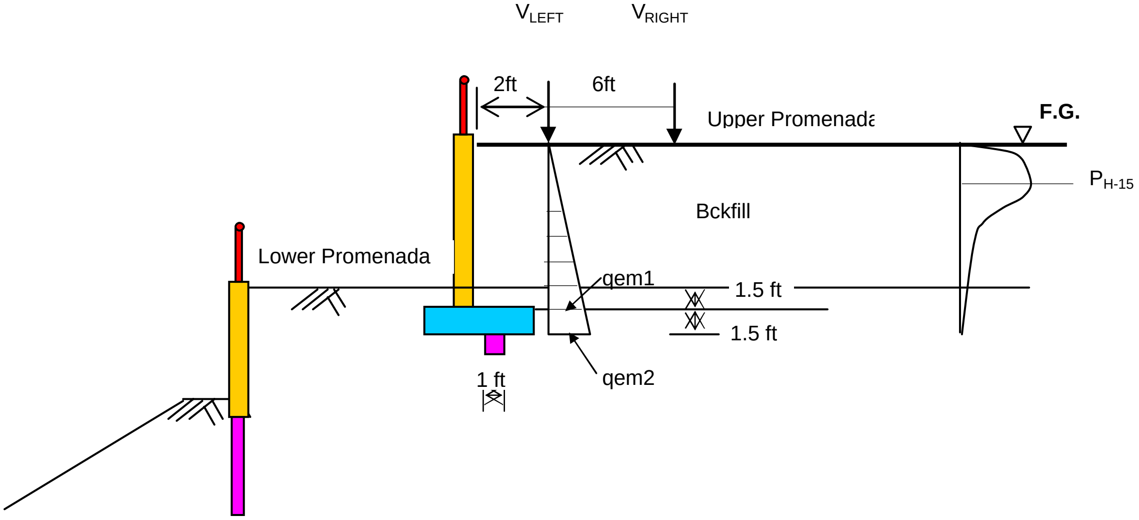
MGE ENGINEERING, INC.

Project	Napa River Flood Control Project		
Subject	Upper Wall Design (H=12')		
By	David An	Date	Feb-05

Load Case 2 -- Normal Condition W/Maintenance Vehicle (Usual Condition)

Wall Design Height, H= 12.00 ft

H-15 truck load **USE V= 16 kips**



Soil Pressure at Wall (Soil pressure =γ Ki hi) **Note:** Passive soil resistance were ignored.

Name	Layer	Thickness(ft)	Pressure(ksf)
qem1	Backfill	12.00	0.571
qem2	Backfill	1.50	0.642

qem - Moist soil pressure at rest wall

H-15 Load---- For bottom of footing

h = 12.00 ft

a=2'/12.00' = 0.17 ≤ 0.4 **For V_{LEFT}**

a=8'/12.00' = 0.67 > 0.4 **For V_{RIGHT}**

$\Delta P_{HZleft} = (0.28V/h^2)\ [b^2 / (0.16+b^2)^3]$ (EM 1110-2-2502 Page 3-48)

$\Delta P_{HZright} = (V/h^2)\ [a^2b^2 / (a^2+b^2)^3]$ (EM 1110-2-2502 Page 3-48)

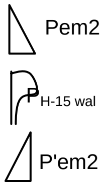
b (for V_{LEFT})	Z	$\Delta P_{PH\ (LEFT)}$	Moment
0.1	1.20	0.076	0.821
0.2	2.40	0.187	1.792
0.3	3.60	0.215	1.806
0.4	4.80	0.182	1.313
0.5	6.00	0.135	0.813
0.6	7.20	0.096	0.459
0.7	8.40	0.067	0.240
0.8	9.60	0.047	0.112
0.9	10.80	0.033	0.040
1.0	12.00	0.024	0.000
Σ		1.061	7.394

b (for V_{RIGHT})	Z	$\Delta P_{PH\ (RIGHT)}$	Moment
0.1	1.20	0.001	0.008
0.2	2.40	0.003	0.025
0.3	3.60	0.005	0.044
0.4	4.80	0.008	0.059
0.5	6.00	0.011	0.065
0.6	7.20	0.013	0.061
0.7	8.40	0.014	0.050
0.8	9.60	0.014	0.035
0.9	10.80	0.014	0.017
1.0	12.00	0.014	0.000
Σ		0.097	0.363

Resultant Summary for Bottom of Wall

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem1	3.4	4.00	13.7
$P_{H\text{-}15\ wall}$	1.2		
At bot of wall	**4.6**		**13.7**
	Σ V		Σ M

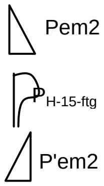
Demand at Bottom of Wall: Vd = 4.6 kips Md = 13.7 k-ft



Resultant Summary for Bottom of Footing

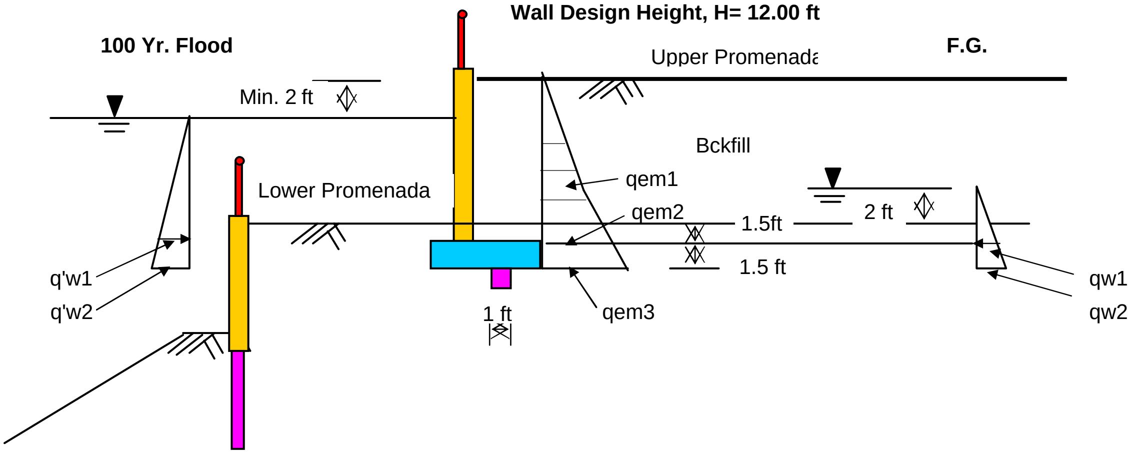
Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem2	4.3	4.50	19.5
$P_{H\text{-}15\ ftg}$	1.2		
At bot of ftg	**5.5**		**19.5**
	Σ V		Σ M

Demand at Bottom of Footing: Vd = 5.5 kips Md = 19.5 k-ft



Load Case 3a -- Design Flood Condition (Unusual Condition)

Wall Design Height, H= 12.00 ft



Soil Pressure at Wall (Soil pressure =γ Ki hi)

Note: Passive soil resistance were ignored.

Name	Layer	Thickness(ft)	Pressure(ksf)
qem1	Backfill	8.50	0.404
qem2	Backfill	3.50	0.570
qem3	Backfill	1.50	0.641
qw1	Backfill	3.50	0.002
qw2	Backfill	1.50	0.003
q'w1	Backfill	10.00	0.007
q'w2	Backfill	1.50	0.008

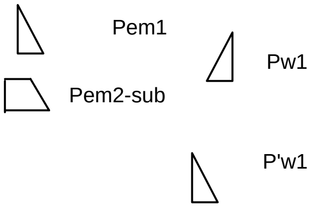
qem1 - Moist soil pressure at rest wall

qem2 & qem3 - Submerged soil pressure at rest wall

qw - Water pressure

Resultant Summary For Bottom Of Wall

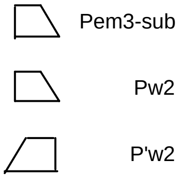
Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem1	1.7	6.33	10.88
Pem2	1.7	1.65	2.81
Pw1	0.004	1.17	0.005
P'w1	-0.03	0.50	-0.02
At bot of wall	**3.39**		**13.7**
	Σ V		Σ M



Demand at Bottom of Wall: Vd = 3.39 kips Md = 13.68 k-ft

Resultant Summary For Bottom Of Footing

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem1	1.7	7.83	13.45
Pem2	1.7	3.15	5.37
Pem3	0.9	0.74	0.67
Pw1	0.00	2.67	0.01
Pw2	0.0	0.71	0.00
P'w1	-0.03	3.33	-0.1
P'w2	-0.01	0.73	-0.01
At bot of ftg	**4.30**		**19.4**
	Σ V		Σ M

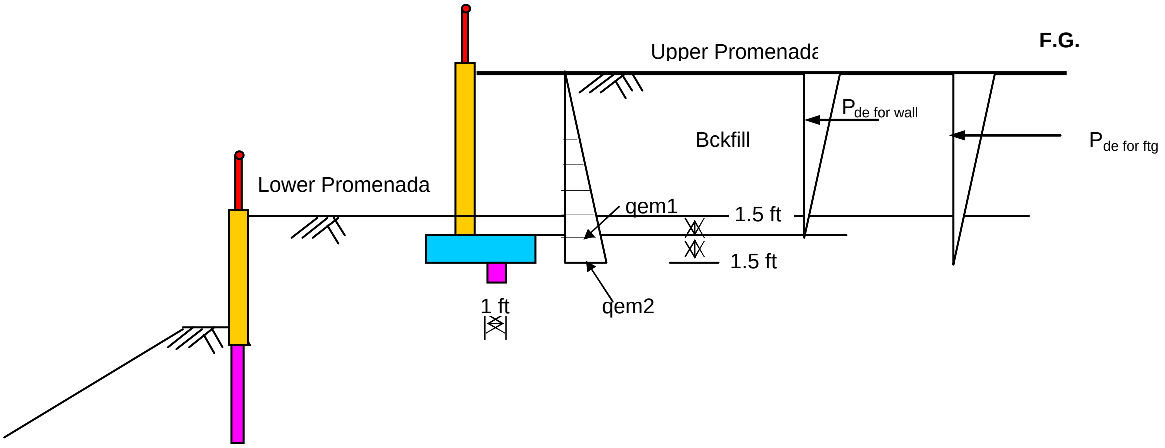


Demand at Bottom of Wall: Vd = 4.3 kips Md = 19.39 k-ft

Project	Napa River Flood Control Project		
Subject	Upper Wall Design (H=12')		
By	David An	Date	Feb-05

Load Case 4 -- Seimic Condition (Extreme condition)

Wall Design Height, H= 12.00 ft



Soil Pressure (Soil pressure =γ Ki hi)

Name	Layer	Thickness(ft)	Pressure(ksf)
qem1	Backfill	12.00	0.571
qem2	Backfill	1.50	0.642
qde-wall	Backfill	12.00	0.172
qde-ftg	Backfill	1.50	0.022

Note: Passive soil resistance were ignored.

qem - Moist soil pressure at rest wall

qde - Earthquake component

qw - Water pressure

Resultant Summary For bottom of wall

Name	Force	Arm to bot.	Moments
Pem1	3.4	4.00	13.7
Pde-wall	1.0	8.00	8.3
At bot of wall	**4.5**		**22.0**
	Σ V		Σ M

Demand at Top of Pile: Vd = 4.46 kips Md = 21.95 k-ft

Pem1

Pde-wall $\alpha = \tan^{-1}[(C_1+\sqrt{(C_1^2+4C_2)})/2]$

(Equation 3-56 of EM 1110-2-2502, Page 3-67)

P'em1 $C_1 = 2\ (\tan\Phi - K_h)\ /\ (1+K_h \tan\Phi)$

(Equation 3-57 of EM 1110-2-2502, Page 3-67)

$C_2 = [\tan\Phi(1-\tan\Phi\ \tan\beta)-(\tan\beta-K_h)]\ /\ [\tan\Phi(1+K_h \tan\Phi)]$

(Equation 3-58 of EM 1110-2-2502, Page 3-67)

K_h =	0.15	g
β =	0	
Φ =	30	degree
C_1 =	0.787	
C_2 =	0.681	
α =	52.6	degree

Resultant Summary For Bottom Of Ftg

Name	Force	Arm to bot.	Moments
Pem2	4.3	4.50	19.5
Pde-ftg	0.02	1.00	0.02
At bot of ftg	**4.3**		**19.5**
	Σ V		Σ M

Demand at Top of Pile: Vd = 4.35 kips Md = 19.51 k-ft

Pem2

Pde-ftg

Dynamic Components qeq =$\gamma\ K_h h^2\ /\ [2(\tan\alpha-\tan\beta)]$

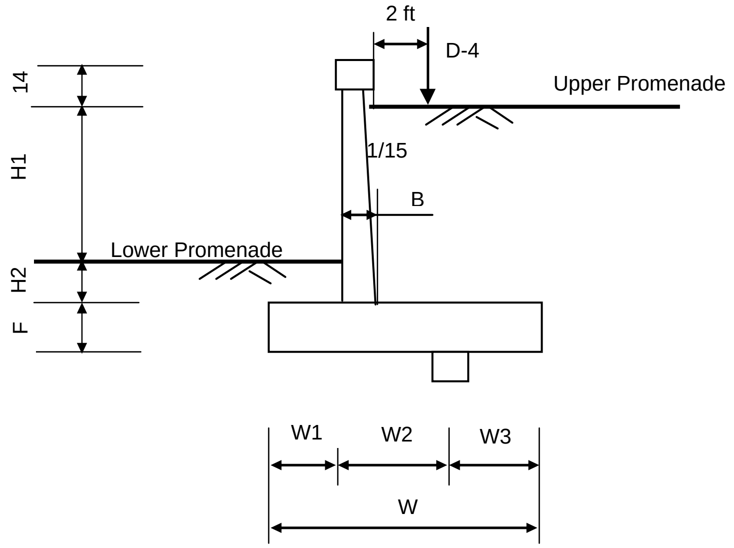
(Equation 3-62 of EM 1110-2-2502, Page 3-68)

Footing & Wall Design (Upper Wall)

Wall Design Height, H= 12.00 ft

1. Loads

Load Case	1		2		3		4	
	Bot. Of Wall	Bot. Of Ftg	Bot. Of Wall	Bot. Of Ftg	Bot. Of Wall	Bot. Of Ftg	Bot. Of Wall	Bot. Of Ftg
Hori. Force (k)	4.8	5.7	4.6	5.5	3.4	4.3	4.5	4.3
Moments(kft)	23.5	30.5	13.7	19.5	13.7	19.4	22.0	19.5



B = 1.70 ft

W1 = 4.75 ft

W2 = 3.75 ft

W3 = 2.50 ft

W = 11.00 ft

H1 = 10.50 ft

H2 = 1.50 ft

F = 1.5 ft

2. Resistances

Location	Dimension		Weight(k/ft)	Arm to toe (ft)	Mot (kft/ft)
	Thick/Depth	Width/Height			
Toe cover	1.50	4.75	0.890625	2.38	2.12
Heel Soil	12.00	4.55	6.825	8.73	59.55
Footing	1.50	11.00	2.475	5.50	13.61
Concrete Key	1.00	1.00	0.15	8.50	1.28
Wall Stem	1.70	13.17	3.358	5.60	18.80
Total			**13.70**		**95.35**

Due to D-4
Vertical weight

Distributed weight, Pd-4 = 2.50 k/ft

Resistance of overturning due to D-4 = distributed weight x (W1 + 1 + 2ft) 19.38 kft/ft

3. Overturning Check

Load Case	Resistance Moment	Overturning Moment	Safety Factor	
1	114.73	30.5	3.76	**Safety Factor > 1.5, OK**
2	95.35	19.5	4.89	**Safety Factor > 1.5, OK**
3	95.35	19.4	4.92	**Safety Factor > 1.5, OK**
4	95.35	19.5	4.89	**Safety Factor > 1.5, OK**

4. Sliding Check

Resistance Due to Soil Friction

μ = 0.3

Ffr = P_d x μ 4.1 kips/ft

where, P_d -- weight of concrete & soil

Resistance Due to D-4

Fd-4 = Pd_{-4} x μ 0.750 kips/ft

Pd_{-4}--distributed weight due to D-4

Resistance Due to Concrete Key

Fcr = 2√f'c x 1 ft 18.21 kips/ft

where, f'c = 4000 psi

Load Case	Resistance Due to Soil (kips)	Sliding Force (kips)	Safety Factor		Resistance Due to Concrete Key (kips)	Sliding Force (kips)	Safety Factor	
1	4.11	5.7	0.72	**< 1.33, NG**	18.21	5.7	3.20	**> 1.33, OK**
2	4.11	5.5	0.75	**< 1.5, NG**	18.21	5.5	3.32	**> 1.5, OK**
3	4.11	4.3	0.95	**< 1.5, NG**	18.21	4.3	4.23	**> 1.5, OK**
4	4.11	4.3	0.95	**< 1.1, NG**	18.21	4.3	4.19	**> 1.1, OK**

5. Soil Bearing Pressure Check

Moment about center of footing

M = Msmax(due to soil pressure) + Mt(due to toe soil) - Mk(due to key) - Mw(due to wall) - Mh(due to heel soil) - M_{d4}(Due to D-4)

M = 5.2 kft/ft

where, Max. M (due to lateral soil pressure), Msmax. = 30.5 kft/ft

Msmax = (case1, case2, case3, case4)

Mt =H2 x W1 x 0.12 x (W/2-W1/2) 2.7 kft/ft

Mk = - (1ft x 1ft x 0.15 x (W/2-W3)) -0.45 kft/ft

Mw = B x (H1 + H2 + 14/12) x 0.15 x (W/2-B/2-W1) -0.34 kft/ft

Mh =- (H1 + H2)x0.12x[W2 + W3 - (B+1)/2]x{W/2-[W2+W3-(B+1)/2]/2} -21.52 kft/ft

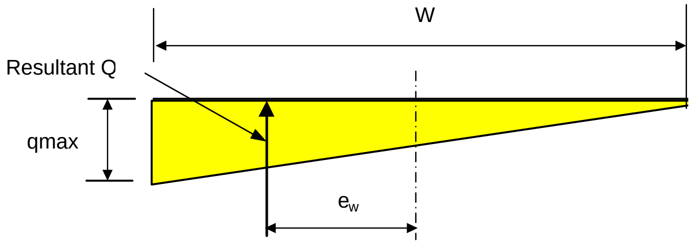
M_{D-4} = - P_{D-4} x (W/2 - W1-1ft - 2) -5.63 kft/ft

Eccentricity e_W = M / (Pd+Pd-4) 0.32 ft

W / 6 = 1.83 ft

Therefor e_W < W / 6

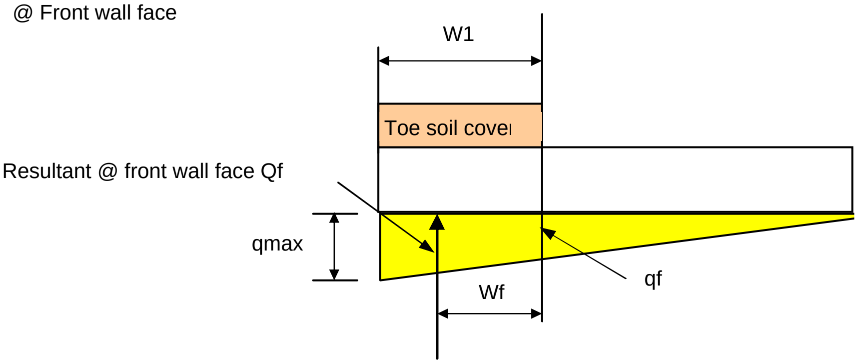
Footing Contact Pressure



Q = Pd + Pd-4		16.20	kips
1 + (6 e_w / W) =		1.18	ft
1 - (6 e_w / W) =		0.82	ft
qmax = Q [1 + (6ew / W)] / W		1.73	ksf
qmin = Q [1 - (6ew / W)] / W		1.21	ksf
assumed allowable soil brearing pressure			
qa =	controlling	2.00	ksf
D / C = qmax / qa =	0.87	**<1, OK**	

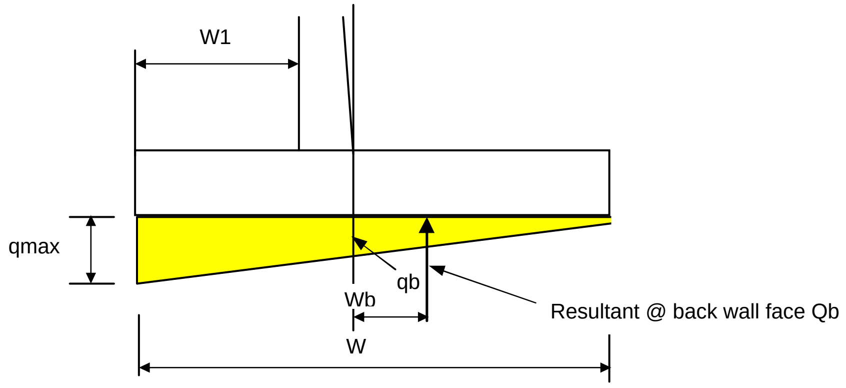
6. Footing Shear and Moment Check (@ face of wall)

@ Front wall face



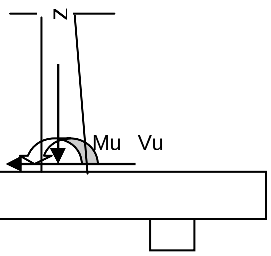
qf = (W2+W3) / W x (qmax - qmin) + qmin		1.51	ksf
Qf =(qf+qmax)/2 x W1		7.69	k/ft
Arm to face of wall			
Wf = W1 (2qmax+qf) / [3 (qmax+qf)]		2.43	ft
Mu = Hf{1.7 [Qf x Wf - (W1x F x 0.15 x W1/2 -W1 x h2 x 0.12 x W1/2)]}		31.21	kft/ft
Mu = Hf[1.7(D+L)]			
Hf =	Hydraulic factor	1.30	
EM-1110-2-2104---P3-2, Equation3.3			
Vu = Hf{1.7[Qf - (W1 x F x 0.15 - W1 x 1ft x 0.12)]}		12.75	k/ft
Required bar area for flexure			
Mu ≤ ΦMn			
ΦMn = Φ As x fy (d - a/2)			
Assume a = 0.15d		2.25	in
where	d =	15	in
As = Mu / [Φfy(d-a/2)]		0.50	in^2
	Φ =	0.9	
	fy =	60	ksi
Use #7@12", As =0.60 in^2	As =	0.60	in^2
Check			
a =Asfy / (0.85f'c*b)		0.9	in
ΦMn = Φ Asfy(d - a/2)		39.31	kft/ft
D/C = Mu/ΦMn		0.79	**OK**
Shear Check			
Shear capacity			
ΦVn = Φ2√f'c x F		23.22	k/ft
where	Φ =	0.85	
D/C = Vu/ΦVn		0.55	**OK**

@ Back wall face



qb = (W-W1-B) / W x qmax + qmin		1.43	ksf
Qb =(qf+qmin)/2 x (W-W1-B)		6.01	k/ft
Arm to face of wall			
Wb =(W-W1-B) - (W- W1-B) (2qb+qmin) / [3 (qb+qmin)]		2.21	ft
Mu = Hf{1.7[Qb x Wb - (F x 0.15 x (W-W1-B)^2/2 + (H1+H2) x 0.12 x (W-W1-B)/2)-(Pd-4 x 2ft)]			
		-19.74	kft/ft
Mu = Hf[1.7(D+L)]			
Hf =	Hydraulic factor	1.30	
EM-1110-2-2104---P3-2, Equation3.3			
Vu = Hf x 1.7[Qb - ((W-W1-B) x F x 0.15 + (W-W1-B) x H2 x 0.12) - Ph15]		-8.99	k/ft
Required bar area for flexure			
Mu ≤ ΦMn			
ΦMn = Φ As x fy (d - a/2)			
Assume a = 0.15d		2.25	in
where	d =	15	in
As = Mu / [Φfy(d-a/2)]		0.32	in^2
	Φ =	0.9	
	fy =	60	ksi
Use #6@12", As = 0.44 in^2	As =	0.44	in^2
Check			
a =Asfy / (0.85f'c*b)		0.6	in
ΦMn = Φ Asfy(d - a/2)		29.06	kft/ft
D/C = Mu/ΦMn		0.68	**OK**
Shear Check			
Shear capacity			
ΦVn = Φ2√f'c x F		23.22	k/ft
where	Φ =	0.85	
D/C = Vu/ΦVn		0.39	**OK**

7. Wall Stem Shear and Moment Check (@ bottom of wall)



MGE Engineering, Inc.

Project	Napa River Flood Control Project		
Subject	Upper Wall Design (H=12')		
By	David An	Date	Feb-05

Mu = Hf x 1.7 x M			51.8	kft/ft
Where	M = max(case1, case2, case3, case4)			
	Mu = Hf[1.7(D+L)]			
	Hf =	Hydraulic factor	1.30	
	EM-1110-2-2104---P3-2, Equation3.3			
Vu = Hf x 1.7 x Vmax		Vmax = max(case1, case2, case3, case4)	10.6	k/ft
Required bar area for flexure				
	Mu ≤ ΦMn			
	ΦMn = Φ As x fy (d - a/2)			
	Assume a = 0.15d		2.61	in
	where	d =	17.4	in
	As = Mu / [Φfy(d-a/2)]		0.72	in^2
		Φ =	0.9	
		fy =	60	ksi
		f'c =	4000	psi
	Use #7@9", As =0.6 in^2 x 12/9	As =	0.80	in^2
Check				
	a =Asfy / (0.85f'c*b)		1.2	in
	ΦMn = Φ Asfy(d - a/2)		60.52	kft/ft
	D/C = Mu/ΦMn		0.86	**OK**
Shear Check				
	Shear capacity			
	ΦVn = Φ2√f'c x B		26.32	k/ft
	where	Φ =	0.85	
	D/C = Vu/ΦVn		0.40	**OK**

MGE ENGINEERING, INC.		
	Project: Napa River Flood Control Project	
	Subject: Upper Wall Design (H=14')	
	By: David An	Date: Feb-05

Upper Wall Design

Design Height H= 14.00 ft

Backfill Properties

Backfill Thickness =	14.00	ft
Backfill Unit Weight =	125	pcf
Φ =	37	degree
C =	0	pcf
SMF= Tan(Φ_d) / TanΦ = 2/3 =	0.67	
Φ_d =	27	degree
Ka = Tan^2 (45° - Φ/2) =	0.25	
Ko = Tan^2 (45° - Φ_d/2) =	0.38	
Kp = Tan^2 (45° + Φ/2) =	4.02	

Water Property

Water Unit Weight =	62.5	pcf

Wall and Footing Data

Design Height (ft)	14
Toe Cover (ft)	1.5
Top Wall Thick (ft)	1.0
100 Year Flood Level to Top of Wall (ft)	2.00

MGE ENGINEERING, INC.

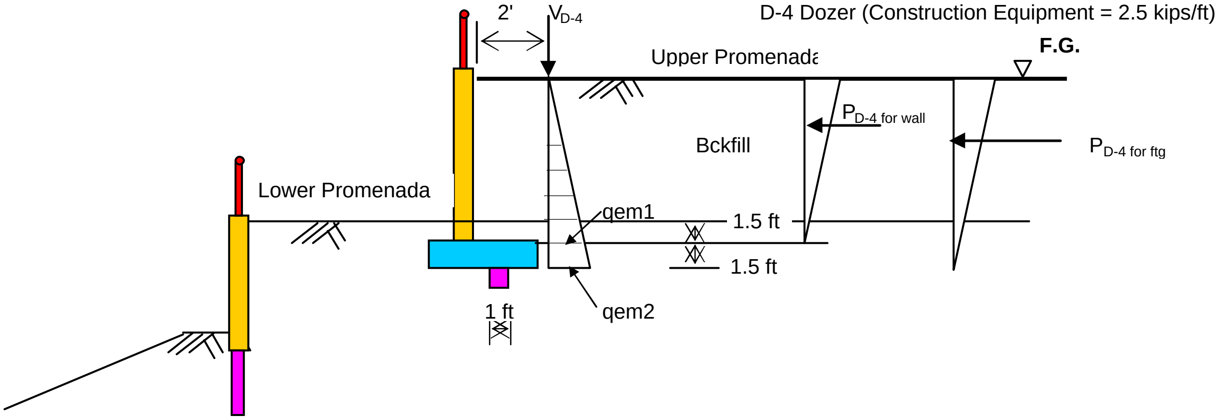
Project	Napa River Flood Control Project		
Subject	Upper Wall Design (H=14')		
By	David An	Date	Feb-05

Upper Wall Design (H=14.00 ft)

Load case based on DOA--Flood Walls and Channel Retaining Walls, Load Conditions & Load Diagrams

Load Case 1 -- Construction Condition (Unusal Condition)

Wall Design Height, H= 14.00 ft



Soil Pressure at Wall (Soil pressure =γ Ki hi)

Note: Passive soil resistance were ignored.

Name	Layer	Thickness(ft)	Pressure(ksf)
qem1	Backfill	14.00	0.666
qem2	Backfill	1.50	0.737

qem - Moist soil pressure at rest wall

D-4 Load---- **For bottom of wall**

h = 14.00 ft

a = 2' / 14.0' = 0.14 ≤ 0.4

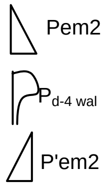
V_{D-4} = 2.5 kips/ft

$\Delta P_{D-4} = (V_{D-4} /h)\ [(0.203b)/(0.16+b^2)^2]$ (EM 1110-2-2502 Page 3-49)

b	Z = bh	$\Delta P_{D-4\ wal}$	Moment
0.0	0.00	0.000	0.000
0.1	1.40	0.176	2.213
0.2	2.80	0.254	2.842
0.3	4.20	0.244	2.387
0.4	5.60	0.198	1.665
0.5	7.00	0.151	1.057
0.6	8.40	0.113	0.631
0.7	9.80	0.084	0.353
0.8	11.20	0.063	0.178
0.9	12.60	0.049	0.068
1.0	14.00	0.038	0.000
Σ		1.369	11.393

Resultant Summary for Bottom of Wall

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem1	4.7	4.67	21.7
Pd-4 wall	1.4		11.4
At bot of wall	**6.0**		**33.1**
	Σ V		Σ M



Demand at Bottom of Wall: Vd = 6.0 kips Md = 33.1 k-ft

D-4 Load---- **For bottom of footing**

h = 15.50 ft

a = 2' / 15.5' = 0.13 ≤ 0.4

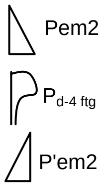
V_{D-4} = 2.5 kips/ft

$\Delta P_{D-4} = (V_{D-4} / h)\ [(0.203b)/(0.16+b\text{^}2)\text{^}2]$ (EM 1110-2-2502 Page 3-49)

b	Z = bh	$\Delta P_{D-4\ wal}$	Moment
0.0	0.00	0.000	0.000
0.1	1.55	0.176	2.450
0.2	3.10	0.254	3.147
0.3	4.65	0.244	2.643
0.4	6.20	0.198	1.844
0.5	7.75	0.151	1.170
0.6	9.30	0.113	0.698
0.7	10.85	0.084	0.391
0.8	12.40	0.063	0.197
0.9	13.95	0.049	0.075
1.0	15.50	0.038	0.000
Σ		1.369	12.614

Resultant Summary for Bottom of Footing

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem2	5.7	5.17	29.5
Pd-4 ftg	1.4		12.6
At bot of ftg	**7.1**		**42.1**
	Σ V		Σ M



Demand at Bottom of Footing: Vd = 7.1 kips Md = 42.1 k-ft

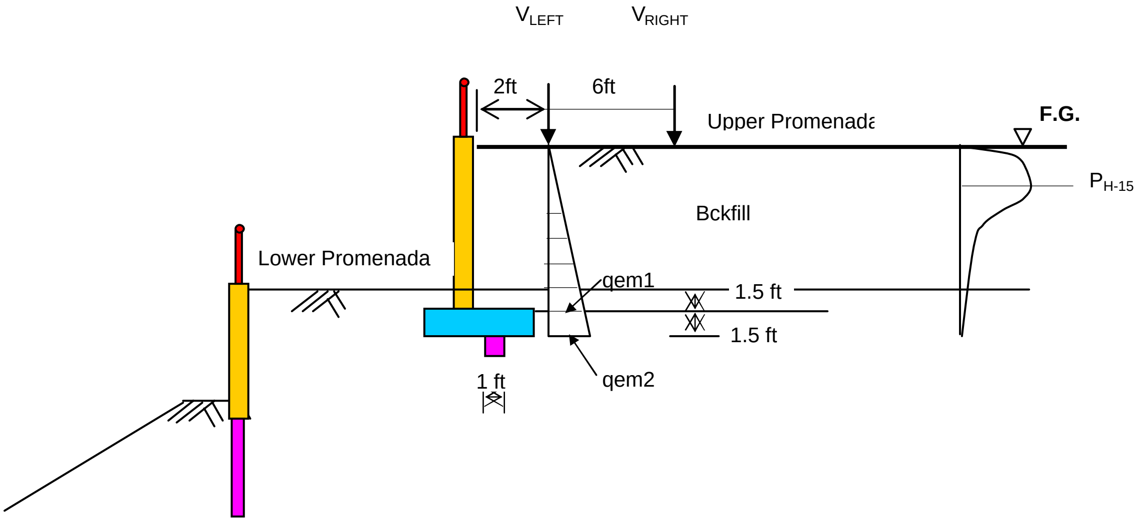
MGE ENGINEERING, INC.

Project: Napa River Flood Control Project
Subject: Upper Wall Design (H=14')
By: David An
Date: Feb-05

Load Case 2 -- Normal Condition W/Maintenance Vehicle (Usual Condition)

Wall Design Height, H= 14.00 ft

H-15 truck load **USE V= 16 kips**



Soil Pressure at Wall (Soil pressure =γ Ki hi)

Note: Passive soil resistance were ignored.

Name	Layer	Thickness(ft)	Pressure(ksf)
qem1	Backfill	14.00	0.666
qem2	Backfill	1.50	0.737

qem - Moist soil pressure at rest wall

H-15 Load---- For bottom of footing

h = 14.00 ft

a=2'/14.00' = 0.14 ≤ 0.4 **For V_{LEFT}**

a=8'/14.00' = 0.57 > 0.4 **For V_{RIGHT}**

$\Delta P_{HZleft} = (0.28V/h^2)\ [b^2 / (0.16+b^2)^3]$ (EM 1110-2-2502 Page 3-48)

$\Delta P_{HZright} = (V/h^2)\ [a^2 b^2 / (a^2+b^2)^3]$ (EM 1110-2-2502 Page 3-48)

b (for V_{LEFT})	Z	$\Delta P_{PH\ (LEFT)}$	Moment
0.1	1.40	0.065	0.821
0.2	2.80	0.160	1.792
0.3	4.20	0.184	1.806
0.4	5.60	0.156	1.313
0.5	7.00	0.116	0.813
0.6	8.40	0.082	0.459
0.7	9.80	0.057	0.240
0.8	11.20	0.040	0.112
0.9	12.60	0.028	0.040
1.0	14.00	0.021	0.000
Σ		0.910	7.394

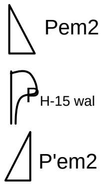
b (for V_{RIGHT})	Z	$\Delta P_{PH\ (RIGHT)}$	Moment
0.1	1.40	0.001	0.008
0.2	2.80	0.002	0.025
0.3	4.20	0.005	0.044
0.4	5.60	0.007	0.059
0.5	7.00	0.009	0.065
0.6	8.40	0.011	0.061
0.7	9.80	0.012	0.050
0.8	11.20	0.012	0.035
0.9	12.60	0.012	0.017
1.0	14.00	0.012	0.000
Σ		0.083	0.363

MGE ENGINEERING, INC.

Project Napa River Flood Control Project
Subject Upper Wall Design (H=14')
By David An
Date Feb-05

Resultant Summary for Bottom of Wall

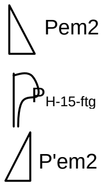
Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem1	4.7	4.67	21.7
$P_{H\text{-}15\ wall}$	1.0		
At bot of wall	**5.7**		**21.7**
	Σ V		Σ M



Demand at Bottom of Wall: Vd = 5.7 kips Md = 21.7 k-ft

Resultant Summary for Bottom of Footing

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem2	5.7	5.17	29.5
$P_{H\text{-}15\ ftg}$	1.0		
At bot of ftg	**6.7**		**29.5**
	Σ V		Σ M



Demand at Bottom of Footing: Vd = 6.7 kips Md = 29.5 k-ft

MGE ENGINEERING, INC.

Project Napa River Flood Control Project

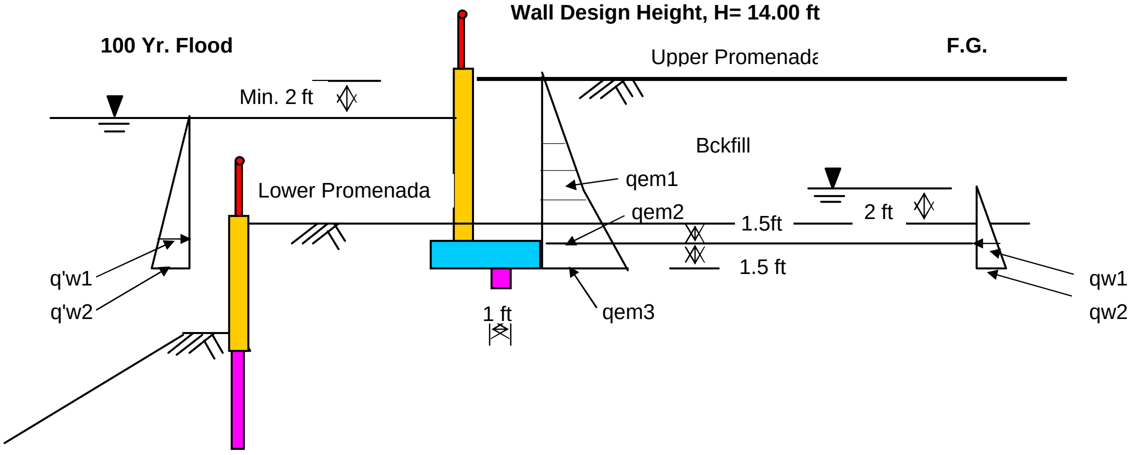
Subject Upper Wall Design (H=14')

By David An

Date Feb-05

Load Case 3a -- Design Flood Condition (Unusual Condition)

Wall Design Height, H= 14.00 ft



Soil Pressure at Wall (Soil pressure =γ Ki hi)

Note: Passive soil resistance were ignored.

Name	Layer	Thickness(ft)	Pressure(ksf)
qem1	Backfill	10.50	0.499
qem2	Backfill	3.50	0.665
qem3	Backfill	1.50	0.736
qw1	Backfill	3.50	0.002
qw2	Backfill	1.50	0.003
q'w1	Backfill	12.00	0.008
q'w2	Backfill	1.50	0.009

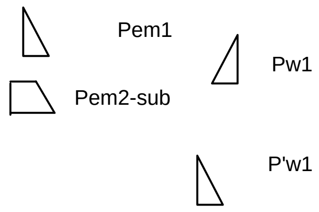
qem1 - Moist soil pressure at rest wall

qem2 & qem3 - Submerged soil pressure at rest wall

qw - Water pressure

Resultant Summary For Bottom Of Wall

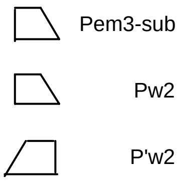
Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem1	2.6	7.00	18.35
Pem2	2.0	1.67	3.40
Pw1	0.004	1.17	0.005
P'w1	-0.05	0.50	-0.02
At bot of wall	**4.61**		**21.7**
	Σ V		Σ M



Demand at Bottom of Wall: Vd = 4.61 kips Md = 21.72 k-ft

Resultant Summary For Bottom Of Footing

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem1	2.6	8.50	22.28
Pem2	2.0	3.17	6.45
Pem3	1.1	0.74	0.77
Pw1	0.00	2.67	0.01
Pw2	0.0	0.71	0.00
P'w1	-0.05	4.00	-0.2
P'w2	-0.01	0.74	-0.01
At bot of ftg	**5.67**		**29.3**
	Σ V		Σ M



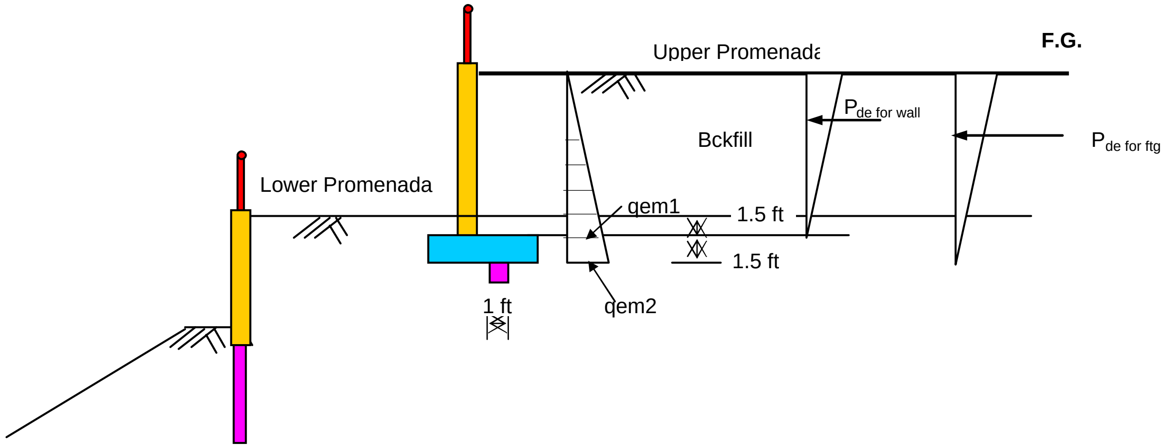
Demand at Bottom of Wall: Vd = 5.67 kips Md = 29.32 k-ft

MGE ENGINEERING, INC.

Project	Napa River Flood Control Project		
Subject	Upper Wall Design (H=14')		
By	David An	Date	Feb-05

Load Case 4 -- Seimic Condition (Extreme condition)

Wall Design Height, H= 14.00 ft



Soil Pressure (Soil pressure =γ Ki hi)

Note: Passive soil resistance were ignored.

Name	Layer	Thickness(ft)	Pressure(ksf)
qem1	Backfill	14.00	0.666
qem2	Backfill	1.50	0.737
qde-wall	Backfill	14.00	0.201
qde-ftg	Backfill	1.50	0.022

qem - Moist soil pressure at rest wall

qde - Earthquake component

qw - Water pressure

Resultant Summary For bottom of wall

Name	Force	Arm to bot.	Moments
Pem1	4.7	4.67	21.7
Pde-wall	1.4	9.33	13.1
At bot of wall	**6.1**		**34.9**
	Σ V		Σ M

Demand at Top of Pile: Vd = 6.06 kips Md = 34.86 k-ft

Pem1

Pde-wall $\alpha = \tan^{-1}[(C_1+\sqrt{(C_1^2+4C_2)})/2]$

(Equation 3-56 of EM 1110-2-2502, Page 3-67)

P'em1 $C_1 = 2(\tan\Phi - K_h) / (1+K_h \tan\Phi)$

(Equation 3-57 of EM 1110-2-2502, Page 3-67)

$C_2 = [\tan\Phi(1-\tan\Phi \ \tan\beta)-(\tan\beta-K_h)] / [\tan\Phi(1+K_h \tan\Phi)]$

(Equation 3-58 of EM 1110-2-2502, Page 3-67)

$K_h =$ 0.15 g

β = 0

Φ = 30 degree

$C_1 =$ 0.787

$C_2 =$ 0.681

α = 52.6 degree

Resultant Summary For Bottom Of Ftg

Name	Force	Arm to bot.	Moments
Pem2	5.7	5.17	29.5
Pde-ftg	0.02	1.00	0.02
At bot of ftg	**5.7**		**29.5**
	Σ V		Σ M

Demand at Top of Pile: Vd = 5.73 kips Md = 29.52 k-ft

Pem2

Pde-ftg

Dynamic Components $q_{eq} = \gamma K_h h^2 / [2(\tan\alpha - \tan\beta)]$

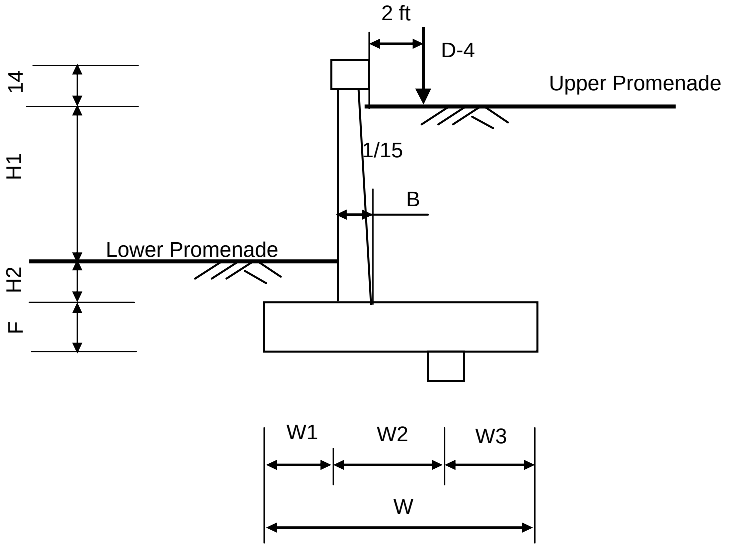
(Equation 3-62 of EM 1110-2-2502, Page 3-68)

Footing & Wall Design (Upper Wall)

Wall Design Height, H= 14.00 ft

1. Loads

Load Case	1		2		3		4	
	Bot. Of Wall	Bot. Of Ftg	Bot. Of Wall	Bot. Of Ftg	Bot. Of Wall	Bot. Of Ftg	Bot. Of Wall	Bot. Of Ftg
Hori. Force (k)	6.0	7.1	5.7	6.7	4.6	5.7	6.1	5.7
Moments(kft)	33.1	42.1	21.7	29.5	21.7	29.3	34.9	29.5



B = 1.83 ft
W1 = 5.00 ft
W2 = 4.50 ft
W3 = 3.00 ft
W = 12.50 ft

H1 = 12.50 ft
H2 = 1.50 ft
F = 1.5 ft

2. Resistances

Location	Dimension		Weight(k/ft)	Arm to toe (ft)	Mot (kft/ft)
	Thick/Depth	Width/Height			
Toe cover	1.50	5.00	0.9375	2.50	2.34
Heel Soil	14.00	5.67	9.917	9.67	95.86
Footing	1.50	12.50	2.813	6.25	17.58
Concrete Key	1.00	1.00	0.15	9.50	1.43
Wall Stem	1.83	15.17	4.171	5.92	24.68
Total			**17.99**		**141.89**

Due to D-4
Vertical weight

Distributed weight, Pd-4 = 2.50 k/ft
Resistance of overturning due to D-4 = distributed weight x (W1 + 1 + 2ft) 20.00 kft/ft

MGE Engineering, Inc.			
	Project	Napa River Flood Control Project	
	Subject	Upper Wall Design (H=14')	
	By	David An	Date Feb-05

3. Overturning Check

Load Case	Resistance Moment	Overturning Moment	Safety Factor	
1	161.89	42.1	3.84	**Safety Factor > 1.5, OK**
2	141.89	29.5	4.81	**Safety Factor > 1.5, OK**
3	141.89	29.3	4.84	**Safety Factor > 1.5, OK**
4	141.89	29.5	4.81	**Safety Factor > 1.5, OK**

4. Sliding Check

Resistance Due to Soil Friction

μ = 0.3

$Ffr = P_d \times \mu$ 5.4 kips/ft

where, P_d -- weight of concrete & soil

Resistance Due to D-4

$Fd\text{-}4 = Pd_{-4} \times \mu$ 0.750 kips/ft

Pd_{-4}--distributed weight due to D-4

Resistance Due to Concrete Key

$Fcr = 2\sqrt{f'c} \times 1\ ft$ 18.21 kips/ft

where, f'c = 4000 psi

Load Case	Resistance Due to Soil (kips)	Sliding Force (kips)	Safety Factor		Resistance Due to Concrete Key (kips)	Sliding Force (kips)	Safety Factor	
1	5.40	7.1	0.76	**< 1.33, NG**	18.21	7.1	2.57	**> 1.33, OK**
2	5.40	6.7	0.80	**< 1.5, NG**	18.21	6.7	2.72	**> 1.5, OK**
3	5.40	5.7	0.95	**< 1.5, NG**	18.21	5.7	3.21	**> 1.5, OK**
4	5.40	5.7	0.94	**< 1.1, NG**	18.21	5.7	3.18	**> 1.1, OK**

5. Soil Bearing Pressure Check

Moment about center of footing

M = Msmax(due to soil pressure) + Mt(due to toe soil) - Mk(due to key) - Mw(due to wall) - Mh(due to heel soil) - M_{d4}(Due to D-4)

M = 9.2 kft/ft

where, Max. M (due to lateral soil pressure), Msmax. = 42.1 kft/ft

Msmax = (case1, case2, case3, case4)

Mt =H2 x W1 x 0.12 x (W/2-W1/2) 3.4 kft/ft

Mk = - (1ft x 1ft x 0.15 x (W/2-W3)) -0.49 kft/ft

Mw = B x (H1 + H2 + 14/12) x 0.15 x (W/2-B/2-W1) 1.39 kft/ft

Mh =- (H1 + H2)x0.12x[W2 + W3 - (B+1)/2]x{W/2-[W2+W3-(B+1)/2]/2} -32.79 kft/ft

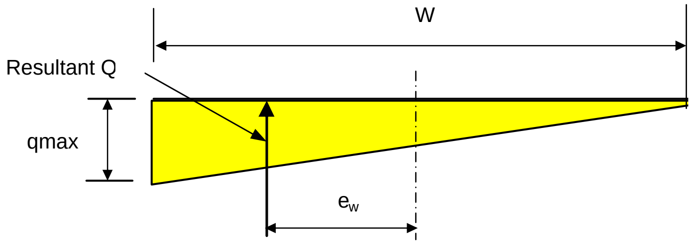
$M_{D-4} = - P_{D-4} \times (W/2 - W1\text{-}1ft - 2)$ -4.38 kft/ft

Eccentricity $e_W = M / (Pd+Pd\text{-}4)$ 0.45 ft

W / 6 = 2.08 ft

Therefor $e_W < W / 6$

Footing Contact Pressure



Q = Pd + Pd-4 20.49 kips

1 + (6 e_w / W) = 1.22 ft

1 - (6 e_w / W) = 0.78 ft

qmax = Q [1 + (6ew / W)] / W 1.99 ksf

qmin = Q [1 - (6ew / W)] / W 1.28 ksf

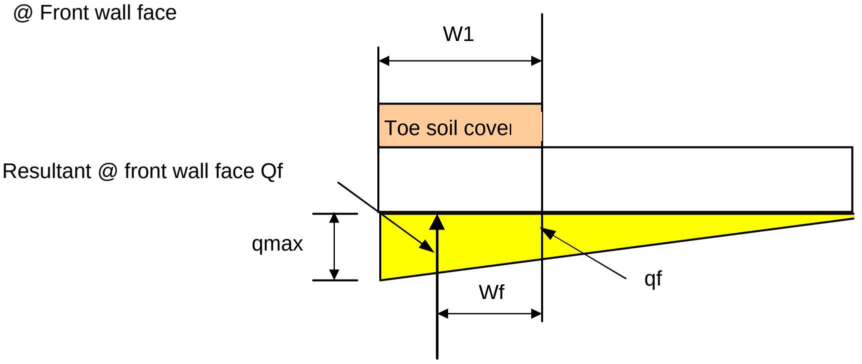
assumed allowable soil brearing pressure

qa = controlling 2.00 ksf

D / C = qmax / qa = 1.00 **<1, OK**

6. Footing Shear and Moment Check (@ face of wall)

@ Front wall face



qf = (W2+W3) / W x (qmax - qmin) + qmin 1.71 ksf

Qf =(qf+qmax)/2 x W1 9.26 k/ft

Arm to face of wall

Wf = W1 (2qmax+qf) / [3 (qmax+qf)] 2.56 ft

Mu = Hf{1.7 [Qf x Wf - (W1x F x 0.15 x W1/2 -W1 x h2 x 0.12 x W1/2)]} 41.27 kft/ft

Mu = Hf[1.7(D+L)]

Hf = Hydraulic factor 1.30

EM-1110-2-2104---P3-2, Equation3.3

Vu = Hf{1.7[Qf - (W1 x F x 0.15 - W1 x 1ft x 0.12)]} 15.99 k/ft

Required bar area for flexure

Mu ≤ ΦMn

ΦMn = Φ As x fy (d - a/2)

Assume a = 0.15d 2.25 in

where d = 15 in

As = Mu / [Φfy(d-a/2)] 0.66 in^2

Φ = 0.9

fy = 60 ksi

Use #7@10", As =12/10 x 0.60 in^2 As = 0.72 in^2

Check

a =Asfy / (0.85f'c*b) 1.1 in

ΦMn = Φ Asfy(d - a/2) 46.88 kft/ft

D/C = Mu/ΦMn 0.88 **OK**

Shear Check

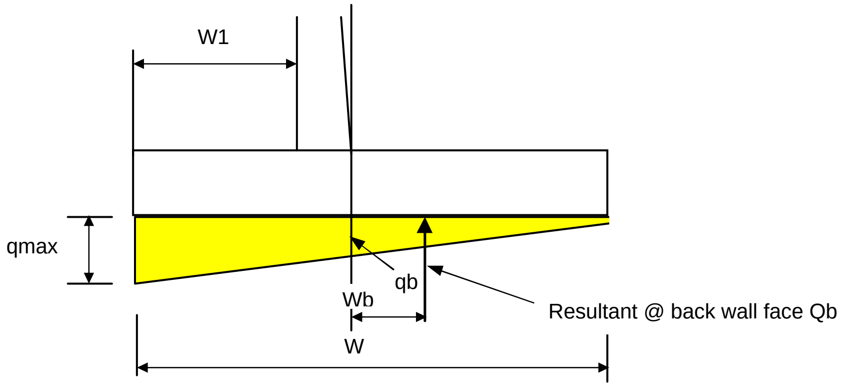
Shear capacity

ΦVn = Φ2√f'c x F 23.22 k/ft

where Φ = 0.85

D/C = Vu/ΦVn 0.69 **OK**

@ Back wall face



qb = (W-W1-B) / W x qmax + qmin 1.61 ksf

Qb =(qf+qmin)/2 x (W-W1-B) 8.19 k/ft

Arm to face of wall

Wb =(W-W1-B) - (W- W1-B) (2qb+qmin) / [3 (qb+qmin)] 2.73 ft

Mu = Hf{1.7[Qb x Wb - (F x 0.15 x (W-W1-B)^2/2 + (H1+H2) x 0.12 x (W-W1-B)/2)-(Pd-4 x 2ft)]

-29.27 kft/ft

Mu = Hf[1.7(D+L)]

Hf = Hydraulic factor 1.30

EM-1110-2-2104---P3-2, Equation3.3

Vu = Hf x 1.7[Qb - ((W-W1-B) x F x 0.15 + (W-W1-B) x H2 x 0.12) - Ph15] -11.28 k/ft

Required bar area for flexure

Mu ≤ ΦMn

ΦMn = Φ As x fy (d - a/2)

Assume a = 0.15d 2.25 in

where d = 15 in

As = Mu / [Φfy(d-a/2)] 0.47 in^2

Φ = 0.9

fy = 60 ksi

Use #6@10", As = 12/10 x 0.44 in^2 As = 0.528 in^2

Check

a =Asfy / (0.85f'c*b) 0.8 in

ΦMn = Φ Asfy(d - a/2) 34.72 kft/ft

D/C = Mu/ΦMn 0.84 **OK**

Shear Check

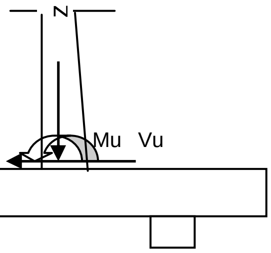
Shear capacity

ΦVn = Φ2√f'c x F 23.22 k/ft

where Φ = 0.85

D/C = Vu/ΦVn 0.49 **OK**

7. Wall Stem Shear and Moment Check (@ bottom of wall)



Mu = Hf x 1.7 x M			77.0	kft/ft
Where	M = max(case1, case2, case3, case4)			
	Mu = Hf[1.7(D+L)]			
	Hf = Hydraulic factor		1.30	
	EM-1110-2-2104---P3-2, Equation3.3			
Vu = Hf x 1.7 x Vmax	Vmax = max(case1, case2, case3, case4)		13.4	k/ft
Required bar area for flexure				
	Mu ≤ ΦMn			
	ΦMn = Φ As x fy (d - a/2)			
	Assume a = 0.15d		2.85	in
	where	d =	19.0	in
	As = Mu / [Φfy(d-a/2)]		0.97	in^2
		Φ =	0.9	
		fy =	60	ksi
		f'c =	4000	psi
	Use #8@9", As =0.79 in^2 x 12/9	As =	1.05	in^2
Check				
	a =Asfy / (0.85f'c*b)		1.5	in
	ΦMn = Φ Asfy(d - a/2)		86.39	kft/ft
	D/C = Mu/ΦMn		0.89	**OK**
Shear Check				
	Shear capacity			
	ΦVn = Φ2√f'c x B		28.38	k/ft
	where	Φ =	0.85	
	D/C = Vu/ΦVn		0.47	**OK**

RAMP WALL DESIGN

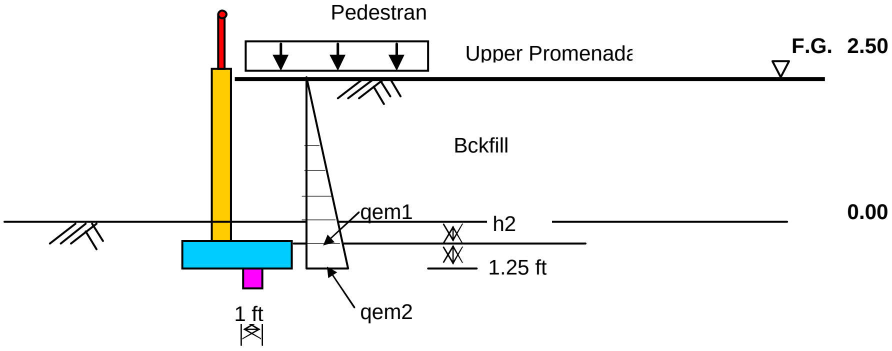
MGE Engineering, Inc.

Project	Napa River Flood Control Project		
Subject	Ramp Wall Design (H=3'-6")		
By	David An	Date	Feb-05

Load Case 2 -- Normal Condition W/Pedestrin Load (Usual Condition)

No vehecal can access

H = 3.5 ft, Desin Height



Note: Passive soil resistance were ignored.

qem - Moist soil pressure at rest wall

Soil Pressure at Wall (Soil pressure =γ Ki hi)

Name	Layer	Thickness(ft)	Pressure(ksf)
qem1	wall	5.50	0.274
qem2	Bot. Footing	1.25	0.336

Backfill parameter:

unit weight	125	pcf
friction angle, ϕ	37	deg
sin(ϕ)	0.602	
at rest coeff. Ko	0.398	
passive, kp	4.023	
Pedestrian load =	250	lbs/sf, UBC
equivalent surcharge, hp =	2	ft

Thickness(ft) = soil height of backwall + equivalent height of surcharge

Resultant Summary for Bottom of Wall

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem1	0.8	1.83	1.4
At bot of wall	**0.8**		**1.4**
	Σ V		Σ M

Demand at Bottom of Wall: Vd = .8 kips Md = 1.4 k-ft

Resultant Summary for Bottom of Footing

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem2	1.1	2.25	2.6
At bot of ftg	**1.1**		**2.6**
	Σ V		Σ M

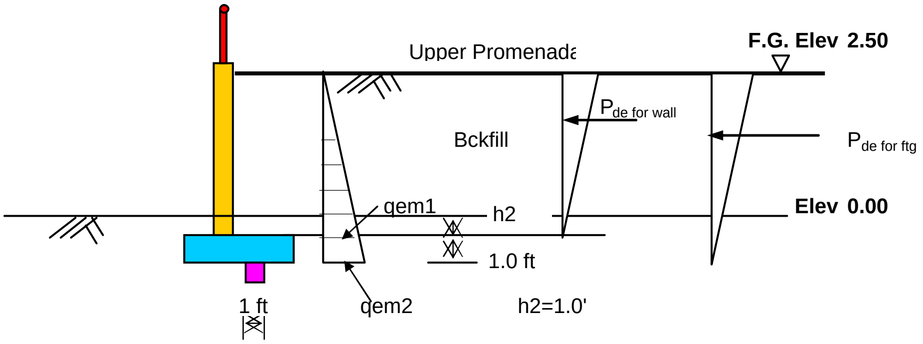
Demand at Bottom of Footing: Vd = 1.1 kips Md = 2.6 k-ft

Project	Napa River Flood Control Project
Subject	Ramp Wall Design (H=3'-6")
By	David An
Date	Feb-05

Load Case 4 -- Seimic Condition (Extreme condition)

No vehecal can access

H = 3.5 ft, Desin Height



Note: Passive soil resistance were ignored.

qem - Moist soil pressure at rest wall

qde - Earthquake component

Soil/Water Properties

Layer	Soil Type	Thickness(ft)	Unit Wt. (pcf)	Φ	Ka	Ko	Kp	C (psf)
1	back fill	4.00	125	37	**0.25**	**0.40**	**4.02**	0

Soil Pressure (Soil pressure =γ Ki hi)

Name	Layer	Thickness(ft)	Pressure(ksf)
qem1	Wall	5.50	0.171
qem2	Bot. Footing	1.25	0.210
qde-wall	Wall	5.50	0.067
qde-ftg	Bot. Footing	7.00	0.085

$\alpha = \tan^{-1}[(C_1+\sqrt{(C_1^2+4C_2)})/2]$
(Equation 3-56 of EM 1110-2-2502, Page 3-67)

$C_1 = 2(\tan\Phi - K_h) / (1+K_h \tan\Phi)$
(Equation 3-57 of EM 1110-2-2502, Page 3-67)

$C_2 = [\tan\Phi(1-\tan\Phi\ \tan\beta)-(\tan\beta-K_h)] / [\tan\Phi(1+K_h \tan\Phi)]$
(Equation 3-58 of EM 1110-2-2502, Page 3-67)

Resultant Summary For bottom of wall

Name	Force	Arm to bot.	Moments
Pem1	0.5	1.83	0.9
Pde-wall	0.2	3.67	0.7
At bot of wall	**0.7**		**1.5**
	Σ V		Σ M

◺ Pem1

◸ Pde-wall

K_h =	0.15	g
β =	0	
Φ =	37	degree
C_1 =	1.085	
C_2 =	0.720	
α =	57.2	degree

Demand at Top of Pile: Vd = .65 kips Md = 1.53 k-ft

Dynamic Components $q_{eq} = \gamma K_h h^2 / [2(\tan\alpha - \tan\beta)]$
(Equation 3-62 of EM 1110-2-2502, Page 3-68)

Resultant Summary For Bottom Of Ftg

Name	Force	Arm to bot.	Moments
Pem2	0.7	2.25	1.6
Pde-ftg	0.3	4.67	1.4
At bot of ftg	**1.0**		**3.0**
	Σ V		Σ M

◺ Pem2

◸ Pde-ftg

Demand at Top of Pile: Vd = 1.0 kips Md = 2.98 k-ft

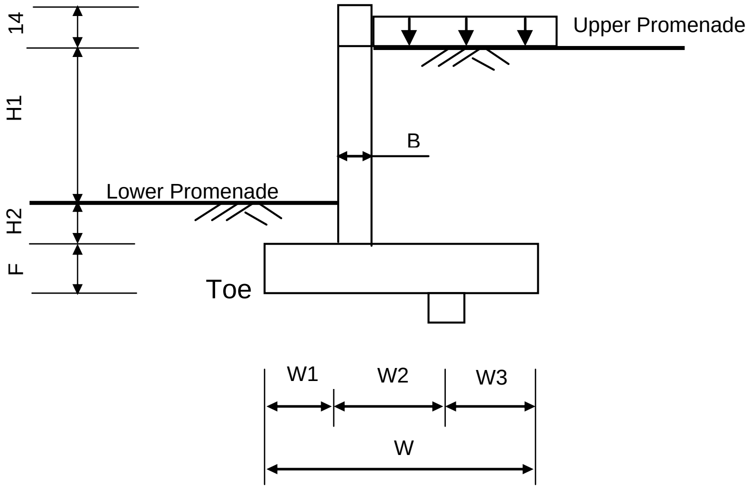
MGE Engineering, Inc.		
Project	Napa River Flood Control Project	
Subject	Ramp Wall Design (H=3'-6")	
By	David An	Date Feb-05

Ramp Wall (RW)
H = 3'-6" , Design Height

1. Loads

A. Due to Soil Pressure

Load Case	2		4	
	Bot. Of Wall	Bot. Of Ftg	Bot. Of Wall	Bot. Of Ftg
Hori. Force (k)	0.8	1.1	0.7	1.0
Moments(kft)	1.4	2.6	1.5	3.0



B =	1.00 ft		H1 =	2.50	ft
W1 =	1.50 ft		H2 =	1.00	ft
W2 =	1.50 ft		F =	1.25	ft
W3 =	1.00 ft		Conc. Unit Wt =	150	pcf
W =	4.00 ft		Backfill Unit Wt =	125	pcf

B. Resistance Due to Weight of Concrete, Soil

Due to concrete and soil

Location	Dimension		Weight(k/ft)	Arm to toe (ft)	Mot (kft/ft)
	Thick/Depth	Width/Height			
Toe cover	1.00	1.50	0.1875	0.75	0.14
Heel Soil	3.50	1.50	0.656	3.25	2.13
Footing	1.25	4.00	0.750	2.00	1.50
Concrete Key	1.00	1.00	0.15	3.00	0.45
Wall Stem	1.00	4.67	0.700	2.00	1.40
Total			**2.44**		**5.62**

- Due to Pedestrain Surcharge

Surcharge = (pedestrain load)	250.00	lbs/sf, UBC
Vertical weight due to the surcharge = (W-W1-B) x Surcharge =	0.38	kips/ft
Resistance of overturning due to pedestrain = Wt. x [W-(W-W1-B)/2]	1.22	kft/ft

2. Overturning Check

Load Case	Resistance Moment	Overturning Moment	Safety Factor	
2	6.84	2.6	2.68	**Safety Factor > 1.5, OK**
4	6.84	3.0	2.30	**Safety Factor > 1.5, OK**

3. Sliding Check

Resistance Due to Soil Friction

μ =	0.3	
Ffr = P_d x μ	0.7	kips/ft
where, P_d -- weight of concrete & soil		

Resistance Due to Pedestrain

0.113 kips/ft

Resistance Due to Concrete Key

Fcr = 2√f'c x 1 ft	18.21	kips/ft
where, f'c =	4000	psi, assumed

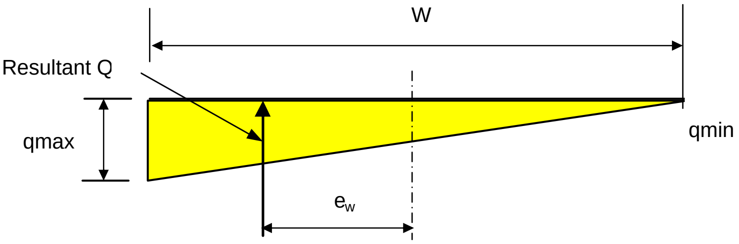
Load Case	Resistance Due to Soil (kips)	Sliding Force (kips)	Safety Factor		Resistance Due to Concrete Key (kips)	Sliding Force (kips)	Safety Factor	
2	0.85	1.1	0.75	**< 1.5, NG**	18.21	1.1	16.06	**> 1.5, OK**
4	0.85	1.0	0.84	**< 1.1, NG**	18.21	1.0	18.13	**> 1.1, OK**

4. Soil Bearing Pressure Check

Moment about center of footing

M = Msmax(due to soil pressure) + Mt(due to toe soil) - Mk(due to key) - Mw(due to wall) - Mh(due to heel soil)

	M =	1.69	kft/ft
where,	Max. M (due to lateral soil pressure), Msmax. =	3.0	kft/ft
	Mt =H2 x 0.125 x (W/2-W1/2)	0.2	kft/ft
	Mk = - (1ft x 1ft x 0.15 x (W/2-W3))	-0.2	kft/ft
	Mw = B x (H1 + H2 + 14/12) x 0.15 x (W/2-B/2-W1)	0.0	kft/ft
	Mh = (H1 + H2)x0.125x(W2 + W3 - B)(W/2-(W2+W3-B)/2)	-0.8	kft/ft
	M_{sur}={(W-W1-B) x qsur x [W/2-(W-W1-B)/2]}	-0.5	kft/ft
Eccentricity	e_W = M / (Pd+Ph15)	0.60	ft
	W / 6 =	0.67	ft
Therefore	e_W < W / 6		
	Footing Contact Pressure		



Q = Pd + Ph15 2.82 kips

1 + (6 e_w / W) = 1.90 ft

1 - (6 e_w / W) = 0.10 ft

qmax = Q [1 + (6ew / W)] / W 1.34 ksf

qmin = Q [1 - (6ew / W)] / W 0.07 ksf

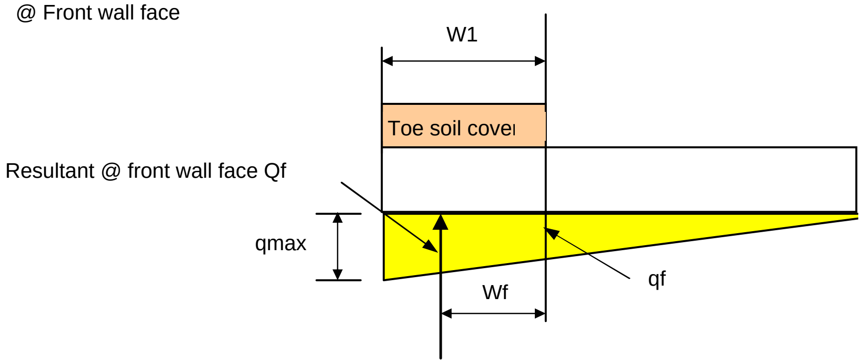
assumed allowable soil brearing pressure

qa = 2 ksf

D / C = qmax / qa = 0.67 **<1, OK**

5. Footing Shear and Moment Check (@ face of wall)

@ Front wall face



qf = (W2+W3) / W x (qmax - qmin) + qmin 0.86 ksf

Qf =(qf+qmax)/2 x W1 1.65 k/ft

Arm to face of wall

Wf = W1 (2qmax+qf) / [3 (qmax+qf)] 0.80 ft

Mu = Hf x 1.7 x [Qf x Wf - (W1x F x 0.15 x W1/2 -W1 x h2 x 0.12 x W1/2)] 2.17 kft/ft

from BDS Table 3.22.1.A

Hf = 1.30

Vu = Hf x 1.7 x [Qf - (W1 x F x 0.15 - W1 x 1ft x 0.12)] 2.63 k/ft

Required bar area for flexure

Mu ≤ ΦMn

ΦMn = Φ As x fy (d - a/2)

Assume a = 0.15d 1.8 in

where d = 12 in

As = Mu / [Φfy(d-a/2)] 0.04 in^2

Φ = 0.9

fy = 60 ksi

Use #4 @12" , As =0.2 in^2 As = 0.2 in^2

Check

a =Asfy / (0.85f'c*b) 0.3 in

ΦMn = Φ Asfy(d - a/2) 10.67 kft/ft

D/C = Mu/ΦMn 0.20 **OK**

Shear Check

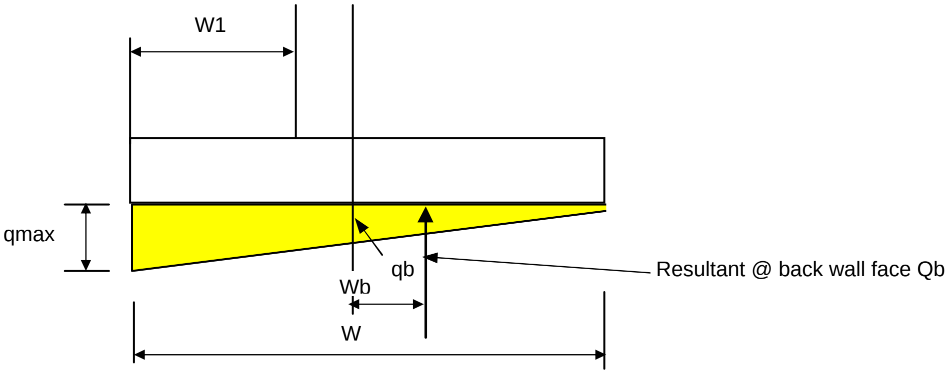
Shear capacity

ΦVn = Φ2√f'c x F 19.35 k/ft

where Φ = 0.85

D/C = Vu/ΦVn 0.14 **OK**

@ Back wall face



qb = (W-W1-B) / W x (qmax + qmin) + qmin 0.55 ksf

Qb =(qb+qmin)/2 x (W-W1-B) 0.46 k/ft

Arm to face of wall

Wb =(W-W1-B) - (W- W1-B) (2qb+qmin) / [3 (qb+qmin)] 0.56 ft

Mu = Hf x 1.7 x[Qb x Wb - (F x 0.15 x (W-W1-B)^2/2 + (H1+H2) x 0.12 x (W-W1-B)/2)-(P_{h15} x 2ft)]

-2.60 kft/ft

Hf = 1.30

Vu = Hf x 1.7 x [Qb - ((W-W1-B) x F x 0.15 + (W-W1-B) x H2 x 0.12) - Ph15] -1.82 k/ft

Required bar area for flexure

Mu ≤ ΦMn

ΦMn = Φ As x fy (d - a/2)

Assume a = 0.15d 1.8 in

where d = F x 12 - 3 12 in

As = Mu / [Φfy(d-a/2)] 0.05 in^2

Φ = 0.9

fy = 60 ksi

Use #4 @12" , As =0.2 in^2 As = 0.2 in^2

Check

a =Asfy / (0.85f'c*b) 0.3 in

ΦMn = Φ Asfy(d - a/2) 10.67 kft/ft

D/C = Mu/ΦMn 0.24 **OK**

Shear Check

Shear capacity

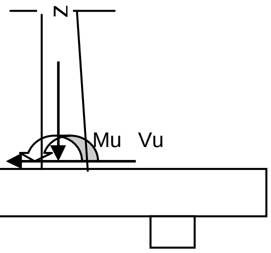
ΦVn = Φ2√f'c x d 15.48 k/ft

where Φ = 0.85

Vu = Hf x 1.7 x Vmax(case2, case4) = 1.7 kips/ft

D/C = Vu/ΦVn 0.12 **OK**

6. Wall Stem Shear and Moment Check (@ bottom of wall)



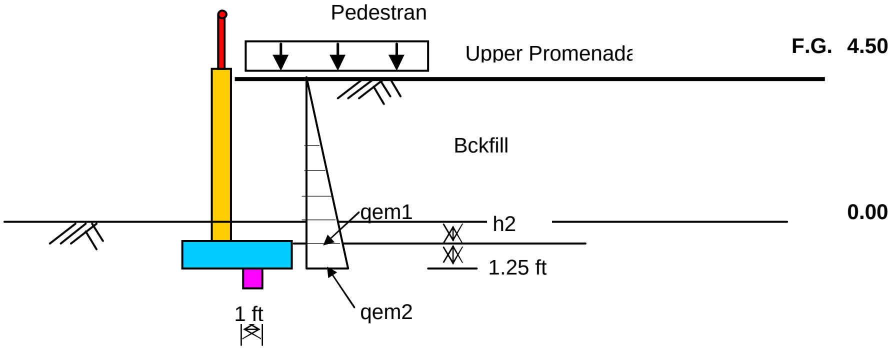
MGE ENGINEERING, INC.	Project	Napa River Flood Control Project	
	Subject	Ramp Wall Design (H=3'-6")	
	By	David An	Date Feb-05

Flexure reinforcement requirement (bending about horizontal axis at bottom of wall)			
Mu ≤ ΦMn			
Mu = Hf x 1.7 x Mmax(case2 , case 4)		3.4	k-ft/ft
Hf = Hydraulic factor		1.30	
EM-1110-2-2104---P3-2, Equation3.3			
Mmax (case2, case4) =		1.5	k-ft/ft
	Φ =	0.90	
	d = F x 12 - 3 =	12.5	in
	b =	12	in
	f'c =	3	ksi
	fy =	60	ksi
Reinforcement Requirement			
As = Mu / [Φfy(d-a/2)], where a = Asfy / (0.85f'c*b)			
ΦMn = Φ As x fy (d - a/2)			
a = 0.15d		1.9	in
Required reinforcement			
As = Mu / [Φfy(d-a/2)]		0.07	in^2
Try #4@12", As = 0.2 in^2		0.20	in^2
a =Asfy / (0.85f'c*b)		0.39	in
ΦMn = Φ Asfy(d - a/2)		11.1	kft/ft
	D/C = Mu/ΦMn	0.31	**OK**
Check Shear			
Vu = Hf x 1.7 x Vmax(case2, case4) =		1.7	kips/ft
ΦVn = Φ(Vc+Vs)		24.6	kips
where			
Vc = 2 x √f'c x b x d		16.4	kips
Vs = As fy d / s		12.5	kips
use #4 @12" as shear reinforcement @ bottom of wall stem			
Where Φ =		0.85	
	D/C = Vu / ΦVn	0.07	**OK**

Load Case 2 -- Normal Condition W/Pedestrin Load (Usual Condition)

No vehecal can access

H = 5.5 ft, Desin Height



Note: Passive soil resistance were ignored.

qem - Moist soil pressure at rest wall

Soil Pressure at Wall (Soil pressure =γ Ki hi)

Name	Layer	Thickness(ft)	Pressure(ksf)
qem1	wall	7.50	0.373
qem2	Bot. Footing	1.25	0.436

Backfill parameter:

unit weight	125	pcf
friction angle, ϕ	37	deg
sin(ϕ)	0.602	
at rest coeff. Ko	0.398	
passive, kp	4.023	
Pedestrian load =	250	lbs/sf, UBC
equivalent surcharge, hp =	2	ft

Thickness(ft) = soil height of backwall + equivalent height of surcharge

Resultant Summary for Bottom of Wall

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem1	1.4	2.50	3.5
At bot of wall	**1.4**		**3.5**
	Σ V		Σ M

Pem2

Demand at Bottom of Wall: Vd = 1.4 kips Md = 3.5 k-ft

Resultant Summary for Bottom of Footing

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem2	1.9	2.92	5.6
At bot of ftg	**1.9**		**5.6**
	Σ V		Σ M

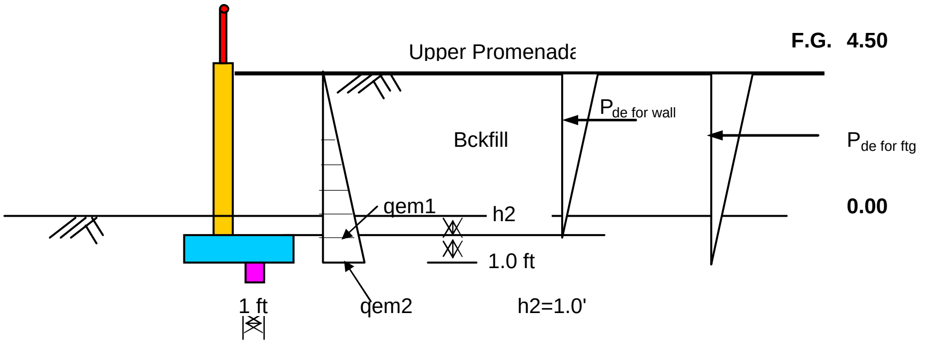
Pem2

Demand at Bottom of Footing: Vd = 1.9 kips Md = 5.6 k-ft

Load Case 4 -- Seimic Condition (Extreme condition)

No vehecal can access

H = 5.5 ft, Desin Height



Note: Passive soil resistance were ignored.

qem - Moist soil pressure at rest wall

qde - Earthquake component

Soil/Water Properties

Layer	Soil Type	Thickness(ft)	Unit Wt. (pcf)	Φ	Ka	Ko	Kp	C (psf)
1	back fill	6.00	125	37	**0.25**	**0.40**	**4.02**	0

Soil Pressure (Soil pressure =γ Ki hi)

Name	Layer	Thickness(ft)	Pressure(ksf)
qem1	Wall	7.50	0.233
qem2	Bot. Footing	1.25	0.272
qde-wall	Wall	7.50	0.091
qde-ftg	Bot. Footing	9.00	0.109

$\alpha = \tan^{-1}[(C_1+\sqrt{(C_1^2+4C_2)})/2]$

(Equation 3-56 of EM 1110-2-2502, Page 3-67)

$C_1 = 2 (\tan\Phi - K_h) / (1+K_h \tan\Phi)$

(Equation 3-57 of EM 1110-2-2502, Page 3-67)

$C_2 = [\tan\Phi(1-\tan\Phi\ \tan\beta)-(\tan\beta-K_h)] / [\tan\Phi(1+K_h \tan\Phi)]$

(Equation 3-58 of EM 1110-2-2502, Page 3-67)

Resultant Summary For bottom of wall

Name	Force	Arm to bot.	Moments
Pem1	0.9	2.50	2.2
Pde-wall	0.3	5.00	1.7
At bot of wall	**1.2**		**3.9**
	Σ V		Σ M

◺ Pem1

◸ Pde-wall

$K_h =$ 0.15 g

$\beta =$ 0

$\Phi =$ 37 degree

$C_1 =$ 1.085

$C_2 =$ 0.720

$\alpha =$ 57.2 degree

Demand at Top of Pile: Vd = 1.21 kips Md = 3.89 k-ft

Dynamic Components $q_{eq} = \gamma K_h h^2 / [2(\tan\alpha - \tan\beta)]$

(Equation 3-62 of EM 1110-2-2502, Page 3-68)

Resultant Summary For Bottom Of Ftg

Name	Force	Arm to bot.	Moments
Pem2	1.2	2.92	3.5
Pde-ftg	0.5	6.00	2.9
At bot of ftg	**1.7**		**6.4**
	Σ V		Σ M

◺ Pem2

◸ Pde-ftg

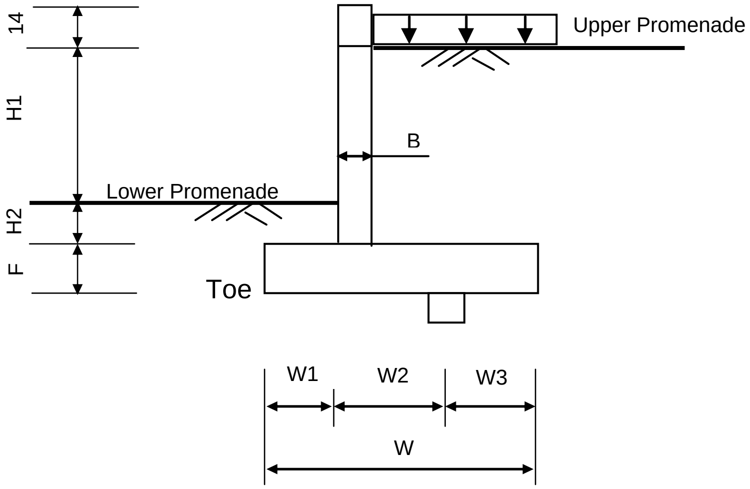
Demand at Top of Pile: Vd = 1.68 kips Md = 6.41 k-ft

Ramp Wall (RW)
H = 5'-6" , Design Height

1. Loads

A. Due to Soil Pressure

Load Case	2		4	
	Bot. Of Wall	Bot. Of Ftg	Bot. Of Wall	Bot. Of Ftg
Hori. Force (k)	1.4	1.9	1.2	1.7
Moments(kft)	3.5	5.6	3.9	6.4



B =	1.00	ft	H1 =	4.50	ft
W1 =	2.25	ft	H2 =	1.00	ft
W2 =	2.25	ft	F =	1.25	ft
W3 =	1.00	ft	Conc. Unit Wt =	150	pcf
W =	5.50	ft	Backfill Unit Wt =	125	pcf

B. Resistance Due to Weight of Concrete, Soil

Due to concrete and soil

Location	Dimension		Weight(k/ft)	Arm to toe (ft)	Mot (kft/ft)
	Thick/Depth	Width/Height			
Toe cover	1.00	2.25	0.28125	1.13	0.32
Heel Soil	5.50	2.25	1.547	4.38	6.77
Footing	1.25	5.50	1.031	2.75	2.84
Concrete Key	1.00	1.00	0.15	4.50	0.68
Wall Stem	1.00	6.67	1.000	2.75	2.75
Total			**4.01**		**13.34**

- Due to Pedestrain Surcharge

Surcharge = (pedestrain load)	250.00	lbs/sf, UBC
Vertical weight due to the surcharge = (W-W1-B) x Surcharge =	0.56	kips/ft
Resistance of overturning due to pedestrain = Wt. x [W-(W-W1-B)/2]	2.46	kft/ft

2. Overturning Check

Load Case	Resistance Moment	Overturning Moment	Safety Factor	
2	15.81	5.6	2.84	**Safety Factor > 1.5, OK**
4	15.81	6.4	2.47	**Safety Factor > 1.5, OK**

3. Sliding Check

Resistance Due to Soil Friction

μ = 0.3

Ffr = P_d x μ 1.2 kips/ft

where, P_d -- weight of concrete & soil

Resistance Due to Pedestrain

0.169 kips/ft

Resistance Due to Concrete Key

Fcr = 2√f'c x 1 ft 18.21 kips/ft

where, f'c = 4000 psi, assumed

Load Case	Resistance Due to Soil (kips)	Sliding Force (kips)	Safety Factor		Resistance Due to Concrete Key (kips)	Sliding Force (kips)	Safety Factor	
2	1.37	1.9	0.72	**< 1.5, NG**	18.21	1.9	9.56	**> 1.5, OK**
4	1.37	1.7	0.82	**< 1.1, NG**	18.21	1.7	10.84	**> 1.1, OK**

4. Soil Bearing Pressure Check

Moment about center of footing

M = Msmax(due to soil pressure) + Mt(due to toe soil) - Mk(due to key) - Mw(due to wall) - Mh(due to heel soil)

M = 2.92 kft/ft

where, Max. M (due to lateral soil pressure), Msmax. = 6.4 kft/ft

Mt =H2 x 0.125 x (W/2-W1/2) 0.2 kft/ft

Mk = - (1ft x 1ft x 0.15 x (W/2-W3)) -0.3 kft/ft

Mw = B x (H1 + H2 + 14/12) x 0.15 x (W/2-B/2-W1) 0.0 kft/ft

Mh = (H1 + H2)x0.125x(W2 + W3 - B)(W/2-(W2+W3-B)/2) -2.5 kft/ft

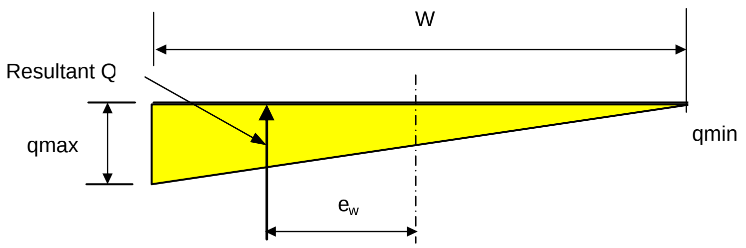
M_{sur}={(W-W1-B) x qsur x [W/2-(W-W1-B)/2]} -0.9 kft/ft

Eccentricity e_W = M / (Pd+Ph15) 0.64 ft

W / 6 = 0.92 ft

Therefore e_W < W / 6

Footing Contact Pressure



Q = Pd + Ph15 4.57 kips

1 + (6 e_w / W) = 1.70 ft

1 - (6 e_w / W) = 0.30 ft

qmax = Q [1 + (6ew / W)] / W 1.41 ksf

qmin = Q [1 - (6ew / W)] / W 0.25 ksf

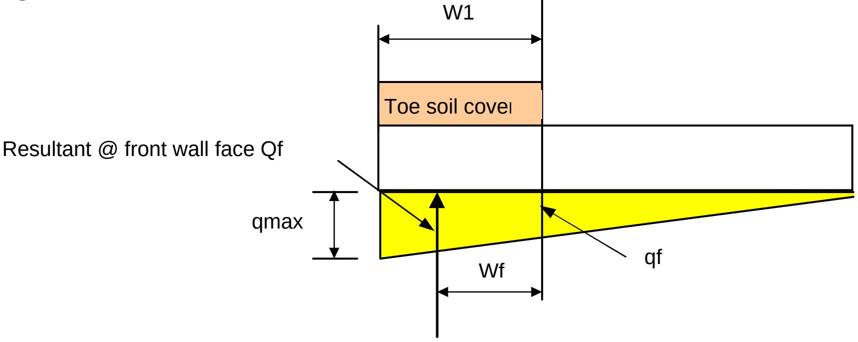
assumed allowable soil brearing pressure

qa = 2 ksf

D / C = qmax / qa = 0.71 **<1, OK**

5. Footing Shear and Moment Check (@ face of wall)

@ Front wall face



qf = (W2+W3) / W x (qmax - qmin) + qmin 0.94 ksf

Qf =(qf+qmax)/2 x W1 2.64 k/ft

Arm to face of wall

Wf = W1 (2qmax+qf) / [3 (qmax+qf)] 1.20 ft

Mu = Hf x 1.7 x [Qf x Wf - (W1x F x 0.15 x W1/2 -W1 x h2 x 0.12 x W1/2)] 5.29 kft/ft

from BDS Table 3.22.1.A

Hf = 1.30

Vu = Hf x 1.7 x [Qf - (W1 x F x 0.15 - W1 x 1ft x 0.12)] 4.31 k/ft

Required bar area for flexure

Mu ≤ ΦMn

ΦMn = Φ As x fy (d - a/2)

Assume a = 0.15d 1.8 in

where d = 12 in

As = Mu / [Φfy(d-a/2)] 0.11 in^2

Φ = 0.9

fy = 60 ksi

Use #4 @12" , As =0.2 in^2 As = 0.2 in^2

Check

a =Asfy / (0.85f'c*b) 0.3 in

ΦMn = Φ Asfy(d - a/2) 10.67 kft/ft

D/C = Mu/ΦMn 0.50 **OK**

Shear Check

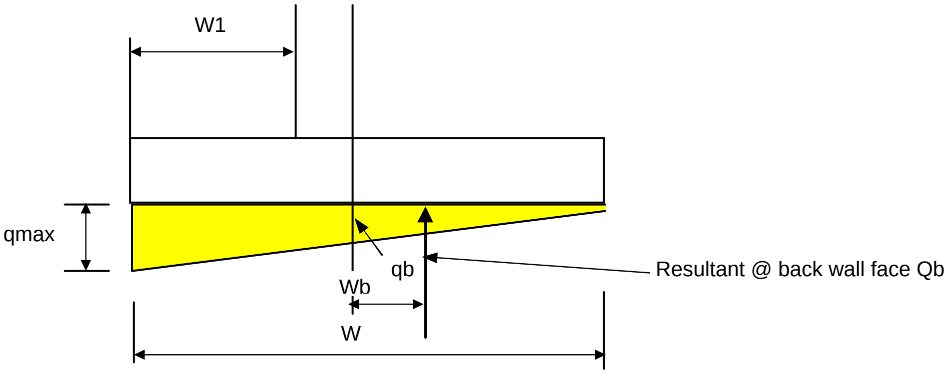
Shear capacity

ΦVn = Φ2√f'c x F 19.35 k/ft

where Φ = 0.85

D/C = Vu/ΦVn 0.22 **OK**

@ Back wall face



qb = (W-W1-B) / W x (qmax + qmin) + qmin 0.73 ksf

Qb =(qb+qmin)/2 x (W-W1-B) 1.10 k/ft

Arm to face of wall

Wb =(W-W1-B) - (W- W1-B) (2qb+qmin) / [3 (qb+qmin)] 0.94 ft

Mu = Hf x 1.7 x[Qb x Wb - (F x 0.15 x (W-W1-B)^2/2 + (H1+H2) x 0.12 x (W-W1-B)/2)-(P_{h15} x 2ft)]

-4.94 kft/ft

Hf = 1.30

Vu = Hf x 1.7 x [Qb - ((W-W1-B) x F x 0.15 + (W-W1-B) x H2 x 0.12) - Ph15] -3.03 k/ft

Required bar area for flexure

Mu ≤ ΦMn

ΦMn = Φ As x fy (d - a/2)

Assume a = 0.15d 1.8 in

where d = F x 12 - 3 12 in

As = Mu / [Φfy(d-a/2)] 0.10 in^2

Φ = 0.9

fy = 60 ksi

Use #4 @12" , As =0.2 in^2 As = 0.2 in^2

Check

a =Asfy / (0.85f'c*b) 0.3 in

ΦMn = Φ Asfy(d - a/2) 10.67 kft/ft

D/C = Mu/ΦMn 0.46 **OK**

Shear Check

Shear capacity

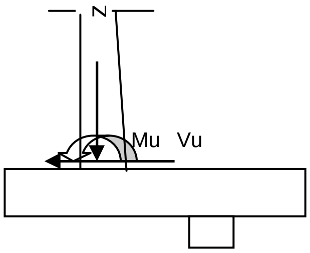
ΦVn = Φ2√f'c x d 15.48 k/ft

where Φ = 0.85

Vu = Hf x 1.7 x Vmax(case2, case4) = 3.1 kips/ft

D/C = Vu/ΦVn 0.20 **OK**

6. Wall Stem Shear and Moment Check (@ bottom of wall)



MGE ENGINEERING, INC.		
Project	Napa River Flood Control Project	
Subject	Ramp Wall Design (H=5'-6")	
By	David An	Date Feb-05

Flexure reinforcement requirement (bending about horizontal axis at bottom of wall)

Mu ≤ ΦMn

Mu = Hf x 1.7 x Mmax(case2 , case 4) 8.6 k-ft/ft

Hf = Hydraulic factor 1.30

EM-1110-2-2104---P3-2, Equation3.3

Mmax (case2, case4) = 3.9 k-ft/ft

Φ = 0.90

d = F x 12 - 3 = 12.5 in

b = 12 in

f'c = 3 ksi

fy = 60 ksi

Reinforcement Requirement

As = Mu / [Φfy(d-a/2)], where a = Asfy / (0.85f'c*b)

ΦMn = Φ As x fy (d - a/2)

a = 0.15d 1.9 in

Required reinforcement

As = Mu / [Φfy(d-a/2)] 0.17 in^2

Try #5@12", As = 0.31 in^2 x 12" / 12" 0.31 in^2

a =Asfy / (0.85f'c*b) 0.61 in

ΦMn = Φ Asfy(d - a/2) 17.0 kft/ft

D/C = Mu/ΦMn 0.50 **OK**

Check Shear

Vu = Hf x 1.7 x Vmax(case2, case4) = 3.1 kips/ft

ΦVn = Φ(Vc+Vs) 24.6 kips

where

Vc = 2 x √f'c x b x d 16.4 kips

Vs = As fy d / s 12.5 kips

use #4 @12" as shear reinforcement @ bottom of wall stem

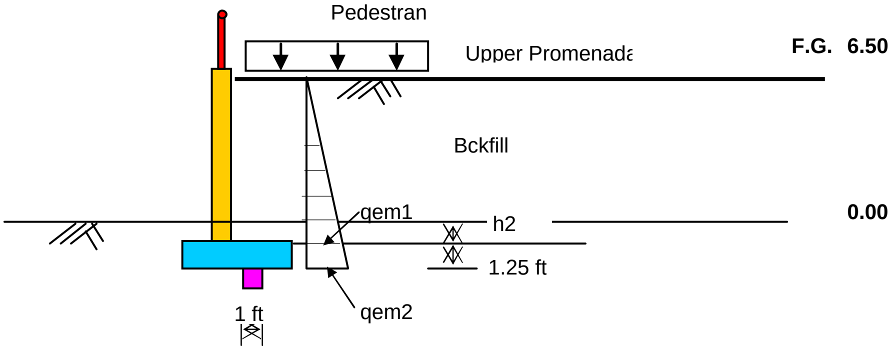
Where Φ = 0.85

D/C = Vu / ΦVn 0.13 **OK**

Load Case 2 -- Normal Condition W/Pedestrin Load (Usual Condition)

No vehecal can access

H = 7.5 ft, Desin Height



Note: Passive soil resistance were ignored.

qem - Moist soil pressure at rest wall

Soil Pressure at Wall (Soil pressure =γ Ki hi)

Name	Layer	Thickness(ft)	Pressure(ksf)
qem1	wall	9.50	0.473
qem2	Bot. Footing	1.25	0.535

Backfill parameter:

unit weight	125	pcf
friction angle, ϕ	37	deg
sin(ϕ)	0.602	
at rest coeff. Ko	0.398	
passive, kp	4.023	
Pedestrian load =	250	lbs/sf, UBC
equivalent surcharge, hp =	2	ft

Thickness(ft) = soil height of backwall + equivalent height of surcharge

Resultant Summary for Bottom of Wall

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem1	2.2	3.17	7.1
At bot of wall	**2.2**		**7.1**
	Σ V		Σ M

Pem2

Demand at Bottom of Wall: Vd = 2.2 kips Md = 7.1 k-ft

Resultant Summary for Bottom of Footing

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem2	2.9	3.58	10.3
At bot of ftg	**2.9**		**10.3**
	Σ V		Σ M

Pem2

Demand at Bottom of Footing: Vd = 2.9 kips Md = 10.3 k-ft

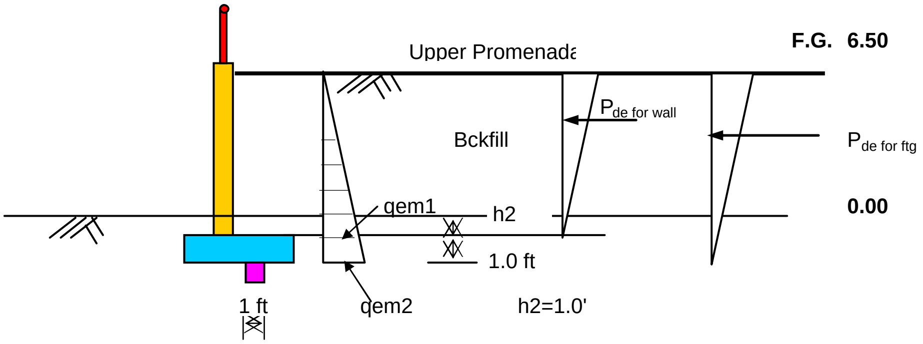
MGE ENGINEERING, INC.

Project	Napa River Flood Control Project
Subject	Ramp Wall Design (H=7'-6")
By	David An
Date	Feb-05

Load Case 4 -- Seimic Condition (Extreme condition)

No vehecal can access

H = 7.5 ft, Desin Height



Note: Passive soil resistance were ignored.

qem - Moist soil pressure at rest wall

qde - Earthquake component

Soil/Water Properties

Layer	Soil Type	Thickness(ft)	Unit Wt. (pcf)	Φ	Ka	Ko	Kp	C (psf)
1	back fill	8.00	125	37	**0.25**	**0.40**	**4.02**	0

Soil Pressure (Soil pressure =γ Ki hi)

Name	Layer	Thickness(ft)	Pressure(ksf)
qem1	Wall	9.50	0.295
qem2	Bot. Footing	1.25	0.334
qde-wall	Wall	9.50	0.115
qde-ftg	Bot. Footing	11.00	0.133

$\alpha = \tan^{-1}[(C_1+\sqrt{(C_1^2+4C_2)})/2]$

(Equation 3-56 of EM 1110-2-2502, Page 3-67)

$C_1 = 2\ (\tan\Phi - K_h)\ /\ (1+K_h \tan\Phi)$

(Equation 3-57 of EM 1110-2-2502, Page 3-67)

$C_2 = [\tan\Phi(1-\tan\Phi\ \tan\beta)-(\tan\beta-K_h)]\ /\ [\tan\Phi(1+K_h \tan\Phi)]$

(Equation 3-58 of EM 1110-2-2502, Page 3-67)

Resultant Summary For bottom of wall

Name	Force	Arm to bot.	Moments
Pem1	1.4	3.17	4.4
Pde-wall	0.5	6.33	3.5
At bot of wall	**1.9**		**7.9**
	Σ V		Σ M

◺ Pem1

◸ Pde-wall

$K_h =$ 0.15 g

$\beta =$ 0

$\Phi =$ 37 degree

$C_1 =$ 1.085

$C_2 =$ 0.720

$\alpha =$ 57.2 degree

Demand at Top of Pile: Vd = 1.95 kips Md = 7.90 k-ft

Dynamic Components $q_{eq} = \gamma\ K_h h^2\ /\ [2(\tan\alpha - \tan\beta)]$

(Equation 3-62 of EM 1110-2-2502, Page 3-68)

Resultant Summary For Bottom Of Ftg

Name	Force	Arm to bot.	Moments
Pem2	1.8	3.58	6.4
Pde-ftg	0.7	7.33	5.4
At bot of ftg	**2.5**		**11.8**
	Σ V		Σ M

◺ Pem2

◸ Pde-ftg

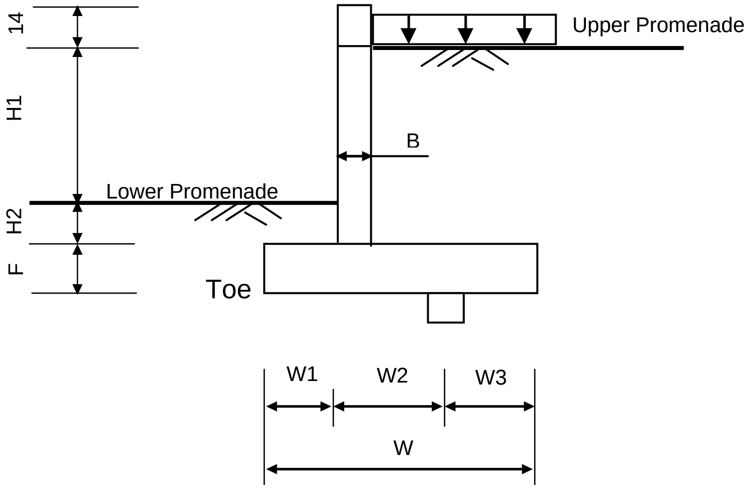
Demand at Top of Pile: Vd = 2.53 kips Md = 11.80 k-ft

Ramp Wall (RW)
H = 7'-6" , Design Height

1. Loads

A. Due to Soil Pressure

Load Case	2		4	
	Bot. Of Wall	Bot. Of Ftg	Bot. Of Wall	Bot. Of Ftg
Hori. Force (k)	2.2	2.9	1.9	2.5
Moments(kft)	7.1	10.3	7.9	11.8



B =	1.00	ft	H1 =	6.50	ft
W1 =	3.25	ft	H2 =	1.00	ft
W2 =	3.25	ft	F =	1.25	ft
W3 =	1.00	ft	Conc. Unit Wt =	150	pcf
W =	7.50	ft	Backfill Unit Wt =	125	pcf

B. Resistance Due to Weight of Concrete, Soil

Due to concrete and soil

Location	Dimension		Weight(k/ft)	Arm to toe (ft)	Mot (kft/ft)
	Thick/Depth	Width/Height			
Toe cover	1.00	3.25	0.40625	1.63	0.66
Heel Soil	7.50	3.25	3.047	5.88	17.90
Footing	1.25	7.50	1.406	3.75	5.27
Concrete Key	1.00	1.00	0.15	6.50	0.98
Wall Stem	1.00	8.67	1.300	3.75	4.88
Total			**6.31**		**29.68**

- Due to Pedestrain Surcharge

Surcharge = (pedestrain load)	250.00	lbs/sf, UBC
Vertical weight due to the surcharge = (W-W1-B) x Surcharge =	0.81	kips/ft
Resistance of overturning due to pedestrain = Wt. x [W-(W-W1-B)/2]	4.77	kft/ft

MGE ENGINEERING, INC.

Project	Napa River Flood Control Project		
Subject	Ramp Wall Design (H=7'-6")		
By	David An	Date	Feb-05

2. Overturning Check

Load Case	Resistance Moment	Overturning Moment	Safety Factor	
2	34.46	10.3	3.34	**Safety Factor > 1.5, OK**
4	34.46	11.8	2.92	**Safety Factor > 1.5, OK**

3. Sliding Check

Resistance Due to Soil Friction

μ =	0.3	
Ffr = P_d x μ	1.9	kips/ft

where, P_d -- weight of concrete & soil

Resistance Due to Pedestrain

0.244 kips/ft

Resistance Due to Concrete Key

Fcr = 2√f'c x 1 ft	18.21	kips/ft
where, f'c =	4000	psi, assumed

Load Case	Resistance Due to Soil (kips)	Sliding Force (kips)	Safety Factor		Resistance Due to Concrete Key (kips)	Sliding Force (kips)	Safety Factor	
2	2.14	2.9	0.74	**< 1.5, NG**	18.21	2.9	6.33	**> 1.5, OK**
4	2.14	2.5	0.85	**< 1.1, NG**	18.21	2.5	7.21	**> 1.1, OK**

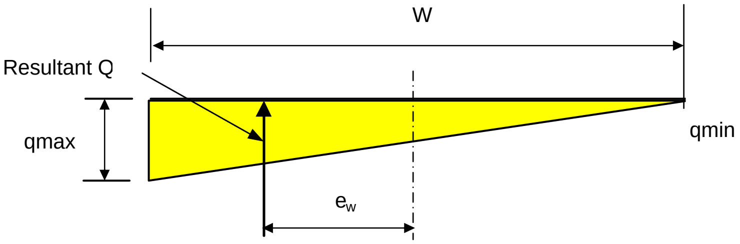
4. Soil Bearing Pressure Check

Moment about center of footing

M = Msmax(due to soil pressure) + Mt(due to toe soil) - Mk(due to key) - Mw(due to wall) - Mh(due to heel soil)

	M =	3.46	kft/ft
where,	Max. M (due to lateral soil pressure), Msmax. =	11.8	kft/ft
	Mt =H2 x 0.125 x (W/2-W1/2)	0.3	kft/ft
	Mk = - (1ft x 1ft x 0.15 x (W/2-W3))	-0.4	kft/ft
	Mw = B x (H1 + H2 + 14/12) x 0.15 x (W/2-B/2-W1)	0.0	kft/ft
	Mh = (H1 + H2)x0.125x(W2 + W3 - B)(W/2-(W2+W3-B)/2)	-6.5	kft/ft
	M_{sur}={(W-W1-B) x qsur x [W/2-(W-W1-B)/2]}	-1.7	kft/ft

Eccentricity	e_W = M / (Pd+Ph15)	0.49	ft
	W / 6 =	1.25	ft
Therefore	e_W < W / 6		
	Footing Contact Pressure		



Q = Pd + Ph15 7.12 kips

1 + (6 e_w / W) = 1.39 ft

1 - (6 e_w / W) = 0.61 ft

qmax = Q [1 + (6ew / W)] / W 1.32 ksf

qmin = Q [1 - (6ew / W)] / W 0.58 ksf

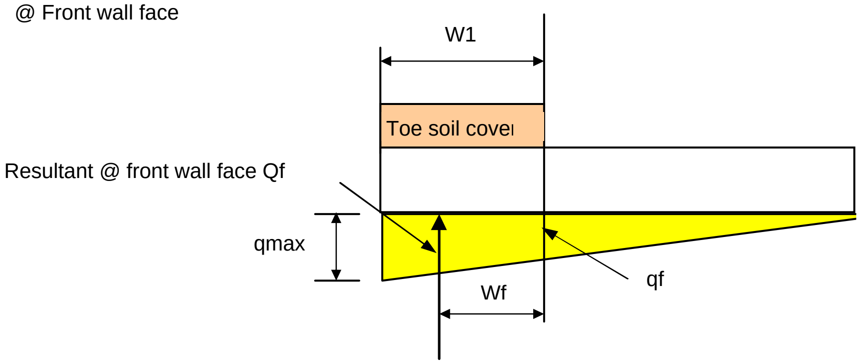
assumed allowable soil brearing pressure

qa = 2 ksf

D / C = qmax / qa = 0.66 **<1, OK**

5. Footing Shear and Moment Check (@ face of wall)

@ Front wall face



qf = (W2+W3) / W x (qmax - qmin) + qmin 1.00 ksf

Qf =(qf+qmax)/2 x W1 3.77 k/ft

Arm to face of wall

Wf = W1 (2qmax+qf) / [3 (qmax+qf)] 1.70 ft

Mu = Hf x 1.7 x [Qf x Wf - (W1x F x 0.15 x W1/2 -W1 x h2 x 0.12 x W1/2)] 10.55 kft/ft

from BDS Table 3.22.1.A

Hf = 1.30

Vu = Hf x 1.7 x [Qf - (W1 x F x 0.15 - W1 x 1ft x 0.12)] 6.11 k/ft

Required bar area for flexure

Mu ≤ ΦMn

ΦMn = Φ As x fy (d - a/2)

Assume a = 0.15d 1.8 in

where d = 12 in

As = Mu / [Φfy(d-a/2)] 0.21 in^2

Φ = 0.9

fy = 60 ksi

Use #5 @12" , As =0.31 in^2 As = 0.31 in^2

Check

a =Asfy / (0.85f'c*b) 0.5 in

ΦMn = Φ Asfy(d - a/2) 16.42 kft/ft

D/C = Mu/ΦMn 0.64 **OK**

Shear Check

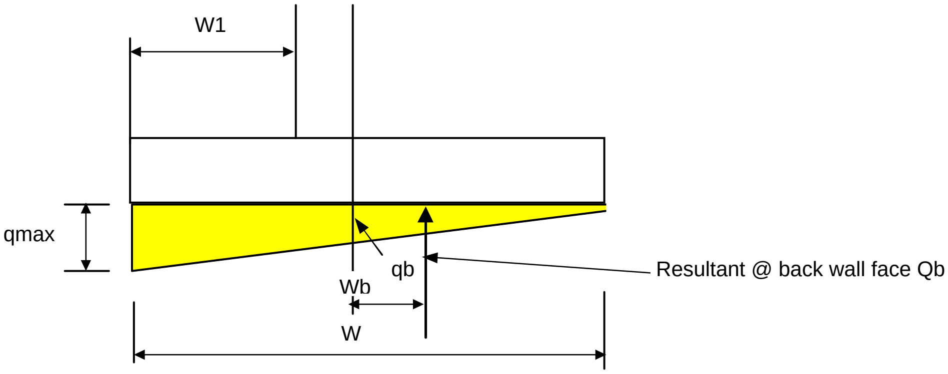
Shear capacity

ΦVn = Φ2√f'c x F 19.35 k/ft

where Φ = 0.85

D/C = Vu/ΦVn 0.32 **OK**

@ Back wall face



qb = (W-W1-B) / W x (qmax + qmin) + qmin 0.90 ksf

Qb =(qb+qmin)/2 x (W-W1-B) 2.41 k/ft

Arm to face of wall

Wb =(W-W1-B) - (W- W1-B) (2qb+qmin) / [3 (qb+qmin)] 1.51 ft

Mu = Hf x 1.7 x[Qb x Wb - (F x 0.15 x (W-W1-B)^2/2 + (H1+H2) x 0.12 x (W-W1-B)/2)-(P_{h15} x 2ft)]

-8.26 kft/ft

Hf = 1.30

Vu = Hf x 1.7 x [Qb - ((W-W1-B) x F x 0.15 + (W-W1-B) x H2 x 0.12) - Ph15] -4.29 k/ft

Required bar area for flexure

Mu ≤ ΦMn

ΦMn = Φ As x fy (d - a/2)

Assume a = 0.15d 1.8 in

where d = F x 12 - 3 12 in

As = Mu / [Φfy(d-a/2)] 0.17 in^2

Φ = 0.9

fy = 60 ksi

Use #5 @12" , As =0.31 in^2 As = 0.31 in^2

Check

a =Asfy / (0.85f'c*b) 0.5 in

ΦMn = Φ Asfy(d - a/2) 16.42 kft/ft

D/C = Mu/ΦMn 0.50 **OK**

Shear Check

Shear capacity

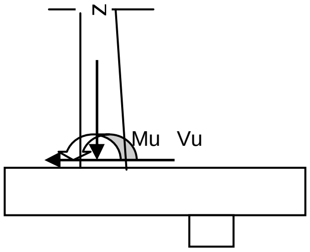
ΦVn = Φ2√f'c x d 15.48 k/ft

where Φ = 0.85

Vu = Hf x 1.7 x Vmax(case2, case4) = 5.0 kips/ft

D/C = Vu/ΦVn 0.28 **OK**

6. Wall Stem Shear and Moment Check (@ bottom of wall)



Flexure reinforcement requirement (bending about horizontal axis at bottom of wall)

Mu ≤ ΦMn

Mu = Hf x 1.7 x Mmax(case2 , case 4) 17.5 k-ft/ft

Hf = Hydraulic factor 1.30

EM-1110-2-2104---P3-2, Equation3.3

Mmax (case2, case4) = 7.9 k-ft/ft

Φ = 0.90

d = F x 12 - 3 = 12.5 in

b = 12 in

f'c = 3 ksi

fy = 60 ksi

Reinforcement Requirement

As = Mu / [Φfy(d-a/2)], where a = Asfy / (0.85f'c*b)

ΦMn = Φ As x fy (d - a/2)

a = 0.15d 1.9 in

Required reinforcement

As = Mu / [Φfy(d-a/2)] 0.34 in^2

Try #7@12", As = 0.6 in^2 0.60 in^2

a =Asfy / (0.85f'c*b) 1.18 in

ΦMn = Φ Asfy(d - a/2) 32.2 kft/ft

D/C = Mu/ΦMn 0.54 **OK**

Check Shear

Vu = Hf x 1.7 x Vmax(case2, case4) = 5.0 kips/ft

ΦVn = Φ(Vc+Vs) 24.6 kips

where

Vc = 2 x √f'c x b x d 16.4 kips

Vs = As fy d / s 12.5 kips

use #4 @12" as shear reinforcement @ bottom of wall stem

Where Φ = 0.85

D/C = Vu / ΦVn 0.20 **OK**

Project	Napa River Flood Control Project		
Subject	Veterans Park Walls Design		
By	Guoping Xu	Date	Feb-05

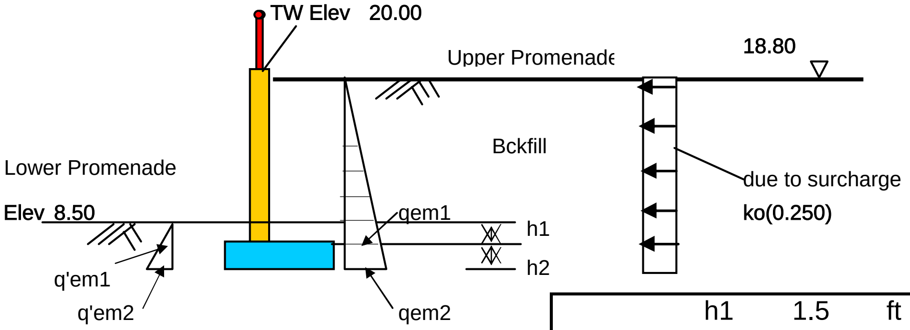
VETERANS PARK WALLS DESIGN

Veterans Park (VP) Wall No.1

Load Case 1: Construction Condition (Unusual Condition)

Load cases are based on "Flood Walls and Channel Retaining Walls, Load Conditions & Load Diagrams" with the exception that D-4 not applicable in this location due to the width of access. Instead, use surcharge load for driveways as public access, see UBC Table 16A-A

Surcharge =	250	psf	
Elevation of Upper Promenade for design	18.80		approximate
Lower promenade	8.50	Finish Grade	approximate



h1	1.5	ft
h2	1.5	ft

Soil Pressure at Wall (Soil pressure =γ Ki hi)

Name	Layer	Thickness(ft)	Pressure(ksf)
qem1	Backfill	11.80	0.587
qem2	Backfill	1.50	0.662
q'em1	Backfill	1.50	0.754
q'em2	Backfill	1.50	1.509

Backfill parameter:

unit weight	125	pcf
friction angle, ϕ	37	deg
sin(ϕ)	0.602	
at rest coeff. Ko	0.398	
passive, kp	4.023	

Resultant Force at Bottom of Wall, per foot of wall

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem1	3.5	3.93	13.6
Surcharge	1.2	5.90	6.9
P'em1	-0.57	0.50	-0.28
At bot of wall	**4.1**		**20.3**
	Σ H		Σ M

Force at Bottom of Wall: Hd = 4.1 kips Md = 20.3 k-ft

Resultant Force at Bottom of Footing, per foot of wall

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem2	4.4	4.43	19.5
Surcharge	1.3	6.65	8.8
P'em2	-2.26	1.00	-2.3
At bot of ftg	**3.5**		**26.1**
	Σ V		Σ M

Force at Bottom of Footing: Hd = 3.5 kips Md = 26.1 k-ft

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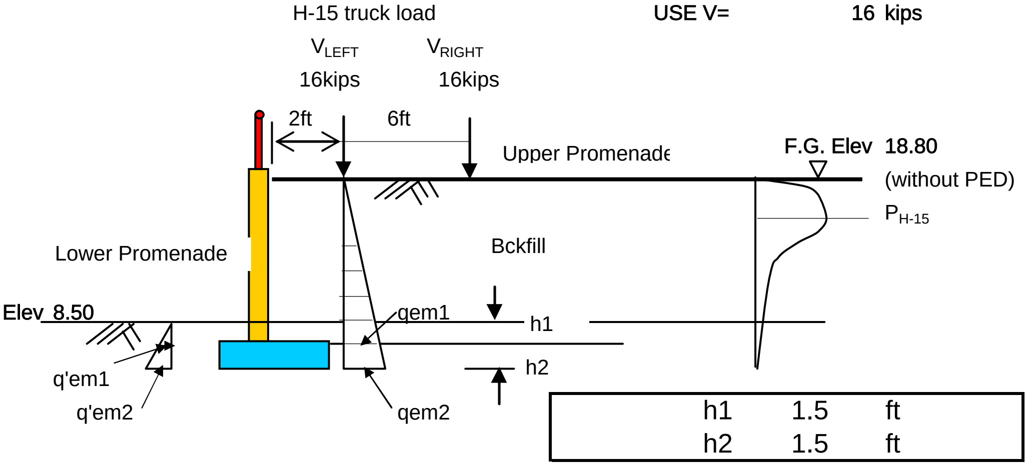
Load Case 2: Normal Condition with Maintenance Vehicle (Usual Condition)



USE V= 16 kips

h1	1.5	ft
h2	1.5	ft

Soil Pressure at Wall (Soil pressure =γ Ki hi)

Name	Layer	Thickness(ft)	Pressure(ksf)
qem1	Backfill	11.80	0.587
qem2	Backfill	1.50	0.662
q'em1	Backfill	1.50	0.754
q'em2	Backfill	1.50	1.509

Backfill parameter:

unit weight	125	pcf
friction angle, φ	37	deg
sin(φ)	0.602	
at rest coeff. Ko	0.398	
passive, kp	4.023	

H-15 Load---- **For bottom of footing**

h = 13.30 ft

a=2'/13.30' = 0.15 ≤ 0.4 **For V_{LEFT}**

a=8'/13.30' = 0.60 > 0.4 **For V_{RIGHT}**

$\Delta P_{HZleft} = (0.28V/h^2)\ [b^2 / (0.16+b^2)^3]$ (EM 1110-2-2502 Page 3-48)

$\Delta P_{HZright} = (V/h^2)\ [a^2b^2 / (a^2+b^2)^3]$ (EM 1110-2-2502 Page 3-48)

b (for V_{LEFT})	Z	$\Delta P_{PH\ (LEFT)}$	Moment
0.1	1.33	0.069	0.821
0.2	2.66	0.168	1.792
0.3	3.99	0.194	1.806
0.4	5.32	0.164	1.313
0.5	6.65	0.122	0.813
0.6	7.98	0.086	0.459
0.7	9.31	0.060	0.240
0.8	10.64	0.042	0.112
0.9	11.97	0.030	0.040
1.0	13.30	0.022	0.000
Σ		0.958	7.394

b (for V_{RIGHT})	Z	$\Delta P_{PH\ (RIGHT)}$	Moment
0.1	1.33	0.001	0.008
0.2	2.66	0.002	0.025
0.3	3.99	0.005	0.044
0.4	5.32	0.007	0.059
0.5	6.65	0.010	0.065
0.6	7.98	0.011	0.061
0.7	9.31	0.013	0.050
0.8	10.64	0.013	0.035
0.9	11.97	0.013	0.017
1.0	13.30	0.012	0.000
Σ		0.087	0.363

Resultant Force at Bottom of Wall

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem1	3.5	3.93	13.6
$P_{H-15\ wall}$	1.0		6.2
P'em1	-0.57	0.50	-0.28
At bot of wall	**3.9**		**19.5**
	Σ V		Σ M

Force at Bottom of Wall: Vd = 3.9 kips Md = 19.5 k-ft

Resultant Force at Bottom of Footing

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem2	4.4	4.43	19.5
$P_{H-15\ ftg}$	1.0		7.8
P'em2	-2.26	1.00	-2.3
At bot of ftg	**3.2**		**25.0**
	Σ V		Σ M

Force at Bot Footing: Vd = 3.2 kips, Md = 25.0 k-ft

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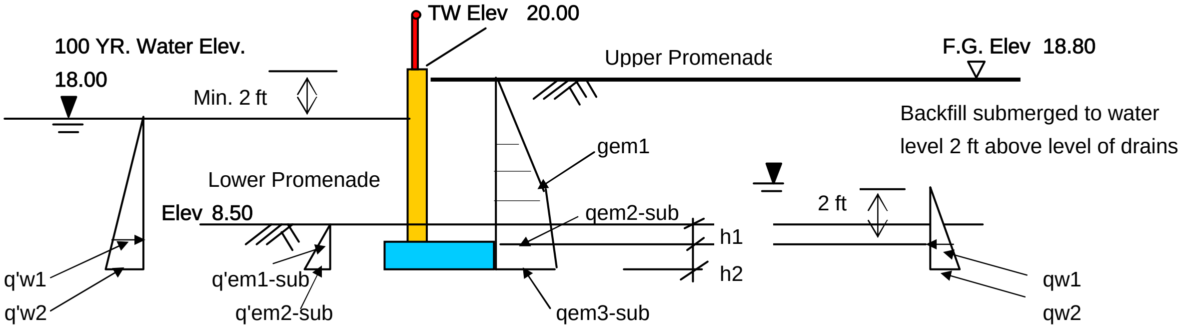
Load Case 3a: Design Flood Condition (Unusual Condition)



h1	1.5	ft
h2	1.5	ft

Soil Pressure at Wall (Soil pressure =γ Ki hi)

Name	Layer	Thickness(ft)	Pressure(ksf)
qem1	Backfill	8.30	0.413
qem2-sub	Backfill	3.50	0.500
qem3-sub	Backfill	1.50	0.538
qw1	Backfill	3.50	0.219
qw2	Backfill	1.50	0.313
q'em1-sub	Backfill	1.50	0.377
q'em2-sub	Backfill	1.50	0.754
q'w1	Backfill	11.00	0.688
q'w2	Backfill	1.50	0.781

Backfill parameter:

Dry		
unit weight	125	pcf
friction angle, ϕ	37	Deg
sin(ϕ)	0.602	
at rest coeff. Ko	0.398	
passive, kp	4.023	
Submerged		
unit weight	62.5	pcf
Water		
unit weight	62.5	pcf

Resultant Force at Bottom Of Wall

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem1	1.7	6.27	10.74
Pem2-sub	1.6	1.69	2.71
Pw1	0.4	1.17	0.45
P'em1-sub	-0.28	0.50	-0.14
P'w1	-3.78	0.50	-1.9
At bot of wall	**-0.37**		**11.9**
	Σ V		Σ M

Force at Bottom of Wall: Vd = -.37 kips Md = 11.87 k-ft

Resultant Force at Bottom Of Footing

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem1	1.7	7.77	13.32
Pem2-sub	1.6	3.19	5.11
Pem3-sub	0.8	0.74	0.58
Pw1	0.38	2.67	1.02
Pw2	0.4	0.71	0.28
P'em1-sub	-0.28	2.00	-0.57
P'em2-sub	-0.8	0.67	-0.57
P'w1	-3.78	3.67	-13.9
P'w2	-1.10	0.73	-0.8
At bot of ftg	**-1.14**		**4.5**
	Σ V		Σ M

Force at Bottom of Wall: Vd = -1.14 kips Md = 4.5 k-ft

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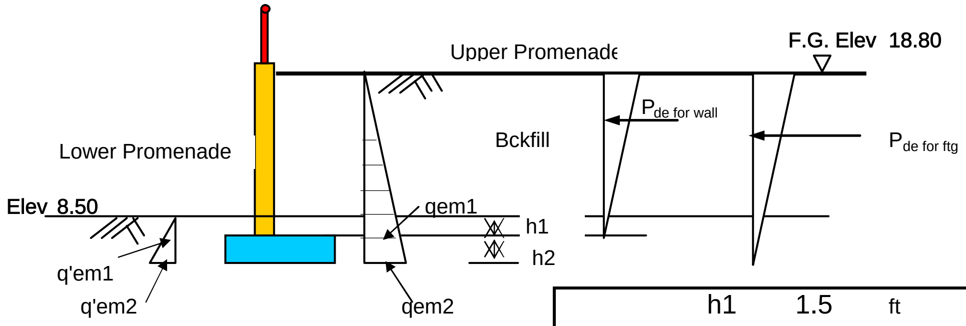
Load Case 4: Seismic Condition (Extreme Condition)



h1	1.5	ft
h2	1.5	ft

Soil/Water Properties

Layer	Soil Type	Thickness(ft)	Unit Wt. (pcf)	Φ	Ka	Ko	Kp
1	back fill	11.80	125	37	0.25	0.40	4.02
Water			62.5				

Soil Pressure (Soil pressure = γ Ki hi)

Name	Layer	Thickness(ft)	Pressure(ksf)
qem1	Backfill	11.80	0.367
qem2	Backfill	1.50	0.413
q'em1	Backfill	1.50	0.754
q'em2	Backfill	1.50	1.509
qde-wall	Backfill	11.80	0.143
qde-ftg	Backfill	13.30	0.161

$\alpha = \tan^{-1}[(C_1+\sqrt{(C_1^2+4C_2)})/2]$

(Equation 3-56 of EM 1110-2-2502, Page 3-67)

$C_1 = 2\ (\tan\Phi - K_h)\ /\ (1+K_h \tan\Phi)$

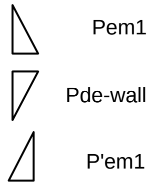
(Equation 3-57 of EM 1110-2-2502, Page 3-67)

$C_2 = [\tan\Phi(1-\tan\Phi\ \text{tar}\beta)-(\tan\beta-K_h)]\ /\ [\tan\Phi(1+K_h \tan\Phi)]$

(Equation 3-58 of EM 1110-2-2502, Page 3-67)

Resultant Summary at bottom of wall

Name	Force	Arm to bot.	Moments
Pem1	2.2	3.93	8.5
Pde-wall	0.8	7.87	6.6
P'em1	-0.57	0.50	-0.28
At bot of wall	**2.4**		**14.9**
	Σ V		Σ M



K_h =	0.15	g
β =	0	
Φ =	37	degree
C_1 =	1.085	
C_2 =	0.720	
α =	57.2	degree

Force at bottom of wall: Vd = 2.44 kips Md = 14.86 k-ft

of which earthquake contribution (E)

E = 0.8 kips 6.6 k-ft

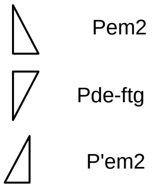
ie. D+L = 1.6 kips 8.2 k-ft

Dynamic Components $q_{eq} = \gamma\ K_h h^2\ /\ [2(\tan\alpha-\tan\beta)]$

(Equation 3-62 of EM 1110-2-2502, Page 3-68)

Resultant Summary at Bottom Of Ftg

Name	Force	Arm to bot.	Moments
Pem2	2.7	4.43	12.2
Pde-ftg	1.1	8.87	9.5
P'em2	-2.26	1.00	-2.26
At bot of ftg	**1.6**		**19.4**
	Σ V		Σ M



Force at bottom of ftg: Vd = 1.56 kips Md = 19.41 k-ft

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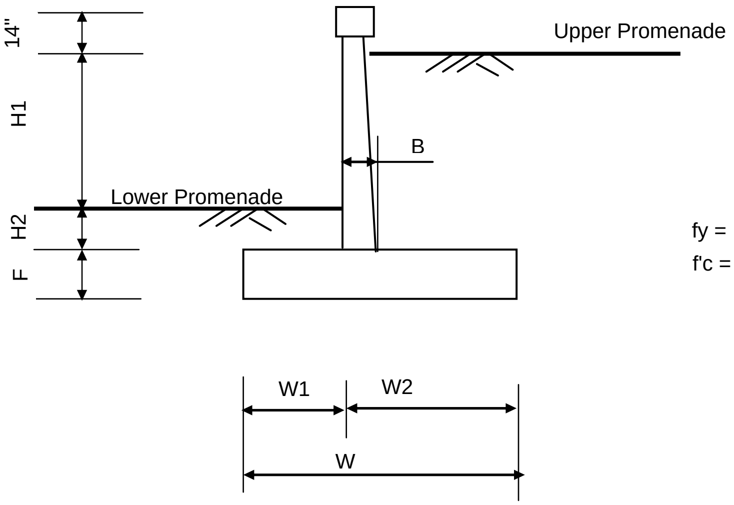
Veterans Park (VP)
Wall No. 1

1. Loads

A. Due to Lateral Pressure

Load Case	1		2		3		4	
	Bot/ Wall	Bot/Ftg	Bot/ Wall	Bot/Ftg	Bot/ Wall	Bot/Ftg	Bot/ Wall	Bot/Ftg
Hori Force(k)	4.1	3.5	3.9	3.2	-0.4	-1.1	2.4	1.6
Moment(kft)	20.3	26.1	19.5	25.0	11.9	4.5	14.9	19.4



fy = 60000 psi

f'c = 4000 psi

B =	1.50 ft		H1 =	10.30	ft
W1 =	4.00 ft		H2 =	1.50	ft
			F =	1.50	ft
W2 =	6.50 ft		Conc. Unit Wt =	150	pcf
W =	10.50 ft		Backfill Unit Wt =	125	pcf

B. Resistance against overturning due to weight of concrete & soil

Due to concrete and soil

Location	Dimension		Weight(k/ft)	Arm to toe (ft)	Mot (kft/ft)
	Thick/Depth	Width/Height			
Toe cover	1.50	4.00	0.75	2.00	1.50
Heel Soil	11.80	5.00	7.375	8.00	59.00
Footing	1.50	10.50	2.363	5.25	12.40
Concrete Key	0.00	0.00	0	0.00	0.00
Wall Stem	1.50	12.97	2.918	4.75	13.86
Total			13.41		86.76

Note: Live load is not included in the resistance to be conservative

2. Overturning Check

Load Case	Resistance Moment	Overturning Moment	Safety Factor	
1	86.76	26.1	3.33	Safety Factor > 1.5, OK
2	86.76	25.0	3.47	Safety Factor > 1.5, OK
3a*	86.76	4.5	19.30	Safety Factor > 1.5, OK
4	86.76	19.4	4.47	Safety Factor > 1.5, OK

*Dry soil wt is used here instead of submerged soil wt, not critical either way, Ok.

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3. Sliding Check

Resistance Due to Soil Friction

$\mu =$ use tan ($2\phi/3$) 0.46

Ffr = $P_d \times \mu$ 6.16 kips/ft

where, P_d -- weight of concrete & soil

Resistance due to passive pressure in front of wall is ignored.

Load Case	Resistance Due to Soil (kips)	Sliding Force (kips)	Safety Factor	
1	6.16	3.5	1.78	> 1.33, OK
2	6.16	3.2	1.93	> 1.5, OK
3a*	6.16	-1.1	5.39	> 1.5, OK
4	6.16	1.6	3.96	> 1.1, OK

* Dry weight is used here where part of it should be submerged wt, however it's not critical in this case.

4. Soil Bearing Pressure Check (Allowable bearing pressure used: 2.0 ksf, see Geotech DDR)

Load Case 1

		Moment due to vert load		M due to Hori	Net Moment	N/(W*1)	$M/(1*W^2/6)$	Net Press
	Vert (k/ft)	M (to Toe)	M (to Ctr)	M o.t.		Press due	Press due	q1
Soil & Conc	13.41	-86.76	-16.39			to vert load	to Moment	**1.80**
						(ksf)	(ksf)	q2
Sum	**13.41**		-16.39	26.1	**9.7**	1.28	0.53	**0.75**

Notes:

1 Postive moment : Anti-clockwise

Percentage in compression 100%

Require percentage 75% OK

Load Case 2

		Moment due to vertical load		M due to Horiz	Net Moment	N/(W*1)	$M/(1*W^2/6)$	Net Press
	Vert (k/ft)	M (to Toe)	M (to Ctr)	M o.t.		Press due	Press due	q1
Soil & Conc	13.41	-86.76	-16.39			to vert load	to Moment	**1.66**
H-15	0.51	-5.08	-2.42			(ksf)	(ksf)	q2
Sum	**13.91**		-18.80	25.0	**6.2**	1.33	0.34	**0.99**

Notes : 1 Postive moment : Anti-clockwise

2 Resistance due to live load is set to zero

3 Wt of H15 truck

Width of H15 distribution area = 2 x (H1+H2) 23.6 ft

Distributed weight, P_{h-15} = 0.51 k/ft

Percentage in compression 100%

Required percentage 100%

Load Case 3a

		Moment due to vertical load		M due to Horiz	Net Moment	N/(W*1)	$M/(1*W^2/6)$	Net Press
	Vert (k/ft)	M (W/R Toe)	M (W/R Ctr)	Mot		Press due	Press due	q1
Soil & Conc	13.41	-86.76	-16.39			to vert load	to Moment	**0.63**
						(ksf)	(ksf)	q2
Sum	**13.41**		-16.39	4.5	**-11.9**	1.28	-0.65	**1.92**

Note: Postive moment : Anti-clockwise

Percentage in compression 100%

Ok

Load Case 4

		Moment due to vertical load		M due to Horiz	Net Moment	N/(W*1)	$M/(1*W^2/6)$	Net Press
	Vert (k/ft)	M (W/R Toe)	M (W/R Ctr)	Mot		Press due	Press due	q1
Soil & Conc	13.41	-86.76	-16.39			to vert load	to Moment	**1.44**
						(ksf)	(ksf)	q2
Sum	**13.41**		-16.39	19.4	**3.0**	1.28	0.16	**1.11**

Notes Postive moment : Anti-clockwise

Percentage in compression 100%

Ok

Strength Design per EM 1110-2-2502, Sect 9-8. Eq.(9-5) and (9-6)

Load Case 1:	U=1.9(D+L)
Load Cases 2 & 3:	U=0.75*1.9 (D+L)
Load Case 4:	U=0.75*1.9 (D+E)

where : D = Dead load
L = Live load, including Surcharge or H-15
E = Earthquake load

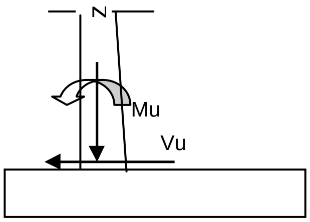
Moment at Bottom of wall Mu=

	D+L	D+E	U	
L.C. 1	20.3		38.5	<---Critical
L.C. 2	19.5		27.8	
L.C. 3	11.9		16.9	
L.C. 4		14.9	21.2	

Shear at bottom of wall Vu=

	D+L	E	U	
L.C. 1	4.1		7.7	<---Critical
L.C. 2	3.9		5.6	
L.C. 3	-0.4		-0.5	
L.C. 4		2.4	3.5	

5. Wall Stem Shear and Moment Check



Mu = max{load case1, load case2, load case 3, load case4}	38.5	kft/ft
Vu = max{load case1, load case2, load case 3, load case4}	7.7	k/ft

Check Moment

	Φ =	0.9	
	fy =	60000	psi
	f'c =	4000	psi
Use #8 @12"	As =	0.79	in^2
	b =	12.0	in (per foot of wall)
	d =	14.5	in
steel ratio	ρ = As/bd =	0.0045	in
ΦMn = Φ*fy*ρ [1 -fy*ρ/(1.7*f'c)] $b*d^2$		49.5	kft/ft
D/C = Mu/ΦMn		0.78	OK

Check Shear

Shear capacity			
ΦVn = Φ2√f'c x B		23.22	k/ft
	where Φ =	0.85	
D/C = Vu/ΦVn		0.33	OK

Concrete shear strength alone is adaquate, no shear reinforcement is required.

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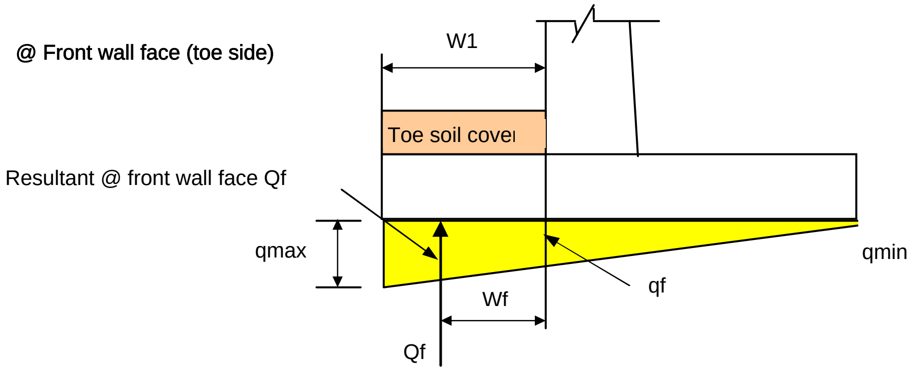
By	G.Xu	Date	Jan 2005

6. Footing Shear and Moment Check (@ face of wall)

Load Case 1 governs the strength design

@ Front wall face (toe side)



Critical load case: Case 1 (most severe bearing pressure for footing in front of wall)

qf = (W2) / W x (qmax - qmin) + qmin 1.15 ksf

Qf =(qf+qmax)/2 x W1 5.91 k/ft

Vu = 1.9 (D+L) = 1.9 Qf 11.23 k/ft

Arm to face of wall

Wf = W1 (2qmax+qf) / [3 (qmax+qf)] 2.15 ft

Mu = Vu*Wf 24.1 kft/ft

Moment Check

Φ = 0.9

fy = 60000 psi

fc = 4000 psi

Use #6 @12" As = 0.44 in^2

b = 12.0 in (per foot of wall)

d = 14.6 in

steel ratio ρ = As/bd = 0.0025 in

ΦMn = Φ*fy*ρ [1 -fy*ρ/(1.7*f'c)] b*d^2 28.3 kft/ft

D/C = Mu/ΦMn 0.85 OK

Shear Check say ok

Shear capacity

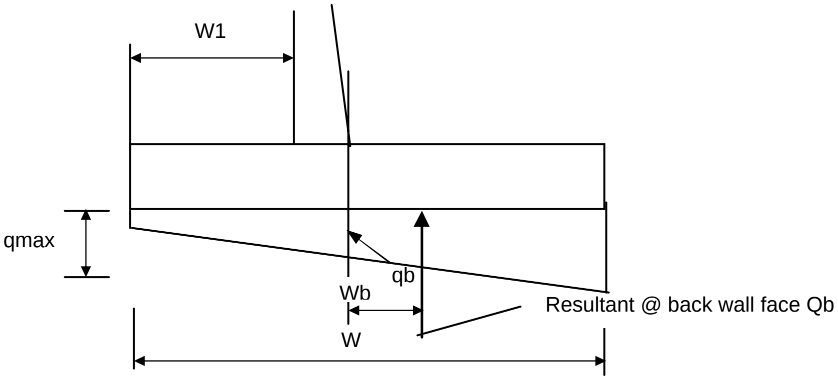
ΦVn = Φ2√f'c x F 23.22 k/ft

where Φ = 0.85

D/C = Vu/ΦVn 0.48 OK

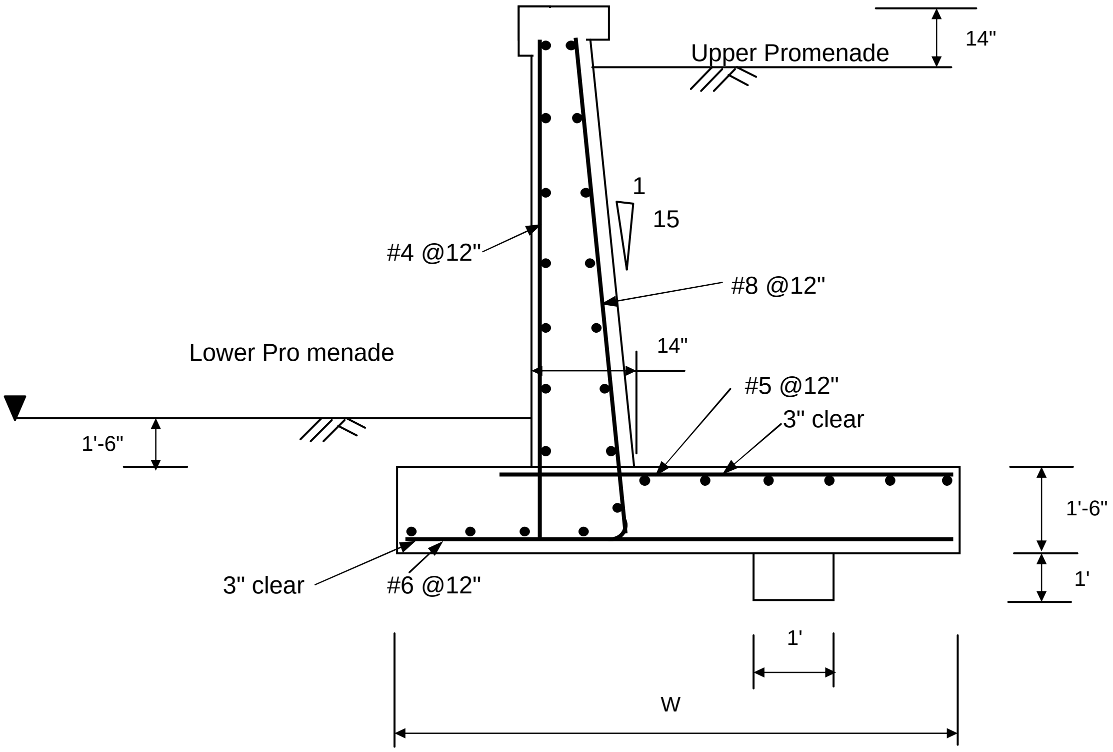
@ Back wall face

Critical Case: 3a



qb = (W-W1-B) / W x (qmax-qmiin) + qmin	1.55	ksf
Qb =(qf+qmin)/2 x (W-W1-B)	5.44	k/ft
Vu = 1.7 Qb	9.24	k/ft
Arm to face of wall		
Wb =(W- W1-B) (2qmax+qb) / [3 (qmax+qb)]	2.59	ft
Mu= Vu*Wb	24.0	kft/ft
Moment check		
Φ =	0.9	
fy =	60000	ksi
f'c=	4000	psi
Use #6 @12" As =	0.44	in^2
b =	12.0	in (per foot of wall)
d =	14.6	in
steel ratio ρ = As/bd =	0.0025	in
ΦMn = Φ*fy*ρ [1 -fy*ρ/(1.7*f'c)] b*d^2	28.9	kft/ft
Shear Check D/C = Mu/ΦMn	0.83	OK
Shear capacity		
ΦVn = Φ2√f'c x F	23.22	k/ft
where Φ =	0.85	
D/C = Vu/ΦVn	0.40	OK

Cross section for VP Wall No.1



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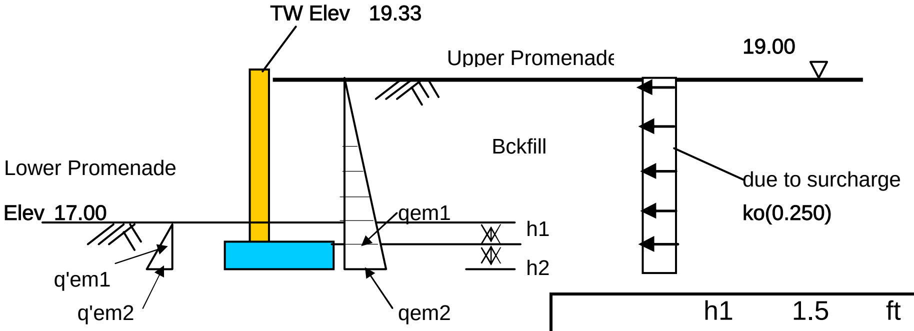
Subject	Veterans Park Walls		
By	G.Xu	Date	Jan 2005

Veterans Park (VP) Walls No.2 & 3

Load Case 1: Construction Condition (Unusual Condition)

Load cases are based on "Flood Walls and Channel Retaining Walls, Load Conditions & Load Diagrams" with the exception that D-4 not applicable in this location due to the width of access. Instead, use surcharge load for driveways as public access, see UBC Table 16A-A

Surcharge =	250	psf
Elevation of Upper Promenade for design	18.80	approximate
Lower promenade, Finished Grade	8.50	approximate



h1	1.5	ft
h2	1.0	ft

Soil Pressure at Wall (Soil pressure =γ Ki hi)

Name	Layer	Thickness(ft)	Pressure(ksf)
qem1	Backfill	3.50	0.174
qem2	Backfill	1.00	0.224
q'em1	Backfill	1.50	0.754
q'em2	Backfill	1.00	1.257

Backfill parameter:

unit weight	125	pcf
friction angle, ϕ	37	deg
sin(ϕ)	0.602	
at rest coeff. Ko	0.398	
passive, kp	4.023	

Resultant Force at Bottom of Wall, per foot of wall

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem1	0.3	1.17	0.4
Surcharge	0.3	1.75	0.6
P'em1	-0.57	0.50	-0.28
At bot of wall	**0.1**		**0.7**
	Σ H		Σ M

Force at Bottom of Wall: Hd = .1 kips Md = .7 k-ft

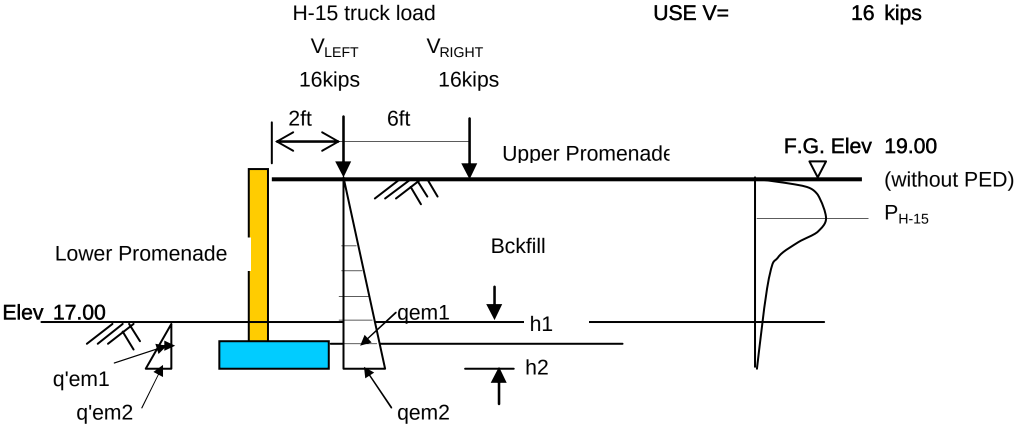
Resultant Force at Bottom of Footing, per foot of wall

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem2	0.5	1.50	0.8
Surcharge	0.4	2.25	1.0
P'em2	-1.57	0.83	-1.3
At bot of ftg	**-0.6**		**0.5**
	Σ V		Σ M

Force at Bottom of Footing: Hd = -.6 kips Md = .5 k-ft

Load Case 2: Normal Condition with Maintenance Vehicle (Usual Condition)

USE V= 16 kips



h1	1.5	ft
h2	1.0	ft

This load case not applicable for walls in this area due to limited access

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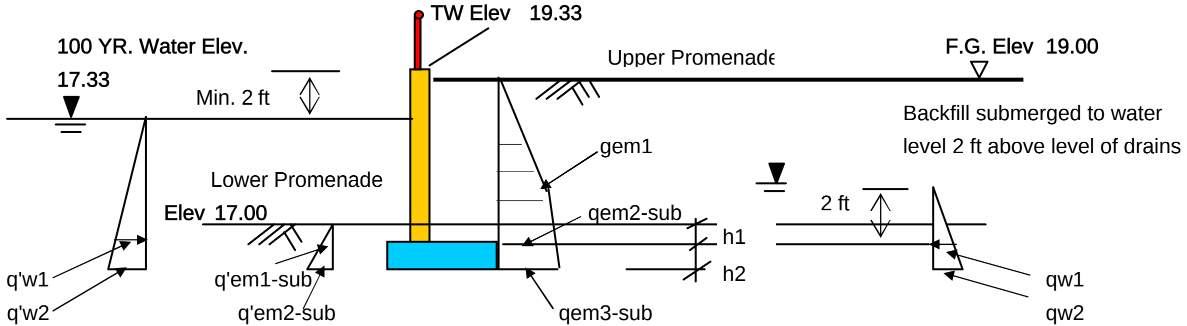
Subject	Veterans Park Walls		
By	G.Xu	Date	Jan 2005

Load Case 3a: Design Flood Condition (Unusual Condition)



h1	1.5	ft
h2	1.0	ft

Soil Pressure at Wall (Soil pressure =γ Ki hi)

Name	Layer	Thickness(ft)	Pressure(ksf)
qem1	Backfill	0.00	0.000
qem2-sub	Backfill	3.50	0.087
qem3-sub	Backfill	1.50	0.124
qw1	Backfill	3.50	0.219
qw2	Backfill	1.50	0.313
q'em1-sub	Backfill	1.50	0.377
q'em2-sub	Backfill	1.50	0.754
q'w1	Backfill	1.83	0.114
q'w2	Backfill	1.50	0.208

Backfill parameter:

Dry		
unit weight	125	pcf
friction angle, ϕ	37	Deg
sin(ϕ)	0.602	
at rest coeff. Ko	0.398	
passive, kp	4.023	
Submerged		
unit weight	62.5	pcf
Water		
unit weight	62.5	pcf

Resultant Force at Bottom Of Wall

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem1	0.0	3.50	0.00
Pem2-sub	0.2	1.17	0.18
Pw1	0.4	1.17	0.45
P'em1-sub	-0.28	0.50	-0.14
P'w1	-0.10	0.50	-0.1
At bot of wall	**0.15**		**0.4**
	Σ V		Σ M

Force at Bottom of Wall: Vd = .15 kips Md = .43 k-ft

Resultant Force at Bottom Of Footing

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem1	0.0	5.00	0.00
Pem2-sub	0.2	2.67	0.41
Pem3-sub	0.2	0.71	0.11
Pw1	0.38	2.67	1.02
Pw2	0.4	0.71	0.28
P'em1-sub	-0.28	2.00	-0.57
P'em2-sub	-0.8	0.67	-0.57
P'w1	-0.10	0.61	-0.1
P'w2	-0.24	0.68	-0.2
At bot of ftg	**-0.39**		**0.5**
	Σ V		Σ M

Force at Bottom of Wall: Vd = -.39 kips Md = .46 k-ft

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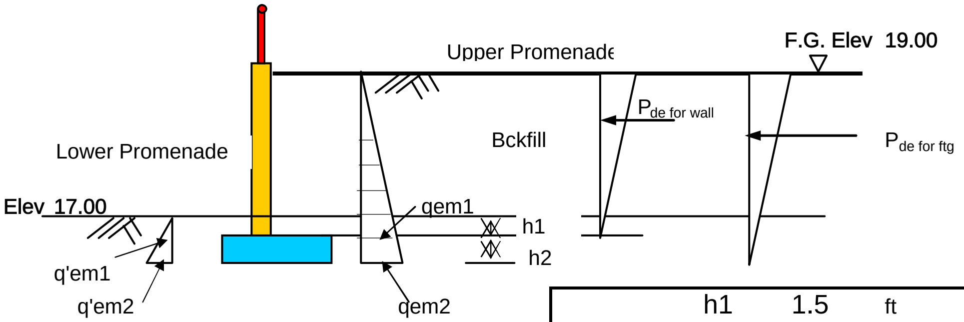
	By G.Xu	Date Jan 2005	

Load Case 4: Seismic Condition (Extreme Condition)



h1	1.5	ft
h2	1.0	ft

Soil/Water Properties

Layer	Soil Type	Thickness(ft)	Unit Wt. (pcf)	Φ	Ka	Ko	Kp
1	back fill	3.50	125	37	**0.25**	**0.40**	**4.02**
Water			62.5				

Soil Pressure (Soil pressure =γ Ki hi)

Name	Layer	Thickness(ft)	Pressure(ksf)
qem1	Backfill	3.50	0.109
qem2	Backfill	1.50	0.155
q'em1	Backfill	1.50	0.754
q'em2	Backfill	1.50	1.509
qde-wall	Backfill	3.50	0.042
qde-ftg	Backfill	5.00	0.061

$\alpha = \tan^{-1}[(C_1+\sqrt{(C_1^2+4C_2)})/2]$

(Equation 3-56 of EM 1110-2-2502, Page 3-67)

$C_1 = 2\ (\tan\Phi - K_h)\ /\ (1+K_h \tan\Phi)$

(Equation 3-57 of EM 1110-2-2502, Page 3-67)

$C_2 = [\tan\Phi(1-\tan\Phi\ \mathrm{tar}\beta)-(\tan\beta-K_h)]\ /\ [\tan\Phi(1+K_h \tan\Phi)]$

(Equation 3-58 of EM 1110-2-2502, Page 3-67)

Resultant Summary at bottom of wall

Name	Force	Arm to bot.	Moments
Pem1	0.2	1.17	0.2
Pde-wall	0.1	2.33	0.2
P'em1	-0.57	0.50	-0.28
At bot of wall	**-0.3**		**0.1**
	Σ V		Σ M

Pem1

Pde-wall

P'em1

K_h =	0.15	g
β =	0	
Φ =	37	degree
C_1 =	1.085	
C_2 =	0.720	
α =	57.2	degree

Force at bottom of wall: Vd = -.30 kips Md = .11 k-ft (See Note)

of which earthquake contribution (E)

E =	0.1	kips	0.2	k-ft
ie. D+L =	-0.4	kips	-0.1	k-ft

Dynamic Components $q_{eq} = \gamma\ K_h h^2\ /\ [2(\tan\alpha-\tan\beta)]$

(Equation 3-62 of EM 1110-2-2502, Page 3-68)

Resultant Summary at Bottom Of Ftg

Name	Force	Arm to bot.	Moments
Pem2	0.4	1.67	0.6
Pde-ftg	0.2	3.33	0.5
P'em2	-2.26	1.00	-2.26
At bot of ftg	**-1.7**		**-1.1**
	Σ V		Σ M

Pem2

Pde-ftg

P'em2

Note:
When the sum of force is negative, indicating passive force greater than active force which is not possible, set the sum of force to be zero, i.e,

$\underline{\Sigma V = 0, \quad \text{and} \quad \Sigma M = 0}$

Force at bottom of ftg: Vd = -1.72 kips Md = -1.11 k-ft

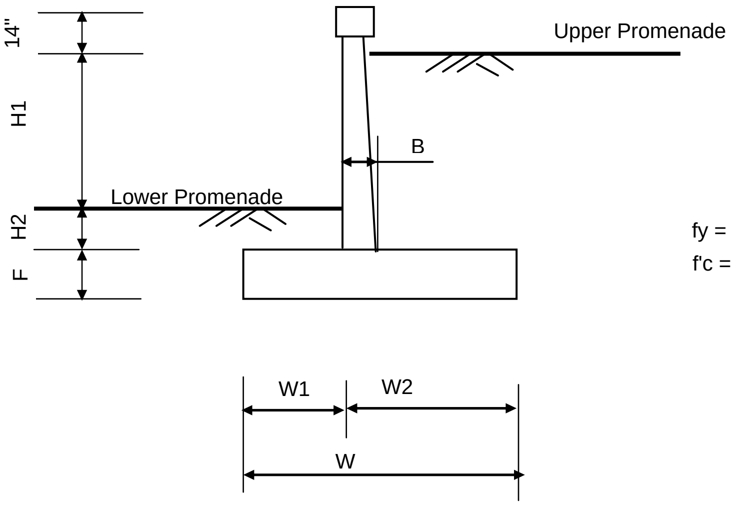
Use 0 0 (See Note)

Veterans Park (VP)
Walls No. 2 & 3

1. Loads

A. Due to Lateral Pressure

Load Case	1		2		3		4	
	Bot/ Wall	Bot/Ftg	Bot/ Wall	Bot/Ftg	Bot/ Wall	Bot/Ftg	Bot/ Wall	Bot/Ftg
Hori Force(k)	0.1	-0.6	N/A	N/A	0.1	-0.4	-0.3	0.0
Moment(kft)	0.7	0.5	N/A	N/A	0.4	0.5	0.1	0.0



fy = 60000 psi
f'c = 4000 psi

B = 1.50 ft
W1 = 1.00 ft
W2 = 2.00 ft
W = 3.00 ft

H1 = 2.00 ft
H2 = 1.50 ft
F = 1.00 ft
Conc. Unit Wt = 150 pcf
Backfill Unit Wt = 125 pcf

B. Resistance against overturning due to weight of concrete & soil

Due to concrete and soil

Location	Dimension		Weight(k/ft)	Arm to toe (ft)	Mot (kft/ft)
	Thick/Depth	Width/Height			
Toe cover	1.50	1.00	0.1875	0.50	0.09
Heel Soil	3.50	0.50	0.219	2.75	0.60
Footing	1.00	3.00	0.450	1.50	0.68
Concrete Key	0.00	0.00	0	0.00	0.00
Wall Stem	1.50	4.67	1.050	1.75	1.84
Total			1.91		3.21

Note: Live load is not included in the resistance to be conservative

2. Overturning Check

Load Case	Resistance Moment	Overturning Moment	Safety Factor	
1	3.21	0.5	7.06	Safety Factor > 1.5, OK
2	3.21	N/A	N/A	N/A
3a*	3.21	0.5	6.95	Safety Factor > 1.5, OK
4	3.21	0.0	N/A	See note for Loadcase 4

(Previous page)

*Dry soil wt is used here instead of submerged soil wt, not critical either way, Ok.

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	By	G.Xu	Date Jan 2005

3. Sliding Check

Resistance Due to Soil Friction

μ = use tan (2ϕ/3) 0.46

Ffr = P_d x μ 0.88 kips/ft

where, P_d -- weight of concrete & soil

Resistance due to passive pressure in front of wall is ignored.

Load Case	Resistance Due to Soil (kips)	Sliding Force (kips)	Safety Factor	
1	0.88	-0.6	1.41	> 1.33, OK
2	0.88	N/A	N/A	N/A
3a*	0.88	-0.4	2.27	> 1.5, OK
4	0.88	0.0	N/A	N/A

See note for Loadcase 4 (Previous page)

* Dry weight is used here where part of it should be submerged wt, however it's not critical in this case.

4. Soil Bearing Pressure Check

(Allowable bearing pressure used: 2.0 ksf, see Geotech DDR)

Load Case 1

		Moment due to vert load		M due to Hori	Net Moment	N/(W*1)	M/(1*W^2/6)	Net Press	
	Vert (k/ft)	M (to Toe)	M (to Ctr)	M o.t.		Press due	Press due	q1	
Soil & Conc	1.91	-3.21	-0.35			to vert load	to Moment		0.71
						(ksf)	(ksf)	q2	
Sum	1.91		-0.35	0.5	0.1	0.64	0.07		0.56

Notes: Postive moment : Anti-clockwise

e=M/V = 0.06 ft 156% in Compres

x=1.5 W - e 4.67 ft >75%,Ok

Load Case 2

This load case not applicable
for short walls in this area.

Load Case 3a

		Moment due to vertical load		M due to Horiz	Net Moment	N/(W*1)	M/(1*W^2/6)	Net Press	
	Vert (k/ft)	M (W/R Toe)	M (W/R Ctr)	Mot		Press due	Press due	q1	
Soil & Conc	1.91	-3.21	-0.35			to vert load	to Moment		0.71
						(ksf)	(ksf)	q2	
Sum	1.91		-0.35	0.5	0.1	0.64	0.08		0.56

Note: Postive moment : Anti-clockwise

e=M/V = 0.06 ft 156% in Compres

x=1.5 W - e 4.68 ft >75%,Ok

Load Case 4

		Moment due to vertical load		M due to Horiz	Net Moment	N/(W*1)	M/(1*W^2/6)	Net Press	
	Vert (k/ft)	M (W/R Toe)	M (W/R Ctr)	Mot		Press due	Press due	q1	
Soil & Conc	1.91	-3.21	-0.35			to vert load	to Moment		0.40
						(ksf)	(ksf)	q2	
Sum	1.91		-0.35	0.0	-0.3	0.64	-0.23		0.87

Notes: Postive moment : Anti-clockwise e=M/V = -0.18 ft ------> Resultant in base, Ok

Consider allowable bearing pressure increase for unusual condition, say 50% increase: 1.5*2.0=3.0 ksf. So, bearing pressure 2.8 <3.0, Ok

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	Subject	Veterans Park Walls
	By	G.Xu
	Date	Jan 2005

Strength Design per EM 1110-2-2502, Sect 9-8. Eq.(9-5) and (9-6)

Load Case 1:	U=1.9(D+L)
Load Cases 2 & 3:	U=0.75*1.9 (D+L)
Load Case 4:	U=0.75*1.9 (D+E)

where : D = Dead load

L = Live load, including Surcharge or H-15

E = Earthquake load

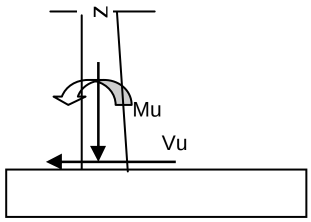
Moment at Bottom of wall Mu=

	D+L	D+E	U	
L.C. 1	0.7		1.3	<---Critical
L.C. 2	N/A		N/A	
L.C. 3	0.4		0.6	
L.C. 4		0.1	0.2	

Shear at bottom of wall Vu=

	D+L	E	U	
L.C. 1	0.1		0.2	<---Critical
L.C. 2	N/A		N/A	
L.C. 3	0.1		0.2	
L.C. 4		-0.3	-0.4	

5. Wall Stem Shear and Moment Check



Mu = max{load case1, load case2, load case 3, load case4} 1.3 kft/ft

Vu = max{load case1, load case2, load case 3, load case4} 0.2 k/ft

Check Moment

	Φ =	0.9	
	fy =	60000	psi
	f'c =	4000	psi
Use #4 @12"	As =	0.20	in^2
	b =	12.0	in (per foot of wall)
	d =	14.5	in
steel ratio	ρ = As/bd =	0.0011	in
$\Phi Mn = \Phi * fy * \rho\ [1 - fy * \rho / (1.7 * f'c)]\ b * d^2$		12.9	kft/ft
D/C = Mu/ΦMn		0.10	OK

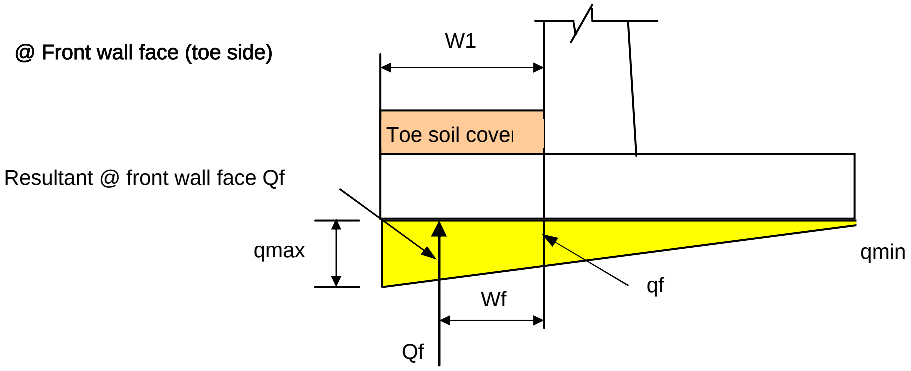
Check Shear

Shear capacity			
ΦVn = Φ2√f'c x B		23.22	k/ft
	where Φ =	0.85	
D/C = Vu/ΦVn		0.01	OK

Concrete shear strength alone is adaquate, no shear reinforcement is required.

6. Footing Shear and Moment Check (@ face of wall)
Load Case 1 governs the strength design

@ Front wall face (toe side)



Critical load case: Case 1 (most severe bearing pressure for footing in front of wall)

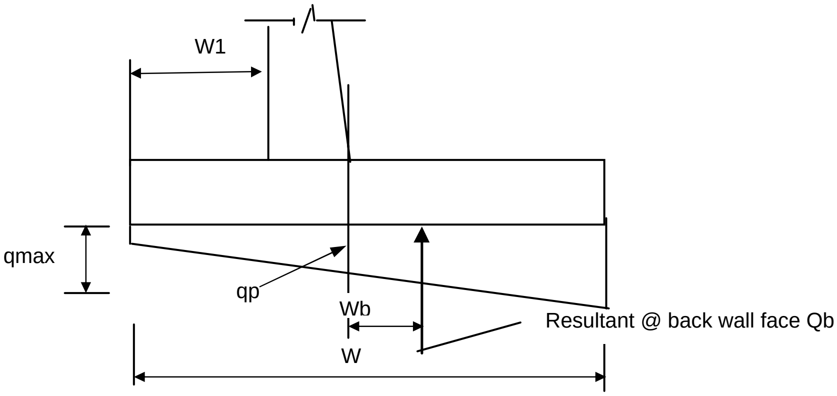
qf = (W2) / W x (qmax - qmin) + qmin 0.61 ksf
Qf =(qf+qmax)/2 x W1 0.66 k/ft
Vu = 1.9 (D+L) = 1.9 Qf 1.25 k/ft
Arm to face of wall
Wf = W1 (2qmax+qf) / [3 (qmax+qf)] 0.51 ft
Mu = Vu*Wf 0.6 kft/ft

Moment Check
Φ = 0.9
fy = 60000 psi
fc = 4000 psi
Use #4 @12" As = 0.20 in^2
b = 12.0 in (per foot of wall)
d = 8.6 in
steel ratio ρ = As/bd = 0.0019 in
$\Phi Mn = \Phi * fy * \rho\ [1 - fy*\rho/(1.7*f'c)]\ b*d^2$ 7.6 kft/ft
D/C = Mu/ΦMn 0.08 OK

Shear Check say ok
Shear capacity
$\Phi Vn = \Phi 2\sqrt{f'c} \times F$ 15.48 k/ft
where Φ = 0.85
D/C = Vu/ΦVn 0.08 OK

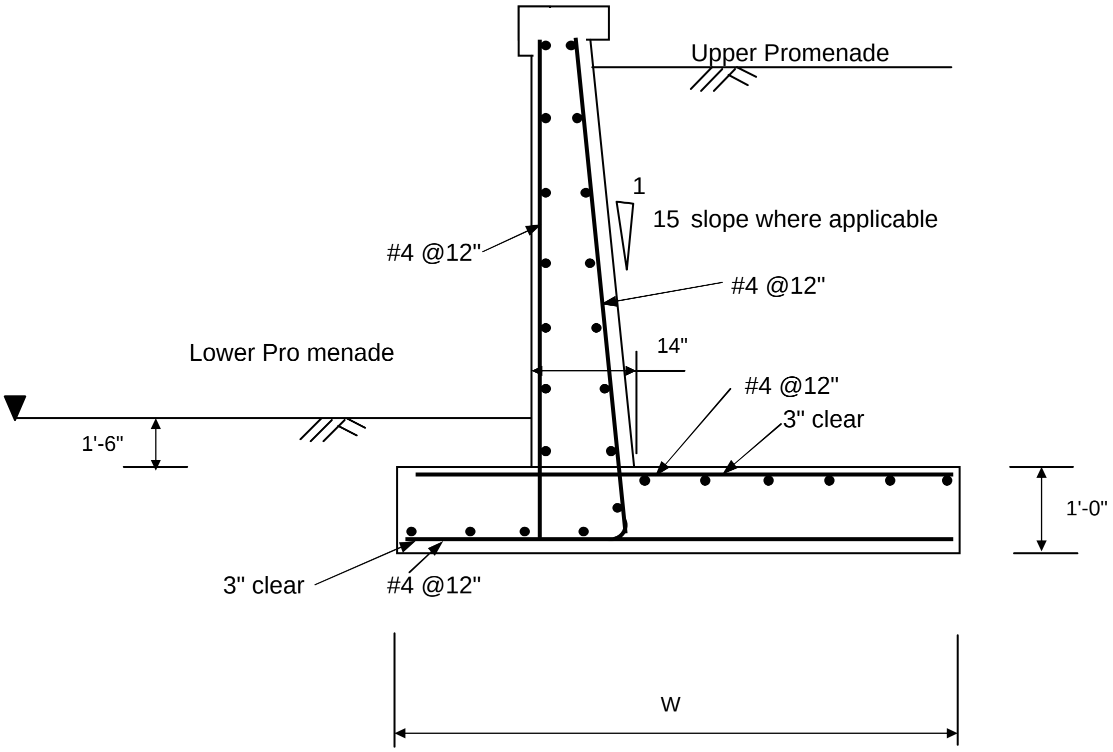
@ Back wall face

Critical Case: 3a



qb = (W-W1-B) / W x (qmax-qmiin) + qmin		0.80	ksf
Qb =(qf+qmin)/2 x (W-W1-B)		0.38	k/ft
Vu = 1.7 Qb		0.64	k/ft
Arm to face of wall			
	Wb =(W- W1-B) (2qmax+qb) / [3 (qmax+qb)]	0.24	ft
Mu= Vu*Wb		0.2	kft/ft
Moment check			
	Φ =	0.9	
	fy =	60000	ksi
	f'c=	4000	psi
	Use #4 @12" As =	0.20	in^2
	b =	12.0	in (per foot of wall)
	d =	8.6	in
	steel ratio ρ = As/bd =	0.0019	in
	ΦMn = Φ*fy*ρ [1 -fy*ρ/(1.7*f'c)] b*d^2	7.7	kft/ft
Shear Check	D/C = Mu/ΦMn	0.02	OK
	Shear capacity		
	ΦVn = Φ2√f'c x F	15.48	k/ft
	where Φ =	0.85	
	D/C = Vu/ΦVn	0.04	OK

Cross section for VP WallS No.2 & 3

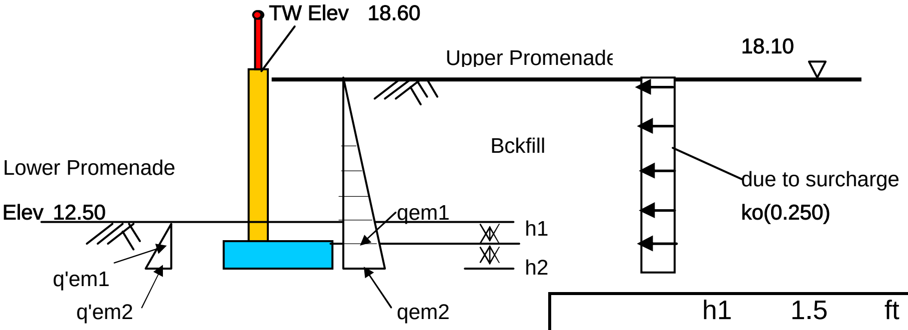


Veterans Park (VP) Wall No.1a & Planter Wall No.4

Load Case 1: Construction Condition (Unusual Condition)

Load cases are based on "Flood Walls and Channel Retaining Walls, Load Conditions & Load Diagrams" with the exception that D-4 not applicable in this location due to the width of access. Instead, use surcharge load for driveways as public access, see UBC Table 16A-A

Surcharge =	250	psf	
Elevation of Upper Promenade for design	18.80		approximate
Lower promenade	8.50	Finish Grade	approximate



h1	1.5	ft
h2	1.0	ft

Soil Pressure at Wall (Soil pressure =γ Ki hi)

Name	Layer	Thickness(ft)	Pressure(ksf)
qem1	Backfill	7.10	0.353
qem2	Backfill	1.00	0.403
q'em1	Backfill	1.50	0.754
q'em2	Backfill	1.00	1.257

Backfill parameter:

unit weight	125	pcf
friction angle, ϕ	37	deg
sin(ϕ)	0.602	
at rest coeff. Ko	0.398	
passive, kp	4.023	

Resultant Force at Bottom of Wall, per foot of wall

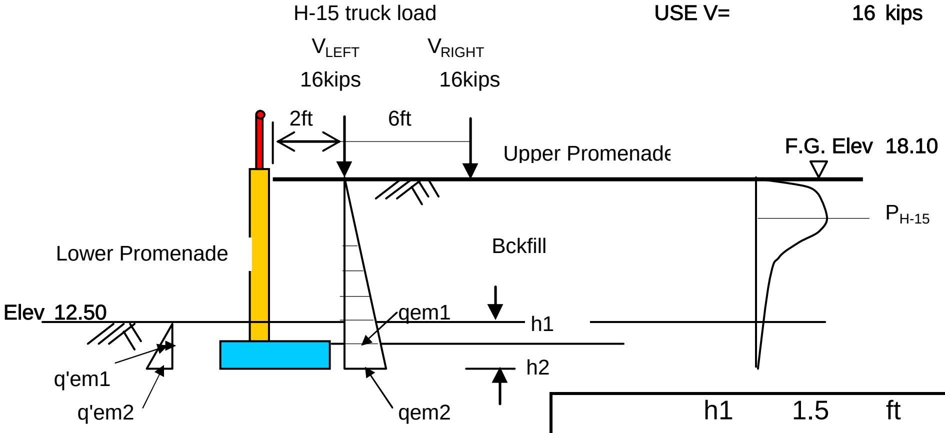
Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem1	1.3	2.37	3.0
Surcharge	0.7	3.55	2.5
P'em1	-0.57	0.50	-0.28
At bot of wall	**1.4**		**5.2**
	Σ H		Σ M
Force at Bottom of Wall: Hd = 1.4 kips Md = 5.2 k-ft			

Resultant Force at Bottom of Footing, per foot of wall

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem2	1.6	2.70	4.4
Surcharge	0.8	4.05	3.3
P'em2	-1.57	0.83	-1.3
At bot of ftg	**0.9**		**6.4**
	Σ V		Σ M
Force at Bottom of Footing: Hd = .9 kips Md = 6.4 k-ft			

Load Case 2: Normal Condition with Maintenance Vehicle (Usual Condition)

USE V= 16 kips



h1	1.5	ft
h2	1.0	ft

This load case not applicable
for walls in this area due to
limited access

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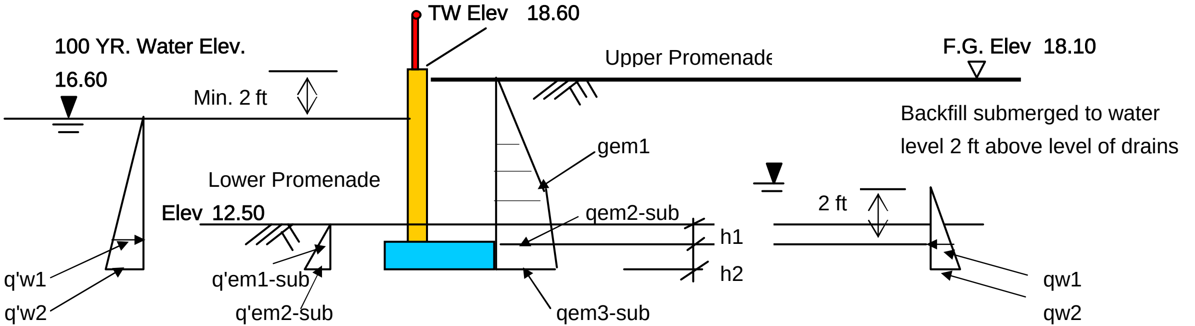
Project	Napa River Flood Ccontrol Project		
Subject	Veterans Park Walls		
By	G.Xu	Date	Jan 2005

Load Case 3a: Design Flood Condition (Unusual Condition)



h1	1.5	ft
h2	1.0	ft

Soil Pressure at Wall (Soil pressure =γ Ki hi)

Name	Layer	Thickness(ft)	Pressure(ksf)
qem1	Backfill	3.60	0.179
qem2-sub	Backfill	3.50	0.266
qem3-sub	Backfill	1.50	0.304
qw1	Backfill	3.50	0.219
qw2	Backfill	1.50	0.313
q'em1-sub	Backfill	1.50	0.377
q'em2-sub	Backfill	1.50	0.754
q'w1	Backfill	5.60	0.350
q'w2	Backfill	1.50	0.444

Backfill parameter:

Dry		
unit weight	125	pcf
friction angle, ϕ	37	Deg
sin(ϕ)	0.602	
at rest coeff. Ko	0.398	
passive, kp	4.023	
Submerged		
unit weight	62.5	pcf
Water		
unit weight	62.5	pcf

Resultant Force at Bottom Of Wall

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem1	0.3	4.70	1.52
Pem2-sub	0.8	1.64	1.28
Pw1	0.4	1.17	0.45
P'em1-sub	-0.28	0.50	-0.14
P'w1	-0.98	0.50	-0.5
At bot of wall	0.22		2.6
	Σ V		Σ M

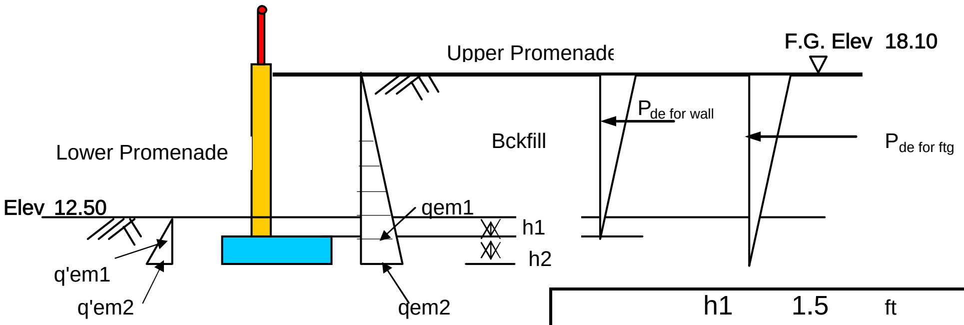
Force at Bottom of Wall: Vd = .22 kips Md = 2.61 k-ft

Resultant Force at Bottom Of Footing

Name	Force (kips)	Mom Arm(ft)	Moments(k-ft)
Pem1	0.3	6.20	2.00
Pem2-sub	0.8	3.14	2.44
Pem3-sub	0.4	0.73	0.31
Pw1	0.38	2.67	1.02
Pw2	0.4	0.71	0.28
P'em1-sub	-0.28	2.00	-0.57
P'em2-sub	-0.8	0.67	-0.57
P'w1	-0.98	1.87	-1.8
P'w2	-0.60	0.72	-0.4
At bot of ftg	-0.40		2.7
	Σ V		Σ M

Force at Bottom of Wall: Vd = -.4 kips Md = 2.67 k-ft

Load Case 4: Seismic Condition (Extreme Condition)



h1	1.5	ft
h2	1.0	ft

Soil/Water Properties

Layer	Soil Type	Thickness(ft)	Unit Wt. (pcf)	Φ	Ka	Ko	Kp
1	back fill	7.10	125	37	0.25	0.40	4.02
Water			62.5				

Soil Pressure (Soil pressure =γ Ki hi)

Name	Layer	Thickness(ft)	Pressure(ksf)
qem1	Backfill	7.10	0.221
qem2	Backfill	1.50	0.267
q'em1	Backfill	1.50	0.754
q'em2	Backfill	1.50	1.509
qde-wall	Backfill	7.10	0.086
qde-ftg	Backfill	8.60	0.104

$\alpha = \tan^{-1}[(C_1+\sqrt{(C_1^2+4C_2)})/2]$

(Equation 3-56 of EM 1110-2-2502, Page 3-67)

$C_1 = 2(\tan\Phi - K_h) / (1+K_h \tan\Phi)$

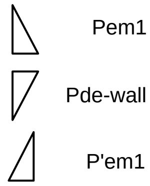
(Equation 3-57 of EM 1110-2-2502, Page 3-67)

$C_2 = [\tan\Phi(1-\tan\Phi\ \tan\beta)-(\tan\beta-K_h)] / [\tan\Phi(1+K_h \tan\Phi)]$

(Equation 3-58 of EM 1110-2-2502, Page 3-67)

Resultant Summary For bottom of wall

Name	Force	Arm to bot.	Moments
Pem1	0.8	2.37	1.9
Pde-wall	0.3	4.73	1.4
P'em1	-0.57	0.50	-0.28
At bot of wall	**0.5**		**3.0**
	Σ V		Σ M



$K_h =$ 0.15 g
$\beta =$ 0
$\Phi =$ 37 degree
$C_1 =$ 1.085
$C_2 =$ 0.720
$\alpha =$ 57.2 degree

Demand at bottom of wall: Vd = .52 kips Md = 3.01 k-ft

of which earthquake contribution (E)

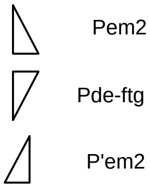
E =	0.3	kips	1.4	k-ft
ie. D+L =	0.2	kips	1.6	k-ft

Dynamic Components $q_{eq} = \gamma K_h h^2 / [2(\tan\alpha - \tan\beta)]$

(Equation 3-62 of EM 1110-2-2502, Page 3-68)

Resultant Summary For Bottom Of Ftg

Name	Force	Arm to bot.	Moments
Pem2	1.1	2.87	3.3
Pde-ftg	0.4	5.73	2.6
P'em2	-2.26	1.00	-2.26
At bot of ftg	**-0.7**		**3.6**
	Σ V		Σ M



Demand at bottom of ftg: Vd = -.67 kips Md = 3.60 k-ft

Project Napa River Flood Ccontrol Project

Subject Veterans Park Walls

By G.Xu

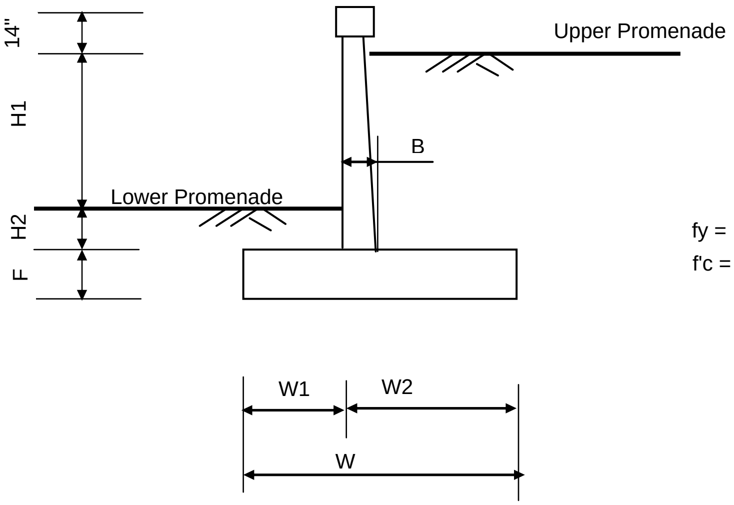
Date Jan 2005

Veterans Park (VP)
Wall No. 1a & Planter Wall No.4

1. Loads

A. Due to Lateral Pressure

Load Case	1		2		3		4	
	Bot/ Wall	Bot/Ftg	Bot/ Wall	Bot/Ftg	Bot/ Wall	Bot/Ftg	Bot/ Wall	Bot/Ftg
Hori Force(k)	1.4	0.9	N/A	N/A	0.2	-0.4	0.5	-0.7
Moment(kft)	5.2	6.4	N/A	N/A	2.6	2.7	3.0	3.6



fy = 60000 psi

f'c = 4000 psi

B = 1.50 ft

W1 = 2.50 ft

W2 = 3.50 ft

W = 6.00 ft

H1 = 5.60 ft

H2 = 1.50 ft

F = 1.00 ft

Conc. Unit Wt = 150 pcf

Backfill Unit Wt = 125 pcf

B. Resistance against overturning due to weight of concrete & soil

Due to concrete and soil

Location	Dimension		Weight(k/ft)	Arm to toe (ft)	Mot (kft/ft)
	Thick/Depth	Width/Height			
Toe cover	1.50	2.50	0.46875	1.25	0.59
Heel Soil	7.10	2.00	1.775	5.00	8.88
Footing	1.00	6.00	0.900	3.00	2.70
Concrete Key	0.00	0.00	0	0.00	0.00
Wall Stem	1.50	8.27	1.860	3.25	6.05
Total			5.00		18.21

Note: Live load is not included in the resistance to be conservative

2. Overturning Check

Load Case	Resistance Moment	Overturning Moment	Safety Factor	
1	18.21	6.4	2.86	Safety Factor > 1.5, OK
2	N/A	N/A	N/A	N/A
3a*	18.21	2.7	6.82	Safety Factor > 1.5, OK
4	18.21	3.6	5.06	Safety Factor > 1.5, OK

*Dry soil wt is used here instead of submerged soil wt, not critical either way, Ok.

3. Sliding Check

Resistance Due to Soil Friction

μ = use tan ($2\phi/3$) 0.46

Ffr = $P_d \times \mu$ 2.30 kips/ft

where, P_d -- weight of concrete & soil

Resistance due to passive pressure in front of wall is ignored.

Load Case	Resistance Due to Soil (kips)	Sliding Force (kips)	Safety Factor	
1	2.30	0.9	2.65	**> 1.33, OK**
2	N/A	N/A	N/A	N/A
3a*	2.30	-0.4	5.80	**> 1.5, OK**
4	2.30	-0.7	3.45	**> 1.1, OK**

Note:
Say, Ok, as H-15 will not be present in this area

* Dry weight is used here where part of it should be submerged wt, however it's not critical in this case.

4. Soil Bearing Pressure Check

(Allowable bearing pressure used: 2.0 ksf, see Geotech DDR)

Load Case 1

		Moment due to vert load		M due to Hori	Net Moment	N/(W*1)	M/(1*W^2/6)	Net Press
	Vert (k/ft)	M (to Toe)	M (to Ctr)	M o.t.		Press due	Press due	q1
Soil & Conc	5.00	-18.21	-3.19			to vert load	to Moment	**1.36**
						(ksf)	(ksf)	q2
Sum	**5.00**		-3.19	6.4	**3.2**	0.83	0.53	**0.31**

Notes:
1 Postive moment : Anti-clockwise

Percentage in compression 100%
Require percentage 75% OK

Load Case 2

This load case not applicable for walls in this area due to limited access

Load Case 3a

		Moment due to vertical load		M due to Horiz	Net Moment	N/(W*1)	M/(1*W^2/6)	Net Press
	Vert (k/ft)	M (W/R Toe)	M (W/R Ctr)	Mot		Press due	Press due	q1
Soil & Conc	5.00	-18.21	-3.19			to vert load	to Moment	**0.75**
						(ksf)	(ksf)	q2
Sum	**5.00**		-3.19	2.7	**-0.5**	0.83	-0.09	**0.92**

Note: Postive moment : Anti-clockwise

Percentage in compression 100%
Required percentage 100% Ok

Load Case 4

		Moment due to vertical load		M due to Horiz	Net Moment	N/(W*1)	M/(1*W^2/6)	Net Press
	Vert (k/ft)	M (W/R Toe)	M (W/R Ctr)	Mot		Press due	Press due	q1
Soil & Conc	5.00	-18.21	-3.19			to vert load	to Moment	**0.90**
						(ksf)	(ksf)	q2
Sum	**5.00**		-3.19	3.6	**0.4**	0.83	0.07	**0.77**

Notes Postive moment : Anti-clockwise

Percentage in compression 100%
Req'd: Resultant in base. Ok

Strength Design per EM 1110-2-2502, Sect 9-8. Eq.(9-5) and (9-6)

Load Case 1: U=1.9(D+L)
Load Cases 2 & 3: U=0.75*1.9 (D+L)
Load Case 4: U=0.75*1.9 (D+E)
where : D = Dead load
L = Live load, including Surcharge or H-15
E = Earthquake load

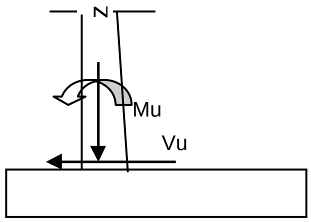
Moment at Bottom of wall Mu=

	D+L	D+E	U	
L.C. 1	5.2		9.9	<---Critical
L.C. 2	N/A		N/A	
L.C. 3	2.6		3.7	
L.C. 4		3.0	4.3	

Shear at bottom of wall Vu=

	D+L	E	U	
L.C. 1	1.4		2.7	<---Critical
L.C. 2	N/A		N/A	
L.C. 3	0.2		0.3	
L.C. 4		0.5	0.7	

5. Wall Stem Shear and Moment Check



Mu = max{load case1, load case2, load case 3, load case4} 9.9 kft/ft
Vu = max{load case1, load case2, load case 3, load case4} 2.7 k/ft

Check Moment

Φ = 0.9
fy = 60000 psi
f'c = 4000 psi
Use #4 @12" As = 0.20 in^2
b = 12.0 in (per foot of wall)
d = 14.5 in
steel ratio ρ = As/bd = 0.0011 in
$\Phi Mn = \Phi * fy * \rho \, [1 - fy*\rho/(1.7*f'c)] \, b*d^2$ 12.9 kft/ft
D/C = Mu/ΦMn 0.76 OK

Check Shear

Shear capacity
$\Phi Vn = \Phi 2\sqrt{f'c} \times B$ 23.22 k/ft
where Φ = 0.85
D/C = Vu/ΦVn 0.11 OK

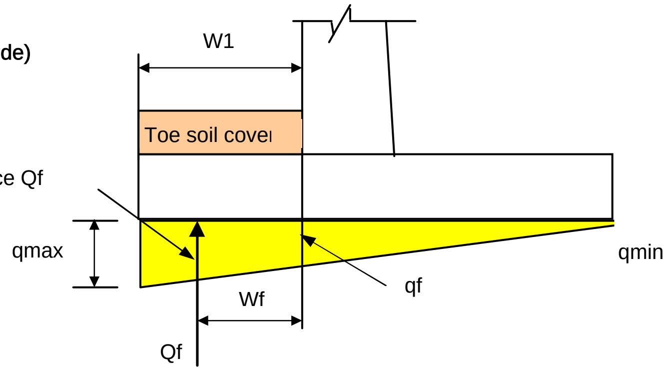
Concrete shear strength alone is adaquate, no shear reinforcement is required.

6. Footing Shear and Moment Check (@ face of wall)

Load Case 1 governs the strength design

@ Front wall face (toe side)

Resultant @ front wall face Qf



Critical load case: Case 1 (most severe bearing pressure for footing in front of wall)

qf = (W2) / W x (qmax - qmin) + qmin	0.75	ksf
Qf =(qf+qmax)/2 x W1	2.64	k/ft
Vu = 1.9 (D+L) = 1.9 Qf	5.01	k/ft
Arm to face of wall		
Wf = W1 (2qmax+qf) / [3 (qmax+qf)]	1.37	ft
Mu = Vu*Wf	6.9	kft/ft

Moment Check

	Φ =	0.9	
	fy =	60000	psi
	fc =	4000	psi
Use #4 @12"	As =	0.20	in^2
	b =	12.0	in (per foot of wall)
	d =	8.6	in
steel ratio	ρ = As/bd =	0.0019	in
$\Phi Mn = \Phi * fy * \rho\ [1 - fy * \rho/(1.7 * f'c)]\ b * d^2$		7.6	kft/ft
D/C = Mu/ΦMn		0.90	OK

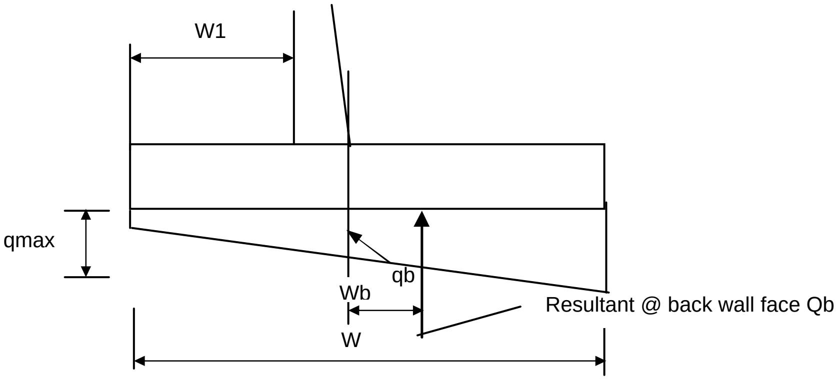
Shear Check — say ok

Shear capacity			
$\Phi Vn = \Phi 2\sqrt{f'c} \times F$		15.48	k/ft
where	Φ =	0.85	
D/C = Vu/ΦVn		0.32	OK

MGE ENGINEERING, INC.		
	Project: Napa River Flood Ccontrol Project	
	Subject: Veterans Park Walls	
	By: G.Xu	Date: Jan 2005

@ Back wall face

Critical Case: 3a



qb = (W-W1-B) / W x (qmax-qmiin) + qmin 1.05 ksf

Qb =(qf+qmin)/2 x (W-W1-B) 1.80 k/ft

Vu = 1.7 Qb 3.06 k/ft

Arm to face of wall

Wb =(W- W1-B) (2qmax+qb) / [3 (qmax+qb)] 0.98 ft

Mu= Vu*Wb 3.0 kft/ft

Moment check

Φ = 0.9

fy = 60000 ksi

f'c= 4000 psi

Use #4 @12" As = 0.20 in^2

b = 12.0 in (per foot of wall)

d = 8.6 in

steel ratio ρ = As/bd = 0.0019 in

ΦMn = Φ*fy*ρ [1 -fy*ρ/(1.7*f'c)] $b*d^2$ 7.7 kft/ft

Shear Check D/C = Mu/ΦMn 0.39 OK

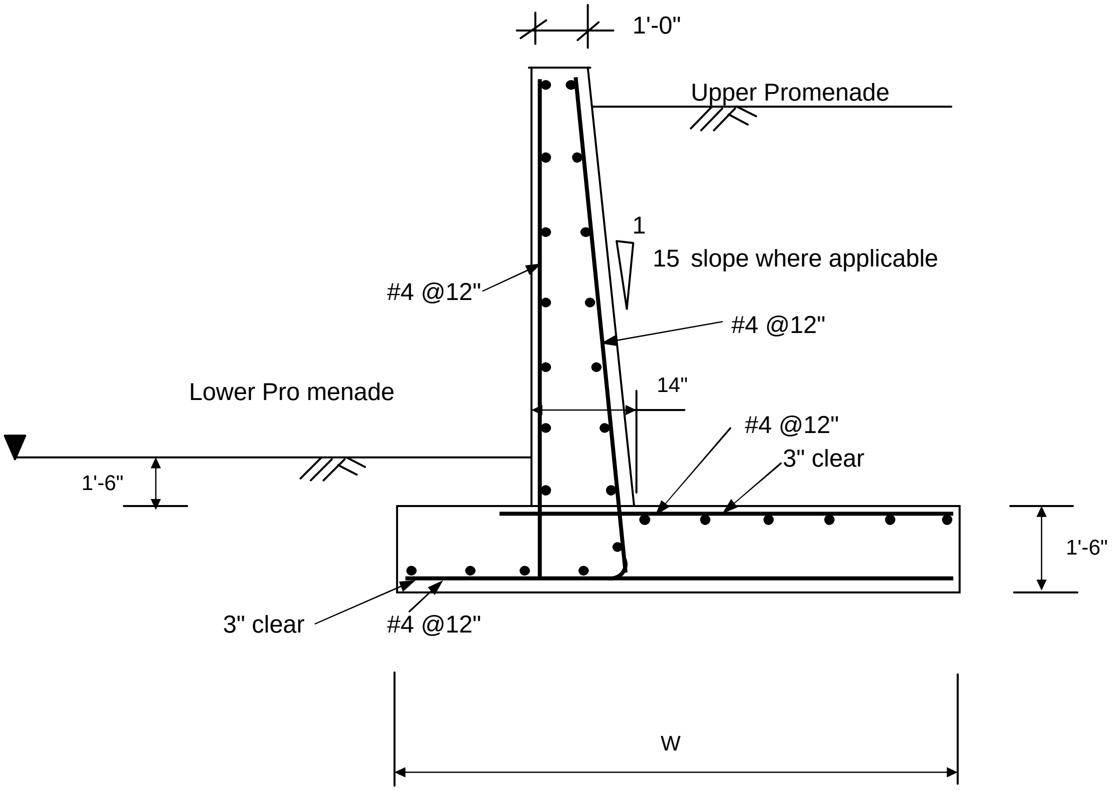
Shear capacity

ΦVn = Φ2√f'c x F 15.48 k/ft

where Φ = 0.85

D/C = Vu/ΦVn 0.20 OK

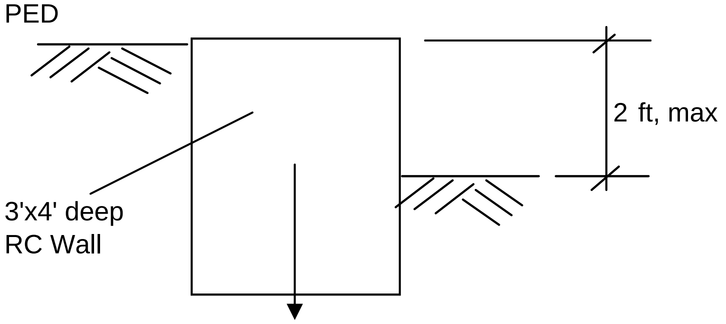
Cross section for VP Wall No.1a
& Planter Wall No.4



MGE ENGINEERING, INC.

Project	Napa River Flood Ccontrol Project		
Subject	Veterans Park -Terrace Wall		
By	G.Xu	Date	Jan 2005

Terrace Wall



Due to the location and configuration of terrace wall, only consider normal loading case for design purpose, that is, with pedestrian on the high side of terrace, say 50 psf, no truck or buldozer.

Height of wall h = 4 ft, assuming no passive backfill
Backfill material ϕ = 37 deg
ko = 0.398
γ_b= 125 pcf
Conc. Wt γ_c= 150 pcf
Lateral pressure at bottom of wall (including 50 psf, equiv to 0.4ft of backfill)
p =ko*γ_b*(h+0.4) 0.22 ksf
Lateral force:
P=p*1*(h+0.4)/2 0.48 kips per ft of wall

Overturning check:
Overturning moment at bottom of wall:
M_{OT}=P*(h+0.4)/3 0.71 k-ft

Vertical load:
For overturning check, assume only wt of conc block (3'x4')
Nconc = 1.8 kips
For bearing pressure, also include 50 psf pedestrian (PED) on top of wall
N ped = 0.15 kips

Resisting moment (due to wt of concrete block)
M res = Nconc*b/2 2.7 k-ft (b=3 ft)

F.S. against O.T. = **3.8** ok

MGE ENGINEERING, INC.

Project	Napa River Flood Ccontrol Project		
Subject	Veterans Park -Terrace Wall		
By	G.Xu	Date	Jan-00

Terrace Wall - Cont'd

Sliding check:

assume μ use tan (2 ϕ / 3) 0.46 friction coefficient
Resistance= μ*Nconc 0.828 kips
Driving force = P 0.48 kips

F.S. against sliding = **1.7** > 1.5, Ok

Bearing pressure check:

Tot vertical: 1.95 kips (including wt of conc and pedestrian0
Tot moment with respect to center of contact surface:
M = O.T. 0.71 k-ft
N/A = 0.65 ksf
M/(bh^2/6) = 0.47 ksf

qmax = N/A + M/(bh^2/6) **1.12** < 2.0 ksf Ok

qmin = N/A - M/(bh^2/6) **0.18** ksf > 0, all compression, Ok

XSECTION ANALYSES

```
02/11/2005, 17:49
************************************************************
*                                                          *
*                       xSECTION                           *
*                                                          *
*               DUCTILITY and STRENGTH of                  *
*     Circular, Semi-Circular, full and partial Rings,     *
*  Rectangular, T-, I-, Hammer head, Octagonal, Polygons   *
*      or any combination of above shapes forming          *
*         Concrete Sections using Fiber Models             *
*                                                          *
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xSECTION
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LICENSE    (choices: LIMITED/UNLIMITED)
UNLIMITED
ENTITY     (choices: GOVERNMENT/CONSULTANT)
CONSULTANT
NAME_OF_FIRM
FRED_HUANG
BRIDGE_NAME
NAPA_RIVER_FLOOD_CONTROL_PROJECT
BRIDGE_NUMBER
123-456
JOB_TITLE
CONNECTION of WALL AND 24 INCHES CIDH PILE(Type A, Compression piles--3 Rows)



Concrete Type Information:

     ----------strains--------  --------strength--------
Type   e0    e2    ecc     eu     f0    f2    fcc    fu    E    W
 1 0.0050 0.0100 0.0203 0.0245  3.00  4.71  4.84  4.83  3123   144
 2 0.0050 0.0100 0.0050 0.0200  3.00  2.87  3.00  2.00  3321   150


Steel Type Information:

     -----strains------  --strength-
Type  ey     eh     eu    fy    fu    E
 1 0.0023 0.0115 0.0900 68.00 95.00 29000
 2 0.0023 0.0115 0.0600 68.00 95.00 29000


Steel Fiber Information:
```

```
                                    NAPA24A.OUT

Fiber          xc       yc      area
No.   type     in       in      in^2
  1    2     8.50    0.00    0.44
  2    2     7.36    4.25    0.44
  3    2     4.25    7.36    0.44
  4    2     0.00    8.50    0.44
  5    2    -4.25    7.36    0.44
  6    2    -7.36    4.25    0.44
  7    2    -8.50    0.00    0.44
  8    2    -7.36   -4.25    0.44
  9    2    -4.25   -7.36    0.44
 10    2     0.00   -8.50    0.44
 11    2     4.25   -7.36    0.44
 12    2     7.36   -4.25    0.44


Force Equilibrium Condition of the x-section:

     Max.             Max.
     Conc.    Neutral Steel          Steel
     Strain    Axis   Strain Conc.    force       P/S     Net  Curvature Moment
step epscmax   in.    Tens.  Comp.  Comp. Tens.   force   force  rad/in  (K-ft)
 0   0.00000    0.00  0.0000     0     0      0     0       0.00 0.000000      0
 1   0.00049    3.10 -0.0006    89    11    -38     0       0.00 0.000055     86
 2   0.00054    3.29 -0.0007    94    12    -43     0       0.05 0.000062     93
 3   0.00060    3.45 -0.0008   100    13    -50     0      -0.03 0.000070    101
 4   0.00066    3.58 -0.0010   106    14    -57     0       0.03 0.000079    110
 5   0.00073    3.70 -0.0011   113    15    -65     0       0.04 0.000088    119
 6   0.00081    3.79 -0.0012   120    16    -73     0       0.01 0.000099    130
 7   0.00089    3.87 -0.0014   128    17    -83     0       0.03 0.000110    140
 8   0.00099    3.93 -0.0015   137    19    -93     0      -0.05 0.000123    152
 9   0.00109    3.97 -0.0017   146    21   -103     0       0.04 0.000136    165
10   0.00121    4.01 -0.0019   156    23   -115     0      -0.01 0.000151    179
11   0.00134    4.02 -0.0021   166    25   -128     0       0.01 0.000167    193
12   0.00148    4.02 -0.0023   177    27   -142     0       0.01 0.000185    209
13   0.00163    4.06 -0.0026   188    30   -155     0       0.01 0.000206    223
14   0.00181    4.17 -0.0029   195    32   -164     0       0.03 0.000231    232
15   0.00200    4.27 -0.0033   203    34   -174     0       0.05 0.000258    242
16   0.00221    4.40 -0.0037   209    37   -183     0       0.04 0.000290    249
17   0.00244    4.57 -0.0043   214    40   -190     0       0.00 0.000328    254
18   0.00270    4.71 -0.0049   218    43   -198     0       0.02 0.000370    260
19   0.00298    4.82 -0.0055   223    46   -207     0       0.01 0.000415    265
20   0.00330    4.92 -0.0062   229    50   -216     0      -0.01 0.000466    271
21   0.00364    5.09 -0.0072   230    54   -221     0      -0.02 0.000527    274
22   0.00403    5.25 -0.0082   231    57   -225     0       0.06 0.000597    276
23   0.00445    5.40 -0.0094   232    60   -229     0       0.04 0.000675    278
24   0.00492    5.53 -0.0107   233    64   -234     0       0.05 0.000760    280
25   0.00544    5.62 -0.0120   235    68   -240     0       0.03 0.000852    283
26   0.00602    5.66 -0.0134   239    71   -247     0      -0.05 0.000949    287
27   0.00665    5.70 -0.0150   243    75   -255     0      -0.03 0.001055    293
28   0.00735    5.72 -0.0167   247    79   -263     0      -0.05 0.001171    298
29   0.00813    5.73 -0.0184   252    84   -273     0      -0.03 0.001296    305
30   0.00899    5.74 -0.0204   257    90   -284     0      -0.06 0.001435    312
31   0.00993    5.69 -0.0223   265    90   -291     0       0.04 0.001574    318
32   0.01098    5.66 -0.0245   271    90   -298     0      -0.05 0.001733    323
33   0.01214    5.64 -0.0270   275    90   -302     0      -0.04 0.001910    327
34   0.01342    5.59 -0.0295   280    90   -306     0       0.01 0.002095    329
35   0.01484    5.52 -0.0321   284    90   -311     0       0.02 0.002290    332
                                       Page 2
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36   0.01640    5.41 -0.0347   289    90  -315     0     -0.04 0.002490     332
37   0.01813    5.25 -0.0369   292    90  -319     0     -0.02 0.002687     331
38   0.02005    5.05 -0.0391   294    90  -321     0     -0.03 0.002885     327
39   0.02216    4.92 -0.0420   292    90  -318     0      0.04 0.003131     322
40   0.02450    4.80 -0.0453   288    91  -316     0      0.01 0.003404     318


First Yield of Rebar Information (not Idealized):

     Rebar Number 10
     Coordinates X and Y (global in.)   0.00,  -8.50
     Yield strain =  0.00230
     Curvature (rad/in)=  0.000184
     Moment (ft-k) =     207


Cross Section Information:

     Axial Load on Section (kips) =    63
     Percentage of Main steel in Cross Section =   1.17
     Concrete modulus used in Idealization (ksi) =  3123
     Cracked Moment of Inertia (ft^4) =     0.209


Idealization of Moment-Curvature Curve by Various Methods:


                  Points on Curve       Idealized Values
                  ===============   =============================
Method     Conc.                    Yield         symbol  Plastic
 ID      | Strain  Curv.   Moment | Curv.  Moment   for   Curv.
         | in/in   rad/in  (K-ft) | rad/in (K-ft)  moment rad/in
Strain @ 0.003   0.000418     266 0.000235     266   Mn  0.003169
Strain @ 0.004   0.000592     276 0.000244     276   Mn  0.003160
Strain @ 0.005   0.000774     281 0.000249     281   Mn  0.003155
CALTRANS 0.00875 0.001397     310 0.000275     310   Mp  0.003129
UCSD@5phy0.00583 0.000918     286 0.000253     286   Mn  0.003151
```

```
                                    NAPA36BE.OUT
02/11/2005, 17:52
************************************************************
*                                                          *
*                       xSECTION                           *
*                                                          *
*               DUCTILITY and STRENGTH of                  *
*     Circular, Semi-Circular, full and partial Rings,     *
*  Rectangular, T-, I-, Hammer head, Octagonal, Polygons   *
*      or any combination of above shapes forming          *
*         Concrete Sections using Fiber Models             *
*                                                          *
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CONSULTANT
NAME_OF_FIRM
FRED_HUANG
BRIDGE_NAME
NAPA_RIVER_FLOOD_CONTROL_PROJECT
BRIDGE_NUMBER
123-456
JOB_TITLE
CONNECTION of WALL AND 36 INCHES CIDH PILE(Type B & E)



Concrete Type Information:

     ----------strains--------  --------strength--------
Type   e0    e2    ecc     eu     f0    f2    fcc    fu    E    W
 1 0.0050 0.0100 0.0182 0.0220  3.00  4.50  4.59  4.58  3123   144
 2 0.0050 0.0100 0.0050 0.0200  3.00  2.87  3.00  2.00  3321   150


Steel Type Information:

     -----strains------  --strength-
Type  ey     eh     eu    fy    fu    E
 1 0.0023 0.0115 0.0900 68.00 95.00 29000
 2 0.0023 0.0115 0.0600 68.00 95.00 29000


Steel Fiber Information:
                                       Page 1
```

```
Fiber          xc       yc      area
No.   type     in       in      in^2
  1    2    13.85    0.00    1.56
  2    2    13.01    4.74    1.56
  3    2    10.61    8.90    1.56
  4    2     6.92   11.99    1.56
  5    2     2.40   13.64    1.56
  6    2    -2.41   13.64    1.56
  7    2    -6.93   11.99    1.56
  8    2   -10.61    8.90    1.56
  9    2   -13.01    4.74    1.56
 10    2   -13.85    0.00    1.56
 11    2   -13.01   -4.74    1.56
 12    2   -10.61   -8.90    1.56
 13    2    -6.92  -11.99    1.56
 14    2    -2.40  -13.64    1.56
 15    2     2.41  -13.64    1.56
 16    2     6.93  -11.99    1.56
 17    2    10.61   -8.90    1.56
 18    2    13.01   -4.74    1.56


Force Equilibrium Condition of the x-section:

     Max.             Max.
     Conc.    Neutral Steel           Steel
     Strain     Axis  Strain Conc.    force       P/S     Net  Curvature Moment
step epscmax   in.    Tens.  Comp.  Comp. Tens.   force   force  rad/in  (K-ft)
 0   0.00000    0.00  0.0000     0     0     0     0      0.00 0.000000       0
 1   0.00044    4.84 -0.0006   178    61  -192     0      0.03 0.000033     401
 2   0.00049    4.87 -0.0007   193    67  -214     0      0.01 0.000037     441
 3   0.00054    4.90 -0.0008   210    74  -238     0     -0.01 0.000041     484
 4   0.00059    4.92 -0.0008   228    81  -263     0     -0.02 0.000045     531
 5   0.00066    4.92 -0.0009   248    90  -291     0      0.00 0.000050     582
 6   0.00073    4.92 -0.0010   269    99  -322     0      0.02 0.000056     637
 7   0.00080    4.91 -0.0011   291   110  -355     0      0.01 0.000061     698
 8   0.00089    4.89 -0.0013   316   122  -391     0      0.00 0.000068     763
 9   0.00098    4.86 -0.0014   342   135  -431     0      0.02 0.000075     834
10   0.00109    4.82 -0.0015   369   150  -473     0      0.02 0.000082     910
11   0.00120    4.77 -0.0017   399   166  -519     0      0.01 0.000091     992
12   0.00133    4.72 -0.0018   430   184  -568     0      0.01 0.000100    1081
13   0.00147    4.66 -0.0020   463   205  -622     0     -0.01 0.000110    1176
14   0.00162    4.59 -0.0022   497   229  -680     0     -0.02 0.000121    1279
15   0.00179    4.54 -0.0024   532   254  -740     0      0.01 0.000133    1382
16   0.00198    4.62 -0.0027   560   279  -793     0      0.02 0.000148    1466
17   0.00219    4.75 -0.0030   585   303  -842     0      0.00 0.000165    1540
18   0.00242    4.97 -0.0035   603   330  -887     0      0.01 0.000186    1601
19   0.00268    5.20 -0.0039   619   358  -931     0      0.01 0.000209    1656
20   0.00296    5.42 -0.0045   635   389  -978     0      0.01 0.000235    1713
21   0.00327    5.75 -0.0052   641   418 -1012     0      0.02 0.000267    1750
22   0.00362    6.04 -0.0060   647   449 -1050     0     -0.03 0.000302    1789
23   0.00400    6.22 -0.0067   659   472 -1085     0     -0.05 0.000339    1822
24   0.00442    6.39 -0.0076   672   492 -1118     0     -0.02 0.000381    1850
25   0.00489    6.66 -0.0087   674   508 -1136     0      0.02 0.000431    1865
26   0.00540    6.83 -0.0099   683   515 -1152     0      0.01 0.000483    1875
27   0.00597    6.94 -0.0111   694   520 -1169     0     -0.01 0.000540    1884
28   0.00660    7.01 -0.0124   709   527 -1190     0      0.03 0.000601    1902
29   0.00730    7.03 -0.0138   727   537 -1218     0     -0.01 0.000665    1934
```

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30   0.00807    7.03 -0.0152   746   549 -1248     0     -0.03 0.000736    1969
31   0.00892    7.01 -0.0168   766   563 -1283     0      0.03 0.000812    2010
32   0.00986    6.99 -0.0185   786   579 -1319     0     -0.01 0.000896    2051
33   0.01090    6.97 -0.0204   805   597 -1356     0     -0.01 0.000988    2091
34   0.01205    6.92 -0.0224   821   619 -1395     0     -0.04 0.001088    2133
35   0.01332    6.90 -0.0247   830   636 -1421     0      0.01 0.001200    2168
36   0.01473    6.74 -0.0266   850   636 -1440     0     -0.04 0.001308    2188
37   0.01628    6.52 -0.0286   867   636 -1457     0      0.04 0.001418    2200
38   0.01800    6.27 -0.0306   882   636 -1473     0     -0.01 0.001535    2206
39   0.01990    6.11 -0.0330   889   643 -1486     0      0.01 0.001673    2213
40   0.02200    5.95 -0.0358   891   650 -1496     0     -0.03 0.001826    2217


First Yield of Rebar Information (not Idealized):

     Rebar Number 14
     Coordinates X and Y (global in.)  -2.40, -13.64
     Yield strain =  0.00230
     Curvature (rad/in)=  0.000126
     Moment (ft-k) =    1330


Cross Section Information:

     Axial Load on Section (kips) =    46
     Percentage of Main steel in Cross Section =   2.76
     Concrete modulus used in Idealization (ksi) =  3123
     Cracked Moment of Inertia (ft^4) =     1.949


Idealization of Moment-Curvature Curve by Various Methods:


                  Points on Curve        Idealized Values
                  ===============   =============================
Method     Conc.                    Yield         symbol  Plastic
 ID      | Strain  Curv.   Moment | Curv.  Moment   for   Curv.
         | in/in   rad/in  (K-ft) | rad/in (K-ft)  moment rad/in
Strain @ 0.003   0.000239    1718 0.000163    1718   Mn  0.001662
Strain @ 0.004   0.000340    1822 0.000173    1822   Mn  0.001652
Strain @ 0.005   0.000442    1867 0.000177    1867   Mn  0.001648
CALTRANS 0.00974 0.000885    2045 0.000194    2045   Mp  0.001631
UCSD@5phy0.00694 0.000632    1918 0.000182    1918   Mn  0.001643
```

```
                                    NAPA24CB.OUT
02/13/2005, 12:40
************************************************************
*                                                          *
*                      xSECTION                            *
*                                                          *
*              DUCTILITY and STRENGTH of                   *
*     Circular, Semi-Circular, full and partial Rings,     *
*  Rectangular, T-, I-, Hammer head, Octagonal, Polygons   *
*      or any combination of above shapes forming          *
*         Concrete Sections using Fiber Models             *
*                                                          *
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NAME_OF_FIRM
FRED_HUANG
BRIDGE_NAME
NAPA_RIVER_FLOOD_CONTROL_PROJECT
BRIDGE_NUMBER
123-456
JOB_TITLE
CONNECTION of WALL AND 24 INCHES CIDH PILE(TypeC Single Pile)



Concrete Type Information:

     ----------strains--------  --------strength--------
Type   e0    e2    ecc     eu     f0    f2    fcc    fu    E    W
 1 0.0050 0.0100 0.0207 0.0245  3.00  4.75  4.88  4.88  3123   144
 2 0.0050 0.0100 0.0050 0.0200  3.00  2.87  3.00  2.00  3321   150


Steel Type Information:

     -----strains------  --strength-
Type  ey     eh     eu    fy    fu    E
 1 0.0023 0.0115 0.0900 68.00 95.00 29000
 2 0.0023 0.0115 0.0600 68.00 95.00 29000


Steel Fiber Information:
```

```
Fiber           xc        yc      area
No.   type      in        in      in^2
  1    2      7.95    2.58    1.27
  2    2      4.91    6.76    1.27
  3    2      0.00    8.36    1.27
  4    2     -4.91    6.76    1.27
  5    2     -7.95    2.58    1.27
  6    2     -7.95   -2.58    1.27
  7    2     -4.91   -6.76    1.27
  8    2      0.00   -8.36    1.27
  9    2      4.91   -6.76    1.27
 10    2      7.95   -2.58    1.27
 11    2      2.58    7.95    1.27
 12    2     -2.58    7.95    1.27
 13    2      2.58   -7.95    1.27
 14    2     -2.58   -7.95    1.27


Force Equilibrium Condition of the x-section:

     Max.             Max.
     Conc.    Neutral Steel           Steel
     Strain    Axis   Strain  Conc.    force        P/S     Net  Curvature Moment
step epscmax   in.    Tens.   Comp.  Comp. Tens.   force   force  rad/in  (K-ft)
 0   0.00000    0.00  0.0000     0      0     0     0       0.00 0.000000      0
 1   0.00049    2.83 -0.0006    93     47  -124     0       0.00 0.000053    161
 2   0.00054    2.83 -0.0007   101     51  -138     0       0.00 0.000059    177
 3   0.00060    2.83 -0.0007   110     57  -152     0       0.01 0.000065    195
 4   0.00066    2.82 -0.0008   120     63  -168     0       0.01 0.000072    213
 5   0.00073    2.81 -0.0009   130     70  -185     0       0.00 0.000080    234
 6   0.00081    2.79 -0.0010   141     77  -203     0       0.01 0.000088    256
 7   0.00089    2.76 -0.0011   153     85  -224     0       0.00 0.000097    280
 8   0.00099    2.74 -0.0012   166     95  -245     0       0.01 0.000107    306
 9   0.00109    2.71 -0.0013   179    105  -269     0      -0.01 0.000118    335
10   0.00121    2.67 -0.0014   193    117  -295     0       0.00 0.000130    365
11   0.00134    2.63 -0.0016   208    129  -323     0       0.01 0.000143    398
12   0.00148    2.58 -0.0017   224    144  -353     0       0.01 0.000157    434
13   0.00163    2.53 -0.0019   240    160  -386     0      -0.01 0.000172    472
14   0.00181    2.48 -0.0021   257    179  -421     0       0.01 0.000190    514
15   0.00200    2.42 -0.0022   275    200  -460     0       0.00 0.000208    558
16   0.00221    2.40 -0.0025   292    221  -498     0       0.00 0.000230    602
17   0.00244    2.51 -0.0028   303    240  -528     0      -0.01 0.000257    634
18   0.00270    2.75 -0.0032   307    258  -550     0       0.01 0.000292    654
19   0.00298    2.96 -0.0037   312    279  -576     0       0.00 0.000330    675
20   0.00330    3.15 -0.0043   317    302  -604     0       0.00 0.000372    698
21   0.00364    3.37 -0.0050   319    325  -629     0       0.01 0.000422    717
22   0.00403    3.69 -0.0058   314    345  -644     0       0.00 0.000485    726
23   0.00445    3.95 -0.0068   311    364  -660     0       0.00 0.000553    735
24   0.00492    4.14 -0.0078   311    380  -676     0       0.01 0.000626    743
25   0.00544    4.23 -0.0088   315    390  -690     0      -0.01 0.000701    748
26   0.00602    4.32 -0.0099   319    400  -705     0       0.01 0.000783    754
27   0.00665    4.39 -0.0111   324    412  -721     0      -0.01 0.000874    760
28   0.00735    4.43 -0.0124   330    426  -741     0      -0.01 0.000971    771
29   0.00813    4.37 -0.0136   342    432  -759     0      -0.01 0.001065    784
30   0.00899    4.26 -0.0147   356    432  -773     0       0.00 0.001161    797
31   0.00993    4.16 -0.0159   371    432  -788     0      -0.01 0.001268    810
32   0.01098    4.07 -0.0172   386    432  -803     0      -0.01 0.001384    823
33   0.01214    3.96 -0.0186   399    432  -816     0       0.01 0.001510    834
```

```
34   0.01342    3.84 -0.0201   410   432  -826     0     -0.01 0.001645     842
35   0.01484    3.72 -0.0217   418   432  -835     0      0.00 0.001793     849
36   0.01640    3.59 -0.0233   424   432  -840     0      0.01 0.001949     854
37   0.01813    3.45 -0.0250   427   432  -844     0      0.01 0.002120     856
38   0.02005    3.30 -0.0269   427   432  -844     0      0.00 0.002305     857
39   0.02216    3.18 -0.0290   421   438  -844     0      0.00 0.002511     857
40   0.02450    3.08 -0.0314   416   447  -848     0      0.00 0.002746     862


First Yield of Rebar Information (not Idealized):

     Rebar Number 8
     Coordinates X and Y (global in.)   0.00,  -8.36
     Yield strain =  0.00230
     Curvature (rad/in)=  0.000214
     Moment (ft-k) =     570


Cross Section Information:

     Axial Load on Section (kips) =    15
     Percentage of Main steel in Cross Section =   3.93
     Concrete modulus used in Idealization (ksi) =  3123
     Cracked Moment of Inertia (ft^4) =     0.495


Idealization of Moment-Curvature Curve by Various Methods:

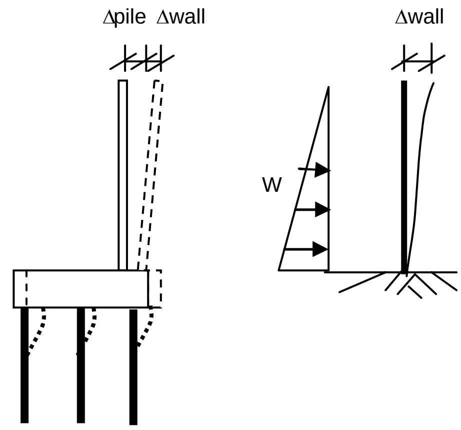

                  Points on Curve        Idealized Values
                  ===============   =============================
Method     Conc.                    Yield         symbol  Plastic
 ID      | Strain  Curv.   Moment | Curv.  Moment   for   Curv.
         | in/in   rad/in  (K-ft) | rad/in (K-ft)  moment rad/in
Strain @ 0.003   0.000332     676 0.000253     676   Mn  0.002493
Strain @ 0.004   0.000480     725 0.000272     725   Mn  0.002474
Strain @ 0.005   0.000637     744 0.000278     744   Mn  0.002467
CALTRANS 0.00989 0.001262     810 0.000303     810   Mp  0.002443
UCSD@5phy0.00815 0.001068     785 0.000294     785   Mn  0.002452
```

LPILE ANALYSES

Deflection Calculations

Note: Deflections are under load case 2(service) only.

1. Deflection @ worse location (Sta. 1+88) for Wall #1 Type A



Load @ pile head (Case2 only, see calc. of wall A-Station 2+52)

Laterial Force, V = 41.9 kips
Axial Force, P = 88.2 kips
bending moment @ bottom of wall, M = 875.8 kft
(w/o safety factor)
wall hight, h = 24.0 ft
equivalent w = 109.5 k/ft
E = 57sqrt(f'c) = 519120 ksf
f'c = 4000 psf
I = bd^3/12 3.849 ft^4
where b = (pile spacing) 8.00 ft
d = (average thickness of wall) 1.794 ft
Δwall = Wh^3/(15Ec x I) = 0.61 in

Δpile = (from Lpie) 0.28 in

Total Deflection @ Top of Wall, Δ = Δpile + Δwall = 0.89 in

2. Deflection @ left side of joint (Sta. 2+56) for Wall #1 Type A

Load @ pile head (Case2 only, see calc. of wall A-Station 2+52)

Laterial Force, V = 25.1 kips
Axial Force, P = 65.2 kips
bending moment @ bottom of wall, M = 506.5 kft
(w/o safety factor)
wall hight, h = 20.0 ft
equivalent w = 76.0 k/ft
E = 57sqrt(f'c) = 519120 ksf
f'c = 4000 psf
I = bd^3/12 3.849 ft^4
where b = (pile spacing) 8.00 ft

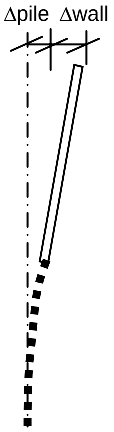
d = (average thickness of wall) 1.794 ft

Δwall = Wh^3/(15Ec x I) = 0.24 in

Δpile = (from Lpie) 0.11 in

Total Deflection @ Top of Wall, Δ = Δpile + Δwall = 0.35 in

3. Deflection @ right side of joint (Sta. 2+56) for Wall #1 Type B-station 2+61 (under load case 2, spacing-9ft)



4. Deflection @ right side of joint (Sta. 2+56) for Wall #1 Type B-station 2+61 (spacing-9ft)

Load @ pile head (Case2 only, see calc. of wall B-Station 2+61, spacing 9ft)

Moment, M = 512.5 kft

Laterial Force, V = 78.6 kips

Axial Force, P = 50.2 kips

(w/o safety factor)

wall hight, h = 19.3 ft

Δpile = from Lpile 0.53 in

Rotated angle of pile head ϕ = from Lpile 0.005513 rad

Δwall = ϕ x h = 1.28 in

Total Deflection @ Top of Wall, Δ = Δpile + Δwall = 1.81 in

Use H / 128 for offset of wall #1 type A

5. Deflection @ left side of joint (Sta. 4+67.79) for Wall #1 Type B-use typical section Sta. 3+15 (spacing 12ft)

Load @ pile head (Case2 only, see calc. of wall B-Station 3+15)

Moment, M = 388.4 kft

Laterial Force, V = 71.8 kips

Axial Force, P = 41.6 kips

(w/o safety factor)

wall hight, h = 16.0 ft

Δpile = from Lpile 0.42 in

Rotated angle of pile head ϕ = from Lpile 0.004414 rad

Δwall = ϕ x h = 0.85 in

MGE Engineering, Inc.

Project: Napa River Flood Control Project
Subject: LPile Analyses (Deflection)
By: David An
Date: Mar-05

Total Deflection @ Top of Wall, Δ = Δpile + Δwall = 1.27 in

Use H / 140 for offset of wall #1 type B

6. Deflection @ right side of joint (Sta. 4+67.79) for Wall #1 Type C (under load case 2, use results of Station 4+83)

Load @ pile head (Case2 only, see calc. of wall C-Station 4+83)
Moment, M = 158.1 kft
Laterial Force, V = 39.0 kips
Axial Force, P = 26.7 kips
(w/o safety factor)
wall hight, h = 11.9 ft

Δpile = from Lpile 0.5 in
Rotated angle of pile head ϕ = from Lpile 0.006877 rad

Δwall = ϕ x h = 0.98 in

Total Deflection @ Top of Wall, Δ = Δpile + Δwall = 1.48 in

7. Deflection @ for Wall #1 Type C (@ station 4+83, used for station 4+67.79 to 10+26.92)

Load @ pile head (Max. see calc. of wall C-station 4+83)
Moment, M = 265.2 kft
Laterial Force, V = 51.5 kips
Axial Force, P = 26.7 kips
(w/o safety factor)
wall hight, h = 11.88 ft

Δpile = from Lpile 0.62 in
Rotated angle of pile head ϕ = from Lpile 0.00793 rad

Δwall = ϕ x h = 1.13 in

Total Deflection @ Top of Wall, Δ = Δpile + Δwall = 1.75 in

Use H / 81 for offset of wall #1 type C station 4+67.79 to 10+26.92)

8. Deflection @ for Wall #1 Type C (The forces use results @ station 4+83, but soil profile is from 9+30 to end of wall)
Used for station11+16.92 to end of wall)

Load @ pile head (Max. see calc. of wall C-station 4+83)
Moment, M = 265.2 kft
Laterial Force, V = 51.5 kips
Axial Force, P = 26.7 kips
Station 9+30 to end of wall profile is used.
(w/o safety factor)
wall hight, h = 11.88 ft

Δpile = from Lpile 1.47 in
Rotated angle of pile head ϕ = from Lpile 0.013608305 rad

Δwall = ϕ x h = 1.94 in

Total Deflection @ Top of Wall, Δ = Δpile + Δwall = 3.41 in

Use H / 42 for offset of wall #1 type C station 11+16.92 to end of wall)

Use Dowels @ Contration Joints.

Pile Head Loads for Station 2+52 (Wall #1, Type A)

Backfill Properties

Backfill Thickness = (17.00') - (-3.00') = 20.00 ft
Backfill Unit Weight = 125 pcf
Φ = 37 degree
C = 0 pcf
SMF= $\tan(\Phi_d) / \tan\Phi = 2/3$ = 0.67
Φ_d = 27 degree
Ka = $\tan^2(45° - \Phi/2)$ = 0.25
Ko = $\tan^2(45° - \Phi_d/2)$ = 0.38
Kp = $\tan^2(45° + \Phi/2)$ = 4.02

Water Property

Water Unit Weight = 62.5 pcf

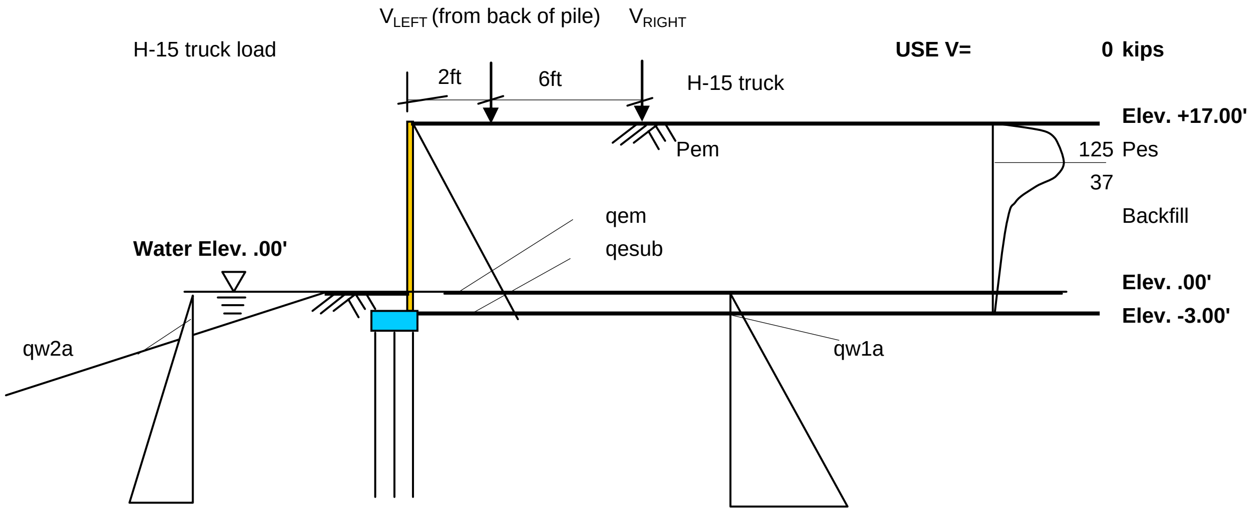
Pile and Wall Data

Station = **2+52**
Finish Grade Elevation(behind) = 17.00 ft
Finish Grade Elevation(front) = 0.00 ft
Top of Footing Elevation = -3.00 ft
Pile Spacing = 8.00 ft
Pile Diameter = 2.00 ft
100 Year Flood Level = 15.27 ft
Water Elevation (Mean higher) = 3.76 ft
Water Elevation (Mean lower) = -2.84 ft

MGE ENGINEERING, INC.

Project	Napa River Flood Control Project		
Subject	Flood Wall Design (Wall #1, Type A)		
By	David An	Date	Mar-05

Load Case 2 -- Long Term (Drained) In Service Condition (Station 2+52)



USE V= **0 kips**

Backfill Soil Pressure at Wall (Soil pressure = γ Ki hi)

Name	Thickness(ft)	Pressure(ksf)
qem	17.00	0.808
qesub	3.00	0.880
qw1a=qw2a	3.00	0.188

Note: Passive soil resistance were ignored.

qem - Moist soil pressure at rest wall

qesub - Submerged soil pressure at rest wall

qw - Water pressure

Backfill Resultant Forces Summary

Name	Force	Arm to bot.	Moments
Pem	55.0	8.67	477
Pesub	20.3	1.48	30
Pw1a	2.3	1.00	2
Ph-15	0.0		0
Pw2a	-2.3	1.00	-2
At bot of wall	**75.2**	Safety Factor	**506.5**
	Σ V	**1.3**	Σ M

H-15 Truck Loading Summary (Left)

b (for V_{LEFT})	Z	$\Delta P_{PH\ (LEFT)}$	Moment
0.1	2.00	0.000	0.000
0.2	4.00	0.000	0.000
0.3	6.00	0.000	0.000
0.4	8.00	0.000	0.000
0.5	10.00	0.000	0.000
0.6	12.00	0.000	0.000
0.7	14.00	0.000	0.000
0.8	16.00	0.000	0.000
0.9	18.00	0.000	0.000
1.0	20.00	0.000	0.000
Σ		0.000	0.000

H-15 Truck Loading Summary (Right)

b (for V_{RIGHT})	Z	$\Delta P_{PH\ (RIGHT)}$	Moment
0.1	2.00	0.000	0.000
0.2	4.00	0.000	0.000
0.3	6.00	0.000	0.000
0.4	8.00	0.000	0.000
0.5	10.00	0.000	0.000
0.6	12.00	0.000	0.000
0.7	14.00	0.000	0.000
0.8	16.00	0.000	0.000
0.9	18.00	0.000	0.000
1.0	20.00	0.000	0.000
Σ		0.000	0.000

h = 20.00 ft

$\Delta P_{HZ} = (0.28V/h^2)\ [b^2 / (0.16+b^2)^3]$ (EM 1110-2-2502 Page 3-49)

a=2'/20.00' = 0.10 ≤ 0.4 **For V_{LEFT}**

a=8'/20.00' = 0.40 ≤ 0.4 **For V_{RIGHT}**

Demand at Top of Pile: Vd = 98 kips Md = 658 k-ft

MGE ENGINEERING, INC.

Project Napa River Flood Control Project
Subject Flood Wall Design (Wall #1, Type A)
By David An
Date Mar-05

Loads at Pile Head (Station 2+52)

1. Loads

	Load Case	1	2	3	4
Forces w/o Safety Factor	Shear (k)		75.2		
	Moments(kft)		506.5		
	Safety Factor		1.3		
Forces w/ Safety Factor	Shear (k)		97.8		
	Moments(kft)		658.5		

Demand at top of Footing:

Vd= **75** kips

Md= **507** kft

Pd = [L x (Bt + Bb) / 2 x h) x 0.15 = **40** kips

Where

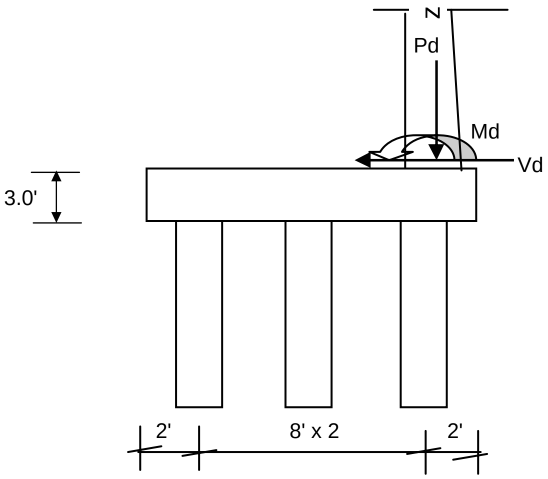
Wall thickness @ Top Bt = 1.00 ft

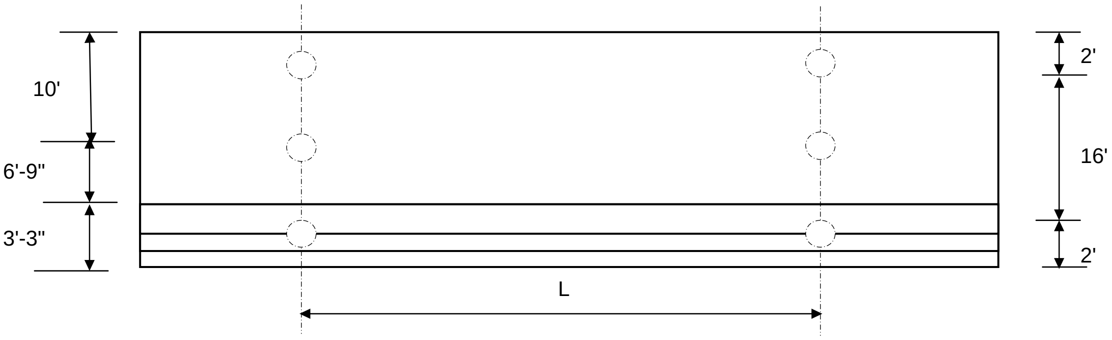
Wall thickness @ Bottom (1:15 batter) Bb = 2.33 ft

Front of Wall to Center of Footing (6'-9") = 6.75 ft

Wall Height h = 20.00 ft

Pile Spacing L = 8.00 ft





Plan View

Piles Force (under load case 2)

Loads at Bottom of Footing

Mdmax=	Md - Pd x c - Σ(Wsoil x Arms) + Vd*D/2	102	k-ft
Vdmax =	Vd	75	kips
Pdmax=	Pd +Wfooting + Wsoil	176	kips

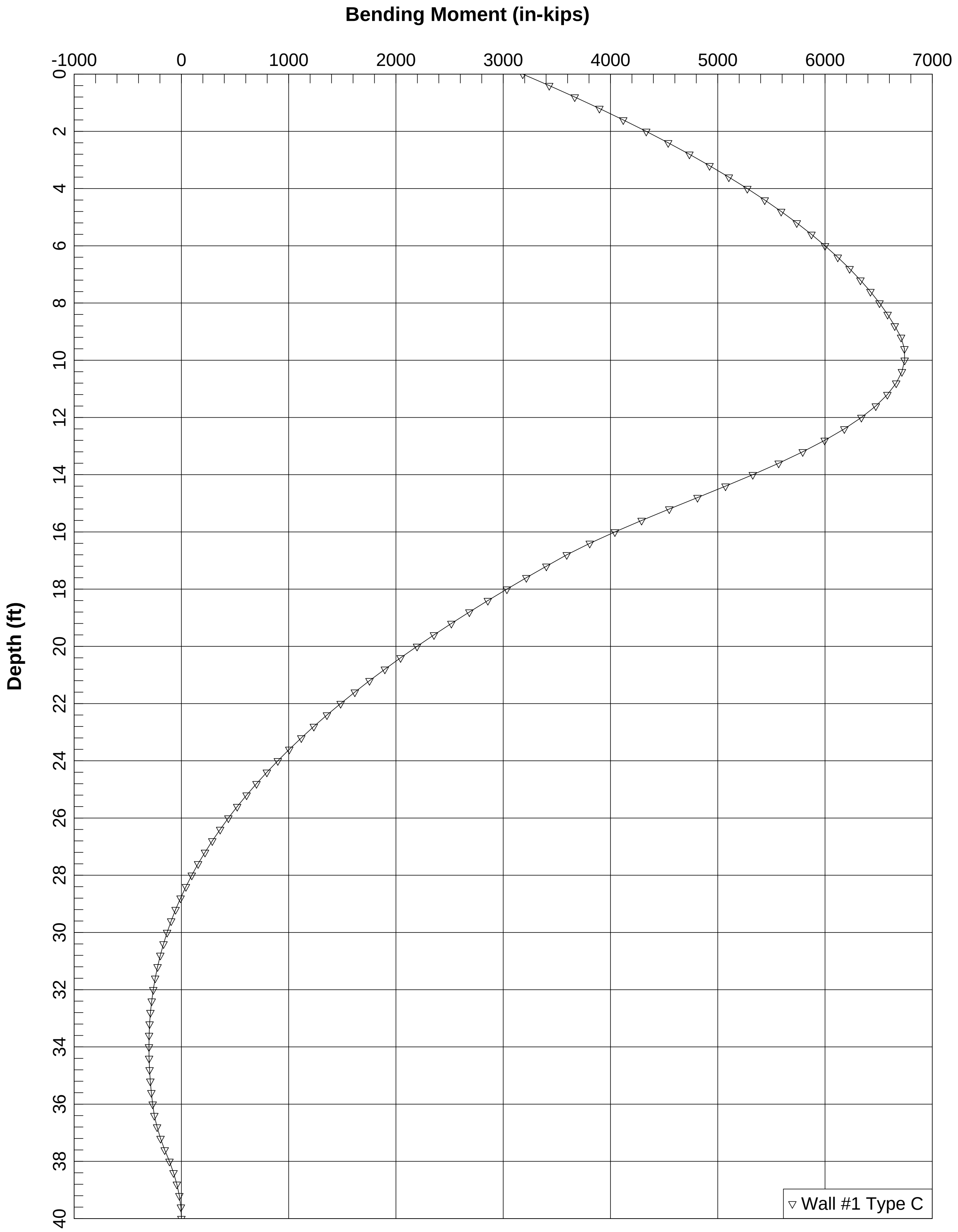
Where

Md = (Load Case2 without Embedment Safety Factors)	507	k-ft
Vd = (Same as Md)	75	kips
Pd =	40.0	kips
c = From Center of Wall to Center of Footing	7.92	ft
D = Depth of Footing	3.00	ft

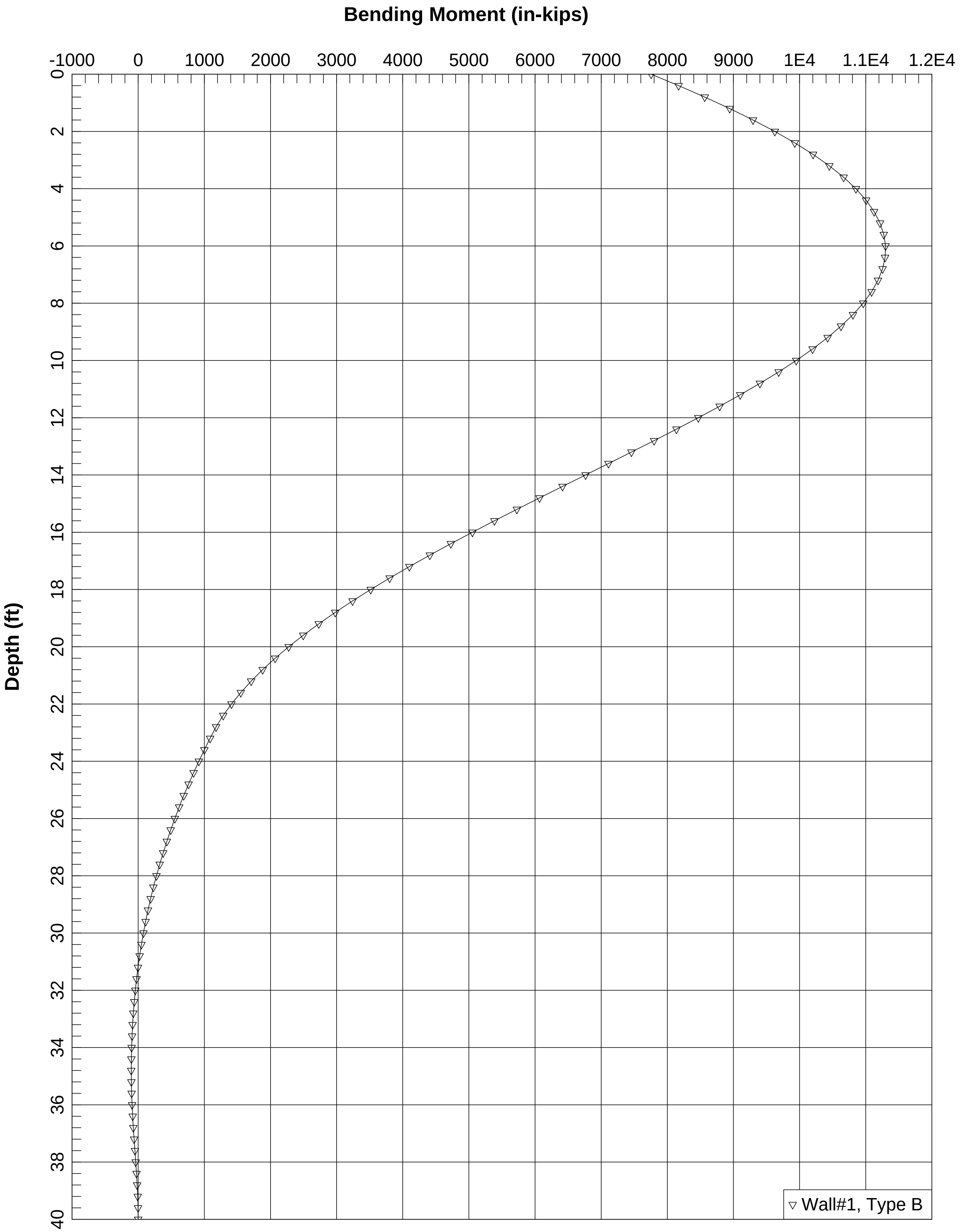
Wfooting = 16 x D x L x 0.15	57.6	kips
Wsoil-1 (resisting side, RSP, Rec.) = 3' x (20'/2+6.75') x L x 0.12	48	kips
Moment Arm for Wsoil-1 = 20'/2 - (20'/2+6.75')/2	1.6	ft
Wsoil-2 (driving, Rec.) = (20'/2-6.75'-2.33') x (20.00') x L x 0.12	18	kips
Moment Arm for Wsoil-3 = (20'/2-6.75-2.33')/2 - 20'/2	-9.5	ft
Wsoil-3 (driving, Tri.) = (2.33'-1.00') x (20.00') x L / 2 x 0.12	13	kips
Moment Arm for Wsoil-3 = - (2.33-1.00') * 2 / 3 - 1.00' - 6.75'	-8.6	ft

Pile Force

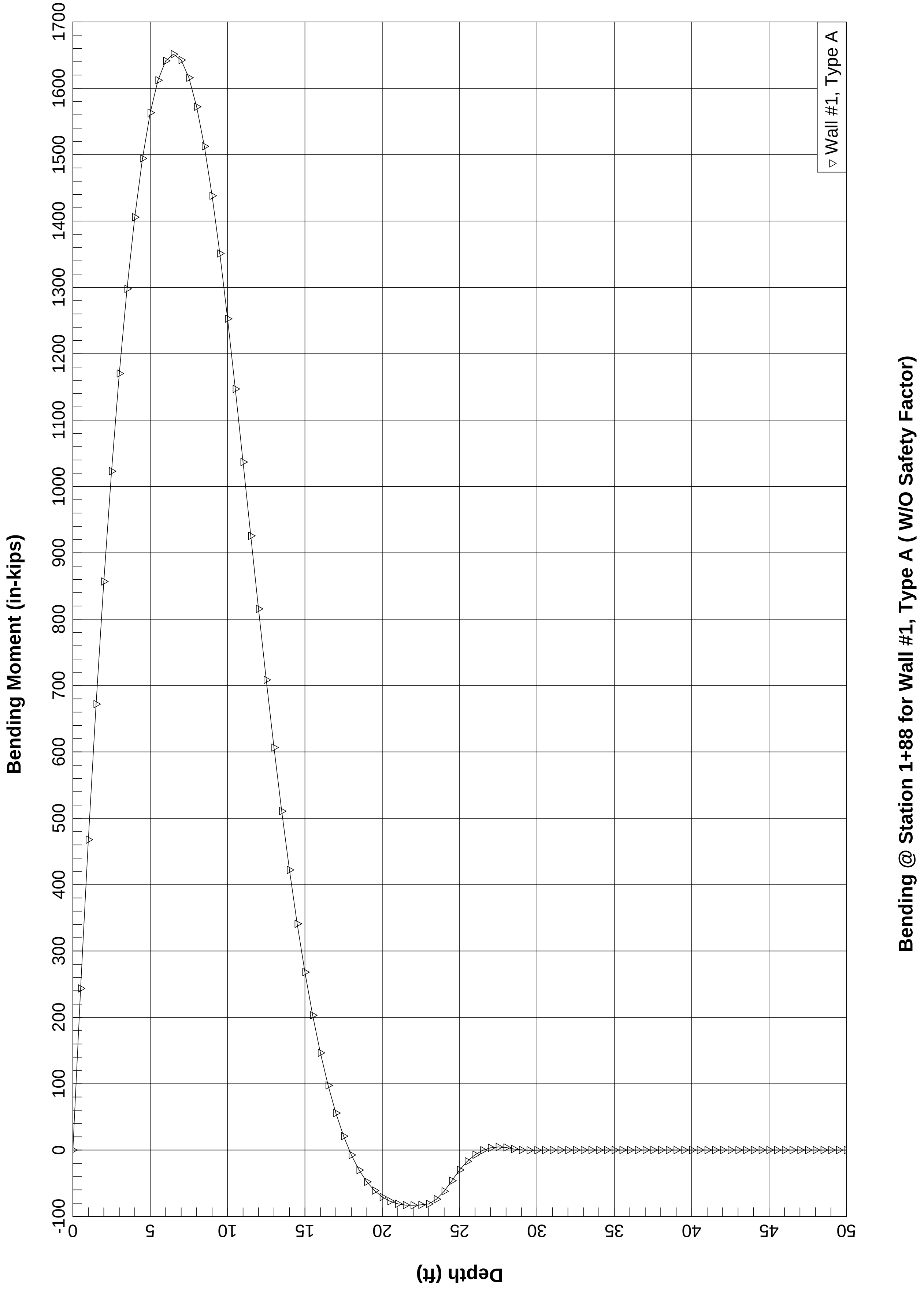
I_{PILES} = 8^2*2 =	(3 Rows 8ft x 2)		128	ft^2
Pile reaction				
Tension		Rt=Pdmax/2-Mdmax*di / Ipiles =	**52.3**	kips
Compression		Rc=Pdmax/2+Mdmax*di / Ipiles =	**65.2**	kips
Laterial Force	Vpile = (Vd-Rsp)/2	(2 piles take lateral force)	**25.1**	kips
where				
Laterial resistance of ftgs	@ Top of ftg	q_{pt} = Kp x γ' x h1	0.69	ksf
	Where	γ' = 120-62.5	57.50	pcf
		h1 =	3.00	ft
	@ Top of ftg	q_{pb} = Kp x γ' x h1	1.39	ksf
	Where	γ' = 120-62.5	57.50	pcf
		h2 =	6.00	ft
	$R_{sp} = (q_{pt}+q_{pb})/2$ x (h2-h1) x L		24.98	kips

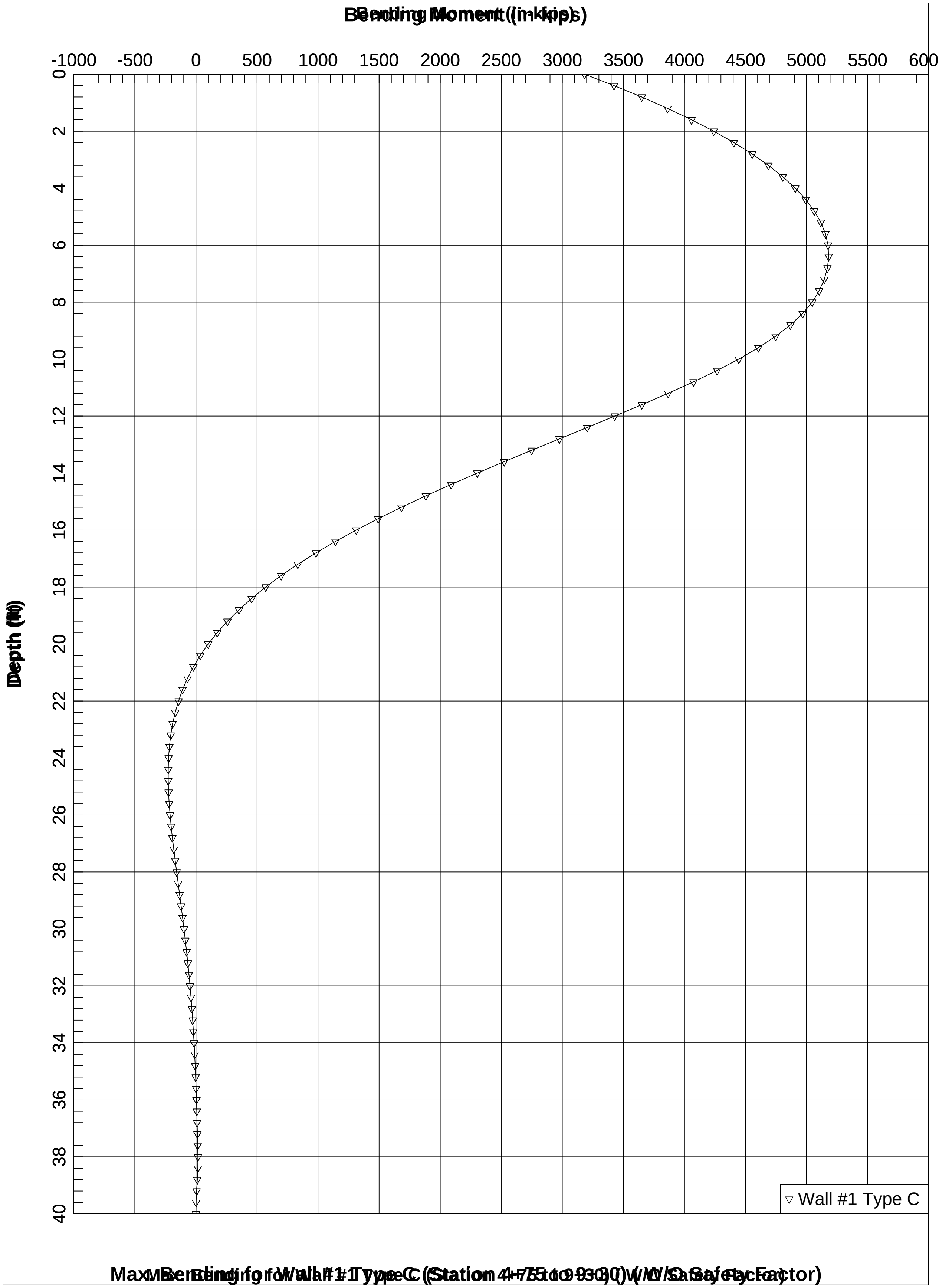


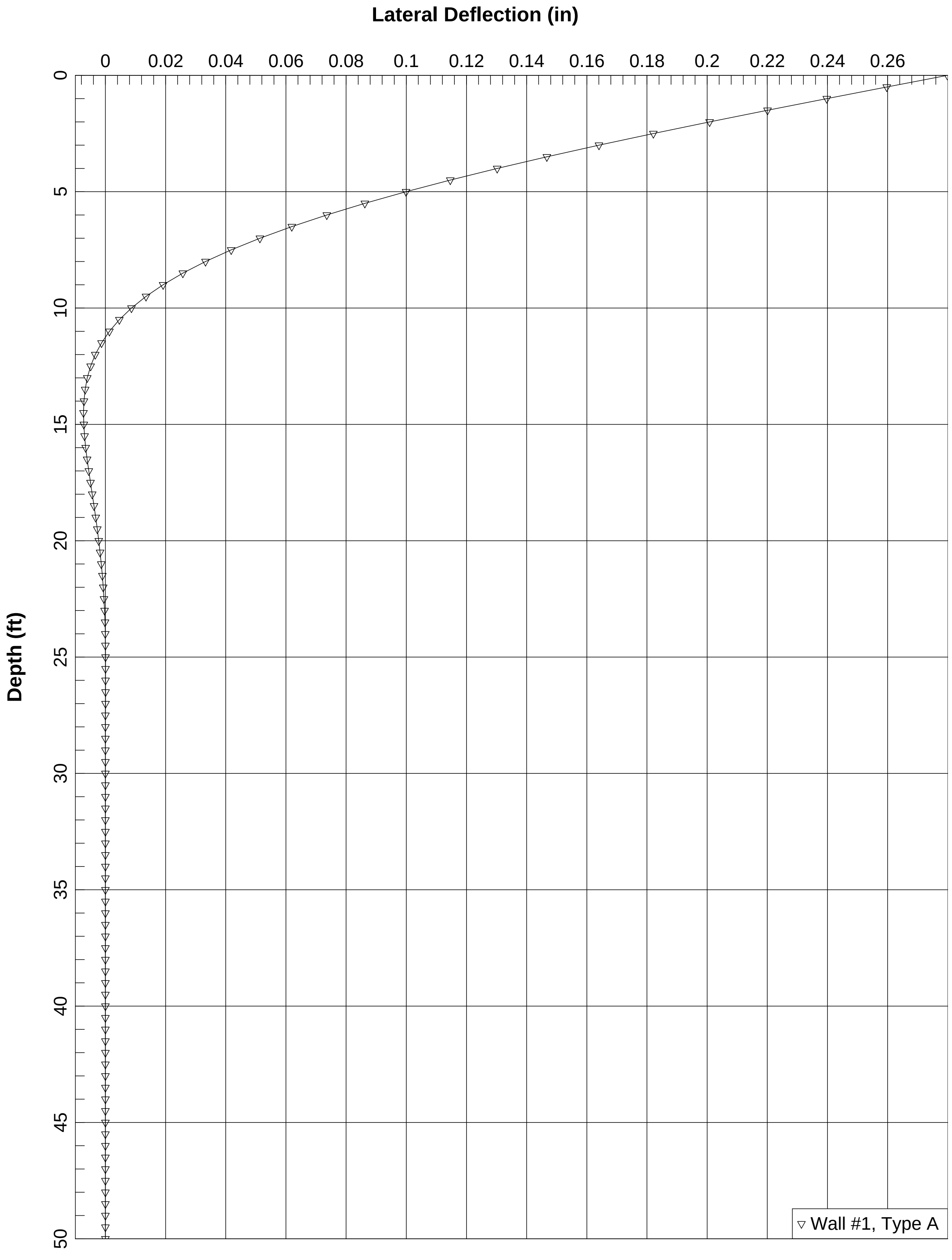
Max. Bending for Wall #1 Type C (Station 9+30 to End of Wall) (W/O Safety Factor)



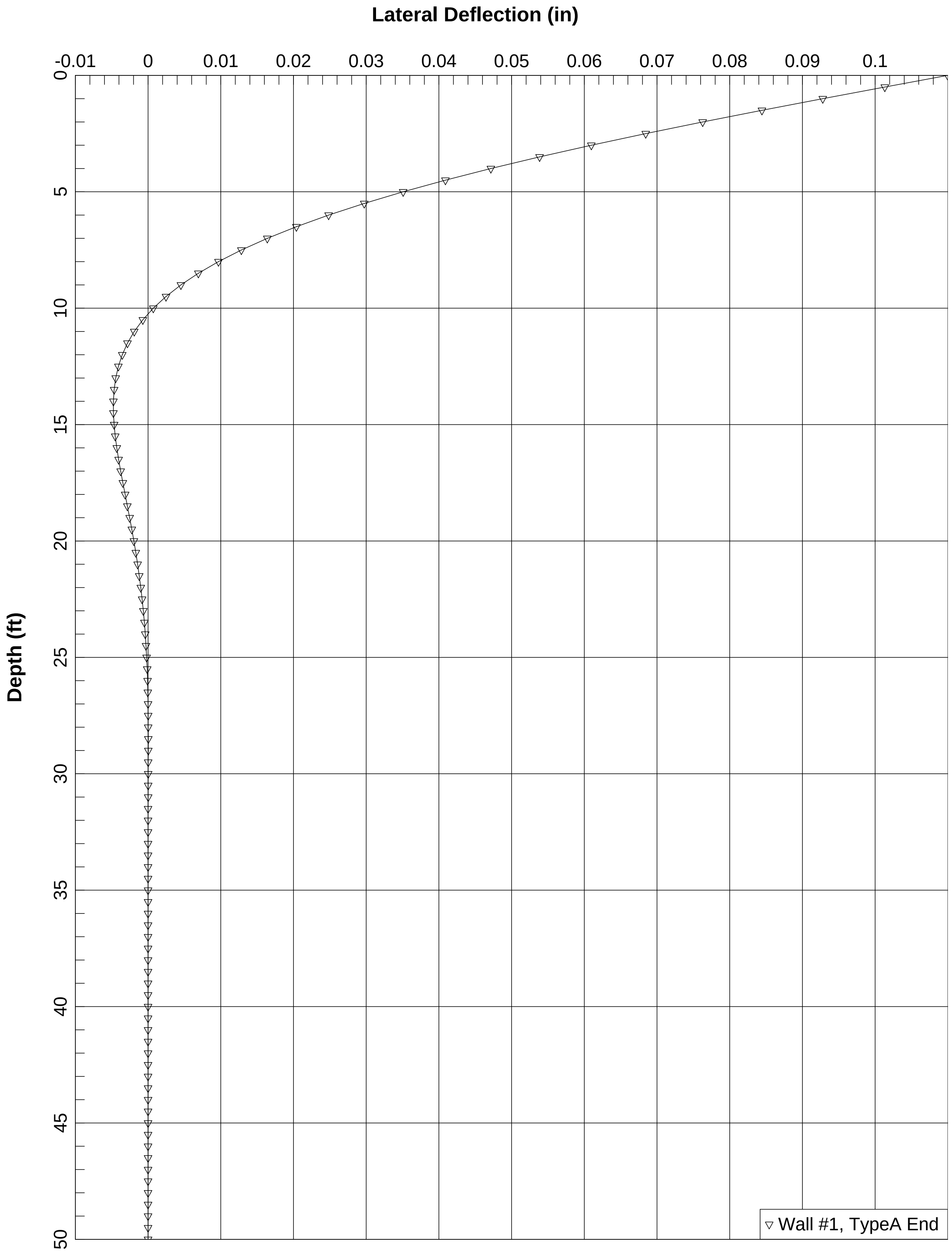
Wall #1 Type B, Max. Bending @ Station 2+61 (W/O Safety Factor)


Bending @ Station 1+88 for Wall #1, Type A (W/O Safety Factor)
Bending Moment (in-kips)
Depth (ft)
▹ Wall #1, Type A
-100
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
1600
1700
0
5
10
15
20
25
30
35
40
45
50

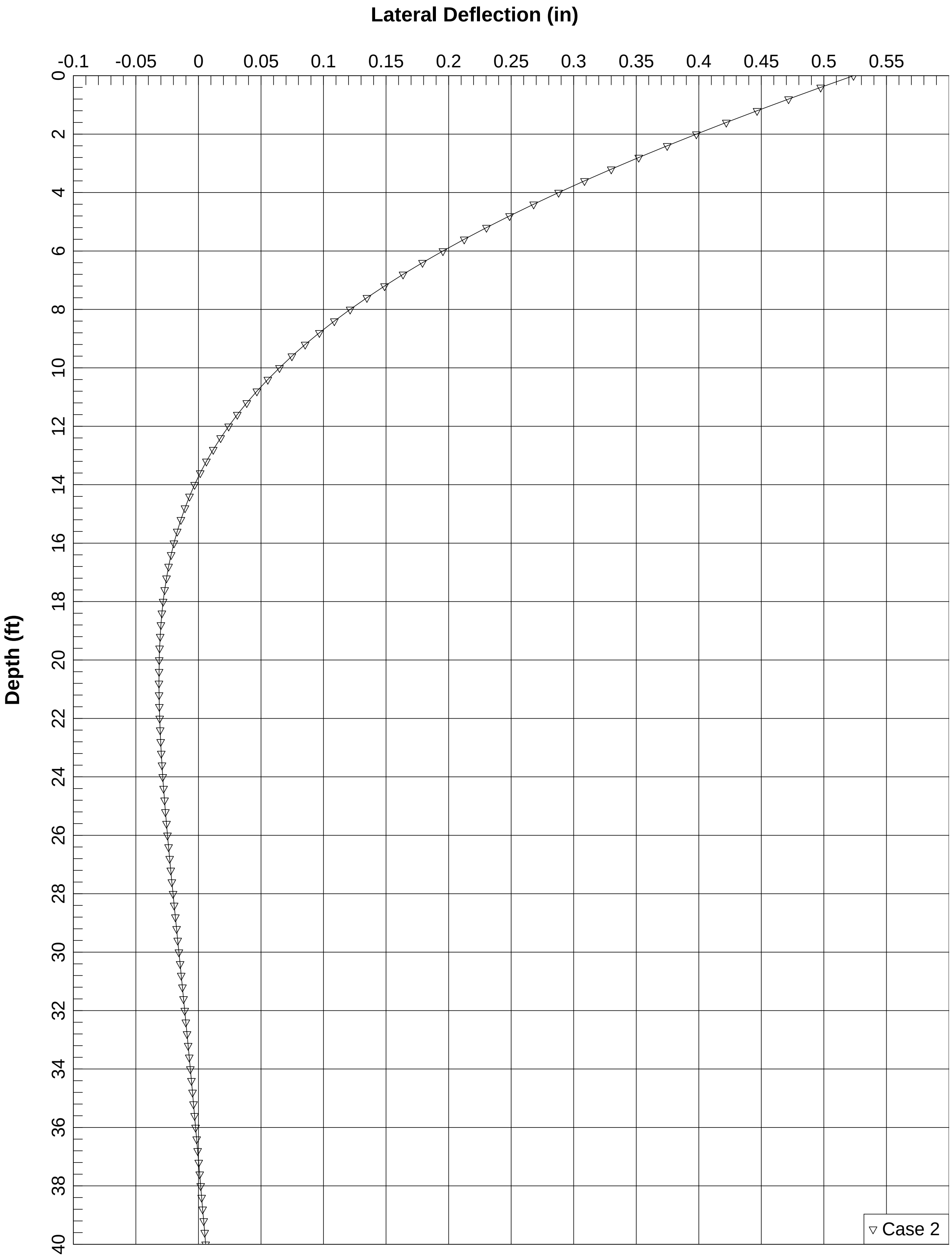

Bending Moment (in-kips)
-1000
-500
0
500
1000
1500
2000
2500
3000
3500
4000
4500
5000
5500
6000
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
▽ Wall #1 Type C
Max Bending for Wall #1 Type C (Station 4+75 to 9+30) (W/O Safety Factor)



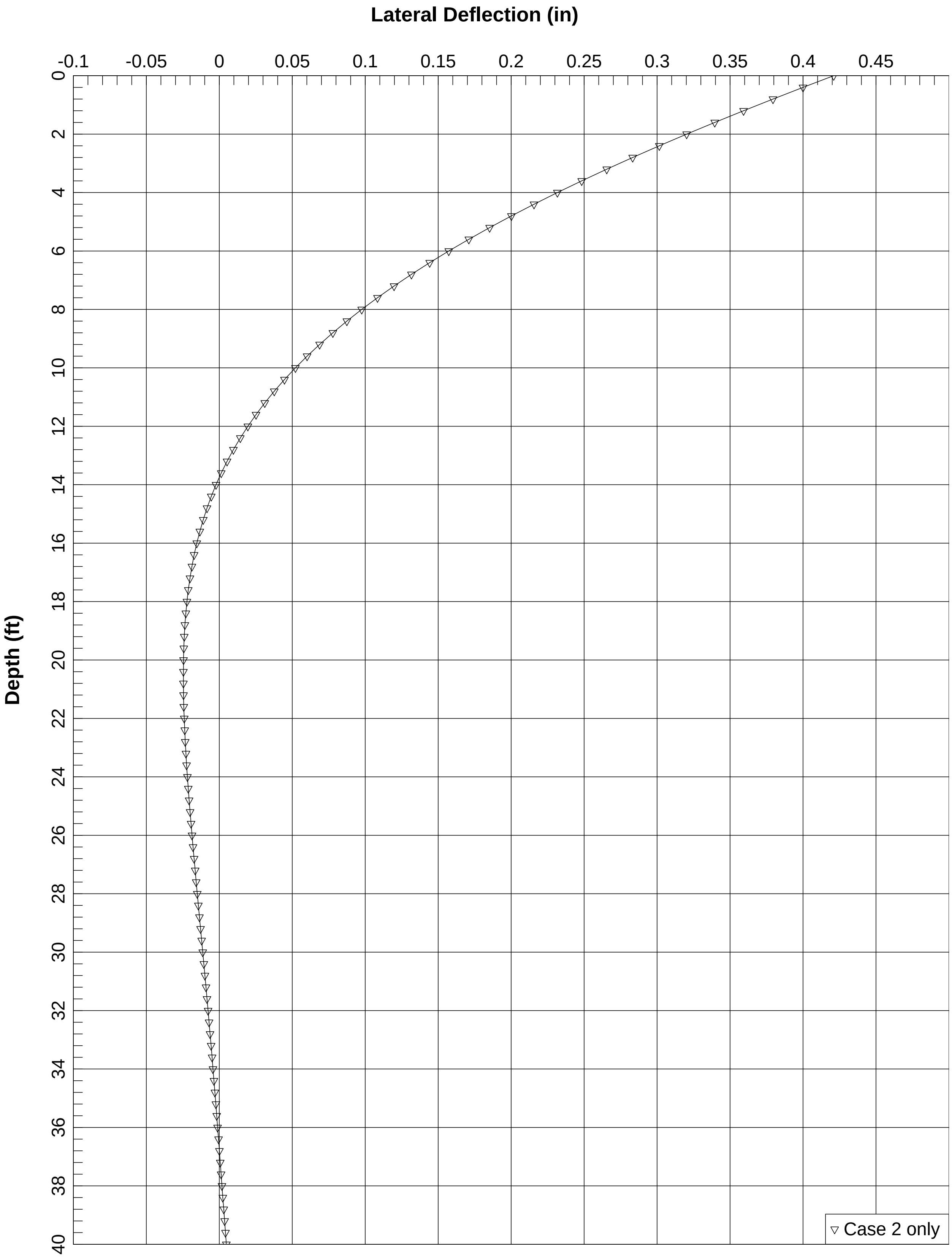
Pile Head Deflection @ Sta. 1+88, Wall #1, Type A



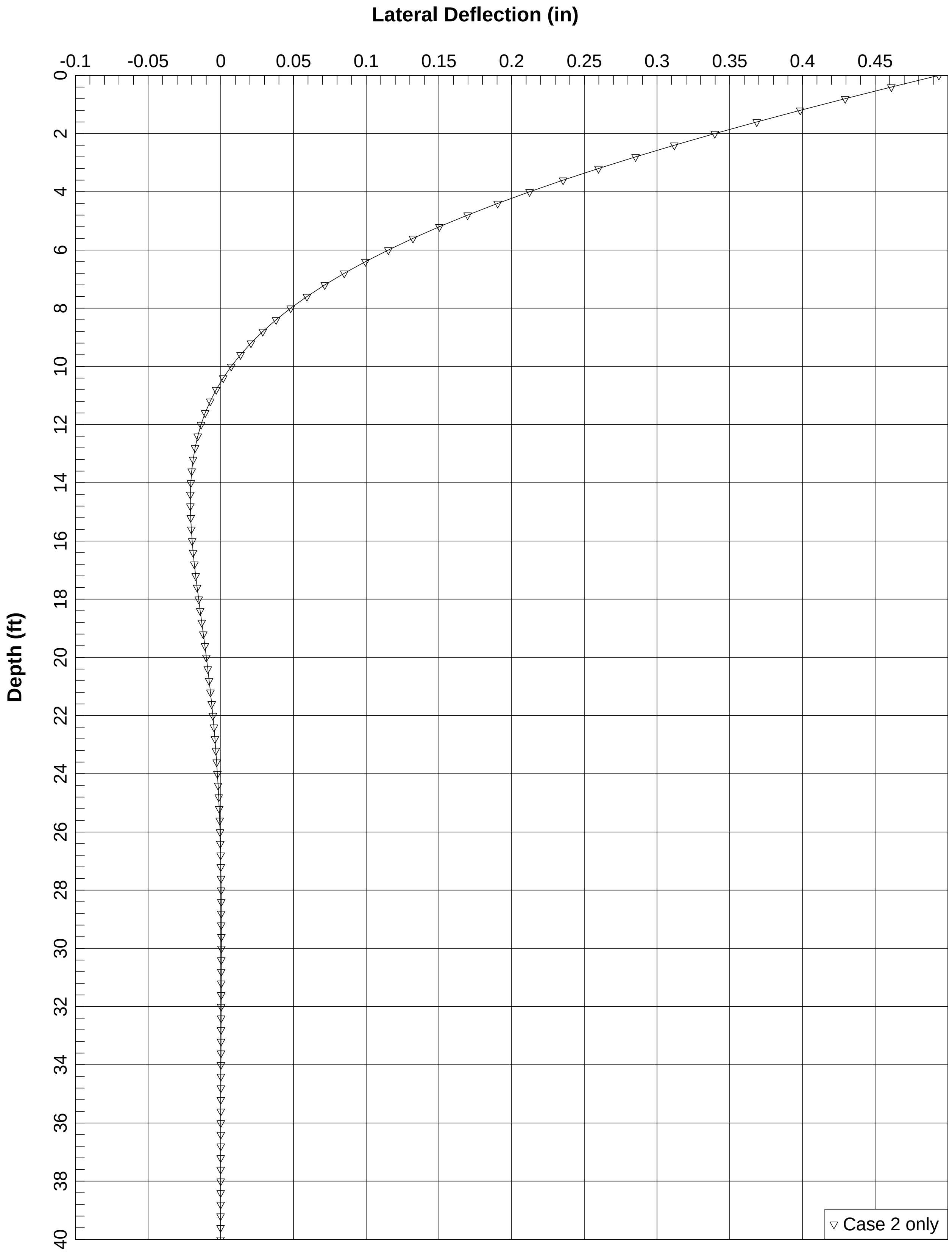
Pile Head Deflection @ Left Side of Joint (Station 2+56) Between Wall#1 Type A & Type B



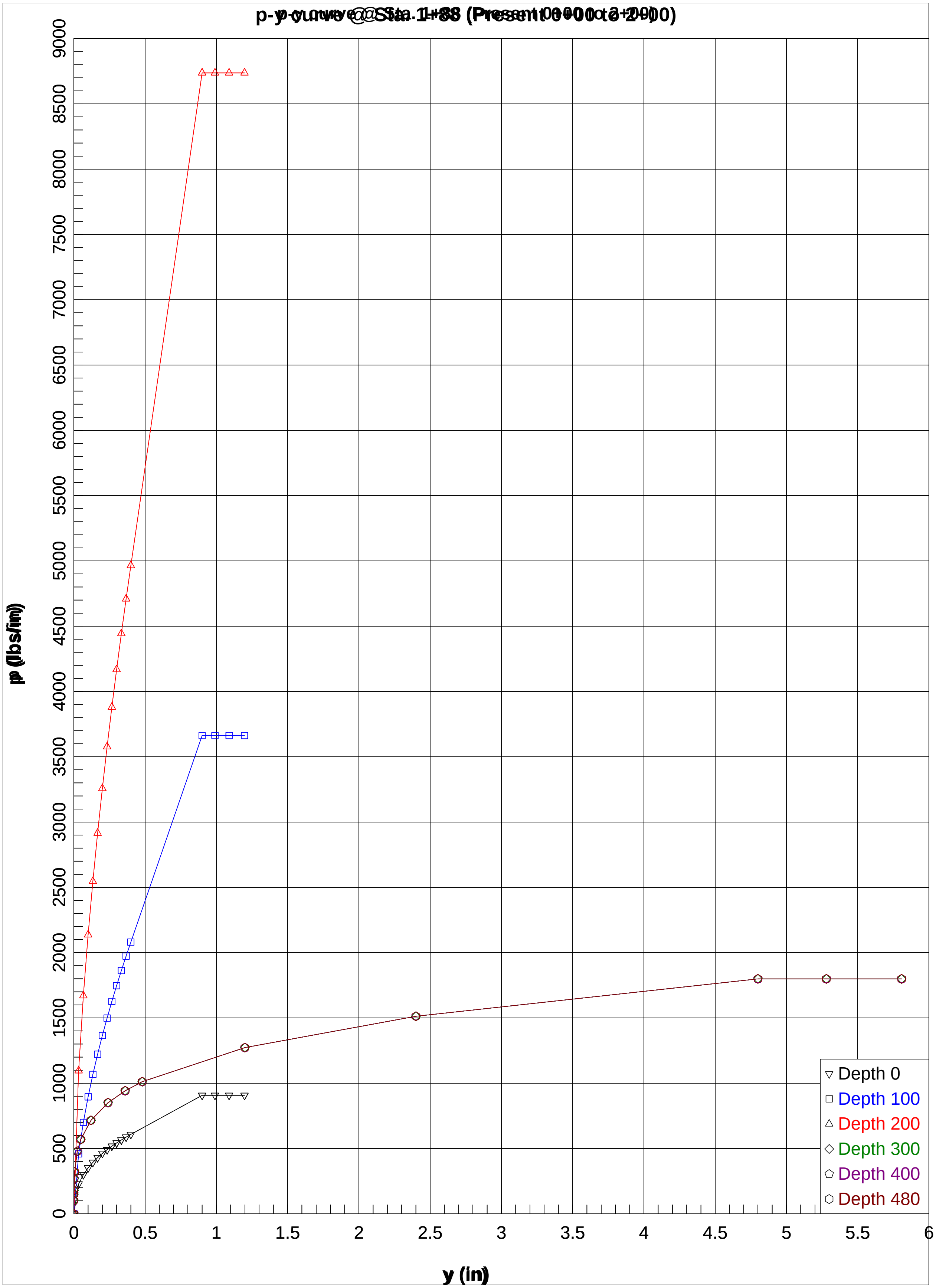
Pile Head Deflection @ Right Side of Joint (Station 2+56) Between Wall #1 Type A & Type B

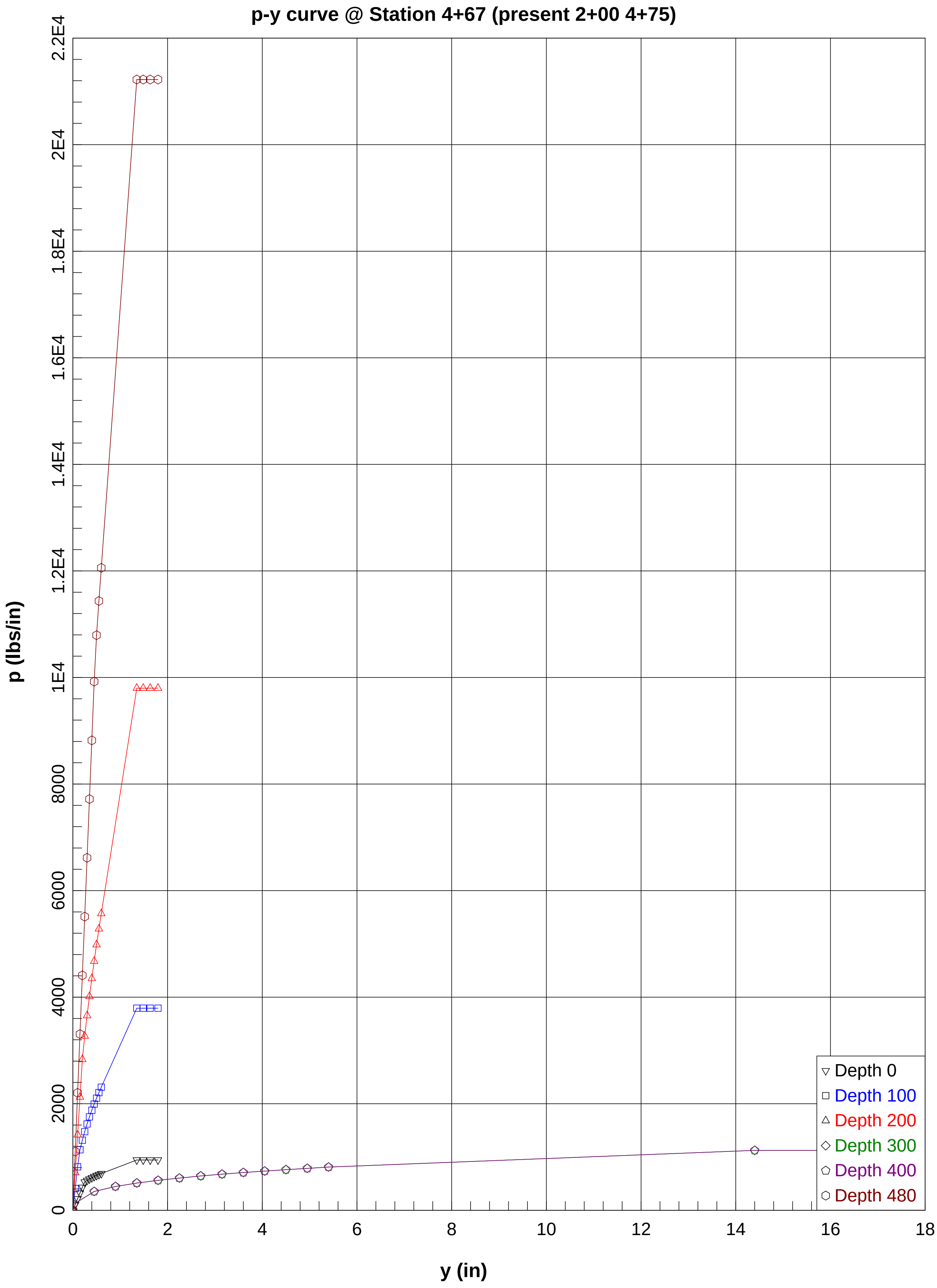


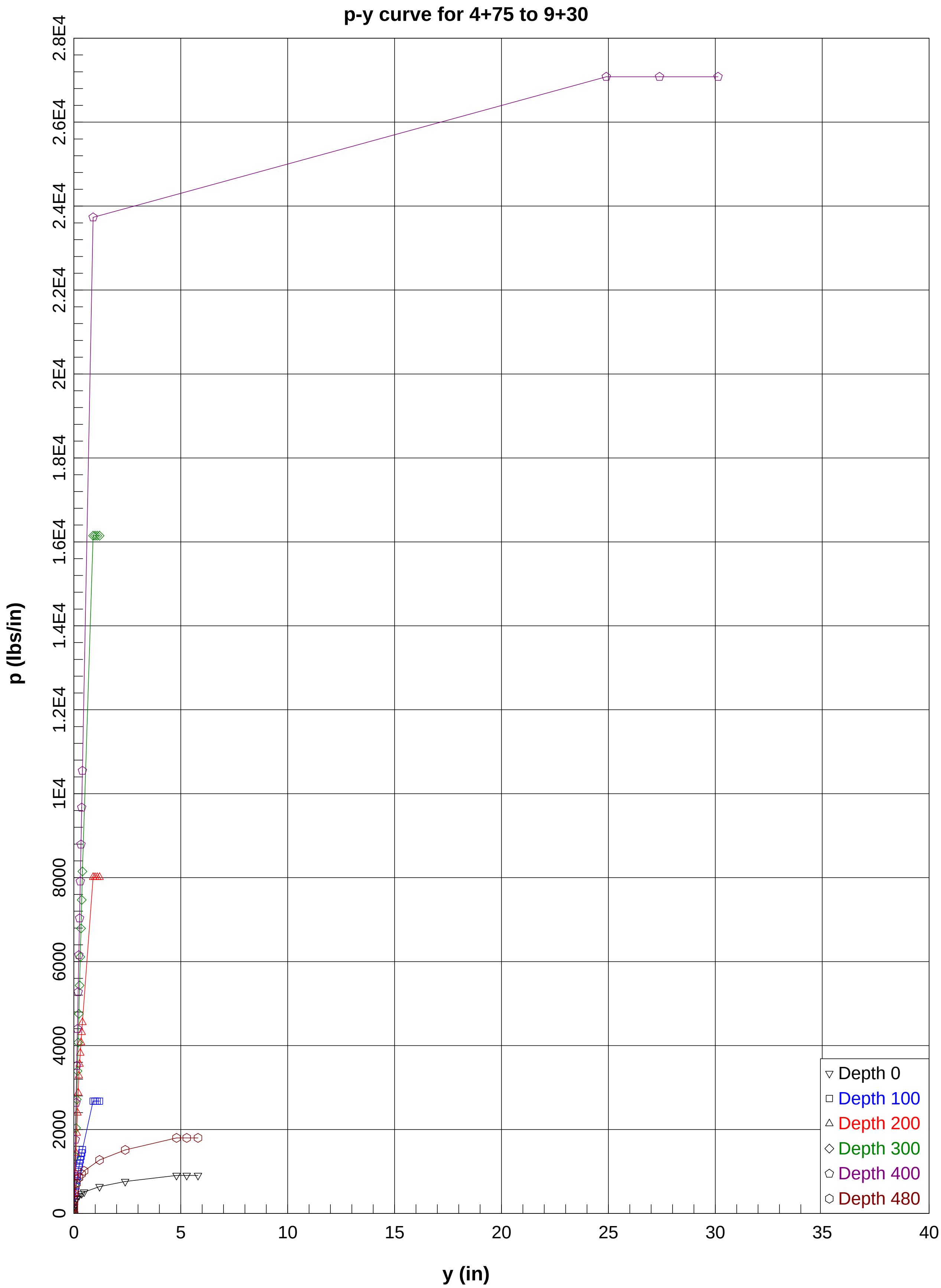
Pile Head Deflection @ Left Side of Joint (Station 4+67) Between Wall #1 Type B & Type C

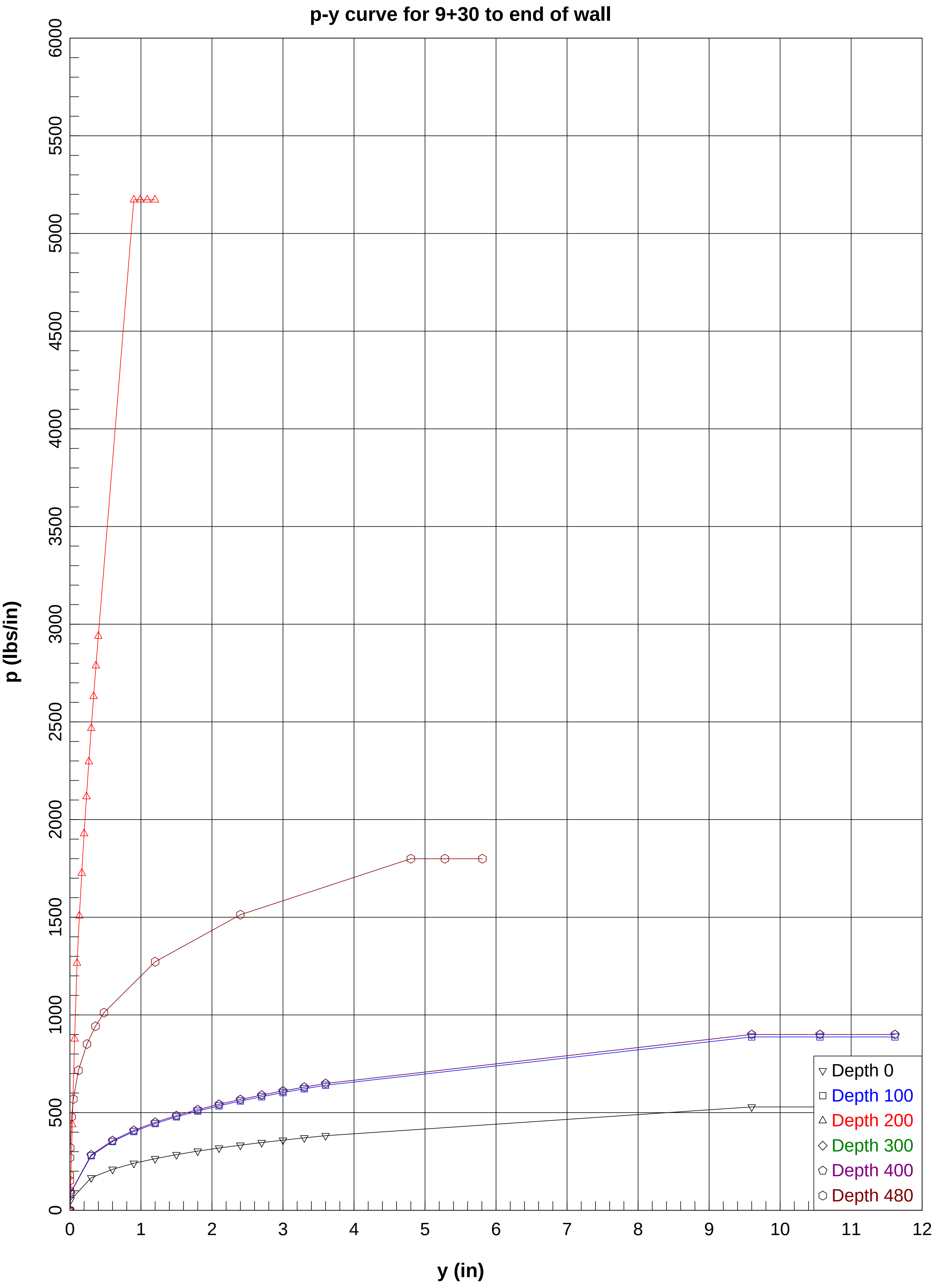


Pile Head Deflection @ Right Side of Joint (Station 4+67) Between Wall #1 Type B & Type C


p-y curve @ Sta. 1+88 (Present 0+00 to 2+00)
p (lbs/in)
0
500
1000
1500
2000
2500
3000
3500
4000
4500
5000
5500
6000
6500
7000
7500
8000
8500
9000
y (in)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
6
▽ Depth 0
□ Depth 100
△ Depth 200
◇ Depth 300
⬠ Depth 400
⬡ Depth 480


p-y curve @ Station 4+67 (present 2+00 4+75)
p (lbs/in)
y (in)
0
2000
4000
6000
8000
1E4
1.2E4
1.4E4
1.6E4
1.8E4
2E4
2.2E4
0
2
4
6
8
10
12
14
16
18
▽ Depth 0
□ Depth 100
△ Depth 200
◇ Depth 300
⬠ Depth 400
⬡ Depth 480


p-y curve for 4+75 to 9+30
p (lbs/in)
y (in)
0
2000
4000
6000
8000
1E4
1.2E4
1.4E4
1.6E4
1.8E4
2E4
2.2E4
2.4E4
2.6E4
2.8E4
0
5
10
15
20
25
30
35
40
▽ Depth 0
□ Depth 100
△ Depth 200
◇ Depth 300
⬠ Depth 400
⬡ Depth 480


p-y curve for 9+30 to end of wall
p (lbs/in)
y (in)
0
500
1000
1500
2000
2500
3000
3500
4000
4500
5000
5500
6000
0
1
2
3
4
5
6
7
8
9
10
11
12
▽ Depth 0
□ Depth 100
△ Depth 200
◇ Depth 300
⬠ Depth 400
⬡ Depth 480

```
                                   Napa24AMax
==============================================================================

               LPILE Plus for Windows, Version 4.0 (4.0.10)

              Analysis of Individual Piles and Drilled Shafts
             Subjected to Lateral Loading Using the p-y Method

                   (c) Copyright ENSOFT, Inc., 1985-2003
                            All Rights Reserved

==============================================================================


This program is licensed to:

david an
mge

Path to file locations:      C:\DA Works\Lpile\
Name of input data file:     Napa24AMax.lpd
Name of output file:         Napa24AMax.lpo
Name of plot output file:    Napa24AMax.lpp
Name of runtime file:        Napa24AMax.lpr


------------------------------------------------------------------------------
                          Time and Date of Analysis
------------------------------------------------------------------------------

               Date:  March 26, 2005     Time:  21: 5:10


------------------------------------------------------------------------------
                                Problem Title
------------------------------------------------------------------------------

Napa River Flood Control Project--24 CIDH Pile , Type A-Station 1+88, Max. Loads


------------------------------------------------------------------------------
                               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 includes automatic computation of pile-top deflection vs.
  pile embedment length
- No computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- Additional p-y curves computed at 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                               =     600.00 in
Depth of ground surface below top of pile =     -72.00 in
Slope angle of ground surface             =        .00 deg.

Structural properties of pile defined using  2 points

Point    Depth         Pile      Moment of       Pile      Modulus of
           X         Diameter     Inertia        Area      Elasticity
          in            in         in**4        Sq.in      lbs/Sq.in
-----  ---------   -----------  -----------   ----------   -----------
  1       0.0000       24.000   16286.0000     452.4000   3500000.000
  2     600.0000       24.000   16286.0000     452.4000   3500000.000


------------------------------------------------------------------------------
                      Soil and Rock Layering Information
------------------------------------------------------------------------------

The soil profile is modelled using  4 layers

Layer  1 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer    =      -72.000 in
Distance from top of pile to bottom of layer =      276.000 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  2 is stiff clay without free water
Distance from top of pile to top of layer    =      276.000 in
Distance from top of pile to bottom of layer =      576.000 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  3 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer    =      576.000 in
Distance from top of pile to bottom of layer =      696.000 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    =      696.000 in
Distance from top of pile to bottom of layer =      768.000 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


(Depth of lowest layer extends  168.00 in below pile tip)

------------------------------------------------------------------------------
```

```
                             Napa24AMax
                  Effective Unit Weight of Soil vs. Depth
------------------------------------------------------------------------------

Distribution of effective unit weight of soil with depth
is defined using  8 points

Point        Depth X    Eff. Unit Weight
 No.           in          lbs/in**3
-----      ----------   ----------------
  1          -72.00          .03180
  2          276.00          .03180
  3          276.00          .04110
  4          576.00          .04110
  5          576.00          .03180
  6          696.00          .03180
  7          696.00          .04110
  8          768.00          .04110


------------------------------------------------------------------------------
                           Shear Strength of Soils
------------------------------------------------------------------------------

Distribution of shear strength parameters with depth
defined using  8 points

Point    Depth X     Cohesion c     Angle of Friction     E50 or      RQD
 No.       in         lbs/in**2            Deg.            k_rm        %
-----    --------    ----------     ------------------    ------    ------
  1      -72.000         .00000           38.00           ------    ------
  2      276.000         .00000           38.00           ------    ------
  3      276.000        8.33000             .00           .00500        .0
  4      576.000        8.33000             .00           .00500        .0
  5      576.000         .00000           38.00           ------    ------
  6      696.000         .00000           38.00           ------    ------
  7      696.000        8.33000             .00           .00500        .0
  8      768.000        8.33000             .00           .00500        .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    =       41900.000 lbs
Bending moment at pile head =            .000 in-lbs
Axial load at pile head     =       88200.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)



-------------------------------------------------------------------------------
                   Output of p-y Curves at Specified Depths
-------------------------------------------------------------------------------


p-y curves are generated and printed for verification at  6 depths.

Depth      Depth Below Pile Head      Depth Below Ground Surface
 No.                in                            in
-----      ---------------------      --------------------------
  1                 .000                        72.000
  2              100.000                       172.000
  3              200.000                       272.000
  4              300.000                       372.000
  5              400.000                       472.000
  6              480.000                       552.000

Depth of ground surface below top of pile =     -72.00 in


p-y Curve in Sand Computed Using Reese Criteria

Soil Layer Number            =           1
Depth below pile head        =        .000 in
Depth below ground surface   =      72.000 in
Equivalent Depth (see note)  =      72.000 in
Pile Diameter                =      24.000 in
Angle of Friction            =      38.000 deg.
Avg. Eff. Unit Weight        =      .03180 lbs/in**3
k                            =     125.000 pci
A (static)                   =      1.0600
B (static)                   =       .7100
Pst                          =     855.956 lbs/in
Psd                          =    4372.464 lbs/in
Ps                           =     855.956 lbs/in
pu                           =     907.313 lbs/in
Cbar                         =    872.2559
n                            =      2.5357
m                            =    599.1693
yk                           =       .0212  in
ym                           =       .4000  in
yu                           =       .9000  in
p-multiplier                 =     1.00000
y-multiplier                 =     1.00000

If Psd <= Pst then actual depth is used in place of equivalent depth in
computations.

        y, in             p, lbs/in
----------------    -----------------
```

```
          .0000            .000
          .0333         228.096
          .0667         299.801
          .1000         351.785
          .1333         394.048
          .1667         430.296
          .2000         462.374
          .2333         491.355
          .2667         517.923
          .3000         542.548
          .3333         565.566
          .3667         587.229
          .4000         607.729
          .9000         907.313
        24.9000         907.313
        48.9000         907.313
        72.9000         907.313



p-y Curve in Sand Computed Using Reese Criteria

Soil Layer Number            =           1
Depth below pile head        =     100.000 in
Depth below ground surface   =     172.000 in
Equivalent Depth (see note)  =     172.000 in
Pile Diameter                =      24.000 in
Angle of Friction            =      38.000 deg.
Avg. Eff. Unit Weight        =      .03180 lbs/in**3
k                            =     125.000 pci
A (static)                   =       .8800
B (static)                   =       .5000
Pst                          =    4161.706 lbs/in
Psd                          =   10445.332 lbs/in
Ps                           =    4161.706 lbs/in
pu                           =    3662.301 lbs/in
Cbar                         =   3632.3602
n                            =      1.6447
m                            =   3162.8964
yk                           =       .0107  in
ym                           =       .4000  in
yu                           =       .9000  in
p-multiplier                 =     1.00000
y-multiplier                 =     1.00000

If Psd <= Pst then actual depth is used in place of equivalent depth in
computations.

        y, in             p, lbs/in
----------------    -----------------
          .0000            .000
          .0333         459.303
          .0667         700.044
          .1000         895.754
          .1333        1066.969
          .1667        1222.005
          .2000        1365.259
          .2333        1499.404
          .2667        1626.215
          .3000        1746.943
          .3333        1862.512
          .3667        1973.631
```

```
          .4000         2080.853
          .9000         3662.301
        24.9000         3662.301
        48.9000         3662.301
        72.9000         3662.301



p-y Curve in Sand Computed Using Reese Criteria

Soil Layer Number            =            1
Depth below pile head        =      200.000 in
Depth below ground surface   =      272.000 in
Equivalent Depth (see note) =      272.000 in
Pile Diameter                =       24.000 in
Angle of Friction            =       38.000 deg.
Avg. Eff. Unit Weight        =       .03180 lbs/in**3
k                            =      125.000 pci
A (static)                   =        .8800
B (static)                   =        .5000
Pst                          =     9928.992 lbs/in
Psd                          =    16518.199 lbs/in
Ps                           =     9928.992 lbs/in
pu                           =     8737.513 lbs/in
Cbar                         =    8666.0803
n                            =       1.6447
m                            =    7546.0342
yk                           =        .0306  in
ym                           =        .4000  in
yu                           =        .9000  in
p-multiplier                 =      1.00000
y-multiplier                 =      1.00000

If Psd <= Pst then actual depth is used in place of equivalent depth in
computations.

        y, in            p, lbs/in
----------------    -----------------
          .0000             .000
          .0333         1095.804
          .0667         1670.164
          .1000         2137.089
          .1333         2545.572
          .1667         2915.458
          .2000         3257.234
          .2333         3577.277
          .2667         3879.821
          .3000         4167.853
          .3333         4443.580
          .3667         4708.686
          .4000         4964.496
          .9000         8737.513
        24.9000         8737.513
        48.9000         8737.513
        72.9000         8737.513
```

```
p-y Curve Computed Using Criteria for Stiff Clay without Free Water

Soil Layer Number          =         2
Depth below pile head      =     300.000 in
Depth below ground surface =     372.000 in
Equivalent Depth           =     949.861 in
Diameter                   =      24.000 in
Undrained cohesion, c      =     8.33000 lbs/in**2
Avg. Undrained cohesion, c =     8.33000 lbs/in**2
Average Eff. Unit Weight   =      .03240 lbs/in**3
Epsilon-50                 =      .00500
Pct                        =    5294.543 lbs/in
Pcd                        =    1799.280 lbs/in
y50                        =        .300 in
p-multiplier               =     1.00000
y-multiplier               =     1.00000

        y, in           p, lbs/in
----------------    -----------------
          .0000            .000
          .0000         101.181
          .0002         151.301
          .0005         179.928
          .0024         269.055
          .0048         319.962
          .0240         478.455
          .0480         568.982
          .1200         715.457
          .2400         850.827
          .3600         941.595
          .4800        1011.810
         1.2000        1272.283
         2.4000        1513.008
         4.8000        1799.280
         5.4000        1799.280
         6.0000        1799.280


p-y Curve Computed Using Criteria for Stiff Clay without Free Water

Soil Layer Number          =         2
Depth below pile head      =     400.000 in
Depth below ground surface =     472.000 in
Equivalent Depth           =    1049.861 in
Diameter                   =      24.000 in
Undrained cohesion, c      =     8.33000 lbs/in**2
Avg. Undrained cohesion, c =     8.33000 lbs/in**2
Average Eff. Unit Weight   =      .03424 lbs/in**3
Epsilon-50                 =      .00500
Pct                        =    5835.246 lbs/in
Pcd                        =    1799.280 lbs/in
y50                        =        .300 in
p-multiplier               =     1.00000
y-multiplier               =     1.00000

        y, in           p, lbs/in
----------------    -----------------
          .0000            .000
          .0000         101.181
          .0002         151.301
          .0005         179.928
          .0024         269.055
          .0048         319.962
```

```
          .0240          478.455
          .0480          568.982
          .1200          715.457
          .2400          850.827
          .3600          941.595
          .4800         1011.810
         1.2000         1272.283
         2.4000         1513.008
         4.8000         1799.280
         5.4000         1799.280
         6.0000         1799.280


p-y Curve Computed Using Criteria for Stiff Clay without Free Water

Soil Layer Number           =          2
Depth below pile head       =      480.000 in
Depth below ground surface =      552.000 in
Equivalent Depth            =     1129.861 in
Diameter                    =       24.000 in
Undrained cohesion, c       =      8.33000 lbs/in**2
Avg. Undrained cohesion, c =      8.33000 lbs/in**2
Average Eff. Unit Weight    =       .03524 lbs/in**3
Epsilon-50                  =       .00500
Pct                         =     6261.140 lbs/in
Pcd                         =     1799.280 lbs/in
y50                         =         .300 in
p-multiplier                =      1.00000
y-multiplier                =      1.00000

        y, in            p, lbs/in
----------------    -----------------
          .0000             .000
          .0000          101.181
          .0002          151.301
          .0005          179.928
          .0024          269.055
          .0048          319.962
          .0240          478.455
          .0480          568.982
          .1200          715.457
          .2400          850.827
          .3600          941.595
          .4800         1011.810
         1.2000         1272.283
         2.4000         1513.008
         4.8000         1799.280
         5.4000         1799.280
         6.0000         1799.280




------------------------------------------------------------------------------
             Computed Values of Load Distribution and Deflection
                 for Lateral Loading for Load Case Number  1
------------------------------------------------------------------------------

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head    =       41900.000 lbs
Specified bending moment at pile head =            .000 in-lbs
Specified axial load at pile head     =       88200.000 lbs
```

```
                                 Napa24AMax

(Zero moment for this load indicates free-head conditions)

  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   .279840  -3.516E-07  41900.0000    -.003354    194.9602   -527.8719
   6.000   .259716 243673.2899  38682.4611    -.003341    374.5058   -544.6410
  12.000   .239745 467725.9286  35402.2519    -.003304    539.5943   -548.7620
  18.000   .220070 671997.0811  32139.4241    -.003244    690.1073   -538.8473
  24.000   .200819 856832.2984  28981.0506    -.003163    826.2993   -513.9439
  30.000   .182109   1.023E+06  25848.8572    -.003064    948.8233   -530.1206
  36.000   .164046   1.170E+06  22638.3326    -.002949   1057.2434   -540.0543
  42.000   .146721   1.298E+06  19388.1710    -.002819   1151.2901   -543.3329
  48.000   .130217   1.406E+06  16139.1125    -.002677   1230.8711   -539.6866
  54.000   .114600   1.494E+06  12881.8161    -.002524   1296.0788   -546.0789
  60.000   .099927   1.563E+06   9601.5878    -.002363   1346.7400   -547.3305
  66.000   .086241   1.612E+06   6329.8745    -.002196   1382.8186   -543.2406
  72.000   .073574   1.641E+06   3099.1668    -.002025   1404.4211   -533.6619
  78.000   .061943   1.651E+06    -57.3086    -.001852   1411.8004   -518.4965
  84.000   .051355   1.643E+06  -3105.8622    -.001678   1405.3583   -497.6880
  90.000   .041804   1.616E+06  -6012.5496    -.001507   1385.6472   -471.2078
  96.000   .033275   1.572E+06  -8743.2634    -.001339   1353.3707   -439.0301
 102.000   .025738   1.512E+06 -11263.6210    -.001177   1309.3839   -401.0891
 108.000   .019156   1.438E+06 -13538.4797    -.001021   1254.6958   -357.1972
 114.000   .013483   1.351E+06 -15530.7078  -8.744E-04   1190.4738   -306.8789
 120.000   .008663   1.253E+06 -17075.0644  -7.374E-04   1118.0560   -207.9067
 126.000   .004634   1.147E+06 -18042.8576  -6.111E-04   1040.0720   -114.6911
 132.000   .001330   1.037E+06 -18488.6430  -4.962E-04    958.9985    -33.9040
 138.000  -.001320 925618.9129 -18486.4088  -3.929E-04    876.9832     34.6488
 144.000  -.003385 815506.2647 -18108.2869  -3.012E-04    795.8490     91.3919
 150.000  -.004935 708638.2974 -17423.2916  -2.210E-04    717.1056    136.9398
 156.000  -.006037 606660.6904 -16496.3000  -1.518E-04    641.9655    172.0574
 162.000  -.006756 510843.3549 -15387.2640  -9.298E-05    571.3645    197.6213
 168.000  -.007153 422111.9306 -14150.6448  -4.388E-05    505.9846    214.5851
 174.000  -.007283 341082.0555 -12835.0517  -3.708E-06    446.2794    223.9459
 180.000  -.007197 268095.2348 -11483.0655   2.835E-05    392.5006    226.7161
 186.000  -.006943 203255.2611 -10131.2244   5.316E-05    344.7246    223.8976
 192.000  -.006559 146464.2776  -8810.1498   7.157E-05    302.8794    216.4606
 198.000  -.006084  97457.7182  -7544.7868   8.440E-05    266.7699    205.3270
 204.000  -.005547  55837.5032  -6354.7363   9.247E-05    236.1029    191.3564
 210.000  -.004974  21103.0098  -5254.6561   9.652E-05    210.5095    175.3370
 216.000  -.004388  -7320.5287  -4254.7084   9.725E-05    200.3542    157.9790
 222.000  -.003807 -30056.4169  -3361.0349   9.528E-05    217.1067    139.9122
 228.000  -.003245 -47753.7919  -2576.2418   9.118E-05    230.1466    121.6855
 234.000  -.002713 -61067.8285  -1899.8778   8.546E-05    239.9568    103.7692
 240.000  -.002219 -70642.7735  -1328.8935   7.853E-05    247.0119     86.5589
 246.000  -.001771 -77097.6615   -858.0706   7.075E-05    251.7680     70.3820
 252.000  -.001370 -81014.5025   -480.4131   6.243E-05    254.6541     55.5039
 258.000  -.001021 -82928.6920   -187.4933   5.380E-05    256.0645     42.1361
 264.000 -7.25E-04 -83321.3635     30.2486   4.505E-05    256.3538     30.4446
 270.000 -4.81E-04 -82613.3891    183.2557   3.632E-05    255.8322     20.5578
 276.000 -2.89E-04 -81160.7325    720.5080   2.770E-05    254.7618    158.5263
 282.000 -1.49E-04 -73996.6074   1598.7416   1.953E-05    249.4831    134.2182
 288.000 -5.47E-05 -61996.5049   2315.0737   1.237E-05    240.6410    104.5592
 294.000 -3.91E-08 -46228.8189   2669.7864   6.678E-06    229.0230     13.6784
 300.000  2.54E-05 -29966.1356   2451.5403   2.667E-06    217.0401    -86.4271
 306.000  3.20E-05 -16813.1579   1917.6010   2.053E-07    207.3486    -91.5527
 312.000  2.79E-05  -6955.1405   1377.2710  -1.046E-06    200.0850    -88.5573
 318.000  1.94E-05   -284.7989    868.5059  -1.427E-06    195.1701    -81.0311
 324.000  1.08E-05   3468.4407    414.8526  -1.259E-06    197.5159    -70.1867
 330.000  4.31E-06   4694.7652     35.0387  -8.295E-07    198.4195    -56.4179
```

```
 336.000  8.17E-07   3889.7829   -252.8136  -3.777E-07    197.8263    -39.5328
 342.000 -2.21E-07   1661.4018   -302.1797  -8.552E-08    196.1844     23.0775
 348.000 -2.09E-07    263.7166   -157.7869   1.580E-08    195.1545     25.0535
 354.000 -3.11E-08   -232.0580    -26.1812   1.746E-08    195.1312     18.8151
 360.000  3.72E-10    -50.4765     19.3874   2.595E-09    194.9974     -3.6256
 366.000  1.68E-13       .5878      4.2064  -3.096E-11    194.9606     -1.4348
 372.000 -4.24E-16   2.676E-04    -.048979  -1.401E-14    194.9602     .016312
 378.000 -1.94E-19  -6.713E-07  -2.230E-05   3.536E-17    194.9602   7.470E-06
 384.000  4.85E-22  -3.092E-10   5.594E-08   1.619E-20    194.9602  -1.863E-08
 390.000  2.25E-25   7.666E-13   2.577E-11  -4.038E-23    194.9602  -8.632E-12
 396.000     0.000   3.573E-16  -6.388E-14      0.0000    194.9602   2.127E-14
 402.000     0.000  -8.755E-19  -2.978E-17      0.0000    194.9602   9.974E-18
 408.000     0.000  -4.128E-22   7.296E-20      0.0000    194.9602  -2.430E-20
 414.000     0.000   9.998E-25   3.440E-23      0.0000    194.9602  -1.152E-23
 420.000     0.000      0.0000      0.0000      0.0000    194.9602      0.0000
 426.000     0.000      0.0000      0.0000      0.0000    194.9602      0.0000
 432.000     0.000      0.0000      0.0000      0.0000    194.9602      0.0000
 438.000     0.000      0.0000      0.0000      0.0000    194.9602      0.0000
 444.000     0.000      0.0000      0.0000      0.0000    194.9602      0.0000
 450.000     0.000      0.0000      0.0000      0.0000    194.9602      0.0000
 456.000     0.000      0.0000      0.0000      0.0000    194.9602      0.0000
 462.000     0.000      0.0000      0.0000      0.0000    194.9602      0.0000
 468.000     0.000      0.0000      0.0000      0.0000    194.9602      0.0000
 474.000     0.000      0.0000      0.0000      0.0000    194.9602      0.0000
 480.000     0.000      0.0000      0.0000      0.0000    194.9602      0.0000
 486.000     0.000      0.0000      0.0000      0.0000    194.9602      0.0000
 492.000     0.000      0.0000      0.0000      0.0000    194.9602      0.0000
 498.000     0.000      0.0000      0.0000      0.0000    194.9602      0.0000
 504.000     0.000      0.0000      0.0000      0.0000    194.9602      0.0000
 510.000     0.000      0.0000      0.0000      0.0000    194.9602      0.0000
 516.000     0.000      0.0000      0.0000      0.0000    194.9602      0.0000
 522.000     0.000      0.0000      0.0000      0.0000    194.9602      0.0000
 528.000     0.000      0.0000      0.0000      0.0000    194.9602      0.0000
 534.000     0.000      0.0000      0.0000      0.0000    194.9602      0.0000
 540.000     0.000      0.0000      0.0000      0.0000    194.9602      0.0000
 546.000     0.000      0.0000      0.0000      0.0000    194.9602      0.0000
 552.000     0.000      0.0000      0.0000      0.0000    194.9602      0.0000
 558.000     0.000      0.0000      0.0000      0.0000    194.9602      0.0000
 564.000     0.000      0.0000      0.0000      0.0000    194.9602      0.0000
 570.000     0.000      0.0000      0.0000      0.0000    194.9602      0.0000
 576.000     0.000      0.0000      0.0000      0.0000    194.9602      0.0000
 582.000     0.000      0.0000      0.0000      0.0000    194.9602      0.0000
 588.000     0.000      0.0000      0.0000      0.0000    194.9602      0.0000
 594.000     0.000      0.0000      0.0000      0.0000    194.9602      0.0000
 600.000     0.000      0.0000      0.0000      0.0000    194.9602      0.0000

Output Verification:

Computed forces and moments are within specified convergence limits.


Output Summary for Load Case No.  1:

Pile-head deflection             =      .27984028 in
Computed slope at pile head      =     -.00335409
Maximum bending moment           =    1651454.896 lbs-in
Maximum shear force              =      41900.000 lbs
Depth of maximum bending moment  =         78.000 in
Depth of maximum shear force     =          0.000 in
Number of iterations             =             13
Number of zero deflection points =             22
```

```
-------------------------------------------------------------------------------
                          Summary of Pile-head Response
-------------------------------------------------------------------------------

Definition of symbols for pile-head boundary conditions:

y = pile-head displacment, in
M = pile-head moment, lbs-in
V = pile-head shear force, lbs
S = pile-head slope, radians
R = rotational stiffness of pile-head, in-lbs/rad

 BC    Boundary     Boundary       Axial    Pile Head    Maximum     Maximum
Type  Condition    Condition       Load    Deflection    Moment      Shear
          1            2            lbs        in        in-lbs       lbs
---- ------------ ------------ ----------- ----------- ----------- -----------
  1  V= 41900.000 M=     0.000  88200.0000       .2798   1.651E+06  41900.0000


-------------------------------------------------------------------------------
                       Pile-head Deflection vs. Pile Length
-------------------------------------------------------------------------------

Boundary Condition Type 1, Shear and Moment

Shear      =          41900. lbs
Moment     =              0. in-lbs
Axial Load =          88200. lbs


   Pile       Pile Head       Maximum      Maximum
  Length      Deflection      Moment        Shear
    in           in           in-lbs         lbs
-----------  ------------  ------------  ------------
   600.000     .27984028   1651454.896     41900.000
   570.000     .27959828   1649720.067     41900.000
   540.000     .27955372   1648395.136     41900.000
   510.000     .27956165   1649395.802     41900.000
   480.000     .27972084   1650386.050     41900.000
   450.000     .27953744   1649326.265     41900.000
   420.000     .27953160   1649358.869     41900.000
   390.000     .27956456   1650196.570     41900.000
   360.000     .27951241   1649725.995     41900.000
   330.000     .27951766   1649722.315     41900.000

The analysis ended normally.
```

```
==============================================================================

               LPILE Plus for Windows, Version 4.0 (4.0.10)

               Analysis of Individual Piles and Drilled Shafts
              Subjected to Lateral Loading Using the p-y Method

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                             All Rights Reserved

==============================================================================


This program is licensed to:

david an
mge

Path to file locations:      C:\DA Works\Lpile\
Name of input data file:     Napa36B-JL.lpd
Name of output file:         Napa36B-JL.lpo
Name of plot output file:    Napa36B-JL.lpp
Name of runtime file:        Napa36B-JL.lpr


------------------------------------------------------------------------------
                          Time and Date of Analysis
------------------------------------------------------------------------------

               Date:  March 27, 2005     Time:   1:51:47


------------------------------------------------------------------------------
                                Problem Title
------------------------------------------------------------------------------

Napa Project--Wall #1 Type B-36 CIDH Pile, Left of Joint (Station 4+67)-AB


------------------------------------------------------------------------------
                               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 includes automatic computation of pile-top deflection vs.
  pile embedment length
- No computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- Additional p-y curves computed at 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                              =     480.00 in
Depth of ground surface below top of pile =     -36.00 in
Slope angle of ground surface            =        .00 deg.

Structural properties of pile defined using  2 points

Point    Depth         Pile      Moment of       Pile      Modulus of
           X         Diameter     Inertia        Area      Elasticity
           in           in         in**4        Sq.in      lbs/Sq.in
-----  ---------   -----------  ----------   ----------   -----------
  1       0.0000        36.000  82448.0000    1018.0000   3500000.000
  2     480.0000        36.000  82448.0000    1018.0000   3500000.000


------------------------------------------------------------------------------
                      Soil and Rock Layering Information
------------------------------------------------------------------------------

The soil profile is modelled using  4 layers

Layer  1 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer    =      -36.000 in
Distance from top of pile to bottom of layer =      276.000 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  2 is soft clay, p-y criteria by Matlock, 1970
Distance from top of pile to top of layer    =      276.000 in
Distance from top of pile to bottom of layer =      420.000 in

Layer  3 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer    =      420.000 in
Distance from top of pile to bottom of layer =      564.000 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  4 is stiff clay without free water
Distance from top of pile to top of layer    =      564.000 in
Distance from top of pile to bottom of layer =      636.000 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


(Depth of lowest layer extends  156.00 in below pile tip)


------------------------------------------------------------------------------
                   Effective Unit Weight of Soil vs. Depth
------------------------------------------------------------------------------
```

```
Distribution of effective unit weight of soil with depth
is defined using  8 points

Point         Depth X    Eff. Unit Weight
 No.            in          lbs/in**3
-----      ----------   ----------------
  1          -36.00         .04110
  2          276.00         .04110
  3          276.00         .03180
  4          420.00         .03180
  5          420.00         .04110
  6          564.00         .04110
  7          564.00         .03180
  8          636.00         .03180


-------------------------------------------------------------------------------
                           Shear Strength of Soils
-------------------------------------------------------------------------------

Distribution of shear strength parameters with depth
defined using  8 points

Point    Depth X     Cohesion c     Angle of Friction     E50 or      RQD
 No.       in         lbs/in**2           Deg.             k_rm        %
-----    --------    ----------     ------------------    ------    ------
  1      -36.000        .00000           39.00            ------    ------
  2      276.000        .00000           39.00            ------    ------
  3      276.000       3.47000             .00            .02000        .0
  4      420.000       3.47000             .00            .02000        .0
  5      420.000        .00000           39.00            ------    ------
  6      564.000        .00000           39.00            ------    ------
  7      564.000       8.33000             .00            .00500        .0
  8      636.000       8.33000             .00            .00500        .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
```

```
                                    Napa36B-JL
Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head    =       71800.000 lbs
Bending moment at pile head =     4660800.000 in-lbs
Axial load at pile head     =       41600.000 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.



------------------------------------------------------------------------------
                   Output of p-y Curves at Specified Depths
------------------------------------------------------------------------------



p-y curves are generated and printed for verification at  6 depths.

Depth      Depth Below Pile Head      Depth Below Ground Surface
 No.                 in                          in
-----      ---------------------      --------------------------
  1                 .000                       36.000
  2              100.000                      136.000
  3              200.000                      236.000
  4              300.000                      336.000
  5              400.000                      436.000
  6              480.000                      516.000

Depth of ground surface below top of pile =     -36.00 in


p-y Curve in Sand Computed Using Reese Criteria

Soil Layer Number            =           1
Depth below pile head        =        .000 in
Depth below ground surface   =      36.000 in
Equivalent Depth (see note)  =      36.000 in
Pile Diameter                =      36.000 in
Angle of Friction            =      39.000 deg.
Avg. Eff. Unit Weight        =      .04110 lbs/in**3
k                            =      60.000 pci
A (static)                   =      2.1100
B (static)                   =      1.5400
Pst                          =     447.300 lbs/in
Psd                          =    4844.679 lbs/in
Ps                           =     447.300 lbs/in
pu                           =     943.802 lbs/in
Cbar                         =    801.3264
n                            =      3.3772
m                            =    339.9477
yk                           =       .2445  in
ym                           =       .6000  in
yu                           =      1.3500  in
p-multiplier                 =     1.00000
y-multiplier                 =     1.00000

If Psd <= Pst then actual depth is used in place of equivalent depth in
computations.

        y, in            p, lbs/in
----------------    -----------------
```

```
          .0000             .000
          .0500          108.000
          .1000          216.000
          .1500          324.000
          .2000          432.000
          .2500          531.541
          .3000          561.026
          .3500          587.227
          .4000          610.911
          .4500          632.593
          .5000          652.639
          .5500          671.320
          .6000          688.841
         1.3500          943.802
        37.3500          943.802
        73.3500          943.802
       109.3500          943.802



p-y Curve in Sand Computed Using Reese Criteria

Soil Layer Number            =            1
Depth below pile head        =      100.000 in
Depth below ground surface   =      136.000 in
Equivalent Depth (see note)  =      136.000 in
Pile Diameter                =       36.000 in
Angle of Friction            =       39.000 deg.
Avg. Eff. Unit Weight        =       .04110 lbs/in**3
k                            =       60.000 pci
A (static)                   =        .9356
B (static)                   =        .5700
Pst                          =     4053.943 lbs/in
Psd                          =    18302.122 lbs/in
Ps                           =     4053.943 lbs/in
pu                           =     3792.689 lbs/in
Cbar                         =    3003.1313
n                            =       1.9491
m                            =    1975.9219
yk                           =        .1284  in
ym                           =        .6000  in
yu                           =       1.3500  in
p-multiplier                 =      1.00000
y-multiplier                 =      1.00000

If Psd <= Pst then actual depth is used in place of equivalent depth in
computations.

        y, in             p, lbs/in
----------------    -----------------
          .0000             .000
          .0500          408.000
          .1000          816.000
          .1500         1134.643
          .2000         1315.105
          .2500         1474.624
          .3000         1619.220
          .3500         1752.482
          .4000         1876.753
          .4500         1993.661
          .5000         2104.397
          .5500         2209.860
```

```
          .6000         2310.748
         1.3500         3792.689
        37.3500         3792.689
        73.3500         3792.689
       109.3500         3792.689



p-y Curve in Sand Computed Using Reese Criteria

Soil Layer Number             =           1
Depth below pile head         =     200.000 in
Depth below ground surface    =     236.000 in
Equivalent Depth (see note) =       236.000 in
Pile Diameter                 =      36.000 in
Angle of Friction             =      39.000 deg.
Avg. Eff. Unit Weight         =      .04110 lbs/in**3
k                             =      60.000 pci
A (static)                    =       .8800
B (static)                    =       .5000
Pst                           =   11137.270 lbs/in
Psd                           =   31759.565 lbs/in
Ps                            =   11137.270 lbs/in
pu                            =    9800.798 lbs/in
Cbar                          =   7596.8368
n                             =      1.6447
m                             =   5642.8835
yk                            =       .2042  in
ym                            =       .6000  in
yu                            =      1.3500  in
p-multiplier                  =     1.00000
y-multiplier                  =     1.00000

If Psd <= Pst then actual depth is used in place of equivalent depth in
computations.

        y, in            p, lbs/in
----------------    -----------------
          .0000             .000
          .0500          708.000
          .1000         1416.000
          .1500         2124.000
          .2000         2832.000
          .2500         3270.245
          .3000         3653.613
          .3500         4012.603
          .4000         4351.964
          .4500         4675.047
          .5000         4984.327
          .5500         5281.695
          .6000         5568.635
         1.3500         9800.798
        37.3500         9800.798
        73.3500         9800.798
       109.3500         9800.798
```

```
p-y Curve Computed Using the Soft Clay Criteria for Static Loading Conditions

Soil Layer Number           =          2
Depth below pile head       =      300.000 in
Depth below ground surface  =      336.000 in
Equivalent Depth            =     1462.217 in
Pile Diameter               =       36.000 in
Cohesion, c                 =        3.470 lbs/in**2
Avg Eff Unit Weight         =       .04044 lbs/in**3
E50 parameter               =       .02000
Default J parameter         =         .500
Y50                         =      1.80000 in
p-multiplier                =      1.00000
y-multiplier                =      1.00000

        y, in           p, lbs/in
----------------    -----------------
          .0000            .000
          .0144         112.428
          .4500         354.126
          .9000         446.171
         1.3500         510.738
         1.8000         562.140
         2.2500         605.547
         2.7000         643.490
         3.1500         677.419
         3.6000         708.252
         4.0500         736.612
         4.5000         762.941
         4.9500         787.569
         5.4000         810.746
        14.4000        1124.280
        27.0000        1124.280
        36.0000        1124.280


p-y Curve Computed Using the Soft Clay Criteria for Static Loading Conditions

Soil Layer Number           =          2
Depth below pile head       =      400.000 in
Depth below ground surface  =      436.000 in
Equivalent Depth            =     1562.217 in
Pile Diameter               =       36.000 in
Cohesion, c                 =        3.470 lbs/in**2
Avg Eff Unit Weight         =       .03846 lbs/in**3
E50 parameter               =       .02000
Default J parameter         =         .500
Y50                         =      1.80000 in
p-multiplier                =      1.00000
y-multiplier                =      1.00000

        y, in           p, lbs/in
----------------    -----------------
          .0000            .000
          .0144         112.428
          .4500         354.126
          .9000         446.171
         1.3500         510.738
         1.8000         562.140
         2.2500         605.547
         2.7000         643.490
         3.1500         677.419
         3.6000         708.252
```

```
         4.0500          736.612
         4.5000          762.941
         4.9500          787.569
         5.4000          810.746
        14.4000         1124.280
        27.0000         1124.280
        36.0000         1124.280



p-y Curve in Sand Computed Using Reese Criteria

Soil Layer Number            =            3
Depth below pile head        =      480.000 in
Depth below ground surface   =      516.000 in
Equivalent Depth (see note) =      367.479 in
Pile Diameter                =       36.000 in
Angle of Friction            =       39.000 deg.
Avg. Eff. Unit Weight        =       .03850 lbs/in**3
k                            =       60.000 pci
A (static)                   =        .8800
B (static)                   =        .5000
Pst                          =    24115.408 lbs/in
Psd                          =    65055.439 lbs/in
Ps                           =    24115.408 lbs/in
pu                           =    21221.559 lbs/in
Cbar                         =   16449.3465
n                            =       1.6447
m                            =   12218.4731
yk                           =        .4736  in
ym                           =        .6000  in
yu                           =       1.3500  in
p-multiplier                 =      1.00000
y-multiplier                 =      1.00000

If Psd <= Pst then actual depth is used in place of equivalent depth in
computations.

        y, in             p, lbs/in
----------------    -----------------
          .0000             .000
          .0500         1102.436
          .1000         2204.871
          .1500         3307.307
          .2000         4409.742
          .2500         5512.178
          .3000         6614.613
          .3500         7717.049
          .4000         8819.484
          .4500         9921.920
          .5000        10792.509
          .5500        11436.395
          .6000        12057.704
         1.3500        21221.559
        37.3500        21221.559
        73.3500        21221.559
       109.3500        21221.559
```

```
------------------------------------------------------------------------------
             Computed Values of Load Distribution and Deflection
                 for Lateral Loading for Load Case Number  1
------------------------------------------------------------------------------

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head    =       71800.000 lbs
Specified bending moment at pile head =     4660800.000 in-lbs
Specified axial load at pile head     =       41600.000 lbs

Non-zero moment 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.

  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   .421009   4.661E+06  71800.0000    -.004414   1058.4076   -620.2471
   4.800   .400006   4.999E+06  68623.0053    -.004334   1132.2800   -703.5007
   9.600   .379403   5.321E+06  65059.8637    -.004248   1202.6102   -781.1417
  14.400   .359224   5.625E+06  61141.3788    -.004157   1269.0072   -851.5604
  19.200   .339494   5.910E+06  56900.0664    -.004061   1331.1168   -915.6532
  24.000   .320236   6.173E+06  52354.5330    -.003961   1388.6162   -978.3191
  28.800   .301471   6.414E+06  47529.8992    -.003856   1441.1902  -1031.9450
  33.600   .283218   6.631E+06  42472.3049    -.003748   1488.5688  -1075.3859
  38.400   .265495   6.823E+06  37219.9423    -.003636   1530.5332  -1113.0985
  43.200   .248316   6.990E+06  31798.4588    -.003521   1566.8938  -1145.8530
  48.000   .231696   7.130E+06  26245.8208    -.003403   1597.4855  -1167.7462
  52.800   .215644   7.243E+06  20685.7412    -.003284   1622.1982  -1148.9536
  57.600   .200172   7.330E+06  15230.2610    -.003163   1641.1264  -1124.1632
  62.400   .185284   7.391E+06   9906.8717    -.003040   1654.3946  -1093.9157
  67.200   .170986   7.426E+06   4740.4822    -.002917   1662.1549  -1058.7467
  72.000   .157282   7.437E+06   -246.5525    -.002793   1664.5843  -1019.1844
  76.800   .144171   7.425E+06  -5034.3885    -.002670   1661.8817   -975.7473
  81.600   .131653   7.390E+06  -9605.6410    -.002546   1654.2657   -928.9412
  86.400   .119725   7.334E+06 -13945.3195    -.002424   1641.9716   -879.2581
  91.200   .108382   7.257E+06 -18040.7551    -.002303   1625.2495   -827.1734
  96.000   .097619   7.162E+06 -21881.5182    -.002183   1604.3613   -773.1445
 100.800   .087428   7.048E+06 -25459.3277    -.002065   1579.5790   -717.6095
 105.600   .077800   6.918E+06 -28767.9549    -.001948   1551.1820   -660.9852
 110.400   .068723   6.773E+06 -31803.1195    -.001835   1519.4551   -603.6667
 115.200   .060188   6.613E+06 -34562.3820    -.001723   1484.6869   -546.0260
 120.000   .052181   6.441E+06 -37045.0314    -.001615   1447.1672   -488.4112
 124.800   .044688   6.258E+06 -39251.9694    -.001509   1407.1862   -431.1463
 129.600   .037694   6.065E+06 -41185.5927    -.001407   1365.0319   -374.5301
 134.400   .031185   5.864E+06 -42849.6736    -.001307   1320.9893   -318.8369
 139.200   .025144   5.654E+06 -44249.2401    -.001212   1275.3387   -264.3158
 144.000   .019555   5.439E+06 -45390.4562    -.001119   1228.3545   -211.1909
 148.800   .014400   5.219E+06 -46280.5027    -.001031   1180.3040   -159.6618
 153.600   .009661   4.995E+06 -46927.4599  -9.456E-04   1131.4466   -109.9038
 158.400   .005321   4.769E+06 -47340.1932  -8.644E-04   1082.0327    -62.0684
 163.200   .001362   4.541E+06 -47528.2401  -7.870E-04   1032.3032    -16.2844
 168.000  -.002234   4.313E+06 -47501.7024  -7.134E-04    982.4885     27.3418
 172.800  -.005486   4.085E+06 -47271.1411  -6.435E-04    932.8082     68.7254
 177.600  -.008411   3.860E+06 -46847.4766  -5.774E-04    883.4706    107.8015
 182.400  -.011029   3.636E+06 -46241.8935  -5.151E-04    834.6725    144.5248
 187.200  -.013356   3.416E+06 -45465.7506  -4.564E-04    786.5987    178.8680
 192.000  -.015411   3.200E+06 -44530.4971  -4.014E-04    739.4221    210.8209
 196.800  -.017210   2.988E+06 -43447.5938  -3.500E-04    693.3037    240.3888
 201.600  -.018770   2.783E+06 -42228.4417  -3.020E-04    648.3923    267.5913
```

```
 206.400  -.020109   2.583E+06 -40884.3155  -2.573E-04    604.8248    292.4613
 211.200  -.021241   2.390E+06 -39426.3052  -2.160E-04    562.7266    315.0430
 216.000  -.022182   2.205E+06 -37865.2632  -1.777E-04    522.2114    335.3911
 220.800  -.022947   2.027E+06 -36211.7589  -1.425E-04    483.3816    353.5690
 225.600  -.023550   1.857E+06 -34476.0393  -1.102E-04    446.3288    369.6475
 230.400  -.024005   1.696E+06 -32667.9976  -8.069E-05    411.1340    383.7033
 235.200  -.024325   1.544E+06 -30797.1473  -5.375E-05    377.8681    395.8177
 240.000  -.024521   1.400E+06 -28872.6042  -2.926E-05    346.5920    406.0753
 244.800  -.024606   1.266E+06 -26903.0742  -7.085E-06    317.3575    414.5622
 249.600  -.024589   1.142E+06 -24896.8486   1.295E-05    290.2074    421.3651
 254.400  -.024482   1.027E+06 -22861.8053   3.099E-05    265.1759    426.5696
 259.200  -.024292 922615.7546 -20805.4165   4.721E-05    242.2894    430.2591
 264.000  -.024029 827695.3808 -18734.7629   6.177E-05    221.5664    432.5133
 268.800  -.023699 742737.3632 -16656.5540   7.483E-05    203.0184    433.4071
 273.600  -.023310 667762.5791 -14577.1538   8.656E-05    186.6500    433.0096
 278.400  -.022868 602762.1187 -13223.0761   9.713E-05    172.4591    131.1894
 283.200  -.022378 540782.2604 -12595.6301   1.066E-04    158.9277    130.2464
 288.000  -.021844 481801.4837 -11972.9477   1.151E-04    146.0511    129.2045
 292.800  -.021272 425795.9794 -11355.4926   1.227E-04    133.8240    128.0684
 297.600  -.020666 372739.7574 -10743.7069   1.293E-04    122.2408    126.8423
 302.400  -.020031 322604.7436 -10138.0138   1.351E-04    111.2953    125.5298
 307.200  -.019369 275360.8653  -9538.8200   1.401E-04    100.9811    124.1343
 312.000  -.018686 230976.1266  -8946.5171   1.443E-04     91.2910    122.6586
 316.800  -.017984 189416.6741  -8361.4845   1.478E-04     82.2178    121.1050
 321.600  -.017267 150646.8523  -7784.0908   1.506E-04     73.7536    119.4757
 326.400  -.016538 114629.2493  -7214.6964   1.528E-04     65.8902    117.7720
 331.200  -.015800  81324.7323  -6653.6554   1.545E-04     58.6192    115.9951
 336.000  -.015055  50692.4722  -6101.3179   1.556E-04     51.9316    114.1455
 340.800  -.014307  22689.9571  -5558.0325   1.562E-04     45.8181    112.2234
 345.600  -.013556  -2727.0072  -5024.1494   1.563E-04     41.4598    110.2279
 350.400  -.012806 -25604.3106  -4500.0230   1.561E-04     46.4544    108.1581
 355.200  -.012058 -45989.5673  -3986.0158   1.555E-04     50.9049    106.0116
 360.000  -.011313 -63932.1636  -3482.5024   1.546E-04     54.8221    103.7856
 364.800  -.010574 -79483.3266  -2989.8744   1.534E-04     58.2172    101.4760
 369.600  -.009840 -92696.2184  -2508.5465   1.520E-04     61.1018     99.0773
 374.400  -.009115-103626.0612  -2038.9632   1.503E-04     63.4880     96.5824
 379.200  -.008397-112330.3010  -1581.6081   1.485E-04     65.3883     93.9822
 384.000  -.007689-118868.8182  -1137.0154   1.466E-04     66.8158     91.2648
 388.800  -.006990-123304.2000   -705.7841   1.446E-04     67.7841     88.4149
 393.600  -.006301-125702.0923   -288.5978   1.425E-04     68.3076     85.4127
 398.400  -.005621-126131.6584    113.7491   1.404E-04     68.4014     82.2318
 403.200  -.004952-124666.1844    500.3140   1.383E-04     68.0815     78.8369
 408.000  -.004293-121383.8944    869.9524   1.363E-04     67.3649     75.1791
 412.800  -.003644-116369.0746   1221.2349   1.343E-04     66.2701     71.1886
 417.600  -.003004-109713.6822   1552.3134   1.324E-04     64.8171     66.7607
 422.400  -.002373-101519.7582   1838.0438   1.307E-04     63.0282     52.2936
 427.200  -.001749 -92120.6521   2057.2933   1.291E-04     60.9762     39.0603
 432.000  -.001133 -81821.2899   2212.5607   1.276E-04     58.7276     25.6344
 436.800 -5.24E-04 -70931.0387   2302.8916   1.264E-04     56.3501     12.0034
 441.600  7.96E-05 -59763.9933   2327.2664   1.253E-04     53.9121     -1.8472
 446.400  6.79E-04 -48639.3098   2284.5924   1.244E-04     51.4833    -15.9336
 451.200   .001274 -37881.5742   2173.6977   1.236E-04     49.1347    -30.2726
 456.000   .001866 -27821.1928   1993.3278   1.231E-04     46.9383    -44.8815
 460.800   .002455 -18794.7895   1742.1463   1.227E-04     44.9677    -59.7774
 465.600   .003044 -11145.5957   1418.7380   1.225E-04     43.2977    -74.9761
 470.400   .003631  -5223.8135   1021.6161   1.223E-04     42.0049    -90.4914
 475.200   .004218  -1386.9354    549.2339   1.223E-04     41.1672   -106.3345
 480.000   .004805      0.0000      0.0000   1.223E-04     40.8644   -122.5129

Output Verification:

Computed forces and moments are within specified convergence limits.
```

```
Output Summary for Load Case No.  1:

Pile-head deflection              =      .42100917 in
Computed slope at pile head       =     -.00441437
Maximum bending moment            =    7437358.642 lbs-in
Maximum shear force               =      71800.000 lbs
Depth of maximum bending moment   =         72.000 in
Depth of maximum shear force      =          0.000 in
Number of iterations              =             12
Number of zero deflection points =              2




------------------------------------------------------------------------------
                        Summary of Pile-head Response
------------------------------------------------------------------------------

Definition of symbols for pile-head boundary conditions:

y = pile-head displacment, in
M = pile-head moment, lbs-in
V = pile-head shear force, lbs
S = pile-head slope, radians
R = rotational stiffness of pile-head, in-lbs/rad

 BC    Boundary     Boundary       Axial    Pile Head    Maximum     Maximum
Type  Condition    Condition       Load    Deflection    Moment       Shear
          1            2            lbs        in         in-lbs       lbs
---- ------------ ------------ ----------- ----------- ----------- -----------
  1  V= 71800.000 M=  4.66E+06  41600.0000       .4210   7.437E+06  71800.0000


------------------------------------------------------------------------------
                     Pile-head Deflection vs. Pile Length
------------------------------------------------------------------------------

Boundary Condition Type 1, Shear and Moment

Shear      =          71800. lbs
Moment     =        4660800. in-lbs
Axial Load =          41600. lbs


   Pile       Pile Head       Maximum      Maximum
  Length      Deflection      Moment        Shear
    in           in           in-lbs         lbs
-----------  ------------  ------------  ------------
   480.000     .42100917   7437358.642     71800.000
   456.000     .42114490   7436906.651     71800.000
   432.000     .42121030   7435485.202     71800.000
   408.000     .42100963   7435977.744     71800.000
   384.000     .42166717   7434184.040     71800.000
   360.000     .42356876   7428832.972     71800.000
   336.000     .42613568   7420564.450     71800.000
   312.000     .43003301   7409714.676     71800.000
   288.000     .43408814   7399741.386     71800.000
   264.000     .44559254   7376036.238     71800.000

The analysis ended normally.
```

```
                                   Napa24C-49
==============================================================================

               LPILE Plus for Windows, Version 4.0 (4.0.8)

              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:

David An
MGE Engineering, Inc.

Path to file locations:      C:\100%submital-napa-final\Lpile\
Name of input data file:     Napa24C-49.lpd
Name of output file:         Napa24C-49.lpo
Name of plot output file:    Napa24C-49.lpp
Name of runtime file:        Napa24C-49.lpr


------------------------------------------------------------------------------
                          Time and Date of Analysis
------------------------------------------------------------------------------

               Date:  March 29, 2005     Time:  14:34: 3


------------------------------------------------------------------------------
                                Problem Title
------------------------------------------------------------------------------

Napa River Flood Control Project--24 CIDH Pile, Type C-Sta. 4+75 to 9+30


------------------------------------------------------------------------------
                               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 includes automatic computation of pile-top deflection vs.
  pile embedment length
- No computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- Additional p-y curves computed at specified depths

Solution Control Parameters:
- Number of pile increments            =          100
```

```
                                    Napa24C-49
- 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                                =     480.00 in
Depth of ground surface below top of pile =     -36.00 in
Slope angle of ground surface             =        .00 deg.

Structural properties of pile defined using  2 points

Point    Depth        Pile      Moment of       Pile      Modulus of
           X        Diameter     Inertia        Area      Elasticity
           in          in         in**4         Sq.in     lbs/Sq.in
-----  ---------   -----------  ----------   ----------   -----------
  1       0.0000       24.000   16286.0000     452.4000   3500000.000
  2     480.0000       24.000   16286.0000     452.4000   3500000.000


-------------------------------------------------------------------------------
                      Soil and Rock Layering Information
-------------------------------------------------------------------------------

The soil profile is modelled using  4 layers

Layer  1 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
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 sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer    =       96.000 in
Distance from top of pile to bottom of layer =      456.000 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    =      456.000 in
Distance from top of pile to bottom of layer =      600.000 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  4 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer    =      600.000 in
Distance from top of pile to bottom of layer =      756.000 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


(Depth of lowest layer extends  276.00 in below pile tip)

-------------------------------------------------------------------------------
                                   Page 2
```

```
                                 Napa24C-49
                  Effective Unit Weight of Soil vs. Depth
------------------------------------------------------------------------------

Distribution of effective unit weight of soil with depth
is defined using  8 points

Point        Depth X    Eff. Unit Weight
 No.           in          lbs/in**3
-----      ----------   ----------------
  1          -36.00         .03410
  2           96.00         .03410
  3           96.00         .04110
  4          456.00         .04110
  5          456.00         .03810
  6          600.00         .03810
  7          600.00         .04110
  8          756.00         .04110



------------------------------------------------------------------------------
                           Shear Strength of Soils
------------------------------------------------------------------------------

Distribution of shear strength parameters with depth
defined using  8 points

Point    Depth X     Cohesion c     Angle of Friction     E50 or      RQD
 No.       in        lbs/in**2            Deg.             k_rm        %
-----    --------    ----------     ------------------     ------    ------
  1      -36.000       9.72200                .00          .00500        .0
  2       96.000       9.72200                .00          .00500        .0
  3       96.000        .00000              38.00          ------    ------
  4      456.000        .00000              38.00          ------    ------
  5      456.000       8.33000                .00          .00500        .0
  6      600.000       8.33000                .00          .00500        .0
  7      600.000        .00000              38.00          ------    ------
  8      756.000        .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 =  1
```

```
Load Case Number  1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head    =       39000.000 lbs
Bending moment at pile head =     1897200.000 in-lbs
Axial load at pile head     =       26700.000 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.



------------------------------------------------------------------------------
                   Output of p-y Curves at Specified Depths
------------------------------------------------------------------------------



p-y curves are generated and printed for verification at  6 depths.

Depth      Depth Below Pile Head      Depth Below Ground Surface
 No.                in                           in
-----      ---------------------      -------------------------
  1                  .000                      36.000
  2               100.000                     136.000
  3               200.000                     236.000
  4               300.000                     336.000
  5               400.000                     436.000
  6               480.000                     516.000

Depth of ground surface below top of pile =     -36.00 in


p-y Curve Computed Using Criteria for Stiff Clay without Free Water

Soil Layer Number            =          1
Depth below pile head        =        .000 in
Depth below ground surface =       36.000 in
Equivalent Depth             =       36.000 in
Diameter                     =       24.000 in
Undrained cohesion, c        =      9.72200 lbs/in**2
Avg. Undrained cohesion, c =      9.72200 lbs/in**2
Average Eff. Unit Weight     =       .03410 lbs/in**3
Epsilon-50                   =       .00500
Pct                          =      904.442 lbs/in
Pcd                          =     2099.952 lbs/in
y50                          =        .300 in
p-multiplier                 =      1.00000
y-multiplier                 =      1.00000

        y, in            p, lbs/in
----------------    -----------------
          .0000             .000
          .0000           50.861
          .0002           76.054
          .0005           90.444
          .0024          135.246
          .0048          160.835
          .0240          240.505
          .0480          286.010
          .1200          359.638
```

```
          .2400          427.684
          .3600          473.311
          .4800          508.605
         1.2000          639.537
         2.4000          760.542
         4.8000          904.442
         5.4000          904.442
         6.0000          904.442




p-y Curve in Sand Computed Using Reese Criteria

Soil Layer Number            =           2
Depth below pile head        =     100.000 in
Depth below ground surface   =     136.000 in
Equivalent Depth (see note) =      139.503 in
Pile Diameter                =      24.000 in
Angle of Friction            =      38.000 deg.
Avg. Eff. Unit Weight        =      .03431 lbs/in**3
k                            =      60.000 pci
A (static)                   =       .8800
B (static)                   =       .5000
Pst                          =    3039.477 lbs/in
Psd                          =    8909.928 lbs/in
Ps                           =    3039.477 lbs/in
pu                           =    2674.740 lbs/in
Cbar                         =   2652.8724
n                            =      1.6447
m                            =   2310.0023
yk                           =       .0533  in
ym                           =       .4000  in
yu                           =       .9000  in
p-multiplier                 =     1.00000
y-multiplier                 =     1.00000

If Psd <= Pst then actual depth is used in place of equivalent depth in
computations.

        y, in           p, lbs/in
----------------    -----------------
          .0000             .000
          .0333          279.006
          .0667          511.273
          .1000          654.209
          .1333          779.254
          .1667          892.484
          .2000          997.109
          .2333         1095.081
          .2667         1187.696
          .3000         1275.869
          .3333         1360.275
          .3667         1441.429
          .4000         1519.738
          .9000         2674.740
        24.9000         2674.740
        48.9000         2674.740
        72.9000         2674.740
```

```
                                   Napa24C-49

p-y Curve in Sand Computed Using Reese Criteria

Soil Layer Number            =           2
Depth below pile head        =     200.000 in
Depth below ground surface   =     236.000 in
Equivalent Depth (see note) =     239.503 in
Pile Diameter                =      24.000 in
Angle of Friction            =      38.000 deg.
Avg. Eff. Unit Weight        =      .03718 lbs/in**3
k                            =      60.000 pci
A (static)                   =       .8800
B (static)                   =       .5000
Pst                          =    9103.040 lbs/in
Psd                          =   16758.822 lbs/in
Ps                           =    9103.040 lbs/in
pu                           =    8010.676 lbs/in
Cbar                         =   7945.1848
n                            =      1.6447
m                            =   6918.3107
yk                           =       .2205  in
ym                           =       .4000  in
yu                           =       .9000  in
p-multiplier                 =     1.00000
y-multiplier                 =     1.00000

If Psd <= Pst then actual depth is used in place of equivalent depth in
computations.

        y, in             p, lbs/in
----------------    -----------------
          .0000             .000
          .0333          479.006
          .0667          958.013
          .1000         1437.019
          .1333         1916.026
          .1667         2395.032
          .2000         2874.039
          .2333         3279.698
          .2667         3557.075
          .3000         3821.147
          .3333         4073.937
          .3667         4316.990
          .4000         4551.520
          .9000         8010.676
        24.9000         8010.676
        48.9000         8010.676
        72.9000         8010.676



p-y Curve in Sand Computed Using Reese Criteria

Soil Layer Number            =           2
Depth below pile head        =     300.000 in
Depth below ground surface   =     336.000 in
Equivalent Depth (see note) =     339.503 in
Pile Diameter                =      24.000 in
Angle of Friction            =      38.000 deg.
Avg. Eff. Unit Weight        =      .03835 lbs/in**3
k                            =      60.000 pci
A (static)                   =       .8800
```

```
B (static)                  =        .5000
Pst                         =    18347.372 lbs/in
Psd                         =    24607.717 lbs/in
Ps                          =    18347.372 lbs/in
pu                          =    16145.687 lbs/in
Cbar                        =   16013.6889
n                           =       1.6447
m                           =   13944.0025
yk                          =        .5413  in
ym                          =        .4000  in
yu                          =        .9000  in
p-multiplier                =      1.00000
y-multiplier                =      1.00000

If Psd <= Pst then actual depth is used in place of equivalent depth in
computations.

        y, in             p, lbs/in
---------------    ----------------
          .0000             .000
          .0333          679.006
          .0667         1358.013
          .1000         2037.019
          .1333         2716.026
          .1667         3395.032
          .2000         4074.039
          .2333         4753.045
          .2667         5432.051
          .3000         6111.058
          .3333         6790.064
          .3667         7469.071
          .4000         8148.077
          .9000        16145.687
        24.9000        16145.687
        48.9000        16145.687
        72.9000        16145.687



p-y Curve in Sand Computed Using Reese Criteria

Soil Layer Number           =            2
Depth below pile head       =      400.000 in
Depth below ground surface  =      436.000 in
Equivalent Depth (see note) =      439.503 in
Pile Diameter               =       24.000 in
Angle of Friction           =       38.000 deg.
Avg. Eff. Unit Weight       =       .03898 lbs/in**3
k                           =       60.000 pci
A (static)                  =        .8800
B (static)                  =        .5000
Pst                         =    30772.920 lbs/in
Psd                         =    32456.611 lbs/in
Ps                          =    30772.920 lbs/in
pu                          =    27080.169 lbs/in
Cbar                        =   26858.7769
n                           =       1.6447
m                           =   23387.4189
yk                          =       1.0479  in
ym                          =        .4000  in
yu                          =        .9000  in
p-multiplier                =      1.00000
```

```
y-multiplier                =      1.00000

If Psd <= Pst then actual depth is used in place of equivalent depth in
computations.

        y, in            p, lbs/in
----------------    -----------------
          .0000             .000
          .0333          879.006
          .0667         1758.013
          .1000         2637.019
          .1333         3516.026
          .1667         4395.032
          .2000         5274.039
          .2333         6153.045
          .2667         7032.051
          .3000         7911.058
          .3333         8790.064
          .3667         9669.071
          .4000        10548.077
          .9000        23733.173
        24.9000        27080.169
        48.9000        27080.169
        72.9000        27080.169




p-y Curve Computed Using Criteria for Stiff Clay without Free Water

Soil Layer Number          =          3
Depth below pile head      =      480.000 in
Depth below ground surface =      516.000 in
Equivalent Depth           =     3254.393 in
Diameter                   =       24.000 in
Undrained cohesion, c      =      8.33000 lbs/in**2
Avg. Undrained cohesion, c =      8.33000 lbs/in**2
Average Eff. Unit Weight   =       .03917 lbs/in**3
Epsilon-50                 =       .00500
Pct                        =    17213.677 lbs/in
Pcd                        =     1799.280 lbs/in
y50                        =         .300 in
p-multiplier               =      1.00000
y-multiplier               =      1.00000

        y, in           p, lbs/in
----------------    -----------------
          .0000             .000
          .0000          101.181
          .0002          151.301
          .0005          179.928
          .0024          269.055
          .0048          319.962
          .0240          478.455
          .0480          568.982
          .1200          715.457
          .2400          850.827
          .3600          941.595
          .4800         1011.810
         1.2000         1272.283
```

```
         2.4000         1513.008
         4.8000         1799.280
         5.4000         1799.280
         6.0000         1799.280




-------------------------------------------------------------------------------
             Computed Values of Load Distribution and Deflection
                 for Lateral Loading for Load Case Number  1
-------------------------------------------------------------------------------

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head    =       39000.000 lbs
Specified bending moment at pile head =     1897200.000 in-lbs
Specified axial load at pile head     =       26700.000 lbs

Non-zero moment 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.

  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   .626419   1.897E+06  39000.0000    -.007906   1456.9309   -543.6134
   4.800   .588855   2.079E+06  36371.9489    -.007738   1590.9900   -551.4078
   9.600   .552131   2.248E+06  33708.2220    -.007556   1715.6715   -558.4784
  14.400   .516316   2.405E+06  31012.3417    -.007360   1830.8542   -564.8051
  19.200   .481473   2.548E+06  28287.9282    -.007152   1936.4293   -570.3672
  24.000   .447660   2.678E+06  25538.7025    -.006932   2032.3013   -575.1435
  28.800   .414929   2.795E+06  22768.4905    -.006701   2118.3880   -579.1115
  33.600   .383328   2.898E+06  19981.2283    -.006461   2194.6212   -582.2478
  38.400   .352899   2.988E+06  17180.9671    -.006214   2260.9467   -584.5277
  43.200   .323678   3.065E+06  14371.8817    -.005959   2317.3253   -585.9246
  48.000   .295695   3.128E+06  11558.2782    -.005698   2363.7325   -586.4102
  52.800   .268977   3.177E+06   8744.6055    -.005433   2400.1596   -585.9534
  57.600   .243543   3.213E+06   5935.4687    -.005163   2426.6140   -584.5203
  62.400   .219407   3.236E+06   3135.6458    -.004892   2443.1197   -582.0726
  67.200   .196580   3.245E+06    350.1095    -.004619   2449.7181   -578.5675
  72.000   .175064   3.240E+06  -2415.9454    -.004346   2446.4686   -573.9554
  76.800   .154858   3.222E+06  -5157.0667    -.004074   2433.4496   -568.1785
  81.600   .135954   3.192E+06  -7867.4971    -.003804   2410.7592   -561.1676
  86.400   .118340   3.148E+06 -10541.1119    -.003537   2378.5168   -552.8386
  91.200   .101999   3.091E+06 -13171.3321    -.003274   2336.8640   -543.0865
  96.000   .086908   3.022E+06 -15832.7789    -.003017   2285.9670   -565.8496
 100.800   .073038   2.940E+06 -18503.2184    -.002766   2225.4397   -546.8335
 105.600   .060356   2.845E+06 -21067.7942    -.002522   2155.6057   -521.7398
 110.400   .048825   2.739E+06 -23373.8992    -.002287   2076.8917   -439.1374
 115.200   .038400   2.622E+06 -25283.2770    -.002061   1990.7010   -356.4367
 120.000   .029035   2.496E+06 -26805.6197    -.001846   1898.4383   -277.8728
 124.800   .020679   2.365E+06 -27961.7834    -.001641   1801.4384   -203.8621
 129.600   .013280   2.228E+06 -28774.4205    -.001448   1700.9589   -134.7367
 134.400   .006780   2.089E+06 -29267.5820    -.001266   1598.1743    -70.7472
 139.200   .001125   1.948E+06 -29466.3370    -.001096   1494.1720    -12.0674
 144.000  -.003742   1.806E+06 -29396.4149  -9.380E-04   1389.9494     41.2016
 148.800  -.007880   1.666E+06 -29083.8705  -7.918E-04   1286.4118     89.0253
 153.600  -.011344   1.527E+06 -28554.7752  -6.574E-04   1184.3724    131.4311
 158.400  -.014191   1.392E+06 -27834.9363  -5.345E-04   1084.5521    168.5018
 163.200  -.016475   1.260E+06 -26949.6436  -4.228E-04    987.5813    200.3702
 168.000  -.018250   1.133E+06 -25923.4457  -3.220E-04    894.0020    227.2123
 172.800  -.019567   1.011E+06 -24779.9552  -2.318E-04    804.2710    249.2421
```

```
 177.600  -.020475 895383.5371 -23541.6825  -1.515E-04    718.7633    266.7049
 182.400  -.021021 785470.4820 -22229.8976  -8.069E-05    637.7761    279.8721
 187.200  -.021249 681997.2031 -20864.5199  -1.891E-05    561.5340    289.0353
 192.000  -.021202 585175.9368 -19464.0327   3.445E-05    490.1933    294.5011
 196.800  -.020918 495133.6599 -18045.4233   7.993E-05    423.8475    296.5862
 201.600  -.020435 411919.3850 -16624.1458   1.181E-04    362.5328    295.6127
 206.400  -.019784 335511.5822 -15214.1054   1.496E-04    306.2333    291.9041
 211.200  -.018999 265825.6289 -13827.6612   1.749E-04    254.8866    285.7810
 216.000  -.018105 202721.2005 -12475.6477   1.946E-04    208.3895    277.5580
 220.800  -.017130 146009.5205 -11167.4108   2.093E-04    166.6026    267.5407
 225.600  -.016096  95460.4028  -9910.8580   2.195E-04    129.3566    256.0229
 230.400  -.015023  50809.0235  -8712.5200   2.256E-04     96.4561    243.2846
 235.200  -.013930  11762.3723  -7577.6216   2.283E-04     67.6854    229.5897
 240.000  -.012831 -21994.6581  -6510.1613   2.279E-04     75.2249    215.1854
 244.800  -.011742 -50793.5799  -5512.9960   2.248E-04     96.4448    200.3002
 249.600  -.010673 -74977.0377  -4587.9309   2.195E-04    114.2638    185.1437
 254.400  -.009635 -94893.9767  -3735.8117   2.123E-04    128.9392    169.9060
 259.200  -.008635-110895.2578  -2956.6191   2.037E-04    140.7294    154.7576
 264.000  -.007680-123329.7265  -2249.5621   1.938E-04    149.8915    139.8495
 268.800  -.006774-132540.7322  -1613.1715   1.830E-04    156.6784    125.3133
 273.600  -.005923-138863.0900  -1045.3909   1.716E-04    161.3369    111.2620
 278.400  -.005127-142620.4729   -543.6645   1.598E-04    164.1055     97.7907
 283.200  -.004389-144123.2194   -105.0215   1.477E-04    165.2128     84.9772
 288.000  -.003709-143666.5346    273.8442   1.356E-04    164.8763     72.8834
 292.800  -.003087-141529.0650    596.4985   1.236E-04    163.3013     61.5559
 297.600  -.002523-137971.8205    866.6982   1.118E-04    160.6802     51.0273
 302.400  -.002014-133237.4179   1088.3260   1.004E-04    157.1918     41.3176
 307.200  -.001559-127549.6195   1265.3327   8.940E-05    153.0008     32.4352
 312.000  -.001156-121113.1375   1401.6847   7.893E-05    148.2583     24.3781
 316.800 -8.02E-04-114113.6770   1501.3167   6.902E-05    143.1009     17.1352
 321.600 -4.93E-04-106718.1889   1568.0916   5.972E-05    137.6516     10.6876
 326.400 -2.28E-04 -99075.3061   1605.7646   5.106E-05    132.0201      5.0095
 331.200 -3.12E-06 -91315.9358   1617.9538   4.304E-05    126.3028     .069345
 336.000  1.85E-04 -83553.9823   1608.1151   3.568E-05    120.5836     -4.1688
 340.800  3.39E-04 -75887.1768   1579.5225   2.897E-05    114.9345     -7.7447
 345.600  4.63E-04 -68397.9910   1535.2532   2.289E-05    109.4162    -10.7008
 350.400  5.59E-04 -61154.6139   1478.1759   1.744E-05    104.0791    -13.0814
 355.200  6.31E-04 -54211.9716   1410.9442   1.258E-05     98.9635    -14.9319
 360.000  6.80E-04 -47612.7743   1335.9920   8.292E-06     94.1010    -16.2982
 364.800  7.10E-04 -41388.5736   1255.5336   4.545E-06     89.5149    -17.2261
 369.600  7.24E-04 -35560.8163   1171.5650   1.305E-06     85.2208    -17.7608
 374.400  7.23E-04 -30141.8845   1085.8679  -1.461E-06     81.2280    -17.9463
 379.200  7.10E-04 -25136.1100   1000.0164  -3.789E-06     77.5396    -17.8252
 384.000  6.86E-04 -20540.7557    915.3841  -5.712E-06     74.1536    -17.4383
 388.800  6.55E-04 -16346.9582    833.1530  -7.265E-06     71.0635    -16.8247
 393.600  6.17E-04 -12540.6248    754.3229  -8.481E-06     68.2589    -16.0212
 398.400  5.73E-04  -9103.2839    679.7222  -9.393E-06     65.7261    -15.0625
 403.200  5.26E-04  -6012.8846    610.0174  -1.003E-05     63.4490    -13.9812
 408.000  4.77E-04  -3244.5463    545.7245  -1.042E-05     61.4092    -12.8076
 412.800  4.26E-04   -771.2591    487.2185  -1.059E-05     59.5869    -11.5699
 417.600  3.75E-04   1435.4656    434.7438  -1.056E-05     60.0763    -10.2946
 422.400  3.25E-04   3404.9883    388.4227  -1.036E-05     61.5275     -9.0059
 427.200  2.76E-04   5166.9783    348.2643  -9.995E-06     62.8257     -7.7268
 432.000  2.29E-04   6750.8879    314.1718  -9.494E-06     63.9928     -6.4785
 436.800  1.85E-04   8185.4608    285.9488  -8.865E-06     65.0499     -5.2811
 441.600  1.44E-04   9498.2682    263.3049  -8.120E-06     66.0172     -4.1538
 446.400  1.07E-04  10715.2689    245.8599  -7.269E-06     66.9139     -3.1149
 451.200  7.41E-05  11860.3867    233.1470  -6.319E-06     67.7576     -2.1821
 456.000  4.62E-05  12955.0998    -12.5804  -5.274E-06     68.5643   -100.2043
 460.800  2.35E-05  11740.9666   -456.1218  -4.234E-06     67.6697    -84.6046
 465.600  5.54E-06   8577.4161   -800.4639  -3.378E-06     65.3387    -58.8713
 470.400 -8.94E-06   4057.3787   -781.9729  -2.846E-06     62.0082     66.5759
 475.200 -2.18E-05   1071.2062   -422.7139  -2.630E-06     59.8079     83.1154
```

```
 480.000 -3.42E-05      0.0000      0.0000  -2.585E-06     59.0186     93.0154

Output Verification:

Computed forces and moments are within specified convergence limits.


Output Summary for Load Case No.  1:

Pile-head deflection             =      .62641918 in
Computed slope at pile head      =     -.00790575
Maximum bending moment           =    3244577.711 lbs-in
Maximum shear force              =      39000.000 lbs
Depth of maximum bending moment  =         67.200 in
Depth of maximum shear force     =          0.000 in
Number of iterations             =             21
Number of zero deflection points =              3





------------------------------------------------------------------------------
                        Summary of Pile-head Response
------------------------------------------------------------------------------

Definition of symbols for pile-head boundary conditions:

y = pile-head displacment, in
M = pile-head moment, lbs-in
V = pile-head shear force, lbs
S = pile-head slope, radians
R = rotational stiffness of pile-head, in-lbs/rad

 BC    Boundary     Boundary       Axial    Pile Head    Maximum     Maximum
Type  Condition    Condition       Load    Deflection    Moment      Shear
          1            2           lbs         in        in-lbs       lbs
---- ------------ ------------ ----------- ----------- ----------- -----------
  1  V= 39000.000 M=  1.90E+06  26700.0000       .6264   3.245E+06  39000.0000


------------------------------------------------------------------------------
                     Pile-head Deflection vs. Pile Length
------------------------------------------------------------------------------

Boundary Condition Type 1, Shear and Moment

Shear      =          39000. lbs
Moment     =        1897200. in-lbs
Axial Load =          26700. lbs



   Pile       Pile Head       Maximum      Maximum
  Length      Deflection      Moment        Shear
    in           in           in-lbs         lbs
-----------  ------------  ------------  ------------
   480.000     .62641918   3244577.711     39000.000
   456.000     .62710951   3244236.935     39000.000
   432.000     .62701795   3243952.261     39000.000
   408.000     .62674456   3243927.355     39000.000
   384.000     .62720334   3243771.372     39000.000
   360.000     .62702793   3244204.117     39000.000
```

```
   336.000     .62738309   3243871.757     39000.000
   312.000     .62741641   3243931.947     39000.000
   288.000     .62750231   3243656.714     39000.000
   264.000     .62816991   3243422.764     39000.000

The analysis ended normally.
```

```
                                     Napa24C-9E
==============================================================================

               LPILE Plus for Windows, Version 4.0 (4.0.8)

                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:

David An
MGE Engineering, Inc.

Path to file locations:      C:\100%submital-napa-final\Lpile\
Name of input data file:     Napa24C-9E.lpd
Name of output file:         Napa24C-9E.lpo
Name of plot output file:    Napa24C-9E.lpp
Name of runtime file:        Napa24C-9E.lpr


------------------------------------------------------------------------------
                          Time and Date of Analysis
------------------------------------------------------------------------------

               Date:  March 29, 2005     Time:  14:37:35


------------------------------------------------------------------------------
                                Problem Title
------------------------------------------------------------------------------

Napa River Flood Control Project--24 CIDH Pile, Type C-Sta. 9+30 to End of Wall


------------------------------------------------------------------------------
                               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 includes automatic computation of pile-top deflection vs.
  pile embedment length
- No computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- Additional p-y curves computed at specified depths

Solution Control Parameters:
- Number of pile increments            =          100
```

```
                                    Napa24C-9E
- 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                              =     480.00 in
Depth of ground surface below top of pile =     -36.00 in
Slope angle of ground surface            =        .00 deg.

Structural properties of pile defined using  2 points

Point    Depth         Pile       Moment of       Pile      Modulus of
           X         Diameter      Inertia        Area      Elasticity
           in           in          in**4         Sq.in     lbs/Sq.in
-----  ---------   -----------   ----------   ----------   -----------
  1       0.0000       24.000   16286.0000     452.4000   3500000.000
  2     480.0000       24.000   16286.0000     452.4000   3500000.000


-------------------------------------------------------------------------------
                      Soil and Rock Layering Information
-------------------------------------------------------------------------------

The soil profile is modelled using  4 layers

Layer  1 is soft clay, p-y criteria by Matlock, 1970
Distance from top of pile to top of layer    =      -36.000 in
Distance from top of pile to bottom of layer =      108.000 in

Layer  2 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer    =      108.000 in
Distance from top of pile to bottom of layer =      204.000 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 soft clay, p-y criteria by Matlock, 1970
Distance from top of pile to top of layer    =      204.000 in
Distance from top of pile to bottom of layer =      432.000 in

Layer  4 is stiff clay without free water
Distance from top of pile to top of layer    =      432.000 in
Distance from top of pile to bottom of layer =      636.000 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


(Depth of lowest layer extends  156.00 in below pile tip)


-------------------------------------------------------------------------------
                   Effective Unit Weight of Soil vs. Depth
-------------------------------------------------------------------------------

Distribution of effective unit weight of soil with depth
```

```
is defined using  8 points

Point        Depth X    Eff. Unit Weight
 No.           in          lbs/in**3
-----      ----------   ----------------
  1           -36.00         .03360
  2           108.00         .03360
  3           108.00         .04110
  4           204.00         .04110
  5           204.00         .03470
  6           432.00         .03470
  7           432.00         .03470
  8           636.00         .03470



--------------------------------------------------------------------------------
                           Shear Strength of Soils
--------------------------------------------------------------------------------

Distribution of shear strength parameters with depth
defined using  8 points

Point    Depth X     Cohesion c     Angle of Friction     E50 or      RQD
 No.       in         lbs/in**2           Deg.             k_rm        %
-----    --------    ----------     ------------------    ------     ------
  1      -36.000       5.55600               .00          .02000         .0
  2      108.000       5.55600               .00          .02000         .0
  3      108.000        .00000             35.00          ------     ------
  4      204.000        .00000             35.00          ------     ------
  5      204.000       4.16700               .00          .02000         .0
  6      432.000       4.16700               .00          .02000         .0
  7      432.000       8.33000               .00          .00500         .0
  8      636.000       8.33000               .00          .00500         .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    =       39000.000 lbs
```

```
                                    Napa24C-9E
Bending moment at pile head =     1897200.000 in-lbs
Axial load at pile head     =       26700.000 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.




-----------------------------------------------------------------------------
                   Output of p-y Curves at Specified Depths
-----------------------------------------------------------------------------



p-y curves are generated and printed for verification at  6 depths.

Depth      Depth Below Pile Head      Depth Below Ground Surface
 No.                 in                          in
-----      ---------------------      -------------------------
  1                 .000                      36.000
  2              100.000                     136.000
  3              200.000                     236.000
  4              300.000                     336.000
  5              400.000                     436.000
  6              480.000                     516.000

Depth of ground surface below top of pile =     -36.00 in


p-y Curve Computed Using the Soft Clay Criteria for Static Loading Conditions

Soil Layer Number           =          1
Depth below pile head       =        .000 in
Depth below ground surface  =      36.000 in
Equivalent Depth            =      36.000 in
Pile Diameter               =      24.000 in
Cohesion, c                 =       5.556 lbs/in**2
Avg Eff Unit Weight         =      .03360 lbs/in**3
E50 parameter               =      .02000
Default J parameter         =        .500
Y50                         =     1.20000 in
p-multiplier                =     1.00000
y-multiplier                =     1.00000

        y, in           p, lbs/in
----------------    -----------------
          .0000              .000
          .0096            52.907
          .3000           166.647
          .6000           209.962
          .9000           240.346
         1.2000           264.535
         1.5000           284.962
         1.8000           302.817
         2.1000           318.784
         2.4000           333.293
         2.7000           346.639
         3.0000           359.030
         3.3000           370.619
         3.6000           381.526
         9.6000           529.070
```

```
        18.0000          529.070
        24.0000          529.070


p-y Curve Computed Using the Soft Clay Criteria for Static Loading Conditions

Soil Layer Number            =          1
Depth below pile head        =      100.000 in
Depth below ground surface   =      136.000 in
Equivalent Depth             =      136.000 in
Pile Diameter                =       24.000 in
Cohesion, c                  =        5.556 lbs/in**2
Avg Eff Unit Weight          =       .03360 lbs/in**3
E50 parameter                =       .02000
Default J parameter          =         .500
Y50                          =      1.20000 in
p-multiplier                 =      1.00000
y-multiplier                 =      1.00000

        y, in            p, lbs/in
---------------    ----------------
          .0000             .000
          .0096           88.751
          .3000          279.548
          .6000          352.209
          .9000          403.178
         1.2000          443.755
         1.5000          478.021
         1.8000          507.973
         2.1000          534.757
         2.4000          559.097
         2.7000          581.484
         3.0000          602.268
         3.3000          621.710
         3.6000          640.006
         9.6000          887.510
        18.0000          887.510
        24.0000          887.510




p-y Curve in Sand Computed Using Reese Criteria

Soil Layer Number            =            2
Depth below pile head        =      200.000 in
Depth below ground surface   =      236.000 in
Equivalent Depth (see note)  =      219.344 in
Pile Diameter                =       24.000 in
Angle of Friction            =       35.000 deg.
Avg. Eff. Unit Weight        =       .03652 lbs/in**3
k                            =       60.000 pci
A (static)                   =        .8800
B (static)                   =        .5000
Pst                          =     5877.129 lbs/in
Psd                          =    11128.273 lbs/in
Ps                           =     5877.129 lbs/in
pu                           =     5171.874 lbs/in
Cbar                         =    5129.5915
n                            =       1.6447
m                            =    4466.6184
yk                           =        .0904  in
```

```
                                          Napa24C-9E
ym                           =        .4000  in
yu                           =        .9000  in
p-multiplier                 =      1.00000
y-multiplier                 =      1.00000

If Psd <= Pst then actual depth is used in place of equivalent depth in
computations.

        y, in              p, lbs/in
----------------    -----------------
          .0000               .000
          .0333            438.687
          .0667            877.375
          .1000           1264.977
          .1333           1506.765
          .1667           1725.706
          .2000           1928.009
          .2333           2117.448
          .2667           2296.528
          .3000           2467.019
          .3333           2630.226
          .3667           2787.147
          .4000           2938.565
          .9000           5171.874
        24.9000           5171.874
        48.9000           5171.874
        72.9000           5171.874




p-y Curve Computed Using the Soft Clay Criteria for Static Loading Conditions

Soil Layer Number            =          3
Depth below pile head        =      300.000 in
Depth below ground surface   =      336.000 in
Equivalent Depth             =      624.125 in
Pile Diameter                =       24.000 in
Cohesion, c                  =        4.167 lbs/in**2
Avg Eff Unit Weight          =       .03606 lbs/in**3
E50 parameter                =       .02000
Default J parameter          =         .500
Y50                          =      1.20000 in
p-multiplier                 =      1.00000
y-multiplier                 =      1.00000

        y, in             p, lbs/in
----------------    -----------------
          .0000               .000
          .0096             90.007
          .3000            283.505
          .6000            357.194
          .9000            408.885
         1.2000            450.036
         1.5000            484.787
         1.8000            515.163
         2.1000            542.325
         2.4000            567.010
         2.7000            589.714
         3.0000            610.793
```

```
         3.3000          630.509
         3.6000          649.064
         9.6000          900.072
        18.0000          900.072
        24.0000          900.072


p-y Curve Computed Using the Soft Clay Criteria for Static Loading Conditions

Soil Layer Number           =          3
Depth below pile head       =      400.000 in
Depth below ground surface  =      436.000 in
Equivalent Depth            =      724.125 in
Pile Diameter               =       24.000 in
Cohesion, c                 =        4.167 lbs/in**2
Avg Eff Unit Weight         =       .03575 lbs/in**3
E50 parameter               =       .02000
Default J parameter         =         .500
Y50                         =      1.20000 in
p-multiplier                =      1.00000
y-multiplier                =      1.00000

        y, in            p, lbs/in
----------------    -----------------
          .0000             .000
          .0096           90.007
          .3000          283.505
          .6000          357.194
          .9000          408.885
         1.2000          450.036
         1.5000          484.787
         1.8000          515.163
         2.1000          542.325
         2.4000          567.010
         2.7000          589.714
         3.0000          610.793
         3.3000          630.509
         3.6000          649.064
         9.6000          900.072
        18.0000          900.072
        24.0000          900.072




p-y Curve Computed Using Criteria for Stiff Clay without Free Water

Soil Layer Number          =          4
Depth below pile head      =      480.000 in
Depth below ground surface =      516.000 in
Equivalent Depth           =      802.225 in
Diameter                   =       24.000 in
Undrained cohesion, c      =      8.33000 lbs/in**2
Avg. Undrained cohesion, c =      8.33000 lbs/in**2
Average Eff. Unit Weight   =       .03558 lbs/in**3
Epsilon-50                 =       .00500
Pct                        =     4626.136 lbs/in
Pcd                        =     1799.280 lbs/in
y50                        =         .300 in
p-multiplier               =      1.00000
y-multiplier               =      1.00000
```

```
                                             Napa24C-9E
        y, in             p, lbs/in
----------------    -----------------
          .0000              .000
          .0000           101.181
          .0002           151.301
          .0005           179.928
          .0024           269.055
          .0048           319.962
          .0240           478.455
          .0480           568.982
          .1200           715.457
          .2400           850.827
          .3600           941.595
          .4800          1011.810
         1.2000          1272.283
         2.4000          1513.008
         4.8000          1799.280
         5.4000          1799.280
         6.0000          1799.280




-----------------------------------------------------------------------------
             Computed Values of Load Distribution and Deflection
                 for Lateral Loading for Load Case Number  1
-----------------------------------------------------------------------------

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head    =       39000.000 lbs
Specified bending moment at pile head =     1897200.000 in-lbs
Specified axial load at pile head     =       26700.000 lbs

Non-zero moment 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.

  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     1.470   1.897E+06  39000.0000    -.013586   1456.9309   -283.0373
   4.800     1.405   2.083E+06  37629.7985    -.013419   1593.7383   -287.8799
   9.600     1.341   2.262E+06  36237.2025    -.013236   1725.6419   -292.3684
  14.400     1.278   2.434E+06  34823.9293    -.013038   1852.5641   -296.4954
  19.200     1.216   2.600E+06  33391.7317    -.012826   1974.4335   -300.2536
  24.000     1.155   2.758E+06  31942.3983    -.012601   2091.1850   -303.6353
  28.800     1.095   2.909E+06  30477.7543    -.012362   2202.7598   -306.6330
  33.600     1.036   3.054E+06  28999.6619    -.012111   2309.1059   -309.2388
  38.400   .978577   3.191E+06  27510.0222    -.011848   2410.1780   -311.4445
  43.200   .922351   3.321E+06  26010.7754    -.011574   2505.9374   -313.2417
  48.000   .867467   3.444E+06  24503.9029    -.011289   2596.3527   -314.6218
  52.800   .813975   3.559E+06  22991.4288    -.010994   2681.3994   -315.5757
  57.600   .761922   3.667E+06  21475.4216    -.010690   2761.0604   -316.0939
  62.400   .711351   3.768E+06  19957.9967    -.010377   2835.3260   -316.1664
  67.200   .662303   3.861E+06  18441.3190    -.010056   2904.1943   -315.7826
  72.000   .614815   3.948E+06  16927.6061    -.009727   2967.6710   -314.9311
  76.800   .568924   4.026E+06  15419.1320    -.009391   3025.7698   -313.5998
  81.600   .524660   4.098E+06  13918.2315    -.009049   3078.5128   -311.7754
  86.400   .482052   4.162E+06  12427.3054    -.008701   3125.9303   -309.4438
  91.200   .441127   4.219E+06  10948.8267    -.008348   3168.0614   -306.5890
  96.000   .401907   4.270E+06   9485.3482    -.007991   3204.9542   -303.1937
 100.800   .364413   4.313E+06   8039.5116    -.007630   3236.6658   -299.2383
```

```
   105.600   .328662   4.349E+06   6614.0583    -.007265   3263.2632   -294.7006
   110.400   .294669   4.378E+06   3868.6134    -.006898   3284.8229   -849.2347
   115.200   .262445   4.388E+06   -213.5960    -.006528   3291.9307   -851.6859
   120.000   .231995   4.378E+06  -4292.9032    -.006159   3284.5450   -848.0255
   124.800   .203314   4.348E+06  -8339.1583    -.005792   3262.7279   -837.9142
   129.600   .176391   4.299E+06 -12320.5356    -.005428   3226.6514   -820.9931
   134.400   .151206   4.231E+06 -16203.3919    -.005069   3176.6030   -796.8637
   139.200   .127731   4.145E+06 -19952.0006    -.004716   3112.9930   -765.0566
   144.000   .105931   4.041E+06 -23528.0921    -.004372   3036.3621   -724.9815
   148.800   .085764   3.920E+06 -26890.0728    -.004036   2947.3913   -675.8438
   153.600   .067182   3.784E+06 -29991.6823    -.003712   2846.9158   -616.4935
   158.400   .050129   3.633E+06 -32754.3242    -.003400   2735.9444   -534.6073
   163.200   .034545   3.470E+06 -34945.4341    -.003101   2615.8682   -378.3552
   168.000   .020363   3.298E+06 -36402.8292    -.002816   2489.3413   -228.8928
   172.800   .007514   3.121E+06 -37160.0879    -.002545   2358.9024    -86.6317
   177.600  -.004072   2.942E+06 -37252.5120    -.002290   2226.9679     48.1216
   182.400  -.014470   2.764E+06 -36716.6522    -.002050   2095.8270    175.1533
   187.200  -.023750   2.590E+06 -35589.8989    -.001824   1967.6376    294.3272
   192.000  -.031983   2.423E+06 -33910.1460    -.001613   1844.4243    405.5698
   196.800  -.039237   2.265E+06 -31715.5313    -.001416   1728.0768    508.8530
   201.600  -.045575   2.119E+06 -29044.2603    -.001231   1620.3500    604.1766
   206.400  -.051057   1.987E+06 -27217.1613    -.001058   1522.8631    157.1147
   211.200  -.055736   1.858E+06 -26451.8276  -8.965E-04   1428.0276    161.7743
   216.000  -.059664   1.733E+06 -25666.3968  -7.453E-04   1335.9238    165.4885
   220.800  -.062891   1.612E+06 -24865.0159  -6.045E-04   1246.6156    168.4202
   225.600  -.065466   1.494E+06 -24051.1541  -4.737E-04   1160.1538    170.6889
   230.400  -.067438   1.381E+06 -23227.7750  -3.526E-04   1076.5778    172.3857
   235.200  -.068852   1.272E+06 -22397.4525  -2.409E-04    995.9173    173.5820
   240.000  -.069751   1.166E+06 -21562.4521  -1.383E-04    918.1935    174.3349
   244.800  -.070179   1.065E+06 -20724.7898  -4.439E-05    843.4201    174.6911
   249.600  -.070177 967097.0363 -19886.2760   4.116E-05    771.6039    174.6896
   254.400  -.069784 873644.8427 -19048.5492   1.187E-04    702.7456    174.3632
   259.200  -.069038 784200.5487 -18213.1019   1.885E-04    636.8404    173.7398
   264.000  -.067975 698750.7573 -17381.3014   2.509E-04    573.8785    172.8437
   268.800  -.066629 617275.7445 -16554.4060   3.063E-04    513.8453    171.6960
   273.600  -.065034 539749.9457 -15733.5792   3.550E-04    456.7221    170.3152
   278.400  -.063221 466142.3835 -14919.9001   3.974E-04    402.4859    168.7178
   283.200  -.061220 396417.0476 -14114.3733   4.337E-04    351.1102    166.9184
   288.000  -.059058 330533.2340 -13317.9361   4.643E-04    302.5651    164.9304
   292.800  -.056762 268445.8500 -12531.4655   4.895E-04    256.8173    162.7657
   297.600  -.054358 210105.6901 -11755.7837   5.097E-04    213.8306    160.4351
   302.400  -.051869 155459.6868 -10991.6629   5.251E-04    173.5658    157.9486
   307.200  -.049318 104451.1406 -10239.8299   5.360E-04    135.9812    155.3152
   312.000  -.046724  57019.9296  -9500.9691   5.428E-04    101.0325    152.5435
   316.800  -.044107  13102.7043  -8775.7263   5.458E-04     68.6730    149.6410
   321.600  -.041484 -27366.9329  -8064.7115   5.452E-04     79.1833    146.6151
   326.400  -.038873 -64458.2617  -7368.5012   5.413E-04    106.5133    143.4725
   331.200  -.036288 -98243.2888  -6687.6409   5.344E-04    131.4071    140.2193
   336.000  -.033742-128796.6036  -6022.6475   5.249E-04    153.9197    136.8613
   340.800  -.031249-156195.2439  -5374.0108   5.129E-04    174.1078    133.4040
   345.600  -.028819-180518.5705  -4742.1957   4.987E-04    192.0299    129.8523
   350.400  -.026461-201848.1511  -4127.6437   4.826E-04    207.7462    126.2110
   355.200  -.024186-220267.6520  -3530.7751   4.648E-04    221.3182    122.4842
   360.000  -.021999-235862.7390  -2951.9907   4.456E-04    232.8091    118.6759
   364.800  -.019908-248720.9871  -2391.6737   4.252E-04    242.2834    114.7895
   369.600  -.017917-258931.8005  -1850.1916   4.039E-04    249.8071    110.8280
   374.400  -.016031-266586.3425  -1327.8995   3.817E-04    255.4472    106.7937
   379.200  -.014252-271777.4796   -825.1421   3.591E-04    259.2721    102.6885
   384.000  -.012584-274599.7408   -342.2583   3.361E-04    261.3517     98.5131
   388.800  -.011026-275149.2964    120.4148   3.129E-04    261.7566     94.2673
   393.600  -.009580-273523.9627    562.5352   2.898E-04    260.5590     89.9495
   398.400  -.008244-269823.2415    983.7475   2.669E-04    257.8322     85.5557
   403.200  -.007017-264148.4054   1383.6716   2.444E-04    253.6508     81.0793
```

```
                                     Napa24C-9E
  408.000  -.005897-256602.6506   1761.8851   2.225E-04    248.0909     76.5096
  412.800  -.004881-247291.3443   2117.8991   2.013E-04    241.2300     71.8295
  417.600  -.003965-236322.4169   2451.1195   1.809E-04    233.1478     67.0123
  422.400  -.003144-223806.9755   2760.7862   1.616E-04    223.9261     62.0155
  427.200  -.002414-209860.2825   3045.8693   1.433E-04    213.6497     56.7692
  432.000  -.001768-194603.3629   3419.8038   1.263E-04    202.4080     99.0369
  436.800  -.001201-177062.5337   4199.3981   1.106E-04    189.4834    225.7941
  441.600 -7.06E-04-154317.4973   5214.7248   9.668E-05    172.7242    197.2587
  446.400 -2.73E-04-127025.9561   6058.2548   8.483E-05    152.6150    154.2122
  451.200  1.08E-04 -96179.9953   6118.7414   7.543E-05    129.8868   -129.0094
  456.000  4.51E-04 -68305.3739   5379.5335   6.851E-05    109.3480   -178.9938
  460.800  7.66E-04 -44554.0333   4461.3068   6.376E-05     91.8473   -203.6006
  465.600   .001063 -25493.1701   3443.0601   6.081E-05     77.8027   -220.6689
  470.400   .001350 -11516.2426   2351.6979   5.925E-05     67.5041   -234.0653
  475.200   .001632  -2932.0564   1201.1743   5.864E-05     61.1790   -245.3195
  480.000   .001913      0.0000      0.0000   5.852E-05     59.0186   -255.1698

Output Verification:

Computed forces and moments are within specified convergence limits.


Output Summary for Load Case No.  1:

Pile-head deflection             =     1.46979886 in
Computed slope at pile head      =     -.01358650
Maximum bending moment           =    4387600.616 lbs-in
Maximum shear force              =      39000.000 lbs
Depth of maximum bending moment  =        115.200 in
Depth of maximum shear force     =          0.000 in
Number of iterations             =             28
Number of zero deflection points =              2




------------------------------------------------------------------------------
                        Summary of Pile-head Response
------------------------------------------------------------------------------

Definition of symbols for pile-head boundary conditions:

y = pile-head displacment, in
M = pile-head moment, lbs-in
V = pile-head shear force, lbs
S = pile-head slope, radians
R = rotational stiffness of pile-head, in-lbs/rad

 BC    Boundary     Boundary       Axial    Pile Head    Maximum     Maximum
Type  Condition    Condition       Load    Deflection    Moment      Shear
          1            2            lbs        in        in-lbs       lbs
---- ------------ ------------ ----------- ----------- ----------- -----------
  1  V= 39000.000 M=  1.90E+06  26700.0000      1.4698   4.388E+06  39000.0000


------------------------------------------------------------------------------
                     Pile-head Deflection vs. Pile Length
------------------------------------------------------------------------------

Boundary Condition Type 1, Shear and Moment

Shear      =          39000. lbs
                                      Page 10
```

```
                                     Napa24C-9E
Moment     =        1897200. in-lbs
Axial Load =          26700. lbs


   Pile       Pile Head       Maximum      Maximum
  Length      Deflection      Moment        Shear
    in            in          in-lbs         lbs
-----------  ------------  ------------  ------------
   480.000    1.46979886   4387600.616     39000.000
   456.000    1.46839770   4384920.430     39000.000
   432.000    1.48850195   4384670.359     39000.000
   408.000    1.47567339   4384003.829     39000.000
   384.000    1.48329151   4384787.531     39000.000
   360.000    1.49572061   4379186.412     39000.000
   336.000    1.53730311   4354570.300    -40062.433
   312.000    1.62513322   4305573.345    -45047.132
   288.000    1.81916295   4220653.912    -51497.531
   264.000    2.01884044   4143886.323    -58111.233

The analysis ended normally.
```

the Phillips Group
A Structural Engineering Firm
10304 Placer Lane, Suite B
Sacramento, CA 95827
916/361-3871

SHEET NO. A OF
BY GP DATE 3/15/0
JOB NO.

CALCULATIONS FOR

NAPA RIVER / NAPA CREEK
FLOOD PROTECTION PROJECT

CONTRACT 2 WEST
(BROWN STREET TO FIFTH STREET)
VALUE ENGINEERING PLANS

RECEIVED

MAR 27 2006

VALLEY RES. OFFICE
CORPS OF ENGINEERS

No. 2383
Exp. 3/31/06

for pages 1 thru 13

NAPA RIVER/NAPA CREEK
FLOOD CONTROL PROJECT

SHEET NO. 1 OF
BY GP DATE 3/15/06
JOB NO.

SOIL INFORMATION

AT GENERAL STORE AREA

STA. 15+35 THROUGH 16+80 (OLD STATIONS 0+00± TO 2+00±)

THIS WALL IS ADJACENT TO THE GENERAL STORE AND IS BACK FILLED WITH CLSM (CONTROLLED LOW STRENGTH MATERIALS). THIS MATERIAL WILL "SET-UP" LIKE CONCRETE SO NO LATERAL PRESSURES ARE EXERTED ON THE WALL. THE INTENTION IS TO BUILD THE WALL IN LIFTS TO LIMIT THE PRESSURE EXERTED ON THE WALL BY THE BACKFILL.

γ_{SAT} = 165 PCF (MAX.) - CLSM $\phi = 35°$

RESISTING MATERIAL

-8' TO -33' DENSE SAND / GRAVEL γ_{MOIST} = 115 PCF, γ_{SAT} = 120 PCF
γ_b = 120 - 62.5 = 57.5 PCF $\phi = 38°$

-33 TO -58' CLAYS γ_m = 115 PCF, γ_{SAT} = 120 PCF, γ_b = 57.5 PCF
R_{TEST} $\phi = 15°$, C = 250 PSF
S_{TEST} $\phi = 32°$, C = 0 PSF
Q_{TEST} $\phi = 0°$, C = 1200 PSF

AT PLAZA / NAPA RIVER INN (3 STORY)

STA. 16+80 THROUGH 19+48 (OLD STA. 2+00± TO 4+50±)

TIE BACK WALL ANALYSIS

BACKFILL γ = 125 PCF $\phi = 37°$ K_{ACTIVE} = .25, K_{PASS} = 4.02,
γ_{SAT} = 130 PCF K_o = .38 (AT REST)

RESISTING MATERIAL

+15 TO +4 SANDY CLAY γ_m = 115 PCF, γ_{SAT} = 120 PCF, γ_b = 57.5 PCF
R_{TEST} $\phi = 15°$, C = 250 PSF
S_{TEST} $\phi = 32°$, C = 0 PSF
Q_{TEST} $\phi = 0°$, C = 1400 PSF

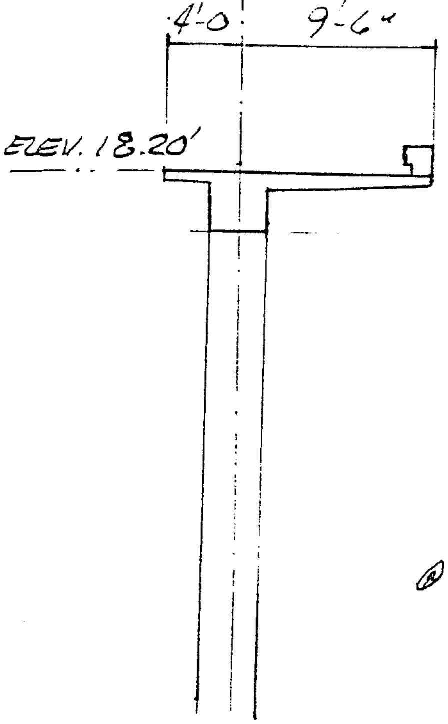
+4' TO -2' FAT CLAY QULT 13' γ_m = 112 PCF, γ_s = 120 PCF, γ_b = 57.5 PC
R_{TEST} $\phi = 10°$ C = 500 PSF
S_{TEST} $\phi = 27°$ C = 0 PSF
Q_{TEST} $\phi = 10°$ C = 500 PSF

-2' TO -6' SANDY CLAY γ_m = 115 PCF, γ_{SAT} = 120 PCF, γ_b = 57.5 PCF
R_{TEST} $\phi = 15°$ C = 250 PSF
S_{TEST} $\phi = 32°$ C = 0 PSF
Q_{TEST} $\phi = 0°$ C = 1200 PSF

-6 TO -26 CLAYEY SAND / GRAVEL γ_m = 128 PCF, γ_{SAT} = 136 PCF, γ_b = 73.
$\phi = 39°$

SOIL PROFILES 2+00 TO 4+75
ESSENTIALLY WALL FROM GENERAL STORE TO PAST 3 STORY BUILL

CASE FROM 18+15 TO 19+50
TYPICAL WIDTH = 13.5'



RAILING WEIGHT = 40#/, DEAD

CONC. CURB = [16×23.75-2×5.75]/144 ×150
= 217.2 #/, DEAD

CONC. "BEAM" = 3.0×3.5×150 = 1575#/, DEA

RT. CONC. SLAB = .75(150) = 112.5#/'²
1.0(150) = 150. #/'²
×7.75' = 7.75(112.5+37.5/2) = 1017

LF. CONC. SLAB = $\frac{(12+11.129)}{2\times12}$×150 = 144.56#
×2.25' = 325.25#/,

MISC. = 2/12×120 = 20 PSF

UNBALANCED D+L MOMENT (#-'/,)

@ PIER ℄ MD = 40×8.83' + 217×8.83' + 133×7.75×5.625
+ 37.5×7.75/2 ×4.33' − 165×2.25×2.87
= +7429.57 #-'/, DEAD

FOR SLAB DESIGN MD = 6361.29 #-'/,

@ PIER ℄ ML = 300×7.75×5.625 = 13078.1 #-'/,

TOTAL UNBALANCED D+L = 20707.7 #-

ML FOR SLAB = 300×7.75²/2 = 9009.38 #-'/,

MU = 1.4×6361.29 + 1.7×9009.38 = 24221.75 #-'/,

TRY #5 @12"oc d = 9" f'c = 4000 fy = 60000

a = .31×60/.85×12×4 = .456

φMN = (.9)(.31)(60)(9 − .456/2) = 146.84 = 12.237 'k-'

IF #5 @ 6"oc a = .912

φMN = (.9)(.62)(60)(9 − .912/2) = 286.05 = 23.84'k

IF LIVE LOAD = 250 PSF ML = 7507.8 #-'

MU = 21,669 #-' < 23840 #-'

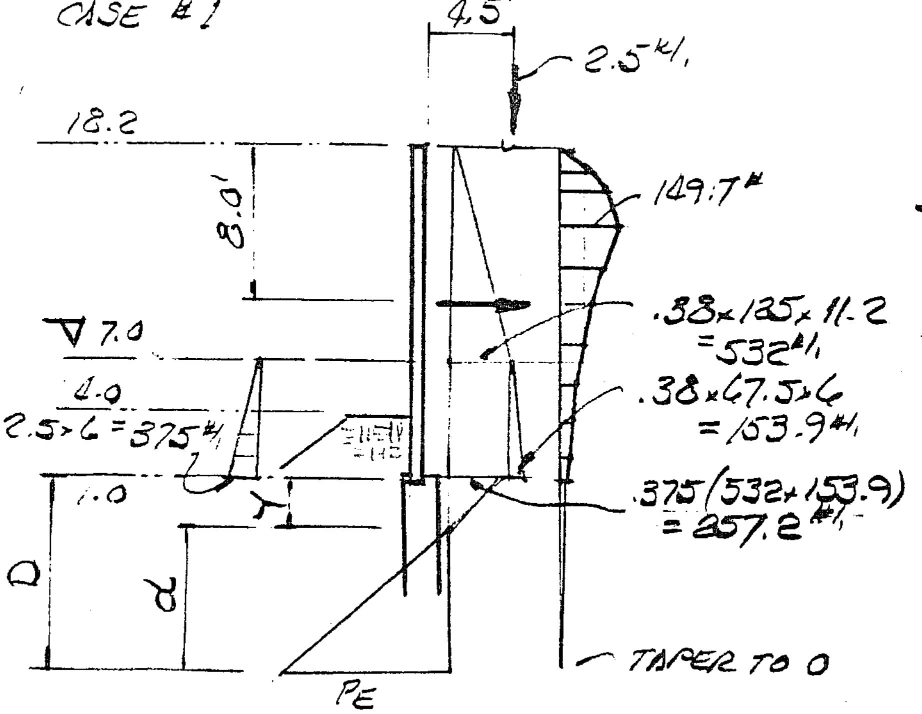
USE #5 HORIZ. @ 6"oc

CANTILEVER SLAB = L/10
MIN DEPTH = L/10 = 11.4"/12
SLAB = 9.5' L/.5 = 8'
(9.6")

WALL "B" FIN GRADE +4.0, B.O.W. +1.0
FIN GRADE +0.0 B.O.W. −6.0

WANT BOTTOM OF WALL
NO LOWER THAN −4.0
WALL "C" IS +1.00 B.O.W. F.G. = 4.00

TIE BACK WALL DESIGN
CASE #1



3'-0 DIAMETER PIER
@ 12'-0 oc
USE "f" = 1.5(3.0)/12 = .37
EFFECT OF LINE LOAD
$a/h = 4.5/17.2 = .262 < .4$

$$\Delta P_{HZ} = \left(\frac{2500}{17.2}\right)\left(\frac{.203b}{(.16+b^2)^2}\right)$$

$b \le 1.0$

P	bh	b
64.1	1.0	.058
115.8	2.0	.116
149.7	4.0	.233
129.8	6.0	.349
96.9	8.0	.465
69.2	10.0	.581
49.2	12.0	.698
35.5	14.0	.814
26.1	16.0	.930
21.9	17.2	1.0

.5 MAX

CASE ' SHORT TERM WALL @ 13.2' WATER @ 7.0

SOIL PROPERTIES BELOW WALL
USE $\gamma_b = 57.5$ IF $\phi = 27°$
$K_A = \tan^2(45-27/2) = .376$; $K_p = 1/.376 = 2.663$

IF $\phi = 32°$
$K_A = \tan^2(45-32/2) = .307$
$K_p = 1/.307 = 3.257$

$Y = 257.2/(.375)(2.663-.376)(57.5) = \underline{5.216'}$

LOAD FROM SURCHARGE

P	h (from top)	Ph
64.1×1/2 = 32.05	.67'	21.47
64.1+49.7/2 = 88.95	1.55'	137.87
113.8×2+35.9 = 263.5	3.05'	803.68
129.8×2+19.9 = 279.5	4.98'	1391.91
96.9×2+32.9 = 226.7	6.95'	1575.57
69.2×2+27.7 = 166.1	8.95'	1486.60
49.2×2+20 = 118.4	10.94'	1295.30
35.5×2+13.7 = 84.7	12.95'	1096.87
26.1×2+9.4 = 61.6	14.95'	920.92
21.9×1.2+2.1 = 28.38	16.59'	470.82
1349.88	6.816'	9201.01

AT BOTTOM OF WALL
$21.9 \times .375 = 8.21$ PSF
AND
"P" = 8.21(5.216+d)/2
= 21.41 + 4.105(d)

SOIL LOADS
$P_1 = 532 \times 11.2 \times 1/2 = 2979.2$#
$P_2 = 532 \times 6 = 3192.0$#
$P_3 = 153.9 \times 6 \times 1/2 = 461.7$#
$P_4 = 257.2 \times 5.216 \times 1/2 = 670.8$#
$P_{H_2O} = 375 \times 6.0 \times 1/2 = 1125$#
"P_E" = .375×(57.5)(d)(2.287) = 49.31d
$P_E = 49.31d(d/2) = 24.657d^2$

$\Sigma M_T = 0$

$0 = +2979.2(.53) - 3192(6.20) - 461.7(7.20) - 670.8(10.94) + 1125(7.20)$
$+1349.38(1.174) + 24.657d^2(\tfrac{2}{3}d + 14.416) - (21.41 + 4.105d)(14.42 + \tfrac{2}{3}d$

$0 = -19,498.18 + 16.438d^3 + 352.718d^2 - 73.468d$

$\therefore\ d^3 + 21.457d^2 - 4.469d = 1186.17$
IF d = 6.5 1152.13 vs 1186.17
d = 6.6 1192.67 vs 1186.17
$\therefore$ D = 6.6 + 5.22 = $\underline{11.82' \times 1.3 = 15.37' \approx 15'\text{-}4''}$

NAPA RIVER/NAPA CREEK
FLOOD CONTROL PROJECT

CASE #1

TIE BACK LOAD

$\Sigma F_H = 0 = \overline{2979.2} + \overline{3192.0} + \overline{461.7} + \overline{670.8} - \overline{1125} + \overline{1349.88} + \overline{48.50}$
$- 1074.04 = 6503.02^{\#/\prime}$

$T_{TOTAL} = 12 \times 6503.02 = 78,036^{\#}$
$F = 78,036 / \cos 15° = \underline{80,789^{\#}}$

$M_{CANTILEVER} = 47.5 \times 8^3/6 + 32.05 \times 7.33' + 88.75 \times 6.45' + 263.5 \times 4.95'$
$+ 279.5 \times 3.02' + 226.7 \times 1.05 = \underline{-7247.15^{\#-\prime}}$

$V_{CANTILEVER} = 47.5 \times 8^2/2 + 32.05 + 88.75 + 263.5 + 279.5 + 226.7 = 2410.5^{\#}$

POINT OF ZERO SHEAR

$V = 6503.02 - 2410.5 = 4092.52$
AT 7.0' $V = 4092.52 - 380(4.2) - 152(4.2)/2 - 166.1 - 118.4 = 1892.8$
$1892.8 - 84.7 = 532x + 25.65x^2/2$
$12.825x^2 + 532x - 1808.1 = 0 \qquad x = 3.158'$

$M = 4092.52(7.358) - 380(7.358)^2/2 - 152(4.2)(4.56)/2 - 152(3.158)^2/2$
$- 25.65(3.158)^3/6 - 166.1(5.408) - 118.4(3.418) - 84.7(1.408) - 7247.15$
$\underline{= +8,808.63^{\#-\prime}}$

CASE #1 NO SURCHARGE FROM DOZER, BUT ADD UNBALANCED DEAD LOAD MOMENT FROM TOP.

$M = 7629.57^{\#-\prime} \curvearrowleft$

$P_1 = 2979.2^{\#}, \ P_2 = 3192.0^{\#}, \ P_3 = 461.7^{\#}, \ P_4 = 670.8^{\#}, \ P_{H_2O} = 1125^{\#}$
$"P_B" = 24.657d^2$

$\underset{\curvearrowleft +}{\Sigma M_T} = 0 = +7629.57 + 2979.2(.53) - 3192(6.20) - 461.7(7.20) - 670.8(10.94$
$+ 1125(7.20) + 24.657d^2(\tfrac{2}{3}d + 14.416)$

$0 = -13144.64 + 16.438d^3 + 355.455d^2$
$\therefore d^3 + 21.624d^2 = 799.65$
IF $d = 5.4'$ 788.02 vs 799.65
$d = 5.42'$ 794.44 vs 799.65
$\therefore D = 5.43 + 5.22 = 10.65 \times 1.3' = 13.85' \approx \underline{13'-10''}$

SEISMIC LOADS

$K_H = .15\,g \qquad K_V = 0$ (ASSUMED)

$\beta = 0\ ,\ \delta = 0\ ,\ \theta = 0$

SOIL PROPERTIES "DRIVING" SIDE BOTTOM OF WALL TO TOP OF W

$\gamma_M = 125 \quad \gamma_S = 130 \quad \gamma_b = 67.5 \quad \phi = 37° \quad K_A = .25 \quad K_p = 4.02$

$K_o = .38$

RESISTING SIDE PROPERTIES VARY

USE $\gamma_M = 115 \quad \gamma_S = 120 \quad \gamma_b = 57.5 \quad \phi = 32° \quad K_A = .307 \quad K_p = 3.2$

DRIVING SIDE

$$C_1 = \frac{2(\tan 37° - .15)}{1 + .15(\tan 37°)} = \frac{1.207}{1.113} = 1.078$$

$$C_2 = \frac{\tan 37° - .15}{\tan 37°(1 + .15\tan 37°)} = \frac{.604}{.839} = .720$$

$$\alpha = \tan^{-1}(1.078 + (1.078^2 + 4 \cdot .604)^{1/2})/2 = 56.04°$$

$$K = \frac{1 - \tan 37° \cot 56.04°}{1 + \tan 37° \tan 56.04°} = .492/2.119 = \underline{.232}$$

$K_A = K \quad (\beta = 0) \quad K_p = K$

$K_{AE} = K_h / \tan\alpha = .15/1.485 = .101$

RESISTING SIDE (NEED TO FIND BOTH ACTIVE AND PASSIVE LOADS)

$$C_1 = \frac{2(\tan 32° - .15)}{1 + .15\tan 32°} = \frac{.950}{1.094} = .869$$

$$C_2 = \frac{(\tan 32° - .15)}{\tan 32°(1 + .15\tan 32°)} = \frac{.475}{.683} = .695$$

$$\alpha_{ACTIVE} = \tan^{-1}(.869 + (.869^2 + 4 \cdot .695)^{1/2})/2 = 53.97°$$

$$\alpha_{PASSIVE} = \tan^{-1}(-.869 + (.869^2 + 4 \times .695)^{1/2})/2 = 26.821°$$

$$K = \frac{1 - \tan 32° \cot 53.97°}{1 + \tan 32° \tan 53.97°} = .546/1.859 = .294$$

$K_A = K_p = K = \underline{.294}$

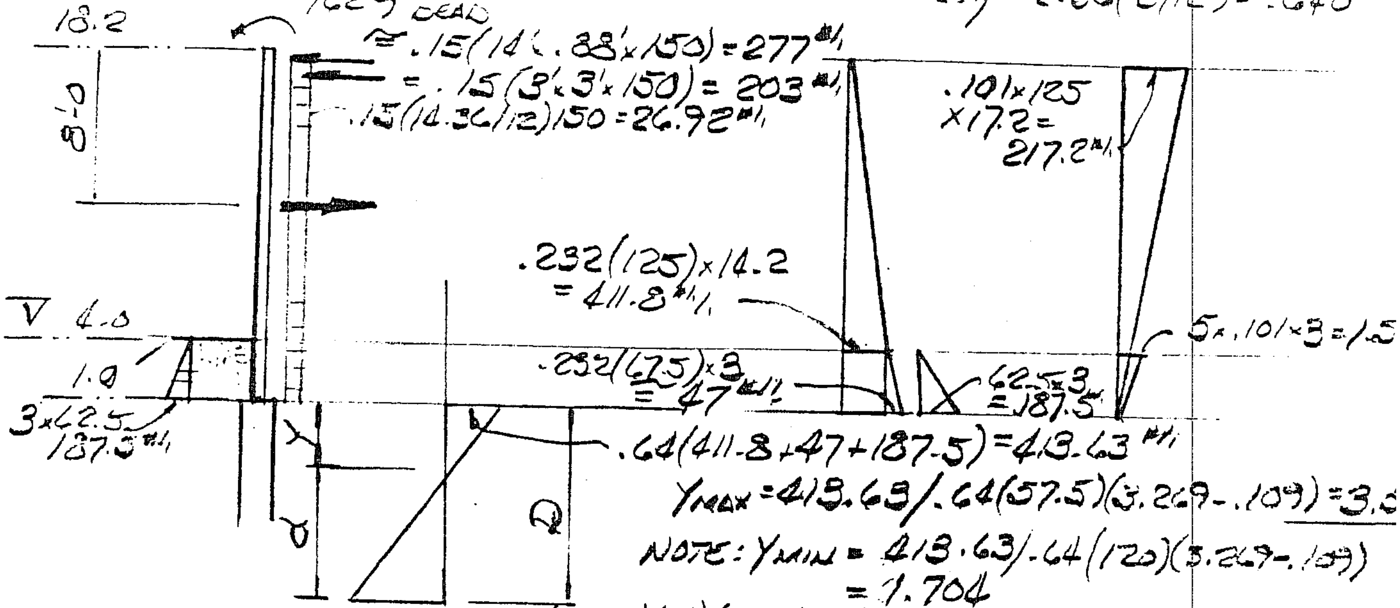
$K_{AE} = K_h / \tan\alpha = .15/1.375 = \underline{.109}$

$$K_p = \frac{1 + \tan 32° \cot 26.821}{1 - \tan 32° \tan 26.821} = 2.234/.684 = \underline{3.269}$$

$K_{pE} = K_h / \tan\alpha = .15/\tan 26.821 = \underline{.297}$

TIE BACK WALL DESIGN
SEISMIC LOADING

f ARCHING FACTOR
$= .08 \times \phi = .08 \times 32° = 2.56$
$\therefore f = 2.56(3/12) = .640$



$Y_{MAX} = 413.63 / .64(57.5)(3.269 - .109) = 3.5$

NOTE: $Y_{MIN} = 413.63 / .64(120)(3.269 - .109)$
$= \underline{1.704}$

$(P_{E_{MIN}} = .64(57.5)(d)(3.16) = 116.29d$ $\quad P_{B_{MIN}} = 58.14d^2$
$P_{E_{MAX}} = .64(120)(d)(3.16) = 242.69d$ $\quad P_{B_{MAX}} = 121.31d^2$

CHECK CASE OF
$Y_{MAX} = 3.56'$; $P_{B_{MIN}} = 58.14d^2$

SEISMIC LOADS (WALL)
$P_A = 277^{\#/1}$, $P = 203^{\#/1}$, $P_{WALL} = 26.92 \times 17.2 = 463.^{\#/1}$

SOIL LOADS
$P_1 = 411.8 \times 14.2 \times 1/2 = 2923.78$
$P_2 = 411.8 \times 3 = 1235.4$
$P_3 = 47 \times 3 \times 1/2 = 70.5$
$P_4 = 187.5 \times 3 \times 1/2 = 281.25$
$P_5 = 217.2 \times 17.2 \times 1/2 = 1867.92$
$P_6 = 1.52 \times 3 \times 1/2 = 2.28$
$P_7 = 3.56 \times 413.63 \times 1/2 = 490.8$
$P_{H2O} = 187.5 \times 3 \times 1/2 = 281.2$

$\Sigma M_T = 0 = +277(7.58) + 203(5.5) - 463(.6)$
$- 2923.78(1.467) - 1235.4(7.7) - 70.5(8.2)$
$+ 1867.92(2.267) - 2.28(8.2) + 7629$
$- 490.84(10.387) + 58.14d^2(\tfrac{2}{3}d + 12.76)$

736.26

$0 = -4694.99 + 38.76d^3 + 741.866d^2$
$\therefore d^3 + 19.14d^2 = \underline{121.13}$
IF $d = 2.5'$ 135.25 VS 121.13
$\underline{D} = 3.56 + 2.5 = \underline{6.06 \times 1.3 = 7.9' \approx 8'0}$

CASE OF $Y_{MIN} = 1.704'$; $P_{B_{MAX}} = 121.31d^2$
$0 = -4694.99 + 490.84(10.387) - 352.41(9.768)$
$+ 121.31d^2(10.904 + \tfrac{2}{3}d)$
$0 = -3038.98 + 1322.76d^2 + 80.87d^3$
$\therefore d^3 + 16.36d^2 = 37.58$
IF $d = 1.5$ 40.18 VS 37.58
$D = 1.704 + 1.5 = \underline{3.2 \times 1.3 = 4.16' = 4'2"}$

$P_7 = 413.63 \times 1.704 \times 1/2 = 352.41$

TIE BACK LOAD FOR Y_{MAX}
$\Sigma F_H = 0 = 277 + 203 + 463 + 2923.78 + 1235.4 + 70.5 + 1867.92 + 490.84 + 2.28$
$- 58.14(2.4)^2 = 7198.8^{\#/1}$
$T_{OTAL} = 7198.8 \times 12 = \underline{86,386^{\#}}$ $\quad F = 86,386/\cos 15° = \underline{89433.4^{\#}}$

SEISMIC LOADING (CONT.)

TIE BACK LOAD FOR YMIN.

$\Sigma F_H = 0 = 277 + 203 + 463 + 2933.78 + 1235.4 + 70.5 + 1867.92 + 352.41 + 2.2$
$- 121.31(1.45)^2 = 7150.2^{\#/'}$

$T_{TOTAL} = 7150.2 \times 12 = \underline{85{,}802.4^{\#}}$ $F = 85{,}802.4 / \cos 15° = \underline{88{,}829^{\#}}$

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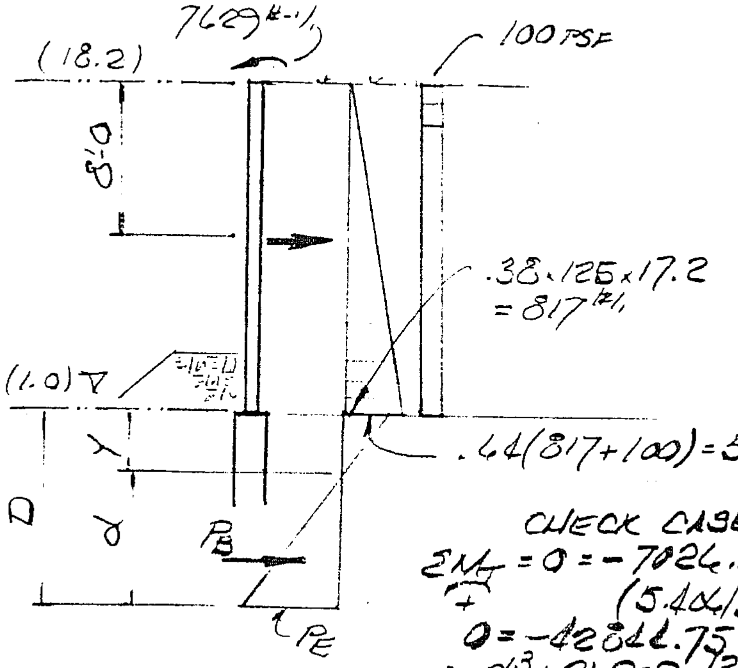
FLOOD CONTROL PROJECT

SHEET NO. 8 OF
BY GP DATE
JOB NO.

MOST COMMON CASES



ARCHING FACTOR
$= .08 \times \phi = .08 \times 32 = 2.56$
$\therefore f = 2.56(3/12) = .640$

$Y_{MAX} = 586.88/.64(57.5)(3.257 - .307)$ $^{2.95}$
$= 5.406'$

$Y_{MIN} = 586.88/.64(120)(2.95) = 2.5$

$P_{E_{MIN}} = .64(57.5)(d)(2.95) = 108.5$

$P_{E_{MAX}} = .64(120)(d)(2.95) = 226.5$

$P_{B_{MIN}} = 108.56d \times d/2 = 54.28d^2$

$P_{B_{MAX}} = 226.56d \times d/2 = 113.28d^2$

SOIL LOADS
$P_1 = \frac{1}{2} \times 817 \times 17.2 = 7026.2$
$P_2 = 100 \times 17.2 = 1720$

CHECK CASE OF $Y_{MAX} = 5.406'$; $P_{B_{MIN}} = 54.28d^2$

$\Sigma M_{\overset{\frown}{+}} = 0 = -7026.2(3.467) - 1720(.6) - (586.88 \times 5.406 \times \frac{1}{2})$
$(5.406/3 + 9.2) + 54.28d^2(14.606 + \frac{2}{3}d)$

$0 = -42,866.75 + 792.814d^2 + 36.187d^3$

$\therefore d^3 + 21.909d^2 = 1183.98$

IF $d = 6.5'$ 1200.28 VS 1183.98

$d = 6.46'$ 1183.88 VS 1183.98 $\therefore$ OK

$\therefore D = 5.406 + 6.46 = \underline{11.866 \times 1.3 = 15.43' \approx 15'\text{-}6''}$

CHECK CASE OF $Y_{MIN} = 2.59'$; $P_{B_{MAX}} = 113.28d^2$

$\Sigma M_{\overset{\frown}{+}} = 0 = -7026.2(3.467) - 1720(.6) - (586.88 \times 2.59 \times \frac{1}{2})(2.59/3 + 9.2)$
$+ 113.28d^2(11.79 + \frac{2}{3}d)$

$0 = -33,440.07 + 1335.57d^2 + 75.52d^3$

$\therefore d^3 + 17.685d^2 = 437.50$

IF $d = 4.5$ 449.25 VS 437.50

$d = 4.45$ 433.33 VS 437.50 $\cdot$ OK

$\therefore D = 2.59 + 4.45 = \underline{7.04 \times 1.3 = 9.15' \approx 9'\text{-}2''}$

TIE BACK LOAD

Y_{MAX} CASE $\Sigma F_H = 0 = 7026.2 + 1720 + 1586.34 - 54.28(6.46)^2 = 8067.35$
$T_{TOTAL} = 12 \times 8067.35 = \underline{96808.2\#}$
$F = 96808.2/\cos 15° = \underline{100,223\#}$

Y_{MIN} CASE $\Sigma F_H = 0 = 7026.2 + 1720 + 760.01 - 113.28(4.45)^2 = 7262.98\#$
$T_{TOTAL} = 12 \times 7262.98 = \underline{87155.8\#}$
$F = 87155.8/\cos 15° = \underline{90,230.3\#}$

CHECK BOTH CASES W/ UNBALANCED DEAD LOAD MOMENT

"Y_{MAX}" $0 = -35,215.75 + 792.814d^2 + 36.187d^3$

$\therefore d^3 + 21.909d^2 = 973.16$

$d = 5.91'$ 971.67 VS 9.73.16

$D = 5.406 + 5.91 = \underline{11.32 \times 1.3 = 14.71' \approx 14'\text{-}9''}$

MOST COMMON CASES (CONT.)

UNBALANCED DEAD LOAD MOMENT

"Y_{MIN}" $0 = 25411.07 + 1335.57d^2 + 75.52d^3$

$\therefore d^3 + 17.685d^2 = 336.48$

$d = 3.95$ 337.6 VS 336.48

$\therefore D = 2.59 + 3.95 = \underline{6.54 \times 1.3 = 8.5' \times 1.3} = 11.05 \simeq 11'\text{-}1''$

T@ "Y_{MAX}" $\Sigma F_H = 0 = +7026.2 + 1720 + 1586.24 - 54.28(5.91)^2 = 8436.54$

$T = 12 \times 8436.54 = \underline{101,238.5^{\#}}$

$F = 101,238.5 / \cos 15° = \underline{104809.8^{\#}}$

T@ "Y_{MIN}" $\Sigma F_H = 0 = 7026.2 + 1720 + 760.01 - 113.28(3.95)^2 = 7738.76^{\#/}$

$T = 12 \times 7738.76 = \underline{92,865.1^{\#}}$

$F = 92865.1 / \cos 15° = \underline{96,141^{\#}}$

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NAPA RIVER / NAPA CREEK
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SHEET NO. 10 OF
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MOST COMMON CASES
GENERAL STORE AREA

$.08\phi = (.08 \times 38) = 3.04$ - USE 3

$\therefore f = 3(3/12) = .75$

$P_1 = 864.5 \times 18.2 \times 1/2 = 7866.95$

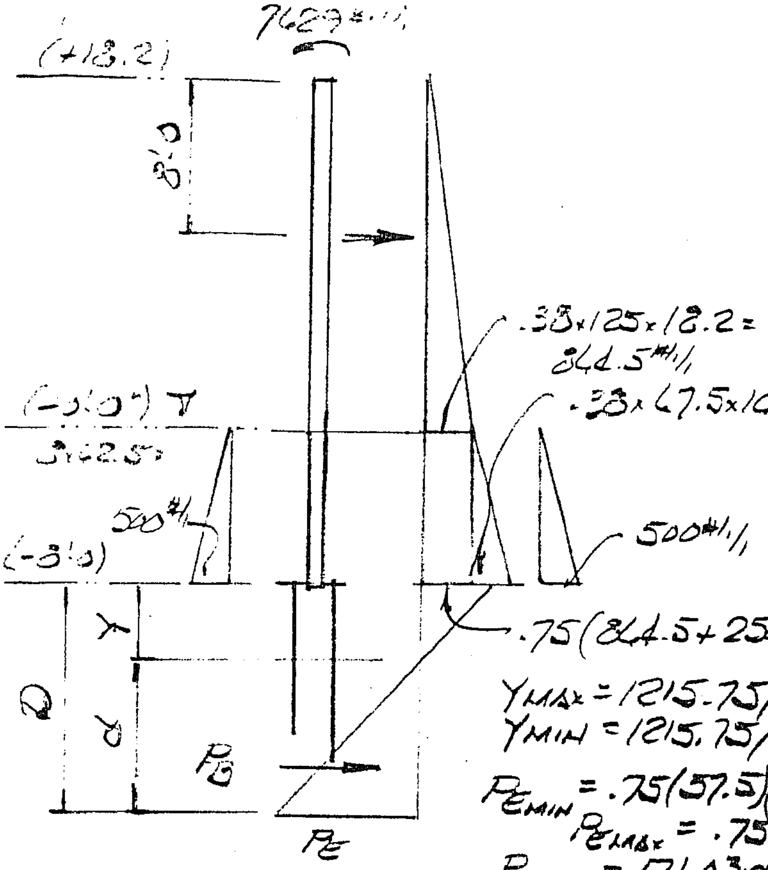
$P_2 = 864.5 \times 8 = 6916.0^{\#}$

$P_3 = 256.5 \times 1/2 \times 8 = 1026^{\#}$

$P_4 = P_5 = 500 \times 1/2 \times 8 = 2000^{\#}$

"P_{MAX}" $= 1215.75 \times 1/2 \times 7.108 = 4320.$

"P_{MIN}" $= 1215.75 \times 1/2 \times 3.406 = 2070.4$



$.75(864.5 + 256.5 + 500) = 1215.75^{\#/1}$

$Y_{MAX} = 1215.75 / .75(57.5)(\overset{3.944}{4.204} - .233) = 7.108'$

$Y_{MIN} = 1215.75 / .75(120)(3.944) = 3.406'$

$P_{E_{MIN}} = .75(57.5)(3.944)d = 171.03\,d$

$P_{E_{MAX}} = .75(120)(3.944)d = 354.94\,d$

$P_{B_{MIN}} = 171.03d \times d/2 = 85.52d^2$

$P_{B_{MAX}} = 356.94d \times d/2 = 178.47d^2$

$K_a = .233$
$K_p = 4.204$

"Y_{MAX}" CASE

$\overset{\frown}{\Sigma M_T} = 0 = -7866.95(4.133) - 6916(14.2) - 1026(15.533) - 4320.78(20.569)$
$+ 85.52d^2(25.308 + 2/3 d)$

$0 = -235{,}532.28 + 2164.34d^2 + 57.01d^3$

$\therefore d^3 + 37.96d^2 = 4131.42$

$d = 9.35 \quad 4135.96 \text{ vs } 4131.42$

$D = 9.35 + 7.108 = \underline{16.45 \times 1.3 = 21.45}$ IF $\underline{1.2 \quad D = 19.75'}$

"Y_{MIN}" CASE

$\overset{\frown}{\Sigma M_T} = 0 = -7866.95(4.133) - 6916(14.2) - 1026(15.533) - 2070.42(19.335)$
$+ 178.47d^2(21.604 + 2/3 d)$

$0 = -186{,}688.93 + 3856.02d^2 + 118.98d^3$

$\therefore d^3 + 32.409d^2 = 1569.08$

$d = 6.36' \quad 1568.19 \text{ vs } 1569.08$

$D = 3.406 + 6.36 = \underline{9.77' \times 1.3 = 12.70' \approx 12'\text{-}9"}$

TIE BACK LOADS

"Y_{MAX}" $\Sigma F_H = 0 = 7866.95 + 6916 + 1026 + 4320.78 - 85.52(9.35)^2 = 12653.4$

$T_{TOTAL} = 12 \times 12653.4 = 151{,}840.3^{\#}$

$F = 151{,}840.3 / \cos 15° = 157{,}196.6^{\#}$

"Y_{MIN}" $\Sigma F_H = 0 = 7866.95 + 6916 + 1026 + 2070.42 - 178.47(6.36)^2 = 10660$

$T_{TOTAL} = 12 \times 10660 = \underline{127{,}923^{\#}} \qquad F = 127{,}923 \cos 15° = \underline{132{,}436.6^{\#}}$

SEISMIC LOADS
AT GENERAL STORE AREA
"RESISTING" SOIL PROPERTIES

$\gamma_M = 115$ PCF, $\gamma_{SAT} = 125$ PCF, $\gamma_b = 57.5$ PCF $\phi = 38°$

PROPERTIES FOR BOTH "ACTIVE" & "PASSIVE"

$K_H = .15\,g$

$$C_1 = \frac{2(\tan 38° - .15)}{1 + .15 \tan 38°} = 1.263/1.117 = 1.131$$

$$C_2 = \frac{\tan 38° - .15}{\tan 38° (1 + .15 \tan 38°)} = .631/.873 = .723$$

$$\alpha_{ACTIVE} = \tan^{-1}\left(1.131 + (1.131^2 + 4 \cdot .723)^{1/2}\right)/2 = 57.78°$$

$$\alpha_{PASSIVE} = \tan^{-1}\left(-1.131 + (1.131^2 + 4 \cdot .723)^{1/2}\right)/2 = 24.50°$$

$$K = \frac{1 - \tan 38° \cot 57.78°}{1 + \tan 38° \tan 57.78°} = .508/2.24 = \underline{.227}$$

$$K_A = K_b = K = \underline{.227}$$

$$K_{AE} = K_h / \tan\alpha = .15/\tan 57.78 = \underline{.095}$$

$$K_P = \frac{1 + \tan 38° \cot 24.50°}{1 - \tan 38° \tan 24.50°} = 2.714/.644 = \underline{4.215}$$

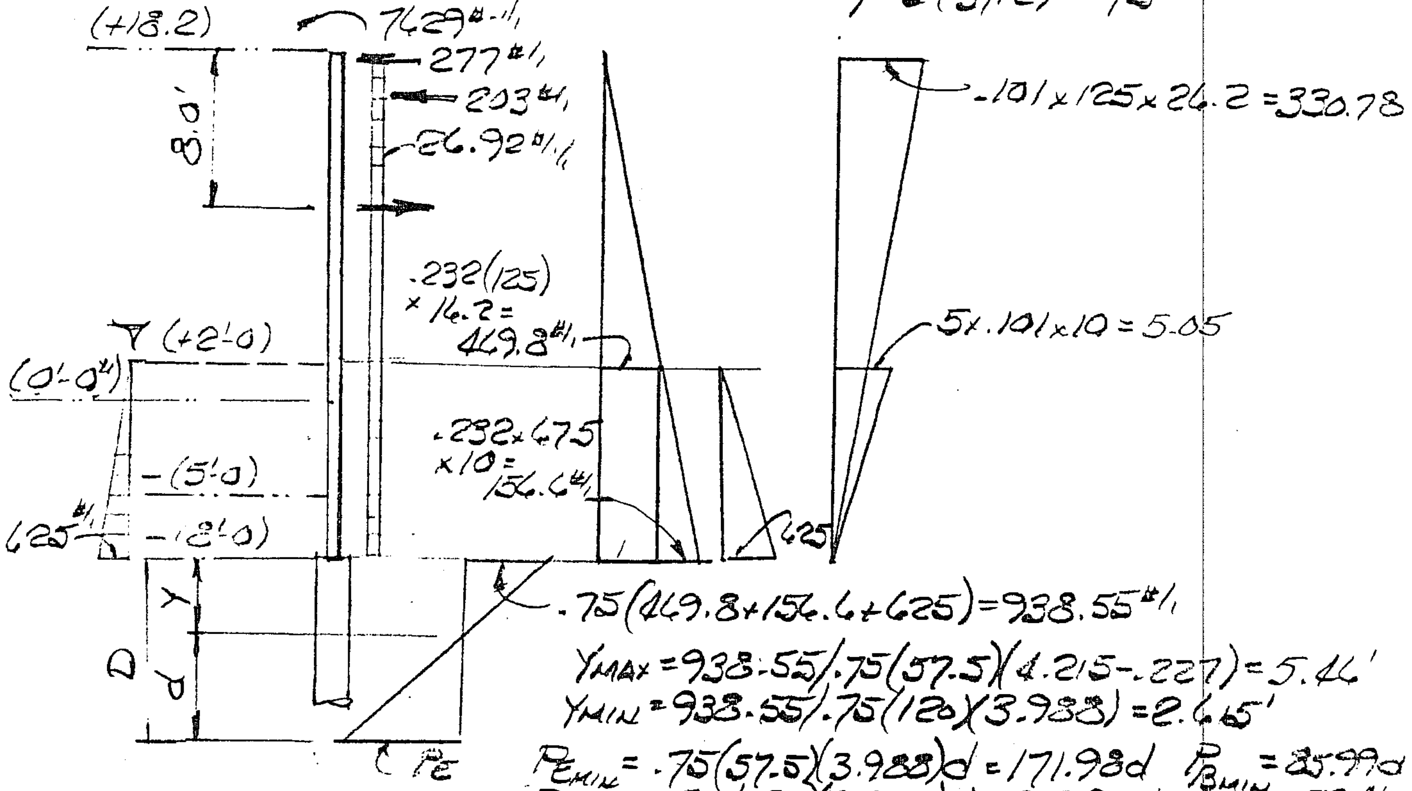
$$K_{PE} = K_h / \tan\alpha = .15/\tan 24.50° = \underline{.329}$$

SEISMIC LOAD CASE

GENERAL STORE (THIS CASE MAY NOT ACTUALLY OCCUR)

$.08\phi = (.08 \times 38) = 3.04$ - USE 3

$\therefore f = 3(3/12) = .75$



$.75(469.8 + 156.6 + 625) = 938.55^{\#/'}$

$Y_{MAX} = 938.55/.75(57.5)(4.215 - .227) = 5.46'$

$Y_{MIN} = 938.55/.75(120)(3.988) = 2.65'$

$P_{E_{MIN}} = .75(57.5)(3.988)d = 171.98d$ $\quad P_{B_{MIN}} = 85.99d$

$P_{E_{MAX}} = .75(120)(3.988)d = 358.92d$ $\quad P_{B_{MAX}} = 179.46$

SEISMIC LOADS

$P_A = 277^{\#/'}, P = 203^{\#/'}, P_{WALL} = 26.92 \times 26.2 = 705.3^{\#/'}$

SOIL LOADS

$P_1 = \frac{1}{2} \times 469.8 \times 16.2 = 3805.38$

$P_2 = 469.8 \times 10 = 4698.0$

$P_3 = \frac{1}{2} \times 156.6 \times 10 = 783.0$

$P_4 = \frac{1}{2} \times 625 \times 10 = 3125$

$P_{H2O} = \frac{1}{2} \times 625 \times 10 = 3125$

$P_5 = \frac{1}{2} \times 330.78 \times 26.2 = 4333.22$

$P_6 = \frac{1}{2} \times 5.05 \times 10 = 25.25$

$P_{7_{YMAX}} = \frac{1}{2} \times 5.46 \times 938.55 = 2562.24$

$P_{7_{YMIN}} = \frac{1}{2} \times 2.615 \times 938.55 = 1227.15$

$$\Sigma M_T = 0 = +277(7.58) + 203(5.5) - 705.3(13.1) - 3805.38(2.80) - 4698(13.2)$$
$$- 783(14.867) - 4333.22(.73) - 25.25(11.53) + 7629 - 2562.24(20.0$$
$$+ 85.99d^2(23.46 + \tfrac{2}{3}d)$$

$$0 = -137,454.06 + 2034.52d^2 + 57.327d^3$$

$$\therefore d^3 + 35.49d^2 = 2397.72$$

IF $d = 8$ $\quad 2783.36$ VS 2397.72

$d = 7.47$ $\quad 2397.21$ VS 2397.72 –

$D = 5.46 + 7.47 = \underline{12.93 \times 1.3 = 16.81 \approx 16'-10"}$ (YMAX CASE)

$$0 = -137,454.06 + 2562.24(20.02) - 1227.15(19.072)$$
$$+ 179.46d^2(20.815 + \tfrac{2}{3}d)$$

$$0 = -109,562.22 + 3735.46d^2 + 119.64d^3 \quad \therefore d^3 + 31.223d^2 = 915.77$$

IF $d = 5$ $\quad 905.58$ VS 915.77

$d = 5.03$ $\quad 917.23$ VS 915.77

$D = 2.62 + 5.03 = \underline{7.65' \times 1.3 = 9.95' \approx 10'-0}$ (YMIN CASE)

NAPA RIVER / NAPA CREEK
FLOOD CONTROL PROJECT

GENERAL STORE, SEISMIC LOAD
TIE BACKS

FOR Y_{MAX}

$$\Sigma F_H = 0 = 277 + 203 + 705.3 + 3805.38 + 4698.0 + 783 + 4333.22 + 25.25 + 2562.24 - 85.99(7.47)^2 = 11888.77^{\#/'}$$

$$T_{TOTAL} = 12 \times 11888.77 = \underline{142,665^{\#}} \qquad F = 142,665/\cos 15^\circ = \underline{147,69}$$

FOR Y_{MIN}

$$\Sigma F_H = 0 = 16687.09 - 2562.24 + 1227.15 - 179.46(5.03)^2 = 10,811.5^{\#/'}$$

$$T_{TOTAL} = 12 \times 10811.5 = \underline{129,738^{\#}} \qquad F = 129,738/\cos 15^\circ = \underline{134,315^{\#}}$$

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Description VERTICAL CAPACITY
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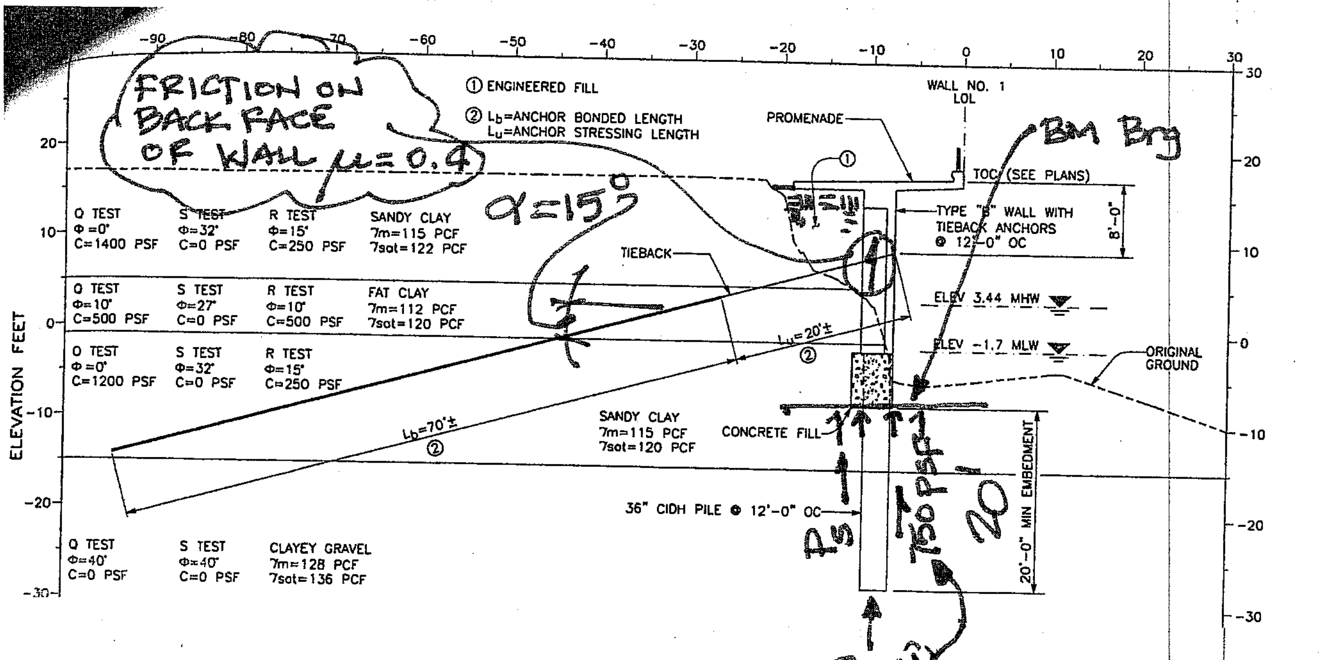
#33346 | 12/31/07

TASK: Approximate/characterize Vertical Load Capacity of Wall accounting for
- Friction Pile
- End Bearing Pile
- Brg Capacity of Grade Bm
- Tieback Forces (Vert & Horz)



P_{Tip} / Surface Friction
Allow = 175 KSF A_b

GENERAL NOTES – WORKING STRESS DESIGN

DESIGN:	AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATI (AASHTO), SECTION 5 STANDARD SPECIFICATIONS FOR HIGHWAY 15th EDITION, AND THE CALIFORNIA DEPARTMENT OF TRANSPOR
SOIL PARAMETERS:	(FOR DETERMINATION OF DESIGN LATERAL EARTH PRESSURE OI Φ=32° SANDY CLAY γm=115 PCF γsat=120 PCF
REINFORCED CONCRETE:	fy = 60 ksi (YEILD STRENGTH OF REINFORCEMENT) f'c = 3000 psi (COMPRESSIVE STRENGTH AT 28 DAYS)
PRESTRESSING STEEL:	(TIEBACKS) STRANDS – ASTM DESIGNATION A416 T = 150k DESIGN (ASD) T LOCKOFF=80k OR AS DIRECTED BY ENGINEER fpu= MINIMUM TENSILE STRENGTH OF PRESTRESSING STEEL (270 ksi) As(min) = $\frac{1.0\ T_{pu}}{0.75\ f}$

$P_D + P_L = 16^K$

Job NAPA VECP
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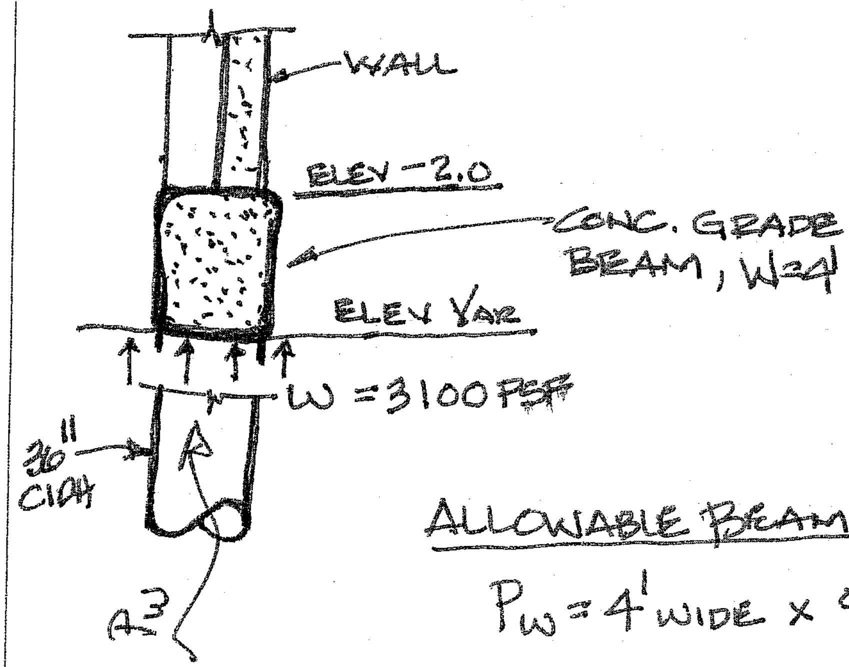

ALLOWABLE BEAM BEARING, P_W

$$P_W = 4' \text{ WIDE} \times 9' \text{ LONG} \times \frac{3100 \text{ PSF}}{1000}$$

$$P_W = 112^k$$

ALLOWABLE "TIP" BEARING, P_{TIP}

$$P_{TIP} = A_b (17.5 \text{ KSF}),\quad A_b = \frac{\pi 3^2}{4} = 7.06 \text{ SF}$$

$$= 7.06 (17.5)$$

$$P_{TIP} = 124^k$$

ALLOWABLE "FRICTION PILE", P_F

$$P_F = L D \pi \left(\frac{750 \text{ PSF}}{1000}\right),\quad L = 20',\ D = 3'$$

$$P_F = 140^k$$

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Job NAPA VECP
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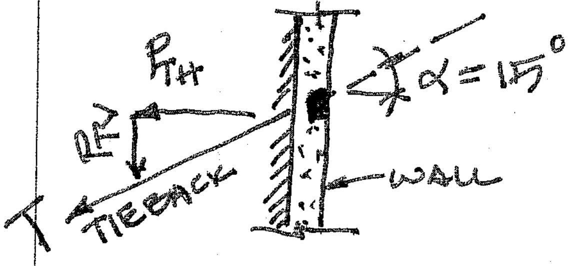

$T_{\substack{TIEBACK \\ MAX}} = 150^k$

$P_{TH} = 150^k \cos 15° = 145^k$

$P_{TV} = 150^k \sin 15° = 40^k$

<u>VERTICAL PILE "DOWNDRAG" FORCE</u>

$= P_{TV} = 40^k$

<u>FRICTION ON BACKFACE OF WALL</u>

$N = \mu N = 0.4(P_{TH}) = 0.4(145^k)$

$N = 60^k$

N IS ALWAYS > P_{TV} ; NEGLECT
$\left(FS = \frac{60}{40} = 1.5\right)$

* PER STRUCTURAL CALCULATIONS BY GARY Phillips; The Phillips GROUP DATED 3/15/06
($2383, 3/31/08)

TOTAL DEAD + LIVE = $132.6^k + 34.^k = 166.6^k$

SAY 167 KIPS

Job NAPA VRCP
Description VERTICAL CAPACITY
Project No. 0510
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LOAD COND # 1: INITIAL LOAD RESISTANCE (ILR)

THE ILR FOR VERTICAL LOADS IS THE CIDH PILES ACTING AS FRICTION PILES. AS PILE SETTLES DUE TO SETTLEMENT, PILE SURFACE FRICTION PLUS BEAM BEARING SUPPORTS WALL.

$$\text{CAPACITY} = P_F + P_W = 140^k + 112^k$$

$$= 252^k >> P_D + P_{LL} = 167^k$$

ACTIVATE ALL P_F; REMAINING IS Brg Press, w

$$w = \frac{167^k - 140^k}{(4' \times 9')} = 750 \text{ PSF}$$

NOT much Brg Pressure

LOAD COND # 2: FRICTION RESISTANCE overcome; (TIP BEARING + Bm Brg, w)

$$= P_{TIP} + \text{Bm Brg} = 112^k + 124^k = 240^k$$

$$= 240^k >> P_D + P_{LL} = 167 \quad (240/167 = 14)$$

ACTIVATE ALL P_{TIP}; remaining is Bm Brg, w

$$w = \frac{167^k - 124^k}{(4' \times 9')} = 1200 \text{ PSF}$$

Job NAPA VECP
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LOAD COND # 3: LOAD CONDITION OCCURS AFTER FULL-TIP BRG IS ACHIEVED; THEN: FULL-TIP BRG PLUS THE SURFACE FRICTION RE-ESTABLISHES @ 750PSF PLUS Bm Brg

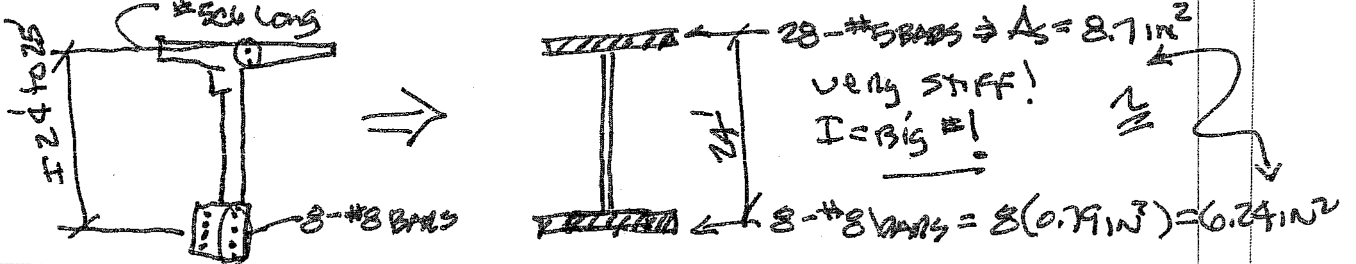
$$P_{tip} + P_F + Bm\ Brg = 112^k + 124^k + 140^k = 376^k$$
$$= 360^k \gg 167^k$$

LOAD COND # 4: BEAM BRG CAPACITY (FAILURE) OCCURS

$$P_{tip} + P_F + \overset{\to 0}{\cancel{Bm}\ Brg} = 124^k + 140^k = 264^k$$
$$264^k \gg 167^k \quad \left(FS = \frac{264}{167}\right) = \underline{1.5}$$

CONCLUSION:

No matter how you consider the proposed wall has more than adequate vertical (REAL) support. Wall acts as huge "I" girder ~~AND~~ therefore load distrib. will occur (approx/conservatively stated at $H_{wall}(8) = 8(24' - 25') \cong 200'$



Appendix E

Napa River H&H Memo For Record

CESPK-ED-D August 31, 2010

MEMORANDUM FOR RECORD

SUBJECT: Napa River Hydrology, Computed Probability Flows

1. Scope

Expected probability flows for the Napa River near Napa gage (USGS # 11458000) and locations downstream are contained in the “Napa River /Napa Creek Flood Protection Project Final Supplemental General Design Memorandum, Appendix H, Napa River Basin Hydrology for the Supplemental General Design Memorandum, “dated October 1998. The Napa River at Napa gage has a drainage area of 218 square miles and is located 5 miles north of Napa at Oak Knoll Avenue. The original hydrology was done using expected probability. This memorandum provides a full range of computed probability flows for the Napa River near Napa gage derived from the median flow frequency curve. These frequencies are 50, 20, 10, 5, 2, 1, 0.5, 0.2, and 0.1 percent. These results will be used for FDA analysis and FEMA certification. This analysis updates the flow frequency curves at the Napa River at Napa gage and select downstream locations. Locations upstream of Oak Knoll Avenue are not included in this study. Figure 1 shows the location of the relevant gages and index points. Future condition floods were not simulated because rural land use and urbanization in the Napa River Basin are not expected to change dramatically (USACE, 1998).

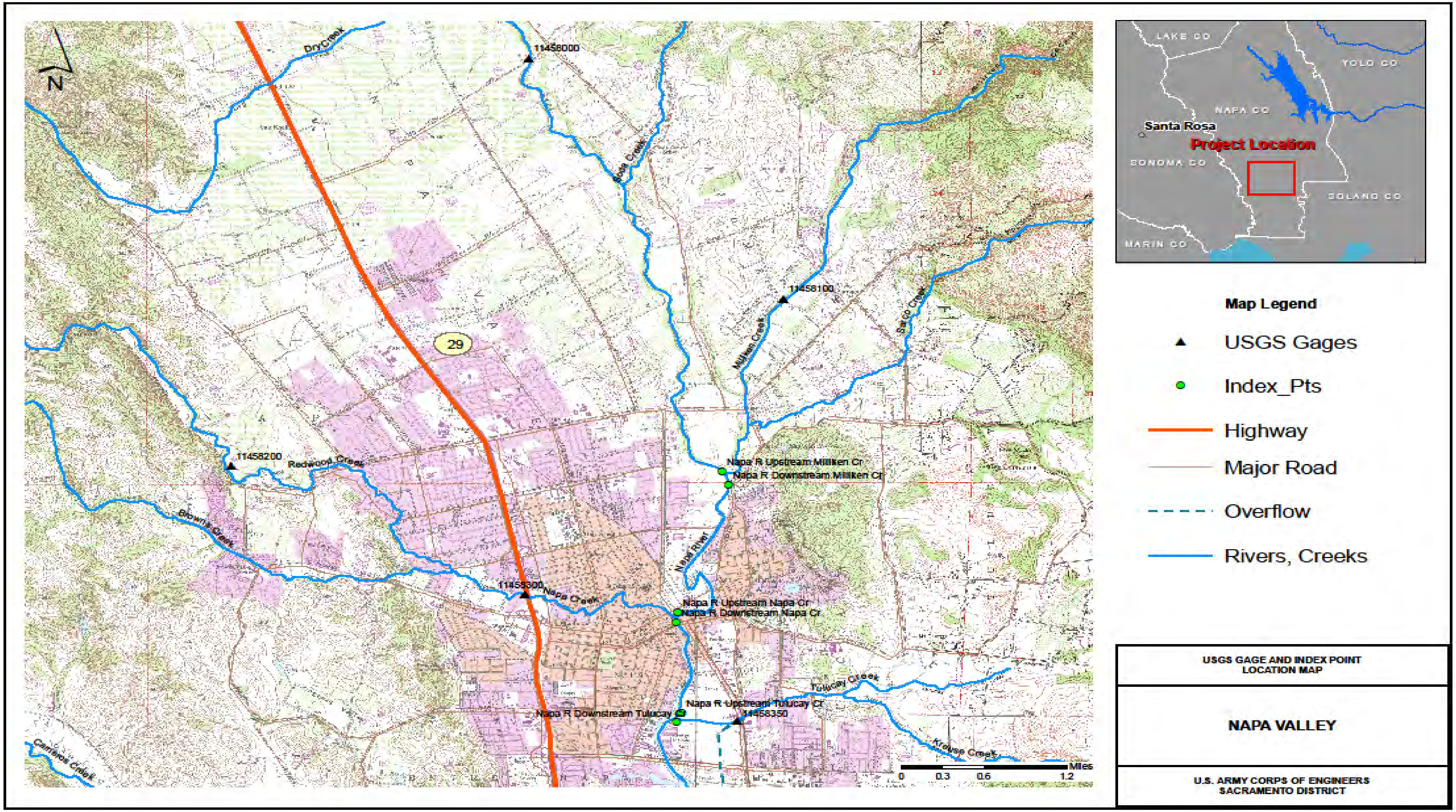


Figure 1 Study area location map showing important gages and index locations (USGS 1980).

2. Hydrologic Analysis

An unregulated peak flow frequency curve was constructed from unregulated peak flow data from USGS 11458000 Napa River near Napa (Oak Knoll) gage using the procedures in Bulletin 17B. As of Water Year 1997, 38 years (WY 1960-1997) of recorded data were available at USGS 11458000 and Conn Dam is the primary regulating influence on the flows at the Oak Knoll gage. The unregulated peak flows were obtained by routing and adding Conn Dam change in storage to the recorded flows at the Napa River near Napa gage (USACE, 1998). HEC-FFA was used to identify low outliers and the identified low outlier is from WY 1977. The period of record was extended from 38 years to 72 years by examining historical floods in the Napa River Basin and adjacent basins and by correlation with an upstream gage, Napa River at St Helena (USGS # 1145600), which has a 58 year period of record (WY 1940-1997) and a drainage area of 79 square miles. The adopted log statistics for the unregulated curve are: mean 3.989, standard deviation 0.329, and adopted skew of -0.8. HEC-REGFRQ (Regional Frequency Computation) was used in the correlation analysis.

A graphical curve was constructed for the regulated flows by fitting the curve through the regulated historical points. The present conditions curve is a combination of the regulated and unregulated curves. The unregulated and regulated curves for the Napa River near Napa (Oak Knoll) gage are shown in Figure 2 and the final present conditions curve is shown in Figure 3. The data used for the present study are from the 1998 GDM and are shown in Table 1 below.

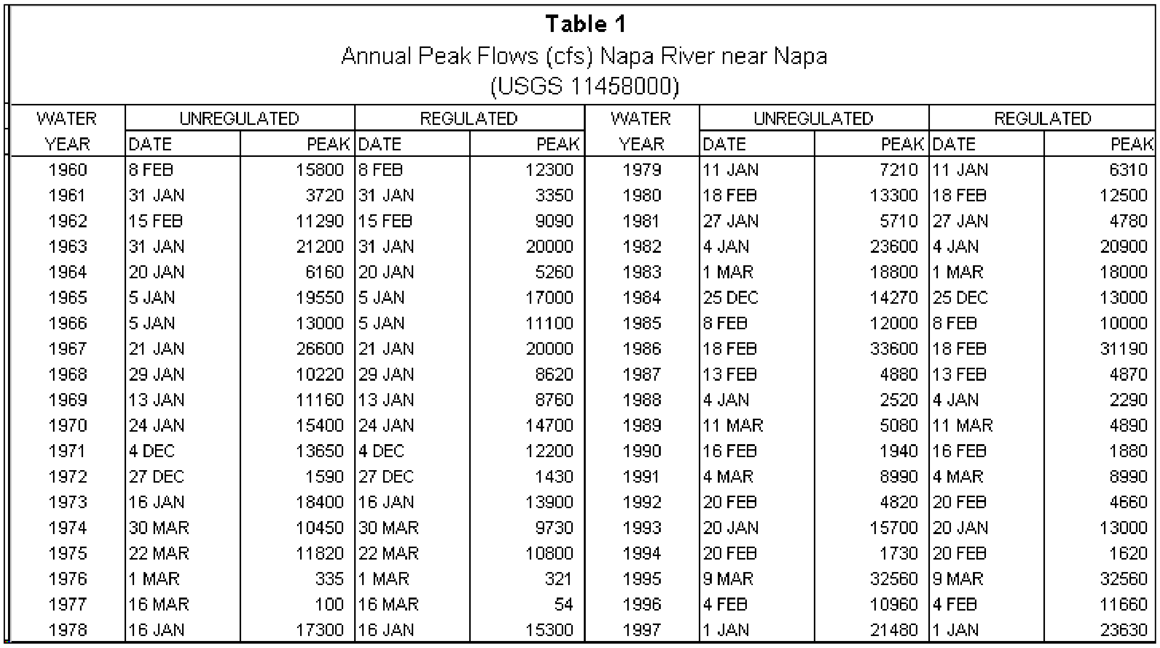


Table 1
Annual Peak Flows (cfs) Napa River near Napa
(USGS 11458000)

WATER	UNREGULATED		REGULATED		WATER	UNREGULATED		REGULATED	
YEAR	DATE	PEAK	DATE	PEAK	YEAR	DATE	PEAK	DATE	PEAK
1960	8 FEB	15800	8 FEB	12300	1979	11 JAN	7210	11 JAN	6310
1961	31 JAN	3720	31 JAN	3350	1980	18 FEB	13300	18 FEB	12500
1962	15 FEB	11290	15 FEB	9090	1981	27 JAN	5710	27 JAN	4780
1963	31 JAN	21200	31 JAN	20000	1982	4 JAN	23600	4 JAN	20900
1964	20 JAN	6160	20 JAN	5260	1983	1 MAR	18800	1 MAR	18000
1965	5 JAN	19550	5 JAN	17000	1984	25 DEC	14270	25 DEC	13000
1966	5 JAN	13000	5 JAN	11100	1985	8 FEB	12000	8 FEB	10000
1967	21 JAN	26600	21 JAN	20000	1986	18 FEB	33600	18 FEB	31190
1968	29 JAN	10220	29 JAN	8620	1987	13 FEB	4880	13 FEB	4870
1969	13 JAN	11160	13 JAN	8760	1988	4 JAN	2520	4 JAN	2290
1970	24 JAN	15400	24 JAN	14700	1989	11 MAR	5080	11 MAR	4890
1971	4 DEC	13650	4 DEC	12200	1990	16 FEB	1940	16 FEB	1880
1972	27 DEC	1590	27 DEC	1430	1991	4 MAR	8990	4 MAR	8990
1973	16 JAN	18400	16 JAN	13900	1992	20 FEB	4820	20 FEB	4660
1974	30 MAR	10450	30 MAR	9730	1993	20 JAN	15700	20 JAN	13000
1975	22 MAR	11820	22 MAR	10800	1994	20 FEB	1730	20 FEB	1620
1976	1 MAR	335	1 MAR	321	1995	9 MAR	32560	9 MAR	32560
1977	16 MAR	100	16 MAR	54	1996	4 FEB	10960	4 FEB	11660
1978	16 JAN	17300	16 JAN	15300	1997	1 JAN	21480	1 JAN	23630

Flows with exceedance frequencies greater than 1 % chance exceedance are from the regulated curve. At about 1 % chance exceedance, the upstream regulation ceases to have an effect on the flows. Thus the flows at frequencies less than or equal to 1% chance exceedance are from the unregulated curve. None of the measured flows at the

Napa River near Napa gage reached the threshold value of 36,500 cfs (1%) where regulated flows equal unregulated flows. As a result all recorded gage data are considered to be regulated flows. Flows for all exceedance intervals are shown in Table 2 below.

Table 2
Napa River near Napa
USGS 11458000

Exceedance Frequency per 100 Years	Flows (cfs)
80	5,000
50	9,900
20	17,200
10	22,200
5.0	26,800
2.0	32,600
1.0	36,500
0.5	39,600
0.2	43,200
0.1	45,600

The Napa River flood hydrographs for each exceedance interval were computed by multiplying the existing Standard Project Flood (SPF) hydrographs by ratios determined from the Napa River frequency curves (USACE 1975, USACE 1998). The ratios were determined by dividing the given exceedance peak flow by the peak of the SPF. For example, the 1% chance exceedance flow is 36,500 cfs, which is 0.802 times the SPF of 45,500 cfs. The adopted Napa River 50-, 20-, 10-, 2-, 1-, 0.5-, 0.2-, and 0.1-percent chance exceedance ratios are: 0.218, 0.378, 0.488, 0.716, 0.802, 0.870, 0.949 and 1.002 respectively. The drainage areas of Soda, Milliken, Napa and Tulucay Creeks are: 15.5, 17.3, 14.9 and 12.6 sqare miles respectively. The flood hydrographs for the local creeks through the project area below Oak Knoll were obtained by ratios derived from the Napa Creek frequency curve. The adopted Napa Creek 50-, 20-, 10-, 2-, 1-, 0.5-, 0.2-, and 0.1-percent chance exceedance ratios are: 0.380, 0.492, 0.562, 0.713, 0.775, 0.832, 0.922, 0.995, respectively. The frequency curve for Napa Creek at Napa River is shown in Figure 4. The original curve was constructed using data from the Napa Creek at Napa gage (USGS# 11458300) and values estimated by correlation with Redwood Creek near Napa gage (USGS# 11458200). This frequency curve was extended from the original graphical curve in the 1998 GDM using regression and graphical methods. Linear regression was used on the upper end of the data to get an approximate trend then the curve is extended graphically. The Napa Creek ratios were used for local concurrent flows from Soda Creek, Milliken Creek and the local flow into the Napa River. An HMS model of Tulucay Creek was used to determine peak flows in that basin (see Sept 1 Addendum).

Two HEC-1 models are used in this study: a rainfall runoff model for Soda, Milliken and Napa Creeks and a routing model for the main stem of the Napa River. The rainfall runoff model uses Kinematic wave unit hydrographs with a 0.75-inch initial loss and a constant loss rate of 0.1 inches per hour. The precipitation pattern is that of the Standard Project Flood (SPF). The SPF for the Napa River Valley is the December 1964 storm over Laytonville, California, artificially centered over the Napa River Basin with wet ground conditions (initial loss of 0.2 inches and final loss rate of 0.1 inches per hour) as was done in USACE 1998 and USACE 1975. The routing model uses the Modified Puls method and routing parameters are the same as in the 1998 GDM (USACE 1998 and USACE 1975).

3. Recent Data

Peak flow data from the Napa River near Napa gage from water years 1998 through 2006 are shown in Table 3, below. The data appear to be randomly distributed. There is not enough evidence at this time to justify revising the flow frequency curves at the Napa River near Napa gage.

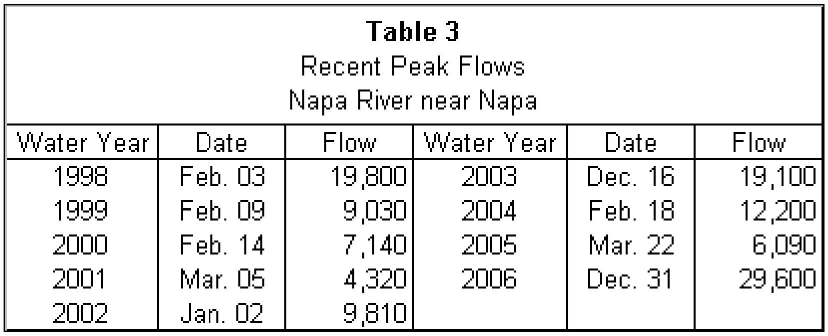


Table 3
Recent Peak Flows
Napa River near Napa

Water Year	Date	Flow	Water Year	Date	Flow
1998	Feb. 03	19,800	2003	Dec. 16	19,100
1999	Feb. 09	9,030	2004	Feb. 18	12,200
2000	Feb. 14	7,140	2005	Mar. 22	6,090
2001	Mar. 05	4,320	2006	Dec. 31	29,600
2002	Jan. 02	9,810			

4. Results

Peak flows in the Napa River with concurrent flows in Milliken, Napa and Tulucay Creeks are shown in Tables 4, 5 and 6. Tables 7, 8 and 9 show the peak flows in Milliken, Napa and Tulucay Creeks with the concurrent flows in the Napa River. Soda Creek is not included in this analysis. These tables follow the same format as the 1998 GDM and can be used to estimate concurrent Napa River flow for nonuniform storms over the Napa River Basin. For example, if a 10 year flood strikes the Napa River Basin and a 100 year flood strikes the Napa Creek Basin, then the concurrent flow downstream of Napa Creek is estimated to be 23,710 cfs (19,430 + 4,280 = 23,710). The tables are for the 50-, 20-, 10-, 2-, 1-, 0.5-, 0,2- and 0.1-percent chance exceedance floods and reflect existing conditions. For example, Table 4 shows that the 1% chance exceedance floods in the Napa River upstream of Milliken Creek is 37,500 cfs and the concurrent flows in Milliken Creek and in the Napa River downstream of Milliken Creek at the time of the peak upstream are 1,570 cfs and 39,400 cfs, respectively.

For the Napa River upstream of Napa Creek shown in Table 5, the 1% chance exceedance flow is 40,100 cfs and the concurrent flows in Napa Creek and in Napa River downstream of Napa Creek (at the time of the peak upstream) are 2,600 cfs and 42,700 cfs, respectively.

In the Napa River above Tulucay Creek, shown in Table 6, the 1% chance exceedance flow is 42,400 cfs, while the concurrent flows in Tulucay Creek and in Napa River below Tulucay Creek are 1660 cfs and 44,400 cfs, respectively

Peak flows in Milliken, Napa, and Tulucay Creek are shown in Tables 7, 8, and 9. These tables follow the same format as in the 1998 GDM. For example, in Table 7, Milliken Creek at the Napa River, the 1% chance exceedance peak flow is 4,900 cfs and the concurrent flows in the Napa River upstream and downstream of Milliken Creek are 27,000 cfs and 32,700 cfs, respectively.

In Napa Creek, at the Napa River, shown in Table 8, the 1% chance exceedance peak flow is 4,280 cfs and the concurrent flows in the Napa River upstream and downstream are 31,700 cfs and 36,000 cfs, respectively.

In Tulucay Creek at the Napa River, shown in Table 9, the 1% chance exceedance peak flow is 4530 cfs and the concurrent flows in the Napa River upstream and downstream are 33,100 cfs and 38,400 cfs, respectively. The index location "Local above Tulucay Creek" refers to a small creek that enters the Napa River approximately ½ mile upstream from the mouth of Tulucay Creek. Figure 5 contains peak flow frequency curves for the Napa River upstream of Milliken, Napa and Tulucay Creeks.

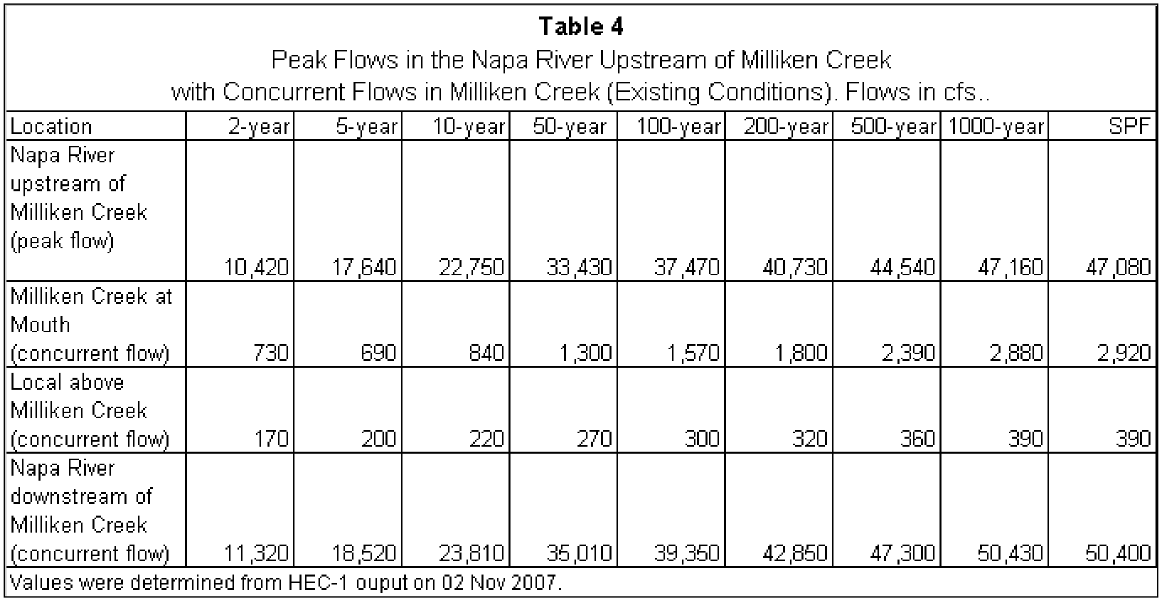


Table 4

Peak Flows in the Napa River Upstream of Milliken Creek
with Concurrent Flows in Milliken Creek (Existing Conditions). Flows in cfs..

Location	2-year	5-year	10-year	50-year	100-year	200-year	500-year	1000-year	SPF
Napa River upstream of Milliken Creek (peak flow)	10,420	17,640	22,750	33,430	37,470	40,730	44,540	47,160	47,080
Milliken Creek at Mouth (concurrent flow)	730	690	840	1,300	1,570	1,800	2,390	2,880	2,920
Local above Milliken Creek (concurrent flow)	170	200	220	270	300	320	360	390	390
Napa River downstream of Milliken Creek (concurrent flow)	11,320	18,520	23,810	35,010	39,350	42,850	47,300	50,430	50,400

Values were determined from HEC-1 ouput on 02 Nov 2007.

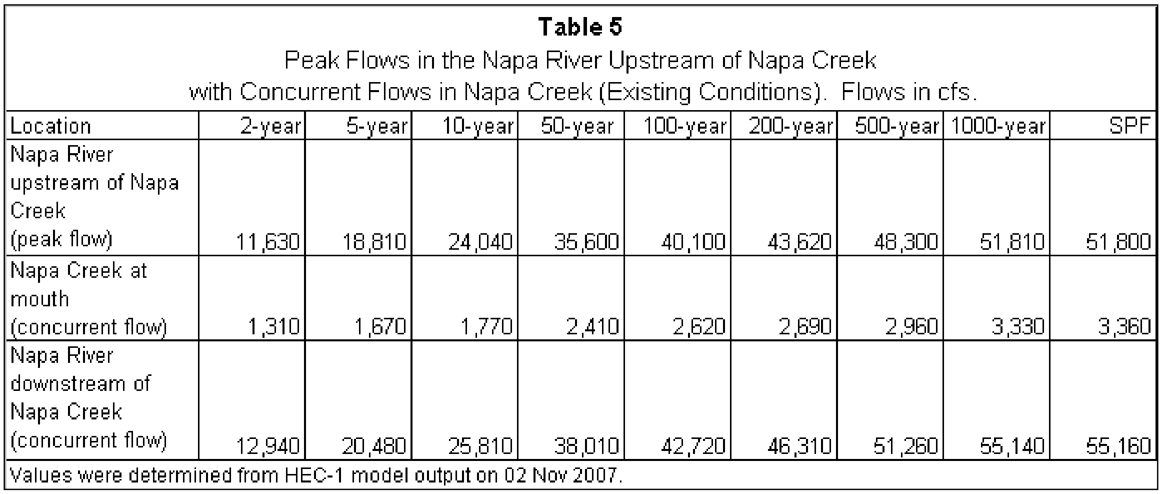


Table 5

Peak Flows in the Napa River Upstream of Napa Creek
with Concurrent Flows in Napa Creek (Existing Conditions). Flows in cfs.

Location	2-year	5-year	10-year	50-year	100-year	200-year	500-year	1000-year	SPF
Napa River upstream of Napa Creek (peak flow)	11,630	18,810	24,040	35,600	40,100	43,620	48,300	51,810	51,800
Napa Creek at mouth (concurrent flow)	1,310	1,670	1,770	2,410	2,620	2,690	2,960	3,330	3,360
Napa River downstream of Napa Creek (concurrent flow)	12,940	20,480	25,810	38,010	42,720	46,310	51,260	55,140	55,160

Values were determined from HEC-1 model output on 02 Nov 2007.

Table 6

Peak flows in the Napa River, upstream of Tulucay Creek
with concurrent flows in Tulucay Creek (existing conditions). Flows in cfs.

Location	2-year	5-year	10-year	50-year	100-year	200-year	500-year	1000-year
Napa River upstream of Tulucay Creek (peak flow)	12,900	20,270	25,650	37,610	42,410	46,110	51,060	54,770
Tulucay Creek at mouth (concurrent flow)	510	710	970	1,300	1,660	1,890	2,180	2,400
Local above Tulucay Creek (concurrent flow)	170	190	210	260	300	320	350	380
Napa River Downstream of Tulucay Creek (concurrent flow)	13,580	21,170	26,830	39,170	44,370	48,310	53,590	57,550

Values were determined from HMS and HEC-1 model outputs on 30 Aug 2010.

Table 7

Peak Flows in Milliken Creek
with Concurrent Flows in the Napa River (Existing Conditions). Flows in cfs.

Location	2-year	5-year	10-year	50-year	100-year	200-year	500-year	1000-year	SPF
Napa River upstream of Milliken Creek (concurrent flow)	8,190	13,070	16,200	23,710	26,950	29,370	32,470	34,660	34,630
Milliken Creek at Mouth (peak flow)	1,730	2,390	2,890	4,220	4,910	5,610	7,010	8,390	8,490
Local above Milliken Creek (concurrent flow)	430	550	630	800	870	930	1,030	1,110	1,110
Napa River downstream of Milliken Creek (concurrent flow)	10,360	16,000	19,730	28,730	32,730	35,910	40,510	44,160	44,230

Values were determined from HEC-1 output on 02 Nov 2007.

Table 8

Peak flows in Napa Creek
with concurrent flows in Napa River (existing conditions). Flows in cfs.

Location	2-year	5-year	10-year	50-year	100-year	200-year	500-year	1000-year	SPF
Napa River upstream of Napa Creek (concurrent flow)	10,280	15,910	19,430	28,170	31,660	34,960	39,610	42,780	42,850
Napa Creek at mouth (peak flow)	2,120	2,720	3,110	3,950	4,280	4,580	5,090	5,500	5,530
Napa River downstream of Napa Creek (concurrent flow)	12,400	18,630	22,540	32,110	35,950	39,540	44,700	48,280	48,370

Values were determined from HEC-1 model output on 6 Nov 2007.

Table 9

Peak flows in Tulucay Creek
with concurrent flows in the Napa River (existing conditions). Flows in cfs.

Location	2-year	5-year	10-year	50-year	100-year	200-year	500-year	1000-year
Napa River upstream of Tulucay Creek (concurrent flow)	11,720	17,760	21,010	29,360	33,130	36,600	41,600	45,580
Tulucay Creek at mouth (peak flow)	1,080	1,890	2,880	3,890	4,530	5,160	6,000	6,660
Local above Tulucay Creek (concurrent flow)	360	460	520	660	720	770	850	920
Napa River Downstream of Tulucay Creek (concurrent flow)	13,160	20,110	24,410	33,920	38,370	42,530	48,450	53,160

Values were determined from HMS and HEC-1 model outputs on 30 Aug 2010.

5. Conclusions

A full range of computed probability flows has been developed for the Napa River near Napa (Oak Knoll) gage. Flow hydrographs at the Napa River near Napa Gage were routed from Oak Knoll Avenue (location of Napa River near Napa gage) to Soda, Milliken, Napa and Tulucay Creeks using HEC-1. Flows in Soda, Milliken and Napa Creeks were routed to the Napa River using the HEC-1 rainfall runoff model. There is not enough evidence at this time to justify revising the flow frequency curves at the Oak Knoll gage. The routed flow hydrographs can be used for flood damage analysis (FDA) and risk-based analysis (RBA) for FEMA certification.

6. References:

1. U.S. Army Corps of Engineers, Sacramento District, “Napa River/Napa Creek Flood Protection Project, Final Supplemental General Design Memorandum Volume II Appendix H: Napa River Basin Hydrology for the Supplemental General Design Memorandum,” October 1998.

2. U.S. Army Corps of Engineers, the Hydrologic Engineering Center, HEC-FFA Flood Frequency Analysis, version 3.1, February 1995.

3. U.S. Army Corps of Engineers, the Hydrologic Engineering Center, HEC-REGFRQ Regional Frequency Computation, version dated September 8, 1989.

4. U.S. Geological Survey, http://waterdata.usgs.gov/ca/nwis, National Water Information System Web Interface, Daily Streamflow for California (accessed September 24, 2007).

5. U.S. Army Corps of Engineers, San Francisco District, “Final General Design Memorandum and Environmental Impact Statement,“ Napa River Flood Control Project, Napa County, California, September 1975.

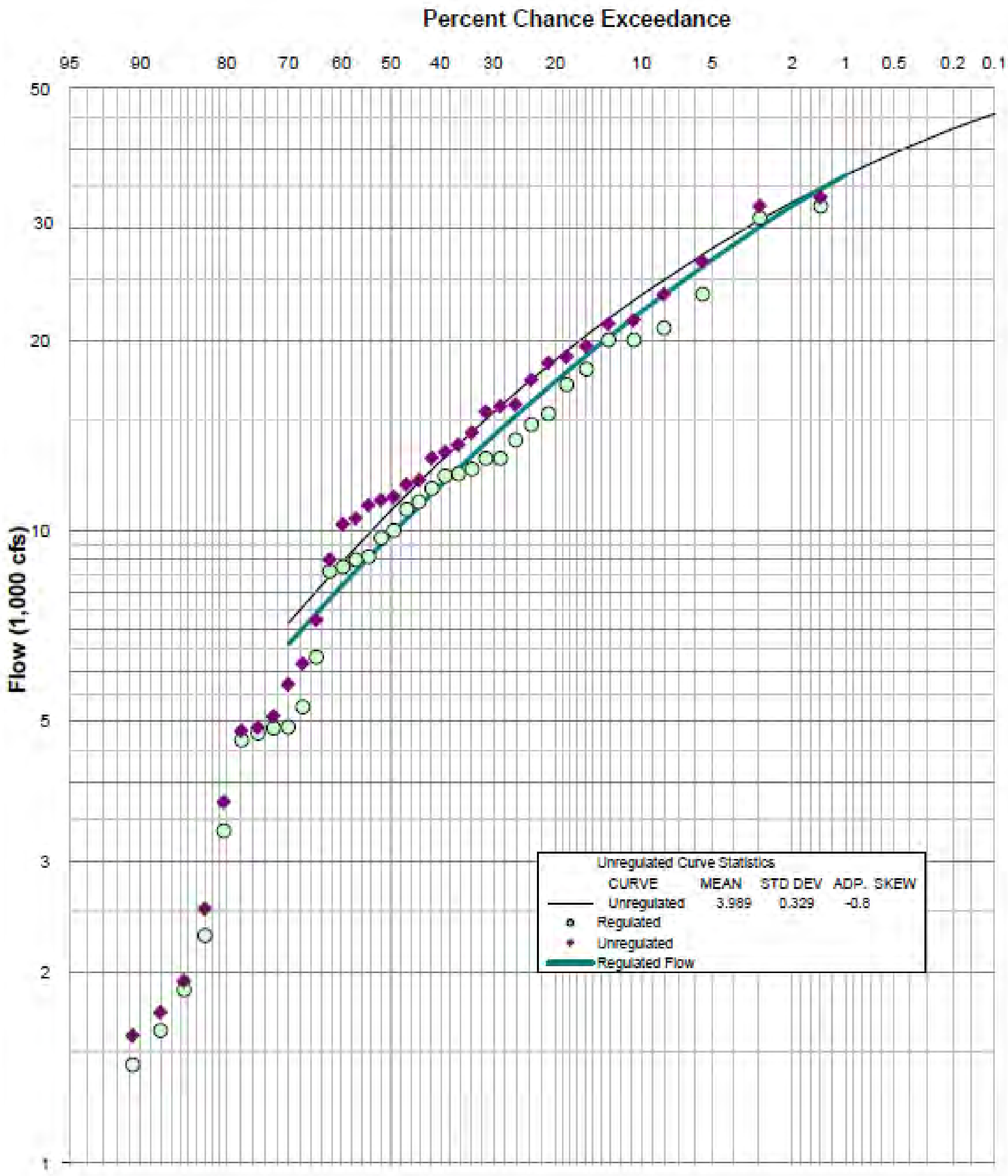
6. U.S. Army Corps of Engineers, the Hydrologic Engineering Center, HEC-1 Flood Hydrograph Package, version 4.1, September 1990.

7. U.S. Army Corps of Engineers, Sacramento District, Memorandum for Record: Napa Creek Hydrologic and Hydraulic Analysis of Historic Events, September 8, 2006.

8. U.S. Geological Survey, “Guidelines for Determining Flood Flow Frequency: Bulletin 17 B of the Hydrologic Subcommittee,” revised September 1981.

9. U.S. Geological Survey and State of California Department of Water Resources, Napa 7.5 Minute Topographic Quadrangle: 1:24,000, dated 1951, photorevised 1980.

William Curry
Hydrologist
CESPK-ED-DW



NOTES:

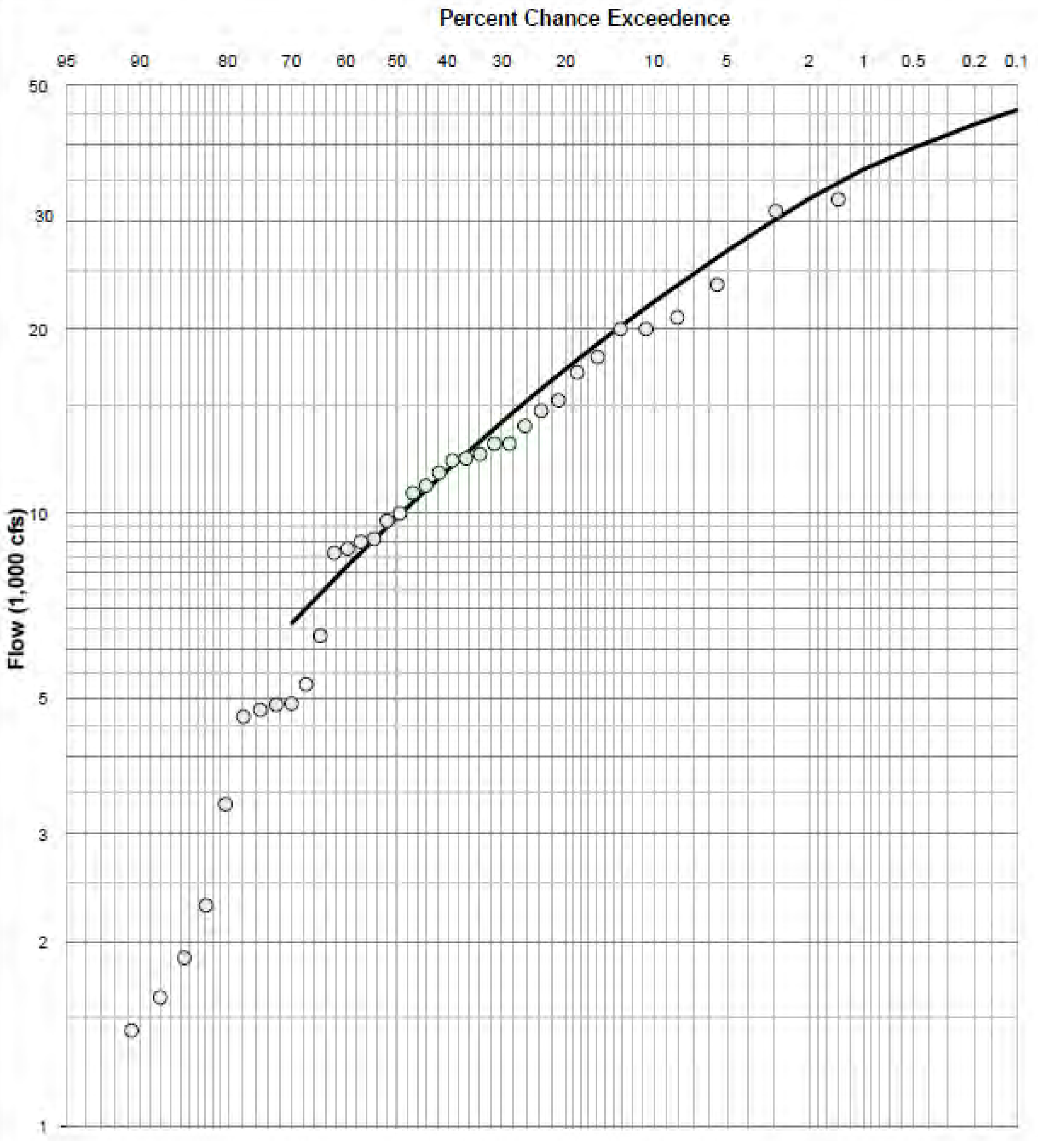
1. The unregulated curve is a Log Pearson Type III curve based on 72 years of record with median plotting postions.
2. The systematic record of 38 years has been extended to 72 years by examination of historical floods in the region and by correlation with the Napa River near St Helena gage.
3. The mean and standard deviation are adjusted to the longer record of the Napa River near St Helena gage.
4. The regulated curve is a graphical curve which was drawn through the regulated peak flows and extended using the unregulated curve.
5. Drainage area: 218 Sq Mi
6. Data are from the 1998 General Design Memorandum.

NAPA RIVER FLOOD PROTECTION PROJECT
NAPA RIVER BASIN, CALIFORNIA

REGULATED AND UNREGULATED
PEAK FLOW FREQUENCY CURVES
PRESENT CONDITIONS
NAPA RIVER NEAR NAPA
(SAME AS NAPA RIVER AT OAK KNOLL)

U.S ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Figure 2. Unregulated and Regulated Flow Frequency Curves for the Napa River near Napa (Oak Knoll) Gage (USGS 11458000) present conditions.



NOTES:

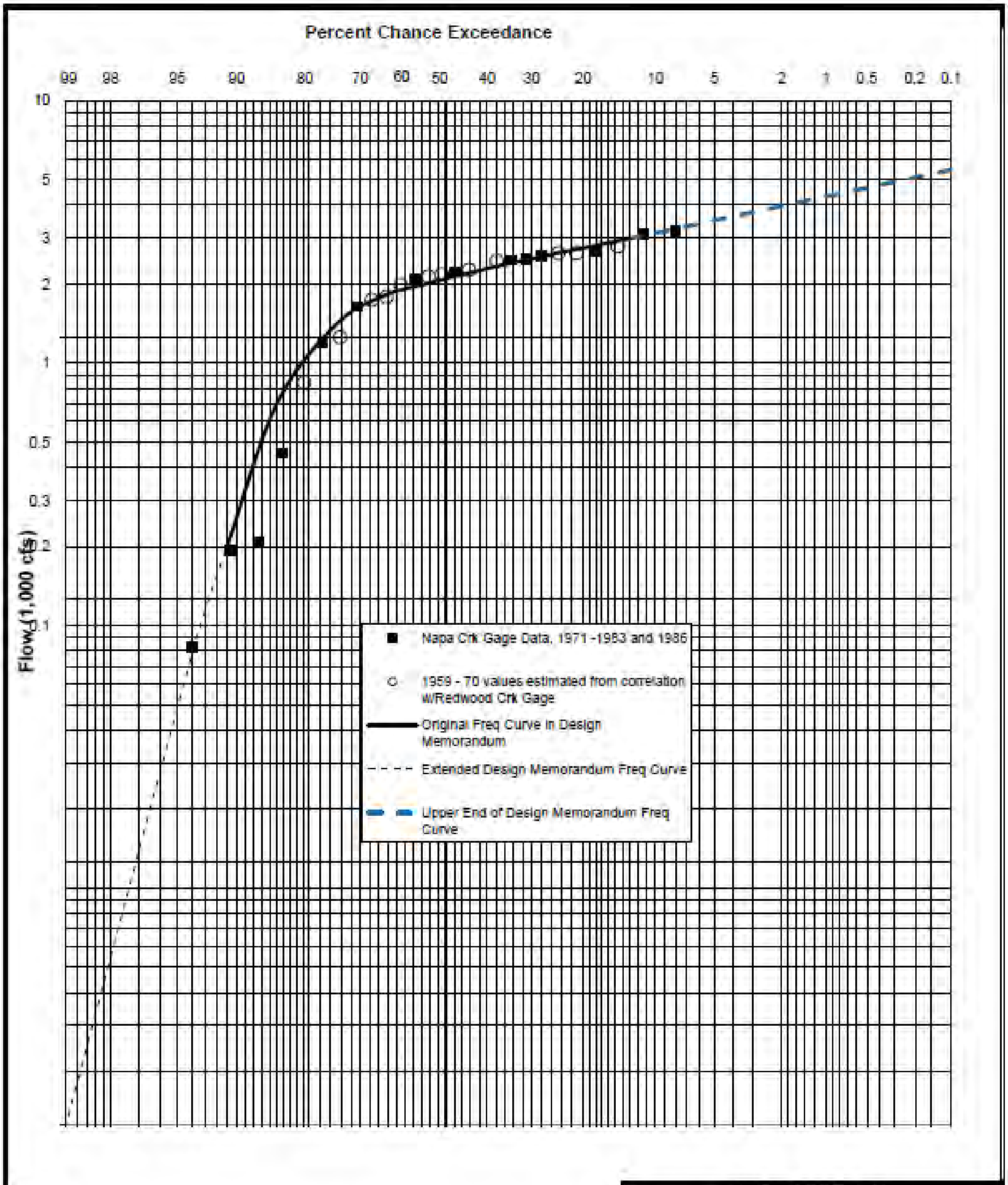
1. The regulated curve is a graphical curve which was drawn through the regulated peak flows and extended using the unregulated curve.
2. Data are from the 1998 General Design Memorandum.
3. Drainage area: 218 Sq Mi.

NAPA RIVER FLOOD PROTECTION PROJECT
NAPA RIVER BASIN, CALIFORNIA

PRESENT CONDITIONS
REGULATED
PEAK FLOW FREQUENCY CURVE
NAPA RIVER NEAR NAPA

U.S ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Figure 3. Present Conditions Regulated Peak Flow Frequency Curve for the Napa River near Napa Gage.



Notes:

1. Lower end of frequency curve was drawn graphically.
2. Upper end of frequency curve was extended using regression and graphical methods.
3. Original 1989 frequency curve was drawn graphically
4. Drainage Area = 14.9 square miles
5. Weibull Plotting positions.

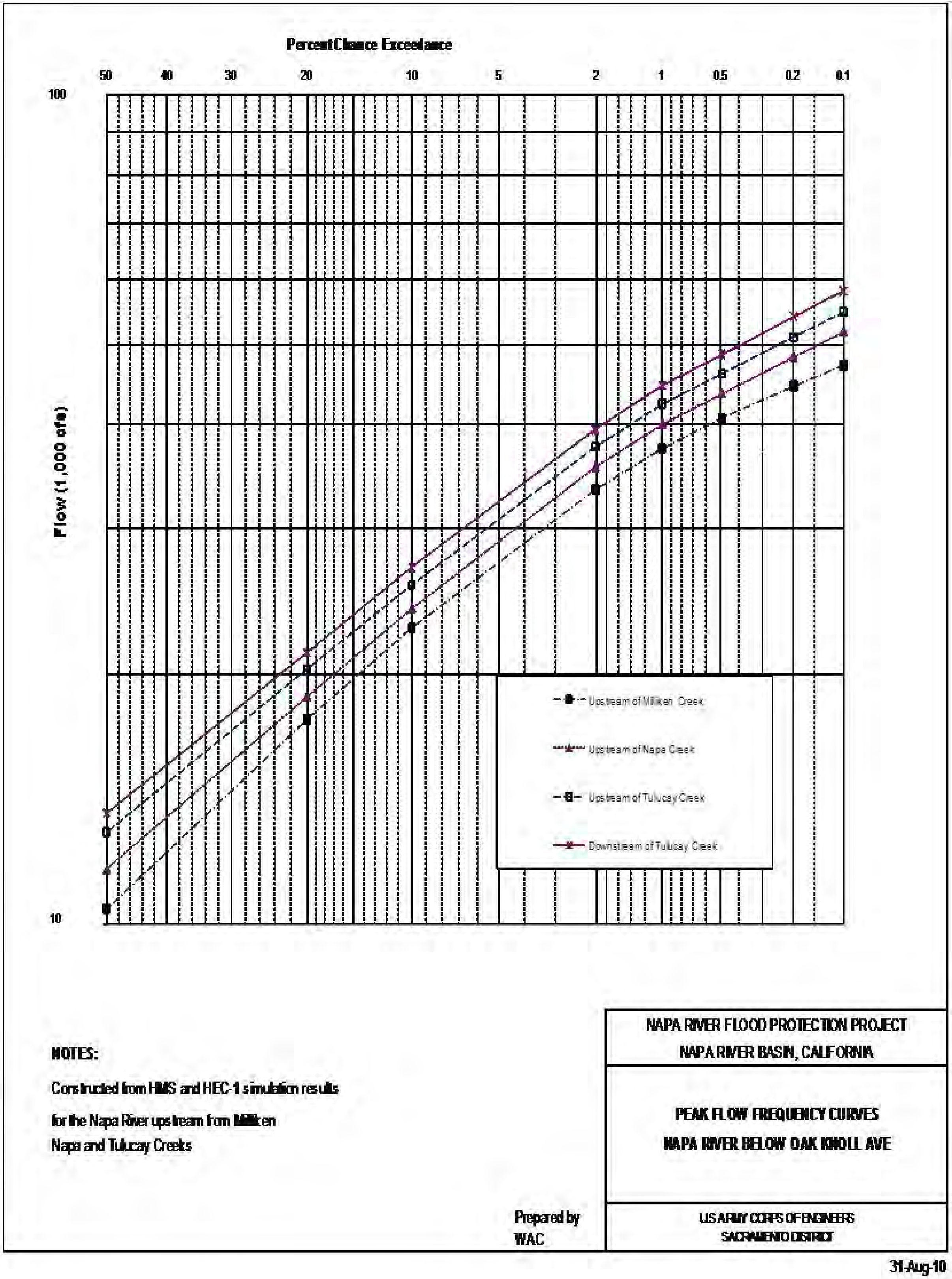
May 2006
JMH and WAC

NAPA CREEK, CA
NAPA CREEK AT NAPA

PEAK FLOW FREQUENCY
EXTENDED GRAPHICAL CURVE

U.S ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Figure 4. Napa Creek at Napa River Peak Flow Frequency Curve adapted from the “Napa River/Napa Creek Final Supplemental General Design Memorandum, Appendix H, Hydrology Office Report” (This curve was determined by graphical methods.) (USACE 1998)



NOTES:

Constructed from HMS and HEC-1 simulation results
for the Napa River upstream from Milliken
Napa and Tulucay Creeks

NAPA RIVER FLOOD PROTECTION PROJECT
NAPA RIVER BASIN, CALIFORNIA

PEAK FLOW FREQUENCY CURVES
NAPA RIVER BELOW OAK KNOLL AVE

Prepared by
WAC

US ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

31-Aug-10

Figure 5. Frequency Curves for the Napa River Upstream of Milliken, Napa and Tulucay Creeks

January 12, 2010 Addendum

Scope of Addendum

Additional work was requested by the Hydraulic Design Section in FY 2009 to prepare the Economic Evaluation of the Project and the Limited Reevaluation Report. These requests included 1) verification of the methods for computing the flow frequency curves and description of the lower end of the curves from 60% to 99.99% probability; and 2) obtaining flows at different frequencies for Risk Based Analysis. This addendum to the November 2007 Napa River Hydrology, Computed Probability Flows Memorandum for Record, was completed in January 12, 2010. The methods for computing the mean flow frequency curves were checked and verified. Additional work was done to describe the lower end of the curves for flows from 0.999 to 0.600 exceedance probabilities for use in the risk analysis for the project's economic evaluation. In addition, flows were needed at different frequencies for greater definition of the frequency curves used for the risk analysis. These flows were estimated by extending the frequency curves, graphically based on the heavily regulated flows of the Napa River near Napa gage and interpolating between the flow frequency values in this report. A brief write-up and the present conditions Flow frequency Curves are added as an addendum to this memo.

Frequency Data Check and Tables Expanded.

Flows used in previous reports cited used expected probability and computed probably frequency curves. The scope of the first request was to make sure the flows used in the new risk based analysis reflected mean flows and computed frequencies at their required exceedance probability at each of the five locations sited in the request. The locations are: upstream of Milliken, Napa and Tulucay Creeks and downstream of Milliken and Tulucay Creeks. It was determined that the flow and exceedance values found in the Napa River Hydrology, Computed Probability Flows Memorandum, dated November 21, 2007 were the correct values to use for Risk Analysis.

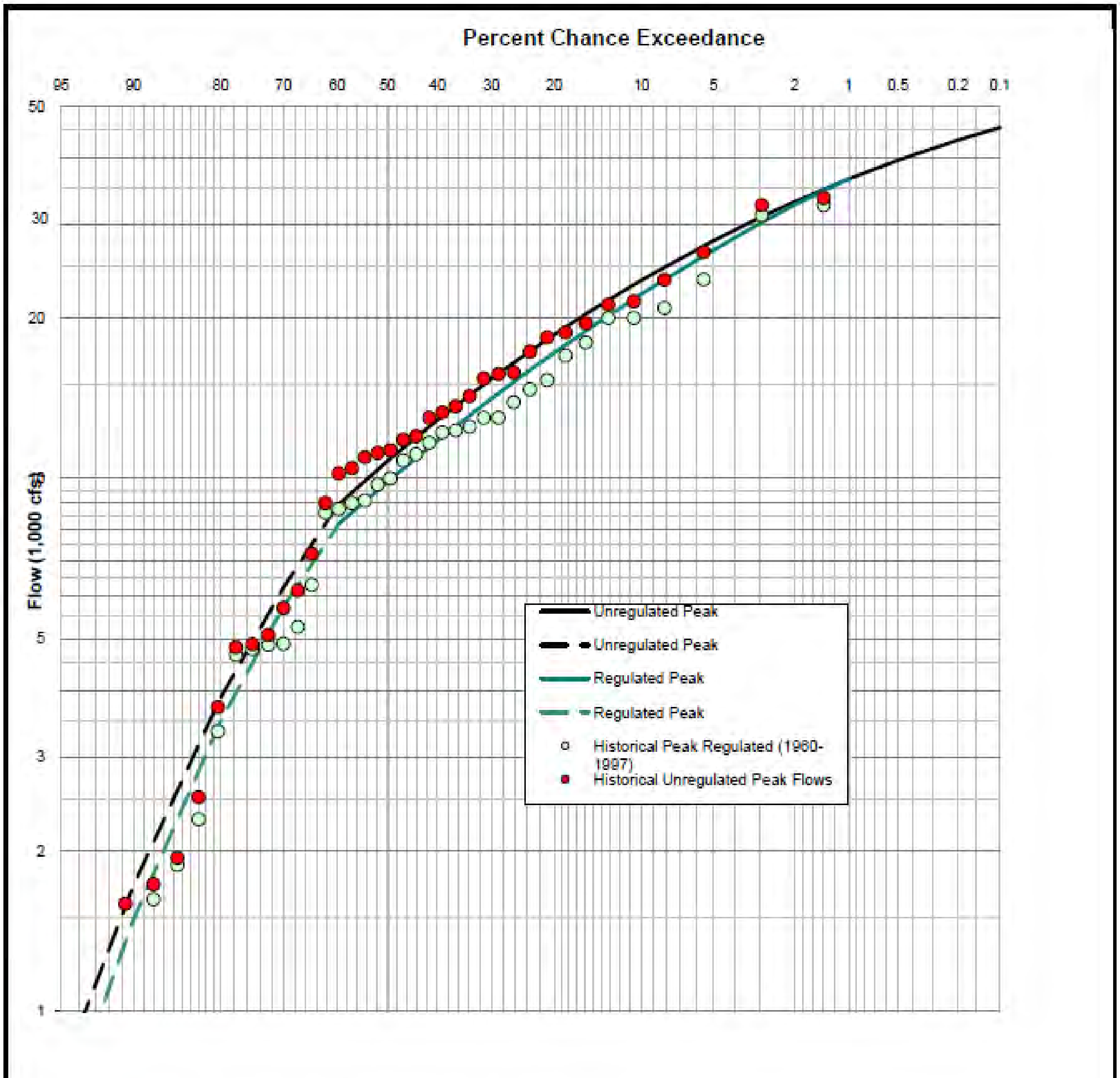
Additional work was done to describe the lower end of the curves for flows from 0.999 to 0.600 exceedance probabilities for use in the risk analysis for the project's economic evaluation. In addition to this, additional flows were needed at different frequencies for greater definition of the frequency curves used for the risk analysis. These flows were estimated by extending the frequency curves, graphically based the heavily regulated flows of Napa River near Napa gage and interpolating between this report's flow frequency values.

Table 10 lists the unregulated computed probability curve, and the regulated graphical frequency curve and their probabilities as plotted in Figure 6.

Table 10
Napa River near Napa
USGS 11458000

Exceedance Probability	Unregulated Flow (cfs)	Regulated Flow (cfs)
0.990	112	75
0.980	257	188
0.950	763	618
0.900	1,720	1,480
0.800	3,870	3,500
0.700	6,240	5,740
0.600	8,900	8,130
0.500	10,800	9,860
0.400	12,900	11,800
0.300	15,400	14,100
0.250	16,900	15,500
0.200	18,600	17,200
0.150	20,700	19,300
0.100	23,600	22,200
0.050	27,900	26,800
0.030	30,900	30,100
0.020	33,100	32,600
0.010	36,500	36,500
0.005	39,600	39,600
0.004	40,500	40,500
0.002	43,200	43,200
0.001	45,900	45,600

Notes:
1. Unregulated flow reflects the removal of Conn Dam (Hennessey Reservoir) the only reservoir that would significantly reduce peak flow in the Napa River at Napa.
2. It was assumed that antecedent conditions would fill and cause Conn Dam to be spilling for events equal to or greater than the 1% flood.
3. Curves plotted in Figure 6 of this addendum.



NOTES:

1. Notes 2-5 only apply to the portion of the curve extending from 60 percent chance exceendance to 0.1 percent chance exceedance
2. The unregulated curve is a Log Pearson Type III curve based on 72 years of record with median plotting postions
3. The systematic record of 38 years has been extended to 72 years by examination of historical floods in the region and by correlation with the Napa River near St Helena gage.
4. The mean and standard deviation are adjusted to the longer record of the Napa River near St Helena gage
5. The regulated curve is a graphical curve which was drawn through the regulated peak flows and extended using the unregulated curve.
6. Booth Curves were extended graphicaly fron 60 to 99 percent chance exceedence using regulated Peak Flow.
7. Data are from the 1998 General Design Memorandum

NAPA RIVER
NAPA RIVER BASIN, CALIFORNIA

REGULATED AND UNREGULATED
PEAK FLOW FREQUENCY CURVES
NAPA RIVER AT OAK KNOLL AVENUE
(SAME AS NAPA RIVER AT NAPA)
USGS # 11458000

U.S ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

RFC 1/4/2010

Figure 6. Re-plotted Figure2 frequency curves for the unregulated and regulated flow for the Napa River near Napa (Oak Knoll) Gage (USGS 11458000) extending the curves from 0.60 to 0.99 exceedance frequency. The LPIII analysis and extension of the period of record pertain only to the portion of the unregulated curve extending from 60 to 0.1 percent chance exceedance.

The second request was to compute additional flood flows for risk analysis based on the shaded flows and probabilities found in Table 11. Shaded data came from the 2007 Memorandum. Curves requested were not ordered in any particular manner so that data is also annotated by station name and location based on tables in the 2007 memo and the hydraulic design section's station numbering system. The frequency curves were drawn and plotted in Figures 6 and 7. Estimated flow values were obtained for frequencies of 0.3, 0.4, 0.005, and 0.004 exceedance probabilities and added to Table 10. Exceedance probability of 0.005 was added because of California's new mandate to know the 0.5% flood peak (200 year) flood.

The legends in those figures name the curves in their plotting order. Figure 7 is Figure 5 replotted, Frequency Curves for the Napa River Upstream of Milliken, Napa and Tulucay Creeks, downstream of Milliken Creek and downstream of Tulucay Creek. Figure 8 is the same as Figure 7 which includes all locations found in Figure 5 and expands the Exceedance Probability axis scale from 0.99 to 0.001 probabilities.

Table 11
Exceedance Probabilities For
Napa River Below Napa River at Oak Knoll Avenue
(Napa River at Napa, California)

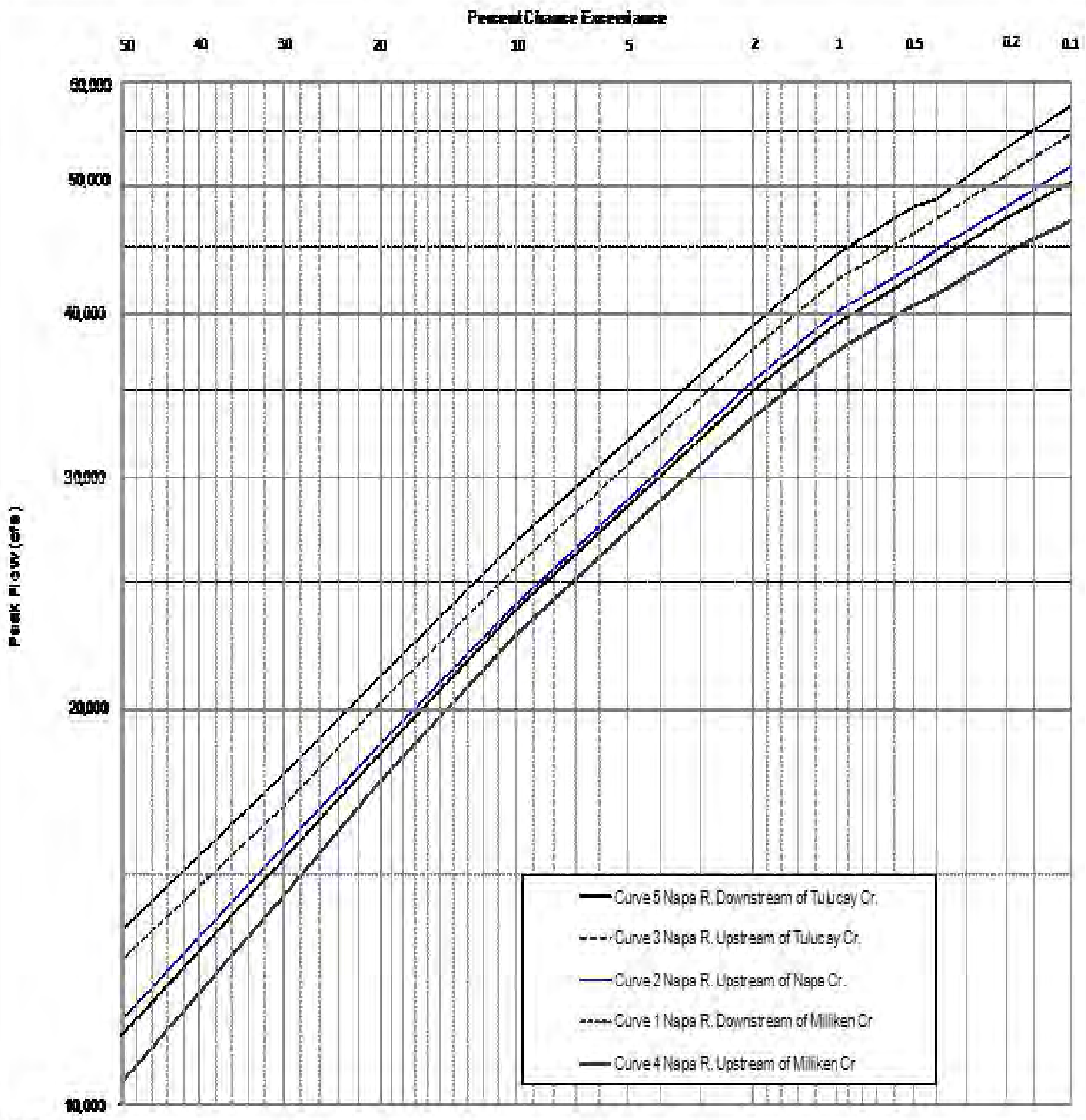
Exceedance Probability	Discharge (cfs)				
	Curve 4 Napa River upstream of Milliken Cr. Table 4	Curve1 Napa River Downstream of Milliken Cr.	Curve 2 Napa River upstream of Napa Cr Table 5	Curve 3 Napa River Upstream of Tulucay Cr	Curve 5 Napa River downstream of Tulucay Cr Table 6
0.999	70	80	85	90	110
0.990	98	107	111	127	144
0.950	714	783	810	930	1029
0.900	1,660	1,819	1,880	2,162	2366
0.800	3,840	4,210	4,360	5,010	5411
0.700	6,290	6,900	7,140	8,200	8811
0.650	7,610	8,340	8,630	9,910	10663
0.600	9,100	9,830	10,180	11,250	11990
0.500	10,420	11,300	11,600	12,900	13580
0.300	14,400	15,380	15,700	16,870	17828
0.200	17640	18,520	18,810	20,270	21170
0.100	22750	23,810	24,040	25,650	26830
0.040	28,850	30,100	30,500	32,370	33741
0.020	33430	35,010	35,600	37,610	39170
0.010	37470	39,350	40,100	42,410	44370
0.005	40,640	42,700	43,600	46,100	48310
0.004	41,400	43,900	44,800	47,300	48891
0.002	44540	47,300	48,300	51,060	53590
0.001	47160	50,430	51,810	54,770	57550
Index Point Station	1L 88034.00	2R 85379.00	3R 83769.00	5L 79160.00	7R 72095.00
Index Point Station			4L 82453.00	6L 72621.00	8R 70411.00

Note:

1. Curve numbers, shaded flows and probabilities, index points, and station locations were provided by the hydraulic Design Section.

2. Locations and Table numbers at the head of the flow columns indicate source Tables in the November 21, 2007 Memorandum for record above this addendum.

3. Flows and probabilities can be found in the same Tables.



NOTES:

1. Based on median plotting positions (also Computed)

2. Flow data for Napa River are based on 72 years of record.

3. Plotted curves are plotted as ordered, top to bottom in the legend located in the graphic box.

Curve No.	Index Point	Station	Index Point	Station
Curve 1	1L	88034.00	---	—
Curve 2	2R	86379.00	---	—
Curve 3	3R	83769.00	4L	82463.00
Curve 4	5L	79160.00	6L	72621.00
Curve 5	7R	72095.00	8R	70411.00

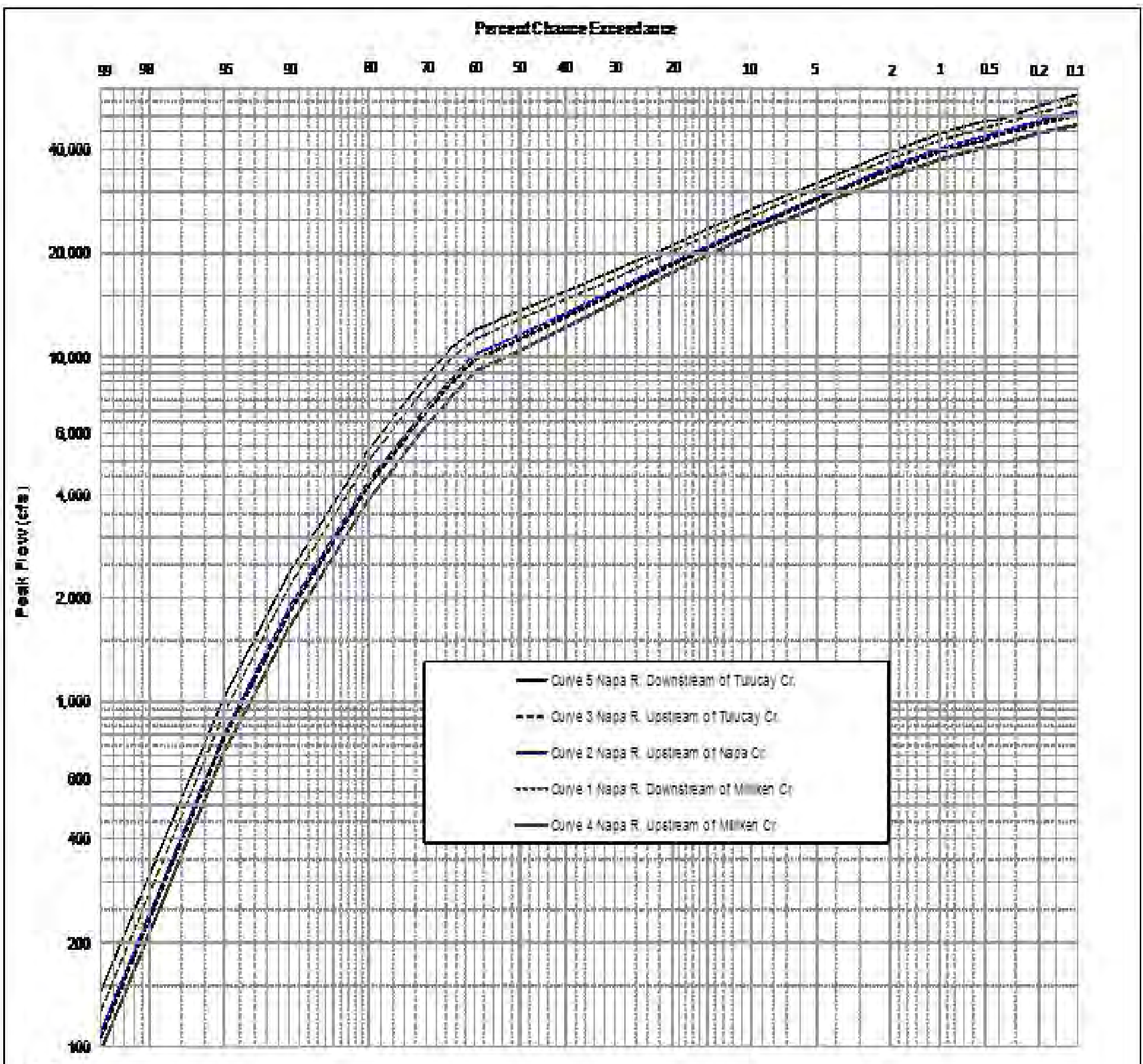
NAPA RIVER BASIN
NAPA RIVER AT 5 LOCATIONS THROUGH NAPA

REGULATED PEAK FLOW FREQUENCY
CURVES FOR NAPA RIVER BELOW
OAK KNOLL AVENUE, NAPA, CALIFORNIA

US ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by: RFC 11-Jan-2010

Figure 7: Figure 5 re-plotted, Frequency Curves for the Napa River Upstream of Milliken, Napa and Tulucay Creeks and downstream of Tulucay and Milliken Creeks.



NOTES:

1. Based on median plotting positions (also Computed)
2. Flow data for Napa River are based on 72 years of record.
3. Plotted curves are plotted as ordered, top to bottom in the legend located in the graphic box.

Curve No.	Index Point	Station	Index Point	Station
Curve 1	1L	88034.00	---	---
Curve 2	2R	86379.00	---	---
Curve 3	3R	83769.00	4L	82463.00
Curve 4	5L	79160.00	6L	72621.00
Curve 5	7R	72095.00	8R	70411.00

NAPA RIVER BASIN
NAPA RIVER AT 5 LOCATIONS THROUGH NAPA

REGULATED PEAK FLOW FREQUENCY
CURVES FOR NAPA RIVER BELOW
OAK KNOLL AVENUE, NAPA, CALIFORNIA

US ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Figure 8 Figure 5 re-plotted Frequency Curves for the Napa River Upstream of Milliken, Napa and Tulucay Creeks and downstream of Tulucay and Milliken Creeks with the Exceedance probability axis scaled from 0.99 to 0.001 probabilities.

September 1, 2010 Addendum

In 2007 an HMS model of Tulucay Creek was obtained from the Napa County Resource Conservation District. This model produced a 100 year (1% probability) peak flow of 4,530 cfs and was adopted by the CORPS for use with Tulucay Creek. The model uses SCS Unit Hydrograph as the transform method and the SCS Curve number (typically in the 70s) as loss method on all sub-basins. The outlet point of the model is Soscal Avenue Bridge which is near the USGS gage (#11458350) at Tulucay Creek and about 0.4 miles east of the Napa River. Maximum n- year 24 hour precipitation values were obtained using the Gumbel Extrapolation method from NOAA Atlas 2 for the 20-, 0.5-, 0.2-, and 0.1-% probability events. The precipitation values are as follows: 4.17, 7.39, 8.17 and 8.76 inches for the 20-, 0.5-, 0.2-, and 0.1-% probability storms. The 50 -, 20-, 10-, 2-, 1-, 0.5-, 0.2-, and 0.1-% probability peak flows produced by the HMS model are as follows: 1,080, 1,890, 2,880, 3,890, 4,530, 5,160, 6,000, and 6,660 cfs. Ratios were calculated by dividing the newly created peak flows for Tulucay Creek by the peaks flows for Tulucay Creek produced by the HEC-1 model used for the GDM and original Memorandum. The ratios for the 50 -, 20-, 10-, 2-, 1-, 0.5-, 0.2-, and 0.1-% probability peak flows are 3.20 , 3.34, 3.82, 3.05, 2.94, 2.86, 2.61 and 2.41 respectively. The hydrographs from the original HEC-1 model for Tulucay Creek were multiplied by the ratios above and were added to the local flows above Tulucay Creek, generated by taking the difference between the original Tulucay Creek (HEC-1) flows and the original Tulucay+Locals (HEC-1) flows. The new flood series, Tulucay+Locals, was then read into the downstream routing model where it was used in the creation of the hydrographs for the Napa River below Tulucay Creek. Tables 6 and 9 were reproduced and replaced in the text and appropriate changes were made to the text itself. The 1% chance peak flow in the Napa River upstream of Tulucay Creek is 42,410 cfs and the concurrent flow downstream of Tulucay Creek is 44,370 cfs. At the time of the 1 % probability peak flow of 4,530 cfs in Tulucay Creek, the concurrent flow in the Napa River is 38,370 cfs (see Tables 6 and 9).

Additional References

1. U.S. Army Corps of Engineers, The Hydrologic Engineering Center, HEC-HMS Hydrologic Modeling System, Version 3.1.0 Build 1206, dated December 2006.
2. U.S. Army Corps of Engineers, Sacramento District, Memorandum for Record: Tulucay Creek – Hydrology Review, July 6, 2006.
3. National Atmospheric and Oceanic Administration, Hydrometeorological Design Center, NOAA Atlas 2, Precipitation Frequency Atlas of the Western United States: Volume XI-California dated 1973.

Appendix F

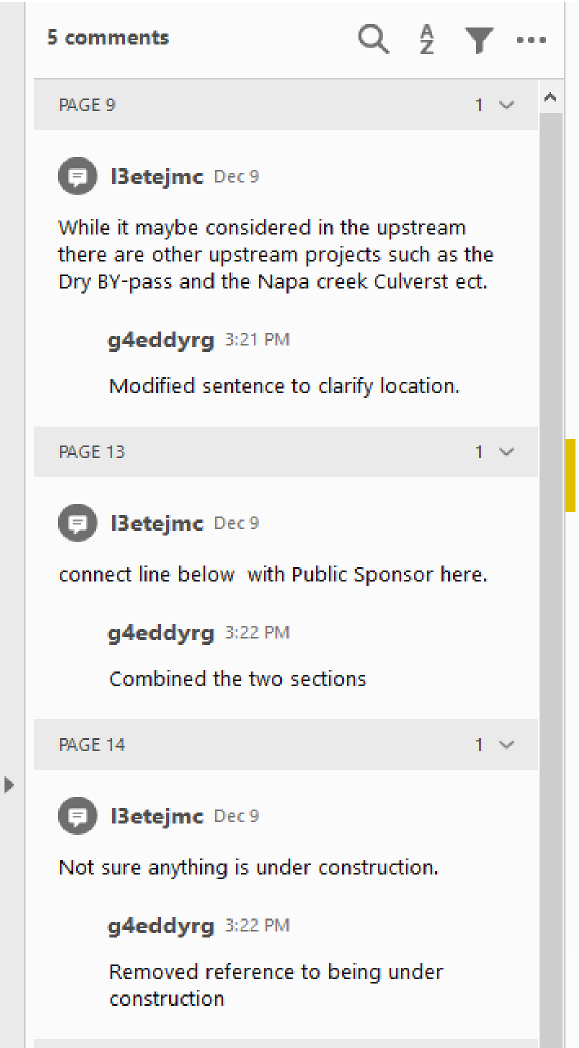
District Quality Control Document

and

Independent Technical Review

Napa River, Hattt Bldg to 1st Street- District Quality Control					
	Reviewer	Yvonne Palmer PE			
	Designer	Justin Knight			
Cmt No.	Section	Comment	Review Date	Response	Backcheck Date
1	Title	Change Title to reflect correct project	11/18/2020	Concur	11/20/2020
2	2.1	Added "." after Mr in paragragh.	11/18/2020	Concur	11/20/2020
3	2.3	Changed 70's to 70s.	11/18/2020	Concur	11/20/2020
4	4.3.2	Add information on H&H from the OMRR&R	11/18/2020	Concur	11/20/2020
5	5.3.1	Comment references levees, not floodwall - change	11/18/2020	Concur	11/20/2020
6	5.3.2	Appears the fence comment is from another project.	11/18/2020	Concur	11/20/2020
7	5.3.1	Add information about vegetation in the riprap. This is shown as an encroachment in the report but should be under vegetation.	11/18/2020	Concur	11/20/2020
8	5.4.1	Add that vegetation was noted in the riprap at the toe of the floodwall	11/18/2020	Concur	11/20/2020
9	6.1.3	add that vegetation was noted in the riprap at the toe of the floodwall and should be removed as necessary to prevent trees	11/18/2020	Concur	11/20/2020
10	6.3	The next PI should be at 5 years from the levee screening to take place in 2021.	11/18/2020	Concur	11/20/2020
11	Inspection Report	Make changes per the attached pdf.	11/18/2020	Concur (Delonnoy)	11/20/2020

ITR Comments, addressed 18 December



5 comments

PAGE 9 1

l3etejmc Dec 9

While it maybe considered in the upstream there are other upstream projects such as the Dry BY-pass and the Napa creek Culverst ect.

g4eddyrg 3:21 PM

Modified sentence to clarify location.

PAGE 13 1

l3etejmc Dec 9

connect line below with Public Sponsor here.

g4eddyrg 3:22 PM

Combined the two sections

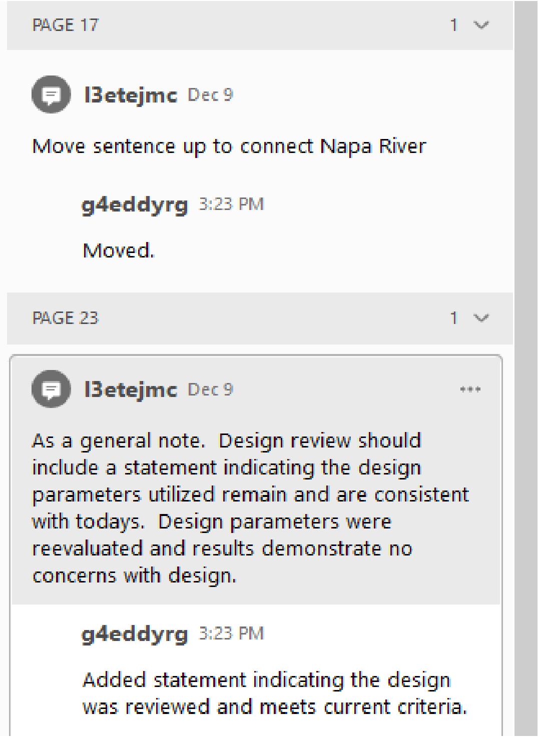
PAGE 14 1

l3etejmc Dec 9

Not sure anything is under construction.

g4eddyrg 3:22 PM

Removed reference to being under construction



PAGE 17 1

l3etejmc Dec 9

Move sentence up to connect Napa River

g4eddyrg 3:23 PM

Moved.

PAGE 23 1

l3etejmc Dec 9

As a general note. Design review should include a statement indicating the design parameters utilized remain and are consistent with todays. Design parameters were reevaluated and results demonstrate no concerns with design.

g4eddyrg 3:23 PM

Added statement indicating the design was reviewed and meets current criteria.