GAMDAT CASES

STRUCTURAL EVALUATION REPORT

January 31, 2013

EXECUTIVE SUMMARY

Background

This report summarizes engineering evaluations that were performed to validate the structural adequacy of the pile foundations identified by Caltrans internal investigations as having irregular Gamma-Gamma Logging (GGL) test results. All of the piles evaluated in this report were constructed using the wet slurry displacement method and had GGL performed by the Caltrans Foundations Testing Branch.

The Gamma-Gamma Logging Data Integrity Review (GamDat) Report identified eleven cases where irregular GGL data necessitated further investigation. Three of the eleven cases did not require structural evaluation and are not included in this report. Two of which involved qualification testing of probe and reference blocks and were not part of a structure foundation. The third case was detected during construction and the pile shaft was retested and accepted.

Caltrans specifications require all Cast in Drill Hole (CIDH) pile installations for bridges, retaining walls and sign structure foundations, constructed under the wet slurry displacement method, undergo Gamma-Gamma Logging (GGL) tests to ascertain homogeneity of concrete density of the concrete in the pile or shaft. The GGL test is one of several Quality Control/Assurance (QC/QA) tests that Caltrans utilizes to insure CIDH piles constructed with the wet slurry displacement method are built without defects.

In some cases when anomalies were present, an additional QA test, Cross Hole Sonic Logging (CSL), was performed. CSL is a non–destructive procedure that tests the homogeneity of the concrete in the interior or core of the pile, similar to how GGL tests the homogeneity of the concrete surrounding the inspection tubes. CSL tests were performed on all of the Benicia Martinez Bridge marine pier foundations that were reviewed as part of this report.

The GamDat Report concluded GGL data from one tube per pile is questionable for seven of the cases and GGL data from two tubes is questionable for the eighth case . The structural evaluators followed published Caltrans practice and either dismissed the piles with questionable GGL data entirely when calculating the revised foundations capacity or assumed the corresponding tributary slice of the pile was anomalous.

Key Findings

All eight structural evaluations summarized in this report confirm that the foundation systems, where the questionable piles are located, have adequate capacity to resist the imposed loads. Because of similarities in the results of the analysis, the eight cases are summarized in five separate groups.

Group I:

Case ID 12 Overhead Sign 19, Route 580

The questionable GGL data in this pile affected two of five tubes in the bottom 12.5% of the pile. The evaluation discounted the pile capacity proportionally by the ratio of questionable tubes to the other tubes in the pile (40%). The evaluation demonstrated that at the depth being investigated, the moment demand was less than 2% of the shaft bending capacity and the shear demand was less than 22% of the shaft shear capacity.

Group II:

Case ID 18 Retaining Wall 435B, Sawtell Blvd. UC

Case ID 52 Abutment 1 Right Retaining Wall, Braddock Dr. UC

In both cases the questionable GGL data was limited to two piles in each retaining wall. The evaluation disregarded the questionable piles in both retaining wall foundations and recalculated the demands on the remaining piles. The smallest factor of safety (Capacity/Demand) with the questionable piles being disregarded was greater than 1.0.

Group III:

Case ID 10 S.E. Connector 215/91/60 Interchange

Case ID 11 Lake Hodges Bridge, Abutment 6

In each of these cases the questionable GGL data was limited to one pile. The evaluations discounted the pile capacity by analyzing the reduced cross section using the fiber modeling tool X-section. This resulted in less than a 14% reduction in bending capacity for the SE connector pile shaft at Bent 15 and 30-40% reduction in bending capacity for Abutment 6 - Pile 1 on the Lake Hodges Bridge. Both of these piles had reserve capacity (factor of safety) in the original design. For Lake Hodges, the design calculations show an 18% reserve capacity. For the S.E. Connector, the design criteria specified the shaft to be designed with a 25% overstrength factor.

The evaluation conservatively assumed the discounted portion of the pile was removed from the compression side of the pile and assumed nominal material strengths. In the case of the Lake Hodges Bridge Abutment 6, any overstress in pile 1 will be taken up by the other piles supporting the abutment.

Group IV:

Case IDs 17, 33, 76 Benicia Martinez Bridge

Pile capacities were calculated by disregarding the sector defined by the tributary slice from the questionable GGL data minus the CSL data that confirmed the homogeneity of the concrete in the pile's interior core. Pile anomalies that were detected and not repaired during construction were also included in the capacity analysis.

Demand envelopes were generated from the displacement time history design model. The maximum moment at top of pile, maximum moment along the length of pile, and associated moment for the minimum axial load on each pile was extracted from time history analysis.

The moment demands were compared to the discounted pile capacities for the portion of the three piles constructed with the slurry displacement method. The smallest factor of safety for discounted bending capacity /moment demand was greater than 1.0.

Shear demands could not be extracted from the time history model. Instead shear demands we generated by calculating the overstrength shear capacity of the base of the pier and dividing it equally to all of the piles in the pier foundation. The smallest factor of safety for discounted shear capacity /shear demand was greater than 1.0.

Group V:

Null Value Data Evaluations

Two cases were evaluated as a result of the Null Value data identified in the GamDat Report. Null values are locations along the pile where density readings were not recorded in the GGL data files. For the Retaining Wall A piles on the I-405 Gap Closure project, two 24 inch CIDH piles were assumed anomalous and removed from the analysis. Revised factors of safety (Capacity/Demand) were greater than 1.0 for all load cases. For the S149-E70 connector the evaluation discounted the pile capacity proportional to one of the 7 tubes that was in question. Analysis showed that the reduced capacity was still well above the demand load.

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Appendix G Benicia Martinez Bridge No. 28-0153R (ID's 17, 33, 76)
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Appendix H I-405 Retaining Wall A & S149-E70 Connector Separation
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1. INTRODUCTION

This report summarizes the evaluations that were performed to validate the structural adequacy of pile foundations identified as cases in the Gamma-Gamma Logging Data Integrity Review (GamDat) Report where the engineering analysis may be affected. All of the piles that are evaluated in this report were constructed using the wet slurry displacement method and had GGL performed by the Caltrans Foundations Testing Branch. Three of the eleven cases did not require structural evaluation and are not included in this report. Two of which involved qualification testing of probe and reference blocks and were not part of a structure foundation. The third case was detected during construction and the pile shaft was retested and accepted.

It should be noted that much of the structural evaluation work was performed by Caltrans engineers shortly after the irregularities in Gamma-Gamma test results first came to light. The purpose of this report is to collect and document the structural evaluations performed by various engineers and to confirm the evaluation methodologies used by the engineers are consistent and adequate for determining the structural adequacy of the piles in question. This report frequently references the GamDat Report and wherever possible uses the same nomenclature to present data and information.

2. BACKGROUND

Caltrans specifications require all Cast in Drill Hole (CIDH) pile installations for bridges, retaining walls and sign structure foundations, constructed under the wet slurry displacement method, undergo Gamma-Gamma Logging (GGL) tests to ascertain homogeneity of concrete density in the pile. The GGL test is one of several Quality Control/Assurance (QC/QA) activities that Caltrans utilizes to insure CIDH piles constructed with the wet slurry displacement method are built without defects. The GGL test requires placing PVC inspection tubes in the outer perimeter of the rebar cage pile during construction to accommodate a radioactive probe that is lowered into the tubes after concrete placement to record bulk density readings along the length of the pile. These readings are used to determine if any substantial drops in bulk density readings exist at any location along the pile which is indicative of the presence of anomalies which may indicate a defect such as a void or soft spot in the concrete surrounding the inspection tubes. GGL is generally viewed as one of the most accurate non-destructive test methods to detect pile defects. Based on the results of GGL test data the pile is either accepted if no significant anomalies[1] are present or rejected depending on the significance and location of the anomaly, in which case further evaluation is necessary. This evaluation is done by the bridge designer by comparing the reduced section capacity as a result of the anomalous region of the pile to the load demands expected at that location.

In some cases an additional test, the Cross Hole Sonic Logging (CSL) test is performed. This test is performed to test the homogeneity of the concrete in the interior or core of the pile, as the GGL test is limited in its ability to detect anomalies to the area adjacent to and surrounding the inspection tubes. CSL is performed by placing signal probes and receiver probes in different combinations of inspection tubes and emitting a high frequency compression wave and logging the sound wave time along the length of the pile or at specific location where the anomalies were detected. The propagation time of these sonic waves is a function of concrete density and as a result this test is used to predict concrete integrity between the probes. The CSL test can either verify that the interior section of the pile has no significant anomalies, in which case the engineering analysis performed to reduce the cross-sectional capacity of the pile is refined further by the CSL test findings and may show that the percentage of pile cross section compromised by the anomalous concrete is not as extensive, or it may verify that a repair of the anomaly is necessary.

[1] Significant anomalies as defined in Gam-Dat report is any location along the length of the pile where the concrete density drops to more than 3 standard deviations from the mean bulk density.

3. CIDH PILE CONSTRUCTION

Cast in Drilled Hole (CIDH) pile construction is a common type of pile construction and involves drilling a hole to a predetermined elevation, lowering the pile rebar cage in the drilled hole and then placing concrete, casting the pile in place. This method of pile construction is usually an alternative to driven piles, such as steel or precast concrete piles, where piles are driven into the ground. The selection of either of these two types of pile installation or other foundation types is dependent on many factors such as structural performance, ground conditions at the site and any constraints that may be present for the particular construction project.

When ground water is not present the CIDH pile can be poured "in the dry", and GGL testing is not required. Otherwise CIDH piles require a series of special pile placement operations, as denoted in the construction contract documents. The objective behind these special pile placement operations is to keep water out of the hole during concrete placement, keep the walls of the drilled hole open and free of soil particles, where you have a potential for caving soil, and to ensure that the CIDH pile is built without defects that are possible with this type of pile installation method if not constructed properly.

This special method is commonly referred to as the "wet slurry displacement method", and involves drilling the hole with the use of an approved slurry which is typically a combination of chemical admixtures and water placed into the hole to a high enough level to maintain a positive effective stress to keep the ground water out. In addition the slurry forms a barrier on the inside of the drilled hole to keep the sides of the hole from coming loose and binds any soil particles in slurry together so they will sink to the bottom of the hole and can be removed prior to concrete placement. After the hole is drilled and cleaned out, the rebar cage is lowered into the hole and concrete is pumped into the hole by use of a tremie tube, starting from the bottom of the hole and gradually raising the tremie tube at a rate keeping the tremie tube below the level of concrete to prevent mixing of the slurry with the concrete. This process eventually displaces the less dense slurry out of the hole for disposal per the contract specifications.

4. PILE ANOMALY EVALUATION PROCESS

In the event that an anomaly is detected in the pile, Caltrans follows a documented process to evaluate the effect of any detected anomalies on the structural adequacy of the pile. When a significant anomaly is detected, the contractor is notified that the pile is rejected, and an evaluation is set into motion to determine whether the anomaly needs to be repaired.[2]

This process is initiated by the Geotechnical Services Branch issuing a Gamma-Gamma Logging testing results report outlining the potential presence of anomalies for each CIDH pile constructed using the wet slurry displacement method, their location along the depth of the pile and the significance of the anomaly by estimating the percentage of pile cross section that may be affected. This report is then submitted to the structure representative, the geotechnical designer and the bridge designer. The geotechnical designer evaluates the effect of the presence of the anomaly on the estimated geotechnical axial capacity of the pile and makes a determination on the resulting reduced axial capacity of the pile and whether the anomaly needs to be repaired. This information is filled out on the "Pile Design Data form" (see appendix A) where the required axial geotechnical capacity of the pile is listed in addition to the demand loads at the anomaly locations and whether a repair is required. Similarly the bridge designer evaluates the presence of the anomaly on the structural capacity of the pile, and is required to fill out the appropriate section of the same "Pile Design Data form" listing the required moment and shear capacity and the available capacity taking the anomaly into account, and whether a repair is required. This form is then routed back to the structure representative who uses this information to determine if the contractor has to repair the pile or can opt to take an administrative monetary deduction if the outcome of the evaluation determines the size and location of the anomaly does not compromise the structural and geotechnical performance of the pile.

A pile mitigation plan, stamped by a licensed California Civil Engineer, is requested from the contractor when an anomaly needs to be repaired. The mitigation plans vary in complexity. For example, if an anomaly is near the top surface of the pile, unsound concrete is typically chipped down and replaced with sound concrete. Anomalies further down the pile at much greater depths, typically require high pressure water jetting to clean out the affected concrete area and pressure grouting of the void with new concrete. For this type of repair the

[2] This process is described in Caltrans Memo to Designers 3-7 (see Appendix A) and the Bridge Construction Records and Procedures Manual Section 130- Foundations

mitigation plan submitted to the structure representative would include an extensive report of the methodology the contractor would use to conduct the repair, the equipment used, and the material specifications for any material used in the repair procedure. The pile mitigation plan is reviewed by the Caltrans Structure Representative and if approved the contractor performs the repair.

5. STRUCTURAL EVALUATIONS FOR IDENTIFIED CASES

As reported in the GamDat report eight cases were identified that met the team's criteria for irregular Gama-Gama test data and require structural evaluation. These cases, listed in Table 1, identify specific piles that were part of either a pile group or individual stand-alone piles supporting a bridge column, sign structure or a retaining wall.

Table 1 contains the case ID number, consistent with the Gam-Dat team's report and lists the Project information, Bridge number, Pier or Bent number, pile number, and inspection tube in question. The structural evaluations are summarized in the following subsections and vary in complexity depending on the specific details of each case, and whether a more refined analysis is warranted to reach a final conclusion.

Table 1 - IDENTIFIED GAM-DAT CASES

Case ID	Br. No.	Description	Pile Location	No. of Effected Tubes
10	56-802F	SE Connector (08-Riv-215)	Bent 15	1 of 14 (tube 14)
11	57-1134R	Lake Hodges Bridge (11-SD-15)	Pile 1 at Abutment 6	1 of 4 (tube 2)
18	RW 435B	Sawtell Blvd UC, Retaining Wall 435B (07-LA-405)	Piles 69 & 72	1 of 4 (tube 4)
52	53-1258	Braddock Drive UC (Widen) (07-LA-405)	Abut. 1 Right Retaining Wall Piles 50 & 51	1 of 4 (tube 2)
12	N/A	Route 580 Overhead Sign Structure Foundation (04-ALA-580)	Overhead Sign 19 5ft diameter CIDH pile	N/A[3]
17	28-153R	Benicia-Martinez Bridge	Pile 5 at Pier 6	1 of 8 (tube 5)
33	28-153R	Benicia-Martinez Bridge	Pile 6 at Pier 12	1 of 8 (tubes 6 or 7)
76	28-153R	Benicia-Martinez Bridge	Pile 8 at Pier 14	1 of 8 (tube 3)

[3] For ID 12 a specific elevation along the length of the pile was identified by Gam-Dat rather than a tube number.

5.1 CIDH PILE DESIGN OVERVIEW

The objective behind designing pile foundations is to ensure the lateral, vertical and rotational capacity of the foundation exceeds all expected demands on the foundation. This is accomplished by optimizing the pile size, number of piles, and pile group layout. Caltrans has a number of standard piles that have been "pre-designed" for foundations in competent soil. The designer selects the standard piles and engineers the number of piles and pile layout to resist the foundation demands without exceeding the allowable capacity of each pile. Standard piles are often used for retaining walls, abutments, and ordinary bridges supported on pile cap footings.

In many situations, a project specific pile design is necessary where the piles are required to resist large axial, shear, and bending forces. This is usually the case for large bridges, single shaft foundations, or sites with complex geologic conditions. In these situations the piles may be designed to perform elastically or inelastically as ductile members. The CIDH piles associated with the cases listed on Table 1 consist of a wide variety of pile types thus requiring different design methodologies, which are briefly discussed in each of the case ID sub-sections.

The CIDH pile listed in this report under case ID-10 for the SE connector (see Table 1) is classified under the Caltrans Seismic Design Criteria (SDC) as a Type II shaft. This type of pile is an enlarged pile shaft where the pile diameter is at 18 - 24 inches larger than the column diameter and the column rebar cage is developed into the pile for a specified embedment length. The typical design approach for this type of pile involves designing the loads imposed on the pile for all non-seismic loading combinations and then checking the pile for seismic loads. The SE connector is classified as a Standard Ordinary Bridge, and is designed according to the SDC. The SDC requires type II shafts to have adequate capacity to withstand the maximum demands imposed on the pile while remaining essentially elastic.

Essentially elastic pile response for Type II shafts is achieved by applying a column over-strength moment at the top of pile which is equal to 1.2 x Mp (Column Plastic Moment). This plastic limit state generates a maximum moment and shear demand profile along the length of the pile whose magnitude is dependent on the soil properties and pile stiffness. The maximum moment generated in the pile is then multiplied by a factor of 1.25 and is used to design the pile nominal moment capacity (see Figure 1). This overdesign is intended to ensure that the column plastic hinge is isolated to the base of column where any post earthquake damage can be inspected and repaired while the pile shaft remains essentially elastic.

CIDH piles can also be part of a pile group with a pile cap or a footing, which are typically used for abutment footings and retaining walls. For case ID's 18 & 52 (see Table 1), these piles are part of a retaining wall footing and are designed to withstand axial loads generated from lateral earth and water pressure, the effects of surcharge loads, and the self-weight of the wall. This is done by designing the pile group layout so that the imposed loads from different service load combinations are below the service load capacity of the pile.

For case ID's 17, 33 & 76 (see Table 1), the CIDH piles supporting the pile cap for the Benicia Martinez main spans are specially designed piles that were part of a special "Project Specific" seismic design criteria used for this bridge. These piles were cast in steel shell (CISS) piles for the top portion of the pile with an uncased "rock socket" for the lower portion. These specially designed piles are designed to withstand the maximum imposed seismic demands as a result of the columns reaching the column plastic moment while at the same time remaining essentially elastic. The pile demand loads for the evaluation where generated from the original 3-D non-linear dynamic analysis model (ADINA) with displacement time histories. The analysis done as part of this report on these piles is discussed in detail in section 5.7.

For other specially designed piles as in case ID-11 for the Lake Hodges Bridge, the site conditions were such that the pile foundation was susceptible to liquefaction and lateral spreading. The lateral spreading loadings were generated from a geotechnical study done for the site and combined with other loads that the pile is subjected to. The lateral load analysis was done with "Lpile", a computer program where the pile is modeled with p-y springs, which are used to model the soil stiffness along the length of the pile (provided by the geotechnical designer). The lateral EQ loads are added to the other applied loadings such as deal load, earth pressure and inertia loads, to generate maximum moment and shear demands in the pile.

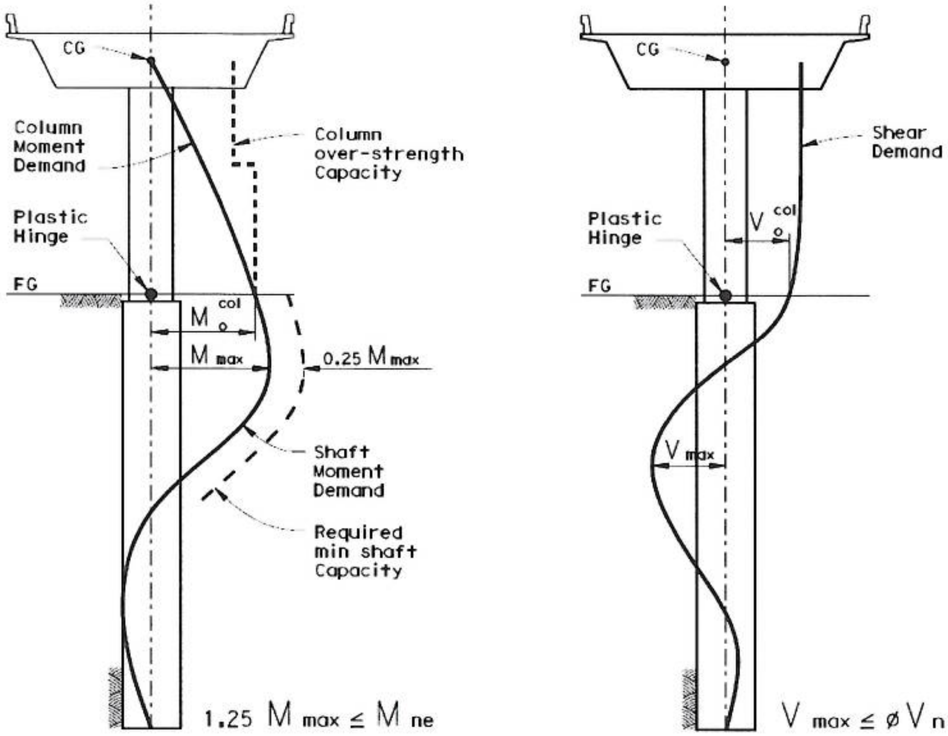

Figure 1 Typical Moment and Shear Diagrams for Type II Shafts

5.2 STRUCTURAL EVALUATION FOR ID - 10

I. PROJECT DESCRIPTION: SOUTH-EAST CONNECTOR (08-RIV-215)

POTENTIAL PILE SHAFT ANOMALY AT BENT 15

II. STRUCTURAL EVALUATION

The SE Connector is a 17 span CIP/PS Box Girder Bridge on the 215/91/60 Interchange in Riverside (see Attachments 1-3).

The pile in question is a 3.65m diameter 29m long CIDH pile with 14 PVC inspection tubes. No significant anomalies were detected in the tested portion[4] of the pile during construction. One of the 14 tubes required re-evaluation by the Gam-Dat team.

In evaluating the capacity of the pile cross section, the designer looked at a worst case scenario by reducing the total cross section by a pie shaped section proportional to the total number of tubes in the pile, which proportionally represented a 7% reduction in capacity (See Figure 2). This reduction in capacity was verified by running the computer program 'X-section' to model the reduced moment capacity of the cross section by discounting the tributary pie shaped area around the inspection tube. The results of this analysis showed a 7-8% reduction in capacity if the discounted concrete area is conservatively modeled as a void, and a 13% reduction in capacity if both the concrete and compression steel inside the discounted wedge is completely removed from the analysis (see Appendix B). This reduction in capacity was compared with the 25% increase in moment capacity built into the design of the pile, as required by the 'SDC' for Type II shafts, which is the case for the pile shafts designed for this bridge. In addition to this the maximum shear demand in the pile was compared to a reduced shear capacity by discounting the tributary pie shaped area around the inspection tube, and the result of this analysis showed that the demand was still well below capacity.

As was explained in section 5.1 of this report for the Seismic Design Criteria for Type II shafts, the 'SDC' states that Type II shafts shall be designed as a "capacity protected member", meaning the pile is overdesigned relative to the column. This is done by using a factor of safety

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[4] Type II piles are constructed with a construction joint at the bottom of the rebar cage which allows the top portion of the pile to be poured in the dry where no GGL testing is required.

of 1.25 when designing for the maximum moment in the pile (see Section 5.1, Figure 1). In addition to this the column "overstrength" moment, which is defined as 1.2 x Mp, is applied at the base of column when generating the maximum moment in the shaft. This is done to ensure that the plastic hinge forms at the base of the column and the shaft remains essentially elastic.

As a result of this conservatism built into the design criteria for this type of pile, the conclusion of the structural evaluation was that even with the test data irregularities the pile in question is still structurally adequate.

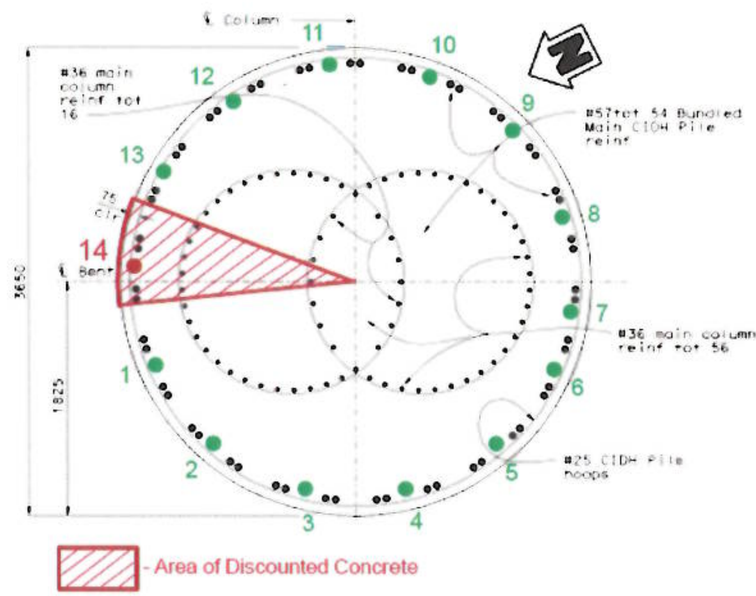

Figure 2 - SE Connector Bent 15 Pile Cross Section

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET No.	TOTAL SHEETS
08	Riv	215, 60, 91	60.7/70.4, 22.3/18.5, 32.7/34.1	2030R2	2838

REGISTERED CIVIL ENGINEER DATE 11-02-04

8-4-03

PLANS APPROVAL DATE

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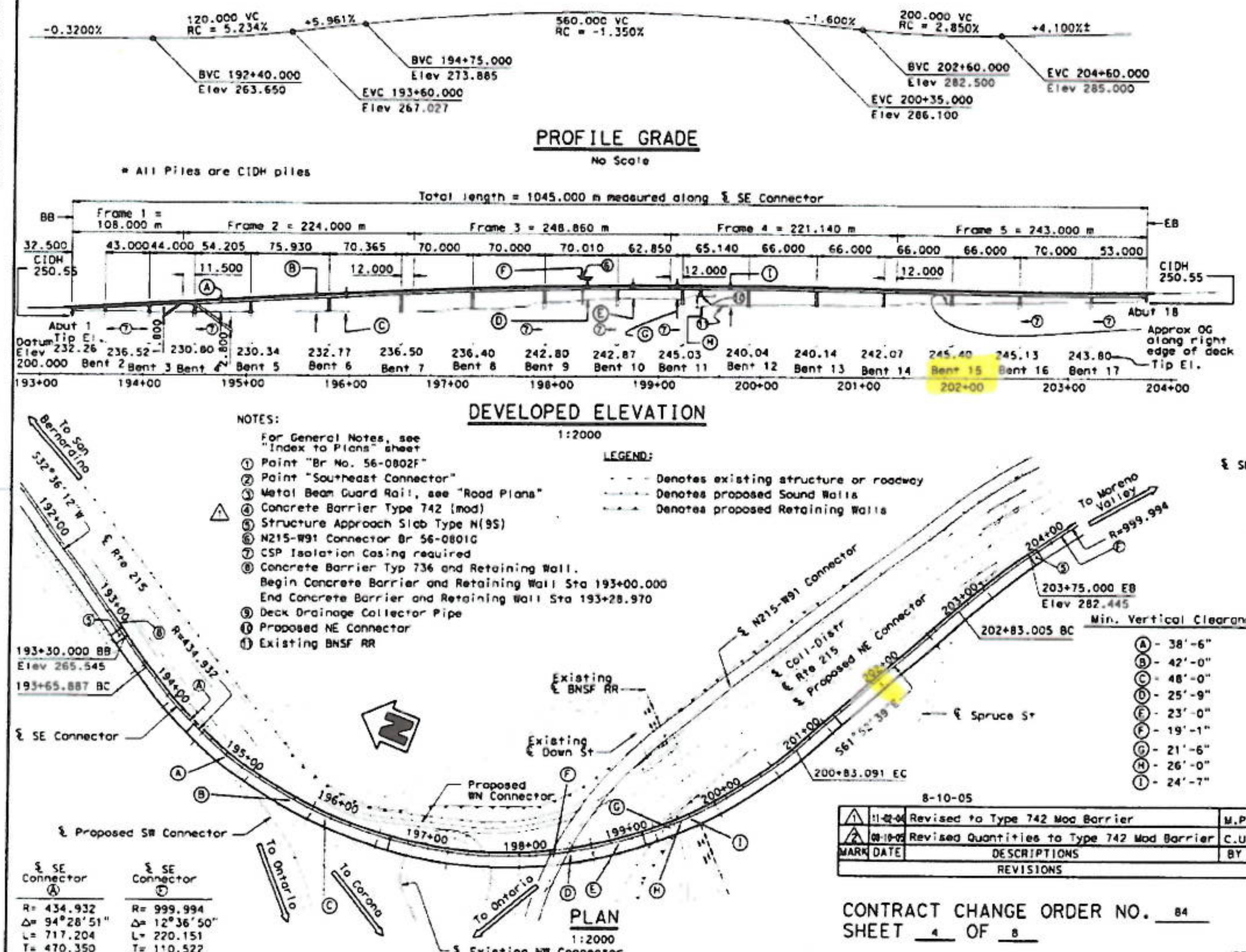
MICHAEL A. POPE No. C 54503 Exp. 12-31-05 CIVIL

Caltrans Metric



NOTES:

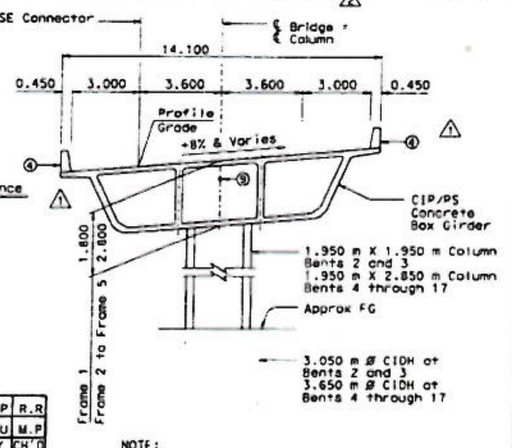
For General Notes, see "Index to Plans" sheet

1. Point "Br No. 56-0802F"
2. Point "Southeast Connector"
3. Metal Beam Guard Rail, see "Road Plans"
4. Concrete Barrier Type 742 (mod)
5. Structure Approach Slab Type N(9S)
6. N215-W91 Connector Br 56-0801G
7. CSP Isolation Casing required
8. Concrete Barrier Typ 736 and Retaining Wall. Begin Concrete Barrier and Retaining Wall Sta 193+00.000 End Concrete Barrier and Retaining Wall Sta 193+28.970
9. Deck Drainage Collector Pipe
10. Proposed NE Connector
11. Existing BNSF RR

℄ SE Connector Ⓐ	℄ SE Connector Ⓔ
R= 434.932	R= 999.994
Δ= 94°28'51"	Δ= 12°36'50"
L= 717.204	L= 220.151
T= 470.350	T= 110.522

QUANTITIES

STRUCTURE EXCAVATION (BRIDGE)	910	m^3
STRUCTURE BACKFILL (BRIDGE)	390	m^3
600 MM CAST-IN-DRILLED-HOLE CONCRETE PILING	382	m
3.6 m CAST-IN-DRILLED-HOLE CONCRETE PILING	435	m
3.0 m CAST-IN-DRILLED-HOLE CONCRETE PILING	45	m
PRESTRESSING CAST-IN-PLACE CONCRETE	LUMP SUM	
STRUCTURAL CONCRETE, BRIDGE FOOTING	53	m^3
STRUCTURAL CONCRETE, BRIDGE	11 100	m^3
STRUCTURAL CONCRETE, APPROACH SLAB (TYPE N)	39	m^3
PTFE SPHERICAL BEARING	8	EA
JOINT SEAL ASSEMBLY (MR 70 MM)	14	m
JOINT SEAL ASSEMBLY (MR 101 MM - 160 MM)	14	m
JOINT SEAL ASSEMBLY (MR 161 MM - 240 MM)	14	m
JOINT SEAL ASSEMBLY (MR 241 MM - 320 MM)	42	m
BAR REINFORCING STEEL (BRIDGE)	2 446 000	kg
MISCELLANEOUS METAL (RESTRAINER - PIPE TYPE)	19 270	kg
MISCELLANEOUS METAL (RESTRAINER - CABLE TYPE)	8 770	kg
MISCELLANEOUS METAL (BRIDGE)	9 930	kg
CONCRETE BARRIER (TYPE 742 MODIFIED) ⚠2	2 117	m



NOTE: Bent 8 shown others similar except as noted

TYPICAL SECTION

1:100

8-10-05

MARK	DATE	DESCRIPTIONS	BY	CH'D
1	11-02-04	Revised to Type 742 Mod Barrier	M.P	R.R
2	08-10-05	Revised Quantities to Type 742 Mod Barrier	C.U	M.P

REVISIONS

CONTRACT CHANGE ORDER NO. 84

SHEET 4 OF 8

ALL DIMENSIONS ARE IN METERS UNLESS OTHERWISE SHOWN

DESIGN SEQUENCE 1

DESIGN	BY M. Pope	CHECKED C. Udorbe
DETAILS	BY B. Robinson	CHECKED C. Udorbe
QUANTITIES	BY M. Pope	CHECKED C. Udorbe

DESIGN ENGINEER 11-02-04

LOAD FACTOR DESIGN — LIVE LOADING: HS20-44 AND ALTERNATIVE AND PERMIT DESIGN LOAD

LAYOUT BY M. Pope — CHECKED C. Udorbe

SPECIFICATIONS BY M. Kopsa — PLANS AND SPECS COMPARED M. Kopsa

STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

DIVISION OF STRUCTURES STRUCTURE DESIGN 17

BRIDGE NO. 56-0802F

KILOMETER POST 69.59

SE CONNECTOR (SB215/EB215)

GENERAL PLAN

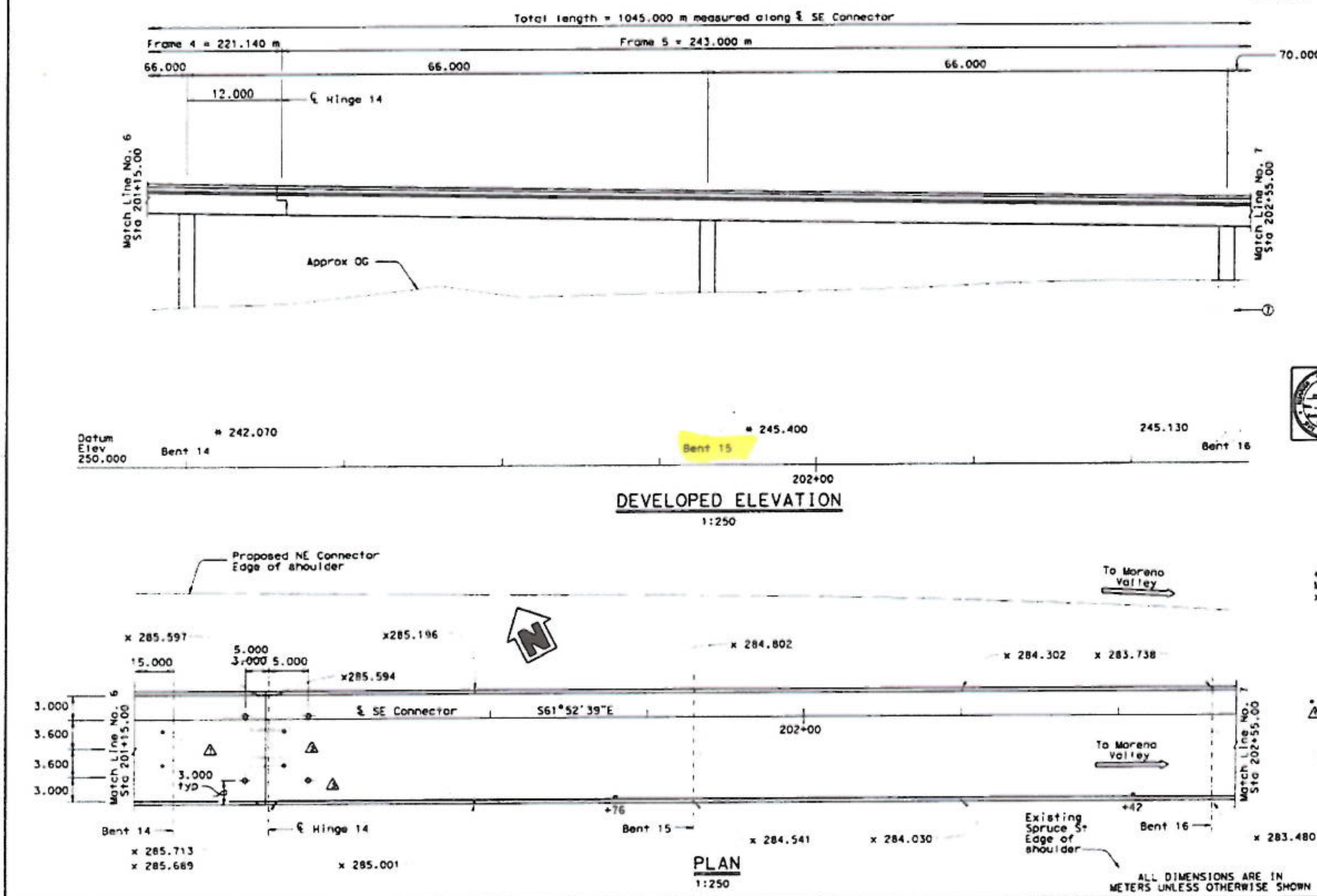
ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS

CU 08 EA 33484

DISREGARD PRINTS BEARING EARLIER REVISION DATES

AS BUILT CORRECTIONS

ATTACHMENT 1



DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET No	TOTAL SHEETS
08	Riv	215, 60, 91	[illegible]	[illegible]	2838

REGISTERED CIVIL ENGINEER DATE

8-4-03

PLANS APPROVAL DATE

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

	DATE	DESCRIPTIONS	BY	CH'D
1	[illegible]	Revised for Cable Restrainer	CU	MP
2	[illegible]	Revised for Pipe Restrainer	CU	MP
3	[illegible]	Revised for Deck Access Opening	CU	MP
MARK	DATE	DESCRIPTIONS / REVISIONS	BY	CH'D

CONTRACT CHANGE ORDER NO. 133
SHEET 8 OF 16

* = CIDH Pile Tip Elev.
MVC= Minimum Vertical Clearance
x, Copper nails Elev. on top of barrier

NOTES:

① Steel Isolation Casing Required
- • Denotes Deck Drain Type D-3 Station indicates ℄ Drain
- • Denotes Cable Restrainer Unit Type 2, see "Cable Restrainer Unit - Type 2" sheet
- Denotes Pipe Restrainer see "Hinge 7,10, & 14 Details No. 3" sheet
- ⊕ Denotes Deck Access Opening typ. 4

B7-7

B7-11
U43

ALL DIMENSIONS ARE IN METERS UNLESS OTHERWISE SHOWN

DESIGN SEQUENCE 1

DESIGN	BY M. Pope	CHECKED C. Udorbe
DETAILS	BY B. Robinson	CHECKED C. Udorbe
QUANTITIES	BY M. Pope	CHECKED C. Udorbe

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

DIVISION OF STRUCTURES
STRUCTURE DESIGN 17

BRIDGE NO. 56-0802F
KILOMETER POST 69.59

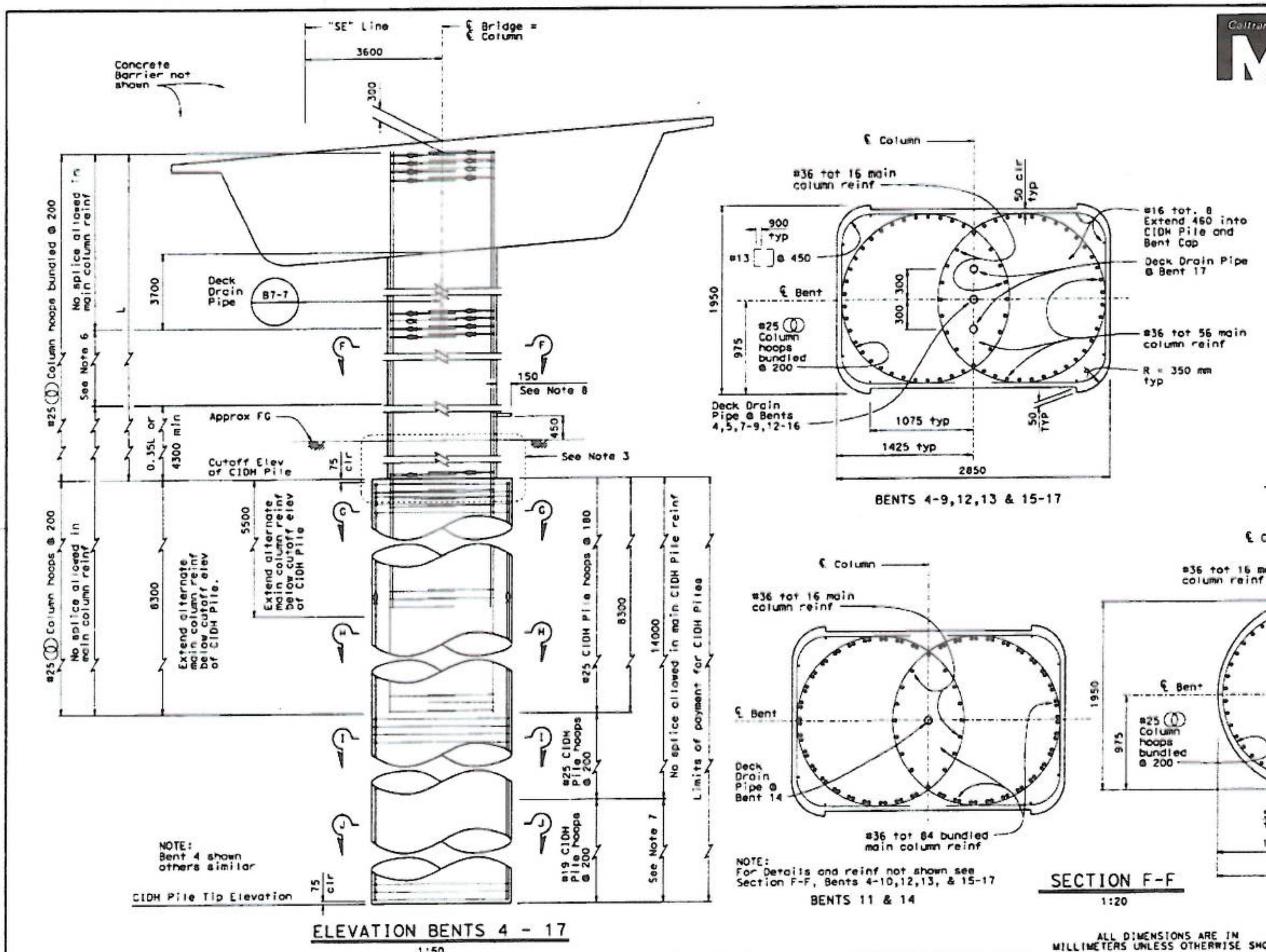
SE CONNECTOR (SB215/EB215)
STRUCTURE PLAN NO. 7

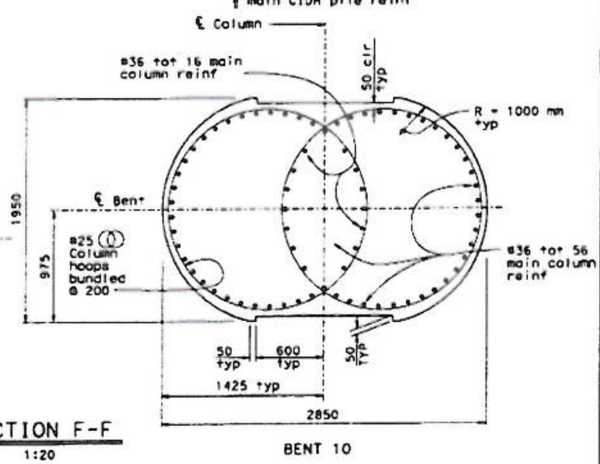
CU 08
EA 334841

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS

DISREGARD PRINTS BEARING EARLIER REVISION DATES

ATTACHMENT 2





SECTION F-F

1:20

NOTES:

1 For Sections G-G and H-H see "Bent Details No. 4" sheet
2 For Sections I-I and J-J see "Bent Details No. 5" sheet
3 Column isolation casing for Bents 4, 9, 10, 11, 16, & 17 is not shown see "Column Isolation Details" sheet
4 For pile data, see "Index To Plans" sheet
5 All hoops are "ultimate" butt splice Continuous
6 Only staggered "ultimate" butt splices are allowed in main column reinf
7 Only staggered "ultimate" butt splices are allowed in main CIDH pile reinforcement
8 For deck drainage connection see "Road Plans" (Bents 4,5,7-9,and 12-16) (Bent 17 has 2 Deck Drain Pipes & 2 exits @ Base of Column)

Denotes horizontally bundled hoops

Denotes vertically bundled main CIDH pile reinf

ALL DIMENSIONS ARE IN MILLIMETERS UNLESS OTHERWISE SHOWN

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET No	TOTAL SHEETS
08	Riv	215,60, 91	40.7/10.6, 22.0/18.5, 32.7/34.7	2066	2838

REGISTERED CIVIL ENGINEER DATE

8-4-03

PLANS APPROVAL DATE

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REGISTERED PROFESSIONAL ENGINEER MICHAEL R. POPE C 54593 Exp. 12-31-05 CIVIL STATE OF CALIFORNIA

DESIGN SEQUENCE 1

DESIGN	BY C. Huang	CHECKED C. Udorbe
DETAILS	BY B. Robinson	CHECKED C. Udorbe
QUANTITIES	BY C. Huang	CHECKED C. Udorbe

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

DIVISION OF STRUCTURES
STRUCTURE DESIGN 17

BRIDGE NO. 56-0802F
KILOMETER POST 69.59

SE CONNECTOR (SB215/EB215)
BENTS DETAILS NO. 3

CU 08
EA 334841

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS

STRUCTURES DESIGN DETAIL SHEET (METRIC) (REV 12-1-07)

ATTACHMENT 3

5.3 STRUCTURAL EVALUATION FOR ID - 11

I. PROJECT DESCRIPTION: LAKE HODGES BRIDGE (11-SD-15)

POTENTIAL PILE SHAFT ANOMALY AT PILE 1 OF ABUTMENT 6

II. STRUCTURAL EVALUATION

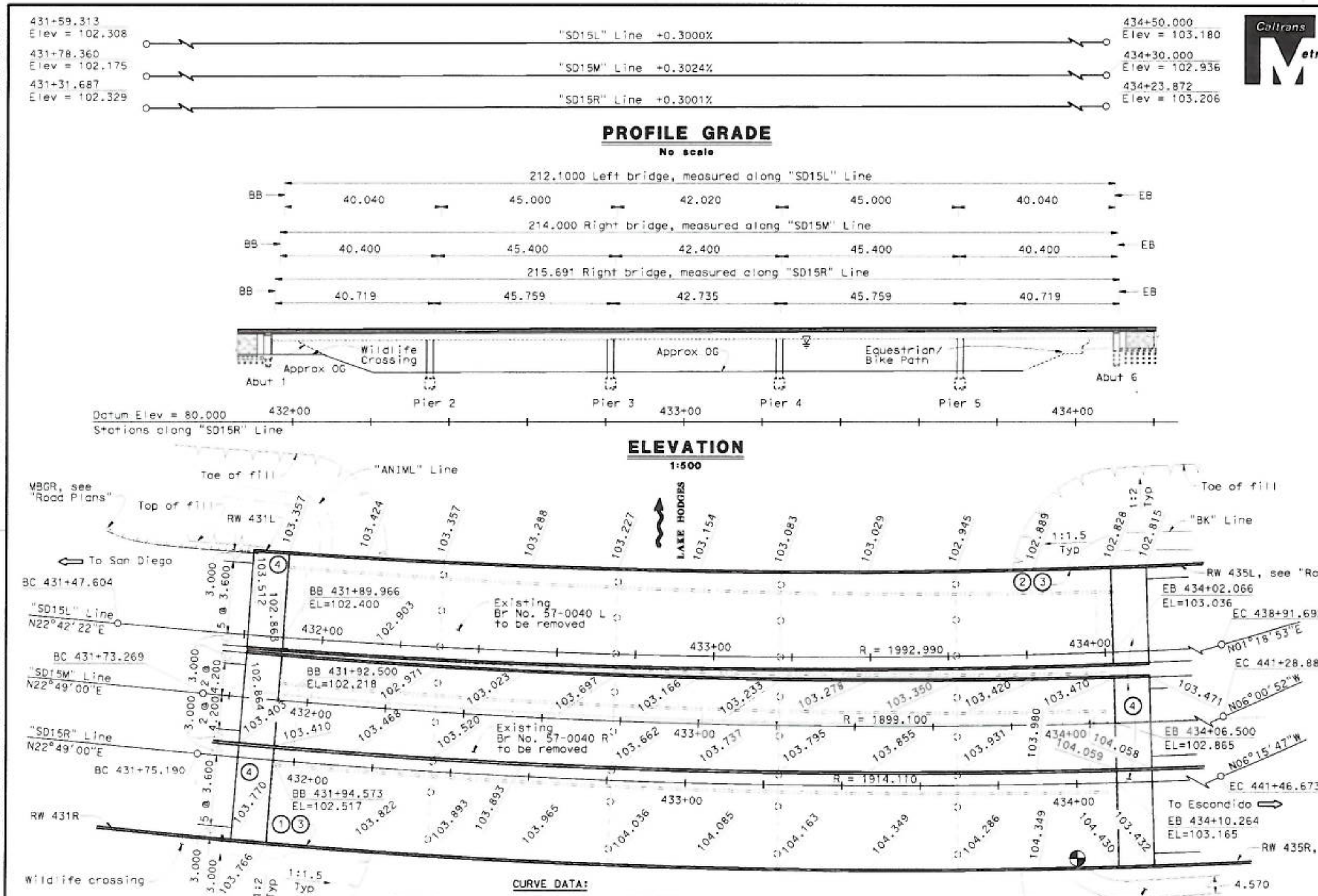
Lake Hodges Bridge is a 5 span CIP/PS Box Girder Bridge on I-15 in San Diego County. The abutments are tall seat type abutments supported on a row of 1.4m diameter CIDH piles with permanent steel casing in the top portion of the piles.

The pile in question is Pile 1 (see Attachments 4-7) of Abutment 6 of the Right Bridge. The total length of the pile is 25m with the steel casing placed for constructability in the top 18m. The pile included four inspection tubes. No significant anomalies were detected during construction, with the exception of concrete in the top 0.1m of pile which was chipped out and replaced as is often done with this method of construction.

The Gam-Dat team recommended structural re-evaluation because one of the four inspection tubes for this pile is questionable. This qualitatively represented a 25 % reduction in capacity assuming that a potential anomaly may occur at the point of maximum moment in the pile. The designer re-evaluated the capacity using the program 'X-section'. The pile was analyzed by discounting a 25% pie shaped wedge on the compression side of the pile ignoring both concrete and steel, which indicated that there is approximately a 32% to 42% reduction in capacity depending on how the discounted wedge is oriented (see Appendix C). However the original design calculations indicated that there was an 18% overdesign in the pile comparing the capacity to the maximum demand moment, which in theory would reduce any overstress that may occur in the pile. The stiff abutment wall and the 18 piles are expected to move as a unit, and load redistribution will occur between the piles if any overstress occurs in Pile 1.

The shear demand from the pile lateral loading analysis was compared to a reduced shear capacity assuming a 25% reduced capacity as a result of the discounted concrete wedge, and this analysis also showed that the demand was still below capacity.

As a result of this structural assessment of the pile, the Abutment was determined to be structurally adequate. (See Appendix C for back up documentation)



NOTE:
THE CONTRACTOR SHALL VERIFY ALL CONTROLLING FIELD DIMENSIONS BEFORE ORDERING OR FABRICATING ANY MATERIAL.

CURVE DATA:

"SD15L" Line	"SD15M" Line	"SD15R" Line
R = 1992.990 m	R = 1899.100 m	R = 1914.110 m
Δ = 21°23'29"	Δ = 28°49'52"	Δ = 29°04'47"
T = 376.427 m	T = 488.154 m	T = 496.444 m
L = 744.088 m	L = 955.620 m	L = 971.483 m

Notes:

1. Paint "Br. No. 57-1134R"
2. Paint "Br. No. 57-1134L"
3. Paint "Lake Hodges Bridge"
4. Structure Approach Type N(9S)

For 'Typical Sections', see "General Plan No. 2" sheet.

For 'General Notes', 'Index To Plans', and 'Standard Plans', see "Index To Plans" sheet.

For 'Pile Data', see "Pile Data No. 1" and "Pile Data No. 2" sheets.

For deck drains, see "Deck Drain Layout" and "Deck Drain Details" sheets.

Legend:

- Indicates new structure
- Indicates existing structure
- Indicates point of minimum vertical clearance
- Minimum vertical clearance=3.350 m
- Standard Plan Sheet No.
- Detail No.

ALL DIMENSIONS ARE IN METERS UNLESS OTHERWISE SHOWN

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO	TOTAL SHEETS
11	SD	15	M38.7/M42.7	743	822

REGISTERED CIVIL ENGINEER DATE 8-6-04

PLANS APPROVAL DATE 4-12-04

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REGISTERED PROFESSIONAL ENGINEER Richard I. Schendel No. C64259 Exp. 6-30-07 CIVIL STATE OF CALIFORNIA

Caltrans Metric

DESIGN	BY Richard Schendel	CHECKED Tong Yang	LOAD FACTOR DESIGN	LIVE LOADING: HS20-44 AND ALTERNATIVE AND PERMIT DESIGN LOAD
DETAILS	BY Ton Doan/other	CHECKED Tong Yang	LAYOUT	BY Richard Schendel / CHECKED T. Yang
QUANTITIES	BY R. Mc Gough	CHECKED A. Wardak	SPECIFICATIONS	BY G. Magana / PLANS AND SPECS COMPARED G. Magana

DESIGN ENGINEER

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

DIVISION OF STRUCTURES
STRUCTURE DESIGN 12

BRIDGE NO. 57-1134R/L
KILOMETER POST 43.19

PHASE 2
LAKE HODGES BRIDGE
GENERAL PLAN NO. 1

STRUCTURES DESIGN GENERAL PLAN SHEET (METRIC) (REV.12-1-01)

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

CU 11
EA 080921

DISREGARD PRINTS BEARING EARLIER REVISION DATES

CONTRACT NO: P-080924
FIELD CORRECTION DATE: 09-10-08
FIELD CORRECTIONS BY: GabrielAcero

AS BUILT CORRECTIONS

ATTACHMENT 4

Notes:

(5) Temporary Railing Type K, see "Road Plans"

(6) Concrete Barrier Type 732

(7) Concrete Barrier Type 60A

(8) Concrete Barrier Type 80

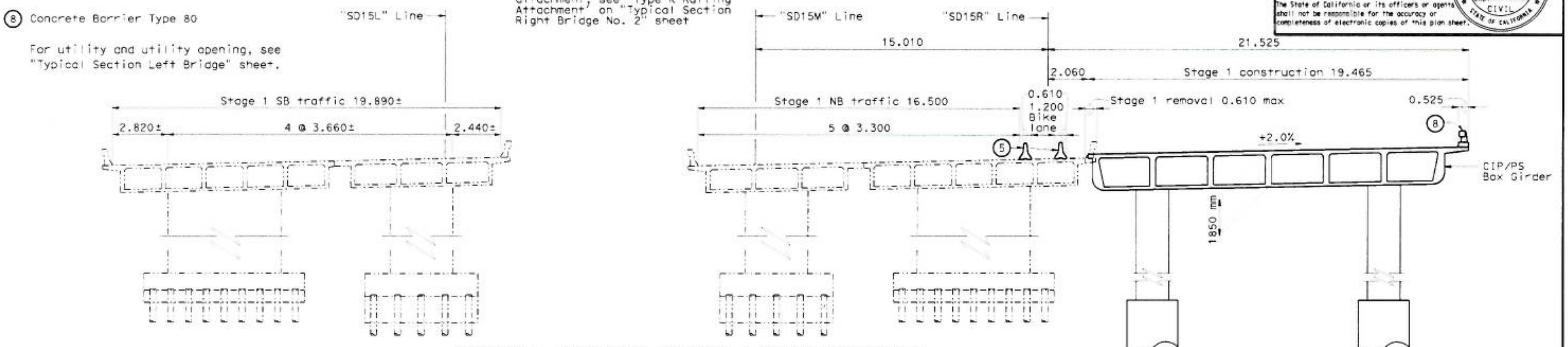
For utility and utility opening, see "Typical Section Left Bridge" sheet.

Legend:

——— Indicates new structure

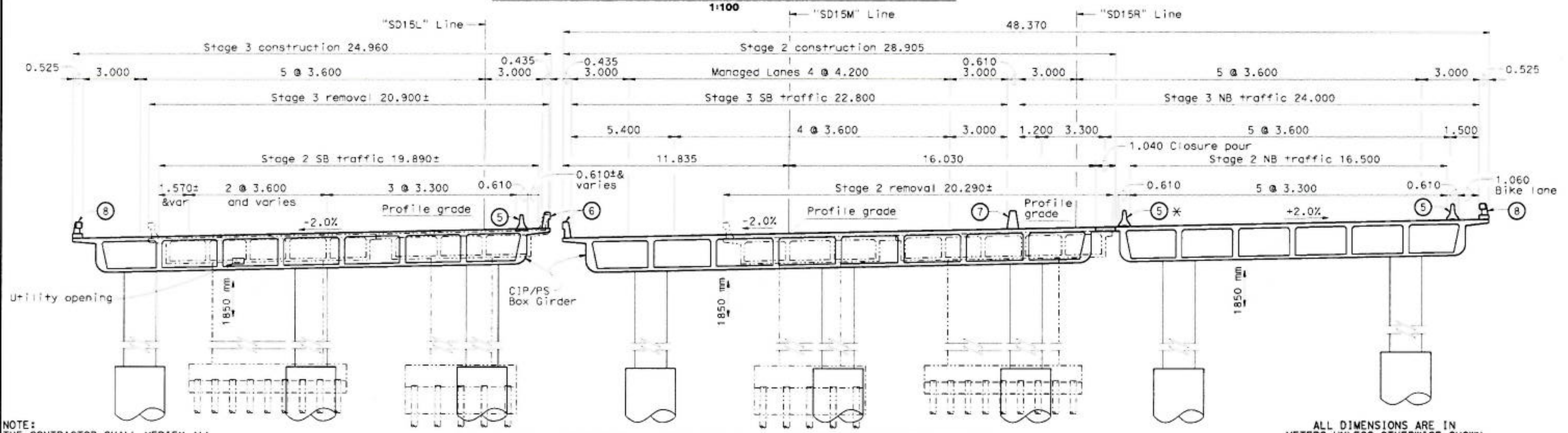
- - - Indicates existing structure

* For Temporary Railing (Type K) attachment, see 'Type K Railing Attachment' on "Typical Section Right Bridge No. 2" sheet



TYPICAL SECTION STAGE 1 CONSTRUCTION

1:100



NOTE:
THE CONTRACTOR SHALL VERIFY ALL CONTROLLING FIELD DIMENSIONS BEFORE ORDERING OR FABRICATING ANY MATERIAL.

TYPICAL SECTION STAGE 2 AND 3 CONSTRUCTION

1:100

ALL DIMENSIONS ARE IN METERS UNLESS OTHERWISE SHOWN

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO	TOTAL SHEETS
11	SD	15	M38.7/M42.7	744	822

REGISTERED CIVIL ENGINEER 8-6-04 DATE

4-12-04
PLANS APPROVAL DATE

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REGISTERED PROFESSIONAL ENGINEER Richard E. Schende No. C64259 Exp. 6-30-07 CIVIL STATE OF CALIFORNIA

Caltrans Metric

DESIGN	BY Richard Schendel	CHECKED Tong Yang	LOAD FACTOR DESIGN	LIVE LOADING: HS20-44 AND ALTERNATIVE AND PERMIT DESIGN LOAD	
DETAILS	BY Tan Doan/other	CHECKED Tong Yang	LAYOUT	BY Richard Schendel	CHECKED T. Yang
QUANTITIES	BY R. McCough	CHECKED A. Mordok	SPECIFICATIONS	BY G. Magana	PLANS AND SPECS COMPARED G. Magana

DESIGN ENGINEER

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

DIVISION OF STRUCTURES
STRUCTURE DESIGN 12

BRIDGE NO. 57-1134R/L
KILOMETER POST 43.19

PHASE 2

LAKE HODGES BRIDGE

GENERAL PLAN NO. 2

STRUCTURES DESIGN GENERAL PLAN SHEET (METRIC) (REV.12-1-01)

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

CU 11
EA 080921

DISREGARD PRINTS BEARING EARLIER REVISION DATES

CONTRACT NO. 11-080924
TRANSFER DATE: 09-22-11
FIELD CORRECTION DATE: 09-10-08
CORRECTIONS TRANSFERRED BY: Gabriel Acero

NO AS BUILT CORRECTIONS

ATTACHMENT 5

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET No	TOTAL SHEETS
11	SD	15	M38.7/M42.7	753	822

REGISTERED CIVIL ENGINEER 8-6-04 DATE

4-12-04
PLANS APPROVAL DATE

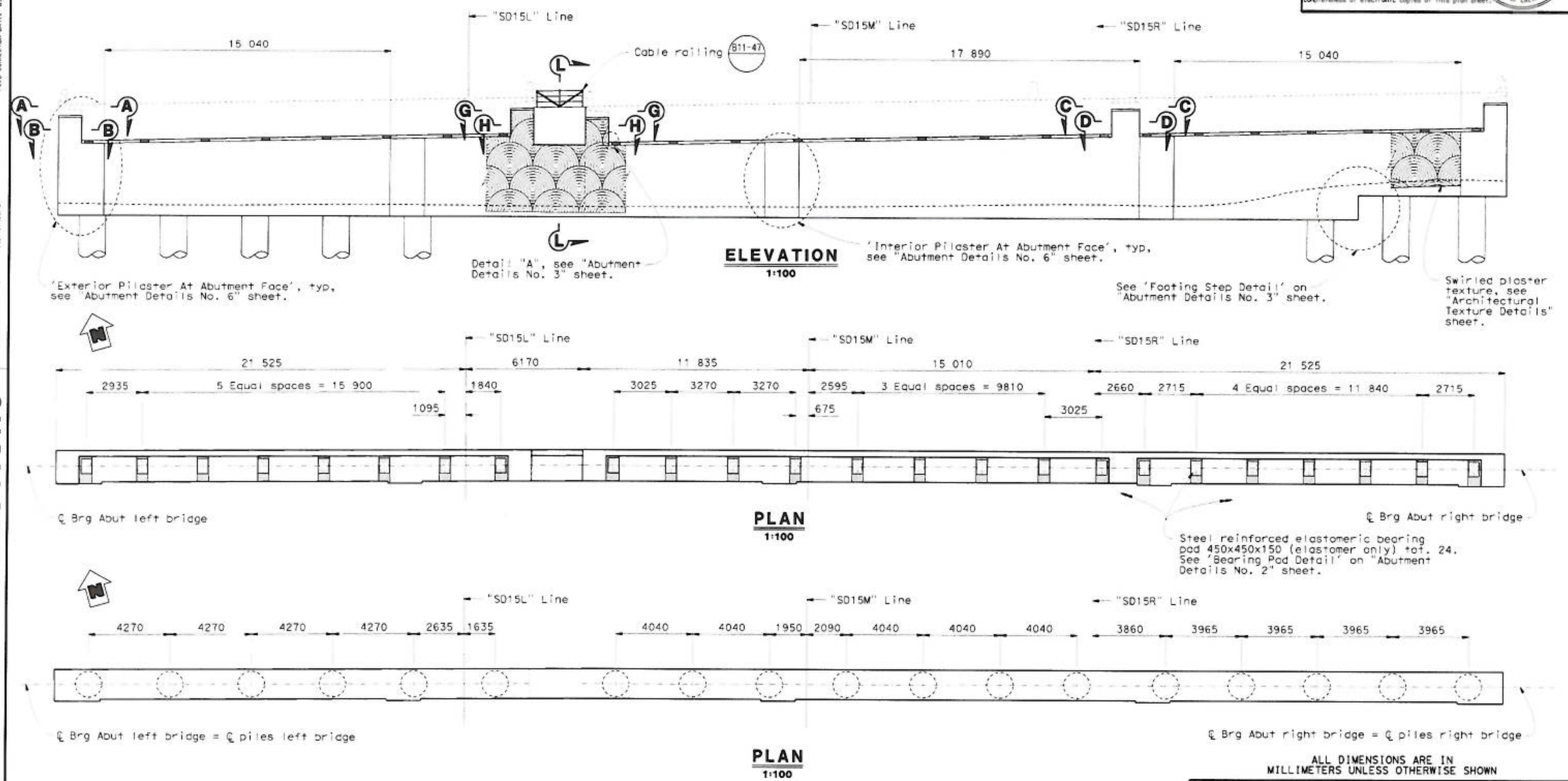
The State of California or its officers or agents shall not be responsible for the accuracy or completeness of electronic copies of this plan sheet.

REGISTERED PROFESSIONAL ENGINEER Richard E. Schendel No. C64259 Exp. 6-30-07 CIVIL STATE OF CALIFORNIA

Caltrans Metric

Notes:

1. Not all piles shown.
2. For Views 'A-A', 'C-C' and Sections 'B-B, 'D-D', see "Abutment Details No. 3" sheet.
3. For Sections 'G-G', 'H-H' and 'L-L', see "Abutment Details No. 5" sheet.



ALL DIMENSIONS ARE IN MILLIMETERS UNLESS OTHERWISE SHOWN

DESIGN	BY Richard Schendel	CHECKED Tong Yang	STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	DIVISION OF STRUCTURES STRUCTURE DESIGN 12
DETAILS	BY kc/other	CHECKED Tong Yang		
QUANTITIES	BY R. McCough	CHECKED A. Mardak		

BRIDGE NO. 57-1134R/L
KILOMETER POST 43.19

PHASE 2
LAKE HODGES BRIDGE
ABUTMENT 6 LAYOUT

CU 11
EA 080921

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

DISREGARD PRINTS BEARING EARLIER REVISION DATES

STRUCTURES DESIGN DETAIL SHEET (METRIC) (REV. 12-1-01)

CONTRACT NO: 11-080924
TRANSFER DATE: 09-22-11
FIELD CORRECTION DATE: 09-10-08
CORRECTIONS TRANSFERRED BY: kc
FIELD CORRECTIONS BY: GabrielAcero

NO AS BUILT CORRECTIONS

ATTACHMENT 6

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET No	TOTAL SHEETS
11	SD	15	M38.7/M42.7	754	822

REGISTERED CIVIL ENGINEER DATE 8-6-04

4-12-04
PLANS APPROVAL DATE

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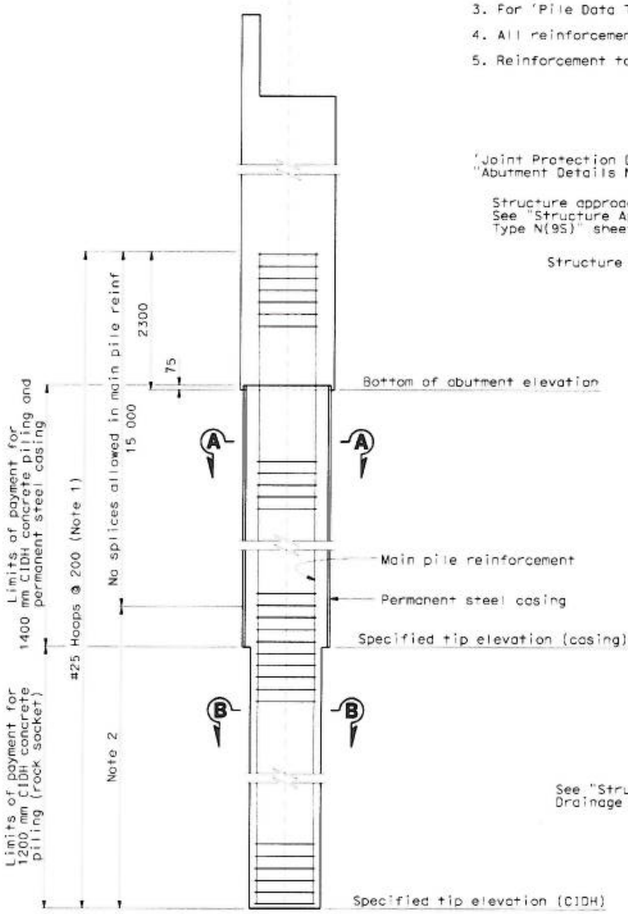
REGISTERED PROFESSIONAL ENGINEER Richard C. Schendel No. C64258 Exp. 6-30-07 CIVIL STATE OF CALIFORNIA

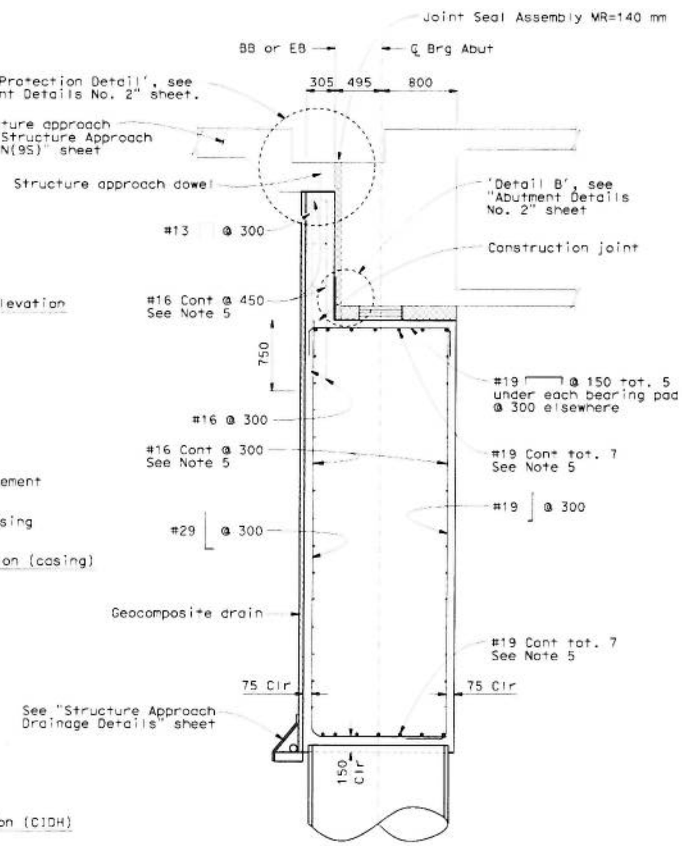
Caltrans Metric

Notes:

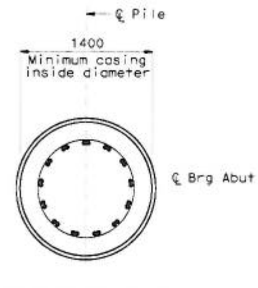
1. All hoops are "ultimate" butt spliced continuous.
2. Only staggered "ultimate" butt splices are allowed in main pile reinforcement in this zone.
3. For 'Pile Data Table', see "Pile Data No. 1" sheet.
4. All reinforcement not shown.
5. Reinforcement to be spliced at construction joints.



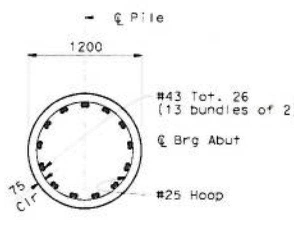
ELEVATION
1:40



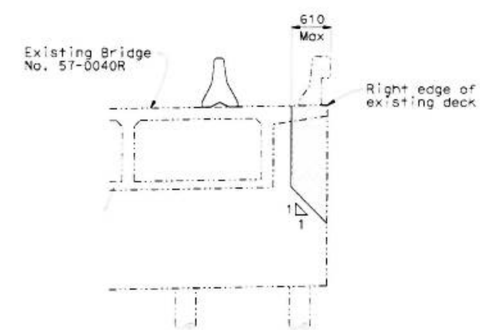
TYPICAL ABUTMENT SECTION
1:25



SECTION A-A
1:25



SECTION B-B
1:25



MAXIMUM LIMITS OF STAGE 1 REMOVAL
1:40

Bridge removal

ALL DIMENSIONS ARE IN MILLIMETERS UNLESS OTHERWISE SHOWN

DESIGN	BY Richard Schendel	CHECKED Tong Yang/ Madhwesh R'achar
DETAILS	BY kc/other	CHECKED Tong Yang
QUANTITIES	BY R. McGough	CHECKED A. Wardak

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

DIVISION OF STRUCTURES
STRUCTURE DESIGN 12

BRIDGE NO. 57-1134R/L
KILOMETER POST 43.19

PHASE 2
LAKE HODGES BRIDGE
ABUTMENT DETAILS NO. 1

CU 11
EA 080921

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

DISREGARD PRINTS BEARING EARLIER REVISION DATES

STRUCTURES DESIGN DETAIL SHEET (METRIC) (REV. 12-1-01)

CORRECTIONS TRANSFERRED BY: kc
FIELD CORRECTIONS BY: Gabriel Acero
TRANSFER DATE: 09 22 II
FIELD CORRECTION DATE: 09-10-08
CONTRACT No: 11-080924

NO AS BUILT CORRECTIONS

ATTACHMENT 7

5.4 STRUCTURAL EVALUATION FOR ID - 12

I. PROJECT DESCRIPTION: I-580 SINGLE POST TUBULAR OVERHEAD SIGN No. 19

POTENTIAL PILE ANOMALY IN CIDH PILE FOUNDATION

II. STRUCTURAL EVALUATION

Overhead sign number 19 is located in Alameda County on I-580 at post mile 36.2. The sign is a standard Plan sign (see Attachment 8) for a single post tubular sign structure supported on a 5 foot diameter CIDH pile that is 33 feet deep. The pile was constructed with five PVC inspection tubes. Based on the GGL test results an anomaly was found at a depth of 9.3 to 10.2 feet below the top of pile. The pile was rejected as a result of the test results and the anomaly was repaired.

Based on the GamDat teams investigation and recommendations, there were two locations near the tip of the pile where the GGL results were identified as questionable and required further structural evaluation. These two locations were at a depth of 28.9 feet and 30.1 feet below the top of the pile. The questionable data affected 20% of the cross section (1 tube) at the first location and affected 40 % of the cross section (2 tubes) at the second location. Demand/Capacity values for moment and shear were calculated by the evaluator and there was significantly more capacity than demand because of the fact that these two locations are close to the tip of the pile. The results are summarized in Table 2.

As a result of this analysis the pile was determined to be adequate. (See Appendix D for additional backup documentation, as-built plans, and Geotechnical Foundation Assessment report).

Table 2 I-580 Overhead Sign Analysis Results

Location	Force	Capacity	Demand	Factor of Safety
Section B-B 28.9 feet below top of pile	Shear	101 kips (450 kN)	14 kips (62 kN)	7
	Bending Moment	2986 kip-ft (4048 kN-m)	33 kip-ft (45 kN-m)	90
Section C-C 30.1 feet below top of pile	Shear	49 kips (216 kN)	11 kips (49kN)	4
	Bending Moment	1470 kip-ft (1993 kN-m)	21 kip-ft 28 kN-m)	70

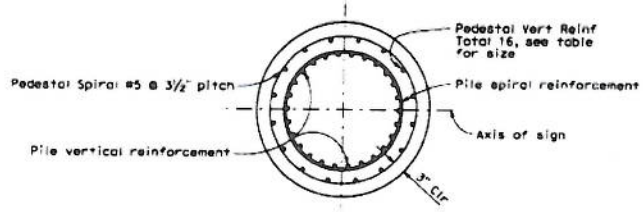
DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET NO	TOTAL SHEETS

REGISTERED CIVIL ENGINEER

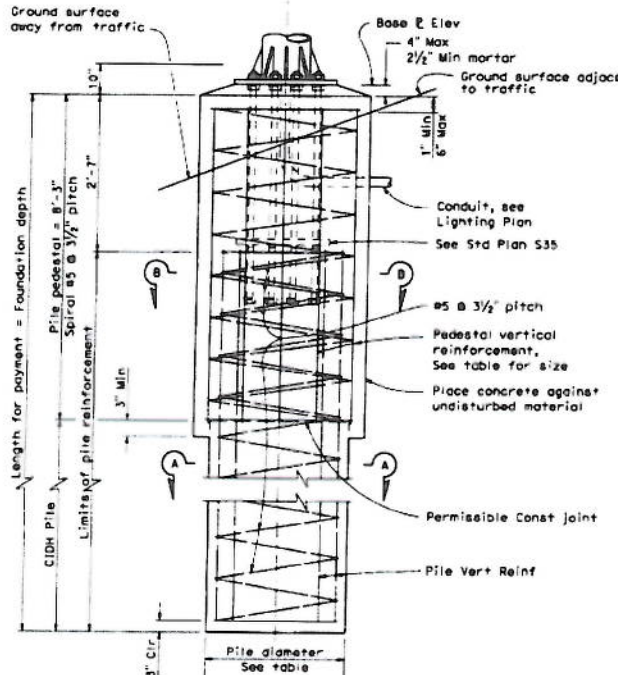
May 1, 2006

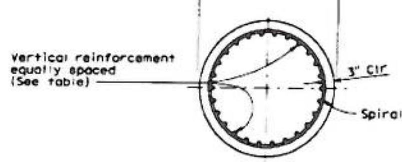
PLANS APPROVAL DATE

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



SECTION B-B





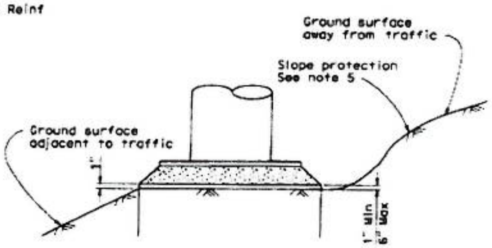
SECTION A-A

Post Type No.	Anchor Bolts: Bolt Circle Dia	Anchor Bolts: Bolt Total and Dia	Anchor Bolts: Total Length	Round Pile Pedestal: Dia	Round Pile Pedestal: Vertical Reinforcing Total	Round Pile Pedestal: Vertical Reinforcing Bar Size	Round Pile Pedestal: Spiral Bar Size	Round Pile Pedestal: Spiral Pitch	CIDH Pile: Pile Dia	CIDH Pile: Vertical Reinforcing Total	CIDH Pile: Vertical Reinforcing Bar Size	CIDH Pile: Spiral Bar Size	CIDH Pile: Spiral Pitch	Foundation Depth * *
I	2'-6"	14-2"	4'-2"	5'-6"	16	#11	#5	3½"	5'-0"	28	#11	#5	3½"	25'-0"
II	2'-10"	14-2½"	5'-0"											25'-0"
III	2'-10"	14-2½"												25'-0"
IV	3'-4"	16-2½"												33'-0"
V	3'-4"	16-2½"												33'-0"
VI	3'-4"	16-2½"												33'-0"

* * Use Foundation Depth shown in table unless otherwise shown on the Project Plans.

NOTES:

1. For anchor bolt layout, see Standard Plan S35.
2. For "Base ℄ elevation", see Project Plans.
3. Prior to erection of the post, backfill which is equivalent to the surrounding material, shall be in place.
4. Pedestal shall be formed 6" minimum below ground surface Remainder to be placed against undisturbed material.
5. Slope protection required when indicated on the Project Plans.
6. Foundation design is based on 2001 ASSHTO article 13.6 Broms' approximate procedure assuming a cohesionless material. The angle of internal friction Dia used 30 degree and unit weight of soil used is 120 lb/ft³.



DETAIL C

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

OVERHEAD SIGNS-TUBULAR SINGLE POST AND TWO POST TYPE ROUND PEDESTAL PILE FOUNDATION

NO SCALE

S37

ATTACHMENT 8

5.5 STRUCTURAL EVALUATION FOR ID - 18

I. PROJECT DESCRIPTION: SAWTELL BLVD UC, RETAINING WALL 435B

POTENTIAL PILE ANOMALY IN CIDH PILE FOUNDATION

II. STRUCTURAL EVALUATION

Retaining Wall 435 B is a specially designed cantilevered retaining wall supported on concrete driven piles. This section of retaining wall was redesigned by a contract change order (see Attachment 9) to accommodate a water line under the retaining wall. This was done by using 24 inch CIDH piles which replaced the original driven piles to eliminate vibration sensitivity. This segment of retaining wall consists of sixty eight 24 inch CIDH piles. (See Attachment 9)

The GGL pile testing results showed that no anomalies were present and all piles were accepted during construction. As a result of the Gam-Dat team investigation there were two piles in question that require structural re-evaluation. The piles in question are numbered 69 and 72.

The designer removed the piles from his retaining wall model and calculated revised foundation demands for the controlling load combination (See Table 3). He determined that the revised demands on the retaining wall foundation were still well below the capacity with piles numbers 69 and 72 removed from the analysis.

As a result of this analysis the retaining wall design was determined to be adequate. (See Appendix E for backup documentation and additional as-built plans).

Table 3 - RW 435B Analysis Results (with piles removed)

LOAD CASE	DESCRIPTION	CAPACITY(kips)	DEMAND(kips)	Factor of Safety
SERVICE LOADS	TOTAL LATERAL LOAD	1716	1463	1.17
	MAX HEEL LOAD	140	18	7.63
	MAX TOE LOAD	140	60	2.33
FACTORED LOADS	TOTAL LATERAL LOAD	2574	2217	1.16
	MAX HEEL LOAD	196	17	11.53
	MAX TOE LOAD	196	83	2.36

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
07	LA	405	41.6/47.9		

REGISTERED CIVIL ENGINEER 5-22-03

PLANS APPROVAL DATE 2-2-04

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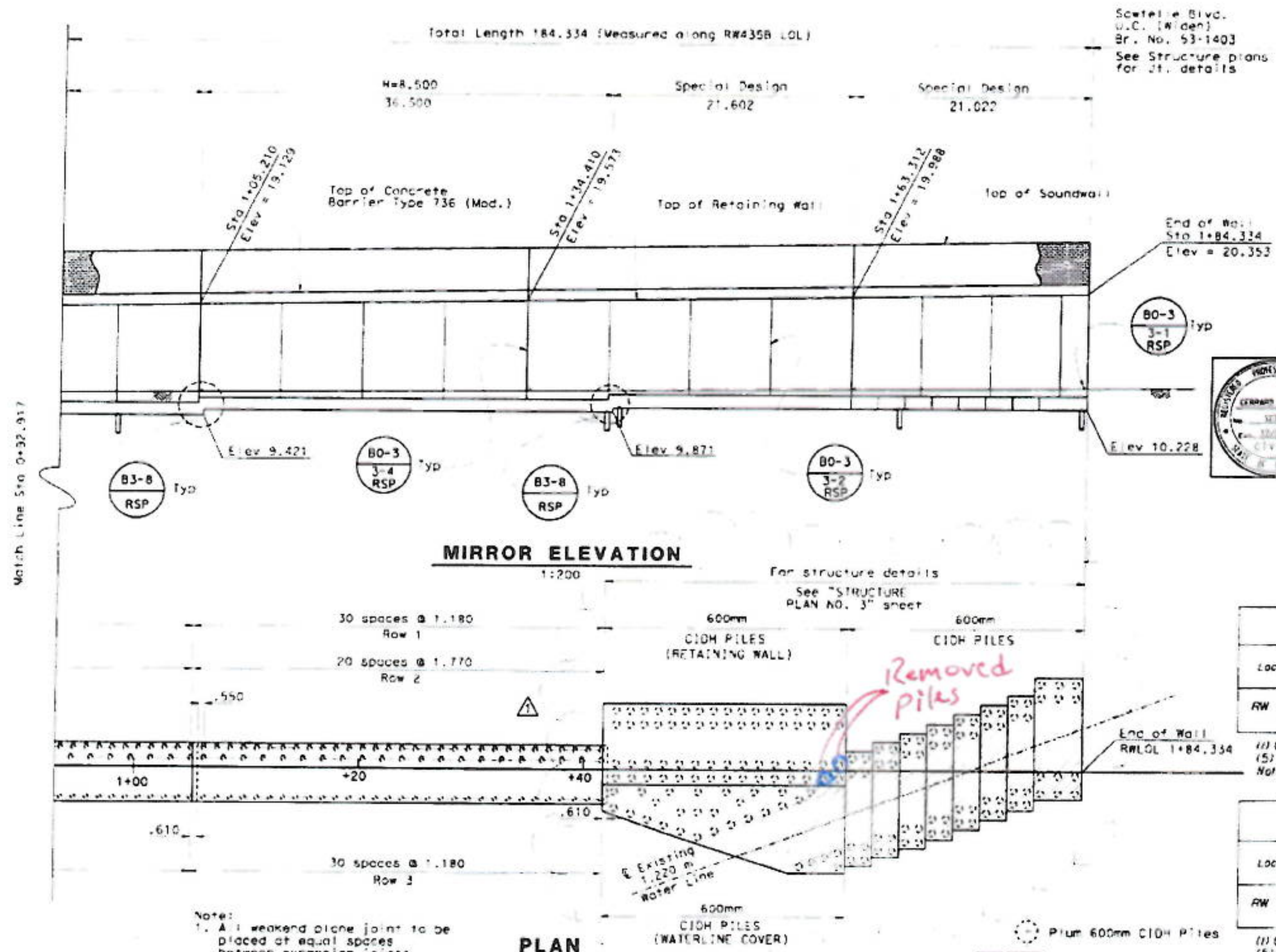


Note:
1. All weakend plane joint to be placed at equal spaces between expansion joints.
2. The minimum distance between the edge of the piles and the edge of the waterline to be 2 times the piles diameter or greater.

Note:
All weakend plane joint to be placed at equal spaces between expansion joints.

NOTE:
THE CONTRACTOR SHALL VERIFY ALL CONTROLLING FIELD DIMENSIONS BEFORE ORDERING OR FABRICATING ANY MATERIAL.

MARK	DATE	REVISION(S) DESCRIPTIONS	DES	CHK	DET
1		REVISED FOOTING & PILES	GH	LZ	SD

REGISTERED CIVIL ENGINEER: GERRARD HIGHT DATE: 8/16/06

CONTRACT CHANGE ORDER NO. ____
SHEET ____ OF ____

TIP ELEVATION FOR 600mm CIDH PILES (WATERLINE COVER)

Location	Design Loading (kN)	Nominal Resistance Compression	Nominal Resistance Tension	Design Tip Elevation	Specified Tip Elevation
RW 435B	800	800kN	800kN	-2.0 (1)	2.0

(1) Compression; (2) Tension; (3) Lateral loads; (4) No scour anticipated; (5) Potential for liquefaction at all supports are considered low.
Note the final grade elevation assumed to be 12.0 meters (MSL).

TIP ELEVATION FOR 600mm CIDH PILES (RETAINING WALL)

Location	Design Loading (kN)	Nominal Resistance Compression	Nominal Resistance Tension	Design Tip Elevation	Specified Tip Elevation
RW 435B	625	625kN	1250kN	1.5 (1)(2)(3)	1.5

(1) Compression; (2) Tension; (3) Lateral loads; (4) No scour anticipated; (5) Potential for liquefaction at all supports are considered low.
Note the final grade elevation assumed to be 12.0 meters (MSL).

ALL DIMENSIONS ARE IN METERS UNLESS OTHERWISE SHOWN

DESIGN	BY Pao Mout	CHECKED G. Hight	
DETAILS	BY D. Elliott	CHECKED G. Hight	
QUANTITIES	BY G. Hight	CHECKED D. Kim	

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

DIVISION OF STRUCTURES
STRUCTURE DESIGN 18

BRIDGE NO. N/A
KILOMETER POST 43.414

RETAINING WALL NO. 435B
STRUCTURE PLAN NO. 2

CU 07
EA 117801

ATTACHMENT 9

5.6 STRUCTURAL EVALUATION FOR ID - 52

I. PROJECT DESCRIPTION: BRADDOCK DR. UC, ABUT 1 RIGHT RETAINING WALL

POTENTIAL PILE ANOMALY IN CIDH PILE FOUNDATION

II. STRUCTURAL EVALUATION

The right retaining wall of abutment 1 of the Braddock Dr. UC (widen) consists of a cantilevered retaining wall supported on concrete driven piles. This section of retaining wall was redesigned by a contract change order to accommodate a water line under the retaining wall (see Attachment 10). 24 inch CIDH piles replaced the original driven piles to eliminate vibration sensitivity. The as constructed segment of retaining wall consists of sixteen 24 inch CIDH piles in four rows of four piles each. (See Attachment 10)

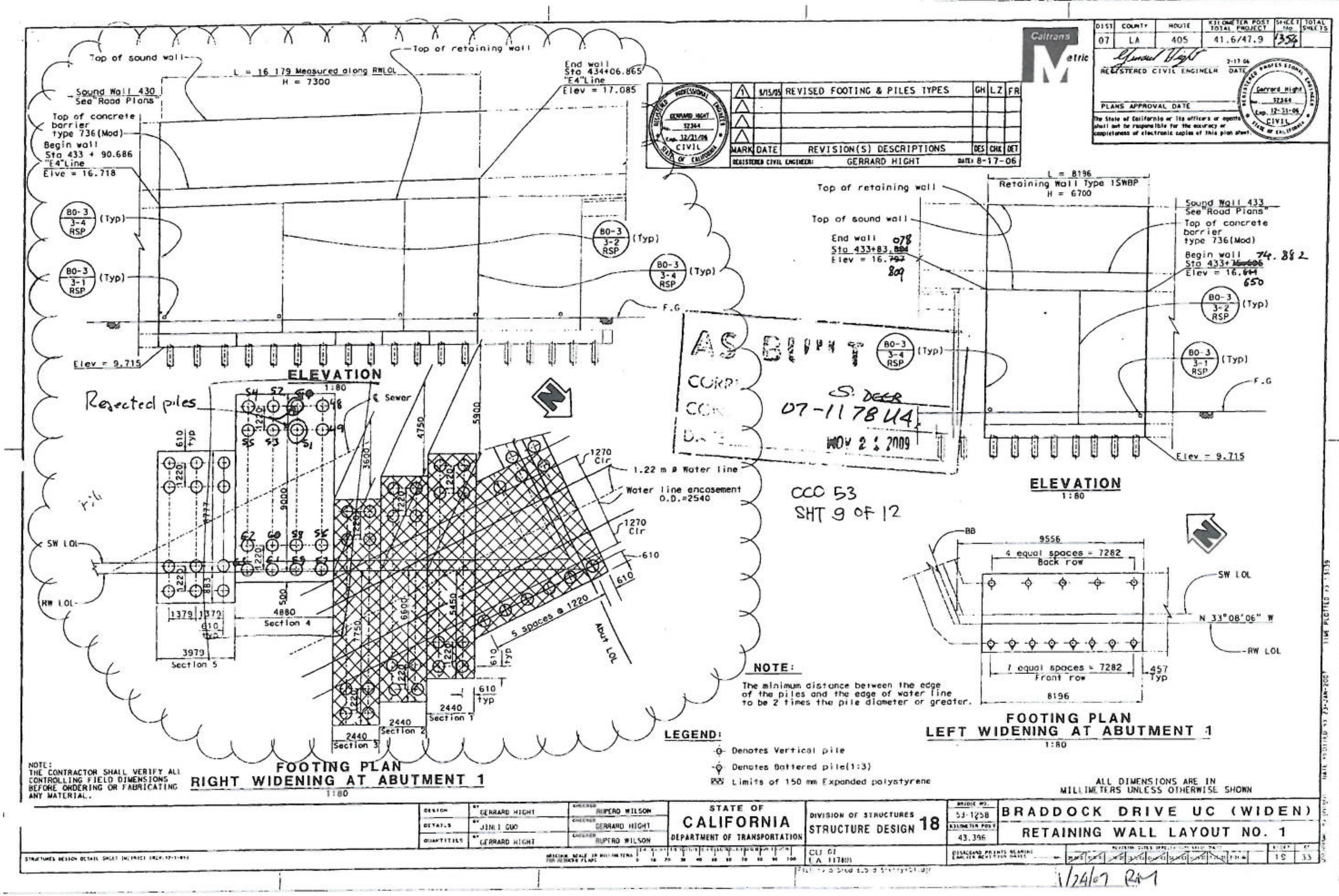
As a result of the GamDat team's investigation there were two piles in question that needed re-evaluation. These piles were pile number 50 and 51. The GGL test results conducted during construction for these piles showed some anomalies were present in one to two tubes of the total 4 tubes placed in the 24 inch piles. These anomalies were at a depth of 26.3 feet from the top of pile, close to the tip of the pile. For pile number 50 the anomaly detected was in two of the four tubes and in pile number 51 the anomaly detected was in one tube and determined by the GGL report to be insignificant and not requiring a structural evaluation. Both piles were accepted during construction with no repair.

As part of the structural assessment for these two piles, the designer removed the piles from his analysis and looked at the revised foundation demands for the controlling load cases (See Table 4), and determined that the revised loads as a result of removing these two piles were still well below capacity.

As a result of this analysis the retaining wall design was determined to be adequate. (See Appendix F for backup documentation and additional as-built plans).

Table 4 – Braddock Retaining Wall Analysis Results (with piles removed)

LOAD CASE	DESCRIPTION	CAPACITY(kips)	DEMAND(kips)	Factor of Safety
SERVICE LOADS	TOTAL LATERAL LOAD	364	195	1.87
	MAX HEEL LOAD	140	121	1.16
	MAX TOE LOAD	140	105	1.33
FACTORED LOADS	TOTAL LATERAL LOAD	431	328	1.31
	MAX HEEL LOAD	210	159	1.32
	MAX TOE LOAD	210	140	1.50



ATTACHMENT 10

5.7 STRUCTURAL EVALUATION OVERVIEW FOR ID's – 17, 33, 76

I. PROJECT DESCRIPTION: BENICIA-MARTINEZ MAIN SPANS

PIER6-PILE 5, PIER 12-PILE 6, PIER 14-PILE 8, POTENTIAL PILE ANOMALY IN CIDH PILE

II. STRUCTURAL EVALUATION

The new Benicia Martinez Bridge is a 1.4 mile long bridge constructed across the Carquinez Strait, west of Suisun Bay, and opened to traffic in late 2007 (see Appendix G for pertinent plan sheets). The marine pier foundations consist of a pile cap at the water level supported on eight or nine CIDH piles at each pier location. The CIDH piles consist of an upper portion that is 2.4m in diameter with a permanent 41mm (1-5/8 in) thick steel casing, and a lower uncased rock socket that was 2.2m in diameter.

Both Gamma-Gamma Logging (GGL) and cross-hole sonic testing (CSL) were performed on all of the marine pier piles in question. Eight PVC inspection tubes were placed in the outer region of the pile reinforcing cage. The upper portion of the cased piles was constructed in the dry therefore GGL and CSL tests were not performed above the construction joint.

During construction various anomalies were detected by the GGL testing in these 3 piles at various depths. As is normally done with this type of pile construction these anomalies were reviewed, analyzed and either accepted without repair or repaired per the contract.

The structural evaluation compared the demand loads with the reduced cross-sectional capacity. The reduced pile capacities were based on the tubes identified in the GamDat report as questionable in conjunction with previously identified anomalies that were accepted without repair during construction. Table 5 lists the piers, pile number and questionable tube number.

Table 5 Benicia Martinez Questionable Inspection Tubes

Case ID	Pier Location	Pile Number	Tube number
ID – 17	Pier 6	Pile 5	5
ID – 33	Pier 12	Pile 6	6 or 7*
ID – 76	Pier 14	Pile 8	3

*The GGL data for these two tubes was identical suggesting one of the data sets was reused as data for the adjacent tube.

The structural evaluation found a relatively small decrease in bending capacities for all three piles, less than 9%, which is attributed to several factors.

- Only one tube was considered questionable in each of the piles. In some cases an additional one to two tubes were also discounted to account for previously identified anomalies that were accepted and not repaired during the construction.
- The upper portion of the piles have a permanent steel casing which provides a large portion of the moment capacity, thereby reducing the effect of the discounted concrete region of the pile.
- The cross-hole sonic (CSL) testing performed on these piles showed that the interior core of the piles had no significant anomalies, which reduced the pie shaped section or the affected concrete region to the outer sector of the pile beyond the inspection tubes (see Appendix G for details).

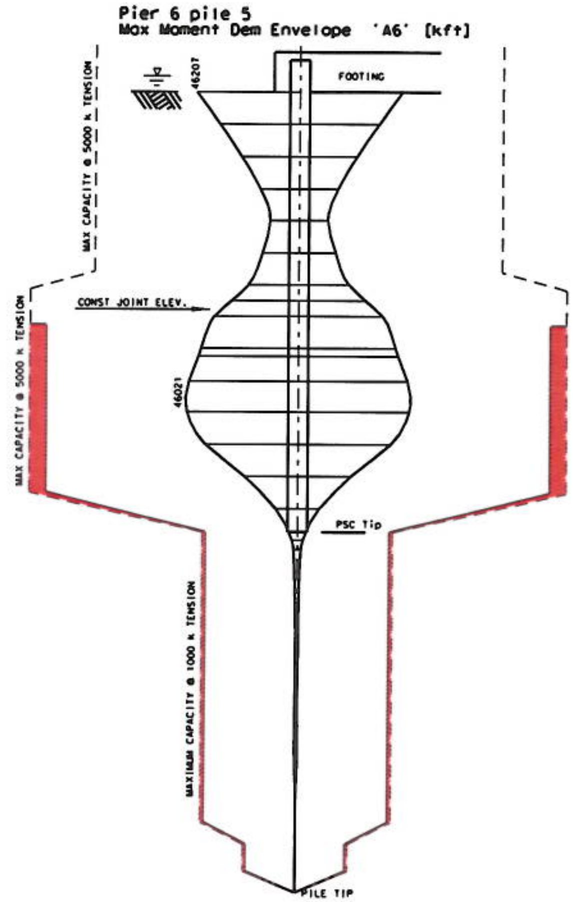
The reduced capacities were compared to the demand values extracted from the ADINA displacement time history dynamic analysis model archived for the bridge. The Office of Earthquake Engineering Analysis & Research (OEEAR) maintains archived analytical seismic models for the state's Toll bridges including the model used on the design of the new Benicia Martinez Bridge. As part of the archival process the consulting firm who prepared the archive model was required to verify the model by re-running the time-history analyses and comparing their results to results previously compiled by the project designer. OEEAR ran the model with the (3) time-histories that were provided with the model and extracted the axial force and bending moment results for the three piles. In addition, the axial force and moment results were provided for the lower pier elements. The piles and piers were modeled with moment-curvature elements and as such, only axial force and moment results are available.

Pile moment demands were extracted from the ADINA model for all three pile locations for three different scenarios: The maximum resultant top of pile moment at a single time step and associated lower pile moments and axial loads, the maximum resultant moment demand along all pile elements at any discrete time step and associated axial load, and the resultant moment demand in the pile associated with minimum axial load (maximum tension or least compression) at any time step. Plots of the three load envelopes for each of the three time histories can be found in Appendix G and on Attachments 11, 12 & 13. As can be seen from these plots the demand load, for all three piles in question, is below the reduced capacity for the entire length of the pile cast under the slurry displacement method.

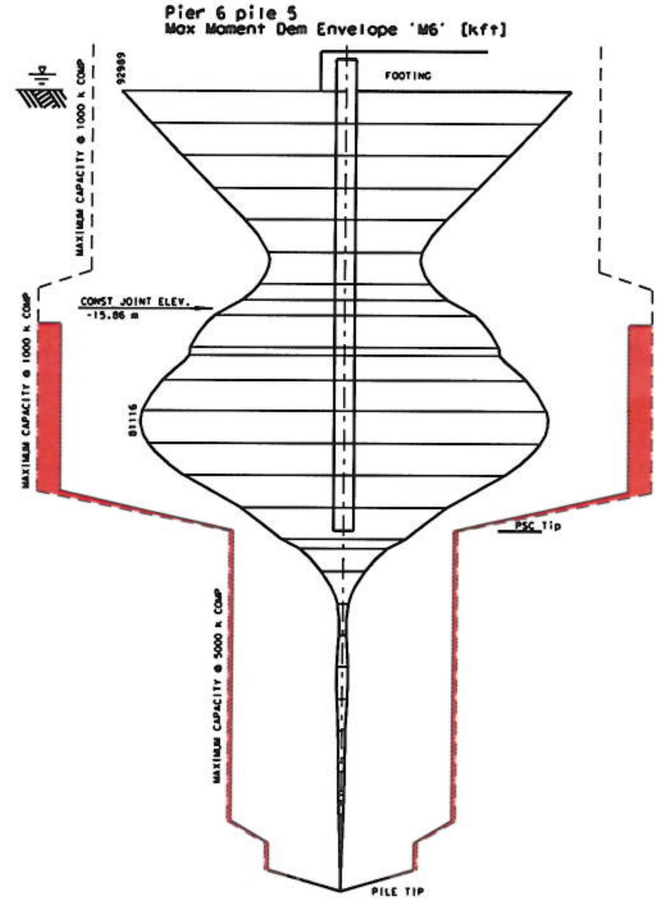
Based on the structural evaluation these three piles are considered structurally adequate and are not adversely affected by the assumed loss in capacity commensurate with ignoring the capacity attributed to the area of the pile associated with the irregular GGL data identified in the Gam-Dat Report.

Attachment 11

D6



TENSION



COMPRESSION

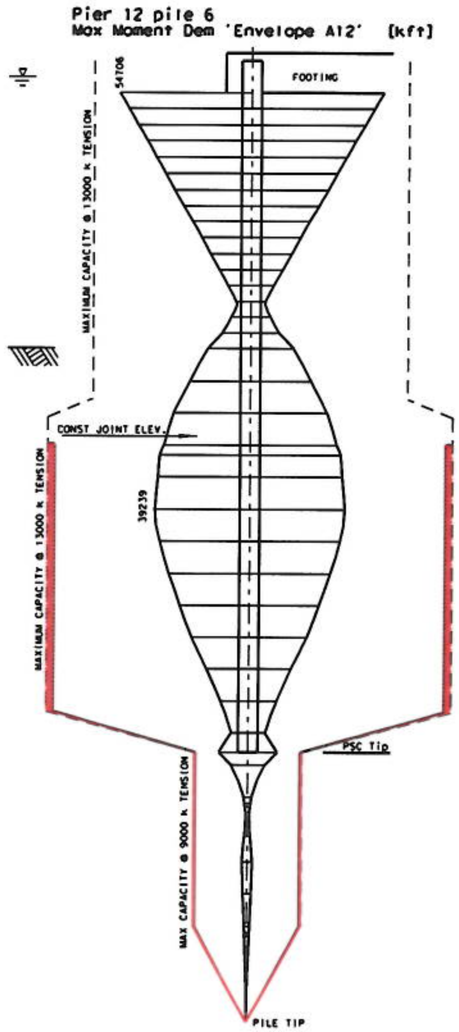
CAPACITY REDUCTION CREATED BY POSSIBLE ANOMALY

Benicia Main Span Anomaly Review
Pier 6, pile 5
Max of 3 time history Pile Moment Demands (A6 & M6)

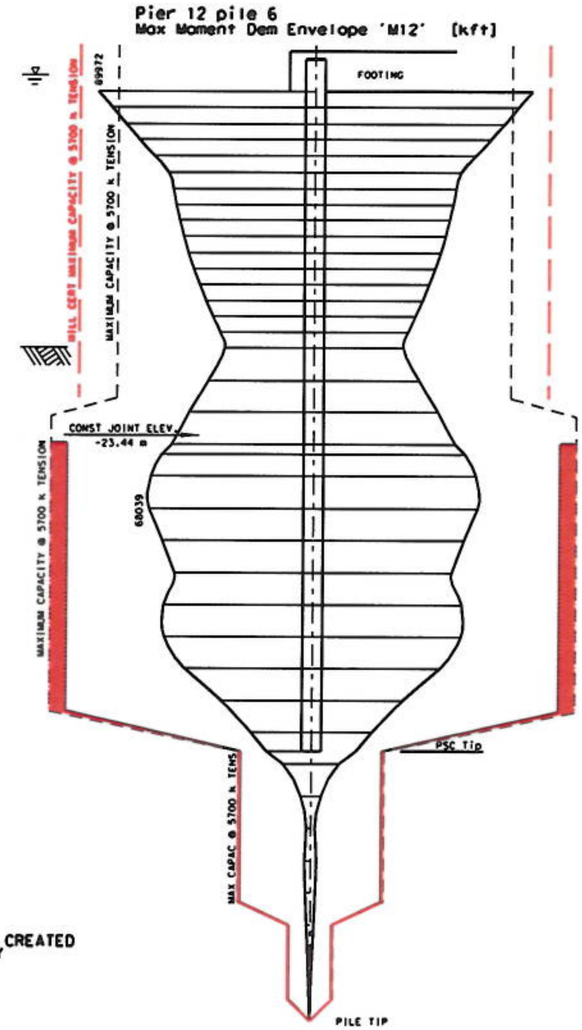
NewBM_P6_12_14_Diag_Final.dgn 1/24/2013 7:56:21 AM

Attachment 12

D12



TENSION 1



TENSION 2

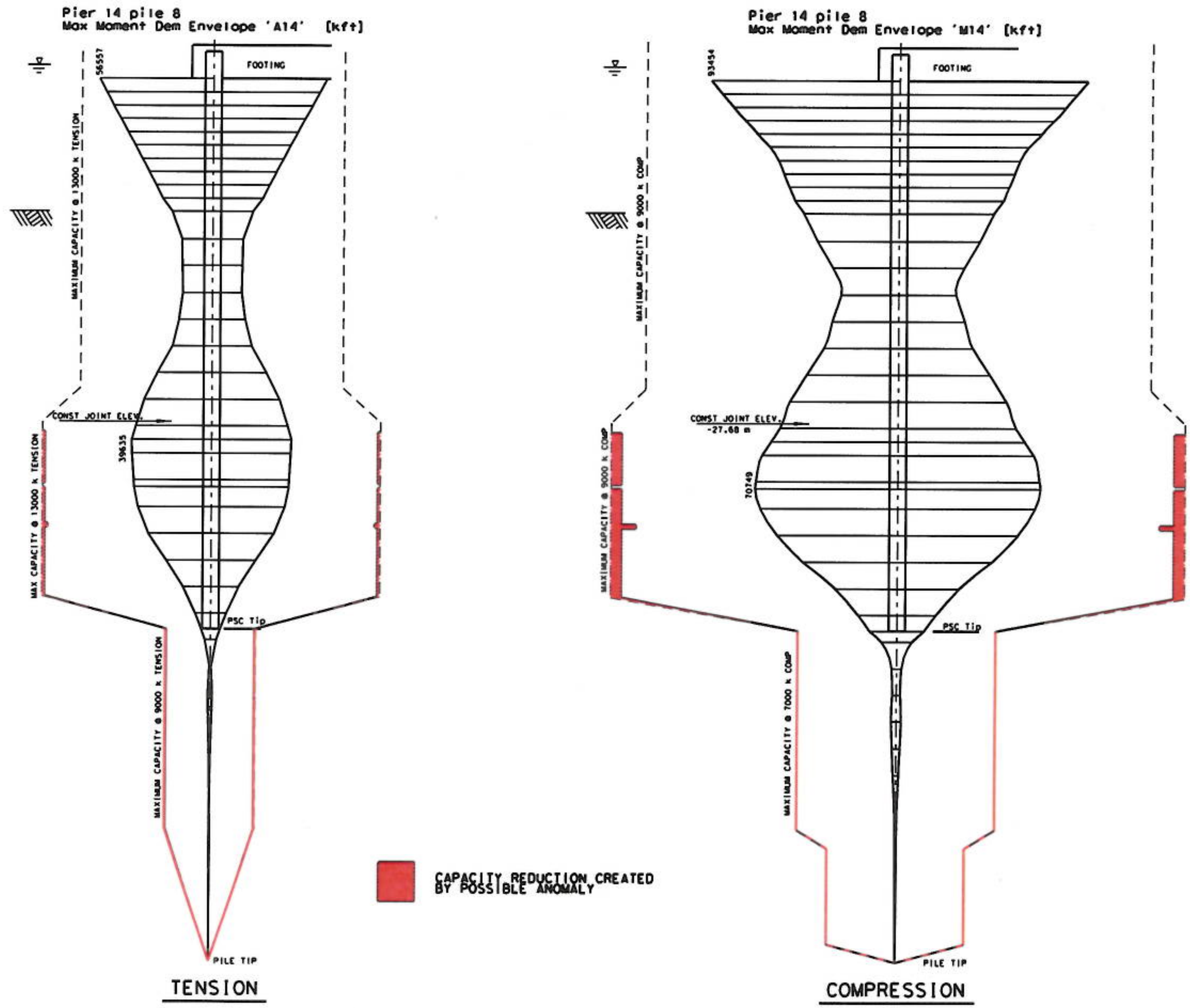
CAPACITY REDUCTION CREATED BY POSSIBLE ANOMALY

Benicia Main Span Anomaly Review
Piers 12, pile 6
Max of 3 time history Pile Moment Demands (A12 & M12)

NewBM_P6_12_14_Diag_Final.dgn 1/24/2013 7:58:31 AM

Attachment 13

D14



Benicia Main Span Anomaly Review
Pier 14, pile 8
Max of 3 time history Pile Moment Demands (A14 & M14)

NewBM_P6_12_14_Diag_Final.dgn 1/24/2013 8:00:38 AM

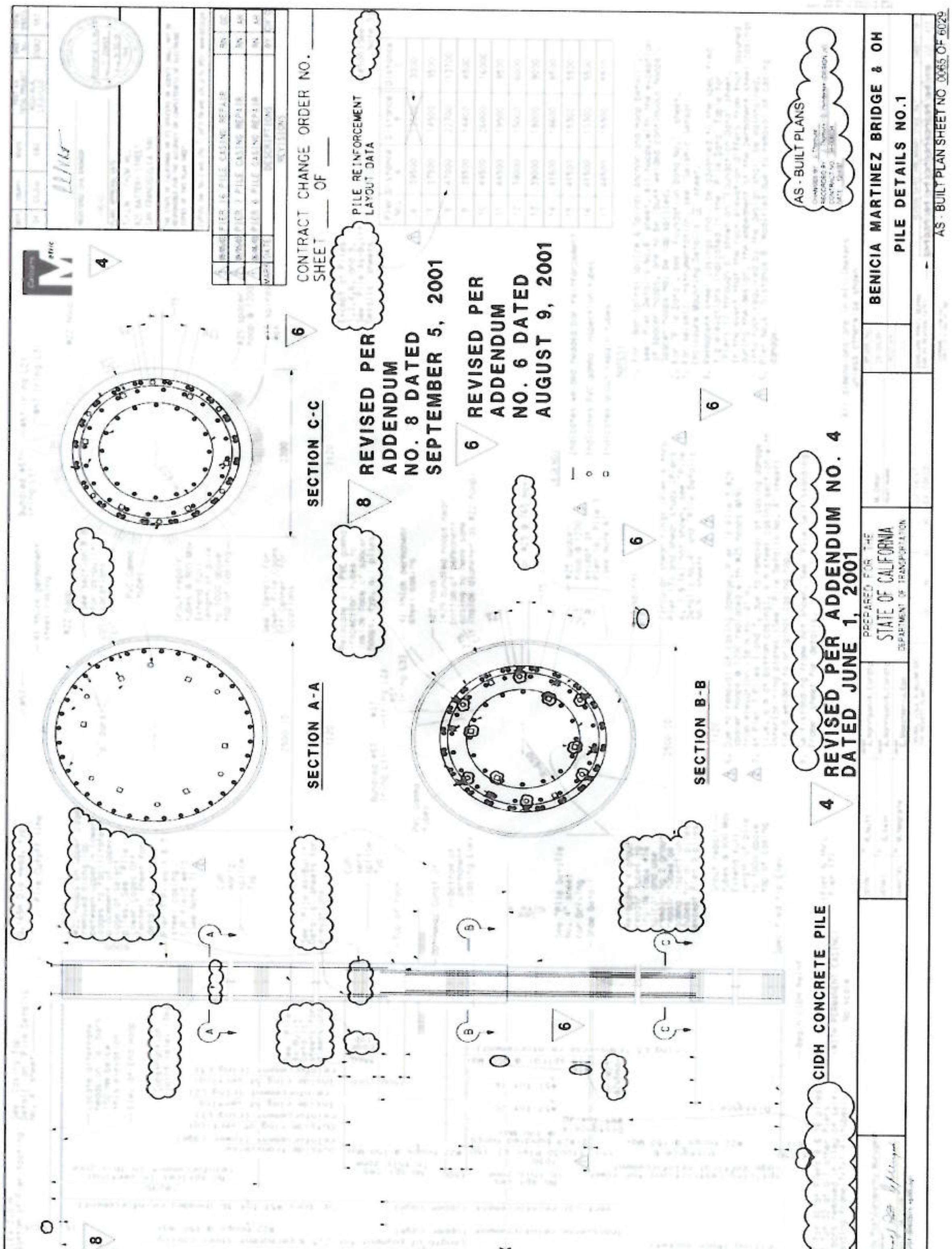
Benicia Martinez Structural Capacity Evaluation

Pile Description for Analysis

There are four distinct regions where strength and stiffness change along the length of the piles in question (see Attachment 14). The upper two regions feature the 41mm (1 $^{5}/_{8}$ inch) thick grade 50 steel casing that has a 2.5 meter (8.2 ft.) inside diameter. Main reinforcement consists of #18 bars that are much more numerous in the lower portion of the cased region. This provides increased bending capacity with depth as will be discussed later. The casing extends well into the footing, and features shear rings at both top and bottom which enhance the abiltiy to utilize the pipe properties a short distance from each end.

The lower two regions of the pile are not cased and have a diameter of 2.2 meters (7.2 ft). Main reinforcement for these regions is the reverse of the cased region, as #18 bars are much less plentiful near the bottom of the pile where bending demands are minimal. Shear capacity is enhanced along all four regions of the pile by use of reinforcement hoops.

Attachment 14 As-Built Pile Details



Pile Data Extraction for Analysis

Data extraction from the three time-history ADINA finite element model runs showed great variations in both the bending moment and axial load demands for each element along the piles. For example, Figure 3 shows variations in axial load demand with respect to time for the top element of pile 6 at Pier 12.

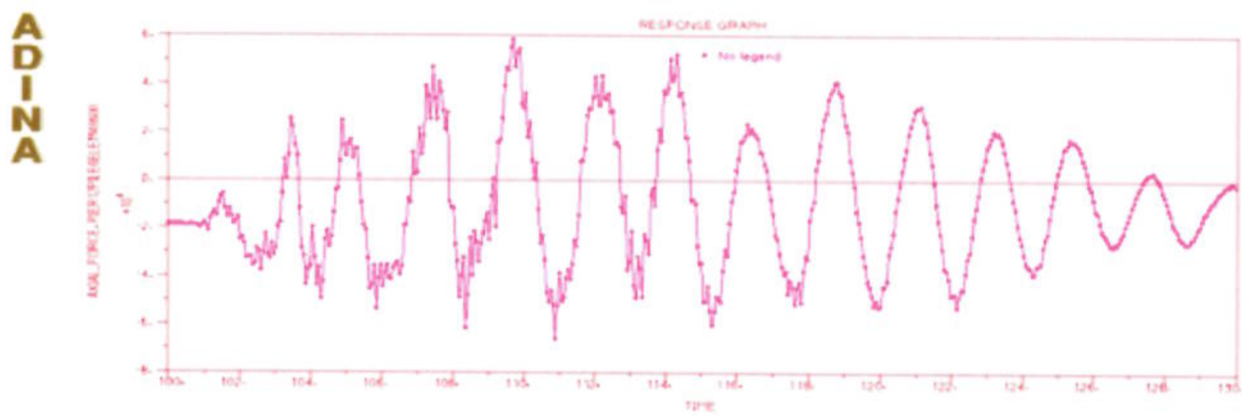


Figure 3: Pier 12 pile 6 axial load vs time plot for top of pile element, time-history set 1

ADINA reports compression loads as negative. Note the peak uplift spike of +58561 KN (13164 kips) near time 110. Figure 4 shows variations in resultant bending moment demand with respect to time for the same top of pile element.

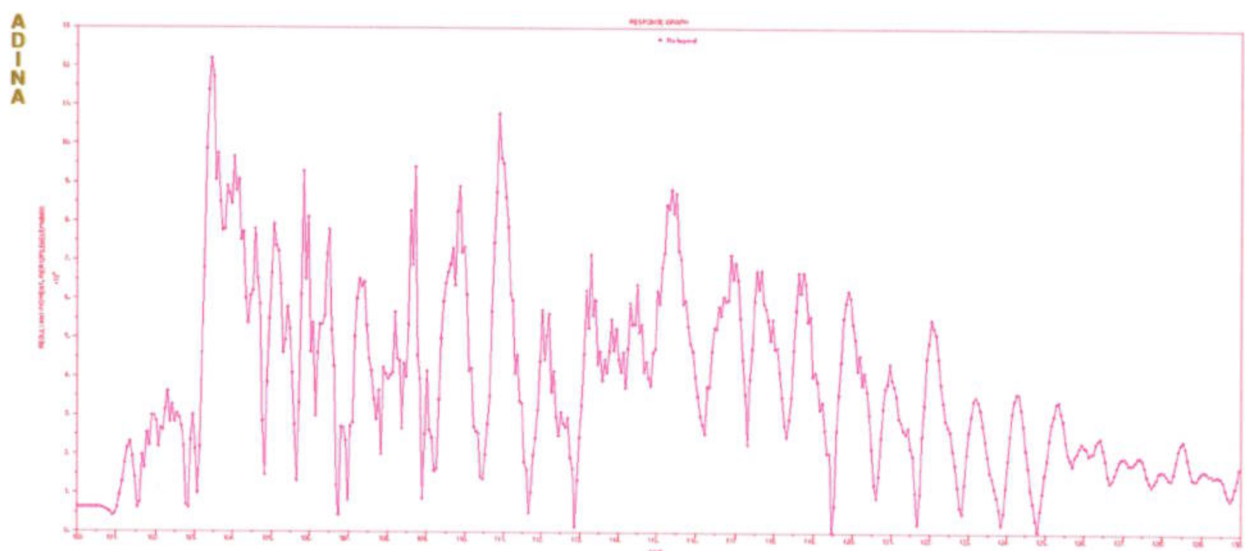


Figure 4: Pier 12 pile 6 resultant moment vs time plot for top of pile element, time-history set 1

The challenge in processing the ADINA data is examining each element along the pile and finding the controlling combination of bending moment and axial load demand for any time step of the three time-history motion runs. Typically, this occurs when moment demand is highest and axial load is least, since this will produce the lowest section capacity. Three load cases for all time-history runs at each pile of interest were extracted to accomplish this:

- Maximum resultant top of pile moment demand at a single time step with associated lower pile moment and axial load demands (labeled 'T' data).
- Maximum resultant moment demand along all pile elements at any time step and associated axial load (labeled 'M' data).
- Least axial load along all pile elements at any time step and associated moment demand (labeled 'A' data).

Data for these cases was sifted from the ADINA output for post processing and can be found in the Appendix G tables. Table 6 contains a Plot Key which can be used to find the analysis results and ADINA output for each pile and for the load cases indicated in the table.

Table 6 - PLOT KEY

		Moment demand for top of pile at maximum bending*	Moment demand for maximum at any time step*	Moment demand for highest upward axial, any time step*	Moment Demand and Capacity Drawing
LOCATION	**Pier 6** *pile 5*	**T6**	**M6**	**A6**	**D6**
	Pier 12 *pile 6*	**T12**	**M12**	**A12**	**D12**
	Pier 14 *pile 8*	**T14**	**M14**	**A14**	**D14**

* Based on ADINA finite element model using three time-history motions

The tabled data becomes more meaningful when transformed into a drawing. Drawings D6, D12, and D14 (also shown on Attachments 11, 12 & 13) represent the controlling moment demand envelopes from the analysis for piers 6, 12, and 14 respectively. Relative bending capacities along each of the four pile regions are also depicted in the drawings. A description of this process follows.

Pile Capacity Analysis

The bending capacity of each pile was evaluated by using the X-Section analysis tool. The original bending capacity along each pile was recreated before revaluating suspected anomaly locations. ADINA data was reviewed to assign appropriate axial loads by location so that proper interaction could be followed. Figure 5 graphs interaction curves for all four pile regions prior to anomaly evaluation.

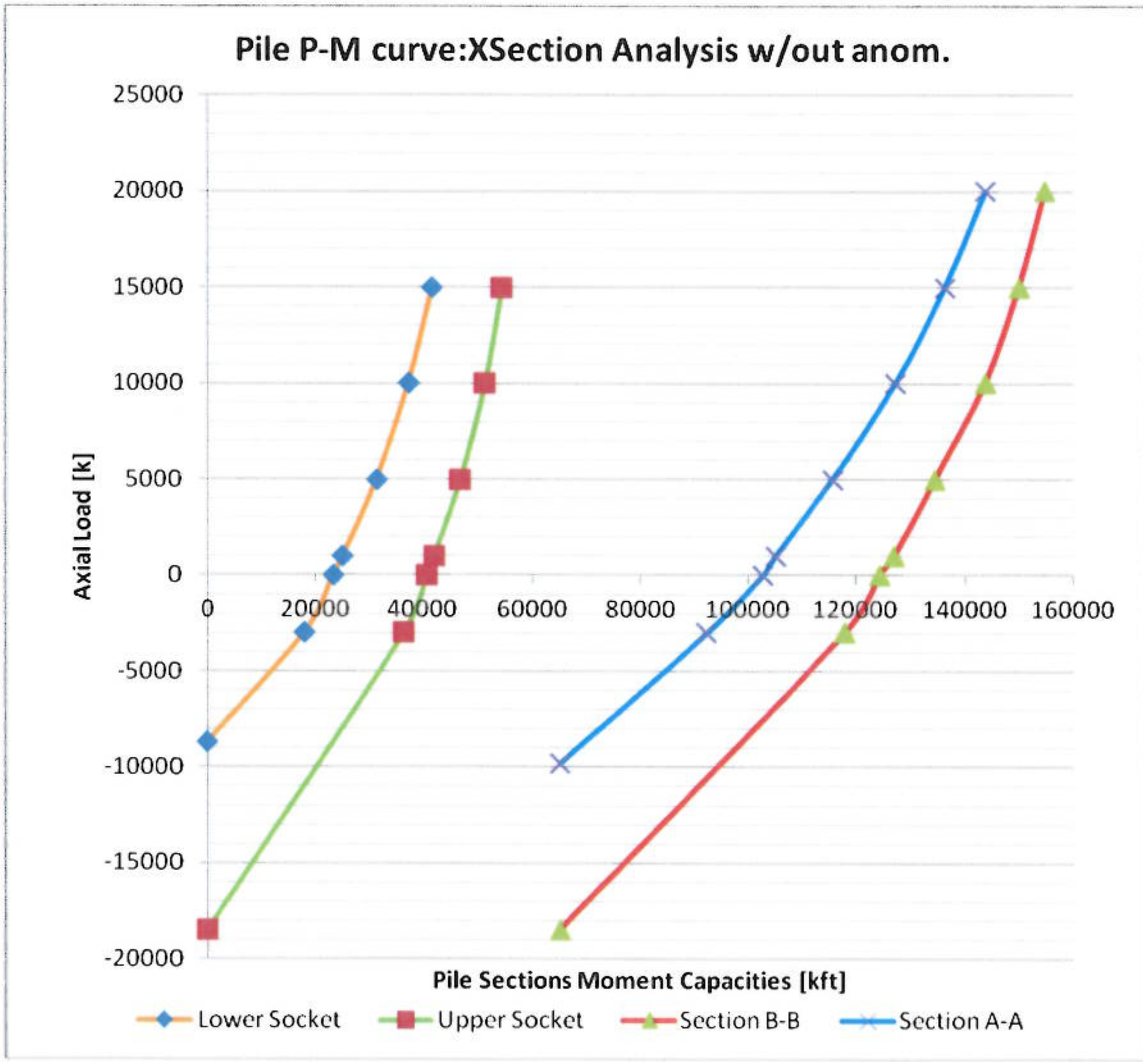


Figure 5: Interaction curves for each region of pile

These curves provide bending moment capacity values for the expected range of axial load demands in any region of the pile. Upper sections of pile have much higher capacity due to the presence of the steel casing.

Bending moment capacity reductions produced by anomalies were also modeled with the XSection analysis tool and compared to demand moments. Areas of suspected anomalies were modeled as low grade (500 psi) concrete. These areas were restricted to portions of the pile outside the core since CSL data showed that the inner portion of the piles was sound. Figure 6 depicts a typical anomalous pile cross section, and Figure 7 shows a typical XSection analysis result.

Analysis showed that only minor reductions in capacity are possible with anomalies in place. For example, for the location depicted in Figure 6 the analysis showed that in the cased region of the pile with the anomaly in place there was a total reduction in bending capacity of 8.7%, and in the non-cased rock socket region there was generally less than a 5% reduction in capacity. Possible reductions in capacity are mapped on drawings D6, D12, and D14 and are far less than demands.

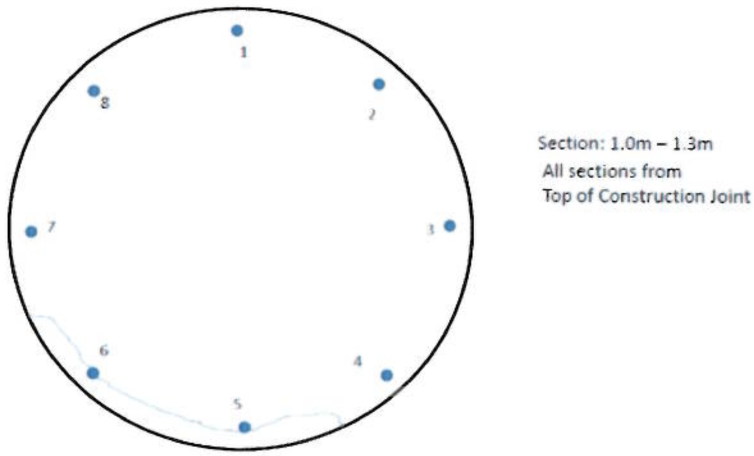


Figure 6: Typical Anomalous Pile Cross Section

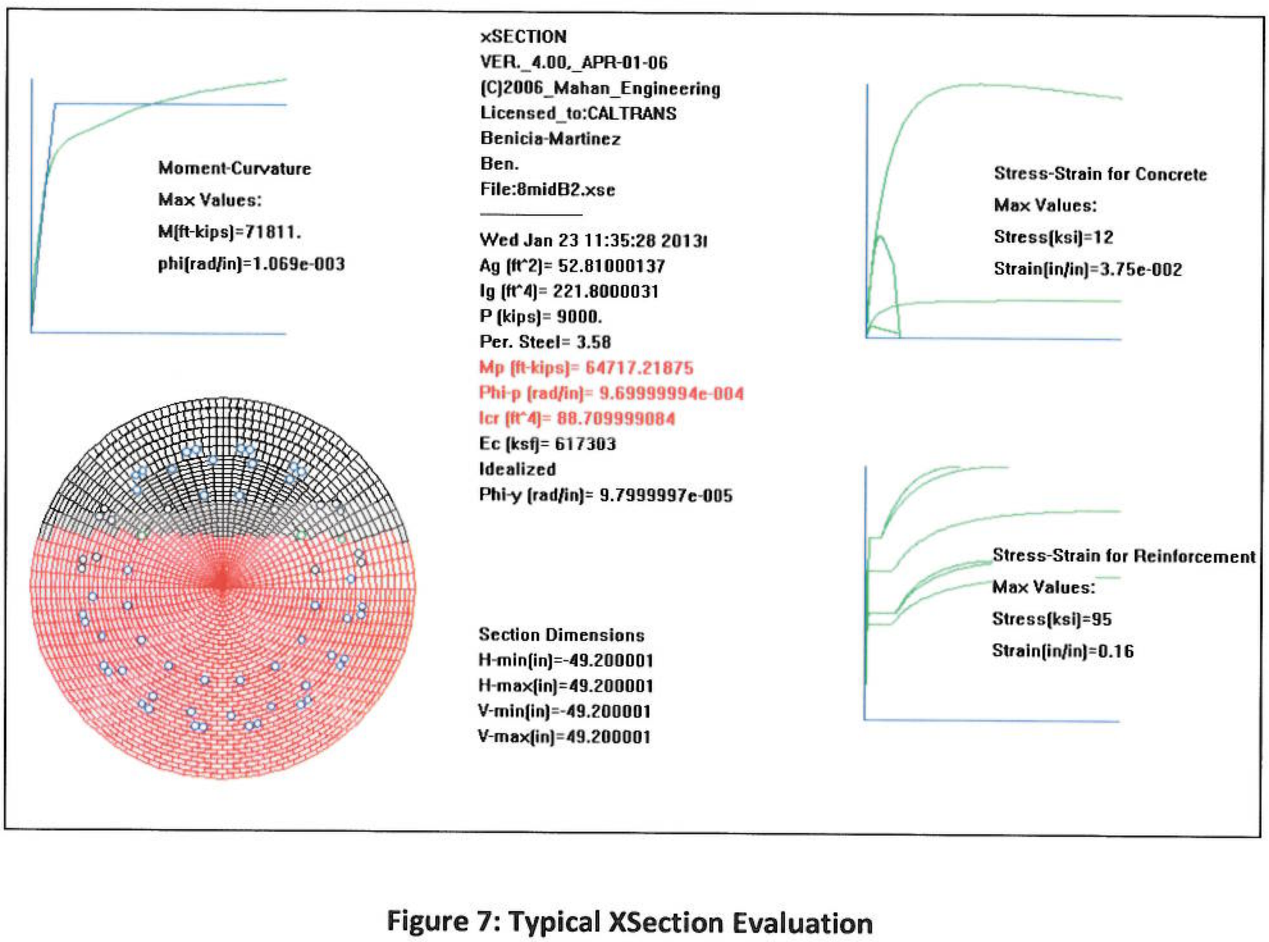


Figure 7: Typical XSection Evaluation

Note: Rotated for Minimum Capacity

Pile Shear Analysis

ADINA extractions did not include pile shear demand values. To calculate expected values, the maximum column over strength moment was divided by the column length to obtain a base shear value. This value was then divided by the number of piles at each pier to arrive at an expected demand shear per pile. Shear demand was kept constant so that the evaluation would be conservative.

Shear capacity was calculated for each region of the piles accounting for concrete, reinforcement hoops, and the casing, as applicable. Table 7 tabulates the result for this assessment in terms of demand to capacity. In all cases the D/C ratio is far below 1.0. Affects of possible anomalies are insignificant, as capacities are far above demands, and actual demands in the lower regions of piles are less than those shown.

Table 7 Pile shear demand to capacity (D/C) assessment

Pier	Location	Maximum Demand	Capacity	D/C	
6	A-A	951	30168	0.032	ok
6	B-B	951	30046	0.032	ok
6	Upper S.	951	1781	0.534	ok
6	Lower S.	951	1781	0.534	ok
12	A-A	1072	30168	0.036	ok
12	B-B	1072	30046	0.036	ok
12	Upper S.	1072	1781	0.602	ok
12	Lower S.	1072	1781	0.602	ok
14	A-A	918	30168	0.030	ok
14	B-B	918	30046	0.031	ok
14	Upper S.	918	1781	0.515	ok
14	Lower S.	918	1781	0.515	ok

Analysis Comments

The review of the analysis revealed a small length of high moment in combination with high tension at the upper region of pile 6, Pier 12. This occurred during the ADINA time-history set 1 analysis for elements just below the footing. The D/C ratio for this occurrence was 1.10. Mill certificates from the Benicia Martinez project were collected and examined to determine the actual steel properties used for pile construction. It was found that the actual yield strength of the casings was an average of 11% higher than required, and that the ultimate strength was 42% above that. This overage covers the situation and reduces the D/C ratio to 0.93 (see drawing D12, Tension 2 case in Appendix G for additional details). In addition to the structural evaluation performed by the primary reviewer there was a secondary review of the analysis that showed consistent results with the structural evaluation described in this report. Analysis results of the secondary reviewer are also attached in Appendix G.

6. Null Value Data Evaluations

In the process of identifying GGL data inconsistencies due to field operator actions, the GamDat investigation identified data files from several project with 'Null Values' in the GGL data, where no concrete density was recorded at the depths were null values were recorded by the GGL instrumentation. The possible explanations behind these occurrences are explained in the GamDat Report and are beyond the scope of this report. As part of the interpretation of the null value data by the GamDat team it was determined that the occurrence of five or more consecutive null values in the GGL data represented a potential anomaly. With this criteria there were two projects identified that required additional structural evaluation, the Retaining Wall A piles on the I-405 project and the S149-E70 Connector Separation pile shaft at Bent 2.

I-405 Retaining Wall A

Retaining wall A is a cantilevered (Type 1) retaining wall supported on 24 inch CIDH piles (see Appendix H). The piles in question are two piles out of a total of 33 piles on an 18.3m (60ft) segment of the retaining wall and are numbered piles no. 5 and 6. As was done for ID's 18 and 52 discussed earlier in this report, these piles were removed from the analysis of the wall and revised foundation demands were calculated on the remainder of the piles. As can be seen from Table 8 the revised Capacity/Demand values were greater than 1.0 for all controlling load cases.

Table 8 RETAINING WALL A Analysis results (with piles removed)

LOAD CASE	DESCRIPTION	CAPACITY (kips)	DEMAND (kips)	Factor of Safety
SERVICE LOADS	TOTAL LATERAL LOAD	1196	819	1.46
	MAX HEEL LOAD	140	82	1.71
	MAX TOE LOAD	140	97	1.45

S149-E70 Connector Separation

The S149-E70 connector separation is a prestressed cast-in-place box girder bridge supported on single column bents (see Appendix H). The pile in question is a 2.1m (6.7ft) diameter Type II CIDH pile located at Bent 2. As a result of the GamDat investigation, tube number 4, one of the seven inspection tubes in the pile required reevaluation because of the occurrence of null values in the GGL data. These null values were identified at a depth of 9.51m (31 ft) from the top of pile (see Pile Data form in Appendix H) and represented a small potential anomaly which was evaluated structurally by the designer and for geotechnical axial resistance by the geotechnical engineer.

As was done for other cases in this report the designer removed a tributary wedge out of the pile for both concrete and steel and analyzed the cross section in the program Xsection. The results of the analysis showed a reduced capacity of 11,122 ft-kips or approximately a 21% reduction in capacity. This reduced capacity was compared to the demand of 5270 ft-kips at this location which is significantly less than capacity. This is due to the fact that the null value identified in the GamDat Report is associated with a specific depth along the pile where the demand was significantly less than capacity. In addition to this the geotechnical capacity was evaluated by the geotechnical engineer and found to be adequate given the size of the potential anomaly. (See Appendix H for Pile Data form, Xsection output results and as-built plans.

REFERENCES

1. "Gamma-Gamma Logging and Crosshole Sonic Logging Test Results: Pile 5 at Pier 6" Report, Caltrans Geotechnical Instrumentation Branch, May 26, 2004

2. "Gamma-Gamma Logging and Crosshole Sonic Logging Test Results: Pile 6 at Pier 12" Report, Caltrans Geotechnical Instrumentation Branch, August 31, 2004

3. "Gamma-Gamma Logging Acceptance Test Results: Pile 1-5 at Abutment 6" Lake Hodges Bridge Report, Caltrans Geotechnical Instrumentation Branch, January 27, 2005

4. "Gamma-Gamma Logging Acceptance Test Results: Pile at Bent 15" SE Connector Report, Caltrans Geotechnical Instrumentation Branch, May 5, 2005

5. "Gamma-Gamma Logging Acceptance Test Results: CIDH pile at Overhead Sign 19", Caltrans Geotechnical Instrumentation Branch, March 21,2008

6. "Gamma-Gamma Logging Acceptance Test Results: Retaining Wall 435B", Caltrans Geotechnical Instrumentation Branch, May 21, 2007

7. "Gamma-Gamma Logging Acceptance Test Results: Braddock Dr. UC(Widen)", Caltrans Geotechnical Instrumentation Branch, April 10, 2007