FOR CONTRACT No.: 07-202111
PROJECT ID 0700020869

# INFORMATION HANDOUT

**Department of the Army Nationwide Permit**

**County of Los Angeles - Flood Control District Permit**

**California Regional Water Quality Control Board Los Angeles Region**

**State of California - Department of Fish and Game**

**Revised Foundation Report for Atlantic Blvd. UC On-Ramp, Intersate 710, dated October 29, 2010**

**Foundation Report for Los Angeles River Bridge (Widen), Interstate 710, dated November 22, 2010**

**Foundation Report for Retaining Wall No. 1150 at Cheli Deport Overhead, dated August 23, 2010**

**Foundation Report for Whittier Blvd UC (Widen), Interstate 710, dated November 12, 2010**

**Foundation Report for Olympic Blvd UC Bridge Widening, Interstate 710, dated September 2 , 2010**

**Foundation Report for Floral Drive UC Bridge Widening, Interstate 710, dated August 31, 2010**

**Foundation Report for Retaining Wall No. 1271, Interstate 710, dated February 28, 2011**

**Foundation Report for NB 710/5 Separation Widening, dated September 29, 2010**

**Foundation Report for SB 710/5 Separation Widening, dated September 23, 2010**

**Final Hydraulic Report, Los Angeles Bridge**

**ROUTE: 07-LA-710-PM 17.2/26.4**

DEPARTMENT OF THE ARMY
LOS ANGELES DISTRICT, CORPS OF ENGINEERS
P.O. BOX 532711
LOS ANGELES, CALIFORNIA 90053-2325

June 15, 2011

REPLY TO
ATTENTION OF:
Regulatory Division

**DEPARTMENT OF THE ARMY NATIONWIDE PERMIT AUTHORIZATION**

Paul Caron, Senior District Biologist
California Department of Transportation, District 7, MS 16A
Attention: Linna Wei
100 South Main Street, MS-16A
Los Angeles, California 90012

Dear Mr. Caron:

This is in reply to your correspondence (File No. SPL-2011-00206-MAS) dated February 16, 2011 for a Department of the Army Permit to discharge temporary fill into 0.06 acre of non-wetland waters of the U.S., and permanent fill into approximately 0.05 acre of non-wetland waters of the U.S., in association with the Caltrans I-710 Bridge Widening over the Los Angeles River. The proposed work would take place in the Los Angeles River, a traditional navigable water and tributary to the Pacific Ocean, at the Interstate 710 crossing (33.9359°N, -118.1753° W) in the City of Downey, Los Angeles County, California (as shown on enclosed map).

Based on the information you have provided, the Corps of Engineers has determined that your proposed activity complies with the enclosed terms and conditions of **Nationwide Permit No. 14:** ***Linear Transportation Projects***, as described in enclosure 1.

Specifically, you are authorized to permanently impact 0.05 acre and temporarily impact 0.06 acre of the Los Angeles River in order to widen the I-710 crossing by expanding the existing piers and retaining walls by 12 feet on the northern side and 16 feet on the southern side (as shown on enclosed plans).

Furthermore, you must comply with the following non-discretionary Special Condition:

Within 45 calendar days of completion of authorized work in waters of the U.S., the Permittee shall submit to the Corps Regulatory Division a post-project implementation memo including the following:

A) Date authorized impacts to waters of the U.S. ceased;
B) Color photographs taken at the project site before, during and after construction for those aspects directly associated with impacts to waters of the US; and
C) One copy of "as-built" drawings for the entire project (all sheets must be signed, dated, to-scale, and no larger than 11x17 inches).

This verification is valid until the NWP is modified, reissued, or revoked. All of the existing NWPs are scheduled to be modified, reissued, or revoked prior to March 18, 2012. It is incumbent upon you to remain informed of changes to the NWPs. We will issue a public notice when the NWPs are reissued. Furthermore, if you commence or are under contract to commence this activity before the date that the relevant nationwide permit is modified or revoked, you will have twelve (12) months from the date of the modification or revocation of the NWP to complete the activity under the present terms and conditions of this nationwide permit.

A nationwide permit does not grant any property rights or exclusive privileges. Also, it does not authorize any injury to the property or rights of others or authorize interference with any existing or proposed Federal project. Furthermore, it does not obviate the need to obtain other Federal, state, or local authorizations required by law.

Thank you for participating in our regulatory program. If you have any questions, please contact Melanie Stalder of my staff at (213) 452-3294 or via e-mail at Melanie.A.Stalder@usace.army.mil.

Please be advised that you can now comment on your experience with Regulatory Division by accessing the Corps web-based customer survey form at: http://per2.nwp.usace.army.mil/survey.html.

Sincerely,

Mark D. Cohen
Deputy Chief, Regulatory Division

Enclosure

LOS ANGELES DISTRICT
U.S. ARMY CORPS OF ENGINEERS

CERTIFICATION OF COMPLIANCE WITH
DEPARTMENT OF THE ARMY NATIONWIDE PERMIT

Permit Number: *SPL-2011-00206-MAS*

Name of Permittee: *California Department of Transportation, District 7 – Paul Caron*

Date of Issuance: *June 15, 2011*

Upon completion of the activity authorized by this permit and any mitigation required by the permit, sign this certification and return it to the following address:

U.S Army Corps of Engineers
Regulatory Division
ATTN: CESPL-RG-SPL-2011-00206-MAS
LOS ANGELES DISTRICT, CORPS OF ENGINEERS
P.O. BOX 532711
LOS ANGELES, CALIFORNIA 90053-2325

Please note that your permitted activity is subject to a compliance inspection by an Army Corps of Engineers representative. If you fail to comply with this nationwide permit you may be subject to permit suspension, modification, or revocation procedures as contained in 33 C.F.R. section 330.5 or enforcement procedures such as those contained in 33 C.F.R. sections 326.4 and 326.5.

I hereby certify that the work authorized by the above referenced permit has been completed in accordance with the terms and conditions of the said permit, and required mitigation was completed in accordance with the permit condition(s).

_____ _____

Signature of Permittee Date

| DIST | COUNTY | ROUTE | POST MILES TOTAL PROJECT | SHEET No. | TOTAL SHEETS |
|---|---|---|---|---|---|
| 07 | LA | 710 | | 2 | 2 |

REGISTERED CIVIL ENGINEER X DATE

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.

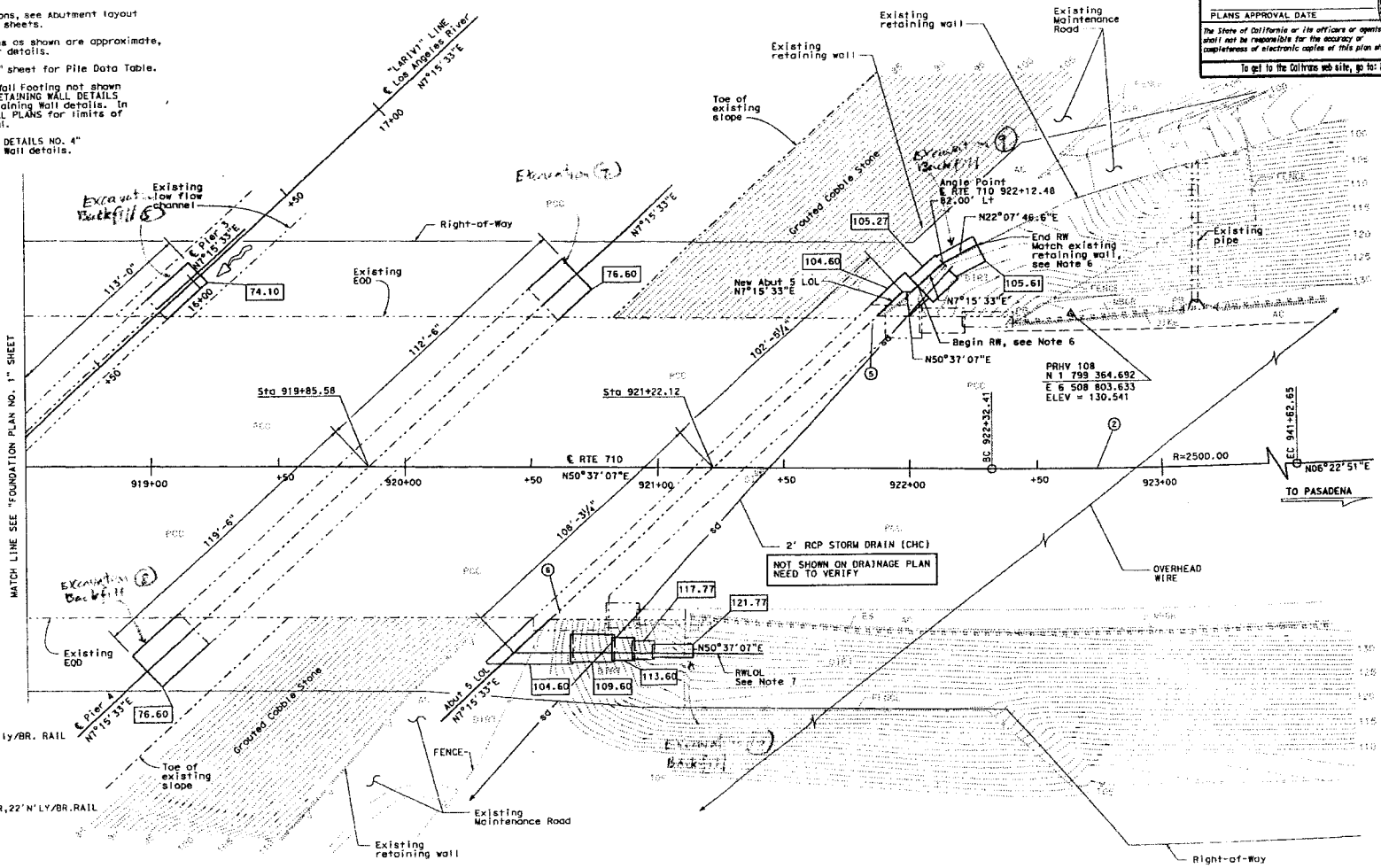
To get to the Caltrans web site, go to: http://www.dot.ca.gov

NOTES:

1. [ ] indicates bottom of footing elevation.
2. Existing contours are shown, for final contours see "ROAD PLANS."
3. For footing dimensions, see Abutment layout and Pier wall layout sheets.
4. Underground utilities as shown are approximate, see "ROAD PLANS" for details.
5. See "INDEX TO PLANS" sheet for Pile Data Table.
6. Existing Retaining Wall Footing not shown for clarity. See "RETAINING WALL DETAILS NO. 3" sheet for Retaining Wall details. In addition, see REMOVAL PLANS for limits of existing wall removal.
7. See "RETAINING WALL DETAILS NO. 4" sheet for Retaining Wall details.

| CURVE DATA | | | | |
|---|---|---|---|---|
| No. | R | Δ | T | L |
| ② | 2500.000 | 44°14'16" | 1016.105 | 1930.237 |

Bridge #53-0828 Location
Ⓢ -57.714 Lt. ℄ Rte710, Sta.921+84.52, Elev.=129.420 ±
Ⓖ -58.285 Rt. ℄ Rte710, Sta.920+61.60, Elev.=129.430 ±




SURVEY CONTROL

PRHV 109 (SEE SHEET1)
PK SB710 Shldr, 84' S'ly/BR. RAIL
63.346' Lt. ℄ Rte 710
Sta.914+96.91
N 1 798 877.209
E 6 508 210.228
ELEV. = 122.185

PRHV 108
PK ON DIKE BEHIND WBGR,22'N'LY/BR.RAIL
61.830'Lt. ℄ Rte 710
Sta.922+63.41
N 1 799 364.692
E 6 508 803.633
Elev. = 130.541'

| PRELIMINARY INVESTIGATION SECTION | | | | | | | |
|---|---|---|---|---|---|---|---|
| SCALE 1:20 | VERT.DATUM NAVD 88 | PHOTOGRAMMETRY AS OF: | | | | | |
| | HORZ.DATUM NAD 83 (1991.35) | SURVEYED | BY C.STEWART/ T.PUNG | CHECKED | BY C. STEWART | | |
| ALIGNMENT TIES DISTRICT SURVEY | | DRAFTED | BY S. ABASSY 06/2009 | CHECKED | BY S. ALIVIO | | |

STRUCTURES FOUNDATION PLAN SHEET (ENGLISH) (REV. 10/25/05)

| DESIGN | BY F. Wei | CHECKED X |
|---|---|---|
| DETAILS | BY P. Perez | CHECKED F. Wei |
| QUANTITIES | BY X | CHECKED X |

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

DIVISION OF ENGINEERING SERVICES
STRUCTURE DESIGN
DESIGN BRANCH 21

CU 7224
EA 202101

BRIDGE NO. 53-0828
POST MILE 17.30

# LOS ANGELES RIVER BRIDGE (WIDEN)
## FOUNDATION PLAN NO. 2

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES 01/06/10 02/08/10

SHEET 2 OF 2

FILE => 53-0828-e-fdn102.dgn

USERNAME => S135307 DATE PLOTTED => 03-MAR-2010 TIME PLOTTED => 09:58

| DIST | COUNTY | ROUTE | POST MILES TOTAL PROJECT | SHEET No | TOTAL SHEETS |
|---|---|---|---|---|---|
| 07 | LA | 710 | | 1 | 2 |

REGISTERED CIVIL ENGINEER X DATE

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.*

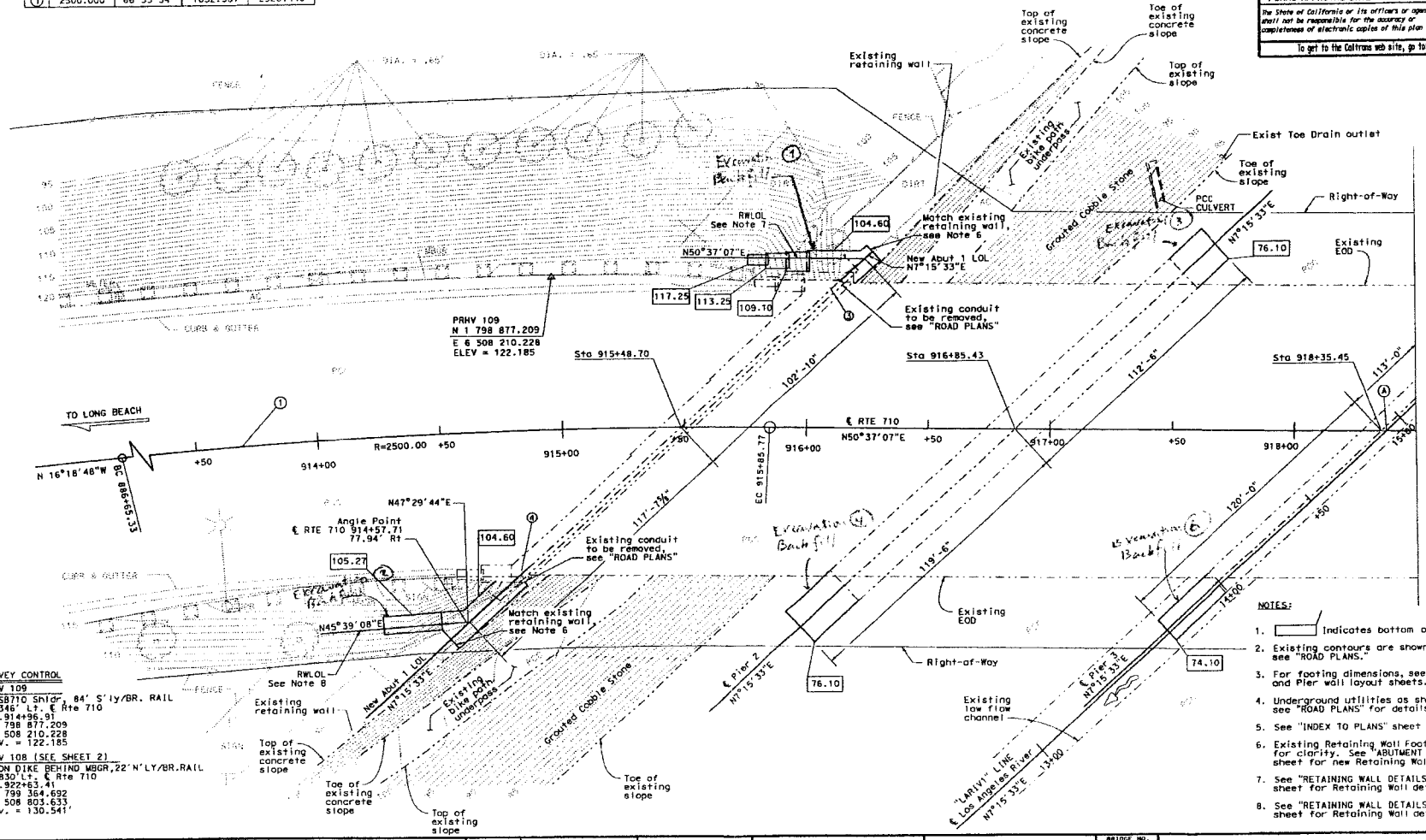
REGISTERED PROFESSIONAL ENGINEER — CIVIL — STATE OF CALIFORNIA

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

| CURVE DATA | | | | |
|---|---|---|---|---|
| No. | R | Δ | T | L |
| 1 | 2500.000 | 66°55'54" | 1652.567 | 2920.440 |

(A) STA.918+37.57 "℄ RTE 710" LINE= STA.14+92.13 "LARIVI" LINE

Bridge #53-0828 Location
(3) -58.199 Lt. ℄ RTE710, 916+09.35Sta., Elev.=123.786±
(4) -57.696 Rt. ℄ RTE710, 914+81.74Sta., Elev.=118.831±




SURVEY CONTROL

PRHV 109
PK SB710 Shldr, 84' S'ly/BR. RAIL
63.346' Lt. ℄ Rte 710
Sta.914+96.91
N 1 798 877.209
E 6 508 210.228
ELEV. = 122.185

PRHV 108 (SEE SHEET 2)
PK ON DIKE BEHIND WBGR,22'N'LY/BR.RAIL
61.830'Lt. ℄ Rte 710
Sta.922+63.41
N 1 799 364.692
E 6 508 803.633
Elev. = 130.541'

NOTES:

1. ▭ Indicates bottom of footing elevation.
2. Existing contours are shown, for final contours see "ROAD PLANS."
3. For footing dimensions, see Abutment layout and Pier wall layout sheets.
4. Underground utilities as shown are approximate, see "ROAD PLANS" for details.
5. See "INDEX TO PLANS" sheet for Pile Data Table.
6. Existing Retaining Wall Footing not shown for clarity. See "ABUTMENT 1 LAYOUT" sheet for new Retaining Wall details.
7. See "RETAINING WALL DETAILS NO. 1" sheet for Retaining Wall details.
8. See "RETAINING WALL DETAILS NO. 2" sheet for Retaining Wall details.

PRELIMINARY INVESTIGATION SECTION

| SCALE | VERT.DATUM NGVD 88 | PHOTOGRAMMETRY AS OF: | | | |
|---|---|---|---|---|---|
| 1:20 | HORZ.DATUM NAD 83 (1991.35) | SURVEYED | BY C.SIEWABI/I.PUNG | CHECKED | BY C. SIEWABI |
| ALIGNMENT TIES | DISTRICT SURVEY | DRAFTED | BY S. ABASSY 06/2009 | CHECKED | BY S. ALIVIO |

STRUCTURES FOUNDATION PLAN SHEET (ENGLISH) (REV. 10/25/05)

| | | |
|---|---|---|
| DESIGN | BY E. Wei | CHECKED X |
| DETAILS | BY E. Eacez | CHECKED E. Wei |
| QUANTITIES | BY X | CHECKED X |

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

DIVISION OF ENGINEERING SERVICES
STRUCTURE DESIGN
DESIGN BRANCH 21

BRIDGE NO. 53-0828
POST MILE 11.30

CU 7224
EA 202101

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES: 3/07/07 12/07/07 10/07/08 3/12/09

# LOS ANGELES RIVER BRIDGE (WIDEN)

## FOUNDATION PLAN NO. 1

SHEET 1 OF 2

FILE => 53-0828-e fdp01(03-01-10).dgn

USERNAME => s813530 DATE PLOTTED => 03-MAR-2010

| Dist | COUNTY | ROUTE | POST MILES TOTAL PROJECT | SHEET No. | TOTAL SHEETS |
|---|---|---|---|---|---|
| 07 | LA | 710 | 17.2 / 26.4 | | |

REGISTERED CIVIL ENGINEER DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER No. EXP. CIVIL STATE OF CALIFORNIA

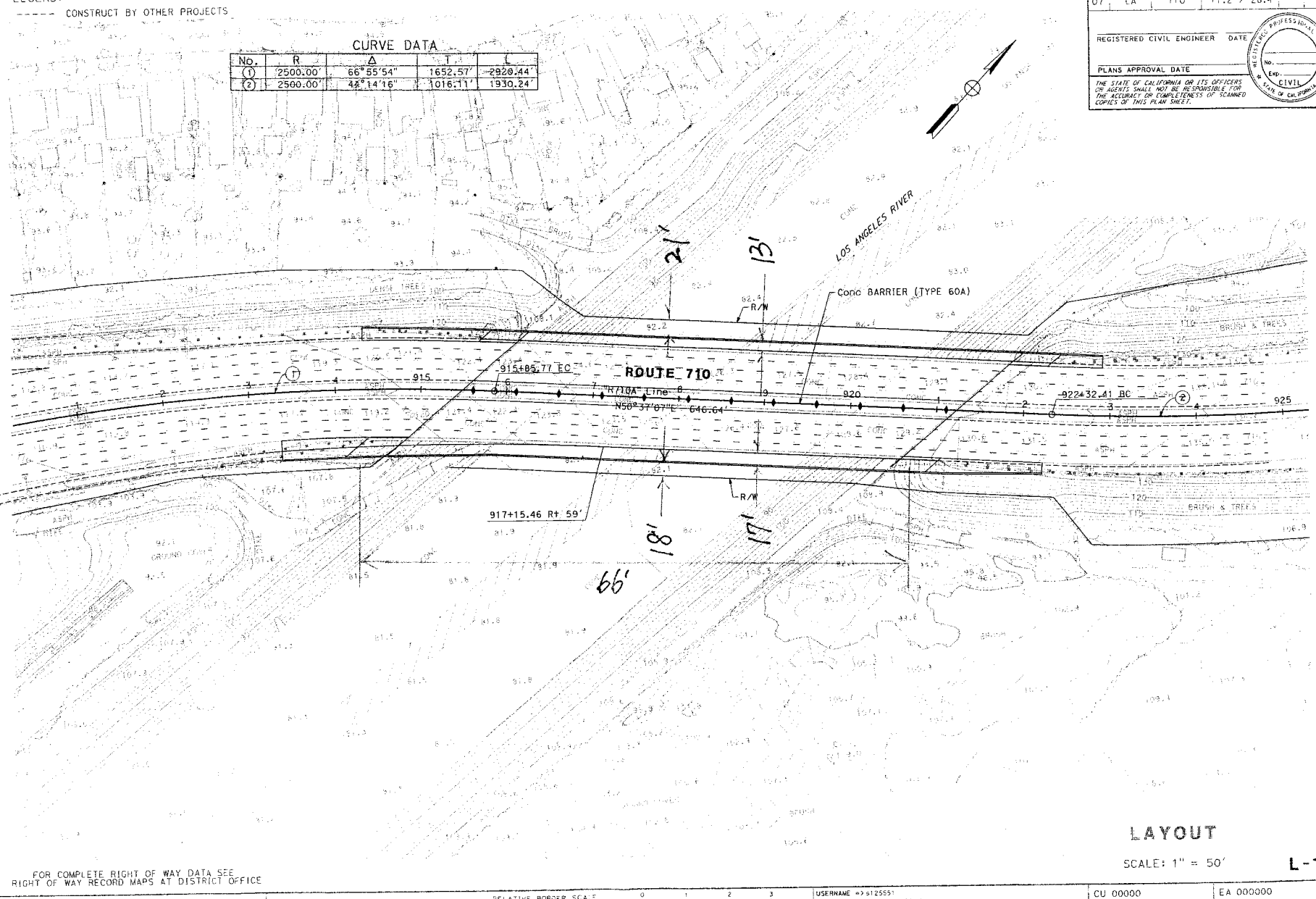
*THE STATE OF CALIFORNIA OR ITS OFFICERS OR AGENTS SHALL NOT BE RESPONSIBLE FOR THE ACCURACY OR COMPLETENESS OF SCANNED COPIES OF THIS PLAN SHEET.*

LEGEND:

----- CONSTRUCT BY OTHER PROJECTS

CURVE DATA

| No. | R | Δ | T | L |
|---|---|---|---|---|
| 1 | 2500.00' | 66°55'54" | 1652.57' | 2920.44' |
| 2 | 2500.00' | 44°14'16" | 1016.11' | 1930.24' |




FOR COMPLETE RIGHT OF WAY DATA SEE
RIGHT OF WAY RECORD MAPS AT DISTRICT OFFICE

# LAYOUT

SCALE: 1" = 50'

L-1

REVISED BY | DATE REVISED

CALCULATED/DESIGNED BY | CHECKED BY

FUNCTIONAL SUPERVISOR

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

Caltrans

LAST REVISION 00-00-00 | DATE PLOTTED => 22 APR 2010 | TIME PLOTTED => 15:53

BORDER LAST REVISED 4/11/2008

RELATIVE BORDER SCALE IS IN INCHES 0 1 2 3

USERNAME => s125551
DGN FILE => 32021Gec001.dgn

CU 00000 | EA 000000

Tract #:

Issued By:
Issued Date:

Permit #: **PCFL T201004347**

Permit Office: 6

PC-OVERBUI
OVEBUILD WITHIN OR
ACROSS FLOOD FACILITY

COUNTY OF LOS ANGELES-DPW
Department Of Public Works
Alhambra, CA 91803 (626) 458-3940
Flood Control District Permit

**FOR BIDDING PURPOSES ONLY (NOT FOR CONSTRUCTION)**

| Individual's / Company Name | Address / City, State Zip | Work Phone | Home Phone |
|---|---|---|---|
| (APP) STATE OF CALIFORNIA, DOT<br>MIKE NOURI<br>(CNT) | 100 S. MAIN ST., #100 MS 13<br>LOS ANGELES, CA 90012 | 213-897-6262 | |

**Emergency Contact**

**Location**

Site Address:
Description: LOS ANGELES RIVER: AT 710 FWY, SOUTH GATE

**Scope of Work**

*** FOR BIDDING PURPOSES ONLY. NOT FOR CONSTRUCTION***

PURPOSE OF PERMIT: To authorize the work described below affecting the subject stream in accordance with the submitted plans, Los Angeles County Flood Control District Drawing Nos. 19-F2632.1 to .34 (Los Angeles County Department of Public Works Drawing Nos. PF557217 to PF557250).

WORK DESCRIPTION: Extend the existing piers and abutments to widen the Permittee's 710 FWY Bridge, per submitted plans.

Work shall not begin until an inspection deposit of $20,000 has been paid; contractor's contact information, insurance certificate, and additional insured endorsement are approved by the County. Work shall also not begin until a Diversion Plan is approved by the District.

Prior to any construction work, the current condition of the existing channel shall be inspected and documented. After the completion of construction, the existing channel shall be re-inspected and monitored for the possible damages. In the event that any damage is identified at any time, immediate replacement or repair to the satisfaction of the District will be required at the Permittee's expense. No open trenches in the channel levee or invert will be allowed between October 15 and April 15. This permit shall not be exercised during inclement weather or when the 5-day forecast predicts rain. Permittee must provide a schedule of its activities to avoid any potential conflicts with the District's maintenance work and must notify the District of the date of final completion. Permittee must immediately notify the South Area of Flood Maintenance Division at (562) 861-0316, in the event of an emergency. Permittee is advised that clearance from other agencies may be required regarding the proposed activities. Any mitigation required by environmental regulatory agencies resulting from this permit's activity must occur outside the District's R/W.

PERMITTEE MUST NOTIFY PERMIT OFFICE NO. 4 AT (562) 861-3580 AND DERREL FEIKER OF FLOOD MAINTENACE DIVISION AT (562) 861-0316 AT LEAST ONE WEEK BEFORE STARTING ANY WORK UNDER THIS PERMIT. PERMITTEE MUST ALSO CONTACT DEPARTMENT'S BIKE TRAIL COORDIANTOR, ABU YUSUF, AT TELEPHONE (626) 458-3940 PRIOR TO ANY CLOSURE OR DETOUR OF THE BIKE TRAIL. FAILURE TO SO NOTIFY THE PERMIT OFFICE IS CAUSE FOR REVOCATION OF PERMIT. SHOULD PERMITTEE FAIL TO TAKE ACTION WITHIN 180 DAYS FROM DATE OF ISSUANCE OF THIS PERMIT OR FAIL TO ACTIVELY AND DILIGENTLY EXERCISE THE PRIVILEGES OF THIS PERMIT, THE PERMIT BECOMES NULL AND VOID. A COPY OF THIS PERMIT SHALL BE KEPT AT THE WORK SITE DURING ALL PERIODS OF OPERATION WITHIN THE DISTRICT'S RIGHT OF WAY AND SHALL BE SHOWN TO ANY DISTRICT REPRESENTATIVE OR LAW ENFORCEMENT OFFICER UPON DEMAND.

cc: U.S. Army Corps of Engineers, City of South Gate, Design (Chang, Zandieh), Flood Maintenance (South); Water Resources (Wood), Watershed Management (Galang), Land Development (Office, P.O. 4, Berhan)

**Permit Detail**

| | |
|---|---|
| FILE CODE NO. : | 19.032 |
| FLOOD FACILITY NAME : | LOS ANGELES RIVER |
| FLOOD STATION : | ~653+00 |
| INSPECTION CHARGE #: | TBD |
| INSURANCE EXPIRE | TBD |
| LOCATION 1: | AT 710 FWY, SOUTH GATE |
| THOMAS GUIDE : | 705-F6 |

**Comments**

PCFL T201004347

REPORT: lapwrp028

Tract #:

Issued By:
Issued Date:

Permit #: **PCFL T201004347**

Permit Office: 6

| Fees | Fee Code | Acount Code | Amount |
|---|---|---|---|
| INSPECT FLOOD OVERBUILD - ACTUAL COST | PCOVBINSP | B07_8371 | $20,000.00 |
| | | Total Fees: | $20,000.00 |

**Is hereby permitted to complete scope of work on the public highways subject to provisions required by County of Los Angeles Highway Permit Ordinance (Division 1 of Title 16, Los Angeles County Code), the Municipal Code, and City Ordinance governing the area where this work is to be done, and the attachments hereon specified. Permit revocable at option of Public Works Director, in consideration of granting of this permit, it is agreed by the applicant that the County of Los Angeles and/or the city wherein the permit work is to be performed and any of their officers or employees thereof shall be saved harmless by the applicant from any liability or responsibility for any accident, loss, or damage to persons or property, happening occurring as the proximate result of any of the work undertaken under the terms of this application and the permit or permits which may be granted in response thereto, and that all of said liabilities are hereby assumed by the applicant, it is further agreed that if any part of this installation interferes with the future use of the highway by the general public, it must be removed or relocated, asdesignated by the Director of Public Works or Superintendent of Streets, at the expense of the permittee of his successor in interest. The permit is void if the permittee is not in compliance with Section 3800 of the Labor Code**

**Performance of the work of activity under this permit is tantamount to agreeing to the conditions of this permit, Copy of this permit shall be kept at work site during period of operation within District's/Road right of way and shall be shown to District's representative or any law enforcement officer upon demand.**

**INSPECTION REQUIRED**

**CALL PERMIT OFFICER 24 HOURS BEFORE STARTING WORK UNDER THIS PERMIT. FAILURE TO DO SO IS CAUSE FOR REVOCATION OF THIS PERMIT. THIS PERMIT IS VOID IF WORK NOT STARTED IN 60 DAYS (FOR ROAD PERMIT) OR 180 DAYS (FOR FLOOD PERMIT) FROM THE DATE OF THE ISSUANCE.**

**PERMIT OFFICE NO. PCHQ**
**PUBLIC WORKS CONSTRUCTION**
**900 S. Fremont Ave.**
**Los Angeles County, CA 91803**
**PHONE NO. 626-458-3129**
**FAX NO. 626-576-7739**

PCFL T201004347

REPORT: lapwrp028

# Conditions of Approval
# By Permit



**Run Date:** Monday June 20, 2011 2:11 pm

**Permit:** PCFL - T201004347

The following Conditions of Approval are required to complete the permit:

| Condition of Approval | Entered | By | Completed | By |
|---|---|---|---|---|
| GENERAL FLOOD PROVISION NO. 1 | 24-NOV-10 | EBERHAN | | |
| Use of District's right of way for the construction or activity authorized under this permit is tantamount to agreeing to the conditions herein.(G1) | | | | |
| GENERAL PROVISION NO.2 | 24-NOV-10 | EBERHAN | | |
| Permittee shall be responsible for notifying his contractor and all subcontractors of the provisions of this permit. No work will be started until a copy of this permit is given to the contractor and each of his subcontractors. Further, the copy will be left at the site of the work being done by each contractor.(G2) | | | | |
| GENERAL PROVISION NO.3 | 24-NOV-10 | EBERHAN | | |
| Permittee is notified that in accordance with the STATE OF CALIFORNIA CONSTRUCTION SAFETY ORDERS, Section 1503, the permittee or his contractor may be required to acquire a permit from CAL/OSHA if the work authorized herein more than 5 feet deep. The inspection provided by the District can in no way be construed as a safety inspection.(G3) | | | | |
| GENERAL PROVISION NO.5 | 20-JUN-11 | EBERHAN | | |
| This permit is subject to such further conditions as the Director or his representative may issue during the period of this use. When possible. such additional conditions shall be promptly delivered in writing to the address shown on page one of this permit. Conditions delivered orally of necessity shall be promptly confirmed in writing.(G5) | | | | |
| GENERAL PROVISION NO. 4 | 24-NOV-10 | EBERHAN | | |
| Unless otherwise indicated in this permit, all work authorized by this permit shall conform to the latest edition of the Standard Specifications for Public Work Construction, as amended, and published by Building News, Inc., 3055 Overland Avenue, Los Angeles, CA 90034 and the latest edition of the Los Angeles County Department of Public Works "Additions and Amendments to the Standard Specifications for Public Works Construction".(G4) | | | | |
| GNERAL PROVISION NO. 13 | 20-JUN-11 | EBERHAN | | |
| The District reserves the right to order the removal of all equipment if District's activities so require. The District assumes no responsibility for any loss to permittee's equipment or personnel.(G13) | | | | |
| GENERAL PROVISION NO.8 | 24-NOV-10 | EBERHAN | | |
| Issuance of this permit shall not be construed as an obligation on the part of this District for the operation and maintenance of the proposed facilities.(G8) | | | | |
| GENERAL PROVISION NO. 17 | 20-JUN-11 | EBERHAN | | |
| Permittee shall keep District right of way clear of obstructions for through access at all times and shall not interfere with the activities of the District's employees or the District's contractors.(G17) | | | | |
| GENERAL PROVISION NO. 14 | 24-NOV-10 | EBERHAN | | |
| Upon completion of work authorized under this permit, permittee shall restore the area to the satisfaction of the District's representative.(G14) | | | | |
| GENERAL PROVISION NO. 18 | 24-NOV-10 | EBERHAN | | |
| Permittee shall not use District's right of way for the temporary or permanent storage of excavated materials. rock. sand. cement. or other material or any equipment. except as specifically noted.(G18) | | | | |
| PROVISION EXCAVATION NO.01 | 24-NOV-10 | EBERHAN | | |
| All open cuts and trenches within District's right of way shall be backfilled and compacted in accordance with approved methods to the satisfaction of District's representative. Paving shall be replaced in kind.(E1) | | | | |
| GENERAL PROVISION NO. 24 | 24-NOV-10 | EBERHAN | | |
| During the period of operations conducted under the permit. Permittee shall maintain in effect an insurance policy (minimum limit $ ONE million) naming the Los Angeles County Flood Control District/Los Angeles County Department of Public Works and/or U.S. Army Corps of Engineers as co-insured with respect to these operations. A copy of this policy shall be submitted to the District for inclusion in the District file copy of this permit. Expiration or cancellation of the insurance policy shall constitute revocation of this permit.(G24) | | | | |
| PROVISION WORK IN CHANNEL NO.2 | 27-DEC-10 | EBERHAN | | |
| During the period from April 15 to October 15. falsework and cofferdams may be placed and excavations made in the channel. However. capacity to convey flows around any obstructions or openings in the channel lining shall be provided as follows: June 1 to August 31- 5 percent of channel capacity/Area. September 1 to October 15 - one-third channel capacity/Area. The above criterion must be determined by hydraulic calculations. Preliminary information regarding the methods for performing these calculations may be obtained before preparation of cofferdams or falsework by contacting the U.S. Army Corps of Engineers. Operations Branch. at (213) 452-3393. For purposes of computing the area of an obstruction. dimensions shall be taken normal to channel flow and two feet added to the faces of the obstructions.(W2) | | | | |
| GENERAL PROVISION NO. 50 | 20-JUN-11 | EBERHAN | | |
| All activities covered by this permit are subject to final approval by the City of South Gate.(G50) | | | | |

# Conditions of Approval
# By Permit



**Run Date:** Monday June 20, 2011 2:11 pm

**Permit:** PCFL - T201004347

The following Conditions of Approval are required to complete the permit:

| Condition of Approval | Entered | By | Completed | By |
|---|---|---|---|---|
| GENERAL PROVISION NO. 59 | 20-JUN-11 | EBERHAN | | |
| Permittee shall protect in place all District facilities where the proposed work comes in close proximity to the District's facilities.(G59) | | | | |
| GENERAL PROVISION NO. 60 | 20-JUN-11 | EBERHAN | | |
| Any structure or portions thereof placed on District right of way must be removed, revised, and/or relocated by permittee without cost to the District, or any other public agency the District shall so designate, should future activities or policy so require.(G60) | | | | |
| GENERAL PROVISION NO. 62 | 20-JUN-11 | EBERHAN | | |
| Parking of vehicles and obstructing the entrance to the access road or of the storing of equipment in front of the double drive gates or on the access road is prohibited.(G62) | | | | |
| PROVISION FOR TRAIL NO.7 | 20-JUN-11 | EBERHAN | | |
| After project construction is completed and prior to the field acceptance of the project, the bicycle trail pavement and striping shall be restored to its preconstruction condition. (T7) | | | | |
| PROVISION WORK IN CHANNEL NO.3 | 20-JUN-11 | EBERHAN | | |
| Plans and calculations of any falsework or cofferdam to be placed within the channel waterway area must be submitted to this District for review and approval at least 30 days prior to installation.(W3) | | | | |
| PROVISION WORK IN CHANNEL NO.9 | 20-JUN-11 | EBERHAN | | |
| Issuance of this permit shall not be construed as an obligation on the part of the District to assume responsibility for any damages incurred to the permittee's improvements due to high flows in the channel during and after rain storms.(W9) | | | | |
| PROVISION WORK IN CHANNEL NO.13 | 20-JUN-11 | EBERHAN | | |
| The permittee shall not use the invert during periods of precipitation or storm flow.(W13) | | | | |
| PROVISION WORK IN CHANNEL NO.15 | 20-JUN-11 | EBERHAN | | |
| Permittee shall not allow any material/debris to fall into the flowing water under any conditions. Any material/debris accidentally deposited in this area shall be removed immediately by the permittee prior to any further work on the bridge.(W15) | | | | |
| PROVISION POLUTION NO. 02 | 20-JUN-11 | EBERHAN | | |
| Permittee shall be responsible for the selection and implementation of Best Management Practices (BMP's) for construction activities. If the Director or authorized representative determines that additional BMP's or corrective steps for existing ones are necessary, permittee shall immediately comply with the requests. (P2) | | | | |

COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS

**Date: 06/20/2011**
**Permit No: PCFL T201004347**

# STANDARD FLOOD CONTROL PERMIT PROVISIONS

A. This permit is valid only for the purpose specified herein. No change of purpose as outlined in application or drawings submitted with application is permitted except upon written permission of the Chief Engineer or his representative.

B. Activities and uses authorized under this permit are subject to any instructions of the Chief Engineer or his representative. **ALL INSTRUCTIONS MUST BE STRICTLY OBSERVED.**

C. Permittee shall assume entire responsibility for all activities and uses under this permit and shall save the District and Los Angeles County free and harmless from any and all expense, cost, or liability in connection with or resulting from the exercise of this permit including, but not limited to, property damage, personal injury, and wrongful death.

D. Any damage caused to Flood Control structures by reason of exercise of this permit shall be repaired, at the permittee's sole expense, to the satisfaction of the District. Should the permittee neglect to promptly make repairs, the District may perform such work or have others perform the work, and the permittee agrees to reimburse the District for all costs of the work so performed upon receipt of a statement thereof.

E. Any structure or portions thereof or plantings placed on District rights of way or which affect District structures must be removed, revised, and/or relocated by permittee without cost to the District, or any other public agency the District shall so designate, should future activities or policy so require.

F. This permit is valid only to the extent of District jurisdiction. Acquisition of permits required by other affected agencies and consent of underlying fee owner(s) of District easement lands are the responsibility of the permittee. **NOTHING CONTAINED IN THIS PERMIT SHALL BE CONSTRUED AS A RELINQUISHMENT OF ANY RIGHTS NOW HELD BY THE DISTRICT.**

G. This permit is subject to all prior unexpired permits, agreements, easements, privileges, or other rights, whether recorded or unrecorded, in the area specified by this permit. Permittee shall make his own arrangements with holders of such prior rights.

H. Unless otherwise specified herein, this permit may be revoked or canceled at any time by the Chief Engineer or his representative when required for District purposes.

I. Upon written notice of cancellation or revocation of this permit for any cause whatsoever, permittee shall restore District right of way and structures to their condition prior to the issuance of the permit and then shall vacate District property. Should permittee neglect to restore the premises or structures to a condition satisfactory to the Chief Engineer or his representative, the District may perform such work or have others perform the work, and the permittee agrees to reimburse the District for all costs of the work so performed upon receipt of a statement thereof.

J. In the event of a District employee work stoppage, the Chief Engineer or his representative reserves the right to suspend all activity authorized under this permit which requires inspection by the District. Activity authorized by the permit shall not resume until District approval to do so is given.

K. Unless otherwise specifically provided, all costs incurred by permittee as a result of the conditions of the permit or exercise by District of any right, authority, or reservation contained therein shall be the sole responsibility of and shall be borne entirely by the permittee.

# California Regional Water Quality Control Board
## Los Angeles Region

320 West Fourth Street, Suite 200, Los Angeles, California 90013
(213) 576-6600 • Fax (213) 576-6640
http://www.waterboards.ca.gov/losangeles

**Linda S. Adams**
*Acting Secretary for Environmental Protection*

**Edmund G. Brown Jr.**
*Governor*


Ms. Linna Wei
Caltrans
100 South Main Street
Los Angeles, CA 90012

**WATER QUALITY CERTIFICATION FOR PROPOSED I-710 BRIDGE WIDENING PROJECT (Corps' Project No. 2011-00206-SJH) LOS ANGELES RIVER, CITY OF DOWNEY, LOS ANGELES COUNTY (File No. 11-020)**

Dear Ms. Wei:

Board staff has reviewed your request on behalf of Caltrans(Applicant) for a Clean Water Act Section 401 Water Quality Certification for the above-referenced project. Your application was deemed complete on May 10, 2011

I hereby issue an order certifying that any discharge from the referenced project will comply with the applicable provisions of sections 301 (Effluent Limitations), 302 (Water Quality Related Effluent Limitations), 303 (Water Quality Standards and Implementation Plans), 306 (National Standards of Performance), and 307 (Toxic and Pretreatment Effluent Standards) of the Clean Water Act, and with other applicable requirements of State law. This discharge is also regulated under State Water Resources Control Board Order No. 2003 - 0017 - DWQ, "General Waste Discharge Requirements for Dredge and Fill Discharges that have received State Water Quality Certification" which requires compliance with all conditions of this Water Quality Certification.

The Applicant shall be liable civilly for any violations of this Certification in accordance with the California Water Code. This Certification does not eliminate the Applicant's responsibility to comply with any other applicable laws, requirements and/or permits.

Should you have questions concerning this Certification action, please contact Dana Cole, Section 401 Program, at (213) 576-5733.

Samuel Unger

Samuel Unger, P.E.
Executive Officer

May 26, 2011
Date

**DISTRIBUTION LIST**

Linna Wei
Caltrans
100 South Main Street
Los Angeles, California 90012

Bill Orme (via electronic copy)
State Water Resources Control Board
Division of Water Quality
P.O. Box 944213
Sacramento, CA 94244-2130

Rick Mayfield
(via electronic copy)
California Department of Fish and Game
Streambed Alteration Team
4949 View Ridge Avenue
San Diego, CA 92123

Cherry Oo
U.S. Army Corps of Engineers
Regulatory Branch, Los Angeles District
P.O. Box 532711
Los Angeles, CA 90053-2325

Eric Raffini (via electronic copy)
U.S. Environmental Protection Agency, Region 9
75 Hawthorne Street
San Francisco, CA 94105

Jim Bartel
U.S. Fish and Wildlife Service
6010 Hidden Valley Road
Carlsbad, CA 92009

## ATTACHMENT A

**Project Information**
**File No. 11-020**

1. Applicant:

Linna Wei
Caltrans
100 South Main Street
Los Angeles, California 90012

Phone: (213) 897-0840 Fax: (213) 897-9572

2. Project Name: I-710 Bridge Widening Project

3. Project Location: Downey, Los Angeles County

| Latitude | Longitude |
|---|---|
| 33.9359 | 118.1753 |
| 33.9360 | 118.1751 |
| 33.9355 | 118.1753 |
| 33.9355 | 118.1752 |
| 33.9360 | 118.1753 |
| 33.9359 | 118.1751 |
| 33.9359 | 118.1750 |
| 33.9355 | 118.1750 |

4. Type of Project: Rehabilitation and Extension of I-710 Roadway

5. Project Purpose: The proposed project (Project) for the 710 Bridge over the Los Angeles River (53-828) (Bridge) will rehabilitate the Long Beach Freeway (Route 710) to enhance safety, improve ride quality, and reduce the need for maintenance. The Bridge will be widened to achieve standard lane, median and shoulder width.

6. Project Description: The Project will widen the northbound side of Route 710 by approximately 16 feet and the southbound side by 12 feet. The Project will also widen the existing piers, abutments and bridge deck over the Los Angeles River (River).

Portions of the Bridge deck and piers will be demolished. Temporary falsework will be installed. The Project will also require

pile driving, temporary access ramps to the river bottom, and a temporary river crossing. The heavy construction equipment to be used will include excavators, loaders, pile driving equipment, cranes, and dump trucks.

Water diversion will be implemented prior to the construction activities in the River. A Surface Water Diversion Plan shall be submitted prior to diversion in compliance with Condition 18 of Attachment B of this certification.

The project is located in a heavily urbanized area. Adjacent land uses are zoned as industrial and commercial, and includes a train yard, warehouses, and office parks. The River at the Long Beach Freeway within the Project area is completely concrete-lined.

The construction window for the bridge work is approximately four months. Work will be performed outside the rain season.

The only vegetation located within the project area is outside of the River and is either ruderal (associated with pre-disturbed areas) or ornamental landscape. No native plant communities are present.

Future maintenance of this bridge will consist of minor construction activities such as sealing joints and re-striping of the roadway. Such construction work can be accessed and conducted from the top of the bridge deck and will not result in any impacts to the River.

| | |
|---|---|
| 7. Federal Agency/Permit: | U.S. Army Corps of Engineers<br>NWP No. 33 (Permit No. 2011-206-SJH) |
| 8. Other Required Regulatory Approvals: | California Department of Fish and Game<br>Streambed Alteration Agreement |
| 9. California Environmental Quality Act Compliance: | The proposed project is Categorically Exempt from CEQA pursuant to the CEQA Guidelines, Section 15301, Existing Facilities. |
| 10. Receiving Water: | Los Angeles River (Hydrologic Unit No. 405.15) |

## ATTACHMENT A

**Project Information**
**File No. 11-020**

| | |
|---|---|
| 11. Designated Beneficial Uses: | MUN*, IND, GWR, REC-1, REC-2, WARM, WILD, RARE, MIGR, SPWN, SHELL, WET<br><br>*Conditional beneficial use |
| 12. Impacted Waters of the United States: | Non-wetland waters (unvegetated streambed): 0.06 temporary acres (2,574 linear feet) and 0.05 permanent acres (1,980 linear feet) |
| 13. Dredge Volume: | None. |
| 14. Related Projects Implemented/to be Implemented by the Applicant: | The Applicant has identified the following related projects carried out in the last 5 years or planned for implementation in the next 5 years:<br><br>• Widening of Long Beach Freeway: Los Angeles County in Long Beach between Pico Ave & Long Beach Boulevard (from Post Mile 4.9 to 24.9). Tentative construction window: April 2012 through August 2013.<br><br>• Widening of Long Beach Freeway Shoemaker Bridge. Tentative construction window: November 2014 through October 2016. |
| 15. Avoidance/ Minimization Activities: | The Applicant has proposed to implement several Best Management Practices, including, but not limited to, the following:<br><br>• Construction materials such as Portland Cement and concrete or asphalt or aggregate will be prevented from coming in contact with storm water.<br><br>• Temporary barriers will be installed around stockpiled perimeters to prevent contact of materials with storm water.<br><br>• Oil, lubricants, or any hazardous or contaminant waste will be kept 100 feet from the top of banks of the River.<br><br>• Temporary barriers and containers will be used to confine any contaminated materials. |

- Upon completion of construction all contaminated material will be removed to a legal point of disposal.
- A temporary spill containment system will be installed and maintained within Project limits during construction.
- A containment plan will be provided in the event of a hazardous material spill.
- Entrances and exits will be stabilized.
- Non-paved surfaces will be watered.
- Paved surfaces will be swept and vacuumed.
- Silt fences will be installed at the toe of excavation and embankment slopes.
- Sand or gravel bag berms will be installed along the top of slopes.
- Slope protection such as geotextiles, plastic covers, soil binders, and erosion control mats, and fiber rolls will be installed.
- Storm drain inlets will be protected with inserts or linear interrupters such as gravel bag and/or sand bag berms.
- Work will be conducted outside the winter rain season from November 1st through April 1st.
- Storage of materials will not occur in the channel, below the top of bank of the River.
- Project limits will be marked upstream and downstream of the work area.

16. Proposed Compensatory Mitigation: None.

**ATTACHMENT A**

**Project Information**
**File No. 11-020**

| | |
|---|---|
| 17. Required Compensatory Mitigation: | The Regional Board will not require compensatory mitigation due to the small size of the impact and because the modifications are concrete-to-concrete. . See *Attachment B, Conditions of Certifications, Additional Conditions* for modifications and additions. |

# ATTACHMENT B

## Conditions of Certification
## File No. 11-020

## STANDARD CONDITIONS

Pursuant to §3860 of Title 23 of the California Code of Regulations (23 CCR), the following three standard conditions shall apply to this project:

1. This Certification action is subject to modification or revocation upon administrative or judicial review, including review and amendment pursuant to §13330 of the California Water Code and Article 6 (commencing with 23 CCR §3867).

2. This Certification action is not intended and shall not be construed to apply to any activity involving a hydroelectric facility and requiring a Federal Energy Regulatory Commission (FERC) license or an amendment to a FERC license unless the pertinent Certification application was filed pursuant to 23 CCR Subsection 3855(b) and the application specifically identified that a FERC license or amendment to a FERC license for a hydroelectric facility was being sought.

3. Certification is conditioned upon total payment of any fee required pursuant to 23 CCR Chapter 28 and owed by the Applicant.

## ADDITIONAL CONDITIONS

Pursuant to 23 CCR §3859(a), the Applicant shall comply with the following additional conditions:

1. The Applicant shall submit to this Regional Board copies of any other final permits and agreements required for this project, including, but not limited to, the U.S. Army Corps of Engineers' (ACOE) Section 404 Permit and the California Department of Fish and Game's (CDFG) Streambed Alteration Agreement. **These documents shall be submitted prior to any discharge to waters of the State.**

2. The Applicant shall adhere to the most stringent conditions indicated with either this Certification, the CDFG's Streambed Alteration Agreement, or the ACOE Section 404 Permit.

3. The Applicant shall comply with all water quality objectives, prohibitions, and policies set forth in the *Water Quality Control Plan, Los Angeles Region (1994),* as amended.

4. The Avoidance/Minimization activities proposed by the Applicant as described in Attachment A, No. 15, are incorporated as additional conditions herein.

5. The Applicant and all contractors employed by the Applicant shall have copies of this Certification and all other regulatory approvals for this project on site at all times and shall be familiar with all conditions set forth.

## ATTACHMENT B

### Conditions of Certification
### File No. 11-020

6. Fueling, lubrication, maintenance, operation, and storage of vehicles and equipment shall not result in a discharge or a threatened discharge to waters of the State. At no time shall the Applicant use any vehicle or equipment which leaks any substance that may impact water quality. Staging and storage areas for vehicles and equipment shall be located outside of waters of the State.

7. All excavation, construction, or maintenance activities shall follow best management practices to minimize impacts to water quality and beneficial uses. Dust control activities shall be conducted in such a manner that will not produce downstream runoff.

8. No construction material, spoils, debris, or any other substances associated with this project that may adversely impact water quality standards, shall be located in a manner which may result in a discharge or a threatened discharge to waters of the State. Designated spoil and waste areas shall be visually marked prior to any excavation and/or construction activity, and storage of the materials shall be confined to these areas.

9. All waste or dredged material removed shall be relocated to a legal point of disposal if applicable. A legal point of disposal is defined as one for which Waste Discharge Requirements have been established by a California Regional Water Quality Control Board, and is in full compliance therewith. Please contact the Land Disposal Unit at (213) 620-2253 for further information regarding the disposal of solid wastes.

10. The Applicant shall implement all necessary control measures to prevent the degradation of water quality from the proposed project in order to maintain compliance with the Basin Plan. The discharge shall meet all effluent limitations and toxic and effluent standards established to comply with the applicable water quality standards and other appropriate requirements, including the provisions of Sections 301, 302, 303, 306, and 307 of the Clean Water Act. This Certification does not authorize the discharge by the applicant for any other activity than specifically described in the 404 Permit.

11. The discharge shall not: a) degrade surface water communities and populations including vertebrate, invertebrate, and plant species; b) promote the breeding of mosquitoes, gnats, black flies, midges, or other pests; c) alter the color, create visual contrast with the natural appearance, nor cause aesthetically undesirable discoloration of the receiving waters; d) cause formation of sludge deposits; or e) adversely affect any designated beneficial uses.

12. The Applicant shall allow the Regional Board and its authorized representative entry to the premises, including all mitigation sites, to inspect and undertake any activity to determine compliance with this Certification, or as otherwise authorized by the California Water Code.

13. The Applicant shall not conduct any construction activities within waters of the State during a rainfall event. The Applicant shall maintain **a five-day (5-day) clear weather forecast** before conducting any operations within waters of the State.

14. If rain is predicted after operations have begun, grading activities must cease immediately and the site must be stabilized to prevent impacts to water quality, and minimize erosion and runoff from the site.

15. No activities shall involve wet excavations (i.e., no excavations shall occur below the seasonal high water table). A minimum **5-foot** buffer zone shall be maintained above the existing groundwater level. If construction or groundwater dewatering is proposed or anticipated, the Applicant shall file a **Report of Waste Discharge** (ROWD) to this Regional Board and obtain any necessary NPDES permits/Waste Discharge Requirements prior to discharging waste.

16. Sufficient time should be allowed to obtain any such permits (generally 180 days). If groundwater is encountered without the benefit of appropriate permits, the Applicant shall cease all activities in the areas where groundwater is present, file a Report of Waste Discharge to this Regional Board, and obtain any necessary permits prior to discharging waste.

17. All project, construction, or maintenance activities not included in this Certification, and which may require a permit, must be reported to the Regional Board for appropriate permitting. Bank stabilization and grading, as well as any other ground disturbances, are subject to restoration and revegetation requirements, and may require additional Certification action.

18. All surface waters, including ponded waters, shall be diverted away from areas undergoing grading, construction, excavation, vegetation removal, and/or any other activity which may result in a discharge to the receiving water. If surface water diversions are anticipated, the Applicant shall develop and submit a **Surface Water Diversion Plan** (plan) to this Regional Board. The plan shall include the proposed method and duration of diversion activities, structure configuration, construction materials, equipment, erosion and sediment controls, and a map or drawing indicating the locations of diversion and discharge points. Contingency measures shall be a part of this plan to address various flow discharge rates. The plan shall be submitted prior to any surface water diversions. If surface flows are present, then upstream and downstream monitoring for the following shall be implemented:
    - pH
    - temperature
    - dissolved oxygen
    - turbidity
    - total suspended solids(TSS)

## ATTACHMENT B

### Conditions of Certification
### File No. 11-020

Analyses must be performed using approved US Environmental Protection Agency methods, where applicable. These constituents shall be measured at least once prior to diversion and then monitored for on a daily basis during the first week of diversion and/or dewatering activities, and then on a weekly basis, thereafter, until the in-stream work is complete.

Results of the analyses shall be submitted to this Regional Board by the 15th day of each subsequent sampling month. A map or drawing indicating the locations of sampling points shall be included with each submittal. Diversion activities shall not result in the degradation of beneficial uses or exceedance of water quality objectives of the receiving waters. Downstream TSS shall be maintained at ambient levels. Where natural turbidity is between 0 and 50 Nephelometric Turbidity Units (NTU), increases shall not exceed 20%. Where natural turbidity is greater than 50 NTU, increases shall not exceed 10%. Any such violations may result in corrective and/or enforcement actions, including increased monitoring and sample collection.

19. The Applicant shall restore any areas of TEMPORARY IMPACTS to waters of the United States and all other areas of temporary disturbance which could result in a discharge or a threatened discharge to waters of the State. The Applicant shall implement all necessary Best Management Practices to control erosion and runoff from areas associated with this project.

20. The Applicant shall submit to this Regional Board **Annual Project and Monitoring Reports** (Annual Reports) by **January 1st** of each year for a minimum period of **five (5) years** following this issuance of 401 Certification or until project completion has been achieved and documented. The Annual Reports shall describe in detail all of the project/construction activities performed during the previous year and all restoration and mitigation efforts; including percent survival by plant species and percent cover. The Annual Reports shall describe the status of other agreements. At a minimum the Annual Reports shall include the following documentation:

    (a) Color photo documentation of the pre- and post-project site conditions;

    (b) Geographical Positioning System (GPS) coordinates in decimal-degrees format outlining the boundary of the project and mitigation areas;

    (c) The overall status of project including a detailed schedule of work;

    (d) Copies of all permits revised as required in Additional Condition 1;

    (e) Water quality monitoring results for each reach (as required) compiled in an easy to interpret format;

    (f) A certified Statement of "no net loss" of wetlands associated with this project;

(g) Discussion of any monitoring activities and exotic plant control efforts; and

(h) A certified Statement from the permittee or his/her representative that all conditions of this Certification have been met.

21. All applications, reports, or information submitted to the Regional Board shall be signed:

    (a) For corporations, by a principal executive officer at least of the level of vice president or his duly authorized representative, if such representative is responsible for the overall operation of the facility from which discharge originates.

    (b) For a partnership, by a general partner.

    (c) For a sole proprietorship, by the proprietor.

    (d) For a municipal, State, or other public facility, by either a principal executive officer, ranking elected official, or other duly authorized employee.

22. Each and any report submitted in accordance with this Certification shall contain the following completed declaration:

    "I declare under penalty of law that this document and all attachments were prepared under my direction or supervision in accordance with a system designed to assure that qualified personnel properly gather and evaluate the information submitted. Based on my inquiry of the person or persons who managed the system or those directly responsible for gathering the information, the information submitted is, to the best of my knowledge and belief, true, accurate and complete. I am aware that there are significant penalties for submitting false information, including the possibility of fine and imprisonment for knowing violations.

    Executed on the _____ day of _____ at _____.

    _____ (Signature)
    _____ (Title)"

23. All communications regarding this project and submitted to this Regional Board shall identify the Project File Number 11-020. Submittals shall be sent to the attention of the 401 Certification Unit.

24. Any modifications of the proposed project may require submittal of a new Clean Water Act Section 401 Water Quality Certification application and appropriate filing fee.

## ATTACHMENT B

### Conditions of Certification
### File No. 11-020

25. The project shall comply with the local regulations associated with the Regional Board's **Municipal Stormwater Permit** issued to the Department of Transportation (Caltrans) under NPDES No. CAS000003 and Waste Discharge Requirements Order No. 99 - 06 - DWQ. This includes the Standard Urban Storm Water Mitigation Plan (SUSMP) and all related implementing local ordinances and regulations for the control of stormwater pollution from new development and redevelopment.

26. The project shall comply with all requirements of the National Pollutant Discharge Elimination System (NPDES) **General Permit** for Storm Water Discharges Associated with Construction Activity, Order No. 2009-009-DWQ. All stormwater treatment systems shall be located outside of any water of the State and shall not be used as a wetland or riparian mitigation credit.

27. Coverage under this Certification may be transferred to the extent the underlying federal permit may legally be transferred and further provided that the Applicant notifies the Executive Officer at least 30 days before the proposed transfer date, and the notice includes a written agreement between the existing and new Applicants containing a specific date of coverage, responsibility for compliance with this Certification, and liability between them.

28. The Applicant or their agents shall report any noncompliance. Any such information shall be provided verbally to the Executive Officer within 24 hours from the time the Applicant becomes aware of the circumstances. A written submission shall also be provided within five days of the time the Applicant becomes aware of the circumstances. The written submission shall contain a description of the noncompliance and its cause; the period of noncompliance, including exact dates and times, and if the noncompliance has not been corrected; the anticipated time it is expected to continue and steps taken or planned to reduce, eliminate and prevent recurrence of the noncompliance. The Executive Officer, or an authorized representative, may waive the written report on a case-by-case basis if the oral report has been received within 24 hours.

29. *Enforcement:*

    (a) In the event of any violation or threatened violation of the conditions of this Certification, the violation or threatened violation shall be subject to any remedies, penalties, process or sanctions as provided for under State law. For purposes of section 401(d) of the Clean Water Act, the applicability of any State law authorizing remedies, penalties, process or sanctions for the violation or threatened violation constitutes a limitation necessary to assure compliance with the water quality standards and other pertinent requirements incorporated into this Certification.

    (b) In response to a suspected violation of any condition of this Certification, the State Water Resources Control Board (SWRCB) or Regional Water Quality Control Board (RWQCB) may require the holder of any permit or license subject to this Certification

to furnish, under penalty of perjury, any technical or monitoring reports the SWRCB deems appropriate, provided that the burden, including costs, of the reports shall be a reasonable relationship to the need for the reports and the benefits to be obtained from the reports.

(c) In response to any violation of the conditions of this Certification, the SWRCB or RWQCB may add to or modify the conditions of this Certification as appropriate to ensure compliance.

30. This Certification shall expire **five (5) years** from date of this Certification. The Applicant shall submit a complete application prior to termination of this Certification if renewal is requested.

State of California –The Natural Resources Agency
DEPARTMENT OF FISH AND GAME
South Coast Region
4949 Viewridge Avenue
San Diego, CA 92123
(858) 467-4201
www.dfg.ca.gov

**EDMUND G. BROWN, JR.**, *Governor*
**JOHN McCAMMAN,** *Director*

March 07, 2011

Ms. Linna Wei
California Department of Transportation
100 South Main Street
Los Angeles, California 90012

Subject: Notification of Lake or Streambed Alteration No. 1600-2010-0364-R5
LA-710 Roadway Rehabilitation and Bridge Widening Project
Los Angeles River at Slauson and Ramona Boulevard

Dear Ms. Linna Wei:

The Department of Fish and Game (Department) had until March 07, 2011 to submit a draft Lake or Streambed Alteration Agreement (Agreement) to you or inform you that an Agreement is not required. The Department did not meet that date. As a result, by law, you may now complete the project described in your notification without an Agreement.

Please note that pursuant to Fish and Game Code section 1602(a)(4)(D), if you proceed with this project, it must be the same as described and conducted in the same manner as specified in the notification and any modifications to that notification received by the Department in writing prior to March 07, 2011. This includes completing the project within the proposed term and seasonal work period and implementing all avoidance and mitigation measures to protect fish and wildlife resources specified in the notification. If the term proposed in your notification has expired, you will need to re-notify the Department before you may begin your project. Beginning or completing a project that differs in any way from the one described in the notification may constitute a violation of Fish and Game Code section 1602.

Your notification includes, but is not limited to, the following information: Project activities shall take place at Post Mile Marker 17.2/17.5 and 20.6/26.4 at the Los Angeles River Bridge located 2500 feet south of Slauson to Ramona Boulevard along State Route 710 (SR-710). The existing vegetation will be cleared to accommodate the bridge widening effort and should be conducted outside the nesting bird season, or only after appropriate surveys, as indicated in the Natural Environmental Study (NES), Page 3. There shall be no impacts to nesting or roosting bats and swallows. All project specific Best Management Practices (BMP's) shall be implemented by Caltrans as proposed in the NES to protect fish and wildlife resources and prevent impacts to water quality. Project related activities shall be completed by July 01, 2014.

Also note that while you are entitled to complete the project without an Agreement, you are still responsible for complying with other applicable local, state, and federal laws. These include, but are not limited to, the state and federal Endangered Species Acts, Federal Migratory Bird Treaty Act (MBTA) of 1918 (50 C.F.R. Section 10.13), Fish and Game Code sections 3503, 3503.5 and 3513 (nesting birds), 5650 (water pollution), and 5901 (fish passage).

Finally, if you decide to proceed with your project without an Agreement, you must have a copy of this letter and your notification with all attachments available at all times at the work site. If you have any questions regarding this matter, please contact Jamie Jackson at 626-513-6308 or jjackson@dfg.ca.gov.

Sincerely,

Jamie Jackson

Jamie Jackson
Staff Environmental Scientist

| FOR DEPARTMENT USE ONLY | | | | |
|---|---|---|---|---|
| Date Received | Amount Received | Amount Due | Date Complete | Notification No. |
| | $ | $ | | |

STATE OF CALIFORNIA

DEPARTMENT OF FISH AND GAME

# NOTIFICATION OF LAKE OR STREAMBED ALTERATION

**Complete EACH field, unless otherwise indicated, following the enclosed instructions and submit ALL required enclosures. Attach additional pages, if necessary.**

## 1. APPLICANT PROPOSING PROJECT

| | | | |
|---|---|---|---|
| Name | Paul Caron | | |
| Business/Agency | California Department of Transportation, Caltrans, District 7 | | |
| Street Address | 100 South Main Street | | |
| City, State, Zip | Los Angeles, CA 90012 | | |
| Telephone | (213) 897-0610 | Fax | (213) 897-0685 |
| Email | Paul_d_caron@dot.ca.gov | | |

## 2. CONTACT PERSON (*Complete only if different from applicant*)

| | | | |
|---|---|---|---|
| Name | Linna Wei | | |
| Street Address | 100 South Main Street | | |
| City, State, Zip | Los Angeles, CA 90012 | | |
| Telephone | (213) 897-0840 | Fax | (213) 897-0685 |
| Email | Linna_wei@dot.ca.gov | | |

## 3. PROPERTY OWNER (*Complete only if different from applicant*)

| | | | |
|---|---|---|---|
| Name | | | |
| Street Address | | | |
| City, State, Zip | | | |
| Telephone | | Fax | |
| Email | | | |

## 4. PROJECT NAME AND AGREEMENT TERM

| | | | | |
|---|---|---|---|---|
| A. Project Name | LA-710 Roadway Rehabilitation and Bridge Widening Project | | | |
| B. Agreement Term Requested | ☑ Regular (*5 years or less*)<br>☐ Long-term (*greater than 5 years*) | | | |
| C. Project Term | | D. Seasonal Work Period | | E. Number of Work Days |
| Beginning (*year*) | Ending (*year*) | Start Date (*month/day*) | End Date (*month/day*) | |
| 2011 | 2011 | 05/01 | 10/01 | 100.00 |

## 5. AGREEMENT TYPE

Check the applicable box. If box B, C, D, or E is checked, complete the specified attachment.

| | | |
|---|---|---|
| A. | ☑ Standard *(Most construction projects, excluding the categories listed below)* | |
| B. | ☐ Gravel/Sand/Rock Extraction (*Attachment A*) | *Mine I.D. Number:* _____ |
| C. | ☐ Timber Harvesting (*Attachment B*) | THP *Number:* _____ |
| D. | ☑ Water Diversion/Extraction/Impoundment (*Attachment C*) | *SWRCB Number:* Pending |
| E. | ☐ Routine Maintenance (*Attachment D*) | |
| F. | ☐ DFG Fisheries Restoration Grant Program (FRGP) | *FRGP Contract Number:* _____ |
| G. | ☐ Master | |
| H. | ☐ Master Timber Harvesting | |

## 6. FEES

Please see the current fee schedule to determine the appropriate notification fee. Itemize each project's estimated cost and corresponding fee. ***Note: The Department may not process this notification until the correct fee has been received.***

| | A. Project | B. Project Cost | C. Project Fee |
|---|---|---|---|
| 1 | LA-710 Roadway Rehabilitation and Bridge Widening Project | $15,850,000.00 | $4,482.75 |
| 2 | | | |
| 3 | | | |
| 4 | | | |
| 5 | | | |
| | | D. Base Fee *(if applicable)* | |
| | | **E. TOTAL FEE ENCLOSED** | $4,482.75 |

## 7. PRIOR NOTIFICATION OR ORDER

A. Has a notification previously been submitted to, or a Lake or Streambed Alteration Agreement previously been issued by, the Department for the project described in this notification?

☐ Yes (*Provide the information below*) ☑ No

Applicant: _____ Notification Number: _____ Date: _____

B. Is this notification being submitted in response to an order, notice, or other directive ("order") by a court or administrative agency (including the Department)?

☑ No ☐ Yes (*Enclose a copy of the order, notice, or other directive. If the directive is not in writing, identify the person who directed the applicant to submit this notification and the agency he or she represents, and describe the circumstances relating to the order.*)

☐ *Continued on additional page(s)*

## 8. PROJECT LOCATION

A. Address or description of project location.

(*Include a map that marks the location of the project with a reference to the nearest city or town, and provide driving directions from a major road or highway*)

The proposed project limits are from Post Mile (PM) 17.2/17.5 and PM 20.6/26.4. This includes the Los Angeles River Bridge from approximately 2500 ft. south of Slauson to Ramona Blvd. along State Route 710 in the cities of Los Angeles, Monterey Park, Vernon, Maywood, Bell, Cudahy, Bell Gardens, and Downey, Los Angeles County.

Los Angeles River Bridge is located at PM 17.2/17.5 is on the State Route 710.

Please see attached continuation sheet for project location map and aerial photograph.

☑ *Continued on additional page(s)*

| | |
|---|---|
| B. River, stream, or lake affected by the project. | Los Angeles River |
| C. What water body is the river, stream, or lake tributary to? | Pacific Ocean |
| D. Is the river or stream segment affected by the project listed in the state or federal Wild and Scenic Rivers Acts? | ☐ Yes ☑ No ☐ Unknown |
| E. County | Los Angeles County |

| F. USGS 7.5 Minute Quad Map Name | G. Township | H. Range | I. Section | J. ¼ Section |
|---|---|---|---|---|
| South Gate and Los Angeles | 1S | 18W | SE 37 | |
| | | | | |
| | | | | |
| | | | | |

☐ *Continued on additional page(s)*

| | |
|---|---|
| K. Meridian (*check one*) | ☐ Humboldt ☐ Mt. Diablo ☐ San Bernardino |

L. Assessor's Parcel Number(s)

N/A

☐ *Continued on additional page(s)*

M. Coordinates (*If available, provide at least latitude/longitude or UTM coordinates and check appropriate boxes*)

| | | | |
|---|---|---|---|
| Latitude/Longitude | *Latitude:* 33.935776 | *Longitude:* -118.175039 | |
| | ☑ Degrees/Minutes/Seconds | ☐ Decimal Degrees | ☐ Decimal Minutes |
| UTM | *Easting:* | *Northing:* | ☐ Zone 10 ☐ Zone 11 |
| Datum used for Latitude/Longitude or UTM | ☐ NAD 27 | ☐ NAD 83 or WGS 84 | |

**9. PROJECT CATEGORY AND WORK TYPE** *(Check each box that applies)*

| PROJECT CATEGORY | NEW CONSTRUCTION | REPLACE EXISTING STRUCTURE | REPAIR/MAINTAIN EXISTING STRUCTURE |
|---|---|---|---|
| Bank stabilization – bioengineering/recontouring | ☐ | ☐ | ☐ |
| Bank stabilization – rip-rap/retaining wall/gabion | ☐ | ☐ | ☐ |
| Boat dock/pier | ☐ | ☐ | ☐ |
| Boat ramp | ☐ | ☐ | ☐ |
| Bridge | ☑ | ☐ | ☐ |
| Channel clearing/vegetation management | ☐ | ☐ | ☐ |
| Culvert | ☐ | ☐ | ☐ |
| Debris basin | ☐ | ☐ | ☐ |
| Dam | ☐ | ☐ | ☐ |
| Diversion structure – weir or pump intake | ☐ | ☐ | ☐ |
| Filling of wetland, river, stream, or lake | ☐ | ☐ | ☐ |
| Geotechnical survey | ☐ | ☐ | ☐ |
| Habitat enhancement – revegetation/mitigation | ☐ | ☐ | ☐ |
| Levee | ☐ | ☐ | ☐ |
| Low water crossing | ☐ | ☐ | ☐ |
| Road/trail | ☐ | ☐ | ☐ |
| Sediment removal – pond, stream, or marina | ☐ | ☐ | ☐ |
| Storm drain outfall structure | ☐ | ☐ | ☐ |
| Temporary stream crossing | ☐ | ☐ | ☐ |
| Utility crossing : Horizontal Directional Drilling | ☐ | ☐ | ☐ |
| Jack/bore | ☐ | ☐ | ☐ |
| Open trench | ☐ | ☐ | ☐ |
| **Other** *(specify)*: | ☐ | ☐ | ☐ |

**10. PROJECT DESCRIPTION**

A. Describe the project in detail. Photographs of the project location and immediate surrounding area should be included.

- Include any structures (e.g., rip-rap, culverts, or channel clearing) that will be placed, built, or completed in or near the stream, river, or lake.
- Specify the type and volume of materials that will be used.
- If water will be diverted or drafted, specify the purpose or use.

Enclose diagrams, drawings, plans, and/or maps that provide all of the following: site specific construction details; the dimensions of each structure and/or extent of each activity in the bed, channel, bank or floodplain; an overview of the entire project area (i.e., "bird's-eye view") showing the location of each structure and/or activity, significant area features, and where the equipment/machinery will enter and exit the project area.

The purpose of this roadway rehabilitation project is to improve ride quality, to enhance safety and to reduce the need for maintenance. The proposed project consists widening of the Los Angeles River Bridge (53-828) to achieve standard lanes, median, and shoulder. This rehabilitation of the existing pavement section will bring the lanes and shoulders to the current design standards.

Please see the enclosed Natural Environment Study Memo (NES), plans, and maps for more details.

☑ *Continued on additional page(s)*

B. Specify the equipment and machinery that will be used to complete the project.

Excavators, Loaders, Pile driving equipment, Cranes, Dump trucks.

☐ *Continued on additional page(s)*

| | |
|---|---|
| C. Will water be present during the proposed work period (specified in box 4.D) in the stream, river, or lake (specified in box 8.B). | ☑ Yes ☐ No (*Skip to box 11*) |
| D. Will the proposed project require work in the wetted portion of the channel? | ☑ Yes (*Enclose a plan to divert water around work site*)<br>☐ No |

## 11. PROJECT IMPACTS

A. Describe impacts to the bed, channel, and bank of the river, stream, or lake, and the associated riparian habitat. Specify the dimensions of the modifications in length (linear feet) and area (square feet or acres) and the type and volume of material (cubic yards) that will be moved, displaced, or otherwise disturbed, if applicable.

The project proposes to widen 10 ft. each side of the Los Angeles River Bridge (53-828) to standard shoulder widths. The project will have linear impact in the concrete-lined Los Angeles River from 34 ft upstream to 35 ft downstream of the bridge. The permanent impacted area is calculated to be 0.05 acres (1,980 sq. ft) and the temporary impacted area is calculated to be 0.06 acres (2,574 sq. ft). The amount of excavation is estimated to be 2,786 cubic yards.

☐ *Continued on additional page(s)*

B. Will the project affect any vegetation? ☐ Yes (*Complete the tables below*) ☑ No

| Vegetation Type | Temporary Impact | Permanent Impact |
|---|---|---|
| | Linear feet: _____<br>Total area: _____ | Linear feet: _____<br>Total area: _____ |
| | Linear feet: _____<br>Total area: _____ | Linear feet: _____<br>Total area: _____ |

| Tree Species | Number of Trees to be Removed | Trunk Diameter (range) |
|---|---|---|
| | | |
| | | |
| | | |

☐ *Continued on additional page(s)*

C. Are any special status animal or plant species, or habitat that could support such species, known to be present on or near the project site?

☐ Yes *(List each species and/or describe the habitat below)* ☑ No ☐ Unknown

☐ *Continued on additional page(s)*

D. Identify the source(s) of information that supports a "yes" or "no" answer above in Box 11.C.

The California Natural Diversity Database (CNDDB) was researched and the result indicted that special status species are not expected to be found on the project site, or directly adjacent.

☐ *Continued on additional page(s)*

E. Has a biological study been completed for the project site?

☑ Yes (*Enclose the biological study*) ☐ No

*Note: A biological assessment or study may be required to evaluate potential project impacts on biological resources.*

F. Has a hydrological study been completed for the project or project site?

☑ Yes (*Enclose the hydrological study*) ☐ No

*Note: A hydrological study or other information on site hydraulics (e.g., flows, channel characteristics, and/or flood recurrence intervals) may be required to evaluate potential project impacts on hydrology.*

## 12. MEASURES TO PROTECT FISH, WILDLIFE, AND PLANT RESOURCES

A. Describe the techniques that will be used to prevent sediment from entering watercourses during and after construction.

All applicable construction Best Management Practices (BMPs) for water quality shall be implemented to minimize project affects to jurisdictional drainages.

☐ *Continued on additional page(s)*

B. Describe project avoidance and/or minimization measures to protect fish, wildlife, and plant resources.

-Work will be conducted outside the winter rain season at the Los Angeles River Bridge. The winter rain season is defined as November 1 to April 1. This will minimize the chance for introduction of toxic materials to the channel.
-Storage of materials will not occur in the channel, herein defined as top of bank to top of bank. This will avoid any chance that flow will be impeded.
-Project limits will need to be marked, upstream and downstream of the work area, at Los Angeles River Bridge, as a first order of work at this location.

Please see the enclosed NES for more details.

☐ *Continued on additional page(s)*

C. Describe any project mitigation and/or compensation measures to protect fish, wildlife, and plant resources.

No mitigation and/or compensation measures are proposed at this time. The project is located in a heavily urbanized area, with adjacent land uses being industrial and commercial. The only drainage in the area is the Los Angeles River, which at Route 710 is completely concrete-lined with minimal biological resources.

Please see the enclosed NES for more details.

☐ *Continued on additional page(s)*

## 13. PERMITS

List any local, state, and federal permits required for the project and check the corresponding box(es). Enclose a copy of each permit that has been issued.

A. Army Corps of Eng. Section 404 Permit (Pending) ☑ Applied ☐ Issued

B. Regional Water Quality Control Board 401 Cert. (Pending) ☑ Applied ☐ Issued

C. _____ ☐ Applied ☐ Issued

D. Unknown whether ☐ local, ☐ state, or ☐ federal permit is needed for the project. (*Check each box that applies*)

☐ *Continued on additional page(s)*

## 14. ENVIRONMENTAL REVIEW

A. Has a draft or final document been prepared for the project pursuant to the California Environmental Quality Act (CEQA), National Environmental Protection Act (NEPA), California Endangered Species Act (CESA) and/or federal Endangered Species Act (ESA)?

☑ Yes (*Check the box for each CEQA, NEPA, CESA, and ESA document that has been prepared and enclose a copy of each*)

☐ No (*Check the box for each CEQA, NEPA, CESA, and ESA document listed below that will be or is being prepared*)

| | | |
|---|---|---|
| ☐ Notice of Exemption | ☐ Mitigated Negative Declaration | ☐ NEPA document (*type*): _____ |
| ☐ Initial Study | ☐ Environmental Impact Report | ☐ CESA document (*type*): _____ |
| ☐ Negative Declaration | ☐ Notice of Determination *(Enclose)* | ☐ ESA document (*type*): _____ |
| ☐ THP/ NTMP | ☐ Mitigation, Monitoring, Reporting Plan | |

| | | | |
|---|---|---|---|
| B. State Clearinghouse Number (*if applicable*) | | | |
| C. Has a CEQA lead agency been determined? | ☑ Yes (*Complete boxes D, E, and F*) | ☐ No (*Skip to box 14.G*) | |
| D. CEQA Lead Agency | Caltrans | | |
| E. Contact Person | Brian Manor | F. Telephone Number | (213) 897-0704 |

G. If the project described in this notification is part of a larger project or plan, briefly describe that larger project or plan.

The LA 710 / LA River Project better known as the I-710 Long Life Pavement Project. The original project scope summary report (PSSR) proposed to rehabilitate the pavement structural sections and replace the existing median double metal beam barrier with Type 60 concrete barrier from Del Amo Boulevard to Route 10 Freeway, PM 10.8 / 26.5, dated October 16, 1997. The original scope of the project was to replace lanes, numbers 3 and 4, with long life Portland Cement Concrete (PCC) pavement and replace damaged PCC slabs at spot locations on lanes numbers 1 and 2. In addition, the report called for widening the outside shoulders to standard width of 10 feet.

☐ *Continued on additional page(s)*

H. Has an environmental filing fee (Fish and Game Code section 711.4) been paid?

☑ Yes (*Enclose proof of payment*) ☐ No (*Briefly explain below the reason a filing fee has not been paid)*

*Note: If a filing fee is required, the Department may not finalize a Lake or Streambed Alteration Agreement until the filing fee is paid.*

## 15. SITE INSPECTION

Check one box only.

☑ In the event the Department determines that a site inspection is necessary, I hereby authorize a Department representative to enter the property where the project described in this notification will take place at any reasonable time, and hereby certify that I am authorized to grant the Department such entry.

☐ I request the Department to first contact (*insert name*) _____ at (*insert telephone number*) _____ to schedule a date and time to enter the property where the project described in this notification will take place. I understand that this may delay the Department's determination as to whether a Lake or Streambed Alteration Agreement is required and/or the Department's issuance of a draft agreement pursuant to this notification.

## 16. DIGITAL FORMAT

Is any of the information included as part of the notification available in digital format (i.e., CD, DVD, etc.)?

☐ Yes (Please enclose the information via digital media with the completed notification form)

☑ No

## 17. SIGNATURE

I hereby certify that to the best of my knowledge the information in this notification is true and correct and that I am authorized to sign this notification as, or on behalf of, the applicant. I understand that if any information in this notification is found to be untrue or incorrect, the Department may suspend processing this notification or suspend or revoke any draft or final Lake or Streambed Alteration Agreement issued pursuant to this notification. I understand also that if any information in this notification is found to be untrue or incorrect and the project described in this notification has already begun, I and/or the applicant may be subject to civil or criminal prosecution. I understand that this notification applies only to the project(s) described herein and that I and/or the applicant may be subject to civil or criminal prosecution for undertaking any project not described herein unless the Department has been separately notified of that project in accordance with Fish and Game Code section 1602 or 1611.

Signature of Applicant or Applicant's Authorized Representative

11/30/10
Date

Linna Wei
Print Name

**State of California** **Business, Transportation and Housing Agency**

# M e m o r a n d u m

***Flex your power!***
***Be energy efficient!***

**To:** MS. CHUNG-YUAN WEN
Office of Bridge Design South 2
Bridge Design Branch 21
21073 Pathfinder Rd, Suite 20

Attention: MR. TIEN CHU

**Date:** October 29, 2010
**File**: 07-LA-710-PM 21.9
07-202111
Atlantic Blvd. UC On-Ramp
(Replace)
Bridge No. 53-3067

**From:** **DEPARTMENT OF TRANSPORTATION**
**DIVISION OF ENGINEERING SERVICES**
**Geotechnical Services**
**Office of Geotechnical Design South 1**
**Branch D**

**Subject:** Revised Foundation Report for Atlantic Blvd UC On-Ramp (Replace), Interstate 710

## INTRODUCTION

This report was prepared in response to the Office of Bridge Design South 2's (OBDS2's) initial memorandum dated May 21, 2009 requesting Foundation Recommendations for the proposed replacement of Interstate 710 (I-710) Atlantic Boulevard Southbound On-Ramp (Br. No. 53-1008K) with the new bridge (Br. No. 53-3067) and based on OBDS2's e-mail request (dated September 28, 2010) with increased (revised) loads at Abutments.

## SCOPE OF WORK

The recommendations contained in this report are based on the subsurface information gathered during the recent field investigation performed from July to September, 2009. Tasks completed by the Office of Geotechnical Design South 1 (OGDS1), Branch D include the following:

1. Review of the regional geology and seismicity,
2. Review of pertinent information from previous geology/geotechnical reports and As-Built plans in the project area to evaluate the subsurface information,
3. Drilling, logging, and sampling of two rotary wash borings, and soundings from two electronic cone penetrometer tests at the subject bridge site to characterize the subsurface conditions,
4. Laboratory testing of selected soil samples for mechanical analysis, corrosivity, direct shear, unit weight, Atterberg Limits, and moisture content,
5. Geotechnical engineering analysis.

## PROJECT DESCRIPTION

The existing Atlantic Blvd Southbound (S/B) On-Ramp (Br. No. 53-1008K), located at the boundary between the cities of Vernon and Bell, was originally built in 1957. The Los Angeles River is about 0.28 mile southwest of Atlantic Blvd S/B On-Ramp. The Atlantic Blvd S/B On-Ramp is a continuous two-span reinforced concrete box girder (3 cells) with closed end abutment walls, and single column bent, all supported on concrete piles. Begin and end bridge elevations (1957 As-Builts with current elevation shift of +0.29 ft) are 181.4 ft and 177.6 ft, respectively. Total length of the bridge is 126.5 ft.

Based on the information provided by the Office of Bridge Design (General Plan and Foundation Plan dated September 30, 2010), OGDS1 understands that the proposed project will consist of removing the existing two span bridge followed by the replacement with the new one span cast-in-place prestressed box girder bridge with seat type abutments on pile footing. The new bridge width will be 34.7 ft to 37.5 ft, and total length of bridge will be 139.2 ft (measured along "AT3" Line). Proposed begin and end bridge elevations are 182.9 ft and 178.9 ft respectively. The replacement is conducted to upgrade the existing lane widths to the current standards.

All elevations provided within this report and proposed project plans are based on current NAVD88 datum. As-Built Plan elevations require a +0.29 ft shift to adjust to the current datum.

## FIELD INVESTIGATION AND TESTING PROGRAM

The recent site-specific field exploration was performed between July 22 and September 30, 2009. The field investigation included drilling two rotary wash borings (4.5 inch diameter) and advancing two cone penetration test (CPT) soundings. Standard Penetration Tests (SPT's) and relatively undisturbed sampling (with a 2 inch inner diameter split-barrel sampler) were performed at the borings. Blow counts (SPT N values) were continuously recorded at 5 foot intervals during drilling. SPT's were performed in accordance with ASTM Test Method D1586-84 using a standard 1.4 inch I.D. sampler with a 140 lb. hammer dropped 30 inches. Undisturbed tube soil samples were also obtained using a 2 inch I.D. modified California Split Spoon Sampler with 4 inch long brass liners. The lined samples were all sealed in the field. Caltrans operated drill rig models CME-75 (Borings R-09-009) and Mobile B-80 (Boring R-09-011) were used at boring locations.

District 7 Surveys provided the location and elevation of the borings and CPT's. Boring/CPT information, including exploration number, stationing, offset, ground surface elevation, boring depth, and date drilled are summarized in Table 1.

**Table 1 - Summary of Borings/Cone Penetration Tests**

| Exploration No. | Station (ft) | Offset (ft) | Reference Line | Surface Elev. (ft) | Depth (ft) | Date Drilled |
|---|---|---|---|---|---|---|
| R-09-009 | 1155+38.6 | 200.9LT | Centerline SR-710 | 154.3 | 66.5 | 7/23/09 |

| | | | | | | |
|---|---|---|---|---|---|---|
| CPT-09-127 | 1155+43.6 | 200.9LT | Centerline SR-710 | 154.3 | 55.3 | 9/30/09 |
| R-09-011 | 1157+67.4 | 170.2LT | Centerline SR-710 | 152.4 | 100.8 | 7/22/09 |
| CPT-09-126 | 1157+72.4 | 170.2LT | Centerline SR-710 | 152.4 | 31.6 | 9/30/09 |

Note: OGDS1 located CPTs by tape measuring from mapped culture.

## LABORATORY TESTING PROGRAM

Selected soil samples were sent to the Department's Transportation Laboratory in Sacramento and District 7 Materials Laboratory in Los Angeles for testing. All laboratory tests were performed in accordance with ASTM standard procedures and California Test Methods. The summarized laboratory tests are shown in Table 2 below:

**Table 2 - Summary of Laboratory Tests**

| Test | Standard | No. of Test Performed |
|---|---|---|
| Mechanical Analysis | CTM 202, 203 | 10 |
| Atterberg | CTM 204 | 5 |
| Unit Weight | CTM 212 | 7 |
| Moisture | CTM 226 | 7 |
| Corrosion | CTM 417, 422, 532 | 4 |
| Direct Shear | ASTM D 3080 | 1 |

## SITE GEOLOGY AND SUBSURFACE CONDITIONS

### Regional Geology

The Rte. 710 Atlantic Blvd UC On-Ramp (Br. No. 53-3067) replacement is located at the boundary of the cities of Vernon and Bell. The subject bridge is located in the Peninsular Ranges Province in the Los Angeles Basin. The site is bounded 1) to the northwest and north by the Santa Monica Mountains and Santa Monica and Raymond Hill faults, 2) to the west, southwest, and south by the Newport-Inglewood Fault Zone, 3) to the northeast and east by the Whittier fault and Puente Hills fault, and 4) to the southeast by the Santa Ana Mountains and San Joaquin Hills. A thick Cenozoic sedimentary section underlies the site.

### Site Subsurface Conditions

Subsurface information obtained from the above borings indicates that the thickness of approach fill embankment at Abutment 1 (bridge south end) is approximately 30 feet, and at Abutment 2 (bridge north end) is approximately 27 feet. Based on the subsurface and Foundation Plan information, OGDS1 estimates the bottom of the fill to be at an elevation of about +151 feet. Alluvium underlying the fill consists mainly of loose to medium dense fine sand and silt down to approximate

elevations +140 to +132 feet. From approximate elevations +140 to +132 feet down to approximate elevations +125 to +115 feet alluvium consists of mainly very stiff, sandy clay and medium dense, clayey sand at boring R-10-009 (Abutment 1), and dense, silty sand and hard, sandy clay at boring R-09-011 (Abutment 2). Alluvium, from approximate elevations +125 to +115 ft down to the maximum 100.8 ft depth of exploration (elevation 51.6 feet), consists of stiff to very stiff sandy lean clay and medium dense to very dense silty sands with sporadic gravel and sandy silt. The lower alluvial unit within Boring R-09-011 (Abutment 2) consists mainly of dense to very dense, silty sand and sand with sporadic gravel.

**Groundwater**

No groundwater was encountered within the two rotary borings (based on evaluation of field samples) drilled/sampled July 2009, down to 66.5 and 100.8 feet depth (elevation +51.6 feet). Hollow stem auger boring (A-10-012), drilled August 2010 for the adjacent Atlantic Blvd. UC (Wdn), Br. No. 53-1009, was also dry to the bottom depth of 91.5 feet (elevation +61.1 feet).

Table 3 shows historical groundwater elevations in the vicinity of the bridge obtained from the monitoring well records of Los Angeles County Department of Public Works (LACDPW) and California Department of Water Resources (DWR). Four (4) wells located within distances ranging from about 0.27 to 0.73 miles from the site indicate groundwater elevations (measured during last 40 years) lower than elev. +65.8 feet, MSL (depths greater than 81.9 feet, adjusted elevation about +68 feet NAVD88).

**Table 3 - Summary of Historic Groundwater Records near Atlantic Blvd UC On-Ramp**

| LACDPW Well No. | Location | Year | Ground Surface Elevation (ft) | Historic High Groundwater Elevation (ft) | Year | Historic Low Groundwater Elevation (ft) | Year | Recent Ground Water Elevation (ft) |
|---|---|---|---|---|---|---|---|---|
| 2829C | 0.27 mile NW Atlantic Blvd UC On-Ramp | 1995 | +160.0 | +44.5 (115.5) | 1972 | -36.5 (196.5) | 2006 | +3.5 (156.5) |
| 1521 | 0.63 mile SW Atlantic Blvd UC On-Ramp | 1939 | +147.7 | +65.8 (81.9) | 1958 | -121.3 (269.0) | 2004 | +14.0 (133.7) |
| 2839B | 0.59 mile NE Atlantic Blvd UC On-Ramp | 1997 | +141.1 | +57.1 (89.0) | 1960 | -61.9 (203.0) | 2006 | +21.1 (125) |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| 1510B | 0.73 mile SW Atlantic Blvd UC On-Ramp | 1935 | +155.1 | +63.5 (91.6) | 1960 | -132.5 (287.6) | 1999 | +24.4 (130.7) |

Note: Values in parentheses indicate depth below the ground surface.

## CORROSION EVALUATION

Corrosion test results for representative soil samples are presented in the Table 4 below. The results show existing soils are non-corrosive to reinforced concrete and metal.

**Table 4 - Corrosion Test Summary**

| Exploration No. | Depth (ft) | PH | Minimum Resistivity (ohm – cm) | Sulfate Content (PPM) | Chloride Content (PPM) |
|---|---|---|---|---|---|
| R-09-009 | 5.0-16.5 | 8.34 | 4894 | N/A | N/A |
| R-09-009 | 45.0-61.5 | 8.28 | 1912 | N/A | N/A |
| R-09-011 | 11.5-30.0 | 7.79 | 1700 | N/A | N/A |
| R-09-011 | 41.5-65.0 | 8.21 | 3200 | N/A | N/A |
| **Corrosive Guidelines** | | **≤5.5** | <1000 | **≥2000** | **≥500** |

Note: It is the practice of Caltrans Corrosion Technology Section (with the exception of MSE Walls) if the minimum resistivity of the sample is greater than 1000 ohm-cm and the pH is greater than 5.5, the sample is considered to be noncorrosive. For structural elements, the California Department of Transportation considers a site to be corrosive if one or more of the following conditions exist for representative soil and/or water samples taken at the site:
Chloride concentration ≥500 ppm, sulfate concentration ≥2000 ppm, or the pH is ≤5.5. Corrosion mitigation is required if one or more of the 3 conditions noted above exists where structural elements are involved (Caltrans Corrosion Guidelines, September 2003).

## SEISMIC RECOMMENDATIONS

### Faulting and Seismicity

The controlling seismic source for this bridge is the Puente Hills Blind Thrust (PHBT) fault. Caltrans has assigned a Maximum Credible Earthquake (MCE) of moment magnitude 7.25 to this fault. This is a buried thrust fault with a site-to-rupture surface distance of about 3.3 miles. Based on the Sadigh et al (1997) attenuation relationships, the median Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.63g.

### Potential Seismic Hazards

This site is classified as being within a potentially liquefiable zone as shown on the California Geological Survey Map of Seismic Hazard Zones of the South Gate 7.5 Minute Quadrangle (March 25, 1999). However, based on Caltrans recent study and the as-built data the possibility of

liquefaction is expected to be low to nil. The site is considered non-liquefiable as design ground water is considered at least 95 to 100 ft or deeper below original ground. Consequently, the potential for seismically induced impacts such as settlement and lateral deformation of subsurface materials is remote.

**Design Ground Motion**

For the purpose of selecting a standard SDC (2006) Acceleration Response Spectrum (ARS) curve, the design PBA is taken as 0.6g. The soil profile may be classified as Type D. The design ARS curve was obtained by modeling the standard SDC curve (for PBA = 0.6g, M = 7.25 ± 0.25, and soil profile type D) for near fault effects. The recommended ARS Curve and the coordinates of spectral accelerations are presented in Figure 1 and Table 5 below, respectively.

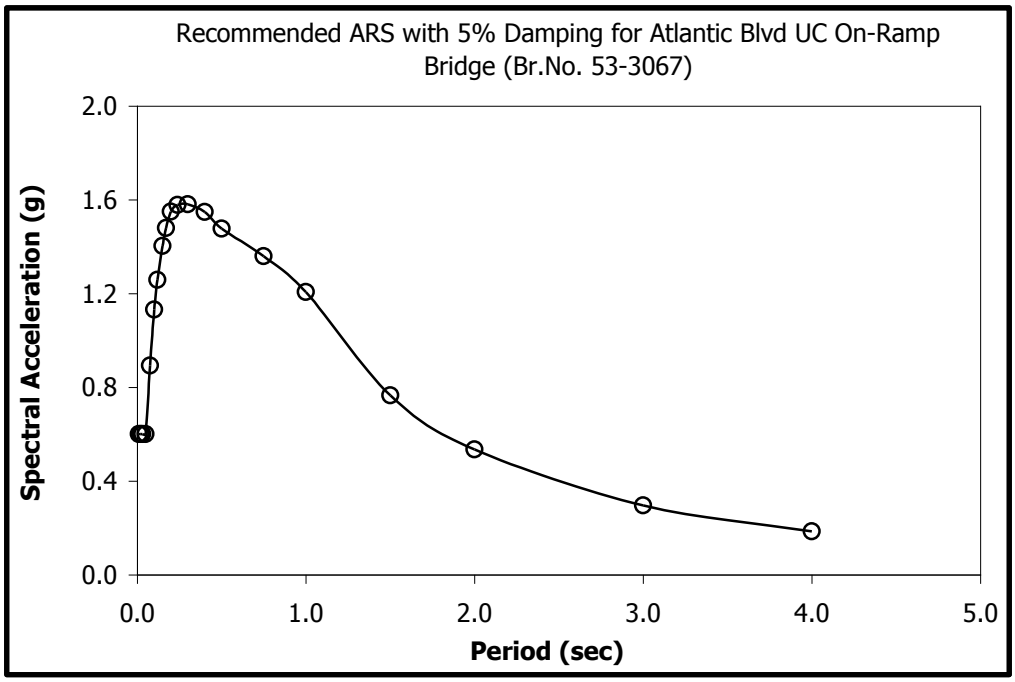



**Figure 1 - Recommended ARS curve for Atlantic Blvd UC On-Ramp Bridge**

**Table 5 - Recommended Design ARS curve for Br. No. 53-3067**

| Period (s) | Spectral Acceleration (g) | |
|---|---|---|
| | Standard | **Modified** |
| 0.010 | 0.6003 | 0.6003 |
| 0.020 | 0.6003 | 0.6003 |
| 0.030 | 0.6003 | 0.6003 |
| 0.050 | 0.6003 | 0.6003 |
| 0.075 | 0.8934 | 0.8934 |

| | | |
|---|---|---|
| 0.100 | 1.1315 | 1.1315 |
| 0.120 | 1.2590 | 1.2590 |
| 0.150 | 1.4033 | 1.4033 |
| 0.170 | 1.4802 | 1.4802 |
| 0.200 | 1.5496 | 1.5496 |
| 0.240 | 1.5784 | 1.5784 |
| 0.300 | 1.5811 | 1.5811 |
| 0.400 | 1.5484 | 1.5484 |
| 0.500 | 1.4767 | 1.4767 |
| 0.750 | 1.2366 | 1.3602 |
| 1.000 | 1.0062 | 1.2074 |
| 1.500 | 0.6378 | 0.7653 |
| 2.000 | 0.4464 | 0.5357 |
| 3.000 | 0.2471 | 0.2965 |
| 4.000 | 0.1546 | 0.1855 |

**Surface Fault Rupture Hazard**

The project site is not located within any California Geological Survey (CGS) designated Earthquake Fault Zone (EFZ) or directly underlain by any fault considered active for bridge design. Therefore, the possibility of surface fault rupture hazard at the site is considered low.

## AS-BUILT FOUNDATION DATA

Based on existing foundation records for the original bridge (1957 As-Builts) for Atlantic Blvd S/B On-Ramp, standard driven concrete piles (45 tons design load) were used for foundation support. Average pile tip elevation for the original bridge Abutments 1 and 3 is approximately +105.3 ft elevation. For Bent 2, bottom of footing elevation is approximately +147.3 feet with specified pile tip elevation of +105.3 feet. No average pile tip elevation is given for the original bridge Bent 2. However, OGDS1 assumes average pile tip elev. +105.3 feet. For Abutments 1 and 3 the bottom of footing elevation is approximately +147.05 feet. For reference, the top elevation of existing bridge embankment is +181.4 (bridge south end) and +177.6 feet (bridge north end), respectively.

## FOUNDATION RECOMENDATIONS

The following recommendations are based on 1) the recent geotechnical investigation with lab test results; 2) updated structure general and foundation plans, abutment layouts, details, and pile layouts, and retaining wall details (received September 3 through 30, 2010), bridge design loads, pile cut-off and finish grade elevations; and 3) suitable pile type (received September 8, 2010) proposed by the Office of Structure Design 2 (OBSD2) and considered feasible for existing site conditions by OGDS1.

**Deep Foundations**

The geotechnical capacities of the proposed piles were evaluated using the design loads provided by OBDS2. Computer program APILE Version 4.0 was used for calculating steel H-pile/soil resistances. OGDS1 recommends standard HP14x117 steel piles (battered and vertical as shown on plans) for Abutments 1 and 2 supports. Existing structures will be partially removed but piles at abutments and wall areas will be left in place for the proposed bridge replacement. Steel H-piles are the most suitable pile choice due to superior drivability, limited lateral extent (to avoid existing concrete piles), nondisplacive nature, ability to drive on batter, and relative flexibility when compared to other pile alternatives.

General Foundation Information and Design Loads for Atlantic Blvd. UC On-Ramp (Replace) presented in Tables 6 and 7, respectively, were provided by Office of Structure Design.

**Table 6 - Foundation Design Data Sheet (Br. No. 53-3067)**

| Support No. | Design Method | Pile Type/Diameter and Thickness (in) | Finished Grade Elev. | Cut-off Elev. (ft) | Pile Cap Size | | Permissible Settlement Under Service Load * (in) | Number of Piles per Support |
|---|---|---|---|---|---|---|---|---|
| | | | | | B (ft) | L (ft) | | |
| Abut 1 | WSD | HP14x117 | varies | 146.3 | 17 | 52 | 2.0 | 41 |
| Abut 2 | WSD | HP14x117 | varies | 146.3 | 17 | 58.75 | 2.0 | 50 |

* Based on CALTRANS' current practice, the total permissible settlement is one inch for multi-span structures with continuous spans or multi-column bents, one inch for single span structures with diaphragm abutments, and two inches for single span structures with seat abutments. Different permissible settlement under service loads may be allowed if a structural analysis verifies that required level of serviceability is met.

**Table 7 - Foundation Design Loads (Br. No. 53-3067)**

| Support No. | Service-1 Limit State (kips) | | | Strength Limit State (Controlling Group, kips) | | | | Extreme Event Limit State (Controlling Group, kips) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | Total Load | | Permanent Loads | Compression | | Tension | | Compression | | Tension | |
| | Per Support | Max Per Pile | Per Support | Per Support | Max Per Pile | Per Support | Max Per Pile | Per Support | Max Per Pile | Per Support | Max Per Pile |
| Abut 1 | 4612 | 200.0 | 4351 | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A |
| Abut 2 | 4485 | 200.0 | 4223 | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A |

The Abutment Foundation Design Recommendations Table 8, and the Pile Data Table 9, prepared by OGDS1 for Atlantic Blvd. UC On-Ramp (Replace) are presented below.

**Table 8 - Abutment Foundation Design Recommendations for Atlantic Blvd UC On-Ramp (Replace) Br. No. 53-3067**

| Support No. | Pile Type | Cut-off Elev. (ft) | LRFD Service-I Limit State Load (kips) per Support | | LRFD Service-I Limit State Total Load (kips) Max per Pile (Compression) | Nominal Resistance (kips) | Design Tip Elev. (ft) | Specified Tip Elev. (ft) | Nominal Driving Resistance Required (kips) |
|---|---|---|---|---|---|---|---|---|---|
| | | | Total Loads Per support | Permanent Loads Per support | | | | | |
| Abut. 1 | HP14x117 | 146.3 | 4612 | 4351 | 200.0 | 400 | 87.0 (a)<br>103.0 (c) | 87.0 | 400 |
| Abut. 2 | HP14x117 | 146.3 | 4485 | 4223 | 200.0 | 400 | 87.0 (a)<br>103.0 (c) | 87.0 | 400 |

*Notes:*

1) *Design tip elevations are controlled by: (a) Compression, (c) Settlement, and (d) Lateral Load, respectively.*
2) *The specified tip elevation shall not be raised above the design tip elevations for Settlement and Lateral Load.*
3) *The nominal driving resistance required is equal to the nominal resistance needed to support the factored load.*
4) *Design tip elevation for Lateral Load is typically provided by SD.*

**Table 9 - Pile Data Table for Atlantic Blvd UC On-Ramp (Replace) Br. No. 53-3067**

| Location | Pile Type | Nominal Resistance (kips) | | Design Tip Elevation (ft) | Specified Tip Elevation (ft) | Nominal Driving Resistance (kips) |
|---|---|---|---|---|---|---|
| | | Compression | Tension | | | |
| Abut. 1 | HP14x117 | 400 | N/A | 87.0 (a)<br>103.0 (c) | 87.0 | 400 |
| Abut. 2 | HP14x117 | 400 | N/A | 87.0 (a)<br>103.0 (c) | 87.0 | 400 |

*Notes:*

1) *Design tip elevations for Abutments are controlled by: (a) Compression, (c) Settlement, (d) Lateral Load*
2) *The specified tip elevation shall not be raised above the design tip elevations for Settlement and Lateral Load*
3) *Design tip elevation for Lateral Load is typically provided by SD*

Pile lengths were computed based on the provided design loads and the observed subsurface conditions. The analysis was conducted based on the assumption that proposed H-piles would derive geotechnical capacity from both skin friction (approximately 80%) and end bearing (approximately 20%).

**Abutment Retaining Walls**

General foundation information and layout plans (dated September 20 and 30, 2010) for Retaining Walls at Atlantic Blvd. UC On -Ramp Abutments 1 and 2 (Replace) were provided by the Office of Structure Design and are shown in Table 10. Standard steel H-Piles (HP12x84), battered and vertical, are recommended to support the Type 1 retaining walls.

**Table 10 - Foundation Design Data Sheet for Abutments 1 and 2 Retaining Walls**

| Abut No. / RW Height | Design Method | Pile Type | Finished Grade Elev. (ft) | Cut-off Elev. (ft) | Pile Cap Size | | Permissible Settlement Under Service Load * | Number of Piles per Support |
|---|---|---|---|---|---|---|---|---|
| | | | | | B (ft) | L (ft) | | |
| Abut 1 RW H=30’ | WSD | HP12x84 | varies | 150.3 | 16.75 | 14.5 | 2 | 17 |
| Abut 1 RW H=26’ | WSD | HP12x84 | varies | 154.8 | 14.25 | 15.5 | 2 | 13 |
| Abut 1 RW H=22’ | WSD | HP12x84 | varies | 159.4 | 12 | 9 | 2 | 6 |
| Abut 1 RW H=18’ | WSD | HP12x84 | varies | 164.5 | 9 | 9 | 2 | 5 |
| Abut 1 RW H=14’ | WSD | HP12x84 | varies | 168.5 | 7.75 | 9 | 2 | 4 |
| Abut 1 RW H=10’ | WSD | HP12x84 | varies | 173.7 | 6.25 | 9 | 2 | 4 |
| Abut 1 RW H=8’ | WSD | HP12x84 | varies | 175.7 | 5.25 | 8 | 2 | 4 |
| Abut 2 RW H=26’ | WSD | HP12x84 | varies | 152.6 | 18.5 | 14.25 | 2 | 18 |
| Abut 2 RW H=20’ | WSD | HP12x84 | varies | 159.3 | 11 | 13 | 2 | 7 |
| Abut 2 RW H=14’ | WSD | HP12x84 | varies | 165.3 | 12 | 9 | 2 | 4 |
| Abut 2 RW H=8’ | WSD | HP12x84 | varies | 169.5 | 9 | 9 | 2 | 4 |

- Permissible settlement at retaining wall segments is provided by OBDS2.

Axial capacities of the proposed steel H-Piles (HP12x84) at retaining walls were calculated per the request of OBDS2. The Pile Data Table for retaining wall foundations are provided below in Table 11.

**Table 11- Pile Data for Retaining Walls at Abutments 1 and 2**

| Abut No. / RW Height | Pile Type | Cut-off Elevation (ft) | Required Nominal Resistance (Kips) | Required Driving Resistance (Kips) | Specified Pile Tip Elevation (ft) |
|---|---|---|---|---|---|
| Abut 1 RW H=30’ | HP12x84 | 150.3 | 180 | 180 | 108.0 |
| Abut 1 RW H=26’ | HP12x84 | 154.8 | 180 | 180 | 108.0 |
| Abut 1 RW H=22’ | HP12x84 | 159.4 | 180 | 180 | 108.0 |
| Abut 1 RW H=18’ | HP12x84 | 164.5 | 180 | 180 | 108.0 |
| Abut 1 RW H=14’ | HP12x84 | 168.5 | 180 | 180 | 108.0 |
| Abut 1 RW H=10’ | HP12x84 | 173.7 | 180 | 180 | 108.0 |
| Abut 1 RW H=8’ | HP12x84 | 175.7 | 180 | 180 | 108.0 |
| Abut 2 RW H=26’ | HP12x84 | 152.6 | 180 | 180 | 110.0 |
| Abut 2 RW H=20’ | HP12x84 | 159.3 | 180 | 180 | 110.0 |
| Abut 2 RW H=14’ | HP12x84 | 165.3 | 180 | 180 | 110.0 |
| Abut 2 RW H=8’ | HP12x84 | 169.5 | 180 | 180 | 110.0 |

**Settlement at Approach Fills**

Fills can be placed in accordance with Section 19-6 of Caltrans Standard Specifications (May 2006). End dumping is not permitted. At Abutments 1 and 2 areas, additional approach fill is estimated to range from 5 to 25 ft height. Calculated post construction settlement will be negligible. OGDS1 recommends a fill settlement waiting period of up to 30 days. However, the actual settlement period will be determined by the structure representative on the basis of settlement data in the field and above waiting periods be adjusted accordingly.

**NOTE TO DESIGNER**

The recommendations contained in this report are based on specific project information that has been provided by the Office of Structure Design. If any conceptual changes are made during final project design, the Office of Geotechnical Design South 1 should review those changes to determine if these foundation recommendations are still applicable.

**CONSTRUCTION CONSIDERATIONS**

1. Variable moderate to hard driving should be anticipated below elevations ranging from +120 to +130 feet down to specified tip elevation for driven piles at Abutments 1 and 2, respectively. Subsurface materials, through which the piles will be driven, include medium dense to very dense, silty sands and sandy silt layers with sporadic gravel and stiff to hard, sandy and silty clay layers with thicknesses ranging from 5 to about 20 feet. However, with the written approval of the Structure Representative, any driven H piles achieving refusal (based on results of driveability analysis) within 4.0 feet or less above specified pile tip elevation, may be considered satisfactory.

2. Existing footings should be removed, location of existing piles (to be abandoned in place) identified in the field, and the top 3 feet of existing piles should be cut off. The Structure Representative should adjust proposed pile locations when necessary to avoid encountering existing abandoned piles. If a proposed pile needs to be relocated, consultation with OBDS2 and OGDS1 is warranted to insure adequate foundation design.

3. Reinforced Tip Protection should be used on H-piles to prevent damage to the pile in case of contact with existing abandoned concrete piles.

4. If minimum required bearing is not obtained at specified pile tip elevation, remaining piles that will be driven should be stopped (a few to several inches) above specified pile tip elevation. The piles should be driven the remaining distance after a minimum set-up period of 24 hours.

5. At times piles will not attain minimum bearing at specified pile tip elevation, even when re-driven. When this happens the only option is to splice on additional length and continue driving to a point where the nominal penetration is achieved.

6. The contractor should monitor adjacent structures for vibrations to prevent potential damage due to pile driving. The contractor should take necessary precautions to minimize the impact on adjacent structures.

7. Earthwork should be performed in accordance with Sections 6 and 19 of the latest Caltrans Standard Specifications. Soils with an Expansion Index of less than 50 and a Sand Equivalent of at least 20 should be used as replacement fill, in accordance with standard Caltrans requirements.

If you have any questions or need further information, please contact Tatjana Halda at (213) 620-2347 or Shiva Karimi at (213) 620-2146.

Prepared by: Date: 10/29/10

Tatjana Halda

REGISTERED PROFESSIONAL ENGINEER
TATJANA L. HALDA
No. C 68293
Exp. 9/30/11
CIVIL
STATE OF CALIFORNIA

Tatjana Halda, PE
Transportation Engineer
Office of Geotechnical Design South 1
Branch D

Supervised by: Date: 10/29/10

P Shiva Karimi

REGISTERED PROFESSIONAL ENGINEER
SHIVA KARIMI
No. GE2651
Exp. 6/30/12
GEOTECHNICAL
STATE OF CALIFORNIA

Shiva Karimi, Ph.D., P.E., G.E.
Senior Transportation Engineer
Office of Geotechnical Design South 1
Branch D

Reviewed by: Date: 10/29/10

Joseph S. Pratt

REGISTERED GEOLOGIST
Joseph S. Pratt
No. 2141
Exp. 5/31/11
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

Joe Pratt, C.E.G. No. 2141
Engineering Geologist
Office of Geotechnical Design South 1
Branch D

cc:

| | | |
|---|---|---|
| District Project Manager | James S. Hsu | James_s_Hsu@dot.ca.gov |
| GS Corporate | Mark Willian | Mark_Willian@dot.ca.gov |
| District Construction R.E. Pending File | | TBD |
| Structure Construction R.E. Pending File | | RE_Pending_File@dot.ca.gov |
| DES Office Engineer, Office of PS&E | Efren Ancheta | Efren_Ancheta@dot.ca.gov |
| District Materials Engineer | Kirsten Stahl | Kirsten_Stahl@dot.ca.gov |

**State of California** **Business, Transportation and Housing Agency**

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**To:** MS. CHUNG-YUAN WEN
Office of Bridge Design South 2
Bridge Design Branch 21
21073 Pathfinder Rd, Suite 20

Attention: MR. FRANK WEI

**Date:** November 22, 2010
**File**: 07-LA-710-PM 17.34
07-202111
Los Angeles River Bridge (Widen)
Bridge No. 53-0828

**From: DEPARTMENT OF TRANSPORTATION**
**DIVISION OF ENGINEERING SERVICES**
**Geotechnical Services**
**Office of Geotechnical Design – South 1**
**Branch D**

**Subject:** Foundation Report for Los Angeles River Bridge (Widen), Interstate 710

## INTRODUCTION

This report was prepared in response to the Office of Bridge Design South 2's (OBDS2's) memorandum, dated May 21, 2009, requesting Foundation Recommendations for the proposed outside widenings of Interstate 710 (I-710) at Los Angeles River Bridge (Bridge No. 53-0828).

## SCOPE OF WORK

The recommendations contained in this report are based on the subsurface information gathered during the field investigations performed in September 2009, and May and June 2010. Tasks completed by the Office of Geotechnical Design South 1 (OGDS1), Branch D include the following:

1. Review of the regional geology and seismicity,
2. Review of pertinent information from previous geology/geotechnical reports and As-Built plans in the project area to evaluate the subsurface information,
3. Drilling, logging, and sampling of six rotary wash borings at the subject bridge site to characterize the subsurface conditions,
4. Laboratory testing of selected soil samples for mechanical analysis, corrosivity, direct shear, consolidation, triaxial compression, unit weight, Atterberg Limits, and moisture content,
5. Geotechnical engineering analysis and preparation of this report.

## PROJECT DESCRIPTION

The Los Angeles River Bridge, located in the city of South Gate, is a four span, continuous steel girder bridge having a total length of 575.6 feet measured along Centerline Rte. 710. The subject bridge was originally built in 1951 (Piers and earlier Abutments) and 1955 (extended bridge and rebuilt Abutments) as two separate structures for carrying northbound (NB) and southbound (SB) traffic for I-710. The bridges were widened towards the median in 1965. The bridge was strengthened in 1993 and seismically retrofitted in 1997 with pile footing widening and the addition of driven Class 70, Alt "W" pipe piles at Piers 2 and 4. As-Built drawings indicate that the bridge deck is supported on open-end seat type abutments and solid pier walls.

The topography at the site consists of surrounding relatively flat areas separated by man-made embankment slopes and the excavated river channel. The grade at the site varies from the top of the approach fill embankments down to the surrounding relatively level ground and levees, then down to the bottom of the Los Angeles River channel. The site is bounded in the northeast and the southwest by approach fills for the freeway embankment. The river bed extends from north to south through the site. The embankment top elevations range from +121 to +129 feet at Abutments 1 and 5, respectively. The levees have 2.25 (H): l (V) side slopes that descend to the bottom of the channel with approximate elevations ranging from +81 to +83 feet. The channel side slopes are covered with a grouted rip-rap blanket and the channel bottom is concrete lined.

Based on information provided by OBDS2, OGDS1 understands that the proposed project will consist of bridge outside widenings for the right side (NB) by 13.30 to 18.76 feet and the left side (SB) by 10.14 to 10.66 feet. The widening is proposed to upgrade the existing lane widths to the current standards and to provide 10-foot wide shoulders.

All elevations provided within this report and current project plans are based on NAVD88 datum. As-Built Plan elevations require a +1.40 foot shift for Abutment 1 through Pier 3 area and a +1.20 foot shift for Pier 4 through Abutment 5 area to adjust to the current NAVD 88 datum.

## FIELD INVESTIGATION AND TESTING PROGRAM

Site-specific field exploration was conducted in September 2009 and May/June 2010 for the proposed widening. The field exploration included drilling six 4.5 inch outer diameter (O.D.) rotary wash borings. Caltrans operated drill rig models Mobile B-80 (Borings R-09-001 and R-09-002), CME-85 (Borings R-10-003, R-10-004, and R-10-005), and CME-1000 (Boring R-10-006) were used for the drilling/sampling. Samples were obtained within borings using Standard Penetration Test sampler (SPT) and a 2 inch inner diameter (I.D.) split-barrel sampler (modified California Sampler), typically at 5-foot depth intervals. SPT's were performed in accordance with ASTM Test Method D1586 using a standard 1.4 inch I.D. split spoon sampler with a 140 lb hammer dropped 30 inches. Relatively undisturbed tube soil samples were obtained using a 2 inch I.D. modified California Split Spoon Sampler with 4 inch long brass liners. The modified California Sampler was predominantly pushed into the soil by hydraulic pressure, but sometimes rarely driven if needed to obtain a soil sample. The lined undisturbed samples were sealed in the field to preserve the in situ

moisture content. Two piezometers were installed within Borings R-10-003 and R-10-006. Slotted PVC pipe was installed within the lower portion of the borings starting at elevations -59.5 feet (R-10-003) and -32.3 feet (R-10-006) and extended to the surface with solid PVC pipe (then capped and sealed).

Following the completion of drilling, District 7 Surveys conducted a precise survey to establish the locations and elevations of Borings R-10-003, R-10-004, R-10-005, and R-10-006. Precise survey for Borings R-09-001 and R-09-002 wasn't conducted due to construction paving over these two locations. OGDS1 located Borings R-09-001 and R-09-002 by tape measuring from existing facilities/objects shown on Layout Plans and plotting boring locations on these plans. After Borings R-09-001 and R-09-002 were located on the layout plans (with 2 foot topographic contours), top of hole elevations were estimated to within 0.1 foot (elevation estimate accurate to approximately ±0.25 feet elevation). Boring information, including exploration number, stationing, offset, ground surface elevation, boring depth, and date drilled are summarized in Table 1.

**Table 1 - Summary of Borings**

| Boring No. | Centerline Rte 710 Stationing (feet) | Offset (feet) | Surface Elev. (feet) | Boring Depth (feet) | Date Completed |
|---|---|---|---|---|---|
| R-09-001 | 913+50 | 53.0Rt | +116.0 | 120.7 | 09-19-09 |
| R-09-002 | 914+80 | 51.0Lt | +122.4 | 121.5 | 09-19-09 |
| R-10-003 | 917+00 | 66.8Rt | +82.0 | 141.5 | 05-26-10 |
| R-10-004 | 919+70 | 72.5Lt | +82.5 | 141.5 | 06-15-10 |
| R-10-005 | 920+29 | 99.4Rt | +107.8 | 140.9 | 06-08-10 |
| R-10-006 | 922+38 | 101.0Lt | +108.1 | 140.4 | 06-23-10 |

## LABORATORY TESTING PROGRAM

Selected soil samples were sent to the Department's Transportation Laboratory in Sacramento and District 7 Materials Laboratory in Los Angeles for testing. All laboratory tests were performed in accordance with current ASTM standard procedures and California Test Methods. The summarized laboratory test data are shown in Table 2.

**Table 2 - Summary of Laboratory Tests**

| Test | Standard | No. of Tests Performed |
|---|---|---|
| Mechanical Analysis | CTM 202, 203 | 46 |
| Atterberg | CTM 204 | 28 |
| Unit Weight | CTM 212 | 6 |
| Moisture | CTM 226 | 30 |
| Corrosion | CTM 417, 422, 532 | 26 |
| Direct Shear | ASTM D 3080 | 2 |
| Consolidation | ASTM D2435 | 2 |
| Triaxial (U.U.) | ASTM D2850 | 1 |

## SITE GEOLOGY AND SUBSURFACE CONDITIONS

### Regional Geology

The Rte. 710 Los Angeles River Bridge is located in the Peninsular Ranges Province in the Los Angeles Basin. The site is bounded 1) to the northwest and north by the Santa Monica Mountains and Santa Monica and Raymond Hill faults, 2) to the west, southwest, and south by the Newport-Inglewood Fault Zone, 3) to the northeast and east by the Whittier fault and Puente Hills fault, and 4) to the southeast by the Santa Ana Mountains and San Joaquin Hills. A thick Cenozoic sedimentary section underlies the site.

### Site Subsurface Conditions

Abutment 1 (South Abutment)
Borings R-10-001 and R-10-002, located near Abutment 1, indicate that embankment fill/levee materials underlie the freeway structural section. The elevations of the freeway grade at the boring locations are 116 feet and 122 feet. The fill-native material (alluvium) contact was observed in these borings and adjusted As-Built borings to be from approximate elevations +95 to + 93 feet. The fill mainly consists of dense to very dense, silty sand and stiff, sandy silt with scattered gravel and cobble and concrete fragments (4 to 6 inch length). Scattered trace amounts of glass fragments and wood fragments were also discovered within fill material. The upper alluvial unit underlying the fill consists predominantly of interlayered medium dense/stiff, silty sand, silt/sandy silt and lean clay/sandy clay down to an elevation of about +25 feet. This interlayered unit is underlain by the lower alluvial unit consisting of medium dense to very dense, silty sand/poorly graded sand and stiff to very stiff, silt/clay extending down to an elevation of about -4.7 feet (maximum boring depths from 120.7 to 121.5 feet).

Abutment 5 (North Abutment)
Borings R-10-005 and R-10-006, located just below Abutment 5 (at the top of the eastern levee) indicate that embankment fill/levee material consists of medium dense to dense, silty sand. Based on the recent field exploration and observations from the borings, the As-Built Plans, and the topography of the surrounding area, OGDS1 considers the fill-alluvium contact to be at approximate elevations +95 to +93 feet. The upper alluvial unit underlying embankment fill consists mainly of interlayered medium dense to dense, silty sand/poorly graded sand and medium stiff to stiff, silt/sandy silt and lean clay/sandy clay down to an elevation of about -20 feet. From elevation -20 feet down to elevation -33.1 feet (maximum boring depth 140.9 feet), the lower alluvial unit consists of stiff, sandy silt/sandy clay and very dense, silty sand with sporadic gravel.

Piers 2, 3 and 4
Borings R-10-003 and R-10-004, located at the bottom of the river bed, indicate that subsurface soil underlying the concrete-lined channel consists primarily of alluvium. The upper alluvial unit, from the channel bed elevation of about +82 feet down to approximate elevation -25 feet, mainly consists of medium dense to dense, silty sand/poorly graded sand and sporadic gravel interbedded with medium stiff to stiff, silt/sandy silt and lean clay with sand. From approximate elevation -25 feet down to elevation -59.5 feet (maximum boring depth 141.5 feet), the lower alluvial unit consists of

dense to very dense, silty sand/poorly graded sand with gravel, and sporadic gravel lenses interbedded with stiff to hard silt, sandy silt and sandy clay.

**Groundwater**

The elevation for design groundwater was established based on water level measurements within piezometers installed in Boreholes R-10-003 and R-10-006. In addition, the approximate depth to groundwater was estimated during drilling by observing the saturation of retrieved samples. The results of recent groundwater measurements at the above piezometers are presented in Table 3.

**Table 3 – Groundwater Measurement Data**

| Boring No. | Piezometer | Top of Boring Elevation (feet) | Depth to Static Water Level (feet) | Static Water Level Elevation (feet) | Date Measured |
|---|---|---|---|---|---|
| R-10-003 | 140' long PVC pipe with a 50' long slotted section at bottom | +82.0 | 37.4 | +44.6 | 9/17/2010 |
| | | | 37.5 | +44.5 | 10/01/2010 |
| | | | 37.6 | +44.4 | 10/29/2010 |
| R-10-006 | 140' long PVC pipe with a 50' long slotted section at bottom | +108.1 | 44.3 | +63.8 | 9/17/2010 |
| | | | 44.6 | +63.5 | 10/01/2010 |
| | | | 44.6 | +63.5 | 10/29/2010 |

The 1955 As-Built LOTB for the original bridge indicates that groundwater at the site was encountered at elevation +77.5 feet, MSL (measured February 1954, approximate elevation +78.9 feet NAVD88 datum).

Historical groundwater elevations in the vicinity of the bridge were obtained from the monitoring well records of Los Angeles County Department of Public Works (LACDPW) and California Department of Water Resources (DWR). Five (5) wells located within distances ranging from about 0.5 to 0.7 miles from the site indicate high groundwater elevations during the last 60 years mainly ranged from +30.1 to +56.5 feet MSL (NGVD29 datum, approximately elevations ranging from +32 to +58.5 feet NAVD88 datum).

OGDS1 considers recent groundwater levels measured within piezometers at the site are appropriate for the current widening. Based on this, design groundwater levels of +45 feet at the piers and +64 feet at the abutments were used in the geotechnical analysis.

## SCOUR EVALUATION

The Los Angeles River channel bottom is lined with concrete and side slopes are lined with cement grouted riprap. Therefore, scour is not considered to be an issue at the subject project site.

## CORROSION EVALUATION

Corrosion test results for representative soil samples are presented in the Table 4 below. The results show subsurface soil at the site is considered non-corrosive to reinforced concrete and metal. Lab test results on two river water samples taken from ±50 feet upstream and downstream of the bridge showed minimum resistivity values that were lower than 1000 ohm-cm (920 and 960 ohm-cm) indicating a potentially corrosive nature. However, these surface water samples were delayed in testing. Therefore, OGDS1 collected additional river water samples and delivered promptly for testing on October 29, 2010. Based on the additional test results on river water samples, OGDS1 considers surface stream water from the Los Angeles River channel at the bridge site to be non-corrosive to reinforced concrete and metal.

**Table 4 - Corrosion Test Summary**

| Exploration No. | Depth (ft) | pH | Minimum Resistivity (ohm – cm) | Sulfate Content (PPM) | Chloride Content (PPM) |
|---|---|---|---|---|---|
| **R-09-001** | 5.0-21.5 | 7.74 | 4203 | N/A | N/A |
| | 25.0-46.5 | 8.37 | 2848 | N/A | N/A |
| | 60.0-66.5 | 7.91 | 1024 | N/A | N/A |
| | 75.0-86.5 | 7.92 | 1404 | N/A | N/A |
| | 90.0-106.5 | 7.97 | 1642 | N/A | N/A |
| | 110.0-121.5 | 7.89 | 2644 | N/A | N/A |
| **R-09-002** | 5.0-21.5 | 8.41 | 764 | 938 | 52 |
| | 25.0-46.5 | 8.36 | 878 | 368 | 300 |
| | 55.0-76.5 | 8.31 | 1009 | N/A | N/A |
| | 80.0-96.5 | 8.45 | 2310 | N/A | N/A |
| | 100.0-121.5 | 8.04 | 2295 | N/A | N/A |
| **R-09-003** | 16.5-20.0/ 36.5-40.0 | 8.75 | 4100 | N/A | N/A |
| | 56.5-60.0/ 81.5/85.0 | 8.21 | 2900 | N/A | N/A |
| | 90.0-141.5 | 7.62 | 2400 | N/A | N/A |
| **R-10-004** | 10.0 | 8.12 | 2100 | N/A | N/A |
| | 65.0 | 8.18 | 1300 | N/A | N/A |
| | 100.0 | 6.75 | 2500 | N/A | N/A |
| | 130.0 | 8.25 | 4000 | N/A | N/A |
| **R-09-005** | 25.0 | 8.23 | 1800 | N/A | N/A |
| **R-09-006** | 25.0 | 8.54 | 1020 | N/A | N/A |
| | 40.0 | 7.33 | 1900 | N/A | N/A |
| | 65.0 | 8.45 | 1400 | N/A | N/A |
| | 90.0 | 7.98 | 1100 | N/A | N/A |
| | 120.0 | 7.78 | 1600 | N/A | N/A |
| **Upstream Surface Water** (delayed test) | N/A | 7.58 | 920 | N/A | N/A |
| **Downstream Surface Water** | N/A | 7.67 | 960 | N/A | N/A |

| | | | | | |
|---|---|---|---|---|---|
| (delayed test) | | | | | |
| **Upstream Surface Water** (resampled) | N/A | 8.23 | 1000 | N/A | N/A |
| **Downstream Surface Water** (resampled) | N/A | 8.23 | 1050 | N/A | N/A |
| **Corrosive Guidelines** | | **≤5.5** | <1000 | **≥2000** | **≥500** |

Notes: It is the practice of Caltrans Corrosion Technology Section (with the exception of MSE Walls) if the minimum resistivity of the sample is greater than 1000 ohm-cm and the pH is greater than 5.5, the sample is considered to be noncorrosive. For structural elements, Caltrans considers a site to be corrosive if one or more of the following conditions exist for representative soil and/or water samples taken at the site:
Chloride concentration ≥500 ppm, sulfate concentration ≥2000 ppm, or the pH is ≤5.5. Corrosion mitigation is required if one or more of the 3 conditions noted above exists where structural elements are involved (Caltrans Corrosion Guidelines, September 2003).

## SEISMIC RECOMMENDATIONS

### Faulting and Seismicity

The controlling seismic sources for this bridge are the Puente Hills Blind Thrust (PHBT) and the Upper Elysian Park Blind Thrust (UEPBT) faults. Caltrans considers these faults as active for bridge design and has assigned a Maximum Credible Earthquake (MCE) of moment magnitude M=7.25 and M=7.0 for the PHBT and the UEPBT faults, respectively. The median Peak Bedrock Acceleration (PBA) and the median Peak Ground Acceleration (PGA) at the site were estimated for each of these faults using the Sadigh at al (1997) attenuation relationships for rock and deep soil sites, respectively. Based on these calculations, the PHBT is the nearest and controlling fault for this site. This is a buried thrust fault with a site-to-fault rupture surface distance ($r_{rup}$) of about 3.6 miles. The median or the design PBA and PGA at the site should be taken as 0.6g and 0.54g, respectively, with an MCE having a moment magnitude M=7.25.

### Potential Seismic Hazards

This site is classified as being within a potentially liquefiable zone as shown on the California Geological Survey Map of Seismic Hazard Zones of the South Gate 7.5 Minute Quadrangle (March 25, 1999). OGDS1 evaluated the potential for liquefaction in the upper alluvial soils located between approximate elevations of +45.0 and +30.0 feet at the piers, and between elevations of +64.0 and +30.0 feet at the abutments. OGDS1's analysis indicates that the liquefaction potential of most of the upper alluvial soils is low due to the predominantly fine grained nature of these soils. However, some of the minor medium dense, silty sand and sand layers interbedded with the fine grained soils may be potentially liquefiable. The liquefaction potential below elevation +30.0 is considered unlikely due to the dense to very dense, silty sand/sand interbedded with stiff to very stiff, silt/clay.

Potential hazards associated with liquefaction at the site include seismically induced settlement and lateral spreading of site soils. Although few minor foundation soil interbeds could be susceptible to

liquefaction, due to the isolated nature and the limited thickness of the potentially liquefiable layers, OGDS1 evaluates the impact of potential liquefaction of such layers on the proposed bridge widening is low and does not warrant any mitigation.

**Design Ground Motion**

The design PBA of 0.6g together with a soil profile of Type D was used for the site for the purpose of selecting a standard Acceleration Response Spectrum (ARS) based on 2006 Caltrans Seismic Design Criteria (SDC). The design ARS curve was obtained by modifying the standard SDC curve for a PBA of 0.6g and a magnitude M of 7.25 ± 0.25, for near fault effects. The recommended ARS Curve and coordinates of spectral accelerations are presented in Figure 1 and Table 5 below, respectively.

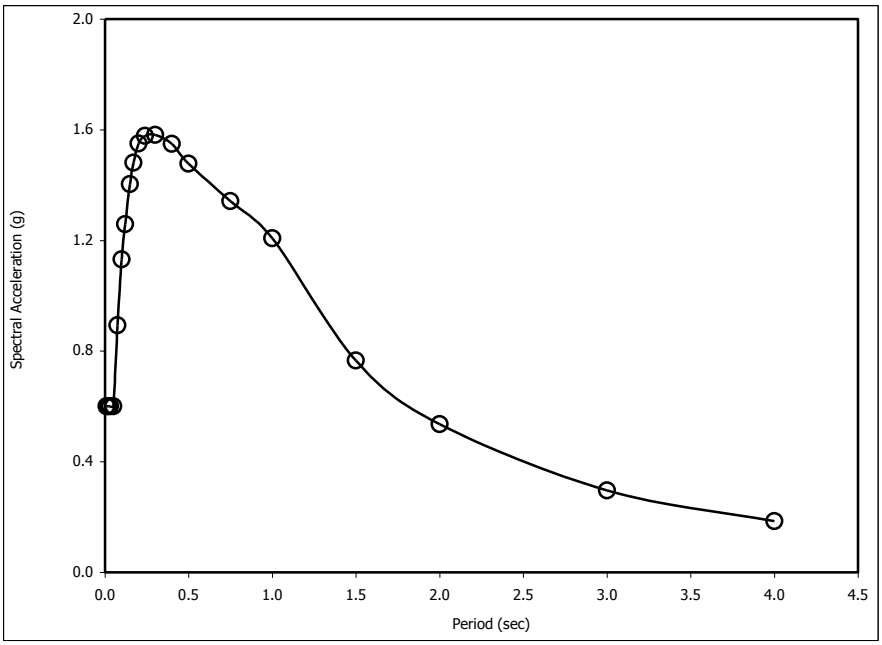


**Figure 1 Recommended ARS curve for Los Angeles River Bridge (Widen)**

**Table 5 - Recommended Design ARS curve data for Los Angeles Bridge (Widen)**

| Period (seconds) | Spectral Acceleration (g) | |
|---|---|---|
| | Standard | **Modified** |
| 0.010 | 0.6003 | 0.6003 |
| 0.020 | 0.6003 | 0.6003 |
| 0.030 | 0.6003 | 0.6003 |
| 0.050 | 0.6003 | 0.6003 |
| 0.075 | 0.8934 | 0.8934 |
| 0.100 | 1.1315 | 1.1315 |
| 0.120 | 1.2590 | 1.2590 |
| 0.150 | 1.4033 | 1.4033 |
| 0.170 | 1.4802 | 1.4802 |

| | | |
|---|---|---|
| 0.200 | 1.5496 | 1.5496 |
| 0.240 | 1.5784 | 1.5784 |
| 0.300 | 1.5811 | 1.5811 |
| 0.400 | 1.5484 | 1.5484 |
| 0.500 | 1.4767 | 1.4767 |
| 0.750 | 1.2366 | 1.3602 |
| 1.000 | 1.0062 | 1.2074 |
| 1.500 | 0.6378 | 0.7653 |
| 2.000 | 0.4464 | 0.5357 |
| 3.000 | 0.2471 | 0.2965 |
| 4.000 | 0.1546 | 0.1855 |

**Surface Fault Rupture Hazard**

The project site is not located within any California Geological Survey (CGS) designated Earthquake Fault Zone (EFZ) or directly underlain by any known fault considered active for bridge design. Therefore, the subject bridge is not considered prone to surface fault rupture hazard.

**AS-BUILT FOUNDATION DATA**

Table 6 presents existing foundation records for Los Angeles River Bridge based on available As-Built data. Solid Pier Walls 2, 3, and 4 were built in 1951 and supported on octagonal-shaped (16 inch) precast concrete piles (driven to 32 tons design load) along with abutments. In 1955 the bridges were later lengthened and cast-in-place tapered concrete piles (Alternative "Z") were used to support the newly extended Abutments. The Alt. “Z” tapered pile shell diameter is 8 inches at tip and 15.5 inches at the butt. No record was found to determine if any additional piles were used to support the 1965 median widening, although it appears that existing pier walls and abutments were used. For the 1997 bridge earthquake retrofit, Class 70, Alternative “W” piles (open ended pipe piles, PP14 X 0.438) were used and foundations widened at Piers 2 and 4 to provide additional support in compression and tension.

**Table 6 – As Built Pile Data for Los Angeles River Bridge, Br. No. 53-0828**

| Support Location | 1955 As Built (Abutments) & 1951 (Piers) | | | As Built (1997) EQ Retrofit | |
|---|---|---|---|---|---|
| | Pile Type | Design Load (kips) | Average Pile Tip Elevation (feet) | Pile Type | Bottom of Footing / (Pile Tip) Elevations. (feet) |
| Abutment 1 | Cast in place concrete pile* Alt. “Z” | 90 | 45.3 (Min: 41.3, Max: 52.1) | *** | *** |
| Pier 2 | Precast concrete pile | 64 | N/A** | Open ended pipe pile - Class 70 Alt. “W” | + 75.5/ ( +20.0) |
| Pier 3 | Precast concrete pile | 64 | N/A** | *** | *** |

| | | | | | |
|---|---|---|---|---|---|
| Pier 4 | Precast concrete pile | 64 | N/A** | Open ended pipe pile - Class 70 Alt. "W" | + 76.0/ ( +20.0) |
| Abutment 5 | Cast in place concrete pile* Alt. "Z" | 90 | 52.5 (Min: 44.6, Max: 62.8) | *** | *** |

Notes: 1. * Shell diameter is minimum 15.5 in. at butt and 8 in. at tip (As-Built 1955).
2. N/A** - Pile tip elevations for existing piles at Piers 2, 3, and 4 were not available in 1951 As-built plans.
3. *** As-built plans indicate that new piles were added to Piers 2 and 4 only.

## FOUNDATION RECOMENDATIONS

The following recommendations for pile foundations were developed by OGDS1 based on the interpreted subsurface conditions, geotechnical design soil parameters established through laboratory and field data, updated structure plans (dated October 10, 2010), bridge design loads (updated September 20, 2010), and the pile types proposed by OBDS2.

The pile type proposed by OBDS2 for both abutments and piers, consists of Class 140, Alternative "W" pipe piles (PP14x0.438). Based on the observed subsurface conditions, OGDS1 concurs with the feasibility of these piles for supporting the proposed outside widenings.

General Foundation Information and Design Loads provided by OBDS2 are presented in Tables 7 and 8, respectively. Pile lengths required to resist the provided loads were computed based on the observed subsurface conditions and design strength parameters using Computer Program APILE (Version 4.0). Working Stress Design and LRFD methods were used to compute the pile lengths to obtain required design pile/soil resistances at abutments and piers, respectively. The analysis indicated that driven piles would derive geotechnical capacities from both skin friction and end bearing with skin friction accounting for about 90% (Abutments) to 80 % (Piers) geotechnical resistance and end bearing accounting for about 10% (Abutments) to 20% (Piers) geotechnical resistance. In order to account for liquefaction/strength loss of minor potentially liquefiable soil layers located below design ground water, reduced soil strength parameters were used in evaluation of geotechnical capacities of piles under Extreme Event Loads.

The Abutment and Pier Foundation Design Recommendations are prepared by OGDS1 and presented in Tables 9 and 10, respectively. The Pile Data Table for both abutments and piers is presented in Table 11.

**Table 7-Foundation Design Data Sheet**

| Support No. | Design Method | Pile Type | Finished Grade Elevation (feet) | Cut-off Elevation (feet)** | Pile Cap Size | | Permissible Settlement Under Service Load (inches)* | Number of Piles per Support |
|---|---|---|---|---|---|---|---|---|
| | | | | | B (feet) | L (feet) | | |
| Abutment 1 | WSD | Driven Class 140 | 108.5 (North) 107.8 (South) | 104.9 (North) 104.9 (South) | Varies | Varies | 1 | 16 (North) 18 (South) |
| Pier 2 | LRFD | Driven Class 140 | 82.2 (North) 82 (South) | 76.4 (North) 76.4 (South) | 17 (North) 17 (South) | 18 (North) 25 (South) | 1 | 25 (North) 35 (South) |
| Pier 3 | LRFD | Driven Class | 80.1 (North) 79.8 (South) | 74.9 (North) 74.9 (South) | 11 (North) 11 (South) | 21.5 (North) 28.5 (South) | 1 | 18 (North) 24 (South) |
| Pier 4 | LRFD | Driven Class 140 | 82.1 (North) 82.4 (South) | 76.9 (North) 76.9 (South) | 17 (North) 17 (South) | 18 (North) 25 (South) | 1 | 25 (North) 35 (South) |
| Abutment 5 | WSD | Driven Class 140 | 108 (North) 107.6 (South) | 104.9 (North) 104.9 (South) | Varies | Varies | 1 | 12 (North) 25 (South) |

Notes:

* Permissible settlements are based on the limits stated in MTD 4-1. The current Caltrans practice indicates that the total permissible settlement is one inch for multi-span structures with continuous spans or multi-column bents, one inch for single span structures with diaphragm abutments, and two inches for single span structures with seat abutments. Different permissible settlement under service loads may be allowed if a structural analysis verifies that required level of serviceability is met.

** Cut-off elevation is 5 inches above bottom of footing (BOF) elevation.

**Table 8 - Foundation Design Loads**

| Service-1 Limit State (kips) | | | | Strength Limit State (Kips) | | | | Extreme Event Limit State (Kips) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Total Load | | | Permanent Loads | Compression | | Tension | | Compression | | Tension | |
| | Per Support | Max Per Pile | Per Support | Per Support | Max Per Pile | Per Support | Max Per Pile | Per Support | Max Per Pile | Per Support | Max Per Pile |
| Abut 1 | 1060 (North) 1190 (South) | 115 (North) 115 (South) | 760 (North) 855 (South) | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A |
| Pier 2 | 1250 (North) 1730 (South) | 70 (North) 70 (South) | 905 (North) 1255 (South) | 1640 (North) 2280 (South) | 85 (North) 85 (South) | N/A | 0 | 1465 (North) 1945 (South) | 120 (North) 120 (South) | N/A | -60 (North) -60 (South) |

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Pier 3 | 1370 (North) 1750 (South) | 110 (North) 110 (South) | 955 (North) 1220 (South) | 1745 (North) 2270 (South) | 135 (North) 135 (South) | N/A | 0 | 1370 (North) 1750 (South) | N/A (North) N/A (South) | N/A | N/A (North) N/A (South) |
| Pier 4 | 1250 (North) 1730 (South) | 70 (North) 70 (South) | 905 (North) 1255 (South) | 1640 (North) 2280 (South) | 85 (North) 85 (South) | N/A | 0 | 1465 (North) 1945 (South) | 120 (North) 120 (South) | N/A | -60 (North) -60 (South) |
| Abut 5 | 995 (North) 2065 (South) | 130 (North) 130 (South) | 780 (North) 1640 (South) | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A |

Note: N/A - Not applicable

**Table 9 - Abutment Foundation Design Recommendations for LA River Bridge (Widen), Br. No. 53- 0828**

| Support Location | Pile Type | Cut-off Elevation (feet) | LRFD Service-I Limit State Load (kips) per Support | | LRFD Service-I Limit State Total Load (kips) per Pile (Compression) | Nominal Resistance (kips) | Design Tip Elevations (feet) | Specified Tip Elevation (feet) | Nominal Driving Resistance Required (kips) |
|---|---|---|---|---|---|---|---|---|---|
| | | | Total | Permanent | | | | | |
| Abut. 1 | Class 140 Alt. "W" | 104.9 (North) 104.9 (South) | 1060 (North) 1190 (South) | 760 (North) 855 (South) | 115 (North) 115 (South) | 230 (North) 230 (South) | 44.0 (a) 64.0 (c) | 44.0 | 230 |
| Abut. 5 | Class 140 Alt. "W" | 104.9 (North) 104.9 (South) | 995 (North) 2065 (South) | 780 (North) 1640 (South) | 130 (North) 130 (South) | 260 (North) 260 (South) | 40.0 (a) 60.0 (c) | 40.0 | 260 |

*Notes:*

1) *Design tip elevations are controlled by: (a) Compression, (c) Settlement.*
2) *The specified tip elevation shall not be raised above the design tip elevations for Settlement and Lateral Load.*
3) *The nominal driving resistance required is equal to the nominal resistance needed to support the factored load.*

**Table 10 – Pier Foundation Design Recommendations for LA River Bridge (Widen), Br. No. 53-0828**

| Support Location | Pile Type | Cut-off Elev. (feet) | Service-I Limit State Load per Support (kips) | Total Permissible Support Settlement (inches) | Required Factored Nominal Resistance (kips) | | | | Design Tip Elev. (feet) | Specified Tip Elevation (feet) | Nominal Driving Resistance Required (kips) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Strength Limit | | Extreme Event | | | | |
| | | | | | Comp. (φ=0.7 | Tension (φ=0.7) | Comp. (φ=1) | Tension (φ=1) | | | |
| Pier 2 | Class 140 Alt. "W" | 76.4 (North) 76.4 (South) | 1250 (North) 1730 (South) | 1 | 85/0.7= 122 (North) 85/0.7= 122 (South) | 0 | 120/1= 120 (North) 120/1= 120 (South) | -60/1= -60 (North) -60/1= -60 (South) | 38.0 (a-I) 30.0 (a-II) 50.0 (b-II) 56.0 (c) 46.0 (d) | 30.0 | 250 |

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Pier 3 | Class 140 Alt. "W" | 74.9 (North) 74.9 (South) | 1370 (North) 1750 (South) | 1 | 135/0.7= 193 (North) 135/0.7= 193 (South) | 0 | N/A (North) N/A (South) | N/A (North) N/A (South) | 30.0 (a-I) 25.0 (a-II) 40.0 (c) 44.0 (d) | 25.0 | 280 |
| Pier 4 | Class 140 Alt. "W" | 76.9 (North) 76.9 (South) | 1250 (North) 1730 (South) | 1 | 85/0.7= 122 (North) 85/0.7= 122 (South) | 0 | 120/1= 120 (North) 120/1= 120 (South) | -60/1= -60 (North) -60/1= -60 (South) | 38.0 (a-I) 30.0 (a-II) 50.0 (b-II) 56.0 (c) 46.0 (d) | 30.0 | 250 |

*Notes:*

1) *Design tip elevations are controlled by: (a-I) Compression (Strength Limit), (a-II) Compression (Extreme Event), (b-II) Tension (Extreme Event), (c) Settlement, (d) Lateral Load, respectively.*
2) *The specified tip elevation shall not be raised above the design tip elevations for Tension, Settlement, and Lateral Load.*
3) *The nominal driving resistance required is equal to the nominal resistance needed to support the factored load plus driving resistance from the unsuitable penetrated soil layers (potentially very soft/liquefiable material) Unsuitable soil layers at Piers (reduced design resistance) extend from approximate elevation +45.0 down to elevation of +30.0 ft.*
4) *Design tip elevation provided for Extreme Lateral Load (40 kips).*

**Table 11 - Pile Data Table for LA River Bridge**

| Location | Pile Type | Nominal Resistance (kips) | | Design Tip Elevation (feet) | Specified Tip Elevation (feet) | Nominal Driving Resistance (kips) |
|---|---|---|---|---|---|---|
| | | Compression | Tension | | | |
| Abut. 1 | Class 140 Alt "W" PP14x0.438 | 230 (North) 230 (South) | 0 | 44.0 (a) 64.0 (c) | 44.0 | 230 |
| Pier 2 | Class 140 Alt "W" PP14x0.438 | 130 (North) 130 (South) | -60 (North) -60 (South) | 30.0 (a) 50.0 (b) 56.0 (c) 46.0 (d) | 30.0 | 250 |
| Pier 3 | Class 140 Alt "W" PP14x0.438 | 200 (North) 200 (South) | N/A (North) N/A (South) | 25.0 (a) 40.0 (c) 44.0 (d) | 25.0 | 280 |
| Pier 4 | Class 140 Alt "W" PP14x0.438 | 130 (North) 130 (South) | -60 (North) -60 (South) | 30.0 (a) 50.0 (b) 56.0 (c) 46.0 (d) | 30.0 | 250 |
| Abut. 5 | Class 140 Alt "W" PP14x0.438 | 260 (North) 260 (South) | 0 | 40.0 (a) 60.0 (c) | 40.0 | 260 |

*Notes:*

1) *Design tip elevations for Abutments are controlled by: (a) Compression, (c) Settlement.*

2) *Design tip elevations for Bents are controlled by: (a) Compression, (b) Tension, (c) Settlement, (d) Lateral Load.*
3) *The specified tip elevation shall not be raised above the design tip elevations for Tension, Settlement, and Lateral Load.*
4) *Design tip elevation provided for Extreme Lateral Load (40 kips).*
5) *Unsuitable soil layers (potentially very soft/ liquefiable material) exist at Piers 2, 3, and 4, extending from elevations +45 down to +30.0 feet.*

## Abutment Retaining Walls

General foundation information for retaining walls at Abutments 1 and 5 provided by OBDS2 (updated October 1, 2010) are shown in Table 12. OBDS2 indicates that Caltrans Standard Type 1 Retaining Walls will be used for all four abutment retaining walls. Retaining Walls at Abutment 1 will be supported on spread footings. Retaining Walls at Abutment 5 will be supported on Class 90 Alternative "W" steel pipe piles (PP14x0.375). Refer to Retaining Wall Structure Plans for various wall details.

**Table 12 - Foundation Design Data Sheet**

| Retaining Wall Location/ Height (ft) | Design Method | Foundation Type / Pile Type | Finished Grade Elevation | Cut-off /Bottom of Footing* Elevation (ft) | Pile Cap Size | | Permissible Settlement Under Service Load | Number of Piles per Support |
|---|---|---|---|---|---|---|---|---|
| | | | | | B (ft) | L (ft) | | |
| **Abut 1 North RW**/ H=14 | WSD | Spread Footing | varies | +109.10* | 8 | 8 | - | N/A |
| **Abut 1 North RW**/ H=10 | WSD | Spread Footing | varies | +113.25* | 6.25 | 8 | - | N/A |
| **Abut 1 North RW**/ H=6 | WSD | Spread Footing | varies | +117.25* | 4.25 | 12 | - | N/A |
| **Abut 1 South RW**/H=12 | WSD | Spread Footing | 108 +/- | +105.67* | 7.25 | 24 | - | N/A |
| **Abut 5 North RW**/H=24 | WSD | Class 90 Alt "W" PP14x0.375 | 108 +/- | +105.25 | 13.25 | 13.5 | - | 11 |
| **Abut 5 North RW**/H=20 | WSD | Class 90 Alt "W" PP14x0.375 | 108 +/- | +105.59 | 11.0 | 15.35 (segment length/ shape varies)** | - | 8 |
| **Abut 5 South RW**/H=20 | WSD | Class 90 Alt "W" PP14x0.375 | varies | +109.25 | 11 | 16 | - | 9 |
| **Abut 5 South RW**/H=16 | WSD | Class 90 Alt "W" PP14x0.375 | varies | +113.75 | 9 | 8 | - | 3 |
| **Abut 5 South RW**/H=12 | WSD | Class 90 Alt "W" PP14x0.375 | varies | 117.75 | 7.25 | 8 | - | 2 |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| **Abut 5 South RW**/H=8 | WSD | Class 90 Alt "W" PP14x0.375 | varies | +121.75 | 5.25 | 16 | - | 3 |

Notes: * Bottom of Spread Footing Elevation
** Refer to Structure Plans provided by OBDS-2 (Retaining Wall Details No. 3 sheet).
N/A - Not applicable.

Recommended allowable soil bearing pressures for Abutment 1 (north and south) retaining wall spread footings are presented in Table 13. Pile Data Table for retaining walls foundations at Abutment 5 are provided in Table 14.

**Table 13- Spread Footing Data Table for Abutment 1 RetainingWalls**

| Wall Location/Height (ft) | Wall Design Height (feet) | Spread Footing Base width (feet) | Bottom of Footing Elevation (ft) | Gross Allowable Soil Bearing Pressure (ksf) |
|---|---|---|---|---|
| **Abut 1 North**/ H=14 | 14 | 8.0 | 109.10 | 3.4 |
| **Abut 1 North**/ H=10 | 10 | 6.25 | 113.25 | 2.5 |
| **Abut 1 North/** H=6 | 6 | 4.25 | 117.25 | 1.9 |
| **Abut 1 South/** H=12 | 12 | 7.25 | 105.67 | 2.9 |

**Table 14 - Pile Data Table for Abutment 5 Retaining Walls**

| **Wall Location / Design Height (feet)** | **Pile Type/** | **Cut-off Elevation (feet)** | **Nominal Resistance (Kips)** | **Required Driving Resistance (Kips)** | **Specified Pile Tip Elevation (feet)** |
|---|---|---|---|---|---|
| **Abut 5 North RW**/ H=24 | Class 90 Alt "W" (PP14x0.375) | +105.25 | 180 | 180 | 55.0 |
| **Abut 5 North RW**/ H=20 | Class 90 Alt "W" (PP14x0.375) | +105.59 | 180 | 180 | 55.0 |
| **Abut 5 South RW/** H=20 | Class 90 Alt "W" (PP14x0.375) | +109.25 | 180 | 180 | 58.0 |
| **Abut 5 South RW/** H=16 | Class 90 Alt "W" (PP14x0.375) | +113.75 | 180 | 180 | 58.0 |
| **Abut 5 South RW/** H=12 | Class 90 Alt "W" (PP14x0.375) | +117.75 | 180 | 180 | 58.0 |
| **Abut 5 South RW/** H=8 | Class 90 Alt "W" (PP14x0.375) | +121.75 | 180 | 180 | 58.0 |

For design details and notes pertaining to Caltrans Standard Type 1 retaining walls, supported on spread footings and on 90 kip pile footings, refer to Standard Plans May 2006, sheets B3-1 and B3-8.

**Settlement**

At Abutments 1 and 5 areas, additional approach fill is estimated to range from 10 to 24 feet height and settlement magnitude should be negligible. OGDS1 recommends settlement waiting periods of up to 30 days for the sliver fill widenings. However, the actual settlement period will be determined by the structure representative on the basis of settlement data in the field and above waiting periods be adjusted accordingly.

**CONSTRUCTION CONSIDERATIONS**

1. At Abutments 1 and 5 (including Abutment 5 North and South Retaining Walls), predrilling for pile installation through embankment and possible channel slope rip-rap material is required down to elevations +86 and +93 feet, respectively. Refer to Section 49-1.06 (Predrilled Holes) from Caltrans Standard Specifications (2006) for accepted procedures.

2. Sporadic hard pile driving may be anticipated from:
   a) approximate elevations +75 down to +50 feet at Abutment 1;
   b) approximate elevations +65 down to +40 feet at Abutment 5;
   c) approximate elevations +55 down to +40 feet at Piers 2 and 3, and +30 down to +25 feet at Pier 3; and
   d) approximate elevations +60 down to +40 feet at Pier 4.

   Subsurface materials, through which the piles have to be driven, include from medium dense to very dense, silty sand and sandy silt layers, and from medium stiff to very stiff, sandy and silty clay layers with thicknesses ranging up to about 10 feet.

   If refusal occurs, driving of piles can be terminated within 4 feet of specified pile tip elevation in concurrence with OGDS1.

3. If center relief drilling is necessary due to possible hard driving conditions, center relief drilling should be abandoned at least 15 feet above specified pile tip elevation as a plug was assumed for pile/soil resistance in end bearing.

4. If minimum required driving resistance is not obtained at specified pile tip elevation, pile driving should be stopped at about 6 inches above specified pile tip. Piles should be driven the remaining distance to specified tip elevation after a minimum set-up period of 24 hours. After the set-up period, blow counts for driving the first 3 inches should be recorded and pile capacity evaluated. If the required driving resistance is not obtained at specified pile tip elevation, OGDS1 should be contacted for any required action.

5. The contractor should monitor adjacent structures for vibrations to avoid potential damage due to pile driving. As necessary, the contractor should take precautions to minimize the impact on adjacent structures. Predrilling before pile installation at Abutments 1 and 5 and Abutment 5 North and South Retaining Walls will help protect the channel lining and nearby structures/facilities from excessive vibration.

6. Earthwork should be performed in accordance with Sections 6 and 19 of the latest Caltrans Standard Specifications. Soils with an Expansion Index of less than 50 and a Sand Equivalent of at least 20 should be used as replacement fill, in accordance with standard Caltrans requirements.

The recommendations contained in this report are based on specific project information that has been provided by the OBDS2. If any loading or conceptual changes are made during final project design, the OGDS1 should review those changes to determine if these foundation recommendations are still applicable.

If you have any questions or need further information, please contact Tatjana Halda at (213) 620-2347 or Shiva Karimi at (213) 620-2146.

Prepared by: Date: 11/22/10

Tatjana Halda

REGISTERED PROFESSIONAL ENGINEER
TATJANA L. HALDA
No. C 68293
Exp. 9/30/11
CIVIL
STATE OF CALIFORNIA

Tatjana Halda, PE
Transportation Engineer
Office of Geotechnical Design – South 1
Branch D

Supervised by: Date: 11/22/10

P Shiva Karimi

REGISTERED PROFESSIONAL ENGINEER
SHIVA KARIMI
No. GE2651
Exp. 6/30/12
GEOTECHNICAL
STATE OF CALIFORNIA

Shiva Karimi, Chief, Ph.D., G.E.
Senior Transportation Engineer
Office of Geotechnical Design – South 1
Branch D

Reviewed by: Date: 11/22/10

Joseph S. Pratt

REGISTERED GEOLOGIST
Joseph S. Pratt
No. 2141
Exp. 5/31/11
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

Joe Pratt, C.E.G. No. 2141
Engineering Geologist
Office of Geotechnical Design South 1
Branch D

Reviewed by: Date: 11/22/10

REGISTERED PROFESSIONAL ENGINEER
GAMINI C. WEERATUNGA
No. 2403
Exp. 9/30/12
GEOTECHNICAL
STATE OF CALIFORNIA

Gamini Weeratunga, G.E.
Transportation Engineer
Office of Geotechnical Design – South 1
Branch D

cc:

| | | |
|---|---|---|
| District Project Manager | James S. Hsu | James_s_Hsu@dot.ca.gov |
| GS Corporate | Mark Willian | Mark_Willian@dot.ca.gov |
| District Construction R.E. Pending File | | TBD |
| Structure Construction R.E. Pending File | | RE_Pending_File@dot.ca.gov |
| DES Office Engineer, Office of PS&E | Efren Ancheta | Efren_Ancheta@dot.ca.gov |
| District Materials Engineer | Kirsten Stahl | Kirsten_Stahl@dot.ca.gov |

**State of California** **Business, Transportation and Housing Agency**

# M e m o r a n d u m

***Flex your power!***
***Be energy efficient!***

**To:** MS. CHUNG-YUAN WEN
Senior Bridge Engineer
Office of Bridge Design South 2

**Date:** August 23, 2010

**File:** 07-LA-710- PM 21.80
07-202111
Retaining Wall No. 1150
Structure No. 53-E0160
at Cheli Depot OH
(Bridge No. 53-0839)

**From:** **DEPARTMENT OF TRANSPORTATION**
**DIVISION OF ENGINEERING SERVICES**
**Geotechnical Services**
**Office of Geotechnical Design – South 1**
**Branch D**

**Subject:** Foundation Report for Retaining Wall No. 1150 (Structure No. 53-E0160) at Cheli Depot Overhead (Bridge No. 53-0839)

## INTRODUCTION

In a memorandum dated May 21, 2009, Office of Bridge Design South 2 (OBDS2) requested a Foundation Report for the proposed Retaining Wall No. 1150 as part of the Route 710 Long Life Pavement Improvement and Bridge Widening project.

Retaining Wall No. 1150 will be constructed at Cheli Depot Overhead along southbound Route 710 (I-710), near the boundary between the cities of Vernon and Bell, Los Angeles County, California. Retaining Wall No. 1150 has three segments consisting of Caltrans Standard Type 1 retaining walls at both beginning and end wall segments and a special design retaining wall on top of a footing slab that lies above the existing Cheli Depot Overhead bridge deck slab. The purpose of the Office of Geotechnical Design South 1's (OGDS1's) geotechnical investigation is to evaluate site soil conditions and to provide recommendations for foundation design of the proposed wall segments.

## SCOPE OF WORK

The scope of work for the current study included performing the following tasks:

a. Review of the pertinent literature, plans, and As-Built plans;
b. Field reconnaissance by an engineer to observe the existing conditions at the site of the proposed wall;
c. Project coordination with Bridge Design South 2, Underground Service Alert, Caltrans Maintenance, and Caltrans Drilling Services;
d. Field investigation and laboratory testing;
e. Interpretation of subsurface soil and groundwater conditions at the site of the proposed wall; and
f. Engineering analyses and preparation of this report to present geotechnical recommendations for foundation design of the proposed wall.

## PROJECT DESCRIPTION

This project is part of planned improvements for the Route 710 Long Life Pavement Improvement and Bridge Widening project in the County of Los Angeles. The proposed retaining wall will be constructed at Cheli Depot Overhead along southbound Route 710. According to the General Plan (plotted June 24, 2010), Retaining Wall No. 1150 begins 79.4 ft Lt. Station 1150+45 Centerline Rte. 710 (50+45 Wall LOL) and ends 82.6 ft Lt. Station 1150+64 Centerline Rte. 710 (51+68.9 Wall LOL). Beginning and ending segments of Retaining Wall No. 1150 consist of Type 1 wall segments and will be constructed on the slope of the southbound Route 710 roadway embankment. The proposed middle segment will be a special design retaining wall on top of a footing slab that lies above the existing Cheli Depot Overhead bridge deck slab and is supported on 24-inch diameter cast-in-drilled-hole (CIDH) piles outside the existing bridge abutments. Total length of the wall is 123.9 feet with design heights ranging from 6.4 feet to 10 feet. The locations and the geometric layout data for the wall segments are presented in Table 1. All elevations provided within this report and proposed project plans are based on NAVD88 datum.

**Table 1 - Locations and Geometric Layout of Wall No. 1150**

| Approx. Rte. 710 Centerline Station[1] | | Wall Segment No. | Wall Type Design | Retaining Wall Design Height (ft) | Foundation Width (ft) | Bottom of Footing Elev. (ft) | Footing on Slope |
|---|---|---|---|---|---|---|---|
| From | To | | | | | | |
| 1150+45 (50+45.0) | 1150+83 (50+83.0) | A | Type 1 | 10 | 6¼ | 178.91 | Yes |
| 1150+83 (50+83.0) | 1151+31 (51+33.3) | B | Special Design | 6'-5" | 9 | 181.21± | NA |
| 1151+31 (51+33.3) | 1151+64 (51+68.9) | C | Type 1 | 8 | 5¼ | 180.03 | Yes |

[1] Values in parenthesis are the approximate stationing on retaining wall layout line (RW LOL).

## FIELD INVESTIGATION

The site-specific field exploration was performed between October 28 and 31, 2009. The field investigation included drilling two 8 inch outer diameter hollow-stem auger borings. Standard Penetration Tests (SPT's) and relatively undisturbed sampling (with a modified California split-barrel sampler, 2 inch inner diameter) were performed within the borings. Blow counts (SPT N-values) were recorded at 5 foot intervals during drilling. The SPT's were performed in accordance with ASTM Test Method D1586 using a standard 1.4 inch I.D. sampler with a 140 lb hammer dropped 30 inches. Relatively undisturbed soil tube samples were obtained using a 2 inch I.D. modified California sampler with 4 inch long brass liners. The liners were all capped/sealed in the field. Caltrans operated drill rig model CME 85 was used to drill the hollow-stem auger borings. Microstation Log of Test Boring (LOTB) files for the recent field investigation and scanned copies of the As-Built LOTB sheets will be sent to the designer for inclusion within Contract Plans.

District 07 Surveys provided the location and elevation of the borings. Boring information are summarized in Table 2.

**Table 2 – Summary of Boring Information**

| Boring No. | Centerline Rte. 710 Station (ft) | Offset (ft) | Surface Elev. (ft) | Depth (ft) | Date Drilled |
|---|---|---|---|---|---|
| A-09-138 | 1151+58.7 | 64.0 Lt | 188.4 | 101.5 | 10-27/28-09 |
| A-09-139 | 1150+54.8 | 59.8 Lt | 187.6 | 81.5 | 10-29/30-09 |

## LABORATORY TESTING PROGRAM

Caltrans Materials and Geotechnical Laboratories performed laboratory testing on selected samples obtained from the field investigation. The purpose of the laboratory testing was to help evaluate the engineering properties of the subsurface materials and to confirm visual classification of the soils. Laboratory tests performed include moisture content, dry unit weight, wash sieve analysis, Atterberg limits, unconfined compression tests, direct shear, and corrosion tests. Laboratory test results are retained in electronic format per the Geotechnical Service (GS) project archive requirements.

## SITE GEOLOGY AND SUBSURFACE CONDITIONS

### Regional Geology

The Retaining Wall 1150 project site is located in the city of Commerce. The subject site is located in the Peninsular Ranges Province in the Los Angeles Basin. The site is bounded 1) to the northwest and north by the Santa Monica Mountains and Santa Monica and Raymond Hill faults, 2) to the west, southwest, and south by the Newport-Inglewood Fault Zone, 3) to the northeast and east by the Whittier fault and Puente Hills fault, and 4) to the southeast by the Santa Ana Mountains and San Joaquin Hills. A thick Cenozoic sedimentary section underlies the site.

### Subsurface Conditions

At the Retaining Wall No. 1150 and Cheli Depot Overhead site, embankment fill ranges between approximately 33 and 38 ft thick. Top and bottom of embankment fill ranges from approximate elevations +188 to +150 ft, respectively. Embankment fill is underlain by undifferentiated young and older Quaternary alluvial deposits. Fill is composed of

medium dense silty sand, sandy silt, and sand with silt interlayered with very stiff to hard silt and silty clay. Fill also contains sporadic trace gravel and asphalt fragments. The underlying undifferentiated young and older Quaternary alluvial deposits are composed of hard to stiff lean clay and sandy clay interlayered with medium dense to very dense sandy silt, silty sand, sand with silt, and sand. Alluvium also contains sporadic cobbles and trace gravel.

### Groundwater

No groundwater was encountered within hollow-stem auger borings A-09-138 and A-09-139 which extended down to a maximum depth of 101.5 ft below the Route 710 Freeway grade (dry down to elevation +86.9 ft) during Caltrans' October 2009 field investigation. The As-Built LOTB (1955) for the original bridge indicates that no groundwater was encountered at the site during the subsurface exploration which extended to a depth of 64 ft (dry down to elevation +89.5 ft).

Based on the recent field investigation and historic groundwater review, it is concluded that the groundwater will be below elevation +86.9 ft for the subject structure.

## CORROSION EVALUATION

Selected soil samples were tested by Caltrans Materials Laboratory in order to assess corrosivity parameters including pH, resistivity, sulfate and chloride content. The results are summarized in Table 3.

**Table 3 - Corrosion Test Results Summary**

| Exploration No. | SIC Number | pH | Minimum Resistivity (ohm – cm) | Sulfate Content (PPM) | Chloride Content (PPM) |
|---|---|---|---|---|---|
| A-09-138 & A-09-139 Combined Sample | NA | 8.06 | 960 | 79 | 562 |

Note: Caltrans currently considers a site to be corrosive to foundation elements if one or more of the following conditions exist. Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm, or the pH is 5.5 or less.

Based on the results of the corrosion analysis, the site is considered corrosive to concrete for foundation elements (Caltrans Corrosion Guidelines, 2003). Design of reinforced concrete in contact with existing soils should consider the site corrosion potential in accordance with Bridge Design Specifications (BDS) Section 8.22 (September 2003) and determine the type of cement and minimum cover requirements.

## SEISMIC RECOMMENDATIONS

### Faulting and Seismicity

The Puente Hills Blind Thrust (PHBT) is the controlling seismic source for the subject wall. The closest site-to-fault rupture distance for the site is about 3.3 mile (5.3 km). The PHBT is capable of generating a Maximum Credible Earthquake (MCE) of moment magnitude 7.3. Based on the Sadigh et al (1997) attenuation relationships, the estimated median or design Peak Bedrock Acceleration (PBA) at bridge site is about 0.6g (rounded to the nearest tenth of g). Assuming a soil site, the estimated median or design Peak Ground Acceleration (PGA) is about 0.56g. The design ARS curves for the subject bridge should be developed based on M=7.25 and PBA=0.6g, and incorporated near-fault effects as specified in the Caltrans Seismic Design Criteria (SDC), 2006. Based on the subsurface information from the recent field investigation and As-Built Logs of Test Borings (LOTB), the soil profile may be classified as Type D for the design. The design ARS curve was obtained by modeling the standard SDC curve (for PBA = 0.6g, M = 7.25 ± 0.25, and soil profile type D) for near fault effects. The recommended ARS Curve and the coordinates of spectral accelerations are presented in Figure 1 and Table 4 below, respectively.

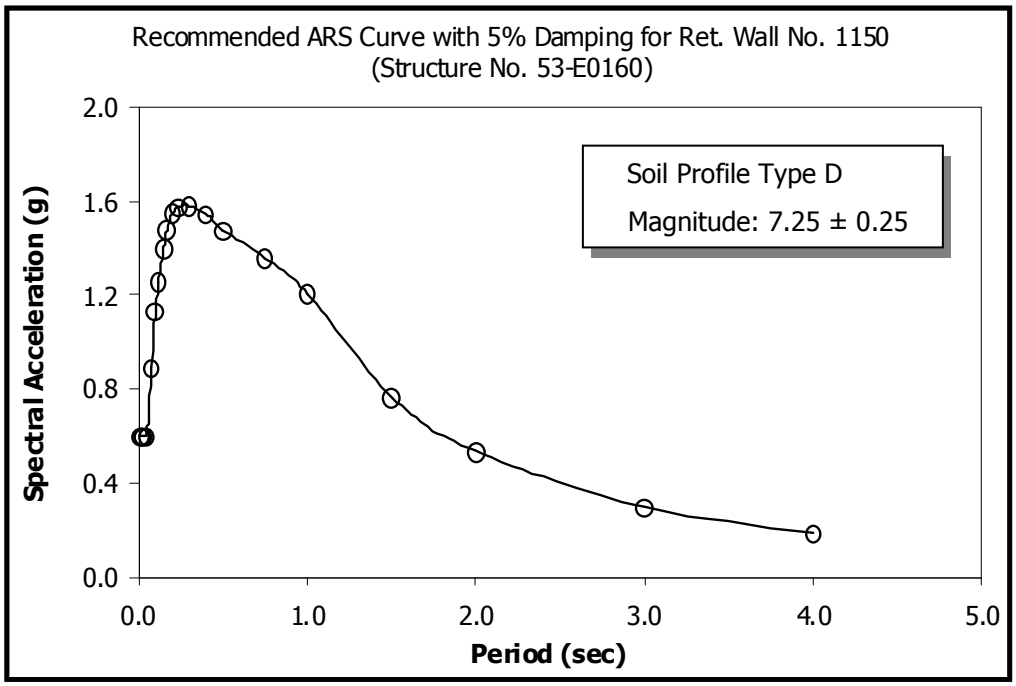


**Figure 1 Recommended ARS curve for Ret. Wall No. 1150**

**Table 4 - Recommended Design ARS curve for Ret. Wall No. 1150**

| Period (s) | Spectral Acceleration (g) | |
|---|---|---|
| | Standard | Modified |
| **0.010** | 0.6003 | **0.6003** |
| **0.020** | 0.6003 | **0.6003** |
| **0.030** | 0.6003 | **0.6003** |
| **0.050** | 0.6003 | **0.6003** |
| **0.075** | 0.8934 | **0.8934** |
| **0.100** | 1.1315 | **1.1315** |
| **0.120** | 1.2590 | **1.2590** |
| **0.150** | 1.4033 | **1.4033** |
| **0.170** | 1.4802 | **1.4802** |
| **0.200** | 1.5496 | **1.5496** |
| **0.240** | 1.5784 | **1.5784** |
| **0.300** | 1.5811 | **1.5811** |
| **0.400** | 1.5484 | **1.5484** |
| **0.500** | 1.4767 | **1.4767** |
| **0.750** | 1.2366 | **1.3602** |
| **1.000** | 1.0062 | **1.2074** |
| **1.500** | 0.6378 | **0.7653** |
| **2.000** | 0.4464 | **0.5357** |
| **3.000** | 0.2471 | **0.2965** |
| **4.000** | 0.1546 | **0.1855** |

**Surface Fault Rupture Hazard Evaluation**

The project site is not located within any California Geological Survey (CGS) designated Earthquake Fault Zone (EFZ). The subject site is not considered prone to surface fault rupture hazard; therefore, the possibility of surface fault rupture hazard at the site is considered very low.

**Liquefaction**

The 7.5-minute Seismic Hazard Zone Map of the South Gate Quadrangle (Davis, 1999) indicates that the Cheli Depot OH is underlain by potentially liquefiable soils. However, no groundwater was encountered in our recent investigation for the retaining wall (drilled to a maximum depth of 101.5 feet) or during the 1955 Caltrans investigation for the bridge (drilled to a maximum depth of 64 feet). Since there is no indication of the presence of groundwater within the subsurface zone to a depth of greater than 50 feet below surrounding grade and considering the dense to very dense/stiff to hard nature of the subsurface soil, the liquefaction potential is considered to be low.

## FOUNDATION RECOMMENDATIONS

### Type 1 Retaining Wall

**Bearing Capacity**

The following recommendations are based on Retaining Wall No. 1150 General Plan, Foundation Plan, Structure Plan and Details (revised June 16, 2010) and Typical Sections (revised May 12, 2010) provided by the Office of Bridge Design South 2.

Based on the Typical Section, the Type 1 wall segments at this location are designed for Loading Case I (level backfill + 240 psf surcharge). The retaining wall segments A and C will be approximately 38 ft and 35.5 ft in length and 10 ft and 8 ft in design height, respectively. Retaining wall segments A and C can be supported by spread footing foundation with minor remedial grading below the footing. Additional details such as spread footing width, bottom of footing elevation, and contact pressures are shown partially in 2006 Standard Plans Sheet (B3-1). The required minimum footing cover depth of 2 ft should be provided. For footings constructed on slopes, a minimum horizontal

distance of 4 ft measured at the top of the footing should be provided between the near face of the footing and the face of the finished slope. The required minimum horizontal bench at the base of the wall is 5 ft measured horizontally from the face of the wall to the hinge point of the slope.

The ultimate bearing capacity was estimated using Terzaghi's equation for general shear failure. Ultimate bearing capacities for maximum and minimum wall heights and spread footing widths were calculated to verify gross allowable soil bearing pressure presented in Table 5 below.

**Table 5 - Retaining Wall No. 1150 Spread Footing Data Table**

| Wall No. 1150 Segment | [1]Wall LOL Stationing | | Wall Design Height (ft) | Spread Footing Base Width (ft) | Approximate Bottom of Footing Elevation (ft) | Bottom of Sub-excavation Elevation (ft) | Gross Allowable Soil Bearing Pressure ($q_{all}$) ASD [2] (ksf) |
|---|---|---|---|---|---|---|---|
| | From | To | | | | | |
| A | 50+45 (1150+45) | 50+83 (1150+83) | 10 | 6¼ | 178.91 | 176.91 | 2.5 |
| C | 51+33 (1151+31) | 51+69 (1151+64) | 8 | 5¼ | 180.03 | 178.03 | 2.2 |

[1] Values in parenthesis are the approximate stationing on RTE. 710 centerline.
[2] Allowable Stress Design, (ASD). The Maximum Contact Pressure, ($q_{max}$), is not to exceed the recommended Gross Allowable Soil Bearing Pressure, ($q_{all}$). The Ultimate Soil Bearing Capacity, ($q_{ult}$), will exceed or equal 3 times the recommended Gross Allowable Soil Bearing Pressure, ($q_{all}$).

In order to provide a uniform and competent bearing material and limit differential settlement, 2 ft of remedial grading is recommended beneath the wall footing. Remedial treatment consists of overexcavating existing soils within the specified limits to 2 ft below footing grade and replacing these soils with structure backfill compacted to 95% R.C. (relative compaction) up to footing grade. The horizontal limits of the structure backfill prism should extend from a line one foot beyond the retaining wall's toe and heel and then down and out at a 1.5:1 (H:V) slope to a depth equal to the bottom of the subexcavation. Refer to the Caltrans Standard Specifications (2006), section 19-5.03 for details.

**Settlement**

The specified limits of the structure backfill prism should reduce overall settlement and differential settlement beneath the wall by replacing poor quality variable soils with more homogenous compacted structure backfill at the site. Differential settlement of the foundations will be acceptable and within tolerance (1V:500H for CIP concrete retaining walls) after the required subexcavation and replacement of a portion of the existing material beneath the wall spread footing footprint.

**Special Design Wall**

The proposed middle segment of the wall is a special design retaining wall on top of a slab that lies above the existing Cheli Depot Overhead bridge deck. The slab carrying the retaining wall is supported on four 24 inch diameter cast-in-drilled-hole (CIDH) piles. The deep foundation data for the special design retaining wall segment were provided by the OBDS2 and summarized in Tables 6 and 7.

**Table 6 – Retaining Wall No. 1150 Deep Foundation Data**

| Wall Segment/ Wall LOL Stationing | Design Method | Pile Dia. and Type | Top of Wall Elevation (ft) | Cut-off Elevation (ft) | RW Footing Size (ft) | | Permissible Movement under Service Load (in) | | Number of Piles to Support the Wall Slab |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | B | L | Vert. (in) | Hori. (in) | |
| B (50+83.0 to 51+33.3) | WSD | 24" CIDH | 189.18 | 181.46 | 9.0 | 50.3 | 1 | 0.5 | 4 |

**Table 7 - LRFD Service Limit State 1 Load Data**

| Wall Seg. | Total Vertical Load on the Wall Slab (Kips) | | | Total Lateral Load On the Wall Slab (Kips) |
|---|---|---|---|---|
| | Total Load | | Permanent Load | |
| | On the Wall Slab | Max. Per Pile | On the Wall Slab | |
| B | 480 | 143 | - | 90 |

CIDH concrete pile geotechnical resistance was calculated using skin friction resistance only within alluvial soils. Pile end bearing was not utilized for pile geotechnical resistance due to large differential movement required between mobilization of skin friction versus end bearing resistance. Axial geotechnical resistance for the proposed CIDH piles was calculated using the Federal Highway Administration's Drilled Shaft Manual (Pub. No. FHWA-IF-99-025, published August 1999) and SHAFT for Windows, V5.0 by ENSOFT Inc.

Foundation Design Recommendations and Pile Data Table for each support are provided in Table 8 and 9, respectively.

**Table 8 – Retaining Wall No. 1150 Foundation Design Recommendation**

| Wall Seg. | Pile Dia. and Type | Cut-off Elev. (ft) | LRFD Service-I Limit State Load (kips) on the Wall Slab | | LRFD Service-I Limit State Total Load (kips) per Pile (Compression) | Nominal Resistance (kips) | Design Tip Elevations (ft) | Specified Tip Elevation (ft) |
|---|---|---|---|---|---|---|---|---|
| | | | Total | Permanent | | | | |
| B | 24" CIDH | 181.46 | 480 | - | 143 | 290 | 126.2 (a)<br>148.2 (b)<br>136.2 (c) | 126.2 |

Notes:
1. Design tip elevations presented above were estimated based on the following requirements: (a) Compression, (b) Settlement, and (c) Lateral Load.

**Table 9 - Retaining Wall No. 1150 Pile Data Table**

| Wall Segment | Pile Dia. and Type | Nominal Resistance (kips) | | Design Tip Elevation (ft) | Specified Tip Elevation (ft) |
|---|---|---|---|---|---|
| | | Compression | Tension | | |
| B | 24" CIDH | 290 | 0 | 126.2 (a), 148.2 (b), 136.2 (c) | 126.2 |

Notes:
1. Design tip elevations presented above were estimated based on the following requirements: (a) Compression, (b) Settlement, and (c) Lateral Load.

## Settlement

Total settlement of the pile under service load is estimated to be less than 0.5 inch.

## Lateral Geotechnical Capacity

Lateral deflections for the proposed piles were calculated for the given loading using the computer program LPILE PLUS 5.0 (Reese et. al., 2004). The lateral deflections for the given loading were analyzed for fixed-head and free-head conditions, and the results are summarized in Table 10. The deflections were calculated for the gross (uncracked) moment of inertia per the request by the OBDS2.

**Table 10 – Pile Lateral Deflections**

| Pile Type | Pile-Head Condition | Lateral Load Per Pile (kips) | Lateral Pile Head Deflection (in) |
|---|---|---|---|
| 24" CIDH | Free-Head | 22.5 | 0.47 |
| | Fixed-Head | 22.5 | 0.14 |

## SLOPE STABILITY AND IDEALIZED SOIL PARAMETERS

Global stability analysis for static and pseudo-static (earthquake) loading was performed using computer software Slope/W Version 5.0, utilizing Spencer Method to compute the minimum factor of safety for circular failure surfaces. OGDS1 has performed slope stability analyses at one critical location identified to have the maximum combined wall and slope height at approximate Station 1151+50 Centerline Rte. 710.

The analyses were performed using the generalized soil profile developed based on the subsurface information and laboratory results from the recent field investigation. The soil profile with corresponding soil parameters used for the slope stability analyses are given in Table 11 below. A traffic surcharge load of 240 psf was applied at the top of the slope. For pseudo-static analysis, a seismic force was applied to the soil mass based on a horizontal seismic acceleration coefficient, $k_h$, equal to one-third of the design PGA.

The slope stability analysis yielded a factor of safety greater than the minimum acceptable values of 1.3 and 1.1 for static (global) stability and pseudo-static condition, respectively (Section 5.2.2.3; Bridge Design Specifications - August 2004).

**Table 11 - Idealized Soil Parameters for Slope Stability**

| Idealized Soil Type | Approximate Elevation Range ft | Thickness ft | Unit Weight pcf | Internal Friction Angle (degrees) | Cohesion psf |
|---|---|---|---|---|---|
| Silty Sand | 189-161.5 | 27.5 | 110 | 32 | 100 |
| Silt, | 161.5-151.5 | 10.0 | 125 | 25 | 500 |
| Sand, Sand w/Silt, Sandy Silt, Silty Sand | 151.5-136.5 | 15.0 | 115 | 31 | 50 |
| Sandy Clay, Lean Clay | 136.5-116.5 | 20.0 | 135 | 25 | 500 |
| Sand, Sand w/Silt, Silty Sand | 116.5-91.5 | 25.0 | 120 | 34 | 50 |

## CONSTRUCTION CONSIDERATIONS

1. Foundation excavations should be cleaned of loose debris. Should any large rock fragments, rebar, or other debris be found at the bottom of footing elevations, the contractor should be prepared to remove and replace them with either structure backfill compacted to 95% R.C. or lean concrete.

2. All earthwork is expected to be carried out by conventional equipment. Fill placed on sloping ground shall be properly keyed and benched into existing ground and placed as specified in Section 19-6 of the Caltrans Standard Specifications (May 2006).

3. Free water shall not be allowed to stand in any excavations. If excavations become flooded, a minimum of the bottom 6 inches of soil shall be removed and replaced or

recompacted per Caltrans specifications. If materials are degraded to a further extent, more removal and replacement may be necessary.

4. Based on soil types encountered during the recent investigation, OGDS1 recommends a slope ratio of 1H:1V or flatter for the temporary back cut slope and excavations for construction. If there are constraints due to construction or traffic concerns, temporary shoring may be utilized to accommodate steeper excavations for the proposed spread footings.

5. Minor to moderate caving may be anticipated during excavation within fill and coarse granular alluvial material including sands. Temporary casing or other suitable method may be considered to stabilize pile borings during excavation and construction of CIDH piles.

6. Groundwater is not considered to be a problem for CIDH pile construction. OGDS1 considers that CIDH piles will be constructed within dry pile borings based on the recent foundation investigation, historic groundwater review, and As-Built LOTB information.

7. The bottom of CIDH pile excavations should be cleaned of loose debris before placing concrete.

The recommendations contained in this report are based on specific project information regarding foundation loading, locations, type, and elevations that have been provided to OGDS1, Branch D. If any conceptual changes are made during final project design, this office should review those changes to determine if these foundation recommendations are still applicable.

If you have any questions and/or further assistance is required please contact Pratheep Piratheepan at (213) 620-2363 or Shiva Karimi at (213) 620-2146.

Prepared by: Date: 08/23/2010

Pratheep Piratheepan, P.E., G.E.
Transportation Engineer
Office of Geotechnical Design South 1
Branch D

Supervised by: Date: 08/23/2010

Shiva Karimi, D., P.E., G.E.
Senior Transp[illegible]
Office of Geot[illegible] 1
Branch D

Prepared by: Date: 08/23/2010

Joe Pratt, C.E.G. No. 2141
Engineering Geologist
Office of Geotechnical Design South 1
Branch D

cc:

| | | |
|---|---|---|
| District Project Manager | James S. Hsu | James_s_Hsu@dot.ca.gov |
| GS Corporate | Mark Willian | Mark_Willian@dot.ca.gov |
| District Construction R.E. Pending File | | TBD |
| Structure Construction R.E. Pending File | | RE_Pending_File@dot.ca.gov |
| DES Office Engineer, Office of PS&E | Efren Ancheta | Efren_Ancheta@dot.ca.gov |
| District Materials Engineer | Kirsten Stahl | Kirsten_Stahl@dot.ca.gov |

**State of California** **Business, Transportation and Housing Agency**

# M e m o r a n d u m

***Flex your power!***
***Be energy efficient!***

**To:** MS. CHUNG-YUAN WEN
Office of Bridge Design South 2
Bridge Design Branch 21
21073 Pathfinder Rd, Suite 20

**Date:** November 12, 2010
**File**: 07-LA-710-PM 23.77
07-202111
Whittier Blvd Undercrossing
(Widen)
Bridge No. 53-1150

Attention: Mr. Newton Armstrong

**From:** **DEPARTMENT OF TRANSPORTATION**
**DIVISION OF ENGINEERING SERVICES**
**Geotechnical Services**
**Office of Geotechnical Design – South 1**
**Branch D**

**Subject:** Foundation Report for Whittier Blvd Undercrossing (Widen), Interstate 710

## INTRODUCTION

This report was prepared in response to the Office of Bridge Design South 2's (OBDS2's) request, dated May 21, 2009, for Foundation Report (FR) for the proposed widening of the SB Interstate 710 (I-710) at Whittier Boulevard Undercrossing (UC).

## SCOPE OF WORK

The recommendations contained in this report are based on the subsurface information gathered during the recent field investigation performed in August 2009. Tasks completed by the Office of Geotechnical Design South 1 (OGDS1), Branch D include the following:

1. Review of the regional geology and seismicity,
2. Review of pertinent information from previous geology/geotechnical reports and As-Built plans in the project area to evaluate the subsurface information,
3. Drilling, logging, and sampling of two rotary wash borings at the subject bridge site to characterize the subsurface conditions,
4. Laboratory testing of selected soil samples for mechanical analysis, corrosivity, direct shear, and unit weight,
5. Geotechnical engineering analysis.

## PROJECT DESCRIPTION

The existing Whittier Blvd UC is a single-span structure with a reinforced concrete deck on composite welded plate girders that are supported on two closed-end cantilever abutments. As Built drawings indicate that the original bridge was constructed in 1960 with no median gap in between the northbound (NB) and southbound (SB) portions. The As-Built drawings indicate that the bridge was widened in 1967 by approximately 25 feet on the right (NB) side and by a variable amount (18 feet

minimum) on the left (SB) side. The drawings also indicate that the two abutments are supported on 16-inch diameter Cast-in-Drilled-Hole (CIDH) piles installed vertically and at a batter.

Based on the information provided by OBDS2, OGDS1 understands that the proposed project will consist of outside widening the left side (SB side) of the bridge by about 3 feet 4 to 6 inches (varies). The widening is conducted to upgrade the existing lane widths to the current standards, and to provide 10 foot wide shoulders. Total length of the bridge is 87.5 feet (measured along Centerline Route 710) with proposed Begin and End Bridge elevations at +216.0 ft and +216.4 ft, respectively.

All elevations provided within this report and proposed project plans are based on NAVD88 datum.

## FIELD INVESTIGATION AND TESTING PROGRAM

Field exploration was performed during August 18 and 19, 2009. The field investigation included drilling and sampling two rotary wash borings (4.5 inches diameter). Standard Penetration Tests (SPT's), and relatively undisturbed sampling (with a split-barrel sampler, 2 inches inner diameter) were performed at the borings. Blow counts (SPT N values) were mostly recorded at 5 foot intervals during drilling. SPT's were performed in accordance with ASTM Test Method D1586-84 using a standard 1.4 inch I.D. sampler with a 140 lb. hammer dropped 30 inches. Undisturbed tube soil samples were also obtained using a 2 inch I.D. modified California Split Spoon Sampler with 4 inch long brass liners. The liners were all sealed in the field. A Caltrans operated drill rig model CME-75 was used at both locations.

District 7 Surveys provided the location and elevation of the borings. Boring information including exploration number, stationing, offset, ground surface elevation, boring depth, and date drilled are summarized in Table 1.

**Table 1 – Summary of Borings**

| Boring No. | Route 710 Centerline Station (ft) | Offset (ft) | Surface Elev. (ft) | Depth (ft) | Date Drilled |
|---|---|---|---|---|---|
| R-09-054 | 1254+56.8 | 72.1 Lt. | +214.1 | 100.5 | 8/19/09 |
| R-09-056 | 1256+19.2 | 115.0 Lt. | +197.9 | 100.4 | 8/18/09 |

## LABORATORY TESTING PROGRAM

Selected soil samples were sent to the Department's Transportation Laboratory in Sacramento for testing. All laboratory tests were performed in accordance with ASTM standard procedures and California Test Methods. The summarized laboratory tests data are shown in Table 2.

**Table 2 – Summary of Laboratory Tests**

| Test | Standard | No. of Tests Performed |
|---|---|---|
| Mechanical Analysis | CTM 202, 203 | 9 |
| Atterberg | CTM 204 | 4 |
| Unit Weight | CTM 212 | 6 |

| | | |
|---|---|---|
| Moisture | CTM 226 | 6 |
| Corrosion | CTM 417, 422, 532 | 14 |
| Direct Shear | ASTM D 3080 | 1 |

## SITE GEOLOGY AND SUBSURFACE CONDITIONS

### Regional Geology

The bridge site is located in the Peninsular Ranges Province in the Los Angeles Basin and is bounded to the northwest and north by the Santa Monica Mountains and Santa Monica and Raymond Hill faults, to the west, southwest, and south by the Newport-Inglewood Fault Zone, to the northeast and east by the Whittier fault and Puente Hills fault, and to the southeast by the Santa Ana Mountains and San Joaquin Hills. A thick Cenozoic sedimentary section underlies the site.

### Site Subsurface Conditions

At the Whittier Blvd. Undercrossing (Widen) site, embankment fill is approximately 20 ft thick at Abutments 1 and 2. Embankment fill is underlain by Holocene and older Quaternary alluvium (Qa and Qoa, Dibblee, 1989, Geologic Map of Los Angeles 7.5 minute Quadrangle). The Pliocene Fernando Formation (Tfsc, nonmarine sandstone and conglomerate and Tfr, marine claystone) most likely underlies the older alluvium at depth.

Top of embankment ranges between approximately +216.0 to +216.5 ft elevations at Abutments 1 and 2, respectively. The bottom of the embankment fill extends down to between approximate elevations +194 to +196 and possibly to +191 ft. Embankment fill is composed of medium dense, clayey sand with gravel and sandy silt. Underlying Holocene alluvium (approximately 12 to 6 ft thick) ranges from approximate elevations +194 to +196 down to +186 to +183 ft. and is composed of medium dense/from stiff to hard, clayey sand, sandy silt, and sandy clayey silt. Underlying older Quaternary alluvium can be divided into two units. The older upper alluvial unit ranging from approximate elevations +186 to +183 ft down to +157 to +172 ft (approximately 29 to 15 ft thick) consists of generally medium dense to very dense/very stiff to hard, sandy silt, clayey to silty sand, and sandy clay interbeds. The older lower alluvial unit ranging from approximate elevations +157 to +172 down to at least +97.5 ft, consists of dense to very dense/minor hard, sand and silty sand with sporadic gravel interbedded with minor sandy clay.

### Groundwater

No groundwater was encountered within the two August 2009 borings which extended down to a maximum depth of 100.5 ft (approximate elevation +97.5 ft) during Caltrans' recent field investigation. A piezometer was installed in Boring R-09-056. Based on successive groundwater measurements taken on August 16, 2009, February 25, 2010, and September 30, 2010, no groundwater was encountered within Boring R-09-056 to at least 100.4 ft depth (dry hole to at least +97.5 ft elevation).

As-Built LOTB's for the 1960 original bridge and 1967 widening (three-3 inch diameter rotary sample borings, three-2.25 inch diameter soil penetration tests, and two-1 inch diameter soil tubes completed in 1956 and 1963) also show no groundwater was encountered (dry holes) down to approximate Elevation +145 ft., the maximum exploration depth obtained.

## SCOUR EVALUATION

There is no possibility of scour at the site due to the large distance from any active channel.

## CORROSION EVALUATION

The soil sample corrosion test results are presented in the Table 3 below. The test results show this area can be considered non-corrosive to concrete and metal.

**Table 3 – Corrosion Test Summary**

| Exploration No. | Sample Depth (ft) | PH | Minimum Resistivity (ohm – cm) | Sulfate Content (PPM) | Chloride Content (PPM) |
|---|---|---|---|---|---|
| R-09-054 | 5.0-16.5 | 7.93 | 3596 | N/A | N/A |
| R-09-054 | 11.0-11.3 | 7.54 | 2373 | N/A | N/A |
| R-09-054 | 20.7-21.0 | 7.69 | 4315 | N/A | N/A |
| R-09-054 | 25.0-26.5 | 8.06 | 3016 | N/A | N/A |
| R-09-054 | 40.7-41.0 | 7.98 | 4389 | N/A | N/A |
| R-09-054 | 45.0-46.5 | 8.07 | 2536 | N/A | N/A |
| R-09-054 | 55.0-71.5 | 8.10 | 6398 | N/A | N/A |
| R-09-054 | 75.0-81.5 | 8.09 | 6627 | N/A | N/A |
| R-09-056 | 5.0-16.5 | 7.74 | 2888 | N/A | N/A |
| R-09-056 | 10.7-11.0 | 7.44 | 1864 | N/A | N/A |
| R-09-056 | 20.7-21.0 | 7.50 | 2002 | N/A | N/A |
| R-09-056 | 25.0-26.5 | 7.76 | 2778 | N/A | N/A |
| R-09-056 | 35.0-56.5 | 8.07 | 5738 | N/A | N/A |
| R-09-056 | 70.0-86.5 | 8.01 | 7508 | N/A | N/A |

Note: It is the practice of Caltrans Corrosion Technology Section (with the exception of MSE Walls) if the minimum resistivity of the sample is greater than 1000 ohm-cm and the pH is greater than 5.5, the sample is considered to be noncorrosive. For structural elements, the California Department of Transportation considers a site to be corrosive if one or more of the following conditions exist for representative soil and/or water samples taken at the site:
Chloride concentration $\geq$500 ppm, sulfate concentration $\geq$2000 ppm, or the pH is $\leq$5.5. Corrosion mitigation is required if one or more of the 3 conditions noted above exists where structural elements are involved (Caltrans Corrosion Guidelines, September 2003).

## SEISMIC RECOMMENDATIONS

### Faulting and Seismicity

The controlling seismic source for this bridge is the Puente Hills Blind Thrust (PHBT) fault. Caltrans has assigned a Maximum Credible Earthquake (MCE) of moment magnitude 7.25 to this fault. This is a buried thrust fault with a site-to-rupture surface distance of about 4.0 miles. Based on the Sadigh et al (1997) attenuation relationships, the median Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.6g.

### Potential Seismic Hazards

This site is not located within a potentially liquefiable zone as shown on the California Geological Survey Map of Seismic Hazard Zones of the Los Angeles 7.5 Minute Quadrangle (March 25, 1999). Based on Caltrans recent study, the As-Built data, and lack of groundwater to at least 100 ft depth, the possibility of liquefaction is expected to be low to nil.

### Design Ground Motion

For the purpose of selecting a standard SDC (2006) Acceleration Response Spectrum (ARS) curve, the design PBA is taken as 0.6g. The soil profile may be classified as Type D. The design ARS curve was obtained by modeling the standard SDC curve (for PBA = 0.6g, M = 7.25 ± 0.25, and soil profile type D) for near fault effects. The recommended ARS Curve and the coordinates of spectral accelerations are presented in Figure 1 and Table 4 below, respectively.

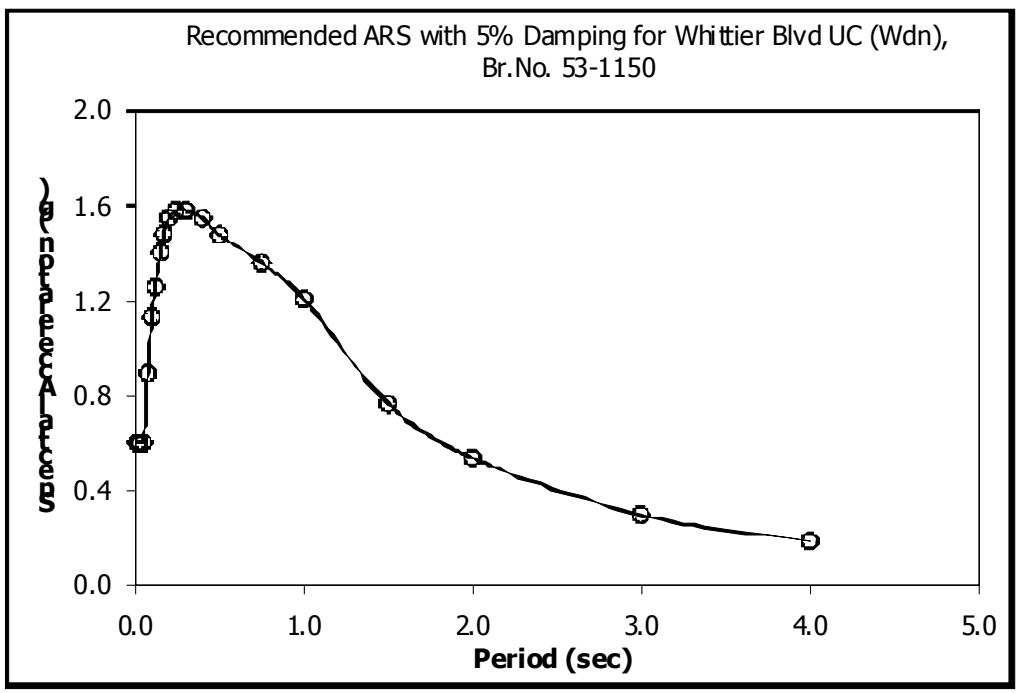


**Figure 1 - Recommended ARS curve for Whittier Blvd UC (Wdn)**

**Table 4 - Recommended Design ARS curve for Br. No. 53-1150**

| Period (s) | Spectral Acceleration (g) | |
|---|---|---|
| | Standard | **Modified** |
| 0.010 | 0.6003 | 0.6003 |
| 0.020 | 0.6003 | 0.6003 |
| 0.030 | 0.6003 | 0.6003 |
| 0.050 | 0.6003 | 0.6003 |
| 0.075 | 0.8934 | 0.8934 |
| 0.100 | 1.1315 | 1.1315 |
| 0.120 | 1.2590 | 1.2590 |
| 0.150 | 1.4033 | 1.4033 |
| 0.170 | 1.4802 | 1.4802 |
| 0.200 | 1.5496 | 1.5496 |
| 0.240 | 1.5784 | 1.5784 |
| 0.300 | 1.5811 | 1.5811 |
| 0.400 | 1.5484 | 1.5484 |
| 0.500 | 1.4767 | 1.4767 |
| 0.750 | 1.2366 | 1.3602 |
| 1.000 | 1.0062 | 1.2074 |
| 1.500 | 0.6378 | 0.7653 |
| 2.000 | 0.4464 | 0.5357 |
| 3.000 | 0.2471 | 0.2965 |
| 4.000 | 0.1546 | 0.1855 |

**Surface Fault Rupture Hazard**

The project site is not located within and California Geological Survey (CGS) designated Earthquake Fault Zone (EFZ) or directly underlain by any fault considered active for bridge design. Therefore, the possibility of surface fault rupture hazard at the site is considered low.

**AS-BUILT FOUNDATION DATA**

The 1960 As-Built Pile Data for the original bridge construction are shown in Table 5. Vertical and battered CIDH piles (16" diameter) are shown on the plans. Based on the As-Built General Plan (corrections March 10, 1966) for the 1967 outside widenings, 16 inch diameter CIDH piles (45 tons design loading) were used. As-Built plan elevations require at +1.83 ft shift to adjust to the current NAVD88 datum.

**Table 5 – 1960 Original As-Built Foundation Data for Whittier Blvd UC**

| Support Location | Pile Type (inch) | Service Loads (kips) | Bottom of Pile Footing Elev. (ft) | Average Pile Tip Elev. (ft) |
|---|---|---|---|---|
| Abut 1 | 16 CIDH | 90 | 188.0 | 164.2 |
| Abut 2 | 16 CIDH | 90 | 189.5 | 165.2 |

Note: Piles may be all 16" CIDH, but 178 driven concrete piles are shown in the quantities.

**Table 6 – 1967 As-Built Foundation Data for Whittier Blvd UC (Wdn)**

| Support Location | Pile Type (inch) | Service Loads (kips) | Bottom of Pile Footing Elev. (ft) | Specified Pile Tip Elev. (ft) |
|---|---|---|---|---|
| Abut 1 | 16 CIDH | 90 | 188.0 | 165.0 |
| Abut 2 | 16 CIDH | 90 | 189.5 | 165.0 |

## FOUNDATION RECOMMENDATIONS

The following recommendations are based on 1) the recent geotechnical investigation with lab test results, 2) updated structure plans (revised October 1, 2010), micropile design loads, and pile cut-off elevations provided by OBDS2.

Based on the information provided by OBDS2, the proposed left side widening will include adding abutment wingwalls supported on micropiles at both abutments. Micropiles are the best foundation support due to space constraints and placing piles through drilled holes in preexisting footings.

### Abutment Wingwall Micropiles

General foundation information for wingwall micropiles NPS12 (outside dia. 12.8 in, nominal thickness 0.5 in) at Whittier Blvd UC were provided by OBDS2 (email dated September 28, 2010) as shown in Tables 7 and 8.

Computer program SHAFT Version 5.0 was used for calculating micropile (NPS12 filled with cement grout)/soil resistances as shown in Table 9. The analysis was conducted based on the assumption that proposed micropiles with grout would derive geotechnical capacity from skin friction only.

**Table 7 - Foundation Design Data Sheet**

| Support No. | Design Method | Pile Type/ Thickness (inch) | Finished Grade Elevation (ft) | Cut-off Elevation (ft) | Pile Cap Size (ft) | | Permissible Settlement Under Service Load (in)* | Number of Piles per Support |
|---|---|---|---|---|---|---|---|---|
| | | | | | B | L | | |
| Pile 1 @ Abut 1 WW | LRFD | NPS 12 xx-strong | N/A | 196.35 | N/A | N/A | 1 | 1 |
| Pile 2 @ Abut 1 WW | LRFD | | N/A | 202.85 | N/A | N/A | 1 | 1 |
| Pile 3 @ Abut 1 WW | LRFD | | N/A | 202.85 | N/A | N/A | 1 | 1 |
| Pile 1 @ Abut 2 WW | LRFD | NPS 12 xx-strong | N/A | 196.85 | N/A | N/A | 1 | 1 |
| Pile 2 @ Abut 2 WW | LRFD | | N/A | 202.85 | N/A | N/A | 1 | 1 |
| Pile 3 @ Abut 2 WW | LRFD | | N/A | 202.85 | N/A | N/A | 1 | 1 |

* Based on CALTRANS' current practice, the total permissible settlement is one inch for multi-span structures with continuous spans or multi-column bents, one inch for single span structures with diaphragm abutments, and two inches for single span structures with seat abutments. Different permissible settlement under service loads may be allowed if a structural analysis verifies that required level of serviceability is met.

**Table 8 - Foundation Design Loads**

| Support No. | Service-1 Limit State (kips) | | | Strength Limit State (Controlling Group, kips) | | | | Extreme Event Limit State (Controlling Group, kips) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | Total Load | | Permanent Loads | Compression | | Tension | | Compression | | Tension | |
| | Per Support | Max Per Pile | Per Support | Per Support | Max Per Pile | Per Support | Max Per Pile | Per Support | Max Per Pile | Per Support | Max Per Pile |
| Abut 1 WW | N/A | 60.5 | N/A | N/A | 83.6 | N/A | N/A | N/A | 70.5 | N/A | N/A |
| Abut 2 WW | N/A | 56.3 | N/A | N/A | 78.4 | N/A | N/A | N/A | 66.3 | N/A | N/A |

**Table 9 – Abutment Wingwall Foundation Design Recommendations for Whittier Blvd UC (Wdn), Br. No. 53-1150**

| Support Location | Micro -pile Type | Cut-off Elev. (ft) | Service-I Limit State Total Load per Pile (kips) | Foundation Design Load (kips) | | | | Design Tip Elev. (ft) | Specified Tip Elev. (ft) |
|---|---|---|---|---|---|---|---|---|---|
| | | | | Strength Limit | | Extreme Event | | | |
| | | | | Comp. (φ=0.7) | Tension (φ=0.7) | Comp. (φ=1) | Tension (φ=1) | | |
| Abut 1 WW; Pile No.1 | NPS 12 – xx strong | 196.35 | 60.5 | 83.6/0.7 =121 | N/A | 70.5 | N/A | 166.3 (a-I)<br>172.8 (a-II)<br>176.8 (c) | 166.3 |
| Abut 1 WW; Pile No.2 | | 202.85 | 60.5 | 83.6/0.7 =121 | N/A | 70.5 | N/A | 172.8 (a-I)<br>177.8 (a-II)<br>180.8 (c) | 172.8 |
| Abut 1 WW; Pile No.3 | | 202.85 | 60.5 | 83.6/0.7 =121 | N/A | 70.5 | N/A | 172.8 (a-I)<br>177.8 (a-II)<br>180.8 (c) | 172.8 |
| Abut 2 WW; Pile No.1 | NPS 12 – xx strong | 196.85 | 56.3 | 78.4/0.7 =112.6 | N/A | 66.3 | N/A | 166.8 (a-I)<br>175.3 (a-II)<br>178.3 (c) | 166.8 |
| Abut 2 WW; Pile No.2 | | 202.85 | 56.3 | 78.4/0.7 =112.6 | N/A | 66.3 | N/A | 172.8 (a-I)<br>178.8 (a-II)<br>182.8 (c) | 172.8 |
| Abut 2 WW; Pile No.3 | | 202.85 | 56.3 | 78.4/0.7 =112.6 | N/A | 66.3 | N/A | 172.8 (a-I)<br>178.8 (a-II)<br>182.8 (c) | 172.8 |

*Notes:*

1) *Design tip elevations are controlled by: (a-I) Compression (Strength Limit), (a-II) Compression (Extreme Event), and (c) Settlement, respectively.*
2) *The micropile specified tip elevation shall not be raised.*
3) *Design tip elevation for Lateral Load is provided by SD.*

*4) Predrilled holes are required for micropile installation with diameter and elevation of bottom of predrilled holes shown in the standard special provisions.*

**Settlement at Approach Fills**

All earthwork is expected to be carried out by conventional equipment. Fill should be placed as specified in Section 19-6 of the Caltrans Standard Specifications, (May 2006). Additional fill placement /replacement is expected to be minimal. Calculated maximum settlements are negligible for the minor left side widening, so consequently, OGDS1 does not require a fill settlement waiting period. However, the actual settlement period (if needed) will be determined by the structure representative on the basis of settlement data in the field.

## NOTE TO DESIGNER

The recommendations contained in this report are based on specific project information regarding bridge abutment wingwall locations, micropile type, finished grade elevation, micropile cut-off elevation and service load that has been provided by OBDS2 to OGDS1. If any conceptual changes are made during final project design, OGDS1 should review those changes to determine if these foundation recommendations are still applicable.

## CONSTRUCTION CONSIDERATIONS

1. Groundwater is not anticipated during drilling and construction of the micropiles.
2. The contractor shall be required to clean out the micropile borings prior to placing and grouting the micropiles.
3. Grout placement for the micropiling shall be completed within the same day that excavation of the drilled hole has been completed.
4. Moderate to minor caving and sloughing should be anticipated during excavation of the pile borings and during micropile construction.

If you have any questions or need further information, please contact Kevin Lai at (213) 620-2344 or Shiva Karimi at (213) 620-2146 and Joe Pratt at (213) 620-2313.

Prepared by: Date: 11/12/10

Kevin Lai,
Transportation Engineer
Office of Geotechnical Design – South 1
Branch D

Supervised by: Date: 11/12/10

Shiva Karimi, Ph.D., P.E.
Senior Transportation Engineer
Office of Geotechnical Design – South 1
Branch D

REGISTERED PROFESSIONAL ENGINEER
SHIVA KARIMI
No. GE2651
Exp. 6/30/12
GEOTECHNICAL
STATE OF CALIFORNIA

Reviewed by: Date: 11/12/10

Joe Pratt, C.E.G No.2141
Engineer Geologist
Office of Geotechnical Design – South 1
Branch D

REGISTERED GEOLOGIST
Joseph S. Pratt
No. 2141
Exp. 5/31/11
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

cc:

| | | |
|---|---|---|
| District Project Manager | James S. Hsu | James_s_Hsu@dot.ca.gov |
| GS Corporate | Mark Willian | Mark_Willian@dot.ca.gov |
| District Construction R.E. Pending File | | TBD |
| Structure Construction R.E. Pending File | | RE_Pending_File@dot.ca.gov |
| DES Office Engineer, Office of PS&E | Efren Ancheta | Efren_Ancheta@dot.ca.gov |
| District Materials Engineer | Kirsten Stahl | Kirsten_Stahl@dot.ca.gov |

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State of California
**DEPARTMENT OF TRANSPORTATION**

Business, Transportation and Housing Agency

# M e m o r a n d u m

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**To:** MS. CHUNG-YUAN WEN
Office of Bridge Design South 2
Bridge Design Branch 21
21073 Pathfinder Road, Suite 20

**Date:** September 2, 2010

**File**: 07-LA-710-23.44
07-202111
Olympic Blvd U.C. (Widen)
Bridge No. 53-1044

**From:** **DEPARTMENT OF TRANSPORTATION**
**Division of Engineering Services**
Geotechnical Services
Office of Geotechnical Design – South 1
Branch D

**Subject:** Foundation Report for Olympic Boulevard Undercrossing Bridge Widening, Bridge No. 53-1044, Interstate 710

## INTRODUCTION

This Foundation Report (FR) was prepared in response to your memorandum (dated May 21, 2009) requesting Foundation Recommendations for the proposed widening of the Interstate 710 (I-710) bridge over Olympic Boulevard in the City of Los Angeles. The proposed right outside widening will consist of extending the structure carrying northbound traffic of I-710 outwards, as described below.

## SCOPE OF WORK

The scope of our work for the proposed project included the following.

1. Review of the available Foundation Reports from previous investigations,
2. Field visits and observations to assess the site conditions,
3. Evaluation of the site subsurface conditions based on boreholes excavated for 2005 widening of the subject bridge and 1955 As-Built Log of Test Borings (LOTB) for the original bridge,
4. Evaluating the site seismicity and performing a seismic hazard analysis, and
5. Geotechnical analysis and preparation of this report

## PROJECT DESCRIPTION

The present Olympic Boulevard Undercrossing is a single-span bridge containing the northbound (NB) and southbound (SB) lanes of I-710. The original structure, constructed in 1955, was subsequently widened in 1967 and 2005.

Based on the information provided by Office of Bridge Design South 2 (OBDS2), the Office of Geotechnical Design South 1 (OGDS1) understands that the proposed project will consist of widening the structure containing NB lanes by approximately 5 feet 3 inches outwards, to provide a standard 10-foot wide shoulder.

OBDS2 indicated that for the proposed widening, abutment walls and the back walls would be extended outwards and would be supported on cast-in drilled-hole (CIDH) piles. OBDS2 proposed using 140-kip and 90-kips piles for supporting the abutment and back walls, respectively.

All elevations provided within this report and proposed project plans are based on NAVD88 datum.

## GEOTECHNICAL INVESTIGATION PROGRAM

No field exploration or a laboratory test program was conducted for this project since the subsurface information obtained from 2004 geotechnical investigation was considered adequate for the proposed widening. Therefore, a brief description of the past geotechnical investigations conducted at the site, particularly of the 2004 investigation, is given below.

Past geotechnical investigations conducted at the site consisted of the 1953 subsurface exploration for the original bridge construction and the 2004 exploration for the median widening. In the subsurface exploration conducted in 1953, four alternating open end (with sampling) and closed end (no sampling), 1-inch diameter soil tubes were driven and the hammer blows recorded. The LOTB for this exploration, dated April 26, 1954, indicates tubes were driven to a maximum depth of about 44 feet below the original ground surface at the site (approximate surface elevation +182.5 feet based on NAVD88 datum). The maximum penetration depth extended down to approximate elevation +138.5 feet. During the 2004 soil investigation, two hollow stem auger borings were excavated from freeway grade at approximate elevations +202 and +204 feet (NAVD88 datum), down to a maximum depth of 90 feet (approximate elevation +113 feet). A number of geotechnical laboratory tests were also conducted during the 2004 investigation to determine engineering properties of subsurface materials.

The locations of the borings drilled for 2004 investigation are summarized in Table 1 below.

**Table 1 – Summary of 2004 Boring Locations**

| Boring | General Location | Station (m) | Offset, feet (m) | Elevation, feet (m) |
|---|---|---|---|---|
| B-1 | Abutment 2, travel lane of S/B on-ramp | 377+79.4 | 84.6 feet (25.8 m) Left | +204.3 feet (+62.3 m) |
| B-2 | Abutment 1, outside shoulder of N/B freeway | 376+99.3 | 37.1 feet (11.3 m) Right | +202.4 feet (+61.7 m) |

Note: Boring stationing and offsets are based on centerline I-710 and elevations are based on NAVD88 datum.

## LABORATORY TESTING PROGRAM

A summary of laboratory tests performed for the 2004 investigation is presented Table 2 below. Laboratory tests included mechanical analysis, Atterberg Limits, direct shear and corrosivity, conducted in accordance with California Test Methods and/or ASTM procedures.

**Table 2 – Summary of Laboratory Tests for 2004 Investigation**

| Test | Standard | No. of Test Performed |
|---|---|---|
| Moisture Content/Unit Weight | CTM 226 | 6 |
| Mechanical Analysis | CTM 202, 203 | 8 |
| Atterberg Limits | CTM 204 | 8 |
| Corrosion | CTM 417, 422, 532 | 2 |
| Direct Shear | ASTM D3080 | 4 |

## SITE GEOLOGY AND SUBSURFACE CONDITIONS

### Regional Geology

The site is located in the Peninsular Ranges Province of the Los Angeles Basin and is bounded to the northwest/north by the Santa Monica Mountains, to the southwest/south by the Newport-Inglewood Fault Zone, to the northeast/east by the Whittier fault and Puente Hills, and to the southeast by the Santa Ana Mountains and San Joaquin Hills. A thick Cenozoic sedimentary section underlies the site.

### Site Subsurface Conditions

The site subsurface conditions presented in this report were interpreted primarily based on the information obtained in the 2004 geotechnical investigation conducted for 2005 bridge widening (Caltrans, 2005). The 2004 investigation included a comprehensive sampling and testing program compared to the previous 1953 investigation at the site. Due to the relative uniformity of subsurface conditions across the site (as observed in the 2004 investigation) and the close proximity of the 2004 borings, the subsurface conditions interpreted from that investigation are considered applicable for the proposed construction. A brief description of subsurface conditions observed at the two abutments is provided below.

The subsurface soils at the site consist of compacted fill placed for I-710 roadway embankment/approach fill and underlying alluvial soils. The fill material underlying the pavement, which is at an elevation of about +203 feet (NAVD88) consists mainly of stiff to very stiff, sandy clay and sandy silt. Although, the fill-alluvium contact is not indicated in the 2004 borings, OGDS1 believes that the fill-alluvium contact could be at an elevation of about +178 to +182 feet based on the as-built drawings. The alluvial materials encountered in the borings are

predominantly fine grained and consist of moist, sandy silt, sandy clay and lean clay with minor layers of medium dense to dense, sand and silty sand. From approximate elevations ranging from +182 to +178 feet down to an elevation of about +150 feet, clayey and silty alluvial soils are generally stiff to very stiff. From elevations +150 down to +113 feet, which is the maximum depth of exploration, the alluvial soils observed in the borings are very stiff to hard.

### Groundwater

Both 1953 and 2004 explorations indicated that groundwater was not present in the subsurface materials extending down to a depth of 90 feet below freeway grade (down to approximate elevation +113 feet).

## CORROSION EVALUATION

As shown in Table 3 below, laboratory tests for corrosivity conducted during the 2004 investigation indicate that site soils are generally non-corrosive to metal and reinforced concrete, as per September 2003 Corrosion Guidelines.

**Table 3 - Summary of Corrosion Test Data**

| Boring/Depth (feet) | Sample No. | Corrosivity | | |
|---|---|---|---|---|
| | | PH CTM 643 | Resistivity (Ohm-cm) CTM 643 | Sulfate/Chloride (ppm) |
| B-1/65 | D8 | 8.17 | 1400 | NA |
| B-2/15-20 | B1 | 8.62 | 1600 | NA |

Note: 1. Not applicable
2.: Sulfate and Chloride tests were not performed as per Caltrans guidelines for samples with resistivity > 1000 ppm or the pH is 5.5 or less.

## SEISMIC RECOMMENDATIONS

### Faulting and Seismicity

The site is located at about 3.5 miles (5.7 km) and 3.9 miles (6.3 km) from the rupture surfaces of Upper Elysian Park Blind Thrust (UEPBT) and Puente Hills Blind Thrust Fault (PHBT), respectively, which are the two nearest active faults to the site. The moment magnitudes of Caltrans assigned Maximum Credible Earthquakes (MCE) of UEPBT and PHBT are 7.0 and 7.25, respectively. Based on these fault parameters and the ground motion attenuation relationships by Sadigh et al (1997), the MCE associated with each of these faults is capable of generating a median or design Peak Bedrock Acceleration (PBA) and Peak Ground Acceleration of 0.6g and 0.53g, respectively, at the site.

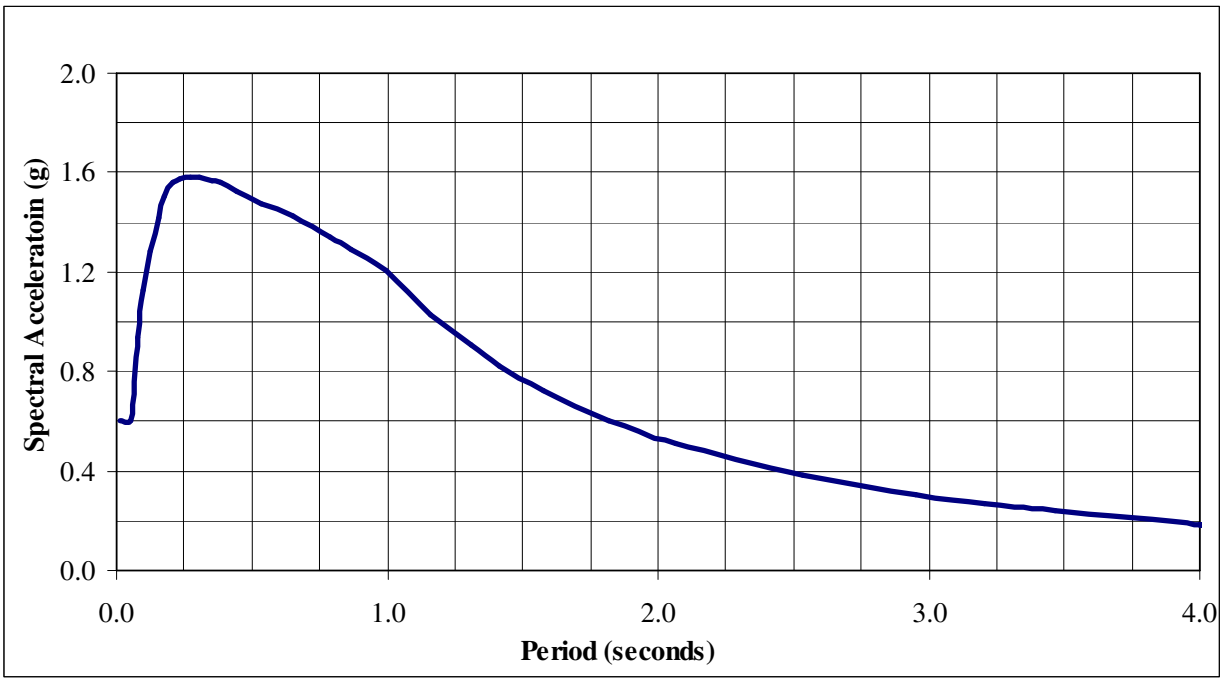
**Potential Seismic Hazards**

Liquefaction is a phenomenon in which loose, saturated, fine-grained, granular soils behave like a liquid while being subjected to high-intensity ground shaking. Liquefaction occurs when shallow ground water, low-density, fine, sandy soils and high-intensity ground motion exist at a site. Due to the presence of stiff cohesive soils at abutment locations and the absence of groundwater within the zone of influence, the potential for the occurrence of liquefaction at the site is considered remote. Consequently, the potential for seismically induced impacts such as settlement and lateral deformation of subsurface materials is remote.

The project site is not located within any California Geological Survey (CGS) designated Earthquake Fault Zone (EFZ) or directly underlain by any fault considered active for bridge design (Davis 1999). Therefore, the possibility of surface fault rupture hazard at the site is considered low.

**Design Ground Motion**

Subsurface conditions indicate the soil profile at the site can be categorized as Type D based on the Caltrans Seismic Design Criteria, (SDC, 2006) for the determination of design ground motion. Considering the local faulting and seismicity of the area surrounding the site, OGDS1 estimates design ground motion would be caused by a 7.25 magnitude event on PHBT fault. Due to the close proximity of the site to the causative fault (3.9 miles), the Standard Acceleration Response Spectrum (ARS) was modified for the near source effects per Section 6.1.2 of SDC, 2006. The modified ARS curve recommended for the proposed structure based on the above conditions is presented below.



## AS-BUILT FOUNDATION DATA

The original bridge, constructed in 1955, consisted of two separate structures for the NB and SB traffic, which were separated by a 12-foot wide gap. The two bridges were widened outwards, in 1967 to provide one additional lane each in the SB and NB directions and a shoulder in the NB direction. The bridges were widened inwards in 2005 to close the median gap between them.

The as-built plans indicate that the original bridge abutments are supported on octagonal-shaped precast concrete piles, 16-inch width (45 ton design load), approximately 26-foot long. The as-built plans also indicate piles were driven along two rows running parallel to the axis of the abutments. The front row piles were driven at a 1:3 (horizontal: vertical) batter. The As-Built plans indicate that the tip elevations of the vertical and battered piles are approximately +152 feet and the bottoms of the abutment footings are at approximate elevation +178 feet. The foundations used for supporting the 1967 widening consisted of 16-inch diameter cast-in-drilled–hole (CIDH) piles. The design loads on the as-built pile foundations were not available in the Caltrans Database, BIRIS for the 1967 widening. However, a Caltrans memorandum, dated January 7, 1964 recommended tips of the CIDH piles be placed at an elevation of about +151 feet to obtain an allowable geotechnical capacity of 45 tons.

No new piles were installed during the 2005 median widening since the geotechnical investigation conducted for the project indicated that the existing piles were capable of supporting the additional loads imposed by the widening. A summary of As-Built foundation data is presented in Table 4, below.

**Table 4 - As-Built Foundation Data for Olympic Blvd U.C.**

| Location | Pile Type/ Diameter or Dimension | Design Service Load (kips) | Approximate Cutoff Elevation (feet) | Approximate Pile Tip Elevation (feet) |
|---|---|---|---|---|
| **Original Bridge Built in 1955** | | | | |
| Abutments 1 and 2 | Concrete Alt. "X"/ 16 inch | 90 | about +178 | about +152 |
| **1967 Widening** | | | | |
| Abutments 1 and 2 | CIDH/16 inch | 90 (NA) | about +178 | about +151 |

Notes: (NA)- Not shown in Caltrans as-built records.

## FOUNDATION RECOMMENDATIONS

The geotechnical capacities of the proposed piles were evaluated using the design loads provided

by the OBDS2. The pile types recommended by OBDS2 and OGDS1 are vertically installed, 16 inch diameter, cast-in-drilled-hole (CIDH) piles. The loading and the pile types provided by OBDS2 are presented in the tables below.

**Table 5 - Foundation Data for Olympic Blvd. U.C. (Widen)**

| Support No. | Design Method | Pile Type/ Design Load | Finish Grade Elevation (ft) | Cut-off Elevation (ft) | Pile Cap Size (ft) | | Permissible Movement under Service Load (in) | | Number of Piles Per Support |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | B | L | Vert. (in) | Horizon. (in) | |
| Back Wall 1 | WSD | 16-inch CIDH/ 90 Kips | 202.9 | 192.69 | 3.0 | 7.0 | 0.25 | 0.25 | 2 |
| Abut. 1 | WSD | 16-inch CIDH/ 140 Kips | 183.0 | 176.89 | 7.0 | 7.0 | 0.25 | 0.25 | 4 |
| Abut. 2 | WSD | 16-inch CIDH/ 140 Kips | 182.5 | 176.89 | 7.0 | 7.0 | 0.25 | 0.25 | 4 |
| Back Wall 2 | WSD | 16-inch CIDH/ 90 Kips | 203.3 | 192.39 | 3.0 | 7.0 | 0.25 | 0.25 | 2 |

**Table 6 - LRFD Service Limit State I Load Data for Olympic Blvd U.C. (Widen)**

| Support No. | Vertical Load Per Pile (Kips) | | Lateral Load per Pile (Kips) |
|---|---|---|---|
| | Total Load | Permanent Load | |
| Back Wall 1 | 75 | 55 | 15.0 |
| Abutment1 | 95 | 80 | 20.0 |
| Abutment 2 | 95 | 80 | 20.0 |
| Back Wall 2 | 75 | 55 | 15.0 |

Pile lengths were computed based on the provided design loads and the observed subsurface conditions. The analysis was conducted based on the assumption that proposed piles would derive geotechnical capacity through both side friction (approximately 80%) and end bearing (approximately 20%). Site subsurface soils consist mainly of cohesive materials. Therefore, OGDS1 believes the potential for caving in the drilled holes is minor. As such, the pile capacities were evaluated assuming dry drilling conditions. Specified pile tip elevations are computed based on the observed site subsurface conditions and are presented below. The axial capacities were

evaluated using the Computer Program SHAFT (Ensoft, Version 5.0) based on procedures recommended by FHWA Drilled Shaft Manual (August 1999). The lateral pile responses were calculated using LPILE Program (Version 5). The tip elevations of the proposed piles computed based on the above methods are presented in Table 7.

**Table 7 - Pile Data Table for Olympic Blvd. U.C. (Widen)**

| Location | Pile Type | Finished Grade Elevation (ft) | Cut-off Elevation (ft) | Nominal Resistance (kips) | | Design Tip Elevations (ft) | Specified Tip Elevation (ft) |
|---|---|---|---|---|---|---|---|
| | | | | Compression | Tension | | |
| Back Wall 1 | 16" CIDH | 202.9 | 192.69 | 160 | N/A | 160.0[1]<br>167.0[2] | 160.0 |
| Abut 1 | 16" CIDH | 183.0 | 176.89 | 190 | N/A | 143.0[1]<br>151.0[2] | 140.0 |
| Abut 2 | 16" CIDH | 182.5 | 176.89 | 190 | N/A | 144.0[1]<br>151.0[2] | 144.0 |
| Back Wall 2 | 16" CIDH | 203.3 | 192.39 | 150 | N/A | 163.0[1]<br>167.0[2] | 163.0 |

Notes: Design Tip Elevation is controlled by the following demands.
(1) Nominal Resistance in Compression
(2) Lateral Resistance – Fixed Head Condition (0.25 inch lateral deflection at top of pile)

**Settlement at Approach Fills**

Fills can be placed in accordance with Section 19-6 of the Standard Specifications. We estimate the post fill placement settlement to be on the order of ½ inch at the back wall locations. Based on the computations, OGDS1 recommends a waiting period of 30 days prior to installing the back wall piles. No waiting period is recommended for installing the abutment piles. However, the actual settlement periods should be determined by the structure representative based on settlement data in the field and the above waiting periods be adjusted accordingly.

## LATERAL EARTH PRESSURE ON SHORING

The proposed construction may require temporary or permanent shoring. We recommend using the following lateral earth pressures along with the procedures given in Section 5 of Bridge Design Specifications, for the design of any needed temporary or permanent shoring.

Level ground - 35 pcf.

1:2 (V: H) sloping grade – 60 pcf

## TEMPORARY EXCAVATIONS

The excavation for proposed construction could be performed either by laying back at suitable slope ratios or supported with shoring. Based on the subsurface materials observed in our investigation we recommend that all temporary excavations should be sloped back at ratios no steeper than 1:1. If any signs of distress are observed in the cut slopes, they should be brought to the attention of the geotechnical engineer immediately for appropriate remedial action.

Alternatively, excavations could be performed with shoring using the lateral earth pressures presented in the preceding section.

## CONSTRUCTION CONSIDERATIONS

1. The proposed piles are designed to carry the imposed loads through both skin friction and end bearing. Therefore, the bottom of the drilled holes should be cleaned out and free of loose materials. The soils at the bottom of the drilled holes should also be firm and competent. The contractor should verify the competency of the drill hole bottoms using appropriate inspection/test method, approved by Caltrans. The subsurface conditions observed at the site indicate that the potential for caving in drilled holes is low. Therefore, the specified pile tip elevations recommended in this report were estimated assuming drilling conditions, rebar cage, and concrete placement will be done under dry conditions. If the contractor proposes to use any procedure other than dry drilling, the above pile tip elevations should be reviewed and modified as needed.

2. Earthwork should be performed in accordance with Sections 6 and 19 of the latest Caltrans Standard Specifications. Soils with an Expansion Index of less than 50 and a Sand Equivalent of at least 20 should be used as replacement fill, in accordance with standard Caltrans requirements.

3. A waiting period of 30 days prior to installing the back wall piles is recommended. No waiting period is recommended for the abutment piles. The actual settlement periods should be determined by the structure representative, based on settlement data in the field.

4. The construction notes in this report shall be included in the Special Provisions.

The recommendations contained in this report are based on specific project information provided by the Office of Structure Design. If any conceptual changes are made during final project design, the Office of Geotechnical Design South 1 should review those changes to determine if these foundation recommendations are still applicable.

If you have any questions or need further information, please contact Gamini Weeratunga at (949) 440-3427 or Shiva Karimi at (213) 620-2146.

Prepared by: Date:9/2/2010

REGISTERED PROFESSIONAL ENGINEER
GAMINI C. WEERATUNGA
No. 2403
Exp. 9/30/10
GEOTECHNICAL
STATE OF CALIFORNIA

GAMINI WEERATUNGA, G.E.
Transportation Engineer
Office of Geotechnical Design-South 1
Branch D

Supervised by: Date: 9/2/2010

P Shiva Karimi

REGISTERED PROFESSIONAL ENGINEER
SHIVA KARIMI
No. GE2651
Exp. 06/30/10
GEOTECHNICAL
STATE OF CALIFORNIA

SHIVA KARIMI, CHIEF, G.E
Senior Transportation Engineer
Office of Geotechnical Design-South 1
Branch D

Reviewed by: Date: 9/2/2010

Joseph S. Pratt

REGISTERED GEOLOGIST
Joseph S. Pratt
No. 2141
Exp. 5/31/11
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

JOSEPH S. PRATT, C.E.G. NO. 2141
Engineering Geologist
Office of Geotechnical Design South 1
Branch D

cc:

| | | |
|---|---|---|
| District Project Manager | James S. Hsu | James_s_Hsu@dot.ca.gov |
| GS Corporate | Mark Willian | Mark_Willian@dot.ca.gov |
| District Construction R.E. Pending File | | TBD |
| Structure Construction R.E. Pending File | | RE_Pending_File@dot.ca.gov |
| DES Office Engineer, Office of PS&E | Efren Ancheta | Efren_Ancheta@dot.ca.gov |
| District Materials Engineer | Kirsten Stahl | Kirsten_Stahl@dot.ca.gov |

## REFERENCES

SDC, 2006, Caltrans Seismic Design Criteria, June 2006, Version 1.4.

Caltrans, 2005, "Foundation Recommendations- Bridge Structure Widening (Northbound and Southbound), Olympic Boulevard Undercrossing (Bridge Number 53-1044)", dated January 31, 2005.

Davis, J.F.,1999, Seismic Hazard Zones Map of the Los Angeles 7.5 minute Quadrangle, Los Angeles County, California, Dept. of Conservation, Calif. Div. Mines and Geology (California Geological Survey), Scale 1:24,000, March 25, 1999.

FHWA 1999, "Drilled Shafts: Construction Procedures and Design Methods" Report No. FHWA-IF-99-025, August 1999

State of California
**DEPARTMENT OF TRANSPORTATION**

Business, Transportation and Housing Agency

# M e m o r a n d u m

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**To:** MS. CHUNG-YUAN WEN
Office of Bridge Design South 2
Bridge Design Branch 21
21073 Pathfinder Road, Suite 20

**Date:** August 31, 2010

**File**: 07-LA-710-PM 25.21
07-202111
Floral Dr U.C. (Widen)
Bridge No. 53-1155

**From:** **DEPARTMENT OF TRANSPORTATION**
**Division of Engineering Services**
Geotechnical Services
Office of Geotechnical Design – South 1
Branch D

**Subject:** Foundation Report for Floral Drive Undercrossing Bridge Widening, Interstate 710

## INTRODUCTION

This Foundation Report (FR) was prepared in response to your request, dated May 21, 2009, for foundation recommendations for the proposed widening of Floral Drive Undercrossing at Interstate 710 (I-710), in the City of Los Angeles. The proposed widening will consist of extending the bridge containing northbound lanes of I-710 outwards, as described below.

## SCOPE OF WORK

The scope of work for the proposed project was determined by the Office of Geotechnical Design South 1 (OGDS1), Branch D based on the information provided by the Office of Bridge Design South 2 (OBDS2) and consisted of the following.

1. Review of previous geotechnical reports and as-built plans,
2. Conducting a geotechnical exploration and performing laboratory testing,
3. Evaluating the site seismicity and performing a seismic hazard analysis, and
4. Geotechnical analysis and preparation of this report.

## PROJECT DESCRIPTION

The current Floral Drive Undercrossing is a single-span bridge containing the northbound (NB) and southbound (SB) lanes of I-710. Freeway at the site is located on embankment built on the relatively flat grade of the adjacent area. The District 07 Layout (sheet L-20, plotted July 12, 2010) indicates that the elevation of the NB lanes of I-710 at the location of the bridge ranges approximately from +327 to +333 feet based on the current NAVD88 datum. The roadway surface elevation of Floral Drive, at the undercrossing location, ranges between +302 to +309 feet (left and right side, respectively).

OBDS2 indicated that the proposed project will consist of widening the structure containing NB lanes by 3 feet 11.5 inches to 4 feet 9 inches, in order to provide a 10-foot wide shoulder. OBDS2 also indicated that the proposed widening should be supported on vertical, 16-inch diameter, cast-in-drilled-hole (CIDH) piles to prevent imposing loads on a water line traversing within the zone of widening.

As a part of the proposed widening 2 wing walls would be constructed at abutments. The wing walls will consist of Caltrans Standard Type 1 walls founded on spread footings.

All elevations provided within this report and proposed project plans are based on NAVD88 datum.

## FIELD INVESTIGATION AND TESTING PROGRAM

### Previous Geotechnical Explorations

Geotechnical investigations at the site have been conducted in 1957 and 2004 for the construction of the original bridge in 1960 and the 1967 and 2005 widenings, respectively. The Log of Test Borings (LOTB), dated December 22, 1958, indicate that in the original investigation, a 2.25 inch diameter penetration boring and a 3 inch diameter rotary wash boring were excavated to a maximum depth of about 26 feet, below surface elevation of +307.5 feet, which appears to be the surface grade of the site at that time. The approximate adjusted elevations to the current datum show an estimated surface elevation of +309.5 feet and a bottom of boring elevation of +283.5 feet.

During the 2004 Caltrans soil investigation, two hollow stem auger borings, one at the inside edge of each abutment in the median area, were excavated to a maximum depth of 81 feet below freeway grade. A number of geotechnical laboratory tests were also performed during the 2004 investigation to determine the relevant engineering properties of the subsurface materials.

### Current Field Exploration Program

Two borings were drilled on October 26 and 27, 2009 on the outside lane of NB I-710 Freeway near Abutments 1 and 2. The borings were excavated with hollow-stem augers using a truck-mounted Mobile B-80 drill rig. Both borings were drilled to a depth of 76.5 feet below the ground surface (BGS) at the freeway grade. The locations and elevations of the borings are summarized in the Table 1 below.

**Table 1 – Summary of Boring Locations and Elevations**

| Boring | General Location | Station | Offset (ft) | Elevation, (ft) |
|---|---|---|---|---|
| A-09-057 | Abutment 1, Lane 4 of N/B freeway | 1330+77.2 | 83.9Rt | 327.1 |
| A-09-058 | Abutment 2, Lane 4 of N/B freeway | 1332+40.9 | 81.8Rt | 328.2 |

Note: Stationing and offsets are based on the Centerline I-710 and the elevations are based on NAVD88.

A representative of Geotechnical Design South 1, Branch D logged and sampled the borings using the Standard Penetration Test (SPT) and modified California Drive sampler having internal diameters of 1.4 and 2.0 inches, respectively. In general, samples were obtained at 5 foot intervals. Both SPT and modified California samplers were driven for 1.5 feet of penetration using a 140-lb, manually operated safety hammer, falling for 30 inches. The efficiency of the hammer has been estimated to be 59 percent by the Foundation Testing Branch of Caltrans. In addition to the drive samples, representative bulk samples were collected from borings. Following drilling, sampling, and logging, the borings were backfilled with soil cuttings and capped with cement slurry.

LOTB's prepared by the Office of Geotechnical Support will be submitted to the Office of Structure Design following completion.

## LABORATORY TESTING PROGRAM

Soil samples, representative of the site subsurface conditions, were selected for testing at the Department's Transportation Laboratory in Sacramento. Tests performed for the project consisted of moisture/density, Atterberg limits, gradation, consolidation, direct shear, consolidated-undrained triaxial compression and corrosivity. All laboratory tests were performed in accordance with current ASTM and Caltrans Test Methods. A summary of the laboratory tests performed for this project is presented in Table 2.

**Table 2 – Summary of Laboratory Tests**

| Test | Standard | No. of Tests Performed |
|---|---|---|
| Moisture Content/Unit Weight | CTM 226 | 3 |
| Mechanical Analysis | CTM 202, 203 | 3 |
| Atterberg Limits | CTM 204 | 5 |
| Corrosion | CTM 417, 422, 532 | 2 |
| Consolidated Undrained Triaxial Shear | ASTM D4767 | 2 |
| Direct Shear | ASTM D3080 | 1 |

## SITE GEOLOGY AND SUBSURFACE CONDITIONS

### Regional Geology

The site is located in the Peninsular Ranges Province of the Los Angeles Basin and is bounded to the northwest/north by the Santa Monica Mountains, to the southwest/south by the Newport-Inglewood Fault Zone, to the northeast/east by the Whittier fault and Puente Hills, and to the southeast by the Santa Ana Mountains and San Joaquin Hills. A thick Cenozoic sedimentary section underlies the site.

### Site Subsurface Conditions

The site subsurface conditions presented in this report were interpreted primarily from the information obtained during the current subsurface exploration. The information obtained from 2004 subsurface exploration was also reviewed for any supplemental information. A brief description of the subsurface conditions interpreted based on the above information is provided below.

The subsurface soils at both abutment locations consist of compacted fill placed for the construction of freeway embankment/bridge approach and underlying alluvial soils. The fill materials underlying the structural section, from an elevation of about +327 to +328 feet (above NAVD 88 datum) down to approximate elevation +307 or +308 feet, consist mainly of interlayered stiff to very stiff, lean clay with sand or sandy clay and medium dense, silty sand or clayey sand. Although, the fill-alluvium contact could not be established with certainty during the current investigation, based on the LOTB of the 1957 investigation, OGDS1 believes that the fill-alluvium contact for the proposed right side widening ranges between approximately +307 to +308 feet elevation. From an elevation of about +308 feet down to about +280 feet, the older Quaternary alluvium consists predominantly of very dense, moist, clayey sand and silty sand with varying amounts of gravel. From an elevation of about +280 feet down to about +265 feet, the

older alluvium consists of medium stiff to very stiff, silt and sandy silt. From elevation +265 feet down to about +250 feet, which corresponds to the maximum depth (76.5 feet) of exploration, subsurface alluvial material consists of very dense, clayey and silty sand and very stiff, sandy clay and sandy silt.

### Groundwater

The current 2009 and previous subsurface explorations indicate that groundwater was not encountered in the subsurface zone investigated. Based on current field and previous explorations, groundwater is considered to be below at least elevation +250.5 feet, which is about 76 to 81 feet below 710 Freeway grade.

## CORROSION EVALUATION

Laboratory corrosion tests conducted on selected samples (Table 3 below) indicate that site subsurface materials contain soils that are corrosive to metal and reinforced concrete, as per September 2003 Corrosion Guidelines.

**Table 3 – Soil Corrosion Test Summary for Floral Drive UC Right Side Widen**

| Boring No./ LA Lab No. or SIC No. | Depth Interval (ft) | Elevation Range (ft) | Minimum Resistivity (ohm-cm) | pH * | Chloride Content (PPM) | Sulfate Content (PPM) |
|---|---|---|---|---|---|---|
| A-09-058 C747752 | 10 to 15 | +318.2 to +313.2 | 564 | 6.2 | 10 | 3183 |
| A-09-057 C747751 | 50 to 51 | +277.0 to +276.0 | 2232 | 7.2 | NA | NA |

Notes: 1. NA = not applicable
2. Caltrans currently considers a site to be corrosive to foundation elements if one or more of the following conditions exist: Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm, or the pH is 5.5 or less. *It is the practice of Caltrans Corrosion Technology Section (with the exception of MSE Walls) if the minimum resistivity of the sample is greater than 1000 ohm-cm and the pH is greater than 5.5, the sample is considered to be noncorrosive.

## SEISMIC RECOMMENDATIONS

### Faulting and Seismicity

The site is located at about 2.5 miles (4.0 km) and 4.3 miles (7.0 km) from the rupture surfaces of Upper Elysian Park Blind Thrust (UEPBT) and Puente Hills Blind Thrust Fault (PHBT), respectively, which are the two nearest active faults to the site. The moment magnitudes of Caltrans assigned Maximum Credible Earthquakes (MCE) of UEPBT and PHBT are 7.0 and 7.25, respectively. Based on the ground motion attenuation relationships by Sadigh et al (1997), OGDS1 estimates that the MCE associated with UEPBT could generate a design Peak Bedrock Acceleration (PBA) and Peak Ground Acceleration (PGA) of 0.7g and 0.58g, respectively, at the

site. Similarly, OGDS1 estimates that the MCE associated with PHBT could generate a median or design PBA and PGA of 0.6g and 0.51g, respectively, at the site.

Based on the above estimates, OGDS1 recommends using an MCE of 7.0 together with a PBA of 0.7g and a PGA of 0.58g for the seismic design.

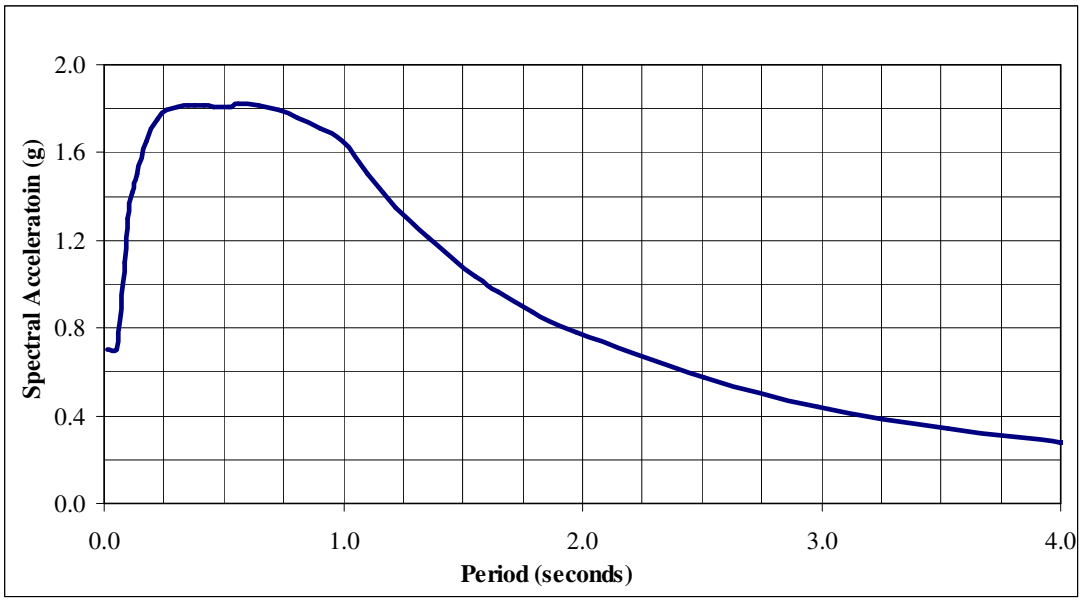
**Potential Seismic Hazards**

Due to the presence of dense to very dense soils at abutment locations and the absence of groundwater within the zone of influence, the potential for the occurrence of liquefaction at the site is considered remote. Consequently, the potential for seismically induced impacts such as settlement and lateral deformation of subsurface materials is remote.

The project site is not located within any California Geological Survey (CGS) designated Earthquake Fault Zone (EFZ) or directly underlain by any fault considered active for bridge design (Davis 1999). Therefore, the possibility of surface fault rupture hazard at the site is considered low.

**Design Ground Motion**

Subsurface investigations indicate that the soil profile at the site can be categorized as Type D based on the Caltrans Seismic Design Criteria, (SDC, 2006) for the determination of design ground motion. Due to the close proximity of the site to the causative fault (2.5 miles), the Standard Acceleration Response Spectrum (ARS) was modified for the near source effects per Section 6.1.2 of SDC, 2006. The modified ARS curve that is recommended for the proposed structure is presented in the figure below.



## AS-BUILT FOUNDATION DATA

As-built drawings indicate that the original bridge abutments on the NB and SB sides are supported on 8-foot and 10-foot wide spread footings, respectively. For the 1967 widening, these abutments were extended outwards on shallow footings having the same width as the original footings. The elevations of the abutment footing bottoms range from approximately +297 to +306 feet, when adjusted to NAVD88 datum. Allowable bearing pressure is 3 TSF (tons per square foot) for the 1960 original bridge and 1967 widening spread footings (F. Gillenwaters and R. Reynolds, Foundation Review, June 30, 1958, and the As Built General Plan for the widening, corrections, July 15, 1966, Contract No. 033124). The Left side of the 1967 widening shows wingwalls are supported on 45 ton design load CIDH piles (16 inch diameter) with pile tip elevation at approximately +286 feet (adjusted to NAVD88 datum).

The 2005 median widening is supported on 24-inch diameter CIDH piles, primarily to prevent the imposition of bridge loads on a subsurface storm drain traversing along the freeway median. The piles at Abutments 1 and 2 were designed for Service Loads of 137 and 115 Kips, respectively.

**Table 4 - As-Built Foundation Data for Floral Drive UC median widening (2005)**

| Location | Pile Type/Diam. | Design Service Load (kips) | Approximate Cutoff Elevation (feet) | Approximate Tip Elevation (feet) |
|---|---|---|---|---|
| Abutment 1 | CIDH/24 inch | 137 | 297 | 256 |
| Abutment 2 | CIDH/24 inch | 115 | 300 | 259 |

## FOUNDATION RECOMMENDATIONS

Foundation recommendations presented in this section were made based on interpreted subsurface conditions, proposed pile types and design loads provided by OBDS2 on March 29, 2010. The pile types, the design loads and other relevant information used in the foundation analysis are presented in the tables below. Vertical, 16 inch diameter, cast-in-drilled-hole (CIDH) piles are recommended to support the proposed Right side widening.

**Table 5 - Foundation Data for Floral Drive UC (Right Side Widen)**

| Support No. | Design Method | Pile Type/Design Load (kips) | Finish Grade Elevation (ft) | Cut-off Elevation (ft) | Pile Cap Size (ft) | | Permissible Movement under Service Load (in) | | Number of Piles Per Support |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | B | L | Vert. | Horizon. | |
| Abut 1 | WSD | 16-inch CIDH/ 140 Kips | 308.89 | 305.14 | 7.09-7.69 | 8.29 | 1” | 0.25” | 4 |
| Abut 2 | WSD | 16-inch CIDH/ 90 Kips | 309.89 | 306.14 | 7.87-9.08 | 7.06 | 1” | 0.25” | 4 |

**Table 6 - LRFD Service Limit State I Load Data for Floral Drive UC (Right Side Widen)**

| Support No. | Vertical Load Per Pile (Kips) | | Lateral Load per Pile (Kips) |
|---|---|---|---|
| | Total Load | Permanent Load | |
| Abut1 | 115 | 53 | 15.5 |
| Abut 2 | 88 | 53 | 14 |

Pile lengths needed to resist the above design loads were computed based on the interpreted subsurface conditions and the design strength parameters obtained based on laboratory tests. The analysis was conducted based on the assumption that proposed piles would derive their geotechnical capacities through both skin friction (approximately 85%) and end bearing (approximately 15%). Therefore, it is imperative that the pile construction methods produce clean drilled hole bottoms that are capable of providing the required end bearing resistance. Due to the potential for caving in the drilled holes, stabilization of drilled holes may be required at both abutment locations. However, any stabilization method used should not compromise the end bearing of the piles.

The geotechnical capacities of the piles were evaluated based on methods recommended in FHWA Drilled Shaft Manual (August 1999), NAVFAC Design Manual 7.02 (Foundations and Earth Structures, September 1986), L-Pile Program (Ensoft, Version 5). The computed geotechnical capacities and the tip elevations are presented below.

**Table 7 - Pile Data Table for Floral Drive UC (Right Side Widen), Br. No. 53-1155**

| Location | Pile Type | Finished Grade Elevation (ft) | Cut-off Elevation (ft) | Nominal Resistance (kips) | | Design Tip Elevations (ft) | Specified Tip Elevation (ft) |
|---|---|---|---|---|---|---|---|
| | | | | Compression | Tension | | |
| Abut 1 | 16" CIDH | 308.89 | 305.14 | 230 | N/A | 259.0[1]<br>289.0[2]<br>284.0[3] | 259.0 |
| Abut 2 | 16" CIDH | 309.89 | 306.14 | 176 | N/A | 265.0[1]<br>290.0[2]<br>285.0[3] | 265.0 |

Notes: Design Tip Elevation is controlled by the following demands.

(1) Nominal Resistance in Compression
(2) Nominal Lateral Resistance – Fixed Head Condition (0.25 inch lateral deflection at top of pile)
(3) Nominal Lateral Resistance – Free Head Condition (0.25 inch lateral deflection at top of pile)

**Settlement at Approach Fills**

We estimate the post construction settlement of fills, placed in accordance with Section 19-6 of Caltrans Standard Specifications, and site soils to be negligible. As such, no waiting period is necessary for installing abutment piles.

**Wing Walls**

Caltrans Standard Type 1 walls that have been proposed for wing walls can be founded on spread footings. Design drawing on Abutment Layout, dated April 19, 2010 indicates that the proposed wall would be constructed approximately 2 feet out from the existing wing walls and therefore would not retain significant backfill. Considering the relatively low foundation loads of the proposed wall, OGDS1 considers that the subsurface soils at the subgrade elevations are capable of providing the required bearing capacities and therefore no remedial grading with overexcavation is recommended.

## LATERAL EARTH PRESSURE ON ABUTMENT WALLS AND SHORING

We recommend that the lateral earth pressure on abutment walls be evaluated using an equivalent fluid weight of 55 pcf.

The proposed construction may require temporary or permanent shoring. We recommend using the following lateral earth pressures along with the procedures given in Section 5 of Bridge Design Specifications, for the design of any needed temporary or permanent shoring.

Level ground - 35 pcf.
1:2 (V: H) sloping grade – 60 pcf
1:1 ½ (V: H) sloping grade – 95 pcf

## TEMPORARY EXCAVATIONS

The excavation for proposed construction could be either laid back at suitable slope ratios or supported by shoring. Based on the subsurface materials observed in our investigation, OGDS1 recommends that all temporary excavations be sloped back at ratios no steeper than 1:1. If any signs of distress are observed in the cut slopes, they should be brought to the attention of the geotechnical engineer immediately for appropriate remedial action.

Alternatively, excavations could be performed with shoring using the lateral earth pressures presented in the preceding section.

## CONSTRUCTION CONSIDERATIONS

1. Subsurface materials at elevations below the pile caps of both abutments consist predominantly of sandy soils with fine to coarse gravel. Although, the sandy soils contain some fine-grained materials, there could be a potential for caving. Therefore, OGDS1 recommends that the contractor consider the possibility of using drilled hole stabilization methods. However, any stabilization method used should not compromise the end bearing capacities of the piles.

2. The proposed piles are designed to carry the imposed loads through skin friction and end bearing. Therefore, the bottom of the drilled holes should be cleaned out and free of loose materials. The soils at the bottom of the drilled holes should also be firm and competent. The contractor should verify the competency of the drilled hole bottom using appropriate inspection/test method, approved by Caltrans.

3. If steel casings are used, the impact of casing installation/retrieval on pile capacities may be neglected. The above consideration takes into account that casings will be installed in accordance with the current Caltrans Standard Specifications (Caltrans 2006) and Special Provisions.

4. Earthwork should be performed in accordance with Sections 6 and 19 of the current Caltrans Standard Specifications. Soils with an Expansion Index of less than 50 and a Sand Equivalent of at least 20 should be used as replacement fill, in accordance with standard Caltrans requirements.

5. Efforts should be made to minimize effects of construction work on adjacent structures.

6. The construction notes in this report shall be included in the Special Provisions.

The recommendations contained in this report are based on specific project information that has been provided by the Office of Structure Design. If any conceptual changes are made during final project design, the office of Geotechnical Design South should review those changes to determine if these foundation recommendations are still applicable.

If you have any questions or need further information, please contact Gamini Weeratunga at (949) 440-3427 or Shiva Karimi at (213) 620-2146.

Prepared by: Date: 8/31/2010

GAMINI WEERATUNGA, G.E.
Transportation Engineer
Office of Geotechnical Design South 1
Branch D

Supervised by: Date: 8/31/2010

SHIVA KARIMI, CHIEF, G.E.
Senior Transportation Eng.
Office of Geotechnical Design
Branch D

Reviewed by: Date:8/31/2010

JOSEPH S. PRATT, C.E.G. NO. 2141
Engineering Geologist
Office of Geotechnical Design South 1
Branch D

cc:

| | | |
|---|---|---|
| District Project Manager | James S. Hsu | James_s_Hsu@dot.ca.gov |
| GS Corporate | Mark Willian | Mark_Willian@dot.ca.gov |
| District Construction R.E. Pending File | | TBD |
| Structure Construction R.E. Pending File | | RE_Pending_File@dot.ca.gov |
| DES Office Engineer, Office of PS&E | Efren Ancheta | Efren_Ancheta@dot.ca.gov |
| District Materials Engineer | Kirsten Stahl | Kirsten_Stahl@dot.ca.gov |

**REFERENCES**

SDC, 2006, Caltrans Seismic Design Criteria, June 2006, Version 1.4.

Caltrans, 2005, "Foundation Recommendations- Bridge Structure Widening (Northbound and Southbound), Floral Drive Undercrossing (Bridge Number 53-1155)", dated January 31, 2005.

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**State of California** **Business, Transportation and Housing Agency**

# M e m o r a n d u m

***Flex your power!***
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**To:** MS. CHUNG-YUAN WEN
Chief, Bridge Design Branch 21
Office of Bridge Design South 2,
21073 Pathfinder Rd., Suite 200
Diamond Bar, CA 91765

Attention: Ms. Mina Pezeshpour

**Date:** February 28, 2011

**File**: 07-LA-710-PM 23.99/24.14
07-202111
Retaining Wall No. 1271
Br. No. 53E0158

**From:** **DEPARTMENT OF TRANSPORTATION**
**DIVISION OF ENGINEERING SERVICES**
**Geotechnical Services**
**Office of Geotechnical Design – South 1**
**Branch D**

**Subject:** Foundation Report for Retaining No. Wall 1271, Interstate 710

## INTRODUCTION

This report was prepared by the Office of Geotechnical Design South 1 (OGDS1) in response to the Office of Bridge Design South 2's (OBDS2's) request dated August 5, 2009, for a Foundation Report for proposed Retaining Wall No. 1271 (RW 1271) as part of the Interstate 710 (I-710) Northbound (N/B) and Southbound (S/B) Widening project.

## SCOPE OF WORK

The recommendations contained in this report are based on the subsurface information gathered during the recent field investigation performed from October, 2009 through February 2011. Tasks completed by the OGDS1, Branch D included the following:

1. Review of the regional geology and seismicity,
2. Review of pertinent information from previous geology/geotechnical reports and As-Built plans in the project area to evaluate the subsurface information,
3. Drilling, logging, and sampling of four – 8 inch diameter hollow stem auger borings and utilizing three – 1 inch diameter hand driven soil tubes with open and closed tip sampling, at the subject wall site to characterize subsurface conditions,
4. Laboratory testing of selected soil samples for mechanical analysis, Atterberg limits, and corrosivity,
5. Geotechnical engineering analysis

## PROJECT DESCRIPTION

Proposed Retaining Wall No. 1271 will be located at and slightly above the existing east shoulder of NB Interstate 710 to Route 60 On-Ramp in the city of East Los Angeles, Los Angeles County. According to the General, Structure, and Foundation Plans, Wall Design Profile, and Layout Plans, provided by the OBDS2 and District 7 Design, revised from August 23, 2010 through

January 4, 2011, the proposed RW 1271 will be a masonry block sound wall on pile cap supported on a soldier pile retaining wall from RW LOL Station 1266+66.52 (81.629 Rt. Sta. 1266+66.52 Centerline Rte. 710) to 1269+05.18 (97.87 Rt. Sta. 1269+09.57 Centerline Rte. 710) for approximately 238.6 ft length. For the remaining wall length (approximately 540.4 ft), cable railing will be placed on pile cap supported on a soldier pile retaining wall from RW LOL Station 1269+05.18 (97.87 Rt. Sta. 1269+09.57 Centerline Rte. 710) to 1274+45.52 (33.46 Rt. Sta. 74+64.36 Stationline RAMP, approximately 118.25 ft. Rt. Sta. 1274+81.29 Centerline Rte. 710). A Type 60D concrete barrier will be placed for the entire wall length in front of the retaining wall at freeway grade. A portion of an existing Caltrans sound wall on concrete barrier (supported on piles) will be removed where the sound wall conflicts with proposed construction of RW 1271 (within a distance of about 5 feet or less from the proposed retaining wall), from RW 1271 LOL Station 1266+66.52 to Station 1269+05.18.

An existing slope [varying from 1.56H:1V (32.6 degrees) and 2H:1V (26.5 degrees)] ascends above proposed RW 1271 LOL. Once RW 1271 is built, the revised slope will be approximately 3H:1V or flatter. An existing City sound wall supported on piles, on the west side of South Burger Avenue at the top of the slope above proposed RW 1271, will remain in place. Proposed RW 1271 LOL will be offset approximately 6.25 to 24 ft horizontally from the existing City sound wall (the outer edges of the walls will be slightly closer). Proposed RW 1271 will extend roughly parallel to the City sound wall for about 561.5 ft (from approximate RW 1271 LOL Sta. 1264+84 to 1274+45.5).

Several utility lines including gasline (under S. Burger Ave.), fiber optics and telephone wire (MC2 Cable, AT&T), waterlines (Calif. Water Service), abandoned 10 inch and live 8 inch VCP sanitary sewer, drainage pipes, and Caltrans irrigation mainline (2 inch PVC) with sprinklers are located in the vicinity of proposed RW 1271. Caltrans power is located near the base of the slope.

**Table 1 – Retaining Wall No. 1271 Data**

| Wall Type | RW LOL Stationing | | Length (ft) | Approx. Sound Wall Design Height H (ft) | Approx. Soldier Pile Design Height H (ft) | Reference Borings/ Soil Tubes |
|---|---|---|---|---|---|---|
| | From | To | | | | |
| Sound wall on Conc. Pile Cap on Soldier Pile w/ Conc. Barrier 60D | 1266+66.522 | 1269+05.18 | 238.58 | 11 to 6 | 5 to 10 | HD-10-225, A-10-220, A-09-104, HD-10-224, A-09-103, A-10-221 HD-10-226 |
| Cable Railing on Conc. Pile Cap on Soldier Pile w/ Conc. Barrier 60D | 1269+05.18 | 1274+45.52 | 540.42 | N/A | 4.9 to 13.2 | |

All elevations provided within this report and proposed project plans are based on NAVD88 datum.

## FIELD INVESTIGATION AND TESTING PROGRAM

The recent site-specific field exploration was performed from October 2009 through February 2011. The field investigation included drilling/sampling four - 8 inch diameter hollow stem auger borings and hand driving three – 1 inch diameter soil tubes with open- and closed-end tips. Standard Penetration Tests (SPT's) were performed at the borings. Blow counts (SPT N values) were recorded at 5 foot intervals during drilling. SPT's were performed in accordance with ASTM Test Method D1586-84 using a standard 1.4 inch I.D. sampler with a 140 lb. hammer dropped 30 inches. Caltrans operated drill rig models included used at the boring locations included a Mobile B-80, CME-75, and CME-85.

District 7 Surveys provided the location and elevation for Borings A-09-104 and A-09-103. Borings A-10-220, A-10-221, and Soil Tubes HD-10-225, HD-10-224, and HD-10-226, were field located by OGDS1 by taping from existing culture and top of boring elevation surveyed utilizing previously surveyed Borings A-09-104 and A-09-103 as temporary bench marks. Boring/Soil Tube information, including exploration number, stationing, offset, boring depth, ground surface elevation, date drilled, and groundwater information are summarized in Table 2.

**Table 2 – Summary of Boring Data**

| Boring/Soil Tube No. | Offset and Centerline Stationing Rte. 710 (ft) | Top of Boring Elev. (ft) | Drilling Depth (ft) | Date Completed | Groundwater Measurement (ft) |
|---|---|---|---|---|---|
| HD-11-225 | 100 Rt. Sta. 1268+72 | 226.6 | 10.8 | 02/23/11 | Dry to Bottom |
| A-10-220 | 80 Rt. Sta. 1268+77 | 218.3 | 46.5 | 12/01/10 | Dry to Bottom |
| A-09-104 | 84.6 Rt. Sta. 1270+01.0 | 220.9 | 40.5 | 10/21/09 | Dry to Bottom |
| HD-11-224 | 118 Rt. Sta. 1272+10 | 236.3 | 6.2 | 02/23/11 | Dry to Bottom |
| A-09-103 | 95.2 Rt. Sta. 1272+32.5 | 226.8 | 41.5 | 10/21/09 | Dry to Bottom |
| A-10-221 | 100 Rt. Sta. 1273+43 | 230.5 | 41.5 | 12/01/10 | Dry to Bottom |
| HD-11-226 | 125 Rt. Sta. 1273+53 | 239.0 | 10.2 | 02/23/11 | Dry to Bottom |

## LABORATORY TESTING PROGRAM

Selected soil samples were sent to the Department's Materials Laboratory in Los Angeles for laboratory testing. All laboratory tests were performed in accordance with ASTM standard procedures and California Test Methods. The summarized laboratory test data are shown in Table 3.

**Table 3 – Summary of Laboratory Tests**

| Test | Standard | No. of Test Performed |
|---|---|---|
| Mechanical Analysis | CTM 202, 203 | 7 |
| Atterberg | CTM 204 | 3 |
| Corrosion | CTM 417, 422, 532 | 4 |

## SITE GEOLOGY AND SUBSURFACE CONDITIONS

### Regional Geology

The subject wall site is located in the Peninsular Ranges Province in the Los Angeles Basin and is bounded to the northwest/north by the Santa Monica Mountains, to the southwest/south by the Newport-Inglewood Fault Zone, to the northeast/east by the Whittier fault and Puente Hills, and to the southeast by the Santa Ana Mountains and San Joaquin Hills. A thick Cenozoic sedimentary section underlies the site.

### Site Subsurface Conditions

The project site is underlain by minor Fill and older Quaternary alluvium (Qoa, Dibblee, 1989, Geologic Map of Los Angeles 7.5 minute Quadrangle). The Pliocene Fernando Formation (Tfsc, nonmarine sandstone and conglomerate and Tfr, marine claystone) most likely underlies the older alluvium at depth.

Based on the information obtained form Borings A-10-220, A-09-104, A-09-103, and A-10-221, and Soil Tubes HD-10-225, HD-10-222, HD-10-224, and HD-10-226, minor sporadic fill material generally consists of loose to very loose silty sand and sandy silt with trace gravel and few brick and asphalt fragments (up to 5 inch length). Underlying alluvial material generally consists of medium dense to very dense silty sand with trace to few gravel, clayey sand, and scattered stiff to very stiff sandy clay and minor sandy silt interbeds.

### Groundwater

Groundwater was not encountered in any of the Borings or Soil Tubes during the recent field investigation from October 2009 through February 2011 for this project. Borings extended down to maximum depths ranging from 46.5 to 41.5 ft (elevations +171.8 ft to +189.0 ft), from south to north along the wall length, and were all dry to bottom of borings. Boring R-09-056 at Whittier Blvd UC (Wdn), Br. No. 53-1150 (about 800 ft south of the subject retaining wall), also showed no groundwater was encountered to at least 100.4 ft depth (dry hole to at least +97.5 ft elevation) with several measurements taken from August 16, 2009 until September 30, 2010.

## SCOUR EVALUATION

There is no possibility of scour at the site due to the distance from the channels.

## CORROSION EVALUATION

Corrosion test results for representative soil samples are presented in Table 4. Test results show soils in this area are considered non-corrosive to reinforced concrete and metal.

**Table 4 – Soil Corrosion Test Summary**

| Boring | Sample Depth (ft) | pH | Minimum Resistivity (Ohm-Cm) | Sulfate Content (ppm) | Chloride Content (ppm) |
|---|---|---|---|---|---|
| A-10-220 | 15.0 – 16.5 | 7.95 | 6700 | N/A | N/A |
| A-10-220 | 30.0 – 31.5 | 7.70 | 5200 | N/A | N/A |
| A-09-104 | 5.0 – 6.5 | 7.79 | 5300 | N/A | N/A |
| A-09-103 | 5.0 – 6.5 | 7.99 | 4000 | N/A | N/A |
| A-10-221 | 20.0 – 21.5 | 7.99 | 6900 | N/A | N/A |

Note: It is the practice of Caltrans Corrosion Technology Section (with the exception of MSE Walls) if the minimum resistivity of the sample is greater than 1000 ohm-cm and the pH is greater than 5.5, the sample is considered to be noncorrosive. For structural elements, the California Department of Transportation considers a site to be corrosive if one or more of the following conditions exist for representative soil and/or water samples taken at the site:
Chloride concentration $\geq$500 ppm, sulfate concentration $\geq$2000 ppm, or the pH is $\leq$5.5. Corrosion mitigation is required if one or more of the 3 conditions noted above exists where structural elements are involved (Caltrans Corrosion Guidelines, September 2003).

## SEISMIC RECOMMENDATIONS

### Faulting and Seismicity

The controlling seismic source for this site is the Puente Hills Blind Thrust (PHBT) fault. Caltrans has assigned a Maximum Credible Earthquake (MCE) of moment magnitude 7.25 to this fault. This is a buried thrust fault with a site-to-rupture surface distance of about 4.0 miles. Based on Sadigh et al (1997) attenuation relationships, the median Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.6g.

### Potential Seismic Hazards

This site is not located within a potentially liquefiable zone as shown on the California Geological Survey Map of Seismic Hazard Zones of the Los Angeles 7.5 Minute Quadrangle (March 25, 1999). Based on Caltrans recent study and lack of shallow groundwater, the possibility of liquefaction is expected to be low to nil.

### Design Ground Motion

For the purpose of selecting a standard SDC (2006) Acceleration Response Spectrum (ARS) curve, the design PBA is taken as 0.6g. The soil profile may be classified as Type D. The design ARS curve was obtained by modeling the standard SDC curve (for PBA = 0.6g, M = 7.25 ± 0.25, and soil profile type D) for near fault effects. The recommended ARS Curve and the coordinates of spectral accelerations are presented in Figure 1 and Table 5, respectively.

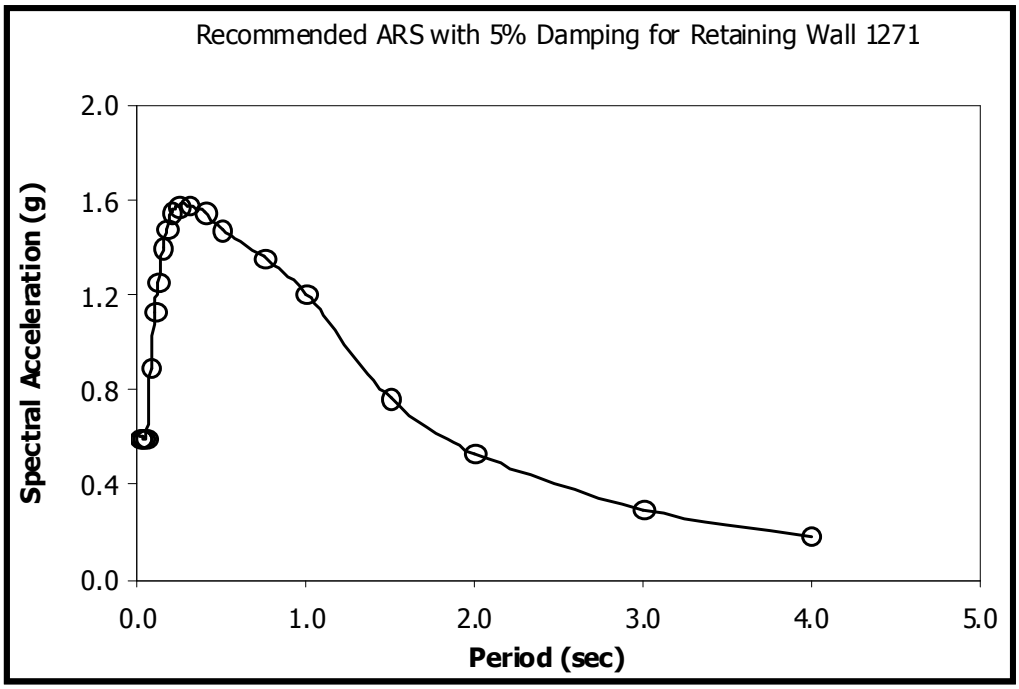


**Figure 1 - Recommended ARS curve for Retaining Wall 1271**

**Table 5 - Recommended Design ARS curve for RW 1271**

| Period (s) | Spectral Acceleration (g) | |
|---|---|---|
| | Standard | **Modified** |
| 0.010 | 0.6003 | 0.6003 |
| 0.020 | 0.6003 | 0.6003 |
| 0.030 | 0.6003 | 0.6003 |
| 0.050 | 0.6003 | 0.6003 |
| 0.075 | 0.8934 | 0.8934 |
| 0.100 | 1.1315 | 1.1315 |
| 0.120 | 1.2590 | 1.2590 |
| 0.150 | 1.4033 | 1.4033 |
| 0.170 | 1.4802 | 1.4802 |
| 0.200 | 1.5496 | 1.5496 |
| 0.240 | 1.5784 | 1.5784 |
| 0.300 | 1.5811 | 1.5811 |
| 0.400 | 1.5484 | 1.5484 |
| 0.500 | 1.4767 | 1.4767 |
| 0.750 | 1.2366 | 1.3602 |
| 1.000 | 1.0062 | 1.2074 |
| 1.500 | 0.6378 | 0.7653 |
| 2.000 | 0.4464 | 0.5357 |
| 3.000 | 0.2471 | 0.2965 |
| 4.000 | 0.1546 | 0.1855 |

**FOUNDATION RECOMMENDATIONS**

The following recommendations are based on Retaining Wall No. 1271 General Plans, Structure Plans, Foundation Plans, Wall Details, and Typical Sections (dated February 8, 2011, and September 21, 2010), and design loads (September 30, 2010) provided by the OBDS2, and results of the recent field investigation.

Based on the project General Plans, Retaining Wall No. 1271 will consist of drilled and structural concrete filled 30 inch diameter soldier piles with HP14x89 and HP14x117 installed for structural support. Based on OBDS2 communications with OGDS1 (September 30, 2010), acceptable maximum deflection at top of soldier piles adjacent to Type 60D concrete barrier (Finished Grade Elevation) is satisfied.

Soldier pile minimum embedment was evaluated using design loads applied at Finished Grade Elevation adjacent to Type 60D concrete barrier (Table 6), and pile size and loads provided by OBDS2. Computer program LPILE plus 5.0 was used for calculating lateral resistances of piles shown in Table 7.

**Table 6 – Soldier Pile Loads**

| Stationing (RW LOL) | | Wall Height (ft) | SW Height (ft) | Vertical Load (kips) | Lateral Earth Pressure (kips) | | Moment (kips-ft) | |
|---|---|---|---|---|---|---|---|---|
| From | To | | | | Earthquake Load | Earth Pressure | Earthquake Load | Earth Pressure |
| 1266+66.52 | 1267+15 | 5 to 6 | 10 to 11 | 33 | 5 | 10 | 100 | 16.5 |
| 1267+15 | 1267+85 | 6 to 8 | 9 to 10 | 36 | 5 | 17 | 96 | 38 |
| 1267+85 | 1269+05.18 | 8 to 10 | 6 to 9 | 36 | 5 | 25 | 68 | 72.9 |
| 1269+05.18 | 1272+70 | 10 to 13 | N/A | 36 | N/A | 25 | N/A | 155.5 |
| 1272+70 | 1273+85 | 6.5 to 10 | N/A | 31 | N/A | 25 | N/A | 73 |
| 1273+85 | 1274+45.52 | 4.9 to 6.5 | N/A | 26 | N/A | 12 | N/A | 21 |

**Table 7 – Retaining Wall No. 1271 Soldier Pile Data**

| Stationing (RW LOL) | | Soldier Pile Dia. (in)/ Reinforcement Type | Finished Grade Range Elev. (ft) | Nominal Resistance in Compression (kips) | Nominal Lateral Resistance (kips) | Nominal Moment Resistance (kips-ft) | Required Embedment Length (ft) |
|---|---|---|---|---|---|---|---|
| From | To | | | | | | |
| 1266+66.52 | 1267+15 | HP 14x89 | 217.45 to 217.52 | 33 | 15 | 116.5 | 22 |
| 1267+15 | 1267+85 | HP 14x89 | 217.52 to 217.65 | 36 | 22 | 134 | 22 |
| 1267+85 | 1269+05.18 | HP 14x117 | 217.65 to 218.62 | 36 | 30 | 140.9 | 22 |
| 1269+05.18 | 1272+70 | HP 14x117 | 218.62 to 227.95 | 36 | 25 | 155.5 | 22 |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| 1272+70 | 1273+85 | HP 14x89 | 227.95 to 232.05 | 31 | 25 | 73 | 19 |
| 1273+85 | 1274+45.52 | HP 14x89 | 232.05 to 234.30 | 26 | 12 | 21 | 19 |

**Global Slope Stability**

Based on subsurface information collected during the 2009/2011 recent field investigation, subsurface material at the proposed wall location is estimated to have a friction angle of 32 degrees, cohesion of 50 psf, and unit weight of 125 pcf. Critical cross sections 15 feet high (extending from the top of the wall up the slope behind the wall) with 2H:1V slope were used for slope stability analysis utilizing computer program SLOPEW. The result yields a factor of safety greater than acceptable values of 1.5 and 1.1 for static and pseudo –static condition, respectively.

## NOTE TO DESIGNER

The recommendations contained in this report are based on specific project information regarding wall location, pile and reinforcement type, finished grade elevation, and loads that have been provided to our office by OBDS2. If any conceptual changes are made during final project design, OGDS1 should review those changes to determine if those foundation recommendations are still applicable.

Recommendations in this report are subject to possible design changes based on the further identification of utility lines at the site or other issues.

## CONSTRUCTION CONSIDERATIONS

1. All earthwork is expected to be carried out by conventional equipment. Fill placed on sloping round shall be properly keyed and benched into existing ground and placed as specified in Section 19-6 of the Caltrans Standard Specifications (May 2006).

2. Based on the soil types encountered during the recent investigation, it is recommended that a slope ratio of 1V:1H or flatter for the temporary back cut slope be considered for construction. If there are constraints due to construction or traffic concerns, a feasible alternative would utilize shoring to accommodate a steeper slope for the excavations.

3. Groundwater is not anticipated during drilling of the pile borings and construction of the soldier piles.

4. The contractor shall be required to clean out the bottom of the soldier pile borings prior to placing the H pile reinforcement and concrete.

5. Concrete placement for construction of the soldier piling shall be completed within the same day that excavation of the drilled hole has been completed.

6. Moderate to minor caving and sloughing should be anticipated during excavation and construction of the soldier piles.

7. Due to space constraints in the area of the proposed overhead sign CIDH pile (Sta. 1268+75 Rte. 710) and the existing City soundwall and proposed soldier piles for Retaining Wall No. 1271, either cement slurry or casing or a combination of the two methods might be considered to prevent caving of adjacent boreholes and assure existing wall stability. Right of Way clearance is also very limited in this area.

If you have any questions or comments, please call Kevin Lai at (213) 620-2344 or Shiva Karimi at (213) 620-2146.

Prepared by: Date: 2/28/11

Kevin Lai
Transportation Engineer
Office of Geotechnical Design South-1
Branch D

Supervised by: Date: 2/28/11

Shiva Karimi, Ph.D., G.E.
Senior Transportation Engineer
Office of Geotechnical
Branch D

Reviewed by: Date: 2/28/11

Joe Pratt, C.E.G. No. 2141
Engineering Geologist
Office of Geotechnical Design South 1
Branch D

cc:

| | | |
|---|---|---|
| District Project Manager | James S. Hsu | James_s_Hsu@dot.ca.gov |
| GS Corporate | Mark Willian | Mark_Willian@dot.ca.gov |
| District Construction R.E. Pending File | | TBD |
| Structure Construction R.E. Pending File | | RE_Pending_File@dot.ca.gov |
| DES Office Engineer, Office of PS&E | Efren Ancheta | Efren_Ancheta@dot.ca.gov |
| District Materials Engineer | Kirsten Stahl | Kirsten_Stahl@dot.ca.gov |

**State of California** **Business, Transportation and Housing Agency**

# M e m o r a n d u m

***Flex your power!***
***Be energy efficient!***

**To:** MS. CHUNG-YUAN WEN
Chief, Bridge Design Branch 21
Office of Bridge Design South 2,
21073 Pathfinder Rd., Suite 200
Diamond Bar, CA 91765

**Date:** September 29, 2010

**File**: 07-LA-710-PM 23.19
EA: 07-202111
NB 710/5 Separation
(Rt. Side Widen)
Bridge No. 53-0785R

Attention: Mr. Wei-Kung Hsia

**From: DEPARTMENT OF TRANSPORTATION**
**DIVISION OF ENGINEERING SERVICES**
**Geotechnical Services**
**Office of Geotechnical Design - South 1**
**Branch D**

**Subject:** Foundation Report for NB 710/5 Separation Widening, Bridge No. 53-0785R

## INTRODUCTION

This Foundation Report was prepared in response to the Office of Bridge Design South 2's (OGDS2's) request dated May 21, 2009, for the proposed right side widening (outside widening) of the Northbound 710/5 Separation (Br. No. 53-0785R) above Interstate 5 (I-5).

## SCOPE OF WORK

The recommendations contained in this report are based on the subsurface information gathered during the recent field investigation performed in August and October 2009, Structure Plans (received September 29 and 30, 2009, through September 21, 2010) and updated design information (received through September 2010). Tasks completed by the Office of Geotechnical Design South 1 (OGDS1), Branch D include the following:

1. Review of the regional geology and seismicity,
2. Review of pertinent information from previous geology/geotechnical reports and As-Built plans in the project area to evaluate the subsurface information,
3. Drilling, logging, and sampling of five rotary wash borings at the subject bridge site to characterize the subsurface conditions,
4. Laboratory testing of selected soil samples for mechanical analysis, corrosivity, direct shear, and unit weight,
5. Geotechnical engineering analysis

## PROJECT DESCRIPTON

The NB 710/5 Separation, straddling the boundary between the cities of Commerce and Los Angeles, was originally built in 1954 with variable width ranging from 28 to 75 ft and right side widened (east side widened) in 1967 with variable width approximately 18 ft or more. In 2002 abutment, hinge, bent cap, and columns were Earthquake retrofit but no pile or foundation retrofit was completed. The NB 710/5 Separation is a continuous eight-span combination of reinforced concrete and cast-in-place/prestressed box girder, with seated abutments and two to three column bents, having a total length of 704.4 ft (measured along the Northbound Stationline or "Conn 2" Line). The Los Angeles River is approximately 1.4 miles southwest of the NB 710/5 Separation Bridge.

Based on the Structure Plans (revised September 20, 2010), provided by Office of Bridge Design South 2, OGDS1 understands that the proposed project will consist of right side widening (outside widening) of the existing bridge by approximately 10 to 25 ft (± varies). Eight spans (CIP/PS box girder and CIP/RC box girder) will be constructed for the proposed right side widening with accompanying nine supports.

All elevations provided within this report and proposed project plans are based on NAVD88 datum with the exception of groundwater well data which is recorded utilizing mean sea level (MSL, NGVD29 datum).

## FIELD INVESTIGATION AND TESTING PROGRAM

The site-specific field exploration was performed between August and October 2009. The field investigation included drilling five rotary wash borings, all 4.5 inch diameter. Standard Penetration Tests (SPT's), and relatively undisturbed sampling (with a modified California split-barrel sampler, 2 inches inner diameter) were performed at the borings. Blow counts (SPT N values) were continuously recorded at 5 foot intervals during drilling. The SPT's were performed in accordance with ASTM Test Method D1586-84 using a standard 1.4 inch I.D. sampler with a 140 lb. hammer dropped 30 inches. Undisturbed tube soil samples were also obtained using the 2 inch I.D. modified California Split Spoon Sampler with 4 inch long brass liners. The liners were all sealed in the field. Caltrans operated drill rig models Mobile B-80 and CME-750 were used at the boring locations.

The District 7 Surveys provided the location and elevation of the borings. Boring information, including exploration number, stationing, offset, ground surface elevation, boring depth, and date drilled are summarized in Table 1 below.

**Table 1 – Summary of Borings**

| Exploration No. | Station (ft) | Offset (ft) | Reference Line | Surface Elev. (ft) | Depth (ft) | Date Drilled |
|---|---|---|---|---|---|---|
| R-09-046 | 1223+58.6 | 54.5 Rt | "Conn 2" Line | +203.4 | 101.5 | 8/12/09 |
| R-09-047 | 1226+11.9 | 66.6 Lt | "Conn 1" Line | +186.1 | 120.3 | 10/20/09 |
| R-09-048 | 1229+10.1 | 74.0 Rt | "Conn 2" Line | +178.0 | 101.5 | 8/12/09 |
| R-09-049 | 1229+40.0 | 61.1 Lt | "Conn 1" Line | +169.1 | 120.7 | 8/19/09 |
| R-09-050 | 1231+74.9 | 55.9 Rt | "Conn 2" Line | +202.0 | 101.5 | 8/11/09 |

## LABORATORY TESTING PROGRAM

Selected soil samples were sent to the Department's Transportation Laboratory in Sacramento and Los Angeles for laboratory testing. All laboratory tests were performed in accordance with ASTM standard procedures and California Test Methods. The summarized laboratory tests data are shown in Table 2, below:

**Table 2 – Summary of Laboratory Tests**

| Test | Standard | No. of Test Performed |
|---|---|---|
| Mechanical Analysis | CTM 202, 203 | 26 |
| Atterberg | CTM 204 | 21 |
| Corrosion | CTM 417, 422, 532 | 18 |
| Direct Shear | ASTM D 3080 | 2 |

## SITE GEOLOGY AND SUBSURFACE CONDITIONS

### Regional Geology

The site is located in the Peninsular Ranges Province in the Los Angeles Basin and is bounded to the northwest/north by the Santa Monica Mountains, to the southwest/south by the Newport-Inglewood Fault Zone, to the northeast/east by the Whittier fault and Puente Hills, and to the southeast by the Santa Ana Mountains and San Joaquin Hills. A thick Cenozoic sedimentary section underlies the site.

### Site Subsurface Conditions

At the NB 710/5 Separation (Widen) site, embankment fill is approximately 24 to 29 ft thick at Abutment 1 and 22 ft thick at Abutment 9. Embankment fill is underlain by Holocene and older Quaternary alluvium (Qa and Qoa, Dibblee, 1989, Geologic Map of Los Angeles 7.5 minute Quadrangle). The Pliocene Fernando Formation (Tfsc, nonmarine sandstone and conglomerate and Tfr, marine claystone) most likely underlies the older alluvium at depth.

Top of embankment ranges between approximately +203 to +201 ft elevations at Abutments 1 and 9, respectively. The bottom of the embankment fill/top of alluvium extends down to between approximate elevations +179 to +160 ft (deepest part of cut for I-5 Freeway). Embankment fill is composed of loose to dense, silty sand, interlayered with clayey sand, sandy silt, and stiff to hard, sandy lean clay. Fill also contains trace to few gravel and sporadic trace to few asphalt and concrete fragments (6 to 8 in maximum length). Underlying undifferentiated Holocene/older Quaternary alluvium can be divided into 4 units in descending order. The upper alluvial unit, ranging from approximate elevations +179 to +160 ft down to +151 to 146 ft, and is composed of medium dense to very dense (minor loose), sandy silt to silt, silty sand, and clayey sand interbedded with stiff to hard, sandy lean clay. The upper alluvial unit contains sporadic trace gravel and trace calcite filaments and

nodules. The upper middle alluvial unit, ranging from approximate elevations +151 to +146 ft down to +134 to +130 ft, consists of generally stiff to hard, sandy lean clay and lean clay interbedded with minor medium dense, sandy silt. The lower middle alluvial unit, ranging from approximate elevations +134 to +130 ft down to +100 to +89 ft, consists of dense to very dense, sand and silty sand, clayey sand, and sandy silt interbedded with stiff to hard, sandy lean clay, lean clay, and silt. The lower middle alluvial unit also contains sporadic few gravel and shows iron oxide staining. The lower alluvial unit, ranging from approximate elevation +100 to +89 ft down to at least +48.4 ft, consists of dense to very dense, silty sand with sporadic gravel, sandy silt, clayey sand, and clayey gravel with sand interbedded with very stiff to hard, sandy lean clay. The lower alluvial unit contains sporadic few to little gravel within the sand and clay beds. The maximum boring depth extended down to 120.7 ft (elevation +48.4 ft) within Boring R-09-049. Refer to the Log of Test Borings for further details, which will be sent as soon as completed.

**Groundwater**

Static groundwater was measured on February 25, 2010 at depth 109.9 ft (elevation +59.2 ft, NAVD88 datum). Groundwater was periodically measured within piezometer Boring R-09-049 during the recent field exploration as shown in Table 3 below.

**Table 3 – Recent Groundwater Monitoring**

| Boring No. | Ground Surface Elevation (ft) | Depth to Water Below Surface (ft) | Groundwater Surface Elevation (ft) | Date Measured |
|---|---|---|---|---|
| **R-09-049** | **+169.1** | 109.78 | +59.3 | 11/16/2009 |
| | | **109.90** | **+59.2** | **02/25/2010** |
| | | 110.90 | +58.2 | 09/21/2010 |

The As-Built LOTB's for the original bridge and widening indicate that groundwater was not encountered at the site during the 1953 and 1965 investigations. The deepest 1 inch diameter soil tube extended down to approximate current adjusted elevation +133 ft (+131 ft MSL) with the deepest cone penetration test down to approximate current adjusted elevation +124 ft (+122 ft MSL).

Table 4 shows historical groundwater elevations in the vicinity of the bridge obtained from the monitoring well records of Los Angeles County Department of Public Works (LACDPW) and California Department of Water Resources (DWR). Four (4) wells located within distances ranging from about 0.27 to 0.43 miles from the site indicate groundwater elevations (during last 70 years) lower than approximate current adjusted elevation +89.0 ft (+86.8 feet, MSL, depths greater than 76.5 feet). The chart below shows recorded elevations based on NGVD29 datum (mean sea level, MSL) which requires about a 2 foot shift (add) to reach the current NAVD88 datum elevations.

**Table 4 - Historic Groundwater Records near NB 710/5 Separation Bridge**

| LACDPW Well No. | Well Distance and Direction from SB 710/5 Separation | Date | Ground Surface Elevation (ft) | Historic High Groundwater Elevation (ft) | Date | Historic Low Groundwater Elevation (ft) | Date | Recent Ground Water Elevation (ft) |
|---|---|---|---|---|---|---|---|---|
| 2828F | 0.28 mile NW | 4/26/95 | +185.8 | +36.8 (149.0) | 7/17/53 | -102.2 (288.0) | 11/30/07 | +6.8 (179.0) |
| 2828C | 0.27 mile SW | 11/30/07 | +168.8 | +69.8 (99.0) | 8/31/57 | -81.2 (250.0) | 11/30/07 | +69.8 (99.0) |
| 2838 | 0.31 mile SE | 2/30/40 | +163.3 | +86.8 (76.5) | 1/31/57 | -102.2 (265.5) | 9/30/05 | +14.8 (148.5) |
| 2838A | 0.43 mile NE | 5/15/94 | +174.2 | +66.2 (108.0) | 8/31/57 | -82.8 (257.0) | 11/30/07 | -20.8 (195.0) |

*Values in parentheses indicate depth below the ground surface.

Based predominantly on the recent 2009/2010 geotechnical investigation, As Built LOTB's, and to a lesser extent the above historic groundwater review (which is not specifically on site), OGDS1 considers groundwater level for bridge design to be at elevation +59.2 ft (based on current NAVD88 datum). The anomalous historic high groundwater elevation was measured within Well No. 2838 during the winter of 1940 and is much lower within recent measurements (elevation +14.8 MSL, measured September 2005). Groundwater elevation within Well No. 2838A has also dropped significantly during recent November 2007 measurements. Within Well No. 2828C, groundwater elevation is high even within recent November 2007 measurements, but the well is 0.27 miles southwest of the subject bridge and site specific measurements at the bridge are considered more appropriate for design groundwater. With the exception of Well No. 2828C, all other groundwater measurements are lower within recent measurements and well below design groundwater.

## SCOUR EVALUATION

There is no possibility of scour at the site due to substantial distance from the Los Angeles River channel or other significant creeks.

## CORROSION EVALUATION

Corrosion test results for representative soil samples are presented in Table 5 below. The results show this area can be considered as non-corrosive to reinforced concrete and metal.

**Table 5 – Corrosion Test Summary**

| Exploration No. | Sample Depth (ft) | pH | Minimum Resistivity (ohm – cm) | Sulfate Content (PPM) | Chloride Content (PPM) |
|---|---|---|---|---|---|
| R-09-047 | 10.0 – 11.5 | 6.87 | 910 | 200 | 130 |
| R-09-047 | 76.0 – 76.5 | 7.93 | 3700 | N/A | N/A |
| R-09-047 | 85.0 – 86.5 | 8.35 | 2700 | N/A | N/A |
| R-09-048 | 16.5 – 45.0 | 7.43 | 1500 | N/A | N/A |
| R-09-048 | 61.5 – 65.0 | 7.53 | 2100 | N/A | N/A |
| R-09-049 | 5.0 – 6.5 | 8.13 | 1951 | N/A | N/A |
| R-09-049 | 10.0 – 36.5 | 6.81 | 1631 | N/A | N/A |
| R-09-049 | 40.0 – 81.5 | 7.34 | 4408 | N/A | N/A |
| R-09-049 | 81.5 – 120.7 | 7.38 | 5429 | N/A | N/A |
| R-09-050 | 16.5 – 35.0 | 8.18 | 3400 | N/A | N/A |
| R-09-050 | 46.5 – 65.0 | 7.69 | 1600 | N/A | N/A |
| R-09-050 | 5.0 – 6.5 | 8.07 | 2500 | N/A | N/A |
| R-09-050 | 70.4 – 70.7 | 7.82 | 1900 | N/A | N/A |
| R-09-050 | 70.0-86.5 | 8.01 | 7508 | N/A | N/A |

Note: It is the practice of Caltrans Corrosion Technology Section (with the exception of MSE Walls) if the minimum resistivity of the sample is greater than 1000 ohm-cm and the pH is greater than 5.5, the sample is considered to be noncorrosive. For structural elements, the California Department of Transportation considers a site to be corrosive if one or more of the following conditions exist for representative soil and/or water samples taken at the site:
Chloride concentration $\geq$500 ppm, sulfate concentration $\geq$2000 ppm, or the pH is $\leq$5.5. Corrosion mitigation is required if one or more of the 3 conditions noted above exists where structural elements are involved (Caltrans Corrosion Guidelines, September 2003).

## SEISMICITY

### Faulting and Seismicity

The Upper Elysian Park Blind Thrust (EPBT) and the Puente Hills Blind Thrust (PHBT) faults, both recently identified by the California Geological Survey (CGS, 2003) as active Quaternary seismic sources, are the two nearest faults applicable to the site. Caltrans has assigned a Maximum Credible Earthquake (MCE) of moment magnitude 7.0 and 7.25 to these faults, respectively. Based on the CGS (2003) fault database, the closest site-to-fault rupture distance is about 3.5 mile (5.7 km) and 3.9 mile (6.3 km) for the EPBT and PHBT faults, respectively.

Based on the Sadigh et al (1997) attenuation relationships, the median Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.6g due to the MCE associated with each of these faults.

The Peak Ground Acceleration (PGA) at the site is estimated to be about 0.52g and 0.53g due to the MCE associated with the EPBT and PHBT, respectively. Design PGA should be taken as 0.53g. Note that the Elysian Park (EPK) faults shown on the 1996 Caltrans' California Seismic Hazard Map (CSHM) is replaced with the PHBT and EPBT faults and is no longer considered for bridge design.

**Potential Seismic Hazards**

This bridge site is outside mapped liquefiable zones as shown in the California Geological Survey (CGS) Map of Seismic Hazard Zones of Los Angeles 7.5 Minute Quadrangle (1999). Considering the deep groundwater levels and the subsurface soils indicated in the recent field investigation, there is a low potential for liquefaction at the site. Consequently, the potential for seismically induced impacts such as settlement and lateral deformation of subsurface materials is remote.

**Design Ground Motion**

Based on the above deterministic seismic hazard information, the PHBT fault with an MCE of 7.25 can be considered as the controlling seismic source for the bridge. For the purpose of selecting a standard SDC (2006) Acceleration Response Spectrum (ARS) curve, the design PBA is taken as 0.6g. The soil profile may be classified as Type D. The design ARS curve was obtained by modifying the standard SDC curve (for PBA = 0.6g, moment magnitude M = 7.25 ± 0.25, and soil profile type D) for the near-fault effects as specified in the SDC. The recommended modified ARS Curve and the coordinates of spectral accelerations are presented in Figure 1 and Table 6 below.

**Figure 1 - Recommended ARS curve for NB 710 / 5 Separation (Widen)**

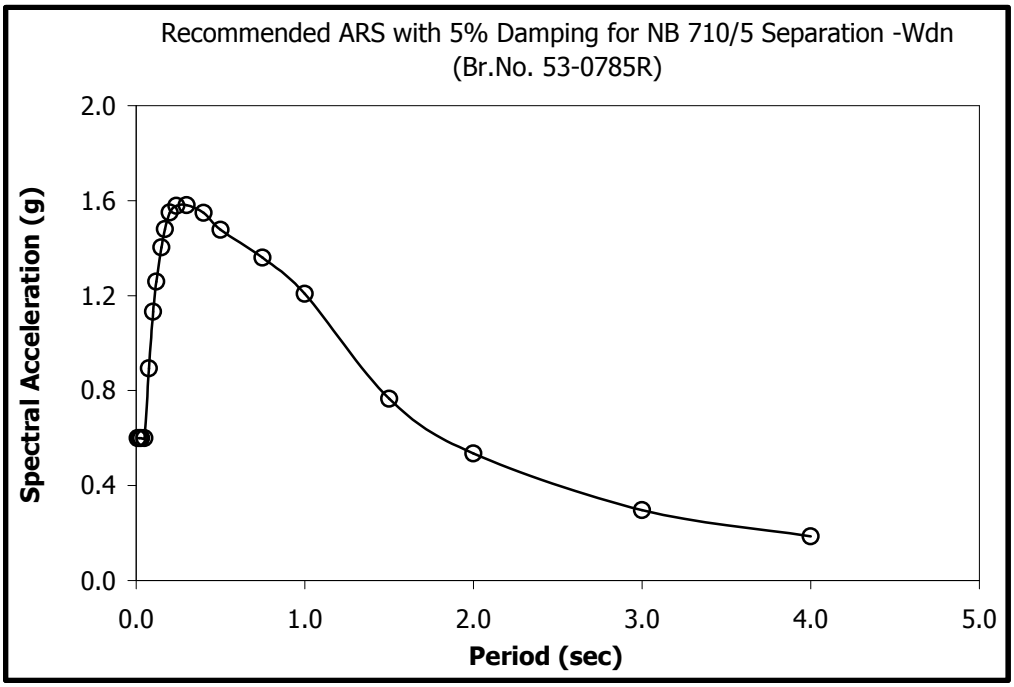

**Table 6 - Recommended Design ARS curve for Br. No. 53-0785R**

| Period (s) | Spectral Acceleration (g) | |
|---|---|---|
| | Standard | **Modified** |
| 0.010 | 0.6003 | 0.6003 |
| 0.020 | 0.6003 | 0.6003 |
| 0.030 | 0.6003 | 0.6003 |
| 0.050 | 0.6003 | 0.6003 |
| 0.075 | 0.8934 | 0.8934 |
| 0.100 | 1.1315 | 1.1315 |
| 0.120 | 1.2590 | 1.2590 |
| 0.150 | 1.4033 | 1.4033 |
| 0.170 | 1.4802 | 1.4802 |
| 0.200 | 1.5496 | 1.5496 |
| 0.240 | 1.5784 | 1.5784 |
| 0.300 | 1.5811 | 1.5811 |
| 0.400 | 1.5484 | 1.5484 |
| 0.500 | 1.4767 | 1.4767 |
| 0.750 | 1.2366 | 1.3602 |
| 1.000 | 1.0062 | 1.2074 |
| 1.500 | 0.6378 | 0.7653 |
| 2.000 | 0.4464 | 0.5357 |
| 3.000 | 0.2471 | 0.2965 |
| 4.000 | 0.1546 | 0.1855 |

**Surface Fault Rupture Hazard**

The project site is not located within any California Geological Survey (CGS) designated Earthquake Fault Zone (EFZ) or directly underlain by any fault considered active for bridge design. Therefore, the possibility of surface fault rupture hazard at the site is considered low.

## AS-BUILT FOUNDATION DATA

Based on 1954 As-Built data the original bridge is supported on concrete piles (45 ton design load) for Abutments 1 and 9 and Bents 2 through 8. Structure Plan No. 1 shows driven concrete piles under Quantities. Pile load tests completed in 1954 show cast-in-drilled-hole piles (CIDH, 45 ton design) were tested near Abutment 1 and Bent 8, and may have been used. Based on 1967 As-Built data, Cast-In-Drilled-Hole (CIDH) concrete piles 16 through 60 in. diameter, with design loads ranging from 45 to 450 tons, were used to support the NB 710/5 Separation right side widening. For the 1997 earthquake retrofit, based on As Built data (2002), abutment seat, hinges, bent caps, and columns were retrofit but no piles were added or foundation retrofit completed. For reference, the top elevations of the existing original bridge abutments are approximately +199 ft (Abutment 1, south end of bridge) and +205 ft. (Abutment 9, north end of bridge), respectively. Approximate bottom of footing, cutoff, average and specified pile tip elevations are provided based on current NAVD88 datum [1954 and 1967 As-Built plan elevations (based on NGVD29 datum, MSL) are shifted +2 ft to approximate the current datum elevations].

**Table 7 – 1954 "As Built" Pile Data for NB 710/5 Separation (original bridge)**

| Location | Pile Type / Design Load (tons) | Design Load (tons) [1] | Approximate Bottom of Footing Elevation (ft) | Approximate Specified Pile Tip Elevation (ft) |
|---|---|---|---|---|
| Abutment 1 | Concrete | 45 | + 186.5 | +157.0 |
| Bent 2 | Concrete | 45 | +163.0 | +142.0 |
| Bent 3 | Concrete | 45 | +170.0 | +142.0 |
| Bent 4 | Concrete | 45 | +164.5 | +140.0 |
| Bent 5 | Concrete | 45 | +161.0 | +142.0 |
| Bent 6 | Concrete | 45 | +174.0 | +152.0 |
| Bent 7 | Concrete | 45 | +174.5 | +152.0 |
| Bent 8 | Concrete | 45 | +175.0 | +152.0 |
| Abutment 9 | Concrete | 45 | +190.0 | +142.0 |

1) Actual resistance in nominal compression has not been verified/calculated by OGDS1

Note: Pile type for original bridge has not been verified. It is possible that driven concrete or driven shells filled with concrete or vertical and battered CIDH piles have been used for bridge support.

**Table 8 - 1967 "As Built" Pile Data for NB 710/5 Separation (Rt. Side Widen - south portion of bridge)**

| Location | Pile Type | Design Load (tons) [1] | Approximate Bottom of Pile Footing/Cutoff Elevation | Specified Pile Tip Elevation (ft) |
|---|---|---|---|---|
| Abutment 1 | 16" Dia. CIDH | 45 | +186.5 | +157.0 |
| Bent 2 | 60" Dia. CIDH | 400 | +168.0 | +124.0 |
| Bent 3 | 60" Dia. CIDH | 450 | +176.0 | +122.0 |
| Bent 4 | 60" Dia. CIDH | 400 | +168.0 | +124.0 |

1) Actual resistance in nominal compression has not been verified/calculated by OGDS1

## FOUNDATION RECOMENDATIONS

The following recommendations are based on 1) the recent geotechnical investigation with lab test results, 2) dated structure plans (General and Foundation Plans, Abutment and Bent Layouts and Details, Column Details, Typical Sections, received from September 30, 2009 through September 21, 2010), bridge foundation design loads and design data sheet (dated August 23, 2010) with CIDH pile sizes provided by Structures Design (Abutment 1, Bents 2 through 8) plus Driven Class 90 pipe pile (PP14X0.375) for Abutment 9 support and Layout L-12 (received July 21, 2010).

**Deep Foundations**

The geotechnical capacities of the proposed piles were evaluated using the design loads and pile sizes provided by OBDS2. Computer program SHAFT Version 5.0 and LPILE plus 5.0 were used for calculating CIDH pile/soil resistances and to determine specified pile tip elevations. Class 90 (Alternative "W") pipe pile/soil resistances and specified pile tip elevations were calculated with computer program Driven 1.2. OGDS1 recommends CIDH piles varying in diameter from 16, 66, and 78 inches to support the right side widening. Pile details and elevations are shown in Tables 9, 10,and 11, below:

General Foundation Information and Design Loads presented in Tables 9 and 10, respectively, were provided by Office of Structure Design 2.

**Table 9 - Foundation Design Data Sheet**

| Support No. | Design Method | Pile Type/Diameter | Finished Grade Elevation (ft) | Cut-off Elevation (ft) | Pile Cap Size | | Permissible Settlement Under Service Load * (in) | Number of Piles per Support |
|---|---|---|---|---|---|---|---|---|
| | | | | | B (ft) | L (ft) | | |
| Abut 1 | WSD | 16" CIDH (90 kips design load) | 195.10 | 190.85 | 6.0 | 12.83 | 1.0 | 4 |
| Bent 2 | LRFD | 66" CIDH | 170.0 | 169.0 | N/A | N/A | 1.0 | 1 |
| Bent 3 | LRFD | 66" CIDH | 176.0 | 175.0 | N/A | N/A | 1.0 | 1 |
| Bent 4 | LRFD | 78" CIDH | 170.0 | 169.0 | N/A | N/A | 1.0 | 1 |
| Bent 5 | LRFD | 78" CIDH | 170.0 | 169.0 | N/A | N/A | 1.0 | 1 |
| Bent 6 | LRFD | 66" CIDH | 178.0 | 177.0 | N/A | N/A | 1.0 | 1 |
| Bent 7 | LRFD | 66" CIDH | 178.0 | 177.0 | N/A | N/A | 1.0 | 1 |
| Bent 8 | LRFD | 66" CIDH | 180.0 | 179.0 | N/A | N/A | 1.0 | 1 |
| Abut 9 | WSD | Class 90 Alt. "W" PP14X0.375 | 191.77<br>189.0 | 186.39<br>183.61 | 7.0 | 33.86 | 1.0 | 16 |

* Based on CALTRANS' current practice, the total permissible settlement is one inch for multi-span structures with continuous spans or multi-column bents, one inch for single span structures with diaphragm abutments, and two inches for single span structures with seat abutments. Different permissible settlement under service loads may be allowed if a structural analysis verifies that required level of serviceability is met.

**Table 10 - Foundation Design Loads**

| Support No. | Service-1 Limit State (kips) | | | Strength Limit State (Controlling Group, kips) | | | | Extreme Event Limit State (Controlling Group, kips) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | Total Load | | Permanent Loads | Compression | | Tension | | Compression | | Tension | |
| | Per Support | Max Per Pile | Per Support | Per Support | Max Per Pile | Per Support | Max Per Pile | Per Support | Max Per Pile | Per Support | Max Per Pile |
| Abut 1 | 305 | 76 | 224 | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A |
| Bent 2 | 616 | N/A | 458 | 964 | 964 | 0 | 0 | 537 | 537 | 0 | 0 |
| Bent 3 | 843 | N/A | 657 | 1288 | 1288 | 0 | 0 | 750 | 750 | 0 | 0 |
| Bent 4 | 1017 | N/A | 846 | 1477 | 1477 | 0 | 0 | 932 | 932 | 0 | 0 |
| Bent 5 | 1283 | N/A | 949 | 2026 | 2026 | 0 | 0 | 1116 | 1116 | 0 | 0 |
| Bent 6 | 1169 | N/A | 861 | 1778 | 1778 | 0 | 0 | 1015 | 1015 | 0 | 0 |
| Bent 7 | 843 | N/A | 577 | 1301 | 1301 | 0 | 0 | 710 | 710 | 0 | 0 |
| Bent 8 | 901 | N/A | 643 | 1352 | 1352 | 0 | 0 | 772 | 772 | 0 | 0 |
| Abut 9 | 600 | 68 | 454 | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A |

The Abutment Foundation Design Recommendations [Table 11, 16 inch diameter CIDH piles and steel pipe piles (PP14X0.375 in.], Bent Foundation Design Recommendations (Table 12, both 66 and 78 inch diameter CIDH piles), and the Pile Data Table 13, prepared by OGDS1, are presented below.

**Table 11 – Abutment Foundation Design Recommendations for NB 710/5 Separation (Rt side Wdn)**

| Support Location | Pile Type/Diameter (in) | Finished Grade Elevation (ft) | Cut-off Elevation (ft) | LRFD Service-1 Limit State Load (kips) Per Support | | LRFD Service-1 Limit State Total Load (kips) per Pile (Compression) | Nominal Resistance in Compression (kips) | Design Pile Tip Elevations (ft) | Specified Pile Tip Elevation (ft) |
|---|---|---|---|---|---|---|---|---|---|
| | | | | Total | Permanent | | | | |
| Abut 1 | 16" CIDH (90 kips design load) | 195.10 | 190.85 | 310 | 230 | 80 | 160 | 155.0 (a)<br>169.0 (c) | 155.0 |
| Abut 9 | Class 90 Alt."W" PP14X0.375 | 191.77<br>189.0 | 186.39<br>183.61 | 600 | 460 | 70 | 140 | 147.0/145.0 (a)<br>156.0/153.0 (c) | 147.0/ 145.0 |

Notes:
1) Design pile tip elevations are controlled by: (a) Compression , (c) Settlement, and (d) Lateral Load, respectively.
2) The CIDH specified tip elevation shall not be raised.
3) Design pile tip elevation for Lateral Load is typically provided by SD.

**Table 12 – Bent Foundation Design Recommendations for NB 710/5 Separation (Rt side Wdn)**

| Support Location | Pile Type/ Diameter | Cut-off Elevation (ft) | Service-I Limit State Load per Support (kips) | Total Permissible Support Settlement (inches) | Required Factored Nominal Resistance (kips) | | | | Design Tip Elevations (ft) | Specified Tip Elevation (ft) |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Strength Limit | | Extreme Event | | | |
| | | | | | Comp. (φ=0.7) | Tension (φ=0.7) | Comp. (φ=1) | Tension (φ=1) | | |
| Bent 2 | 66" CIDH | 169.0 | 620 | 1 | 964/0.7= 1377 | N/A | 537 | N/A | 112.0 (a-I)<br>140.0 (a-II)<br>137.0 (c) | 112.0 |
| Bent 3 | 66" CIDH | 175.0 | 850 | 1 | 1288/0.7= 1840 | N/A | 750 | N/A | 90.0 (a-I)<br>138.0 (a-II)<br>139.0 (c) | 90.0 |
| Bent 4 | 78" CIDH | 169.0 | 1020 | 1 | 1477/0.7= 2110 | N/A | 932 | N/A | 74.0 (a-I)<br>125.0 (a-II)<br>128.0 (c) | 74.0 |
| Bent 5 | 78" CIDH | 169.0 | 1290 | 1 | 2026/0.7= 2894 | N/A | 1116 | N/A | 56.0 (a-I)<br>120.0 (a-II)<br>115.0 (c) | 56.0 |
| Bent 6 | 66" CIDH | 177.00 | 1170 | 1 | 1778/0.7= 2540 | N/A | 1015 | N/A | 60.0 (a-I)<br>114.0 (a-II)<br>111.0 (c) | 60.0 |
| Bent 7 | 66" CIDH | 177.0 | 850 | 1 | 1301/0.7= 1858 | N/A | 710 | N/A | 74.0 (a-I)<br>133.0 (a-II)<br>125.0 (c) | 74.0 |
| Bent 8 | 66" CIDH | 179.0 | 910 | 1 | 1352/0.7= 1931 | N/A | 772 | N/A | 68.0 (a-I)<br>120.0 (a-II)<br>114.0 (c) | 68.0 |

Notes:

1) Design tip elevations are controlled by: (a-I) Compression (Strength Limit), (a-II) Compression (Extreme Event) (c) Settlement, and (d) Lateral Load, respectively.
2) The CIDH specified pile tip elevation shall not be raised.
3) Design pile tip elevation for Lateral Load is typically provided by SD.

**Table 13 – Pile Data Table for NB 710/5 Separation (Rt side Wdn), Br. No. 53-0785R**

| Location | Pile Type | Nominal Resistance (kips) | | Design Pile Tip Elevation (ft) | Specified Pile Tip Elevation (ft) |
|---|---|---|---|---|---|
| | | Compression | Tension | | |
| Abut 1 | 16" CIDH (90 kips design load) | 160 | N/A | 155.0 (a)<br>169.0 (c) | 155.0 |
| Bent 2 | 66" CIDH | 1380 | N/A | 112.0 (a-I)<br>140.0 (a-II)<br>137.0 (c) | 112.0 |
| Bent 3 | 66" CIDH | 1840 | N/A | 90.0 (a-I)<br>138.0 (a-II)<br>139.0 (c) | 90.0 |
| Bent 4 | 78" CIDH | 2110 | N/A | 74.0 (a-I)<br>125.0 (a-II)<br>128.0 (c) | 74.0 |
| Bent 5 | 78" CIDH | 2900 | N/A | 56.0 (a-I)<br>120.0 (a-II)<br>115.0 (c) | 56.0 |
| Bent 6* | 66" CIDH | 2540 | N/A | 60.0 (a-I)<br>114.0 (a-II)<br>111.0 (c) | 60.0 |
| Bent 7* | 66" CIDH | 1860 | N/A | 74.0 (a-I)<br>133.0 (a-II)<br>125.0 (c) | 74.0 |
| Bent 8* | 66" CIDH | 1940 | N/A | 68.0 (a-I)<br>120.0 (a-II)<br>114.0 (c) | 68.0 |
| Abut 9 | Class 90 Alt."W" PP14X0.375 | 140 | N/A | 147.0/145.0 (a)<br>156.0/153.0 (c) | 147.0/145.0 |

Notes:
1) Design tip elevations for Abutments are controlled by: (a) Compression, (c) Settlement, (d) Lateral Load
2) Design tip elevations for Bents are controlled by: (a-I) Compression (Strength Limit), (a-II) Compression (Extreme Event), (c) Settlement, and (d) Lateral Load.
3) The CIDH specified tip elevation shall not be raised.
4) * = Isolation casing proposed at Bents 6, 7, and 8. Isolation casing bottom tip elevations assumed at +173.0 (Bent 6), +169.0 (Bent 7), and +173 ft (Bent 8) from information supplied by Structures Design.

Pile lengths were computed based on the provided design loads and the observed subsurface conditions. The analysis was conducted based on the assumption that proposed CIDH piles would derive geotechnical capacity from skin friction only (100%). The pipe pile would derive geotechnical capacity of approximately 90% from skin friction and 10% from end bearing.

**Settlement at Approach Fills**

Fills can be placed in accordance with Section 19-6 of the Caltrans Standard Specifications (May 2006). End dumping is not permitted. At Abutments 1 and 9 areas, additional sliver fills are estimated to range from 20 to 26 ft height. Calculated maximum settlement (Hough's Method) is 1 to 3 inches at Abutments 1 and 9, respectively. OGDS1 recommends a fill settlement waiting period of up to 30 days for the widening. However, the actual settlement period will be determined by the structure representative on the basis of settlement data in the field and the above waiting period may be adjusted accordingly.

## NOTE TO DESIGNER

The recommendations contained in this report are based on specific project information, regarding bridge location, pile type, finished grade elevation, cut-off elevation and loads, that has been provided to OGDS1. If any conceptual changes are made during final project design, this office should review those changes to determine if these included foundation recommendations are still applicable.

## CONSTRUCTION CONSIDERATIONS

1. Design groundwater is considered to be at approximate elevation +59.2 ft. All CIDH piles, with the exception of Bent 5 (also, Bent 6 specified pile tip elevation is just slightly above measured groundwater), terminate above design groundwater elevation so no overdrilling should be allowed so that CIDH piles (with the exception of Bents 5 and possibly Bent 6) can be constructed under dry conditions. CIDH pile construction, during typical low water months (such as summer) is recommended by OGDS1. Casing and/or slurry displacement techniques are considered necessary at Bent 5 and likely necessary at Bent 6 location. Specified pile tip elevation (SPTE) for Bent 5 CIDH pile is below the groundwater table. SPTE for Bent 6 CIDH pile is just slightly above (less than 1 ft above) the measured static groundwater level. With a small fluctuation in groundwater level during different seasons or years, Bent 6 SPTE could be below the water table. If casing and/or slurry displacement techniques are not deemed feasible at Bents 5 and 6 location due to limited space or other reason, then a suitable alternative must be proposed by the contractor and reviewed by Caltrans Construction, Design, and OGDS1 for possible construction acceptance.
2. The contractor shall be required to clean out the bottom of the CIDH pile boring prior to placing the cage and the concrete.
3. OGDS1 recommends concrete placement for construction of the CIDH piling shall be completed within the same day that excavation of the drilled hole has been completed. Otherwise, a suitable alternative must be proposed to control caving if concrete placement cannot be completed the same day as drilling.
4. Moderate to minor caving and sloughing should be anticipated during excavation of the pile boring and during CIDH pile construction.
5. All earthwork is expected to be carried out by conventional equipment. Fill should be placed as specified in Section 19-6 of the Caltrans Standard Specifications (May 2006). If imported materials are used to construct the new fill embankment, the material should be tested during grading to assess expansion potential. Only non-expansive soils or soils having a low

expansion potential (EI<50) should be used in the Soil Expansion Exclusion Zone in bridge approach embankment and within 10 ft of the roadbed sub-grade elevation.

6. Pile acceptance criteria at Abutment 9 (driven pipe piles) should be based on Gates formula within Section 49-1.08 (Pile Driving Acceptance Criteria) of the Caltrans Standard Specifications (May 2006).

If you have any questions or need further information, please contact Kevin Lai at (213) 620-2344 or Shiva Karimi at (213) 620-2146.

Prepared by: Date:9/29/2010

KEVIN LAI,
Transportation Engineer
Office of Geotechnical Design – South 1
Branch D

Supervised by: Date: 9/29/2010

SHIVA KARIMI, C
Senior Transportatio
Office of Geotechnica
Branch D

REGISTERED PROFESSIONAL ENGINEER
SHIVA KARIMI
No. GE2651
Exp. 6/30/12
GEOTECHNICAL
STATE OF CALIFORNIA

Reviewed by: Date: 9/29/2010

JOE PRATT, C.E.G. NO. 2141
Engineering Geologist
Office of Geotechnical Design South 1
Branch D

REGISTERED GEOLOGIST
Joseph S. Pratt
No. 2141
Exp. 5/31/11
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

cc:

| | | |
|---|---|---|
| District Project Manager | James S. Hsu | James_s_Hsu@dot.ca.gov |
| GS Corporate | Mark Willian | Mark_Willian@dot.ca.gov |
| District Construction R.E. Pending File | | TBD |
| Structure Construction R.E. Pending File | | RE_Pending_File@dot.ca.gov |
| DES Office Engineer, Office of PS&E | Efren Ancheta | Efren_Ancheta@dot.ca.gov |
| District Materials Engineer | Kirsten Stahl | Kirsten_Stahl@dot.ca.gov |

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**State of California** **Business, Transportation and Housing Agency**

# M e m o r a n d u m

***Flex your power!***
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**To:** MS. CHUNG-YUAN WEN
Chief, Bridge Design Branch 21
Office of Bridge Design South 2,
21073 Pathfinder Rd., Suite 200
Diamond Bar, CA 91765

Attention: Mr. Wei-Kung Hsia

**Date:** September 23, 2010

**File**: 07-LA-710-PM 23.21
EA: 07-202111
SB 710/5 Separation
(Lt. Side Widen)
Bridge No. 53-0788L

**From: DEPARTMENT OF TRANSPORTATION**
**DIVISION OF ENGINEERING SERVICES**
**Geotechnical Services**
**Office of Geotechnical Design - South 1**
**Branch D**

**Subject:** Foundation Report for SB 710 /5 Separation Widening, Bridge No. 53-0788L

## INTRODUCTION

This Foundation Report was prepared in response to the Office of Bridge Design South 2's (OBDS2's) request dated May 21, 2009, for the proposed left side widening of the Southbound 710/5 Separation (Br. No. 53-0788L) above Interstate 5 (I-5).

## SCOPE OF WORK

The recommendations contained in this report are based on the subsurface information gathered during the recent field investigation performed in August and October 2009, Structure Plans (received September 29 and 30, 2009, through September 23, 2010) and updated design information (received through September 23, 2010). Tasks completed by the Office of Geotechnical Design South 1 (OGDS1), Branch D include the following:

1. Review of the regional geology and seismicity,
2. Review of pertinent information from previous geology/geotechnical reports and As-Built plans in the project area to evaluate the subsurface information,
3. Drilling, logging, and sampling of five rotary wash borings at the subject bridge site to characterize the subsurface conditions,
4. Laboratory testing of selected soil samples for mechanical analysis, corrosivity, direct shear, and unit weight,
5. Geotechnical engineering analysis

## PROJECT DESCRIPTON

The SB 710/5 Separation, straddling the boundary between the cities of Commerce and Los Angeles, was originally built in 1955, left side widened in 1967, and earthquake retrofit in 2002. The Los Angeles River is approximately 1.4 miles southwest of the SB 710/5 Separation Bridge. The SB 710/5 Separation consists of two connected bridge structures. The right (main) structure, constructed in 1955, consists of seven spans and is supported by what were referred to as piers. The left widening structure was added in 1967 and consists of eight spans supported by bents. The SB 710/5 Separation Bridge is a continuous seven-span combination of reinforced concrete box girder and precast/prestressed inverted T-girders, with seated abutments and two column bents, with a total length of 759 ft. measured along the Southbound station line ("SBLS1" Line) for the current project widening (varying from about 695 to 732 ft. length for the 1955 original bridge and 1967 Left Side Widening, respectively).

Based on the Planning Study sheet (revision date September 30, 2009), provided by Office of Bridge Design South 2, OGDS1 understands that the proposed project will consist of outside widening of the west side (left side) of the existing bridge by 7 ft 4in (± varies). Nine spans (CIP/PS box Girder) will be constructed for the proposed left side widening with accompanying ten supports.

All elevations provided within this report and proposed project plans are based on NAVD88 datum with the exception of groundwater well data which is recorded utilizing mean sea level (MSL, NGVD29 datum).

## FIELD INVESTIGATION AND TESTING PROGRAM

The site-specific field exploration was performed between August and October 2009. The field investigation included drilling five rotary wash borings, all 4.5 inch diameter. Standard Penetration Tests (SPT's), and relatively undisturbed sampling (with a modified California split-barrel sampler, 2 inches inner diameter) were performed at the borings. Blow counts (SPT N values) were continuously recorded at 5 foot intervals during drilling. The SPT's were performed in accordance with ASTM Test Method D1586-84 using a standard 1.4 inch I.D. sampler with a 140 lb. hammer dropped 30 inches. Undisturbed tube soil samples were also obtained using the 2 inch I.D. modified California Split Spoon Sampler with 4 inch long brass liners. The liners were all sealed in the field. Caltrans operated drill rig models Mobile B-80 and CME-750 were used at the boring locations.

The District 7 Surveys provided the location and elevation of the borings. Boring information, including exploration number, stationing, offset, ground surface elevation, boring depth, and date drilled are summarized in Table 1 below.

**Table 1 – Summary of Borings**

| Exploration No. | Station (ft) | Offset (ft) | Reference Line | Surface Elev. (ft) | Depth (ft) | Date Drilled |
|---|---|---|---|---|---|---|
| R-09-046 | 1223+58.6 | 54.5 Rt | "Conn 2" Line | +203.4 | 101.5 | 8/12/09 |
| R-09-047 | 1226+11.9 | 66.6 Lt | "Conn 1" Line | +186.1 | 120.3 | 10/20/09 |
| R-09-048 | 1229+10.1 | 74.0 Rt | "Conn 2" Line | +178.0 | 101.5 | 8/12/09 |

| | | | | | | |
|---|---|---|---|---|---|---|
| R-09-049 | 1229+40.0 | 61.1 Lt | "Conn 1" Line | +169.1 | 120.7 | 8/19/09 |
| R-09-050 | 1231+74.9 | 55.9 Rt | "Conn 2" Line | +202.0 | 101.5 | 8/11/09 |

## LABORATORY TESTING PROGRAM

Selected soil samples were sent to the Department's Transportation Laboratory in Sacramento and Los Angeles for laboratory testing. All laboratory tests were performed in accordance with ASTM standard procedures and California Test Methods. The summarized laboratory tests data are shown in Table 2, below:

**Table 2 – Summary of Laboratory Tests**

| Test | Standard | No. of Test Performed |
|---|---|---|
| Mechanical Analysis | CTM 202, 203 | 26 |
| Atterberg | CTM 204 | 21 |
| Corrosion | CTM 417, 422, 532 | 18 |
| Direct Shear | ASTM D 3080 | 2 |

## SITE GEOLOGY AND SUBSURFACE CONDITIONS

### Regional Geology

The site is located in the Peninsular Ranges Province in the Los Angeles Basin and is bounded to the northwest/north by the Santa Monica Mountains, to the southwest/south by the Newport-Inglewood Fault Zone, to the northeast/east by the Whittier fault and Puente Hills, and to the southeast by the Santa Ana Mountains and San Joaquin Hills. A thick Cenozoic sedimentary section underlies the site.

### Site Subsurface Conditions

At the SB 710/5 Separation (Widen) site, embankment fill is approximately 19 to 24 ft thick at Abutment 1 and 23 to 27 ft thick at Abutment 10. Embankment fill is underlain by Holocene and older Quaternary alluvium (Qa and Qoa, Dibblee, 1989, Geologic Map of Los Angeles 7.5 minute Quadrangle). The Pliocene Fernando Formation (Tfsc, nonmarine sandstone and conglomerate and Tfr, marine claystone) most likely underlies the older alluvium at depth.

Top of embankment ranges between approximately +200 to +206.0 ft elevations at Abutments 1 and 10, respectively. The bottom of the embankment fill/top of alluvium extends down to between approximate elevations +179 to +160 ft (deepest part of cut for I-5 Freeway). Embankment fill is composed of loose to dense, silty sand, interlayered with clayey sand, sandy silt, and stiff to hard, sandy lean clay. Fill also contains trace to few gravel and sporadic trace to few asphalt and concrete fragments (6 to 8 in maximum length). Underlying undifferentiated Holocene/older Quaternary alluvium can be divided into 4 units in descending order. The upper alluvial unit, ranging from approximate elevations +179 to +160 ft down to +151 to 146 ft, and is composed of medium dense to

very dense (minor loose), sandy silt to silt, silty sand, and clayey sand interbedded with stiff to hard, sandy lean clay. The upper alluvial unit contains sporadic trace gravel and trace calcite filaments and nodules. The upper middle alluvial unit, ranging from approximate elevations +151 to +146 ft down to +134 to +130 ft, consists of generally stiff to hard, sandy lean clay and lean clay interbedded with minor medium dense, sandy silt. The lower middle alluvial unit, ranging from approximate elevations +134 to +130 ft down to +100 to +89 ft, consists of dense to very dense, sand and silty sand, clayey sand, and sandy silt interbedded with stiff to hard, sandy lean clay, lean clay, and silt. The lower middle alluvial unit also contains sporadic few gravel and shows iron oxide staining. The lower alluvial unit, ranging from approximate elevation +100 to +89 ft down to at least +48.4 ft, consists of dense to very dense, silty sand with sporadic gravel, sandy silt, clayey sand, and clayey gravel with sand interbedded with very stiff to hard, sandy lean clay. The lower alluvial unit contains sporadic few to little gravel within the sand and clay beds. The maximum boring depth extended down to 120.7 ft (elevation +48.4 ft) within Boring R-09-049. Refer to the Log of Test Borings for further details, which will be sent as soon as completed.

**Groundwater**

Static groundwater was measured on February 25, 2010 at depth 109.9 ft (elevation +59.2 ft, NAVD88 datum). Groundwater was periodically measured within piezometer Boring R-09-049 during the recent field exploration as shown in Table 3 below.

**Table 3 – Recent Groundwater Monitoring**

| Boring No. | Ground Surface Elevation (ft) | Depth to Water Below Surface (ft) | Groundwater Surface Elevation (ft) | Date Measured |
|---|---|---|---|---|
| **R-09-049** | **+169.1** | 109.78 | +59.3 | 11/16/2009 |
| | | **109.90** | **+59.2** | **02/25/2010** |
| | | 110.90 | +58.2 | 09/21/2010 |

The As-Built LOTB's for the original bridge and widening indicate that groundwater was not encountered at the site during the 1953 and 1964/1965 investigations. The deepest boring extended down to approximate current adjusted elevation +91 ft (+89 ft MSL).

Table 4 shows historical groundwater elevations in the vicinity of the bridge obtained from the monitoring well records of Los Angeles County Department of Public Works (LACDPW) and California Department of Water Resources (DWR). Four (4) wells located within distances ranging from about 0.27 to 0.43 miles from the site indicate groundwater elevations (during last 70 years) lower than approximate current adjusted elevation +89.0 ft (+86.8 feet, MSL, depths greater than 76.5 feet). The chart below shows recorded elevations based on NGVD29 datum (mean sea level, MSL) which requires about a 2 foot shift (add) to reach the current NAVD88 datum elevations.

**Table 4 - Historic Groundwater Records near SB 710/5 Separation Bridge**

| LACDPW Well No. | Well Distance and Direction from SB 710/5 Separation | Date | Ground Surface Elevation (ft) | Historic High Groundwater Elevation (ft) | Date | Historic Low Groundwater Elevation (ft) | Date | Recent Ground Water Elevation (ft) |
|---|---|---|---|---|---|---|---|---|
| 2828F | 0.28 mile NW | 4/26/95 | +185.8 | +36.8 (149.0) | 7/17/53 | -102.2 (288.0) | 11/30/07 | +6.8 (179.0) |
| 2828C | 0.27 mile SW | 11/30/07 | +168.8 | +69.8 (99.0) | 8/31/57 | -81.2 (250.0) | 11/30/07 | +69.8 (99.0) |
| 2838 | 0.31 mile SE | 2/30/40 | +163.3 | +86.8 (76.5) | 1/31/57 | -102.2 (265.5) | 9/30/05 | +14.8 (148.5) |
| 2838A | 0.43 mile NE | 5/15/94 | +174.2 | +66.2 (108.0) | 8/31/57 | -82.8 (257.0) | 11/30/07 | -20.8 (195.0) |

*Values in parentheses indicate depth below the ground surface.

Based predominantly on the recent 2009/2010 geotechnical investigation, As Built LOTB's, and to a lesser extent the above historic groundwater review (which is not specifically on site), OGDS1 considers groundwater level for bridge design to be at elevation +59.2 ft (based on current NAVD88 datum). The anomalous historic high groundwater elevation was measured within Well No. 2838 during the winter of 1940 and is much lower within recent measurements (elevation +14.8 MSL, measured September 2005). Groundwater elevation within Well No. 2838A has also dropped significantly during recent November 2007 measurements. Within Well No. 2828C, groundwater elevation is high even within recent November 2007 measurements, but the well is 0.27 miles southwest of the subject bridge and site specific measurements at the bridge are considered more appropriate for design groundwater. With the exception of Well No. 2828C, all other groundwater measurements are lower within recent measurements and well below design groundwater.

## SCOUR EVALUATION

There is no possibility of scour at the site due to substantial distance from the Los Angeles River channel or other significant creeks.

## CORROSION EVALUATION

Corrosion test results for representative soil samples are presented in Table 5 below. The results show this area can be considered as non-corrosive to reinforced concrete and metal.

**Table 5 – Corrosion Test Summary**

| Exploration No. | Sample Depth (ft) | pH | Minimum Resistivity (ohm – cm) | Sulfate Content (PPM) | Chloride Content (PPM) |
|---|---|---|---|---|---|
| R-09-047 | 10.0 – 11.5 | 6.87 | 910 | 200 | 130 |
| R-09-047 | 76.0 – 76.5 | 7.93 | 3700 | N/A | N/A |
| R-09-047 | 85.0 – 86.5 | 8.35 | 2700 | N/A | N/A |
| R-09-048 | 16.5 – 45.0 | 7.43 | 1500 | N/A | N/A |
| R-09-048 | 61.5 – 65.0 | 7.53 | 2100 | N/A | N/A |
| R-09-049 | 5.0 – 6.5 | 8.13 | 1951 | N/A | N/A |
| R-09-049 | 10.0 – 36.5 | 6.81 | 1631 | N/A | N/A |
| R-09-049 | 40.0 – 81.5 | 7.34 | 4408 | N/A | N/A |
| R-09-049 | 81.5 – 120.7 | 7.38 | 5429 | N/A | N/A |
| R-09-050 | 16.5 – 35.0 | 8.18 | 3400 | N/A | N/A |
| R-09-050 | 46.5 – 65.0 | 7.69 | 1600 | N/A | N/A |
| R-09-050 | 5.0 – 6.5 | 8.07 | 2500 | N/A | N/A |
| R-09-050 | 70.4 – 70.7 | 7.82 | 1900 | N/A | N/A |
| R-09-050 | 70.0-86.5 | 8.01 | 7508 | N/A | N/A |

Note: It is the practice of Caltrans Corrosion Technology Section (with the exception of MSE Walls) if the minimum resistivity of the sample is greater than 1000 ohm-cm and the pH is greater than 5.5, the sample is considered to be noncorrosive. For structural elements, the California Department of Transportation considers a site to be corrosive if one or more of the following conditions exist for representative soil and/or water samples taken at the site:
Chloride concentration $\geq$500 ppm, sulfate concentration $\geq$2000 ppm, or the pH is $\leq$5.5. Corrosion mitigation is required if one or more of the 3 conditions noted above exists where structural elements are involved (Caltrans Corrosion Guidelines, September 2003).

## SEISMICITY

### Faulting and Seismicity

The Upper Elysian Park Blind Thrust (EPBT) and the Puente Hills Blind Thrust (PHBT) faults, both recently identified by the California Geological Survey (CGS, 2003) as active Quaternary seismic sources, are the two nearest faults applicable to the site. Caltrans has assigned a Maximum Credible Earthquake (MCE) of moment magnitude 7.0 and 7.25 to these faults, respectively. Based on the CGS (2003) fault database, the closest site-to-fault rupture distance is about 3.5 mile (5.7 km) and 3.9 mile (6.3 km) for the EPBT and PHBT faults, respectively.

Based on the Sadigh et al (1997) attenuation relationships, the median Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.6g due to the MCE associated with each of these faults. The Peak Ground Acceleration (PGA) at the site is estimated to be about 0.52g and 0.53g due to the

MCE associated with the EPBT and PHBT, respectively. Design PGA should be taken as 0.53g. Note that the Elysian Park (EPK) faults shown on the 1996 Caltrans' California Seismic Hazard Map (CSHM) is replaced with the PHBT and EPBT faults and is no longer considered for bridge design.

**Potential Seismic Hazards**

This bridge site is outside mapped liquefiable zones as shown in the California Geological Survey (CGS) Map of Seismic Hazard Zones of Los Angeles 7.5 Minute Quadrangle (1999). Considering the deep groundwater levels and the subsurface soils indicated in the recent field investigation, there is a low potential for liquefaction at the site. Consequently, the potential for seismically induced impacts such as settlement and lateral deformation of subsurface materials is remote.

**Design Ground Motion**

Based on the above deterministic seismic hazard information, the PHBT fault with an MCE of 7.25 can be considered as the controlling seismic source for the bridge. For the purpose of selecting a standard SDC (2006) Acceleration Response Spectrum (ARS) curve, the design PBA is taken as 0.6g. The soil profile may be classified as Type D. The design ARS curve was obtained by modifying the standard SDC curve (for PBA = 0.6g, moment magnitude M = 7.25 ± 0.25, and soil profile type D) for the near-fault effects as specified in the SDC. The recommended modified ARS Curve and the coordinates of spectral accelerations are presented in Figure 1 and Table 6 below.

**Figure 1 - Recommended ARS curve for SB 710 / 5 Separation (Widen)**

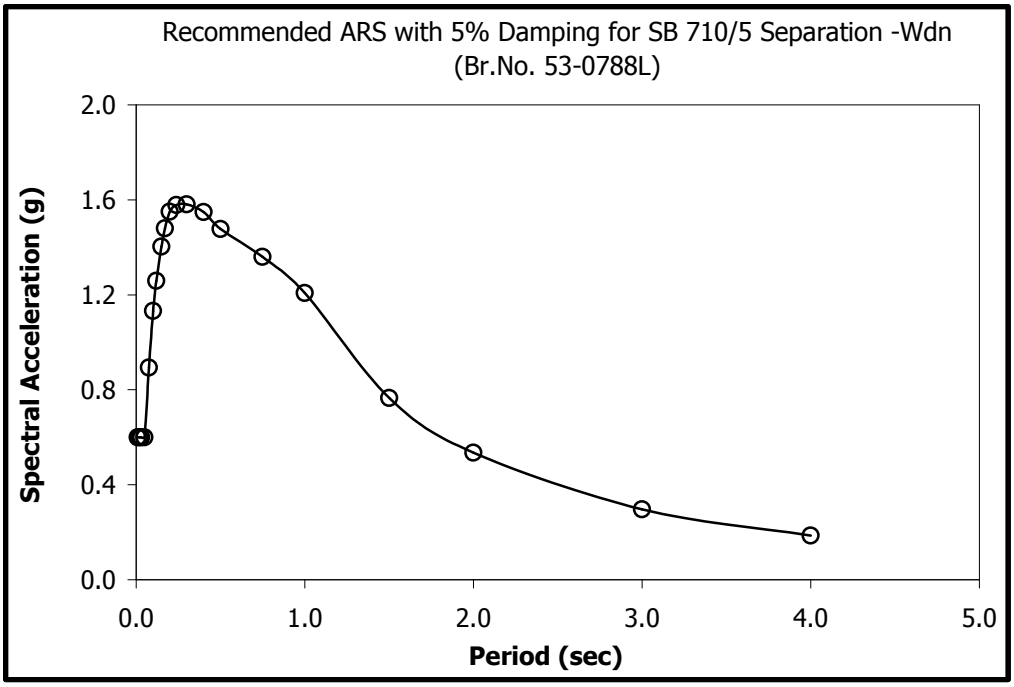

**Table 6 - Recommended Design ARS curve for Br. No. 53-0788L**

| Period (s) | Spectral Acceleration (g) | |
|---|---|---|
| | Standard | **Modified** |
| 0.010 | 0.6003 | 0.6003 |
| 0.020 | 0.6003 | 0.6003 |
| 0.030 | 0.6003 | 0.6003 |
| 0.050 | 0.6003 | 0.6003 |
| 0.075 | 0.8934 | 0.8934 |
| 0.100 | 1.1315 | 1.1315 |
| 0.120 | 1.2590 | 1.2590 |
| 0.150 | 1.4033 | 1.4033 |
| 0.170 | 1.4802 | 1.4802 |
| 0.200 | 1.5496 | 1.5496 |
| 0.240 | 1.5784 | 1.5784 |
| 0.300 | 1.5811 | 1.5811 |
| 0.400 | 1.5484 | 1.5484 |
| 0.500 | 1.4767 | 1.4767 |
| 0.750 | 1.2366 | 1.3602 |
| 1.000 | 1.0062 | 1.2074 |
| 1.500 | 0.6378 | 0.7653 |
| 2.000 | 0.4464 | 0.5357 |
| 3.000 | 0.2471 | 0.2965 |
| 4.000 | 0.1546 | 0.1855 |

**Surface Fault Rupture Hazard**

The project site is not located within any California Geological Survey (CGS) designated Earthquake Fault Zone (EFZ) or directly underlain by any fault considered active for bridge design. Therefore, the possibility of surface fault rupture hazard at the site is considered low.

**AS-BUILT FOUNDATION DATA**

Based on 1955 As-Built data the original bridge is supported on precast concrete piles for Abutments 1 and 8, and Pier 7 and large spread footings for Piers 2 through 6. Based on 1967 As-Built data, Cast-In-Drilled-Hole (CIDH) concrete piles 16 through 72 in. diameter, with design loads ranging from 45 to 800 tons, were used to support the SB 710/5 Separation left side widening. For the 1997 earthquake retrofit, based on As Built data (2002), 16 in. diameter CIDH piles with design loads of 100 tons were added as perimeter piles and footings were also widened. For reference, the top elevations of the existing original bridge abutments are +200.5 ft (Abutment 1, south end of bridge) and +204.7 ft. (Abutment 8, north end of bridge), respectively. Approximate bottom of footing, cutoff, average and specified pile tip elevations are provided based on current NAVD88 datum [1955 and 1967 As-Built plan elevations (based on NGVD29 datum, MSL) are shifted +2 ft to approximate the current datum elevations].

**Table 7 – 1955 "As Built" Pile Data for SB 710/5 Separation (original bridge)**

| Location | Pile Type / Support Type | Design Load / Allowable Bearing Capacity | Approximate Bottom of Footing Elevation (ft) | Approximate Specified Pile Tip Elevation (ft) |
|---|---|---|---|---|
| Abutment 1 | Precast concrete | 45 tons [1] | + 180.0 [2] | +157.0 |
| Pier 2 | Spread footing | 3 TSF | +157.0 | N/A |
| Pier 3 | Spread footing | 3 TSF | +157.0 | N/A |
| Pier 4 | Spread footing | 3 TSF | +166.0 | N/A |
| Pier 5 | Spread footing | 3 TSF | +159.0 | N/A |
| Pier 6 | Spread footing | 3 TSF | +159.0 | N/A |
| Pier 7 | Precast concrete | 45 tons [1] | +176.0 | +152.0 |
| Abutment 8 | Precast concrete | 45 tons [1] | +195.67 [2] | +152.0 |

1) Actual resistance in nominal compression has not been verified / calculated by OGDS1
2) Bottom of lowest footing elevation

**Table 8 - 1967 "As Built" Pile Data for SB 710/5 Separation (Left side Widen)**

| Location | Pile Type [1] | Design Load (tons) [2] | Approximate Bottom of Pile Footing/Cutoff Elevation [3] | Approximate Average Pile Tip Elevation (ft) |
|---|---|---|---|---|
| Abutment 1 | 16" Dia. CIDH (4) | 45 | +190.0 (33.4) | +156.6 |
| Bent 2 | 60" Dia. CIDH (1) | 400 | +176.0 (34.2) | +141.8 |
| Bent 3 | 72" Dia. CIDH (1) | 750 | +163.0 (61.0) | +102.0 |
| Bent 4 | 72" Dia. CIDH (1) | 750 | +162.0 (62.0) | +100.0 |
| Bent 5 | 72" Dia. CIDH (1) | 650 | +170.0 (69.5) | +100.5 |
| Bent 6 | 72" Dia. CIDH (1) | 650 | +167.0 (66.0) | +101.0 |
| Bent 7 | 72" Dia. CIDH (1) | 750 | +165.0 (64.0) | +101.0 |
| Bent 8 | 60" Dia. CIDH (1) | 800 | +182.0 (81.0) | +101.0 |
| Abutment 9 | 16" Dia. CIDH (22) | 45 | +195.67 (46.6) | +149.1 |

1) Values in the parenthesis of the pile type column indicate number of piles
2) Actual resistance in nominal compression has not been verified/calculated by OGDS1
3) Values in parenthesis indicate average length of piles

**Table 9 - 2002 "As Built" Pile Data for SB 710/5 Separation (Earthquake Retrofit)**

| Location | Pile Type | Design Load (tons) [1] | Bottom of Footing Elevation (ft) | As-Built Specified Pile Tip Elevation (ft) |
|---|---|---|---|---|
| Abutment 1 | No foundation retrofit | | | |
| Pier 2 | 16" Dia. CIDH | 100 | +154.5 | +110.0 |
| Pier 3 | 16" Dia. CIDH | 100 | +154.5 | +110.0 |
| Pier 4 | 16" Dia. CIDH | 100 | +163.5 | +110.0 |
| Pier 5 | 16" Dia. CIDH | 100 | +156.5 | +110.0 |

| | | | | |
|---|---|---|---|---|
| Pier 6 | 16” Dia. CIDH | 100 | +156.5 | +110.0 |
| Pier 7 | 16” Dia. CIDH | 100 | +174.0 | +118.0 |
| Bent 3 | 16” Dia. CIDH | 100 | +153.5 | +100.0 |
| Bent 4 | 16” Dia. CIDH | 100 | +153.5 | +100.0 |
| Bent 5 | 16” Dia. CIDH | 100 | +160.0 | +110.0 |
| Bent 6 | 16” Dia. CIDH | 100 | +155.5 | +100.0 |
| Bent 7 | 16” Dia. CIDH | 100 | +155.5 | +100.0 |
| Bent 8 | 16” Dia. CIDH | 100 | +174.0 | +118.0 |
| Abutment 9 | No foundation retrofit | | | |

1) Actual resistance in nominal compression has not been verified/calculated by OGDS1

Note: It appears that there may be a discrepancy (6.0 to 9.5 ft difference) between Bottom of Footing Elevation for 1967 Widening vs. 2002 Retrofit of the widening. Retrofit bent pile footings are substantially lower. This needs to be verified by Structures Design.

**FOUNDATION RECOMENDATIONS**

The following recommendations are based on 1) the recent geotechnical investigation with lab test results, 2) dated structure plans (General and Foundation Plans, Typical Section, and Abutment 1 Details with proposed left side Retaining Wall, received from September 30, 2009 through September 21, 2010), bridge foundation design loads and design data sheet and Pile Data Table (dated August 23 and September 13 through 23, 2010) with CIDH pile sizes provided by Structures Design, 3) and Layout L-12 (received July 21, 2010).

**Deep Foundations**

The geotechnical capacities of the proposed piles were evaluated using the design loads and pile sizes provided by OBDS2. Computer program SHAFT Version 5.0 and LPILE plus 5.0 were used for calculating CIDH pile/soil resistances and to determine specified pile tip elevations. OGDS1 recommends CIDH piles varying in diameter from 16, 54, and 66 inches to support the left side widening. Pile details and elevations are shown in Tables 10, 11,and 12, below:

General Foundation Information and Design Loads presented in Tables 10 and 11, respectively, were provided by Office of Structure Design 2.

**Table 10 - Foundation Design Data Sheet**

| Support No. | Design Method | Pile Type/Diameter (in) | Finished Grade Elevation (ft) | Cut-off Elevation (ft) | Pile Cap Size | | Permissible Settlement Under Service Load * (in) | Number of Piles per Support |
|---|---|---|---|---|---|---|---|---|
| | | | | | B (ft) | L (ft) | | |
| Abut 1 | WSD | 16 in CIDH (90 kips design load) | 189.81 | 184.75<br>186.25 | 7.0 | 12.5 | 1.0 | 6 |
| Bent 2 | LRFD | 54 in CIDH | 181.50 | 180.0 | N/A | N/A | 1.0 | 1 |
| Bent 3 | LRFD | 66 in CIDH | 166.0 | 164.0 | N/A | N/A | 1.0 | 1 |
| Bent 4 | LRFD | 66 in CIDH | 163.5 | 161.0 | N/A | N/A | 1.0 | 1 |
| Bent 5 | LRFD | 66 in CIDH | 169.0 | 168.0 | N/A | N/A | 1.0 | 1 |
| Bent 6 | LRFD | 66 in CIDH | 168.0 | 167.0 | N/A | N/A | 1.0 | 1 |
| Bent 7 | LRFD | 66 in CIDH | 168.5 | 165.0 | N/A | N/A | 1.0 | 1 |
| Bent 8 | LRFD | 66 in CIDH | 180.5 | 181.0 | N/A | N/A | 1.0 | 1 |
| Bent 9 | LRFD | 54 in CIDH | 184.0 | 183.0 | N/A | N/A | 1.0 | 1 |
| Abut 10 | WSD | 16 in CIDH (90 kips design load) | 198.25 | 193.50 | 10.25 | 13.0 | 1.0 | 6 |

* Based on CALTRANS' current practice, the total permissible settlement is one inch for multi-span structures with continuous spans or multi-column bents, one inch for single span structures with diaphragm abutments, and two inches for single span structures with seat abutments. Different permissible settlement under service loads may be allowed if a structural analysis verifies that required level of serviceability is met.

**Table 11 - Foundation Design Loads**

| Support No. | Service-1 Limit State (kips) | | | Strength Limit State (Controlling Group, kips) | | | | Extreme Event Limit State (Controlling Group, kips) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | Total Load | | Permanent Loads | Compression | | Tension | | Compression | | Tension | |
| | Per Support | Max Per Pile | Per Support | Per Support | Max Per Pile | Per Support | Max Per Pile | Per Support | Max Per Pile | Per Support | Max Per Pile |
| Abut 1 | 493 | 82 | 406 | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A |
| Bent 2 | 663 | N/A | 478 | 1076 | 1076 | 0 | 0 | 415 | 415 | 0 | 0 |
| Bent 3 | 761 | N/A | 558 | 1237 | 1237 | 0 | 0 | 461 | 461 | 0 | 0 |
| Bent 4 | 750 | N/A | 548 | 1217 | 1217 | 0 | 0 | 479 | 479 | 0 | 0 |
| Bent 5 | 692 | N/A | 506 | 1109 | 1109 | 0 | 0 | 421 | 421 | 0 | 0 |
| Bent 6 | 823 | N/A | 629 | 1284 | 1284 | 0 | 0 | 461 | 461 | 0 | 0 |
| Bent 7 | 680 | N/A | 500 | 1071 | 1071 | 0 | 0 | 399 | 399 | 0 | 0 |
| Bent 8 | 735 | N/A | 536 | 1179 | 1179 | 0 | 0 | 428 | 428 | 0 | 0 |
| Bent 9 | 650 | N/A | 460 | 1053 | 1053 | 0 | 0 | 392 | 392 | 0 | 0 |
| Abut 10 | 300 | 83 | 233 | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A |

The Abutment Foundation Design Recommendations (Table 12, 16 inch diameter CIDH piles), Bent Foundation Design Recommendations (Table 13, both 54 and 66 inch diameter CIDH piles), and the Pile Data Table 14, prepared by OGDS1, are presented below.

**Table 12 – Abutment Foundation Design Recommendations for SB 710/5 Separation (Lt side Wdn)**

| Support Location | Pile Type | Finished Grade Elevation (ft) | Cut-off Elevation (ft) | LRFD Service-1 Limit State Load (kips) Per Support | | LRFD Service-1 Limit State Total Load (kips) per Pile (Compression) | Nominal Resistance in Compression (kips) | Design Pile Tip Elevations (ft) | Specified Pile Tip Elevation (ft) |
|---|---|---|---|---|---|---|---|---|---|
| | | | | Total | Permanent | | | | |
| Abut 1 | 16" CIDH (90 kips design load) | 189.81 | 184.75<br>186.25 | 500 | 406 | 90 | 180 | 139.0 (a)<br>154.0 (c)<br>159.0 (d) | 139.0 |
| Abut 10 | 16" CIDH (90 kips design load) | 198.25 | 193.50 | 300 | 233 | 90 | 180 | 152.0 (a)<br>170.0 (c)<br>167.0 (d) | 152.0 |

Notes:
1) Design pile tip elevations are controlled by: (a) Compression , (c) Settlement, and (d) Lateral Load, respectively.
2) The CIDH specified tip elevation shall not be raised.
3) Design pile tip elevation for Lateral Load is typically provided by SD but was calculated by OGDS1 upon request.

**Table 13 – Bent Foundation Design Recommendations for SB 710/5 Separation (Lt side Wdn)**

| **Support Location** | **Pile Type/ Diameter** | **Cut-off Elevation (ft)** | **Service-I Limit State Load per Support (kips)** | **Total Permissible Support Settlement (inches)** | **Required Factored Nominal Resistance (kips)** | | | | **Design Tip Elevations (ft)** | **Specified Tip Elevation (ft)** |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | **Strength Limit** | | **Extreme Event** | | | |
| | | | | | **Comp. (φ=0.7)** | **Tension (φ=0.7)** | **Comp. (φ=1)** | **Tension (φ=1)** | | |
| Bent 2 | 54 in CIDH | 180.0 | 670 | 1 | 1076/0.7= 1537 | N/A | N/A | N/A | 79.0 (a-I)<br>141.0 (c)<br>121.0 (d) | 79.0 |
| Bent 3 | 66 in CIDH | 164.0 | 770 | 1 | 1237/0.7= 1767 | N/A | N/A | N/A | 64.0 (a-I)<br>100.0 (c)<br>96.0 (d) | 64.0 |
| Bent 4 | 66 in CIDH | 161.0 | 750 | 1 | 1217/0.7= 1739 | N/A | N/A | N/A | 65.0 (a-I)<br>120.0 (c)<br>91.0 (d) | 65.0 |
| Bent 5 | 66 in CIDH | 168.0 | 700 | 1 | 1109/0.7= 1584 | N/A | N/A | N/A | 78.0 (a-I)<br>129.0 (c)<br>97.0 (d) | 78.0 |
| Bent 6 | 66 in CIDH | 167.0 | 830 | 1 | 1248/0.7= 1783 | N/A | N/A | N/A | 68.0 (a-I)<br>126.0 (c)<br>92.0 (d) | 68.0 |

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Bent 7 | 66 in CIDH | 165.0 | 680 | 1 | 1071/0.7= 1530 | N/A | N/A | N/A | 80.0 (a-I)<br>125.0 (c)<br>93.0 (d) | 80.0 |
| Bent 8 | 66 in CIDH | 181.0 | 740 | 1 | 1179/0.7= 1684 | N/A | N/A | N/A | 88.0 (a-I)<br>135.0 (c)<br>108.0 (d) | 88.0 |
| Bent 9 | 54 in CIDH | 183.0 | 650 | 1 | 1053/0.7= 1504 | N/A | N/A | N/A | 78.0 (a-I)<br>129.0 (c)<br>119.0 (d) | 78.0 |

Notes:
1) Design tip elevations are controlled by: (a-I) Compression (Strength Limit), (c) Settlement, and (d) Lateral Load, respectively.
2) The CIDH specified pile tip elevation shall not be raised.
3) Design pile tip elevation for Lateral Load is typically provided by SD but was calculated by OGDS1 upon request.

**Table 14 – Pile Data Table for SB 710/5 Separation (Lt side Wdn), Br. No. 53-0788L**

| Location | Pile Type | Nominal Resistance (kips) | | Design Pile Tip Elevation (ft) | Specified Pile Tip Elevation (ft) |
|---|---|---|---|---|---|
| | | Compression | Tension | | |
| Abut 1 | 16” CIDH | 180 | N/A | 139.0 (a)<br>154.0 (c)<br>159.0 (d) | 139.0 |
| Bent 2 | 54” CIDH | 1540 | N/A | 79.0 (a-I)<br>141.0 (c)<br>121.0 (d) | 79.0 |
| Bent 3 | 66” CIDH | 1770 | N/A | 64.0 (a-I)<br>100.0 (c)<br>96.0 (d) | 64.0 |
| Bent 4 | 66” CIDH | 1740 | N/A | 65.0 (a-I)<br>120.0 (c)<br>91.0 (d) | 65.0 |
| Bent 5* | 66” CIDH | 1590 | N/A | 78.0 (a-I)<br>129.0 (c)<br>97.0 (d) | 78.0 |
| Bent 6 | 66” CIDH | 1790 | N/A | 68.0 (a-I)<br>126.0 (c)<br>92.0 (d) | 68.0 |
| Bent 7 | 66” CIDH | 1530 | N/A | 80.0 (a-I)<br>125.0 (c)<br>93.0 (d) | 80.0 |
| Bent 8* | 66” CIDH | 1690 | N/A | 88.0 (a-I)<br>135.0 (c)<br>108.0 (d) | 88.0 |

| | | | | | |
|---|---|---|---|---|---|
| Bent 9* | 54" CIDH | 1510 | N/A | 78.0 (a-I)<br>129.0 (c)<br>119.0 (d) | 78.0 |
| Abut 10 | 16" CIDH | 180 | N/A | 152.0 (a)<br>170.0 (c)<br>167.0 (d) | 152.0 |

Notes:
1) Design tip elevations for Abutments are controlled by: (a) Compression, (c) Settlement, (d) Lateral Load
2) Design tip elevations for Bents are controlled by: (a-I) Compression (Strength Limit), (c) Settlement, (d) Lateral Load
3) The CIDH specified tip elevation shall not be raised.
4) * = Isolation casing proposed at Bents 5, 8, and 9. Isolation casing bottom tip elevations assumed at +163.0 (Bent 5), +174.5 (Bent 8), and +180.0 (Bent 9) from information supplied by Structures Design.

Pile lengths were computed based on the provided design loads and the observed subsurface conditions. The analysis was conducted based on the assumption that proposed CIDH piles would derive geotechnical capacity from skin friction only (100%).

**Abutment 1 (Left Side) Retaining Wall**

Abutment 1 (Left Side) Retaining Wall Plans (Abutment Details No. 3) showing pile footing footprint, elevation, and typical section were received September 8, 2010, along with pile data table information. The plan shows the proposed Abutment 1 (Left Side) Retaining Wall is 53 ft 9 in length and varies from 14 to 6 ft height. The retaining wall is Type 1. CIDH piles, 16 in diameter (90 kip design load) are recommended to support this retaining wall as shown in Table 15 below.

**Table 15 – Pile Data for Retaining Wall at Abutment 1 (Left Side), SB 710/5 Separation Widen**

| Wall Location/Design Height (ft) | Pile Type/Diameter (in) | Cut-off Elevation (ft) | Service Load per Pile (kips) | Required Nominal Resistance in Compression (kips) | Specified Pile Tip Elevation (ft) |
|---|---|---|---|---|---|
| **Abut 1 Lt RW**<br>H=14 | 16 in CIDH | 185.50 | 90 | 180 | +140.0 |
| H=10 | 16 in CIDH | 188.75 | 90 | 180 | +143.0 |
| H=8 | 16 in CIDH | 191.25 | 90 | 180 | +146.0 |
| H=6 | 16 in CIDH | 194.25 | 90 | 180 | +149.0 |

A minimum cover of 1.5 ft is required over the retaining wall footing.

**Settlement at Approach Fills**

Fills can be placed in accordance with Section 19-6 of the Caltrans Standard Specifications (May 2006). End dumping is not permitted. At Abutments 1 and 10 areas, additional sliver fills are estimated to range from 24 to 27 ft height. Calculated maximum settlement (Hough's Method) is up

to 2 inches at Abutments 1 and 10. OGDS1 recommends a fill settlement waiting period of up to 30 days for the widening. However, the actual settlement period will be determined by the structure representative on the basis of settlement data in the field and the above waiting period may be adjusted accordingly.

## NOTE TO DESIGNER

The recommendations contained in this report are based on specific project information regarding bridge location, pile type, finished grade elevation, cut-off elevation and service load that has been provided to our office. If any conceptual changes are made during final project design, this office should review those changes to determine if these included foundation recommendations are still applicable.

## CONSTRUCTION CONSIDERATIONS

1. Design groundwater is considered to be at approximate elevation +59.2 ft. All CIDH piles terminate above design groundwater elevation but no overdrilling should be allowed so that piles can be constructed under dry conditions. CIDH pile construction, during typical low water months (such as summer) is recommended by OGDS1.
2. The contractor shall be required to clean out the bottom of the CIDH pile boring prior to placing the cage and the concrete.
3. OGDS1 recommends concrete placement for construction of the CIDH piling shall be completed within the same day that excavation of the drilled hole has been completed. Otherwise, a suitable alternative must be proposed to control caving if concrete placement cannot be completed the same day as drilling.
4. Moderate to minor caving and sloughing should be anticipated during excavation of the pile boring and during CIDH pile construction.
5. All earthwork is expected to be carried out by conventional equipment. Fill should be placed as specified in Section 19-6 of the Caltrans Standard Specifications (May 2006). If imported materials are used to construct the new fill embankment, the material should be tested during grading to assess expansion potential. Only non-expansive soils or soils having a low expansion potential (EI<50) should be used in the Soil Expansion Exclusion Zone in bridge approach embankment and within 10 ft of the roadbed sub-grade elevation.

If you have any questions or need further information, please contact Kevin Lai at (213) 620-2344 or Shiva Karimi at (213) 620-2146.

Prepared by: Date: 09/23/10

KEVIN LAI,
Transportation Engineer
Office of Geotechnical Design – South 1
Branch D

Supervi /23/10

SHIVA KARIMI, C
Senior Transportation
Office of Geotechnical D
Branch D

Reviewed by: Dated: 09/23/10

JOE PRATT, C.E.G. NO. 2141
Engineering Geologist
Office of Geotechnical Design South 1
Branch D

cc:

| | | |
|---|---|---|
| District Project Manager | James S. Hsu | James_s_Hsu@dot.ca.gov |
| GS Corporate | Mark Willian | Mark_Willian@dot.ca.gov |
| District Construction R.E. Pending File | | TBD |
| Structure Construction R.E. Pending File | | RE_Pending_File@dot.ca.gov |
| DES Office Engineer, Office of PS&E | Efren Ancheta | Efren_Ancheta@dot.ca.gov |
| District Materials Engineer | Kirsten Stahl | Kirsten_Stahl@dot.ca.gov |

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State of California Department of Transportation
Division of Engineering Services
Office of Design & Technical Services

Structure Hydraulics

# FINAL HYDRAULIC REPORT

## Los Angeles River Bridge

EA 07-202101

Located on State Route 710 in Los Angeles County

JOB:
Bridge No. 53-0828,

LOCATION:
07-LA-710-PM 17.3

PREPARED BY (Signature)

REGISTERED PROFESSIONAL ENGINEER
GINGER LU
C 71324
EXP. 12/31/09
CIVIL
STATE OF CALIFORNIA

Ginger Lu, PE# 71324
Structure Hydraulics & Scour Mitigation
September 25, 2009

*This report has been prepared under my direction as the professional engineer in responsible charge of the work, in accordance with the provisions of the Professional Engineers Act of the State of California.*

# Hydrology/Hydraulics Report

## General:

Structure Design is proposing to widen the existing Los Angeles River Bridge located at PM 17.3 on State Route 710 in the city of South Gate. The bridge will be widened by roughly 11' on the upstream side and roughly 15' on the downstream side of the bridge, and the superstructure will be widened in kind. Based on the May 2008 Bridge Inspection Report, the NBIS Item 113 code regarding bridge's vulnerability to scour was 8, "bridge foundations determined to be stable for the assessed or calculated scour condition."

The existing 4-span bridge was built in 1954, widened at the median in 1966, and retrofitted in both 1993 and 1997. It is a 574.4' long 10' thick continuous riveted and welded non-composite-plate girders structure with a structure width of 142'. The supporting elements are concrete pier walls on piles with pile caps and seated abutments on piles. The widening portion will have the same structure depth. The existing pier walls are roughly 177' long by 3.5' thick. After the proposed widening the pier walls will be roughly 203' long by 3.5' thick.

All the calculated values in this report are based geometric information on the 5/26/09 survey data (NAVD 1988). The General Plans provided on 7/9/09 by the Office of Structure Design, Caltrans Bridge Maintenance Records and As-Built plans (NGVD 1929) were reviewed. This evaluation makes reference to the updated Flood Insurance Study (9/26/08) for Los Angeles County by the Federal Emergency Management Agency (FEMA) and Los Angeles County Drainage Area Final Report (LACDA, 7/05) by US Army Corps of Engineers (USACE), Los Angeles District.

## Basin:

The Los Angeles River watershed covers an area of over 824 square miles ($mi^2$) from its headwaters around Elevation 1000' in the mountains eastward to northern corner of Griffith Park through Glendale Narrows into San Pedro Bay near Long Beach. Major tributaries for the upper river are, Burbank Western Channel, Burbank Eastern Channel, Sycamore Wash, Tujunga Wash, Verdugo Wash, Arroyo Seco, and Caballero Creek. The drainage area near the project site was determined to be 599 $mi^2$ according to the USGS stream gage #11098500 (roughly 1 mile upstream of the bridge).

The Los Angeles River channel was mostly concrete lined around 1970 to control the runoff and reduce impacts of flood events. The three unlined stretches are within the Sepulveda Flood Control Basin through the Glendale Narrows, and South of Willow Street in Long Beach. The basin receives on an average of 12-14 inches of annual rainfall.

**Discharge:**

The 1983 Flood Insurance Study - City of Long Beach by FEMA reported that Los Angeles River at Wardlow Road had the 100-year discharge of 121,000 cubic feet per second (cfs). The latest discharge for the upper reach of Los Angeles River was refined to 120,000 cfs at Station 685+00 south of Firestone Blvd and between Steward & Gray Rd and Rio Hondo Channel by USACE in LACDA (2005).

**Stage/Velocity:**

Using the survey data (5/26/09) and the HEC-RAS model of the upper Los Angeles River (LACDA) reach obtained from USACE, a second HEC-RAS model of 1754' segment of the upper LAR reach between River Station (RS) 655+69.22' and 638+15' was constructed to simulate the widening effect. Roughness coefficient of 0.016 for a concrete lined channel and the known water surface elevation (WSEL) of 109.06' extracted from the LACDA model were incorporated into the segment model. In order to use the known WSEL of 109.06' as the upstream and downstream boundary control, a total of six cross-sections from RS 649+98.32 to RS 638+15.0 were brought from the LACDA model into the segment model. Existing and proposed conditions were simulated to compare their outcomes. Since the channel is straight, zero value was entered for the contraction and expansion coefficients in the LACDA model, and was used in the segment model as well. Parameters such as the bridge geometry and numbers of pier walls perceived by one single RS used by USACE are different from the segment model. The water surface elevations (WSELs) at the upstream face (RS 654+99.33) and the downstream face of the bridge (RS 649+98.32) calculated by the LACDA model and by the segment model are tabulated as follows.

| ***Hydrologic Summary for the Existing Condition*** | | | | |
|---|---|---|---|---|
| *Drainage Area: 599 mi²* | | | | |
| *100-year Event ($Q_{100}$)* | *LACDA HEC-RAS* | | *Segment HEC-RAS* | |
| *Discharge (cfs)* | 120,000 | | 120,000 | |
| *Existing Soffit Elev. (ft)* | 115.15 | | 111.9 | |
| *River Stations (RS, ft)* | 654+99.33 | 649+98.32 | 654+99.33 | 649+98.32 |
| *WSEL (ft)* | 109.13 | 107.99 | 109.14 | 108.00 |
| *Average Velocity (ft/s)* | 14.44 | 14.44 | 14.43 | 14.44 |
| *Flood plain data are based upon information available when the plans were prepared and are shown to meet federal requirements. The accuracy of said information is not warranted by the State and interested or affected parties should make their own investigation.* | | | | |

**Streambed/Scour & Drift:**

The channel bottom is concrete lined and the bank slopes are riprap grouted. It appears to have little vegetation along the banks, and the concrete lining of the channel bottom seems to be in a fair condition. Due to the setting of the bridge being in the middle of the urban area, a

debris flow of 7.6' wide and 6' high was built into the LACDA model by USACE and to the segment model as well. The existing structure soffit at Elevation 111.9' should pass the 100-year flow with more than 2' freeboard.

**Summary & Recommendation:**

- Due to the concrete lined nature of the channel, the scour potential for the bridge foundation is minimal.
- According to the segment model, widening the pier walls by 15' downstream and 11' upstream would have raised the water surface elevation by one-tenth of a foot. The minimal soffit elevation at the lowest point of the proposed widening part should not be lower than Elevation 109.9'.

| *Condition ($Q_{100}$)* | | *Existing Condition* | *Proposed Condition* |
|---|---|---|---|
| *Structure Soffit Elev. (ft)* | | 111.9 | 109.9 |
| *RS* | *WSEL (ft)* | 109.14 | 109.25 |
| *654+99.33* | *Ave Velocity (ft/s)* | 14.43 | 14.36 |
| *RS* | *WSEL (ft)* | 108.0 | 108.0 |
| *649+98.32* | *Ave. Velocity (ft/s)* | 14.44 | 14.44 |

# STRUCTURE DESIGN CALCULATIONS

## LA River Bridge Widening

Los Angeles County - Route 710 – PM 17.30

By Frank Wei, Lorena Sanchez-Manzo

REGISTERED PROFESSIONAL ENGINEER
FRANK HSIAOSHIH WEI
No. C-052316
EXP. 12/31/10
CIVIL
STATE OF CALIFORNIA

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```
>>>> Describe Chains <<<< [Thu Apr 02 18:33:37 2009]
*******************************************************************************
CHAIN 710STH1
List          C STCC7,C ,STCC8,C ,STCC9,C ,STCC10,PTSTNC3,C ,STCC11,C ,STCC12
List          ,C ,STCC13,C ,STCC14,C ,STCC15,C ,STCC16,C ,STCC17,C ,STCC18,C
List           ,STCC19,C ,STCC20,C ,STCC21,C ,STCC22,C ,STCC23,C ,STCC24,C
List          ,STCC25,INT1,C ,STIC1
Description   C/L LA 710
Layer         59
Feature       CL
*******************************************************************************
CURVE STCC7
        BC N 1786804.1863 E 6505137.5668        STA 784+75.32868
        CC N 1785848.6238 E 6507981.3143
        PI N 1787539.6393 E 6505384.6954        STA 792+51.19184
                TAN 775.863'
                DB N 18°34'24.6" E
                DA N 47°34'25.9" E
                LChord  Distance     1502.299'   Bearing  N 33°04'25.3" E
                External Distance 98.704'
                Middle Ordinate 95.560'
                Radius 3000.000'
                DEG   1°54'35.5"
                DELTA  29°00'01.3"
                LENGTH 1518.456'
        EC N 1788063.0670 E 6505957.3970        STA 799+93.78463
CURVE STCC7
        Distance      530.329'   Bearing  N 47°34'26.0" E
CURVE STCC8
        BC N 1788420.8475 E 6506348.8584        STA 805+24.11369
        CC N 1790266.2179 E 6504662.2619
        PI N 1788930.2077 E 6506906.1690        STA 812+79.12554
                TAN 755.012'
                DB N 47°34'26.1" E
                DA N 13°57'53.2" E
                LChord  Distance     1445.540'   Bearing  N 30°46'09.6" E
                External Distance 111.521'
                Middle Ordinate 106.759'
                Radius 2500.000'
                DEG   2°17'30.6"
                DELTA -33°36'32.9"
                LENGTH 1466.475'
        EC N 1789662.9046 E 6507088.3725        STA 819+90.58864
CURVE STCC8
        Distance      974.038'   Bearing  N 13°57'53.2" E
CURVE STCC9
        BC N 1790608.1547 E 6507323.4325        STA 829+64.62708
        CC N 1791573.4559 E 6503441.6555
        PI N 1790898.6288 E 6507395.6662        STA 832+63.94788
                TAN 299.321'
                DB N 13°57'53.2" E
                DA N 5°24'20.9" E
                LChord  Distance      596.973'   Bearing  N 9°41'07.1" E
                External Distance 11.183'
                Middle Ordinate 11.152'
                Radius 4000.000'
                DEG   1°25'56.6"
                DELTA  -8°33'32.2"
                LENGTH 597.528'
        EC N 1791196.6184 E 6507423.8650        STA 835+62.15506
CURVE STCC9
```

```
        Distance     3477.005'   Bearing  N 5°24'20.9" E
CURVE STCC10
        BC N 1794658.1593 E 6507751.4309        STA 870+39.16026
        CC N 1794940.7875 E 6504764.7738
        PI N 1795231.1065 E 6507805.6491        STA 876+14.66707
                TAN 575.507'
                DB N 5°24'20.9" E
                DA N 16°18'47.0" W
                LChord  Distance     1130.402'   Bearing  N 5°27'13.0" W
                External Distance 54.703'
                Middle Ordinate 53.723'
                Radius 3000.000'
                DEG   1°54'35.5"
                DELTA -21°43'08.0"
                LENGTH 1137.198'
        EC N 1795783.4442 E 6507643.9975        STA 881+76.35823
CURVE STCC10
        Distance       29.484'   Bearing  N 16°18'41.8" W
PTSTNC3 N 1795811.7413 E 6507635.7166 Z -99999.990      STA 882+05.84217
        Distance      459.490'   Bearing  N 16°18'47.5" W
CURVE STCC11
        BC N 1796252.7322 E 6507506.6516        STA 886+65.33182
        CC N 1796954.9477 E 6509906.0042
        PI N 1797838.7689 E 6507042.4683        STA 903+17.89909
                TAN 1652.567'
                DB N 16°18'47.2" W
                DA N 50°37'06.4" E
                LChord  Distance     2757.194'   Bearing  N 17°09'09.6" E
                External Distance 496.828'
                Middle Ordinate 414.462'
                Radius 2500.000'
                DEG   2°17'30.6"
                DELTA  66°55'53.6"
                LENGTH 2920.440'
        EC N 1798887.2925 E 6508319.8002        STA 915+85.77173
CURVE STCC11
        Distance      646.639'   Bearing  N 50°37'06.5" E
CURVE STCC12
        BC N 1799297.5727 E 6508819.6121        STA 922+32.41048
        CC N 1801229.9185 E 6507233.4093
        PI N 1799942.2722 E 6509604.9987        STA 932+48.51557
                TAN 1016.105'
                DB N 50°37'06.6" E
                DA N 6°22'50.5" E
                LChord  Distance     1882.649'   Bearing  N 28°29'58.6" E
                External Distance 198.605'
                Middle Ordinate 183.989'
                Radius 2500.000'
                DEG   2°17'30.6"
                DELTA -44°14'16.0"
                LENGTH 1930.237'
        EC N 1800952.0829 E 6509717.9227        STA 941+62.64792
CURVE STCC12
        Distance     2356.399'   Bearing  N 6°22'50.5" E
CURVE STCC13
        BC N 1803293.8849 E 6509979.7985        STA 965+19.04677
        CC N 1803682.8548 E 6506501.4797
        PI N 1803588.7934 E 6510012.7772        STA 968+15.79351
                TAN 296.747'
                DB N 6°22'50.5" E
                DA N 3°18'42.2" W
                LChord  Distance      591.372'   Bearing  N 1°32'04.1" E
                External Distance 12.557'
```

1

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                Middle Ordinate 12.512'
                Radius 3500.000'
                DEG   1°38'13.3"
                DELTA  -9°41'32.8"
                LENGTH 592.077'
        EC N 1803885.0446 E 6509995.6346        STA 971+11.12425
CURVE STCC13
        Distance      972.167'   Bearing  N 3°18'42.3" W
CURVE STCC14
        BC N 1804855.5878 E 6509939.4738        STA 980+83.29100
        CC N 1805664.3561 E 6523916.0932
        PI N 1805650.9947 E 6509893.4469        STA 988+80.02846
                TAN 796.737'
                DB N 3°18'42.4" W
                DA N 3°12'09.3" E
                LChord  Distance     1590.901'   Bearing  N 0°03'16.5" W
                External Distance 22.653'
                Middle Ordinate 22.616'
                Radius 14000.000'
                DEG   0°24'33.3"
                DELTA   6°30'51.7"
                LENGTH 1591.758'
        EC N 1806446.4878 E 6509937.9579        STA 996+75.04899
CURVE STCC14
        Distance        0.047'   Bearing  N 3°12'08.0" E
CURVE STCC15
        BC N 1806446.5346 E 6509937.9605        STA 996+75.09582
        CC N 1805440.9187 E 6527909.8480
        PI N 1807082.3125 E 6509973.5354        STA 1003+11.86826
                TAN 636.772'
                DB N 3°12'09.5" E
                DA N 7°15'17.2" E
                LChord  Distance     1272.749'   Bearing  N 5°13'43.3" E
                External Distance 11.260'
                Middle Ordinate 11.253'
                Radius 18000.000'
                DEG   0°19'05.9"
                DELTA   4°03'07.7"
                LENGTH 1273.014'
        EC N 1807713.9872 E 6510053.9481        STA 1009+48.10983
CURVE STCC15
        Distance      991.774'   Bearing  N 7°15'17.1" E
CURVE STCC16
        BC N 1808697.8214 E 6510179.1903        STA 1019+39.88363
        CC N 1808318.9766 E 6513155.1736
        PI N 1809082.5583 E 6510228.1676        STA 1023+27.72543
                TAN 387.842'
                DB N 7°15'17.2" E
                DA N 21°59'15.1" E
                LChord  Distance      769.282'   Bearing  N 14°37'16.1" E
                External Distance 24.966'
                Middle Ordinate 24.760'
                Radius 3000.000'
                DEG   1°54'35.5"
                DELTA  14°43'57.9"
                LENGTH 771.405'
        EC N 1809442.1906 E 6510373.3774        STA 1027+11.28860
CURVE STCC16
        Distance      440.261'   Bearing  N 21°59'14.9" E
CURVE STCC17
        BC N 1809850.4294 E 6510538.2128        STA 1031+51.54946
        CC N 1811348.0467 E 6506829.1507
        PI N 1810784.3907 E 6510915.3207        STA 1041+58.77043
```

```
                DEG   2°51'53.2"
                DELTA  76°26'21.0"
                LENGTH 2668.230'
        EC N 1821994.3908 E 6508286.4088        STA 1162+21.81982
CURVE STCC24
        Distance      742.778'   Bearing  N 29°33'43.1" E
CURVE STCC25
        BC N 1822640.4759 E 6508652.8693        STA 1169+64.59795
        CC N 1825600.6669 E 6503433.9354
        PI N 1824019.2844 E 6509434.9325        STA 1185+49.75899
                TAN 1585.161'
                DB N 29°33'43.2" E
                DA N 0°02'09.9" W
                LChord  Distance     3065.155'   Bearing  N 14°45'46.7" E
                External Distance 205.863'
                Middle Ordinate 199.034'
                Radius 6000.000'
                DEG   0°57'17.7"
                DELTA -29°35'53.1"
                LENGTH 3099.504'
        EC N 1825604.4451 E 6509433.9344        STA 1200+64.10202
CURVE STCC25
        Distance     1515.972'   Bearing  N 0°02'10.1" W
INT1 N 1827120.4173 E 6509432.9785 Z 0.000      STA 1215+80.07452
        Distance      156.608'   Bearing  N 0°02'09.6" W
CURVE STIC1
        BC N 1827277.0252 E 6509432.8801        STA 1217+36.68243
        CC N 1827273.2025 E 6503431.8676
        PI N 1827462.3112 E 6509432.7620        STA 1219+21.96857
                TAN 185.286'
                DB N 0°02'11.4" W
                DA N 3°34'24.5" W
                LChord  Distance      370.396'   Bearing  N 1°48'18.0" W
                External Distance 2.860'
                Middle Ordinate 2.858'
                Radius 6001.014'
                DEG   0°57'17.2"
                DELTA  -3°32'13.1"
                LENGTH 370.455'
        EC N 1827647.2371 E 6509421.2134        STA 1221+07.13701
CURVE STIC1
*******************************************************************************
CHAIN LENGTH 43631.808'
*******************************************************************************

>>>> End of Describe Chains <<<<
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```
                TAN 1007.221'
                DB N 21°59'15.0" E
                DA N 6°16'46.6" W
                LChord  Distance     1953.463'   Bearing  N 7°51'14.2" E
                External Distance 124.863'
                Middle Ordinate 121.083'
                Radius 4000.000'
                DEG   1°25'56.6"
                DELTA -28°16'01.6"
                LENGTH 1973.416'
        EC N 1811785.5683 E 6510805.1505        STA 1051+24.96540
CURVE STCC17
        Distance      391.777'   Bearing  N 6°16'46.8" W
CURVE STCC18
        BC N 1812174.9945 E 6510762.2974        STA 1055+16.74234
        CC N 1812503.1377 E 6513744.2970
        PI N 1812518.2553 E 6510724.5245        STA 1058+62.07510
                TAN 345.333'
                DB N 6°16'46.7" W
                DA N 6°51'11.9" E
                LChord  Distance      686.135'   Bearing  N 0°17'12.6" E
                External Distance 19.810'
                Middle Ordinate 19.680'
                Radius 3000.000'
                DEG   1°54'35.5"
                DELTA  13°07'58.6"
                LENGTH 687.639'
        EC N 1812861.1206 E 6510765.7323        STA 1062+04.38132
CURVE STCC18
        Distance      830.383'   Bearing  N 6°51'11.9" E
CURVE STCC19
        BC N 1813685.5703 E 6510864.8199        STA 1070+34.76417
        CC N 1814520.8205 E 6503915.1747
        PI N 1814561.4481 E 6510970.0882        STA 1079+16.94514
                TAN 882.181'
                DB N 6°51'11.8" E
                DA N 7°30'47.4" W
                LChord  Distance     1750.514'   Bearing  N 0°19'47.8" W
                External Distance 55.373'
                Middle Ordinate 54.938'
                Radius 6999.658'
                DEG   0°49'06.8"
                DELTA -14°21'59.3"
                LENGTH 1755.108'
        EC N 1815436.0553 E 6510854.7393        STA 1087+89.87239
CURVE STCC19
        Distance        0.038'   Bearing  N 7°30'42.1" W
CURVE STCC20
        BC N 1815436.0933 E 6510854.7343        STA 1087+89.91073
        CC N 1814782.3207 E 6505897.6605
        PI N 1815789.6122 E 6510808.1098        STA 1091+46.49090
                TAN 356.580'
                DB N 7°30'47.5" W
                DA N 15°40'17.8" W
                LChord  Distance      711.354'   Bearing  N 11°35'32.6" W
                External Distance 12.699'
                Middle Ordinate 12.667'
                Radius 5000.000'
                DEG   1°08'45.3"
                DELTA  -8°09'30.3"
                LENGTH 711.955'
        EC N 1816132.9368 E 6510711.7892        STA 1095+01.86571
CURVE STCC20
```

```
        Distance      476.771'   Bearing  N 15°40'17.7" W
CURVE STCC21
        BC N 1816591.9842 E 6510583.0025        STA 1099+78.63669
        CC N 1815241.3730 E 6505768.8725
        PI N 1816935.3016 E 6510486.6843        STA 1103+35.20932
                TAN 356.573'
                DB N 15°40'17.5" W
                DA N 23°49'47.2" W
                LChord  Distance      711.339'   Bearing  N 19°45'02.4" W
                External Distance 12.698'
                Middle Ordinate 12.666'
                Radius 5000.000'
                DEG   1°08'45.3"
                DELTA  -8°09'29.6"
                LENGTH 711.940'
        EC N 1817261.4764 E 6510342.6216        STA 1106+90.57666
CURVE STCC21
        Distance        0.085'   Bearing  N 23°49'49.0" W
CURVE STCC22
        BC N 1817261.5545 E 6510342.5871        STA 1106+90.66210
        CC N 1814562.8782 E 6504232.4698
        PI N 1818083.0472 E 6509979.7556        STA 1115+88.71377
                TAN 898.052'
                DB N 23°49'47.2" W
                DA N 39°08'40.3" W
                LChord  Distance     1780.087'   Bearing  N 31°29'13.8" W
                External Distance 60.100'
                Middle Ordinate 59.564'
                Radius 6679.550'
                DEG   0°51'28.0"
                DELTA -15°18'53.1"
                LENGTH 1785.397'
        EC N 1818779.5365 E 6509412.8346        STA 1124+76.05905
CURVE STCC22
        Distance        0.082'   Bearing  N 39°08'37.0" W
CURVE STCC23
        BC N 1818779.6004 E 6509412.7826        STA 1124+76.14144
        CC N 1815624.3218 E 6505536.3731
        PI N 1819041.5783 E 6509199.5407        STA 1128+13.93508
                TAN 337.794'
                DB N 39°08'40.3" W
                DA N 46°52'37.8" W
                LChord  Distance      674.050'   Bearing  N 43°00'39.1" W
                External Distance 11.401'
                Middle Ordinate 11.376'
                Radius 4998.233'
                DEG   1°08'46.8"
                DELTA  -7°43'57.5"
                LENGTH 674.562'
        EC N 1819272.4821 E 6508952.9885        STA 1131+50.70298
CURVE STCC23
        Distance      402.887'   Bearing  N 46°52'37.9" W
CURVE STCC24
        BC N 1819547.8812 E 6508658.9253        STA 1135+53.58991
        CC N 1821007.6616 E 6510026.0540
        PI N 1820624.4621 E 6507509.3834        STA 1151+28.54164
                TAN 1574.952'
                DB N 46°52'37.9" W
                DA N 29°33'43.1" E
                LChord  Distance     2474.708'   Bearing  N 8°39'27.4" W
                External Distance 545.677'
                Middle Ordinate 428.709'
                Radius 2000.000'
```

| | SOUTHBOUND | | | | | | | | NORTHBOUND | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Cross Slope | Along Inside Face of Barrier | | Along Saw Cut Line | | Reference Point | | CL | Reference point | | Along Saw Cut Line | | Along Inside face of Barrier | | Cross Slope |
| | | Elevation | Offset From CL | Elevation | Offset From CL | Elevation | Offset From CL | Elevation | Offset From CL | Elevation | Offset From CL | Elevation | Offset From CL | Elevation | |
| Station: 914+00.00 | 4.06% | 121.04 | -71.57 | 120.43 | -56.41 | 119.11 | -24 | 118.56 | 24 | 118.08 | 53.39 | 117.18 | 77.95 | 116.43 | -3.07% |
| Station: 914+25.00 | 3.88% | 121.35 | -70.55 | 120.81 | -56.41 | 119.55 | -24 | 119.06 | 24 | 118.59 | 53.39 | 117.74 | 77.53 | 117.03 | -2.92% |
| Station: 914+50.00 | 3.67% | 121.67 | -69.78 | 121.18 | -56.41 | 119.99 | -24 | 119.56 | 24 | 119.09 | 53.39 | 118.29 | 76.86 | 117.65 | -2.73% |
| Station: 914+57.71 | 3.60% | 121.76 | -69.6 | 121.29 | -56.41 | 120.12 | -24 | 119.71 | 24 | 119.25 | 53.39 | 118.47 | 76.61 | 117.85 | -2.65% |
| Station: 914+75.00 | 3.40% | 121.99 | -69.28 | 121.55 | -56.41 | 120.45 | -24 | 120.05 | 24 | 119.58 | 53.39 | 118.85 | 76.5 | 118.27 | -2.51% |
| Station: 914+87.22 | 3.26% | 122.15 | -69.12 | 121.74 | -56.41 | 120.68 | -24 | 120.28 | 24 | 119.83 | 53.39 | 119.09 | 76.35 | 118.51 | -2.52% |
| Station: 915+00.00 | 3.11% | 122.33 | -69.03 | 121.94 | -56.41 | 120.93 | -24 | 120.53 | 24 | 120.12 | 53.86 | 119.4 | 76.13 | 118.86 | -2.42% |
| Station: 915+25.00 | 2.84% | 122.69 | -69 | 122.33 | -56.41 | 121.41 | -24 | 121.07 | 24 | 120.72 | 54.6 | 119.95 | 75.52 | 119.43 | -2.52% |
| Station: 915+46.88 | 2.71% | 123.09 | -69 | 122.75 | -56.38 | 121.87 | -24 | 121.73 | 24 | 121.21 | 55.04 | 120.43 | 74.78 | 119.94 | -2.51% |
| Station: 915+50.00 | 2.66% | 123.13 | -69 | 122.8 | -56.33 | 121.94 | -24 | 121.79 | 24 | 121.28 | 55.09 | 120.5 | 74.66 | 120.01 | -2.50% |
| Station: 915+75.00 | 2.21% | 123.49 | -69 | 123.21 | -56.07 | 122.5 | -24 | 122.27 | 24 | 121.88 | 55.33 | 121.08 | 73.57 | 120.61 | -2.55% |
| Station: 915+85.77 | 2.10% | 123.67 | -69 | 123.4 | -56.04 | 122.73 | -24 | 122.47 | 24 | 122.08 | 55.36 | 121.35 | 73.02 | 120.93 | -2.34% |
| Station: 916+00.00 | 2.07% | 123.94 | -69 | 123.67 | -56.03 | 123.01 | -24 | 122.74 | 24 | 122.35 | 55.37 | 121.68 | 73.02 | 121.31 | -2.13% |
| Station: 916+07.23 | 2.10% | 124.08 | -69 | 123.81 | -56.03 | 123.14 | -24 | 122.88 | 24 | 122.49 | 55.38 | 121.81 | 73.02 | 121.43 | -2.15% |
| Station: 916+22.56 | 2.09% | 124.32 | -69 | 124.05 | -56.02 | 123.38 | -24 | 123.15 | 24 | 122.75 | 55.39 | 122.08 | 73.02 | 121.7 | -2.13% |
| Station: 916+25.00 | 2.07% | 124.36 | -69 | 124.09 | -56.01 | 123.43 | -24 | 123.19 | 24 | 122.79 | 55.39 | 122.12 | 73.02 | 121.75 | -2.11% |
| Station: 916+50.00 | 1.89% | 124.71 | -69 | 124.46 | -56 | 123.86 | -24 | 123.58 | 24 | 123.22 | 55.41 | 122.63 | 73.02 | 122.3 | -1.88% |
| Station: 916+75.00 | 1.56% | 125.03 | -69 | 124.83 | -55.98 | 124.33 | -24 | 123.95 | 24 | 123.63 | 55.43 | 123.16 | 73.02 | 122.9 | -1.50% |
| Station: 917+00.00 | 1.46% | 125.41 | -69 | 125.22 | -55.96 | 124.75 | -24 | 124.41 | 24 | 124.09 | 55.45 | 123.65 | 73.02 | 123.41 | -1.39% |
| Station: 917+25.00 | 1.10% | 125.63 | -69 | 125.49 | -55.94 | 125.14 | -24 | 124.82 | 24 | 124.5 | 55.47 | 124.08 | 73.02 | 123.85 | -1.34% |
| Station: 917+50.00 | 0.96% | 125.85 | -69 | 125.73 | -55.92 | 125.42 | -24 | 125.2 | 24 | 124.91 | 55.49 | 124.5 | 73.02 | 124.27 | -1.29% |
| Station: 917+75.00 | 0.88% | 126.18 | -69 | 126.07 | -55.9 | 125.79 | -24 | 125.57 | 24 | 125.29 | 55.51 | 124.93 | 73.02 | 124.72 | -1.17% |
| Station: 918+00.00 | 0.75% | 126.48 | -69 | 126.38 | -55.89 | 126.14 | -24 | 125.93 | 24 | 125.68 | 55.53 | 125.36 | 73.02 | 125.18 | -1.02% |
| Station: 918+25.00 | 0.67% | 126.75 | -69 | 126.66 | -55.87 | 126.44 | -24 | 126.28 | 24 | 126.09 | 55.55 | 125.79 | 73.02 | 125.63 | -0.94% |
| Station: 918+50.00 | 0.56% | 127.01 | -69 | 126.94 | -55.85 | 126.76 | -24 | 126.62 | 24 | 126.45 | 55.56 | 126.23 | 73.02 | 126.11 | -0.69% |
| Station: 918+75.00 | 0.42% | 127.25 | -69 | 127.2 | -55.83 | 127.06 | -24 | 126.94 | 24 | 126.79 | 55.58 | 126.64 | 73.02 | 126.56 | -0.46% |
| Station: 919+00.00 | 0.15% | 127.37 | -69 | 127.35 | -55.81 | 127.3 | -24 | 127.26 | 24 | 127.17 | 55.6 | 127.05 | 73.02 | 126.98 | -0.38% |
| Station: 919+25.00 | -0.01% | 127.61 | -69 | 127.61 | -55.79 | 127.61 | -24 | 127.58 | 24 | 127.5 | 55.62 | 127.54 | 73.02 | 127.56 | 0.14% |
| Station: 919+50.00 | -0.01% | 127.87 | -69 | 127.87 | -55.78 | 127.87 | -24 | 127.9 | 24 | 127.94 | 55.64 | 128.02 | 73.02 | 128.06 | 0.25% |
| Station: 919+75.00 | -0.25% | 128.09 | -69 | 128.13 | -55.76 | 128.21 | -24 | 128.24 | 24 | 128.27 | 55.66 | 128.34 | 73.02 | 128.37 | 0.20% |
| Station: 920+00.00 | -0.32% | 128.33 | -69 | 128.37 | -55.74 | 128.47 | -24 | 128.61 | 24 | 128.64 | 55.68 | 128.7 | 73.02 | 128.73 | 0.19% |
| Station: 920+25.00 | -0.55% | 128.54 | -69 | 128.62 | -55.72 | 128.79 | -24 | 128.91 | 24 | 128.99 | 55.7 | 129.08 | 73.02 | 129.12 | 0.26% |
| Station: 920+44.31 | -0.68% | 128.7 | -69 | 128.79 | -55.71 | 129.01 | -24 | 129.13 | 24 | 129.2 | 55.71 | 129.3 | 73.02 | 129.35 | 0.32% |
| Station: 920+50.00 | -0.68% | 128.75 | -69 | 128.84 | -55.7 | 129.06 | -24 | 129.19 | 24 | 129.26 | 55.72 | 129.36 | 73.02 | 129.42 | 0.32% |
| Station: 920+64.20 | -0.68% | 128.88 | -69 | 128.97 | -55.69 | 129.18 | -24 | 129.34 | 24 | 129.41 | 55.73 | 129.53 | 73.02 | 129.59 | 0.38% |
| Station: 920+75.00 | -0.70% | 128.97 | -69 | 129.06 | -55.68 | 129.28 | -24 | 129.43 | 24 | 129.44 | 55.73 | 129.7 | 73.02 | 129.84 | 0.82% |
| Station: 921+00.00 | -0.81% | 129.1 | -69 | 129.2 | -55.67 | 129.46 | -24 | 129.65 | 24 | 129.58 | 55.75 | 130.09 | 73.02 | 130.36 | 1.59% |
| Station: 921+25.00 | -0.84% | 129.26 | -69 | 129.37 | -55.65 | 129.63 | -24 | 129.82 | 24 | 129.93 | 55.77 | 130.39 | 72.46 | 130.63 | 1.44% |
| Station: 921+50.00 | -0.39% | 129.41 | -69 | 129.46 | -55.63 | 129.58 | -24 | 130.08 | 24 | 130.32 | 55.79 | 130.74 | 71.38 | 130.94 | 1.31% |
| Station: 921+75.00 | -1.10% | 129.39 | -69 | 129.54 | -55.61 | 129.89 | -24 | 130.27 | 24 | 130.7 | 55.81 | 131.09 | 70.3 | 131.27 | 1.22% |
| Station: 921+82.09 | -1.24% | 129.42 | -69 | 129.58 | -55.61 | 129.98 | -24 | 130.38 | 24 | 130.81 | 55.82 | 131.19 | 69.99 | 131.37 | 1.22% |
| Station: 922+00.00 | -1.32% | 129.59 | -69 | 129.77 | -55.59 | 130.19 | -24 | 130.66 | 24 | 131.05 | 55.83 | 131.48 | 69.22 | 131.66 | 1.34% |

## VERTICAL TRANSFORMATION

Route: 710
PM: 17.34

** All numbers are in feet unless otherwise indicated

### LA RIVER

| AS-BUILT | | CALTRANS DTM | | Change in elevation |
|---|---|---|---|---|
| Station | Elevation | Station | Elevation | |
| 20+00 | 121.99 | 91+631.96 | 123.3 | +1.31 |
| 21+00 | 123.5 | 91+731.96 | 124.94 | +1.44 |
| 22+00 | 125 | 91+831.96 | 126.38 | +1.38 |
| 23+00 | 126.51 | 91+931.96 | 127.66 | +1.15 |
| 24+00 | 128.02 | 92+031.96 | 128.99 | +0.97 |

Average: +1.25

*elevations measured along station line

BB Area ⇒ 1.40' adjustment

Pier 2 ⇒ 1.45' adjustment

Pier 3 ⇒ 1.40 Adjustment

Pier 4 ⇒ 1.20 adjustment

Abutment 5 ⇒ 1.20 Adjustment

1.40 82 81.4 0.6

75.5 — Pier 2
1.4
76.9 + 4.5 = 81.4

75.5
1.3
76.8
4.5
81.3 v.s. 82.2

76
1.4
77.1
4.5
81.6
0.6
82.1

## Top of Pier Wall Elevations

| Pier | Station | Elevation ACOE Left (N) | Elevation ACOE Right (S) | 1954 As-Built Left Up See Note 3 | 1954 As-Built Left Low See Note 1 | 1954 As-Built Right Up See Note 3 | 1954 As-Built Right Low See Note 2 | 1994 Strength (See Note 4) Left Up See Note 6 | 1994 Strength Left Old T/P See Note 5 | 1994 Strength Right Up See Note 8 | 1994 Strength Right Old T/P See Note 7 | Retrofit No. 224 (See Note 9) Left Up See Note 10 | Retrofit No. 224 Left Low See Note 9 | Retrofit No. 224 Right Up See Note 12 | Retrofit No. 224 Right Low See Note 11 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Datum Conversion Factor | | | | 0.589 | 0.589 | 0.589 | 0.589 | | | | |
| 2 | 91685.717 | 110.08 | 108.39 | 112.33 | 110.08 | 108.86 | 109.53 | 113.96 | 112.12 | 110.49 | 108.65 | 114.3 | 110.1 | 110.8 | 108.5 |
| 3 | 91835.717 | 112.06 | 110.67 | 113.91 | 112.07 | 111.59 | 110.58 | 115.54 | 113.70 | 113.22 | 111.38 | 115.9 | 112.1 | 113.6 | 110.6 |
| 4 | 91985.717 | 114.04 | 112.35 | 115.49 | 113.99 | 114.33 | 112.47 | 117.12 | 115.28 | 115.96 | 114.12 | 117.4 | 114 | 116.3 | 112.5 |
| | | | | | | | | New Datum Elevation | | | | Difference (ft) | | | |
| | | | | | | | | 114.54 | 112.71 | 111.07 | 109.24 | 0.24 | -0.02 | 0.27 | 0.03 |
| | | | | | | | | 116.12 | 114.29 | 113.80 | 111.97 | 0.22 | -0.03 | 0.20 | -0.02 |
| | | | | | | | | 117.70 | 115.87 | 116.54 | 114.71 | 0.30 | -0.01 | 0.24 | -0.03 |

Note
1. Columun G numbers match with Column D numbers. It means that 1954 As-Built Data check with ACOE Plans.
2. Columun H numbers match with Column E numbers. It means that 1954 As-Built Data check with ACOE Plans.
3. These elevations are from 1954 As-Built Plans. Elevations at Top of Sole Plate for rocker bearing. Rocker bearing were replaced in 1994. Sole Plate is 2.5 inches thick.
4. From 1994 Strength As-Built Plans, it is interpreted that the rocker bearing was removed to the top of the existing concrete pier wall face. The distance from the existing bottom flange to the top of the exisitng pier wall is about 2'-2". The added depth of the concrete pedestal is about 2'-2" minus 2" thick base plate and 2" steel elastomeric bearing pad.
5. The top of the existing concrete pier wall for rocker bearing. It is equal to Column F - (2.5")
6. It is equal to Column K + (1'-10").
7. It is equal to Column H - (2.5")
8. It is equal to Column M + (1'-10")

## Bottom of Pier Wall Elevations

| Pier | Station | ACOE BOF Old Datum | New Datum | Pier Height From Nose to BOF Left Nose | Pier Height From Nose to BOF Right Nose | Pier Height From Nose to BOF Average | Pier Height to TOF Average |
|---|---|---|---|---|---|---|---|
| | | | 0.589 | | | | 4.5 |
| 2 | 91685.717 | 75.5 | 76.089 | 36.62 | 33.15 | 34.89 | 30.39 |
| 3 | 91835.717 | 73.5 | 74.089 | 40.20 | 37.88 | 39.04 | 34.54 |
| 4 | 91985.717 | 76 | 76.589 | 39.28 | 38.12 | 38.70 | 34.20 |

Note:
1. The existing footing depth is 3 feet, and was retrofitted to become 4'-6".

5/19/09

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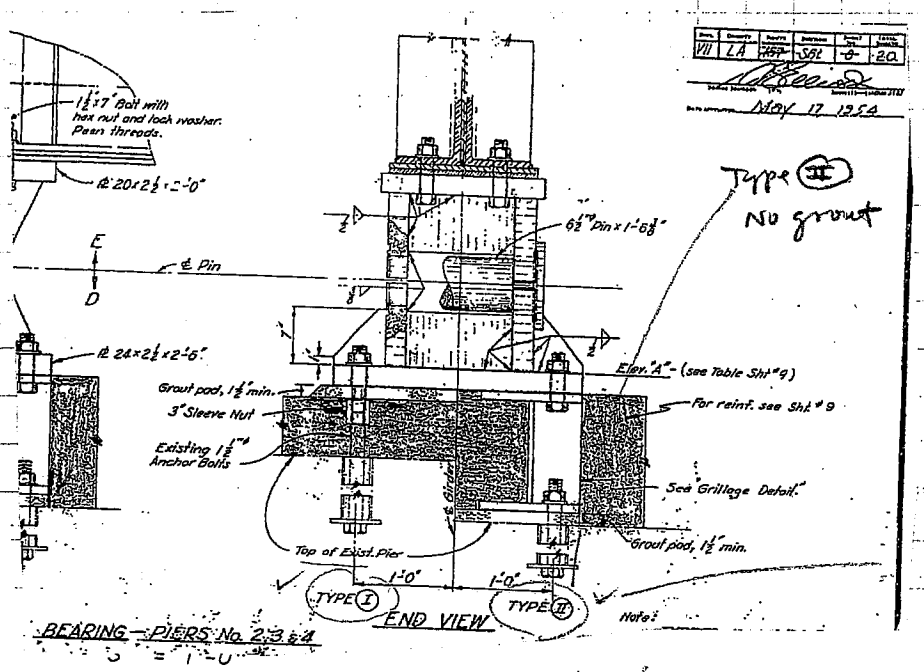
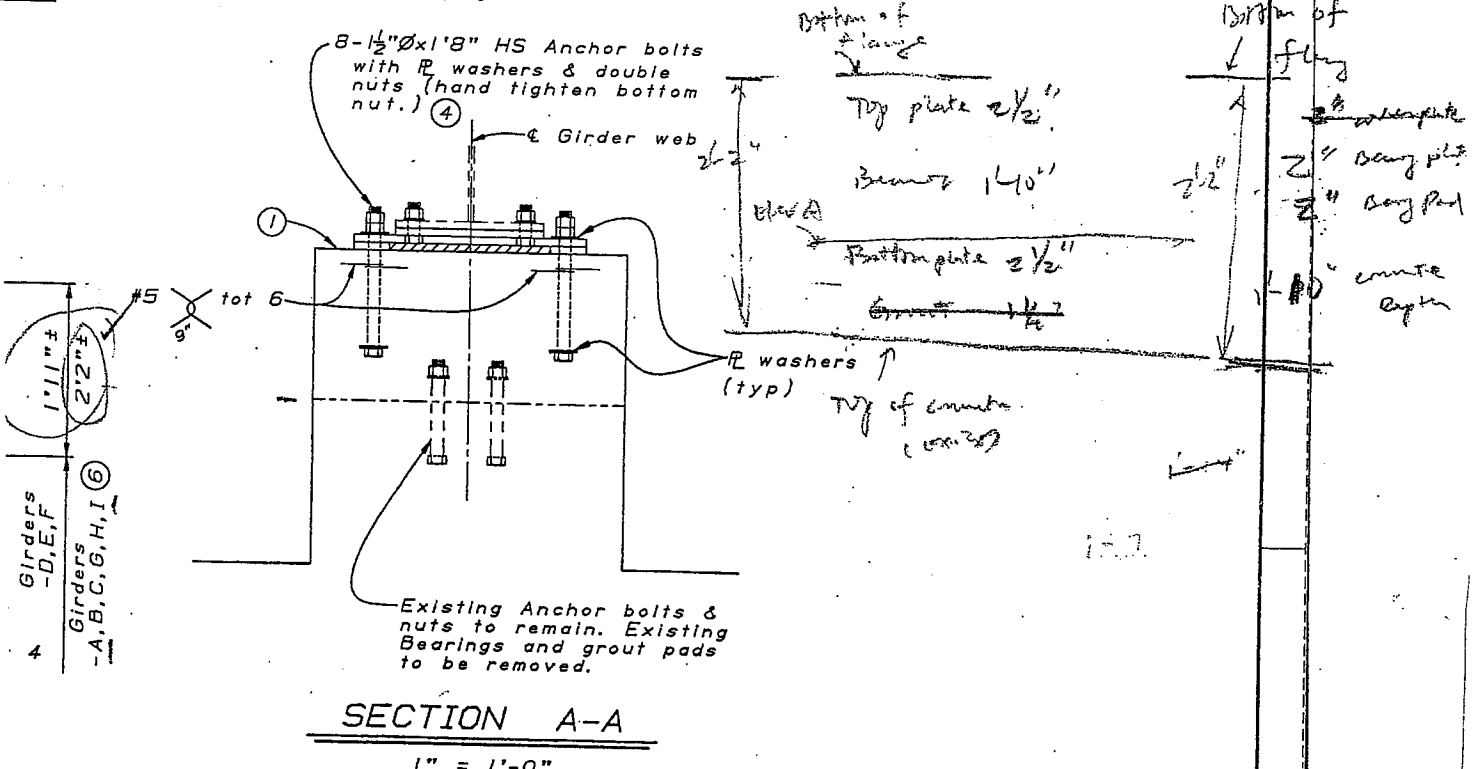
FHW L A River Bridge 10

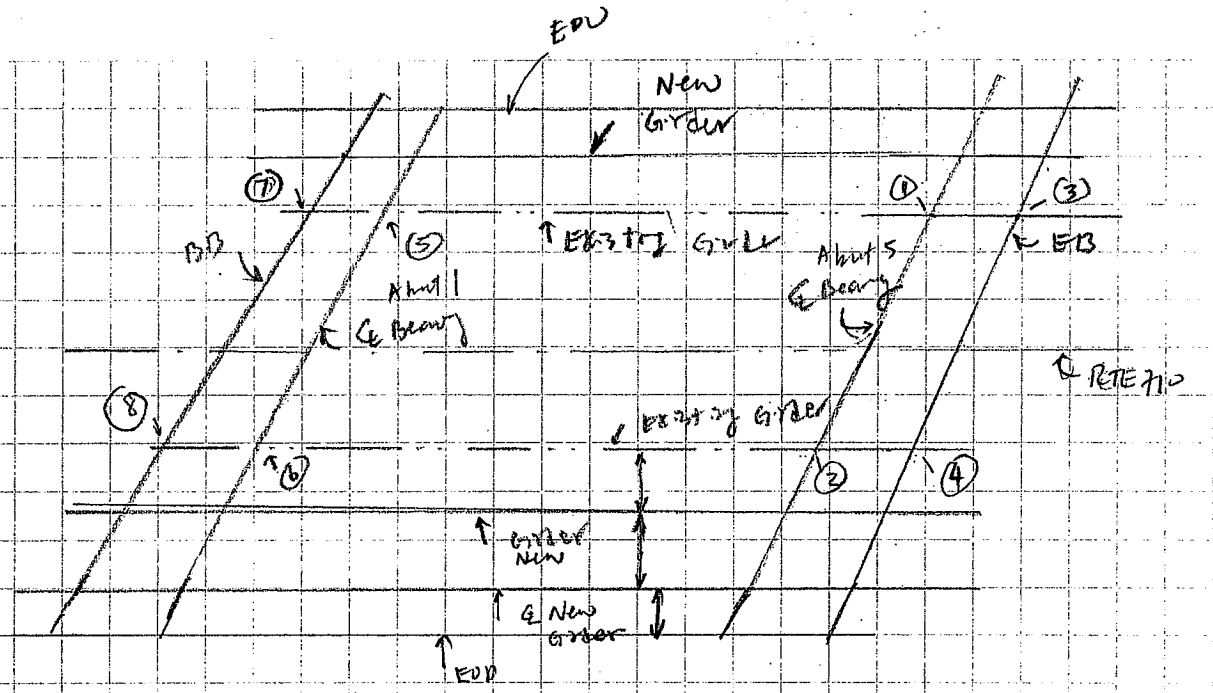
5/21/09 Top of Pier Wall Elev.

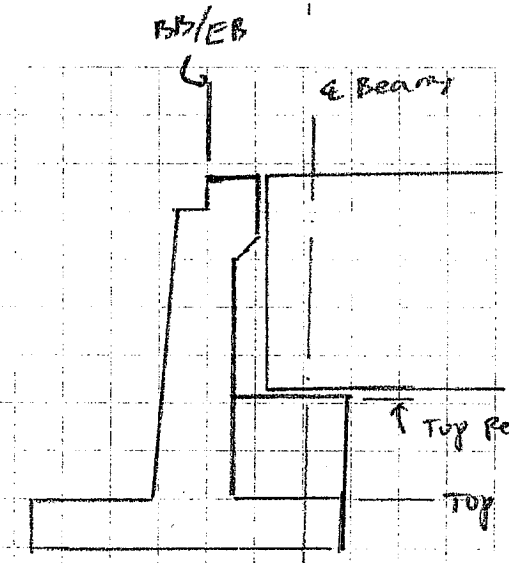
Pier Wall Elevations from As-Built.

From LA River Bridge strength Plans, Bearing Details No. 2 sheet and 1954 Bridge As-Built Plans "Bearing Details" sheet.





FHW
6/11/09

Existing Abutment

11

## Abutment Deck Elevation at Centerline of Girder and Pedestal Elevation

### Existing Abutment Pedestal Elevation

Conver. Factor 0.589

Retrofit No. 224

| Abutment | Loc | Old Datum Left (Girder I) | Old Datum Right (Girder A) | New Datum Left (Girder I) | New Datum Right (Girder A) |
|---|---|---|---|---|---|
| 1 | Top Pedestal | 113.00 | 108.50 | 113.59 | 109.09 |
| | Top of Footing | 106.00 | 106.00 | 106.59 | 106.59 |
| 5 | Top Pedestal | 119.00 | 118.80 | 119.59 | 119.39 |
| | Top of Footing | 106.50 | 106.50 | 107.09 | 107.09 |

### Existing Abutment Deck Elevation at Centerline of Girder ( Existing Top of Backwall)

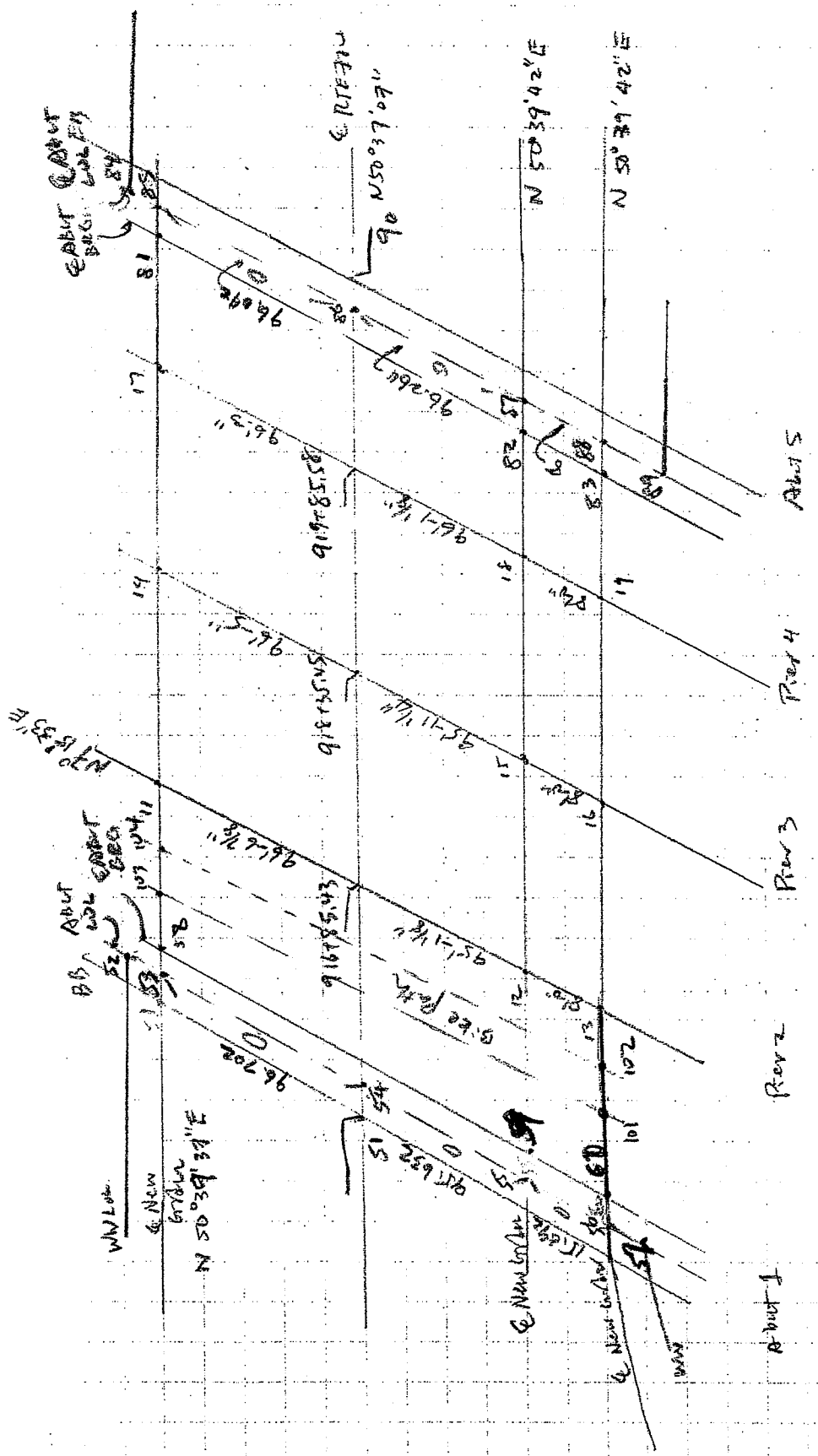
Skew Angle 46^37'00"
0.813611681 rad
46.617 deg

| Point on Figure | Abutment | Station | Deck Elevation along the ETW Line outside | Offset Distance from Centerline of existing girder to ETW outside | Cross Slope | Deck Elevation at Circle numbers |
|---|---|---|---|---|---|---|
| Circle 5 | 1 Left, CL Bearing, SB | 91632.289 | 124.16 | -1.12 | 0.0156 | 124.14 |
| | 1 Sta., CL Bearing | 91552.217 | | | | 121.70 |
| Circle 6 | 1 Right, CL Bearing, NB | 91472.145 | 118.86 | -0.397 | -0.0291 | 118.87 |
| Circle 7 | 1 Left, BB | 91628.592 | 124.11 | -1.12 | 0.0156 | 124.09 |
| | 1 Sta., BB | 91548.520 | | | | 121.62 |
| Circle 8 | 1 Right, BB | 91468.448 | 118.78 | -0.397 | -0.0292 | 118.79 |
| Circle 1 | 5 Left, CL Bearing, SB | 92199.289 | 129.94 | -0.26 | -0.0192 | 129.95 |
| | 5 Sta., CL Bearing | 92119.217 | | | | 129.58 |
| Circle 2 | 5 Right, CL Bearing, NB | 92039.145 | 129.09 | -0.912 | 0.0058 | 129.09 |
| Circle 3 | 5 Left, EB | 92202.989 | 129.97 | -0.26 | -0.0197 | 129.98 |
| | 5 Sta., EB | 92122.917 | | | | 129.62 |
| Circle 4 | 5 Right, EB | 92042.845 | 129.15 | -0.912 | 0.0060 | 129.15 |

FHW
6/29/09

LA River Bridge
Based plate layout @ Pier





## New Abutment Deck Elevation at Centerline of Girder/Abut LOL (New Top of Backwall)

| Circle Pt. | | Station | Offset Dist. To CL 710 | Deck Elevation along the Station Line | Cross Slope | Deck Elevation at Centerline of New Girder (Note 1) | Deck Elevation based upon the survey points along ETW outside | Distance from CL of new girder to ETW outside line | Cross Slope | Deck Elevation at Centerline of New Girder (Note 2) |
|---|---|---|---|---|---|---|---|---|---|---|
| 52 | Abut 1 North/WWLOL | 91623.476 | 70.333 | 122.99 | 0.0180 | 124.26 | 124.03 | 14.232 | 0.0180 | 124.29 |
| 53 | Abut 1 North 1 | 91619.304 | 66.393 | 122.92 | 0.0183 | 124.13 | 123.97 | 10.337 | 0.0183 | 124.16 |
| 58 | Abut 1 BRG/North 1 | 91621.911 | 66.393 | 122.96 | 0.0160 | 124.03 | 124.01 | 10.309 | 0.0160 | 124.17 |
| 54 | Abut 1/Rte 710 | 91548.995 | 0 | 121.63 | | 121.63 | | | | |
| 51 | BB/Rte 710 | 91547.903 | 0 | 121.61 | | 121.61 | | | | |
| 55 | Abut 1 South 1 | 91479.464 | 65.658 | 120.14 | -0.0290 | 118.24 | 119.01 | 10.096 | -0.0290 | 118.72 |
| 56 | Abut 1 South 2 | 91467.91 | 75.569 | 119.90 | -0.0292 | 117.69 | 118.77 | 20.066 | -0.0292 | 118.18 |
| 57 | Abut 1 South/WWLO | 91463.196 | 81.02 | 119.79 | -0.0293 | 117.42 | 118.66 | 25.541 | -0.0293 | 117.92 |
| 59 | Abut 1 BRG/South 1 | 91482.072 | 65.66 | 120.20 | -0.0289 | 118.30 | 119.07 | 10.085 | -0.0289 | 118.78 |
| 60 | Abut 1 BRG/South 2 | 91470.627 | 76.468 | 119.95 | -0.0292 | 117.72 | 118.82 | 20.951 | -0.0292 | 118.21 |
| 84 | Abut 5 North/WWLOL | 92196.597 | 70.333 | 130.69 | -0.0188 | 129.36 | 129.92 | 15.200 | -0.0188 | 129.63 |
| 85 | Abut 5 North 1 | 92191.981 | 65.974 | 130.62 | -0.0182 | 129.42 | 129.88 | 10.833 | -0.0182 | 129.68 |
| 81 | Abut 5 BRG/North 1 | 92189.375 | 65.976 | 130.58 | -0.0179 | 129.40 | 129.86 | 10.830 | -0.0179 | 129.66 |
| 86 | Abut 5/Rte 710 | 92122.116 | 0 | 129.61 | | 129.61 | | | | |
| 90 | EB/Rte 710 | 92123.208 | 0 | 129.63 | | 129.63 | | | | |
| 87 | Abut 5 South 1 | 92052.126 | 66.092 | 128.78 | 0.0065 | 129.21 | 129.30 | 10.193 | 0.0065 | 129.37 |
| 88 | Abut 5 South 2 | 92046.306 | 71.588 | 128.71 | 0.0062 | 129.16 | 129.21 | 15.690 | 0.0062 | 129.30 |
| 89 | Abut 5 South/WWLOL | 92043.399 | 74.333 | 128.68 | 0.0060 | 129.13 | 129.16 | 18.434 | 0.0060 | 129.27 |
| 82 | Abut 5 BRG/South 1 | 92049.518 | 66.09 | 128.75 | 0.0064 | 129.17 | 129.26 | 10.195 | 0.0064 | 129.32 |
| 83 | Abut 5 BRG/South 2 | 92043.698 | 71.586 | 128.68 | 0.0061 | 129.11 | 129.17 | 15.687 | 0.0061 | 129.26 |
| 101 | Bike Path W/South 2 | 91499.124 | 75.402 | 120.57 | -0.0286 | 118.41 | 119.44 | 19.741 | -0.0286 | 118.87 |
| 102 | Bike Path E/South 2 | 91524.302 | 74.461 | 121.10 | -0.0281 | 119.01 | 119.98 | 18.673 | -0.0281 | 119.45 |
| 103 | Bike Path W/North 1 | 91643.609 | 66.375 | 123.33 | 0.0161 | 124.40 | 124.31 | 10.254 | 0.0161 | 124.47 |
| 104 | Bike Path E/North 2 | 91662.454 | 66.362 | 123.65 | 0.0143 | 124.60 | 124.56 | 10.306 | 0.0143 | 124.71 |
| | | 91438.196 | 81.02 | 119.28 | -0.0306 | 116.80 | 118.10 | 25.630 | -0.0306 | 117.31 |
| | | 92113.698 | 74.333 | 129.51 | 0.0100 | 130.25 | 130.30 | 18.332 | 0.0100 | 130.48 |

Notes:
1. This deck elevation is calculated using the elevation at the station line to add/subtract the elevation difference from cross slope.
2. This deck elevation is calculated using the elevation at the ETW outside line to add/subtract the elevation difference from cross slope.

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8/19/10

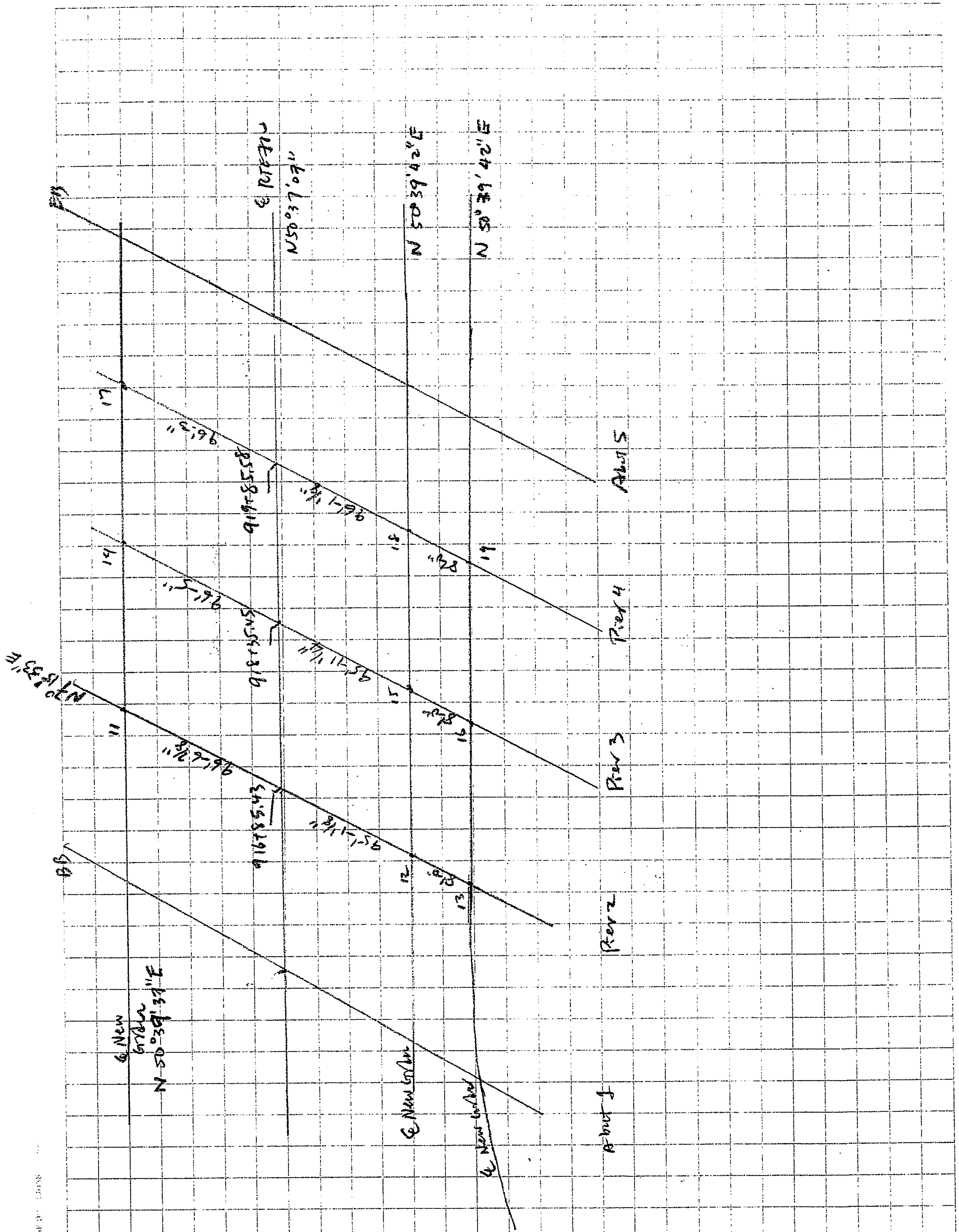
## New Top Abutment Pedestal Elevation

| | |
|---|---|
| Depth | 9.75 |
| Bearing Plate | 0.13 |
| Base Plate | 0.13 |

| | Top of Deck | | Bottom of Flange | | Top of Pedestal (Min) | | |
|---|---|---|---|---|---|---|---|
| | Left Bridge (North) | Right Bridge (South) | Left Bridge (North) | Right Bridge (South) | Left Bridge (North) | Right Bridge (South) | |
| Circle Pt. | | | | | | | |
| 58 Abut 1 BRG/North 1 | 124.41 | | 114.66 | | 114.41 use 114.40 | | |
| 59 Abut 1 BRG/South 1 | | 118.73 | | 108.98 | | 1~~08.73~~ | use 108.70 |
| 60 Abut 1 BRG/South 2 | | 118.21 | | 108.46 | | 108.21 | 108.20 |
| 81 Abut 5 BRG/North 1 | 129.53 | | 119.78 | | 119.53 use 119.50 | | |
| 82 Abut 5 BRG/South 1 | | 129.39 | | 119.64 | | 119.39 | use 119.40 |
| 83 Abut 5 BRG/South 2 | | 129.34 | | 119.59 | | 119.34 | 119.35 |

FHW LA River Bridge 

10/29/09 Based plate layout @ Pier



## New Top of Pier Wall Pedestal Elevation

| | |
|---|---|
| Depth | 9.75 |
| Bearing Plate | 0.17 |
| Base Plate | 0.17 |

| Circle Pt. | Top of Deck Left Bridge (North) | Top of Deck Right Bridge (South) | Bottom of Flange Left Bridge (North) | Bottom of Flange Right Bridge (South) | Top of Pedestal (Min) Left Bridge (North) | Top of Pedestal (Min) Right Bridge (South) | |
|---|---|---|---|---|---|---|---|
| | | | | | use 115.80 | | |
| 11 Pier 2 North | 125.90 | | 116.15 | | 115.82 | | |
| 12 Pier 2 South 1 | | 121.68 | | 111.93 | use 117.35 | 111.60 | ) use 111.50 |
| 13 Pier 2 South 2 | | 121.52 | | 111.77 | | 111.43 | |
| 14 Pier 3 North | 127.42 | | 117.67 | | 117.34 | | |
| 15 Pier 3 South 1 | | 124.63 | | 114.88 | | 114.55 | ) 114.45 |
| 16 Pier 3 South 2 | | 124.47 | | 114.72 | | 114.39 | |
| 17 Pier 4 North | 128.80 | | 119.05 | | 118.71 | | |
| 18 Pier 4 South 1 | | 127.35 | | 117.60 | use 118.70 | 117.27 | ) use 117.20 |
| 19 Pier 4 South 2 | | 127.22 | | 117.47 | | 117.13 | |

## New Top of Deck Elevation at the centerline of girder and centerline of Pier Wall Elevation

| Circle Pt. | Station | Offset Dist. To CL 710 | Deck Elevation along the Station Line | Cross Slope | Deck Elevation at Centerline of New Girder (Note 1) | Deck Elevation based upon the survey points along sawcut line | Distance from CL of new girder to sawcut line | Cross Slope | Elevation at Centerline of New Girder (Note 2) |
|---|---|---|---|---|---|---|---|---|---|
| 11 Pier 2 North | 91755.632 | 66.293 | 125.20 | 0.0078 | 125.72 | 125.81 | 10.375 | 0.0094 | 125.90 |
| 12 Pier 2 South 1 | 91615.789 | 65.762 | 122.86 | -0.0227 | 121.37 | 121.90 | 10.375 | -0.0214 | 121.68 |
| 13 Pier 2 South 2 | 91609.969 | 71.257 | 122.76 | -0.0232 | 121.11 | 121.86 | 15.875 | -0.0215 | 121.52 |
| 14 Pier 3 North | 91905.541 | 66.184 | 127.11 | 0.0009 | 127.17 | 127.41 | 10.375 | 0.0011 | 127.42 |
| 15 Pier 3 South 1 | 91765.071 | 65.874 | 125.36 | -0.0139 | 124.44 | 124.76 | 10.375 | -0.0122 | 124.63 |
| 16 Pier 3 South 2 | 91759.874 | 71.371 | 125.27 | -0.0140 | 124.27 | 124.67 | 15.875 | -0.0124 | 124.47 |
| 17 Pier 4 North | 92055.553 | 66.074 | 128.82 | -0.0054 | 128.46 | 128.87 | 10.375 | -0.0068 | 128.80 |
| 18 Pier 4 South 1 | 91915.702 | 65.989 | 127.22 | -0.0023 | 127.07 | 127.36 | 10.375 | -0.0005 | 127.35 |
| 19 Pier 4 South 2 | 91909.882 | 71.485 | 127.16 | -0.0028 | 126.96 | 127.24 | 15.875 | -0.0017 | 127.22 |

Void

Notes:
1. This deck elevation is calculated using the elevation at the station line to add/subtract the elevation difference from cross slope. (No use, void)
2. This deck elevation is calculated using the elevation at the sawcut line to add/subtract the elevation difference from cross slope.

Girder Section for Positive Flexure regions

Top Compression Flange

Match with existing Girder Top flange width. Use

$b_{fc} \times t_{fc} = 18^{in} \times 1^{in}$

Web

try web $D \times t_w = 102'' \times 3/4''$

Bottom tension Flange

Try bottom Tension flange

$b_{ft} \times t_{ft} = 18^{in} \times 1''$

Check section Proportion limits

(1) Web without longitudinal stiffners. 6.10.2.1.1

$\frac{D}{t_w} < 150 \quad \frac{102}{0.75} = 136 < 150$ OK

(2) Flange Proportions. 6.10.2.2

$\frac{b_f}{2t_f} < 12.0 \quad \frac{18}{2} = 9 < 12$ OK

$b_f \geq D/6 \quad \frac{102}{6} = 17 \quad 18 > 17$ OK

$t_f \geq 1.1\, t_w \quad 1^{in} > 1.1 \times 3/4 = 0.825$ OK

$0.1 < \frac{I_{yc}}{I_{yt}} < 10 \quad I_{yc}/I_{yt} = 1$ OK

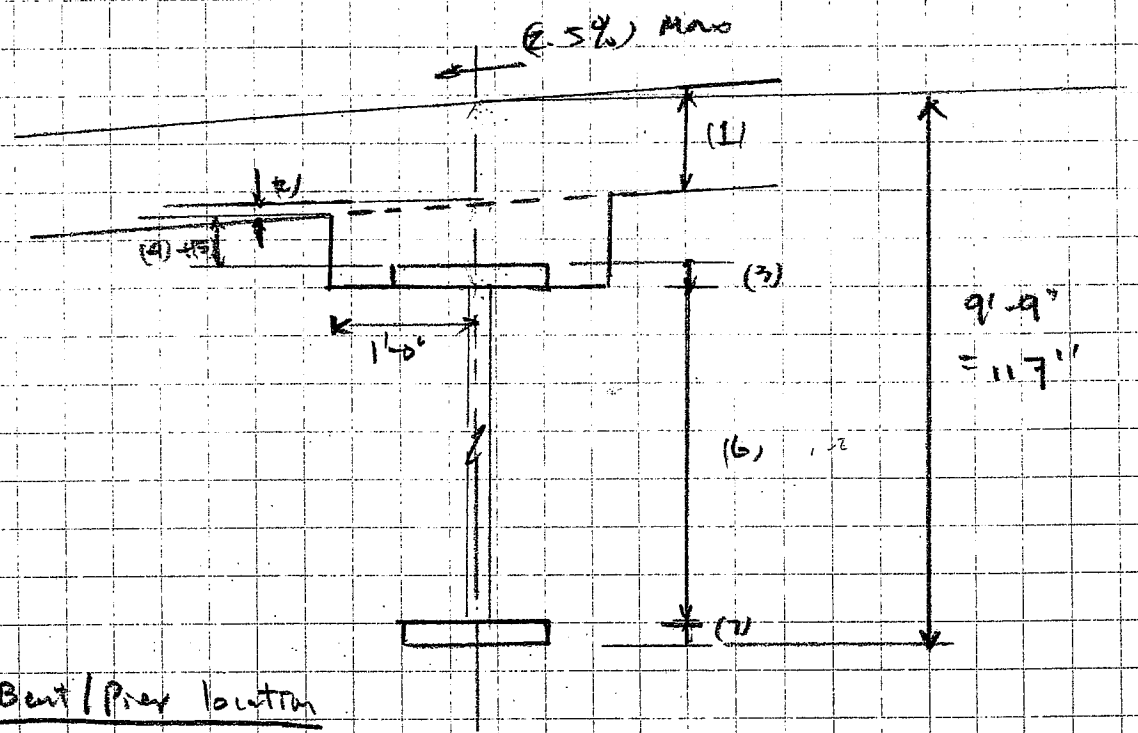
$I_{yc} = \frac{1}{12}(1)(18^3) = I_{yt}$

FHW
4/27/09

LA River Bridge
Typical Haunch Cal.



BDD 11-30, to calculate haunch depth, and overall structure depth




At Bent / Pier location

(1) = 10"

(2) = 0.025 × 1' ≐ 1/4"

(3) = 1" ~ 1 1/2"

(4)+(5) = 1 1/2" MIN

(6) = 102"

(7) = 1" – 1 1/2"

≐ 116.75" → use 117"

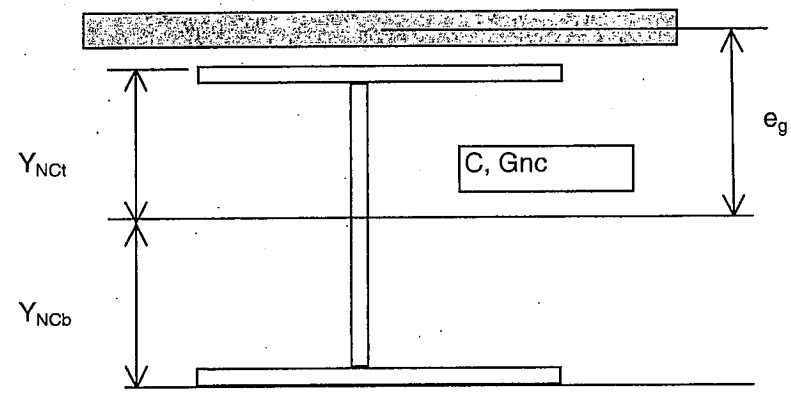
Haunch Thickness: 117" − 1.5" − 102" − 10" = 3.5" MAX
(7) (6) (1)

(1) + (2) + (4) + (5) ≐ 12" MIN or
117" − (7) − (6) = 14" MAX } use 14"

## Preliminary Section Properties

| Component | Ai ($in^2$) | Yi (in) | Ai*Yi ($In^3$) | $Y_{NCb}$ | $Y_{NCt}$ | $Yi-Y_{NCb}$ | $Ai(Yi-Y_{NCb})^2$ | Io |
|---|---|---|---|---|---|---|---|---|
| Top Flange 18"x1" | 18.00 | 103.50 | 1863.00 | | | 51.50 | 47741 | 1.50 |
| Web 102"x 3/4" | 76.50 | 52.00 | 3978.00 | | | 0.00 | 0 | 66326 |
| Bottom Flange 18"x1" | 18.00 | 0.50 | 9.00 | | | -51.50 | 47741 | 1.50 |
| Sum | 112.50 | | 5850.00 | 52.00 | 52.00 | | 95481 | 66329 |

| | | |
|---|---|---|
| $I_{NC}$ = | 161810 | $in^4$ |
| $e_g$ = | 60.00 | in |
| n = | 8 | |
| $K_g$ = | 4534476 | $in^4$ |
| $S_{NCb}$ = | 3112 | $in^3$ |
| $S_{NCt}$ = | 3112 | $in^3$ |



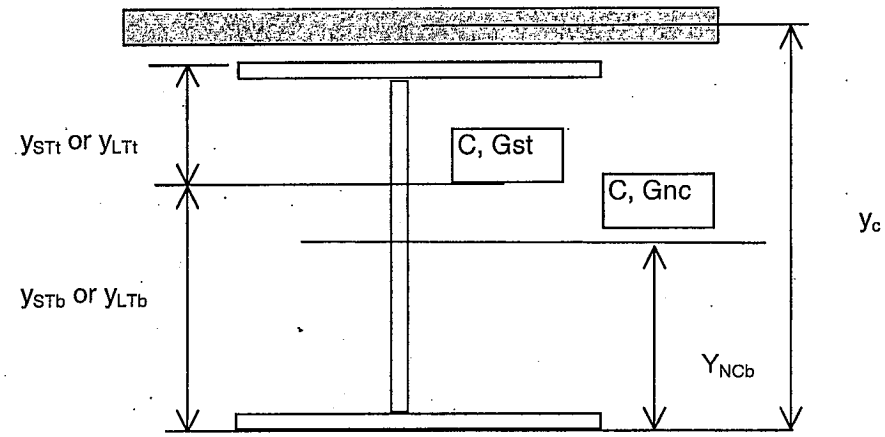
## Preliminary Section Properties - For Short Term Composite Section (n=8)

| Component | Ai ($in^2$) | Yi (in) | Ai*Yi ($In^3$) | $y_{STb}$ | $y_{STt}$ | $Yi-y_{STb}$ | $Ai(Yi-y_{STb})^2$ | Io |
|---|---|---|---|---|---|---|---|---|
| Steel Section | 112.50 | 51.96 | 5845.5 | | | -34.31 | 132421 | 161347 |
| Concrete Slab (10') | 150 | 112 | 16800 | | | 25.73 | 99316 | 1250 |
| Sum | 262.50 | | 22645.5 | 86.27 | 17.73 | | 231737 | 162597 |

$I_{ST}$ = 394334 $in^4$

$S_{STb}$ = 4571 $in^3$

$S_{STt}$ = 22239 $in^3$




## Preliminary Section Properties - For Long Term Composite Section (3n=24)

| Component | Ai ($in^2$) | Yi (in) | Ai*Yi ($In^3$) | $y_{LTb}$ | $y_{LTt}$ | $Yi-y_{LTb}$ | $Ai(Yi-y_{LTb})^2$ | Io |
|---|---|---|---|---|---|---|---|---|
| Steel Section | 112.50 | 51.96 | 5845.5 | | | -18.47 | 38394 | 161347 |
| Concrete Slab (10') | 50 | 112 | 5600 | | | 41.57 | 86387 | 417 |
| Sum | 162.50 | | 11445.5 | 70.43 | 33.57 | | 124782 | 161764 |

$I_{LT}$ = 286545 $in^4$

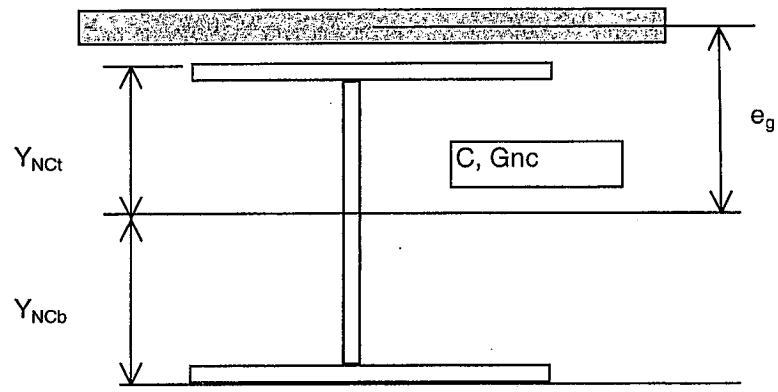
$S_{LTb}$ = 4068 $in^3$

$S_{LTt}$ = 8537 $in^3$

## Preliminary Section Properties - For Negative Moment Area

| Component | Ai ($in^2$) | Yi (in) | Ai*Yi ($In^3$) | $Y_{NCb}$ | $Y_{NCt}$ | $Yi-Y_{NCb}$ | $Ai(Yi-Y_{NCb})^2$ | Io |
|---|---|---|---|---|---|---|---|---|
| Top Flange 18"x1.5" | 27.00 | 104.25 | 2814.75 | | | 51.75 | 72308 | 5.06 |
| Web 102"x 3/4" | 76.50 | 52.50 | 4016.25 | | | 0.00 | 0 | 66326 |
| Bottom Flange 18"x1.5" | 27.00 | 0.75 | 20.25 | | | -51.75 | 72308 | 5.06 |
| Sum | 130.50 | | 6851.25 | 52.50 | 52.50 | | 144615 | 66336 |

| | | |
|---|---|---|
| $I_{NC}$ = | 210951 | $in^4$ |
| $e_g$ = | 60.00 | in |
| n = | 8 | |
| $K_g$ = | 5446008 | $in^4$ |
| $S_{NCb}$ = | 4018 | $in^3$ |
| $S_{NCt}$ = | 4018 | $in^3$ |

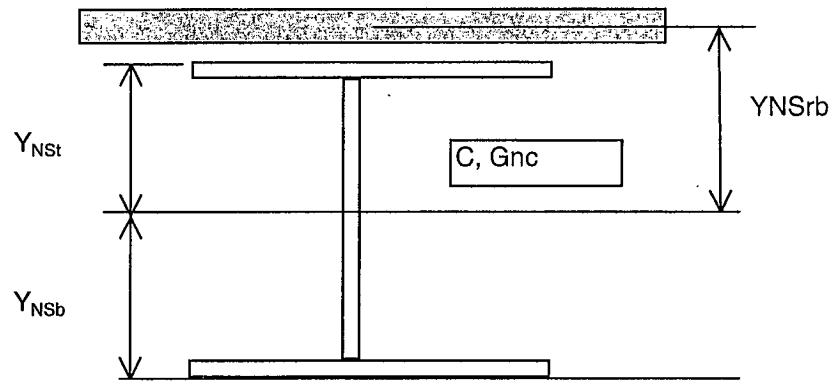


## Preliminary Section Properties - For Negative Moment Area, considering deck slab longitudinal reinforcement

(Assuming longitudinal reinforcement is 1% of the deck area.
As=0.01x(10'x12"x10")=12 Sq. In.

| Component | Ai ($in^2$) | Yi (in) | Ai*Yi ($In^3$) | $Y_{NSb}$ | $Y_{NSt}$ | Yi-$Y_{NSb}$ | Ai(Yi-$Y_{NSb}$)$^2$ | Io |
|---|---|---|---|---|---|---|---|---|
| Top Reinforcement | 12.00 | 110.50 | 1326.00 | | | 53.12 | 33855 | 0.00 |
| Steel Section | 130.50 | 52.50 | 6851.25 | | | -4.88 | 3113 | 210951 |
| | | | 0.00 | | | | | |
| Sum | 142.50 | | 8177.25 | 57.38 | 47.62 | | 36969 | 210951 |

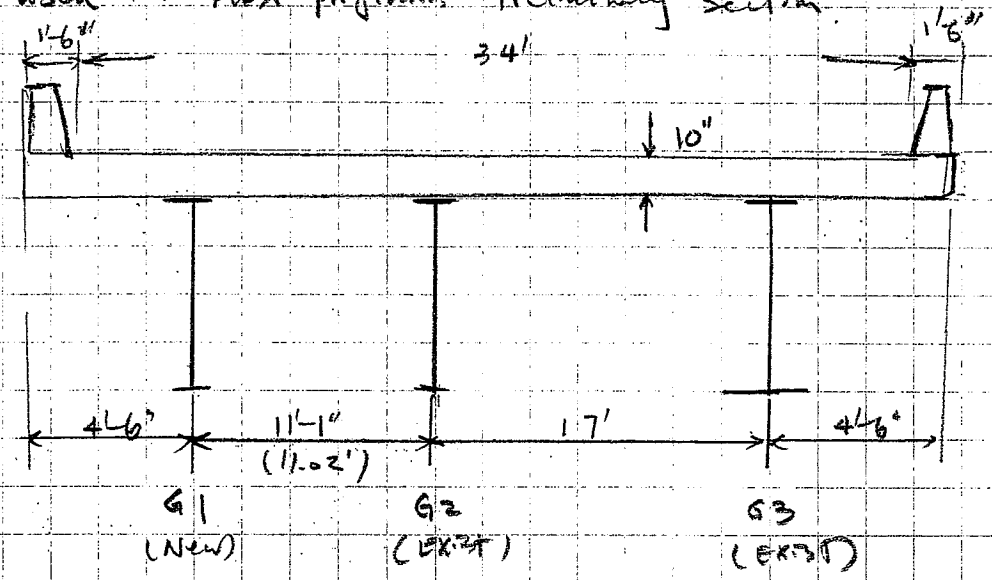
| | | |
|---|---|---|
| $I_{NS}$ = | 247920 | $in^4$ |
| YNSrb = | 53.12 | in |
| n = | 8 | |
| $K_g$ = | 5199624 | $in^4$ |
| $S_{NSb}$ = | 4320 | $in^3$ |
| $S_{NSt}$ = | 5207 | $in^3$ |
| $S_{NSrb}$= | 4668 | $in^3$ |



4/6/09 MDX Input to generate Dead and Live Load

11'-1"

Section used in MDX program. Preliminary section




For Girder 1

(1) slab weight

$10'' \times (4.5' + 5.51) \times 0.15\ ^{\#}/_{ft^3} = 1.25\ ^{k}/_{ft}$

(2) Haunch

$\frac{2''}{12} \times 2' \times 0.15\ ^{\#}/_{ft^3} = 0.05\ ^{k}/_{ft}$

(3) Wearing surface

$\frac{3''}{12} \times (4.5' + 5.51) \times 0.15\ ^{\#}/_{ft^3} = 0.38\ ^{k}/_{ft}$

(4) concrete Barrier $0.4\ ^{k}/_{ft}$

(5) Steel Girder

| | | | |
|---|---|---|---|
| Top flange plate | $18'' \times 1'' = 18\square''$ | | 10" |
| Bottom flange | $18'' \times 1'' = 18\square''$ | | + 2" |
| web plate | $102'' \times 5/8'' = 63.75\square''$ | | 18" |
| | | | 18" |
| | | | 102" |
| | | | 140 |

$99.75\ in^2 \times 0.2836\ ^{\#}/_{in^3} = 28.3\ ^{\#}/_{in} = 0.34\ ^{k}/_{ft}$

(6) Stiffeners + cross frames + studs = 0.03 $^{k}/_{ft}$

(7) Permanent Steel Deck Form = $0.1(1.25 + 0.05) = 0.13\ ^{k}/_{ft}$

Total uncomposite Dead Load = DC1

(1) + (2) + (3) + (4) + (5) + (6) + (7) = 1.80 $^{k}/_{ft}$

For Girder 2

(1) slab weight

$\frac{10''}{12} \times (5.51' + 8.5') \times 0.15 = 1.75$ K/ft

Total weight 2.39 K/ft

For Girder 3

(1) slab weight

$\frac{10''}{12} \times (8.5' + 4.5') \times 0.15$ K/ft $= 1.63$ K/ft

Total weight = 2.18 K/ft

For Girder 2

(3) wearing surface

$\frac{3''}{12} \times (5.51 + 8.5) \times 0.15 = 0.53$ K/ft

For Girder 3

(3) $\frac{3''}{12} \times (8.5 + 4.5) \times 0.15 = 0.49$ K/ft

Number of loaded live load lanes

$\frac{39}{12} = 3.08 \rightarrow$ use 3 lanes

FHW

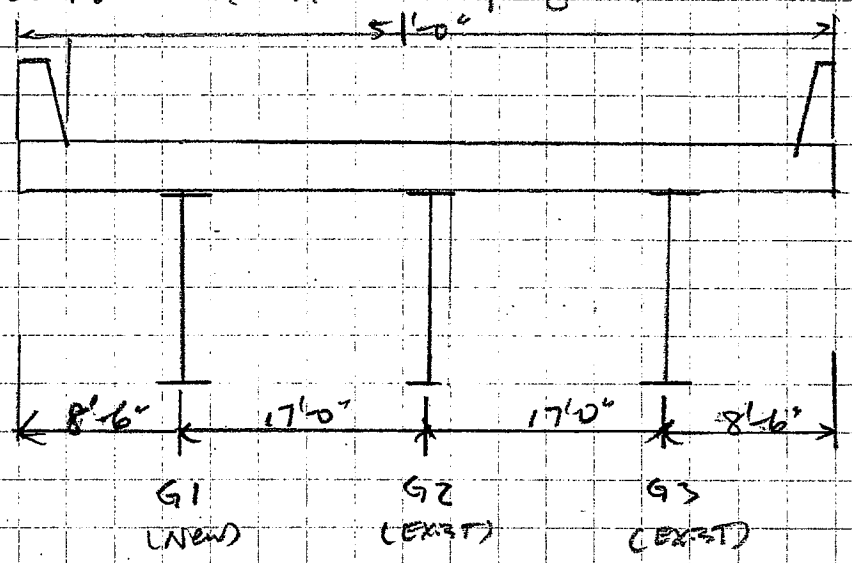
4/16/09

LA River Bridge

MDX Input to Generate Dead and Live Load

26

17'-0" Section used in MDX program



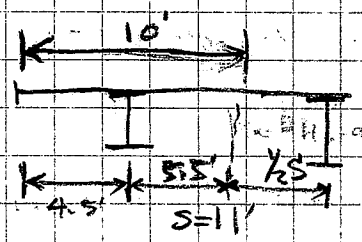
FHW
4/21/09

LA River Bridge
Live Load



Based on the 11'-0" New Girder spacing from the existing Girder, The new girder will NOT be able to take 1 lane Live Load. I ran MDX Preliminary Analysis mode, which will provide Live Load analysis results, base upon the actually girder geometry input. (In other words, it is 3-D analysis neglecting Distribution factor).

Then, I ran CT-Bridge model, using Arbitrary section and input 1 lane Live Load applied to the arbitrary section. Then, I compared the results of CT-Bridge v.s. MDX run. Use 1 full Lane Loaded on the widening Girder will be




the most conservative approach. No Distribution factor needs to be applied.

Comparison of the two runs also showed 1 full Lane Live load generated higher forces than MDX output.

Live Load moment Distribution Factor

Two or more design lane loaded controls

$$DFm = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12 L t_s^3}\right)^{0.1}$$

I will have several combinations for different assumptions.

S could be 11'-1" or use 11'-1" and 17'-0" average

$K_g = 453447 \text{ in}^4$

$t_s = 10 \text{ in}$

L = 133.5' for span 1 and 4 positive moment.

L = 150' for span 2 and 3 positive moment

L = 141.75 for span 1 & 2, span 3, & 4 negative moment

L = 150' for span 2 & 3 negative moment

| L | S = 11.083' | S = 14.41' |
|---|---|---|
| 133.5 | 0.815 | 0.0988 |
| 141.75 | 0.801 | 0.972 |
| 150 | 0.79 | 0.956 |

From this chart, using DF = 1 is calculation is conservative.

| Live Load - MDX (LL+IM) HL-93 | | Live Load - CT Bridge (LL+IM) HL-93, One Lane | | Ratio | |
|---|---|---|---|---|---|
| +M (K-ft) | -M (K-ft) | +M (K-ft) | -M (K-ft) | +M | -M |
| 65.38 | -0.29 | 10.71 | -166.86 | 6.105 | 0.002 |
| 1435.67 | -30 | 1343.41 | -28.16 | 1.069 | 1.065 |
| 2416.74 | -57.45 | 2317.27 | -54.12 | 1.043 | 1.062 |
| 3022.67 | -62.63 | 2888.66 | -60.97 | 1.046 | 1.027 |
| 3301.71 | -66.93 | 3075.31 | -69.59 | 1.074 | 0.962 |
| 3166.81 | -66.34 | 2938.06 | -71.08 | 1.078 | 0.933 |
| 2661.5 | -79.92 | 2416.06 | -74.18 | 1.102 | 1.077 |
| 1635.11 | -44.25 | 1499.56 | -41.49 | 1.090 | 1.067 |
| 190.61 | -4.81 | 166.05 | -12.84 | 1.148 | 0.375 |
| 120.04 | -1640.51 | 98.39 | -1813.04 | 1.220 | 0.905 |
| 258.08 | -3645.73 | 211.56 | -3930.34 | 1.220 | 0.928 |
| 258.08 | -3645.73 | 211.56 | -3930.34 | 1.220 | 0.928 |
| 1579.86 | -3014.1 | 1590.62 | -3296.48 | 0.993 | 0.914 |
| 2737.94 | -2537.88 | 2767.36 | -2381.35 | 0.989 | 1.066 |
| 3508.44 | -2132.78 | 3479.25 | -1990.62 | 1.008 | 1.071 |
| 3876.13 | -1685.76 | 3749.51 | -1599.89 | 1.034 | 1.054 |
| 3860.42 | -1318.12 | 3624.29 | -1209.16 | 1.065 | 1.090 |
| 3298.01 | -939.68 | 3057.42 | -818.44 | 1.079 | 1.148 |
| 2262.63 | -564.69 | 2043.61 | -427.71 | 1.107 | 1.320 |
| 561.76 | -141.51 | 551.34 | -36.98 | 1.019 | 3.827 |
| 315.28 | -1579.59 | 353.92 | -1658.1 | 0.891 | 0.953 |
| 789.2 | -3926.17 | 744.48 | -4149.61 | 1.060 | 0.946 |
| 789.2 | -3926.17 | 744.48 | -4149.61 | 1.060 | 0.946 |
| 1876.78 | -3293.93 | 1882.17 | -3588.26 | 0.997 | 0.918 |
| 3168.92 | -2899.88 | 3251.31 | -2785.28 | 0.975 | 1.041 |
| 4150.22 | -2618.32 | 4199.46 | -2550.56 | 0.988 | 1.027 |
| 4804.06 | -2345.77 | 4758.12 | -2315.83 | 1.010 | 1.013 |
| 5105.36 | -2139.2 | 4905.76 | -2081.11 | 1.041 | 1.028 |
| 4990.04 | -2299.49 | 4715.4 | -2274.1 | 1.058 | 1.011 |
| 4489.97 | -2624.59 | 4114.02 | -2479.7 | 1.091 | 1.058 |
| 3469.26 | -2825.48 | 3123.15 | -2685.31 | 1.111 | 1.052 |
| 1968.41 | -3030.57 | 1711.29 | -3423.23 | 1.150 | 0.885 |
| 535.32 | -3336.3 | 122.28 | -3826.96 | 4.378 | 0.872 |
| 535.32 | -3336.3 | 122.28 | -3826.96 | 4.378 | 0.872 |
| 185.91 | -1516.66 | 160.26 | -1813.04 | 1.160 | 0.837 |
| 218.87 | -27.13 | 182.35 | -23.39 | 1.200 | 1.160 |
| 1634.24 | -170.73 | 1499.56 | -149.85 | 1.090 | 1.139 |
| 2594.44 | -119.3 | 2416.06 | -114.87 | 1.074 | 1.039 |
| 3182.96 | -111.97 | 2938.06 | -107.06 | 1.083 | 1.046 |
| 3395.76 | -116.17 | 3075.31 | -106.7 | 1.104 | 1.089 |
| 3203.59 | -112.67 | 2888.66 | -99.49 | 1.109 | 1.132 |
| 2618.21 | -106.61 | 2317.27 | -88.23 | 1.130 | 1.208 |
| 1525.43 | -68.54 | 1343.41 | -63.08 | 1.135 | 1.087 |
| 29.09 | -62.48 | 10.71 | -63.37 | 2.716 | 0.986 |

From the ratio, we can see that CT_Bridge has larger negative moment than MDS, however smaller positive moment tham MDX.

For positive moment, MDX is about 10% higher than CT-Bridge.
For negative moment, MDX is about 13% lower than CT-Bridge.

MDX out is using influence line to get the live load. Therefore, no distribution factor is needed.

CT-Bridge is one lane and need to apply distribution factor. The calculations now are based upon distribution factor = 1.

Based upon current design positive moment, capacity over applied moment is 22291/17589=1.267. Design section has 26% over capacity. Therefore, section in positive moment is OK for MDX live load.

MDX negative moment is less than CT-Bridge moment. Therefore, section in negative moment area is adequate for MDX live load.

4/21/09
FW

## Unfactored Dead and Live Load - Moments

DC1: Permanent dead load. DC1 is defined as structural dead load acting on the non-composite section. (DC1 consists of deck slab concrete, steel girder, bracing system, use of steel deck form.)
DC2: Permanent dead load. DC2 is defined as non-structural dead load acting on the longterm composite sec (DC2 consists of barrier rails and specified utility)
DW: Wearing surface.

| Span | Point | Dead Load DC1 M_dc1 (K-ft) | DC2 M_dc2 (K-ft) | DW M_dw (K-ft) | Live Load (LL+IM) HL-93, One Lane +M (K-ft) | -M (K-ft) | (LL+IM) P15, One Lane (K-ft) | (K-ft) |
|---|---|---|---|---|---|---|---|---|
| | 0.0 | -0.18 | 0.03 | -0.04 | 10.71 | -166.86 | 83.44 | -183.87 |
| | 0.1 | 1231.42 | 200.86 | 264.41 | 1343.41 | -28.16 | 2323.11 | 0 |
| | 0.2 | 2141.07 | 332.19 | 460.97 | 2317.27 | -54.12 | 4068.34 | 0 |
| | 0.3 | 2683.34 | 411.09 | 578.08 | 2888.66 | -60.97 | 5065.02 | 0 |
| 1 | 0.4 | 2891.61 | 424.01 | 624.21 | 3075.31 | -69.59 | 5372.38 | 0 |
| | 0.5 | 2720.48 | 402.25 | 586.84 | 2938.06 | -71.08 | 5162.02 | 0 |
| | 0.6 | 2214.71 | 317.74 | 478.04 | 2416.06 | -74.18 | 4230.49 | 0 |
| | 0.7 | 1337.2 | 199.69 | 288.12 | 1499.56 | -41.49 | 2543.03 | 0 |
| | 0.8 | 129.13 | 17.51 | 27.93 | 166.05 | -12.84 | 235.78 | -14.91 |
| | 0.9 | -1374.72 | -203.49 | -294.38 | 98.39 | -1813.04 | 3.23 | -2933.84 |
| | 1.0 | -3253.84 | -503.93 | -695.89 | 211.56 | -3930.34 | 0 | -6536.87 |
| | 0.0 | -3253.84 | -503.93 | -695.89 | 211.56 | -3930.34 | 0 | -6536.87 |
| | 0.1 | -1377.09 | -182.69 | -291.06 | 1590.62 | -3296.48 | 2925.17 | -5422.78 |
| | 0.2 | 145.27 | 58.23 | 35.97 | 2767.36 | -2381.35 | 5187.13 | -4658.43 |
| | 0.3 | 1273.04 | 225.8 | 276.58 | 3479.25 | -1990.62 | 6625.94 | -3894.09 |
| 2 | 0.4 | 1973.31 | 318.06 | 425.81 | 3749.51 | -1599.89 | 7173.3 | -3129.74 |
| | 0.5 | 2238.99 | 339.52 | 481.83 | 3624.29 | -1209.16 | 6862.16 | -2365.39 |
| | 0.6 | 2038.73 | 309.21 | 437.61 | 3057.42 | -818.44 | 5676 | -1601.04 |
| | 0.7 | 1406.88 | 206.72 | 301.64 | 2043.61 | -427.71 | 3735.81 | -836.69 |
| | 0.8 | 325.22 | 45.71 | 69.82 | 551.34 | -36.98 | 992.78 | -72.34 |
| | 0.9 | -1050.9 | -160.77 | -224.38 | 353.92 | -1658.1 | 692.01 | -2929.93 |
| | 1.0 | -2967.16 | -477.33 | -632.73 | 744.48 | -4149.61 | 1456.36 | -7267.9 |
| | 0.0 | -2967.16 | -477.33 | -632.73 | 744.48 | -4149.61 | 1456.36 | -7267.9 |
| | 0.1 | -1143.63 | -160.54 | -238.57 | 1882.17 | -3588.26 | 3362.54 | -6077.9 |
| | 0.2 | 336.17 | 74.71 | 78.98 | 3251.31 | -2785.28 | 6260 | -5374.6 |
| | 0.3 | 1431.49 | 235.76 | 311.92 | 4199.46 | -2550.56 | 8318.74 | -4671.3 |
| 3 | 0.4 | 2101.9 | 321.05 | 454.32 | 4758.12 | -2315.83 | 9569.89 | -3968 |
| | 0.5 | 2339.47 | 336.1 | 504.2 | 4905.76 | -2081.11 | 9976.65 | -3264.7 |
| | 0.6 | 2119.04 | 298.8 | 455.32 | 4715.4 | -2274.1 | 9505.4 | -3587.99 |
| | 0.7 | 1472 | 189.46 | 315.34 | 4114.02 | -2479.7 | 8321.3 | -4223.94 |
| | 0.8 | 384.2 | 25.48 | 80.14 | 3123.15 | -2685.31 | 6257.58 | -4859.88 |
| | 0.9 | -1104.31 | -209.83 | -241.55 | 1711.29 | -3423.23 | 3362.54 | -5495.83 |
| | 1.0 | -2995.04 | -522.96 | -649.39 | 122.28 | -3826.96 | 251.8 | -6142.13 |
| | 0.0 | -2995.04 | -522.96 | -649.39 | 122.28 | -3826.96 | 251.8 | -6142.13 |
| | 0.1 | -1301.5 | -221.62 | -281.66 | 160.26 | -1813.04 | 3.23 | -2933.84 |
| | 0.2 | 163.23 | 26.41 | 35.47 | 182.35 | -23.39 | 235.78 | -14.91 |
| | 0.3 | 1302.35 | 202.35 | 281.99 | 1499.56 | -149.85 | 2547.6 | 0 |
| 4 | 0.4 | 2216.5 | 329.15 | 475.59 | 2416.06 | -114.87 | 4230.49 | 0 |
| | 0.5 | 2752.27 | 400.43 | 590.05 | 2938.06 | -107.06 | 5163.99 | 0 |
| | 0.6 | 2921.93 | 412.79 | 627.82 | 3075.31 | -106.7 | 5372.38 | 0 |
| | 0.7 | 2705.94 | 388.61 | 581.82 | 2888.66 | -99.49 | 5052.64 | 0 |
| | 0.8 | 2151.32 | 304.92 | 463.44 | 2317.27 | -88.23 | 4068.35 | 0 |
| | 0.9 | 1238.26 | 180.94 | 266.85 | 1343.41 | -63.08 | 2323.11 | 0 |
| | 1.0 | 2.33 | -3.97 | 1.18 | 10.71 | -63.37 | 12.22 | -183.87 |

4/21 Log
FW

## Factored Dead and Live Load - Moments and Load Combinations

DC1: Permanent dead load. DC1 is defined as structural dead load acting on the non-composite section.
(DC1 consists of deck slab concrete, steel girder, bracing system, use of steel deck form.)
DC2: Permanent dead load. DC2 is defined as non-structural dead load acting on the longterm composite section.
(DC2 consists of barrier rails and specified utility)
DW: Wearing surface.

Strength I: 1.25(DC)+1.5(DW)+1.75(LL+IM) HL-93
Strength I: 1.25(DC)+1.5(DW)+1.35(LL+IM) P15

| Factor | | 1.25 | 1.25 | 1.5 | 1.75 | 1.75 | 1.35 | 1.35 | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Dead Load | | | Live Load | | | | | | | |
| Span | Point | DC1 M_dc1 (K-ft) | DC2 M_dc2 (K-ft) | DW M_dw (K-ft) | (LL+IM) HL-93, One Lane +M (K-ft) | -M (K-ft) | (LL+IM) P15, One Lane (K-ft) | (K-ft) | Strength I +M (K-ft) | -M (K-ft) | Strength II (K-ft) | (K-ft) |
| | 0.0 | 0 | 0 | 0 | 19 | -292 | 113 | -248 | 18 | -292 | 112 | -248 |
| | 0.1 | 1539 | 251 | 397 | 2351 | -49 | 3136 | 0 | 4538 | 2138 | 5323 | 2187 |
| | 0.2 | 2676 | 415 | 691 | 4055 | -95 | 5492 | 0 | 7838 | 3688 | 9275 | 3783 |
| | 0.3 | 3354 | 514 | 867 | 5055 | -107 | 6838 | 0 | 9790 | 4628 | 11573 | 4735 |
| 1 | 0.4 | 3615 | 530 | 936 | 5382 | -122 | 7253 | 0 | 10463 | 4959 | 12334 | 5081 |
| | 0.5 | 3401 | 503 | 880 | 5142 | -124 | 6969 | 0 | 9925 | 4659 | 11752 | 4784 |
| | 0.6 | 2768 | 397 | 717 | 4228 | -130 | 5711 | 0 | 8111 | 3753 | 9594 | 3883 |
| | 0.7 | 1672 | 250 | 432 | 2624 | -73 | 3433 | 0 | 4978 | 2281 | 5786 | 2353 |
| | 0.8 | 161 | 22 | 42 | 291 | -22 | 318 | -20 | 516 | 203 | 543 | 205 |
| | 0.9 | -1718 | -254 | -442 | 172 | -3173 | 4 | -3961 | -2242 | -5587 | -2410 | -6375 |
| | 1.0 | -4067 | -630 | -1044 | 370 | -6878 | 0 | -8825 | -5371 | -12619 | -5741 | -14566 |
| | 0.0 | -4067 | -630 | -1044 | 370 | -6878 | 0 | -8825 | -5371 | -12619 | -5741 | -14566 |
| | 0.1 | -1721 | -228 | -437 | 2784 | -5769 | 3949 | -7321 | 397 | -8155 | 1563 | -9707 |
| | 0.2 | 182 | 73 | 54 | 4843 | -4167 | 7003 | -6289 | 5151 | -3859 | 7311 | -5981 |
| | 0.3 | 1591 | 282 | 415 | 6089 | -3484 | 8945 | -5257 | 8377 | -1195 | 11233 | -2969 |
| 2 | 0.4 | 2467 | 398 | 639 | 6562 | -2800 | 9684 | -4225 | 10065 | 703 | 13187 | -722 |
| | 0.5 | 2799 | 424 | 723 | 6343 | -2116 | 9264 | -3193 | 10288 | 1830 | 13210 | 753 |
| | 0.6 | 2548 | 387 | 656 | 5350 | -1432 | 7663 | -2161 | 8942 | 2159 | 11254 | 1430 |
| | 0.7 | 1759 | 258 | 452 | 3576 | -748 | 5043 | -1130 | 6046 | 1721 | 7513 | 1340 |
| | 0.8 | 407 | 57 | 105 | 965 | -65 | 1340 | -98 | 1533 | 504 | 1909 | 471 |
| | 0.9 | -1314 | -201 | -337 | 619 | -2902 | 934 | -3955 | -1232 | -4753 | -917 | -5807 |
| | 1.0 | -3709 | -597 | -949 | 1303 | -7262 | 1966 | -9812 | -3952 | -12517 | -3289 | -15066 |
| | 0.0 | -3709 | -597 | -949 | 1303 | -7262 | 1966 | -9812 | -3952 | -12517 | -3289 | -15066 |
| | 0.1 | -1430 | -201 | -358 | 3294 | -6279 | 4539 | -8205 | 1306 | -8268 | 2551 | -10193 |
| | 0.2 | 420 | 93 | 118 | 5690 | -4874 | 8451 | -7256 | 6322 | -4242 | 9083 | -6624 |
| | 0.3 | 1789 | 295 | 468 | 7349 | -4463 | 11230 | -6306 | 9901 | -1912 | 13782 | -3754 |
| 3 | 0.4 | 2627 | 401 | 681 | 8327 | -4053 | 12919 | -5357 | 12037 | -343 | 16630 | 1647 |
| | 0.5 | 2924 | 420 | 756 | 8585 | -3642 | 13468 | -4407 | 12686 | 459 | 17569 | -307 |
| | 0.6 | 2649 | 374 | 683 | 8252 | -3980 | 12832 | -4844 | 11957 | -274 | 16538 | -1139 |
| | 0.7 | 1840 | 237 | 473 | 7200 | -4339 | 11234 | -5702 | 9749 | -1790 | 13784 | -3152 |
| | 0.8 | 480 | 32 | 120 | 5466 | -4699 | 8448 | -6561 | 6098 | -4067 | 9080 | -5929 |
| | 0.9 | -1380 | -262 | -362 | 2995 | -5991 | 4539 | -7419 | 990 | -7996 | 2534 | -9424 |
| | 1.0 | -3744 | -654 | -974 | 214 | -6697 | 340 | -8292 | -5158 | -12069 | -5032 | -13663 |
| | 0.0 | -3744 | -654 | -974 | 214 | -6697 | 340 | -8292 | -5158 | -12069 | -5032 | -13663 |
| | 0.1 | -1627 | -277 | -422 | 280 | -3173 | 4 | -3961 | -2046 | -5499 | -2322 | -6287 |
| | 0.2 | 204 | 33 | 53 | 319 | -41 | 318 | -20 | 609 | 249 | 609 | 270 |
| | 0.3 | 1628 | 253 | 423 | 2624 | -262 | 3439 | 0 | 4928 | 2042 | 5743 | 2304 |
| 4 | 0.4 | 2771 | 411 | 713 | 4228 | -201 | 5711 | 0 | 8124 | 3694 | 9607 | 3895 |
| | 0.5 | 3440 | 501 | 885 | 5142 | -187 | 6971 | 0 | 9968 | 4639 | 11797 | 4826 |
| | 0.6 | 3652 | 516 | 942 | 5382 | -187 | 7253 | 0 | 10492 | 4923 | 12363 | 5110 |
| | 0.7 | 3382 | 486 | 873 | 5055 | -174 | 6821 | 0 | 9796 | 4567 | 11562 | 4741 |
| | 0.8 | 2689 | 381 | 695 | 4055 | -154 | 5492 | 0 | 7821 | 3611 | 9258 | 3765 |
| | 0.9 | 1548 | 226 | 400 | 2351 | -110 | 3136 | 0 | 4525 | 2064 | 5310 | 2174 |
| | 1.0 | 3 | -5 | 2 | 19 | -111 | 16 | -248 | 18 | -111 | 16 | -249 |

Shaded Area are the controlling Design Forces

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FW

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## Unfactored Dead and Live Load - Shear

DC1: Permanent dead load. DC1 is defined as structural dead load acting on the non-composite section.
(DC1 consists of deck slab concrete, steel girder, bracing system, use of steel deck form.)
DC2: Permanent dead load. DC2 is defined as non-structural dead load acting on the lonterm composite section.
(DC2 consists of barrier rails and specified utility)
DW: Wearing surface.

| Span | Point | Dead Load | | | Live Load | | | |
|---|---|---|---|---|---|---|---|---|
| | | DC1 | DC2 | DW | (LL+IM) HL-93, One Lane | | (LL+IM) P15, One Lane | |
| | | V_dc1 | V_dc2 | V_dw | +V | -V | +V | -V |
| | | (K) | (K) | (K) | (K) | (K) | (K) | (K) |
| | 0.0 | 104.28 | 17.69 | 22.37 | 119.91 | 0 | 229.8 | 0 |
| | 0.1 | 80.15 | 12.51 | 17.24 | 102.44 | -7.48 | 187.2 | -8.31 |
| | 0.2 | 56.12 | 7.17 | 12.17 | 83.69 | -17.56 | 142.27 | -22.02 |
| | 0.3 | 27.62 | 3.64 | 5.99 | 65.99 | -31.84 | 103.15 | -41.15 |
| 1 | 0.4 | 3.59 | -1.7 | 0.92 | 49.35 | -47.3 | 69.93 | -66.07 |
| | 0.5 | -25.87 | -3.66 | -5.61 | 33.75 | -63.82 | 44.33 | -98.92 |
| | 0.6 | -49.9 | -9 | -10.69 | 19.21 | -81.38 | 24.14 | -137.18 |
| | 0.7 | -78.48 | -10.98 | -16.95 | 8.67 | -100 | 9.36 | -180.85 |
| | 0.8 | -102.51 | -16.32 | -22.03 | 0.68 | -119.66 | 1.06 | -230.68 |
| | 0.9 | -128.74 | -19.83 | -27.54 | 0 | -135.41 | 0 | -280.82 |
| | 1.0 | -152.77 | -25.17 | -32.61 | 0 | -145.38 | 0 | -321.99 |
| | 0.0 | 137.67 | 24.24 | 29.69 | 135.03 | 0 | 267.5 | 0 |
| | 0.1 | 114.97 | 19.04 | 24.65 | 116.64 | -7.49 | 218.42 | -8.1 |
| | 0.2 | 88.69 | 14.17 | 18.89 | 97.28 | -18.14 | 183.93 | -22.68 |
| | 0.3 | 61.69 | 8.17 | 13.19 | 79.06 | -32.51 | 149.21 | -43.74 |
| 2 | 0.4 | 31.21 | 4.43 | 6.58 | 62.01 | -48.03 | 111.57 | -71.28 |
| | 0.5 | 4.21 | -1.57 | 0.88 | 46.1 | -64.71 | 82.16 | -105.3 |
| | 0.6 | -27.61 | -4.55 | -5.98 | 31.34 | -82.53 | 60.07 | -145.8 |
| | 0.7 | -58.61 | -7.73 | -12.6 | 27.07 | -101.51 | 50.96 | -194.4 |
| | 0.8 | -85.61 | -13.73 | -18.3 | 26.11 | -121.64 | 50.96 | -249.48 |
| | 0.9 | -114.25 | -18.1 | -24.37 | 26.05 | -139.27 | 50.96 | -303.12 |
| | 1.0 | -141.25 | -24.1 | -30.07 | 26.05 | -150.73 | 50.96 | -346.15 |
| | 0.0 | 133.92 | 23.97 | 28.96 | 143.91 | -23.13 | 300.96 | -42.4 |
| | 0.1 | 112.12 | 18.66 | 24.02 | 127.11 | -23.61 | 257.1 | -42.4 |
| | 0.2 | 86.52 | 13.74 | 18.38 | 109.37 | -25.05 | 207.9 | -42.4 |
| | 0.3 | 59.52 | 7.74 | 12.68 | 92.59 | -35.63 | 170.1 | -51.91 |
| 3 | 0.4 | 29.34 | 4 | 6.18 | 76.78 | -48.57 | 135.92 | -74.77 |
| | 0.5 | 2.34 | -2 | 0.48 | 60.19 | -62.63 | 97.6 | -102.1 |
| | 0.6 | -29.01 | -4.91 | -6.31 | 48.03 | -77.32 | 74.88 | -135.58 |
| | 0.7 | -59.02 | -7.93 | -12.83 | 35.09 | -93.14 | 52.07 | -169.34 |
| | 0.8 | -86.02 | -13.93 | -18.53 | 24.69 | -109.91 | 46.89 | -207.9 |
| | 0.9 | -112.55 | -17.87 | -24.34 | 23.25 | -127.65 | 46.89 | -257.1 |
| | 1.0 | -139.55 | -23.87 | -30.04 | 22.77 | -144.45 | 46.89 | -300.96 |
| | 0.0 | 138.28 | 25.14 | 29.98 | 145.38 | 0 | 321.99 | 0 |
| | 0.1 | 118.91 | 20.62 | 25.7 | 135.41 | 0 | 280.82 | 0 |
| | 0.2 | 94.88 | 15.28 | 20.62 | 119.66 | -0.68 | 230.68 | -1.06 |
| | 0.3 | 80.49 | 12.17 | 17.04 | 100 | -8.67 | 180.85 | -9.36 |
| 4 | 0.4 | 56.46 | 6.83 | 11.97 | 81.38 | -19.21 | 137.18 | -24.14 |
| | 0.5 | 24.72 | 3.6 | 5.37 | 63.82 | -33.75 | 98.92 | -44.33 |
| | 0.6 | 0.69 | -1.74 | 0.29 | 47.3 | -49.35 | 66.07 | -69.93 |
| | 0.7 | -28.96 | -4.08 | -6.17 | 31.84 | -65.99 | 41.15 | -103.15 |
| | 0.8 | -56.38 | -6.62 | -12.19 | 17.56 | -83.69 | 22.02 | -142.47 |
| | 0.9 | -80.41 | -11.96 | -17.26 | 7.48 | -102.44 | 8.31 | -187.2 |
| | 1.0 | -104.26 | -14.26 | -22.6 | 0 | -119.41 | 0 | -229.8 |

4/22/09
PW

## Factored Dead and Live Load - Shear and Load Combinations, Strength Limit State

DC1: Permanent dead load. DC1 is defined as structural dead load acting on the non-composite section.
(DC1 consists of deck slab concrete, steel girder, bracing system, use of steel deck form.)
DC2: Permanent dead load. DC2 is defined as non-structural dead load acting on the longterm composite section.
(DC2 consists of barrier rails and specified utility)
DW: Wearing surface.

Strength I: 1.25(DC)+1.5(DW)+1.75(LL+IM) HL-93
Strength I: 1.25(DC)+1.5(DW)+1.35(LL+IM) P15

| Factor | | 1.25 | 1.25 | 1.5 | 1.75 | 1.75 | 1.35 | 1.35 | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Span | Point | Dead Load | | | Live Load | | | | | | | |
| | | DC1 | DC2 | DW | (LL+IM) HL-93, One Lane | | (LL+IM) P15, One Lane | | Strength I | | Strength II | |
| | | V_dc1 | V_dc2 | V_dw | +V | -V | +V | -V | +V | -V | +V | -V |
| | | (K) | (K) | (K) | (K) | (K) | (K) | (K) | (K) | (K) | (K) | (K) |
| | 0.0 | 130 | 22 | 34 | 210 | 0 | 310 | 0 | 396 | 186 | 496 | 186 |
| | 0.1 | 100 | 16 | 26 | 179 | -13 | 253 | -11 | 321 | 129 | 394 | 130 |
| | 0.2 | 70 | 9 | 18 | 146 | -31 | 192 | -30 | 244 | 67 | 289 | 68 |
| | 0.3 | 35 | 5 | 9 | 115 | -56 | 139 | -56 | 164 | -8 | 187 | -7 |
| 1 | 0.4 | 4 | -2 | 1 | 86 | -83 | 94 | -89 | 90 | -79 | 98 | -85 |
| | 0.5 | -32 | -5 | -8 | 59 | -112 | 60 | -134 | 14 | -157 | 15 | -179 |
| | 0.6 | -62 | -11 | -16 | 34 | -142 | 33 | -185 | -56 | -232 | -57 | -275 |
| | 0.7 | -98 | -14 | -25 | 15 | -175 | 13 | -244 | -122 | -312 | -125 | -381 |
| | 0.8 | -128 | -20 | -33 | 1 | -209 | 1 | -311 | -180 | -391 | -180 | -493 |
| | 0.9 | -161 | -25 | -41 | 0 | -237 | 0 | -379 | -227 | -464 | -227 | -606 |
| | 1.0 | -191 | -31 | -49 | 0 | -254 | 0 | -435 | -271 | -526 | -271 | -706 |
| | 0.0 | 172 | 30 | 45 | 236 | 0 | 361 | 0 | 483 | 247 | 608 | 247 |
| | 0.1 | 144 | 24 | 37 | 204 | -13 | 295 | -11 | 409 | 191 | 499 | 194 |
| | 0.2 | 111 | 18 | 28 | 170 | -32 | 248 | -31 | 327 | 125 | 405 | 126 |
| | 0.3 | 77 | 10 | 20 | 138 | -57 | 201 | -59 | 245 | 50 | 309 | 48 |
| 2 | 0.4 | 39 | 6 | 10 | 109 | -84 | 151 | -96 | 163 | -30 | 205 | -42 |
| | 0.5 | 5 | -2 | 1 | 81 | -113 | 111 | -142 | 85 | -109 | 116 | -138 |
| | 0.6 | -35 | -6 | -9 | 55 | -144 | 81 | -197 | 6 | -194 | 32 | -246 |
| | 0.7 | -73 | -10 | -19 | 47 | -178 | 69 | -262 | -54 | -279 | -33 | -364 |
| | 0.8 | -107 | -17 | -27 | 46 | -213 | 69 | -337 | -106 | -364 | -83 | -488 |
| | 0.9 | -143 | -23 | -37 | 46 | -244 | 69 | -409 | -156 | -446 | -133 | -611 |
| | 1.0 | -177 | -30 | -45 | 46 | -264 | 69 | -467 | -206 | -516 | -183 | -719 |
| | 0.0 | 167 | 30 | 43 | 252 | -40 | 406 | -57 | 493 | 200 | 647 | 184 |
| | 0.1 | 140 | 23 | 36 | 222 | -41 | 347 | -57 | 422 | 158 | 547 | 142 |
| | 0.2 | 108 | 17 | 28 | 191 | -44 | 281 | -57 | 344 | 109 | 434 | 96 |
| | 0.3 | 74 | 10 | 19 | 162 | -62 | 230 | -70 | 265 | 41 | 333 | 33 |
| 3 | 0.4 | 37 | 5 | 9 | 134 | -85 | 183 | -101 | 185 | -34 | 234 | -50 |
| | 0.5 | 3 | -3 | 1 | 105 | -110 | 132 | -138 | 106 | -108 | 133 | -137 |
| | 0.6 | -36 | -6 | -9 | 84 | -135 | 101 | -183 | 32 | -187 | 49 | -235 |
| | 0.7 | -74 | -10 | -19 | 61 | -163 | 70 | -229 | -42 | -266 | -33 | -332 |
| | 0.8 | -108 | -17 | -28 | 43 | -192 | 63 | -281 | -110 | -345 | -89 | -433 |
| | 0.9 | -141 | -22 | -37 | 41 | -223 | 63 | -347 | -159 | -423 | -136 | -547 |
| | 1.0 | -174 | -30 | -45 | 40 | -253 | 63 | -406 | -209 | -502 | -186 | -656 |
| | 0.0 | 173 | 31 | 45 | 254 | 0 | 435 | 0 | 504 | 249 | 684 | 249 |
| | 0.1 | 149 | 26 | 39 | 237 | 0 | 379 | 0 | 450 | 213 | 592 | 213 |
| | 0.2 | 119 | 19 | 31 | 209 | -1 | 311 | -1 | 378 | 167 | 480 | 167 |
| | 0.3 | 101 | 15 | 26 | 175 | -15 | 244 | -13 | 316 | 126 | 386 | 129 |
| 4 | 0.4 | 71 | 9 | 18 | 142 | -34 | 185 | -33 | 239 | 63 | 282 | 64 |
| | 0.5 | 31 | 5 | 8 | 112 | -59 | 134 | -60 | 155 | -16 | 177 | -16 |
| | 0.6 | 1 | -2 | 0 | 83 | -86 | 89 | -94 | 82 | -87 | 88 | -95 |
| | 0.7 | -36 | -5 | -9 | 56 | -115 | 56 | -139 | 5 | -166 | 5 | -190 |
| | 0.8 | -70 | -8 | -18 | 31 | -146 | 30 | -192 | -66 | -243 | -67 | -289 |
| | 0.9 | -101 | -15 | -26 | 13 | -179 | 11 | -253 | -128 | -321 | -130 | -394 |
| | 1.0 | -130 | -18 | -34 | 0 | -209 | 0 | -310 | -182 | -391 | -182 | -492 |

shaded Area are the controlling Design Forces.

5/13/09

## Unfactored Dead and Live Load - Support Reaction

DC1: Permanent dead load. DC1 is defined as structural dead load acting on the non-composite section.
(DC1 consists of deck slab concrete, steel girder, bracing system, use of steel deck form.)
DC2: Permanent dead load. DC2 is defined as non-structural dead load acting on the longterm composite section.
(DC2 consists of barrier rails and specified utility)
DW: Wearing surface.

| Loc | Dead Load | | | Live Load | |
|---|---|---|---|---|---|
| | DC1 R_dc1 (Kips) | DC2 R_dc2 (Kips) | DW R_dw (Kips) | (LL+IM) HL-93, C +R (Kips) | (LL+IM) P15, One Truck +R (Kips) |
| Abutment 1 | 104.28 | 17.69 | 22.37 | 131.16 | 229.80 |
| Bent 2 | 290.44 | 49.41 | 62.30 | 241.84 | 449.00 |
| Bent 3 | 275.17 | 48.07 | 59.03 | 253.00 | 458.00 |
| Bent 4 | 277.83 | 49.01 | 60.02 | 248.00 | 450.00 |
| Abutment 5 | 104.26 | 14.26 | 22.60 | 131.16 | 229.80 |

## Factored Dead and Live Load - Support Reaction

| Factor | 1.25 | 1.25 | 1.5 | 1.75 | 1.35 | | |
|---|---|---|---|---|---|---|---|
| Loc | Dead Load | | | Live Load | | | |
| | DC1 R_dc1 (Kips) | DC2 R_dc2 (Kips) | DW R_dw (Kips) | (LL+IM) HL-93, One Lane +R (Kips) | (LL+IM) P15, One Truck +R (Kips) | Strength I +R (K-ft) | Strength II +R (K-ft) |
| Abutment 1 | 130.35 | 22.11 | 33.56 | 229.53 | 310.23 | 415.55 | 496.25 |
| Bent 2 | 363.05 | 61.76 | 93.45 | 423.22 | 606.15 | 941.48 | 1124.41 |
| Bent 3 | 343.96 | 60.09 | 88.55 | 442.75 | 618.30 | 935.35 | 1110.90 |
| Bent 4 | 347.29 | 61.26 | 90.03 | 434.00 | 607.50 | 932.58 | 1106.08 |
| Abutment 5 | 130.33 | 17.83 | 33.90 | 229.53 | 310.23 | 411.58 | 492.28 |

Check strength limit state.

Composite section, in positive moment Region

<u>Determine effective Flange width</u>

$S = 10'$ $L = 108.5'$ or $150'$ or $200$

$S/L = 0.09 < 0.32$

$b_{eff} = b = 10'$ ← use to calculate section property.

<u>Check section Compactness</u> (AASHTO 6.10.6.2.2)

$F_{yf} = 50$ ksi $< 70$ ksi

web $\frac{D}{t_w} = \frac{102}{0.75} = 136 < 150$

web slenderness $\frac{2D_{cp}}{t_w} \leq 3.76\sqrt{\frac{E}{F_{yc}}}$

$D_{cp}$ is the depth of the web in compression at the plastic moment. It is determined as follows.

Compressive force in concrete slab

$P_s = 0.85 \times f_c' \times b_{eff} \times t_s$

$= 0.85 \times (4) \times (10' \times 12'') \times 10'' = 4080$ kips

Yield force at top flange

$P_c = A_{fc} \times F_{yc} = (18 \times 1) \times (50) = 900$ kips

Yield force in the web

$P_w = A_w F_{yw} = (102'' \times 0.75)(50) = 3825$ kips

Yield force in the bottom flange

$P_t = A_{ft} \times F_{yt} = (18 \times 1) \times (50) = 900$ kips

$P_s + P_c > P_w + P_t \Rightarrow D_{cp} = 0$

$F_{yc} = 50^{ksi}$, $E = 29{,}000$ ksi

$$3.76\sqrt{\frac{E}{F_{yc}}} = 3.76\sqrt{\frac{29{,}000}{50}} = 90.55$$

$\boxed{\text{Section is compact}}$

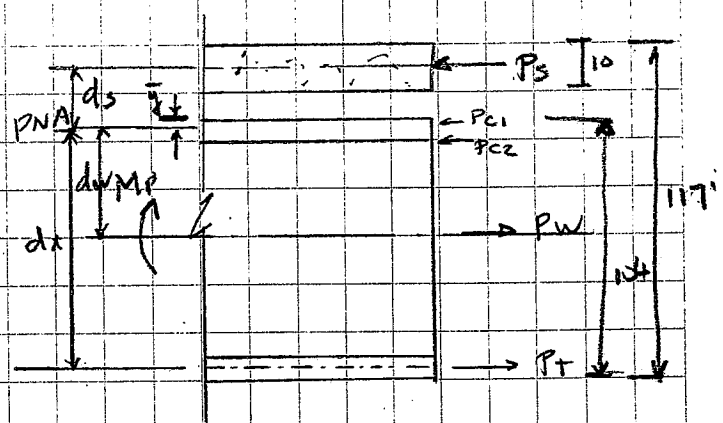
(Note, when bottom flange thickness increase more than 1 1/4", need to revisit compact section assumption)

## Calculate plastic moment Mp.

$$\bar{y} = \frac{1}{2} t_{fc}\left(\frac{P_w + P_t - P_s}{P_c} + 1\right)$$

$$\bar{y} = \frac{1}{2}(1)\left(\frac{3825 + 900 - 4080}{900} + 1\right)$$

$$= 0.86$$




$$M_p = \Sigma M_{PNA} = P_s d_s + P_{c1} \times \left(\frac{\bar{y}}{2}\right) + \underbrace{P_{c2} \times \left(\frac{t_{cf} - \bar{y}}{2}\right)}_{b_{fc} F_{yc}\left[\frac{\bar{y} + (t_{cf} - \bar{y})^2}{2}\right]} + P_w d_w + P_T \times d_t.$$

$d_t = 104" - 1/2" - 0.86" = 102.64"$

$d_w = 102/2 + 1 - 0.86 = 51.14"$

$d_s = 117" - 1/2 - 0.5 - 102.64 = 8.86"$

$$M_p = 4080 \times 8.86 + 18 \times 50\left[\frac{0.86 + (1 - 0.86)^2}{2}\right]$$

$$+ 3825 \times 51.14 + 900 \times 102.64$$

$$= 36149 + 396 + 195610 + 92376 = 324531 \text{ k-in}$$

$$= 27{,}044 \text{ k-ft}$$

Calculating Yield Moment $M_y$ $M_{D1} = 1 M_{DL1}$

$M_{D1} = 3650$ kips (Span 1, 0.4 pt & span 4 0.6 pt)
(control)

or $M_{D1} = 2950$ kips (Span 2 & span 3 0.5 pt)

$M_{D2} = M_{DC2} + M_{DW}$

$M_{D2} = 530 + 936 = 1466$ kips or
$420 + 756 = 1176$ kips.

$$M_{AD} = S_{ST}\left(F_y - \frac{M_{D1}}{S_{NC}} - \frac{M_{D2}}{S_{LT}}\right)$$

for Top flange

$$M_{AD} = 22239\left(50 - \frac{3650 \times 12}{3100} - \frac{1466 \times 12}{8537}\right)$$

(14.13 ; 2.06)

$= 751907$ k-in $= 62659$ k-ft

For Bottom flange.

$$M_{AD} = 4571\left(50 - \frac{3650 \times 12}{3105} - \frac{1466 \times 12}{4068}\right)$$

(14.10)

$= 144370$ k-in $= 12030$ kip-ft ← Control

$M_y = M_{D1} + M_{D2} + M_{AD}$

$= 3650 + 1466 + 12030 = 17147$ k-ft

Calculate Flexural Resistance $M_N$, LRFD 6.10.7.1.2

$$M_N = \min \begin{cases} M_p & \text{IF } D_p < 0.1 D_t \\ M_p \left[ 1 - \left(1 - \frac{M_y}{M_p}\right) \left[ \frac{\frac{D_p}{D_t} - 0.1}{0.32} \right] \right] \\ 1.3 R_h M_y & \text{for continuous span} \end{cases}$$

$D_p$ = Distance from Top of concrete Deck to PNA

$D_p = 5 + d_s = 13.86$ in

$D_t$ = Total Depth of the composite section

$D_t = 117$ in

$\therefore D_p > 0.1 D_t$

$$M_p \left[ 1 - \left(1 - \frac{M_y}{M_p}\right) \left( \frac{\frac{D_p}{D_t} - 0.1}{0.32} \right) \right]$$

$$= 27044 \left( 1 - \left(1 - \overset{=0.366}{\frac{17147}{27044}}\right) \times \left( \frac{\overset{=0.0577}{\frac{13.86}{117} - 0.1}}{0.32} \right) \right]$$

$= 26473$ K-ft

$1.3 R_h M_y = 1.3(1) M_y = 1.3 \times 17147 = 22291$ K-ft

$\therefore M_N = 22291$ K-ft

$M_{u(MAX)} = 17569$ K-ft $< \phi_f M_N = (1)(22291)$

4/28/09

FW



## Factored Dead and Live Load - Moments, Service Limit State II

DC1: Permanent dead load. DC1 is defined as structural dead load acting on the non-composite section.
(DC1 consists of deck slab concrete, steel girder, bracing system, use of steel deck form.)
DC2: Permanent dead load. DC2 is defined as non-structural dead load acting on the longterm composite section.
(DC2 consists of barrier rails and specified utility)
DW: Wearing surface.

Service II: 1.0(DC)+1.0(DW)+1.30(LL+IM) HL-93

| Factor | | 1 | 1 | 1 | 1.3 | 1.3 |
|---|---|---|---|---|---|---|
| Span | Point | Dead Load | | | Live Load | |
| | | DC1 | DC2 | DW | (LL+IM) HL-93, One Lane | |
| | | M_dc1 | M_dc2 | M_dw | +M | -M |
| | | (K-ft) | (K-ft) | (K-ft) | (K-ft) | (K-ft) |
| | 0.0 | 0 | 0 | 0 | 14 | -217 |
| | 0.1 | 1231 | 201 | 264 | 1746 | -37 |
| | 0.2 | 2141 | 332 | 461 | 3012 | -70 |
| | 0.3 | 2683 | 411 | 578 | 3755 | -79 |
| 1 | 0.4 | 2892 | 424 | 624 | 3998 | -90 |
| | 0.5 | 2720 | 402 | 587 | 3819 | -92 |
| | 0.6 | 2215 | 318 | 478 | 3141 | -96 |
| | 0.7 | 1337 | 200 | 288 | 1949 | -54 |
| | 0.8 | 129 | 18 | 28 | 216 | -17 |
| | 0.9 | -1375 | -203 | -294 | 128 | -2357 |
| | 1.0 | -3254 | -504 | -696 | 275 | -5109 |
| | 0.0 | -3254 | -504 | -696 | 275 | -5109 |
| | 0.1 | -1377 | -183 | -291 | 2068 | -4285 |
| | 0.2 | 145 | 58 | 36 | 3598 | -3096 |
| | 0.3 | 1273 | 226 | 277 | 4523 | -2588 |
| 2 | 0.4 | 1973 | 318 | 426 | 4874 | -2080 |
| | 0.5 | 2239 | 340 | 482 | 4712 | -1572 |
| | 0.6 | 2039 | 309 | 438 | 3975 | -1064 |
| | 0.7 | 1407 | 207 | 302 | 2657 | -556 |
| | 0.8 | 325 | 46 | 70 | 717 | -48 |
| | 0.9 | -1051 | -161 | -224 | 460 | -2156 |
| | 1.0 | -2967 | -477 | -633 | 968 | -5394 |
| | 0.0 | -2967 | -477 | -633 | 968 | -5394 |
| | 0.1 | -1144 | -161 | -239 | 2447 | -4665 |
| | 0.2 | 336 | 75 | 79 | 4227 | -3621 |
| | 0.3 | 1431 | 236 | 312 | 5459 | -3316 |
| 3 | 0.4 | 2102 | 321 | 454 | 6186 | -3011 |
| | 0.5 | 2339 | 336 | 504 | 6377 | -2705 |
| | 0.6 | 2119 | 299 | 455 | 6130 | -2956 |
| | 0.7 | 1472 | 189 | 315 | 5348 | -3224 |
| | 0.8 | 384 | 25 | 80 | 4060 | -3491 |
| | 0.9 | -1104 | -210 | -242 | 2225 | -4450 |
| | 1.0 | -2995 | -523 | -649 | 159 | -4975 |
| | 0.0 | -2995 | -523 | -649 | 159 | -4975 |
| | 0.1 | -1302 | -222 | -282 | 208 | -2357 |
| | 0.2 | 163 | 26 | 35 | 237 | -30 |
| | 0.3 | 1302 | 202 | 282 | 1949 | -195 |
| 4 | 0.4 | 2217 | 329 | 476 | 3141 | -149 |
| | 0.5 | 2752 | 400 | 590 | 3819 | -139 |
| | 0.6 | 2922 | 413 | 628 | 3998 | -139 |
| | 0.7 | 2706 | 389 | 582 | 3755 | -129 |
| | 0.8 | 2151 | 305 | 463 | 3012 | -115 |
| | 0.9 | 1238 | 181 | 267 | 1746 | -82 |
| | 1.0 | 2 | -4 | 1 | 14 | -82 |

6.10.4.2.2 check Service Limit state stress at flange

Maximum at span 4, 0.6 pt.

$M_{DC1} = 2922^{k\text{-}ft}$

$M_{DC2} + M_{DW} = 413 + 628 = 1041^{k\text{-}ft}$

$M_{(LL+IM)HL\text{-}93} = 3998^{k\text{-}ft}$

Bottom flange

$$f_f = \frac{M_{DC1}}{S_{NC}} + \frac{M_{DC2} + M_{DW}}{S_{LT}} + \frac{M_{(LL+IM)HL\text{-}93}}{S_{ST}}$$

$$= \frac{2922 \times 12}{3105} + \frac{1041 \times 12}{4068} + \frac{3998 \times 12}{4571}$$

$$= 11.3 + 3.07 + 10.5 = 24.9^{ksi}$$

$0.95 R_h F_{yf} = 0.95(1)(50) = 47.5^{ksi}$

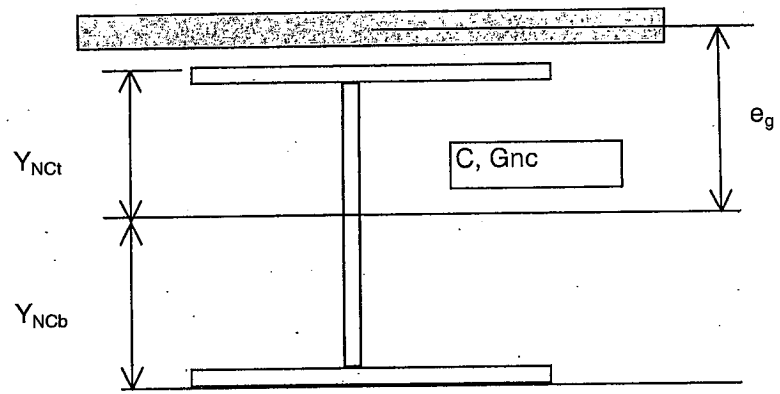
$f_f < 0.95 R_h F_{yf}$ OK.

$\frac{D}{t_w} < 150$, AASHTO 6.10.4.2.2-4 check is not required.

## Preliminary Section Properties - For Negative Moment Area

| Component | Ai ($in^2$) | Yi (in) | Ai*Yi ($In^3$) | $Y_{NCb}$ | $Y_{NCt}$ | $Yi-Y_{NCb}$ | $Ai(Yi-Y_{NCb})^2$ | Io |
|---|---|---|---|---|---|---|---|---|
| Top Flange 18"x1.5" | 27.00 | 104.25 | 2814.75 | | | 51.75 | 72308 | 5.06 |
| Web 102"x 3/4" | 76.50 | 52.50 | 4016.25 | | | 0.00 | 0 | 66326 |
| Bottom Flange 18"x1.5" | 27.00 | 0.75 | 20.25 | | | -51.75 | 72308 | 5.06 |
| Sum | 130.50 | | 6851.25 | 52.50 | 52.50 | | 144615 | 66336 |

| | | |
|---|---|---|
| $I_{NC}$ = | 210951 | $in^4$ |
| $e_g$ = | 60.00 | in |
| n = | 8 | |
| $K_g$ = | 5446008 | $in^4$ |
| $S_{NCb}$ = | 4018 | $in^3$ |
| $S_{NCt}$ = | 4018 | $in^3$ |



steel girders in negative moment regions are non-composite section. Controlling load is at Bent 2 to Bent 3 section.

AASHTO 6.10.6.2.3.

$F_y < 70$ ksi , satisfied.

Web $\frac{2D_c}{t_w} < 5.7\sqrt{\frac{E}{F_{yc}}}$

$\frac{D}{t_w} = \frac{102}{0.75} = 136$

$5.7\sqrt{\frac{E}{F_{yc}}} = 5.7\sqrt{\frac{29{,}000}{50}} = 137.3$ , satisfied

Flange $\frac{I_{yc}}{I_{yt}} = 1 > 0.3$ satisfied

use AASHTO Appendix A 6

Local Buckling Resistance ( AASHTO A.6.3.2 )

$\lambda_f = \frac{b_{fc}}{2t_{fc}} = \frac{18''}{2\times 1} = 9$

$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_{yc}}} = 0.38\sqrt{\frac{29{,}000}{50}} = 9.15$

$\lambda_f < \lambda_{pf}$

$\therefore M_{nc} = R_{pc} M_{yc}$

For web plastification factor, check web compaction.

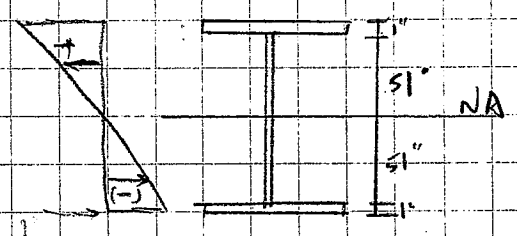
web is compact if

$$\frac{2D_{cp}}{t_w} \le \lambda_{pw}(D_{cp})$$

$$D_{cp} = D_c = 51 \text{ in}$$

$$\lambda_{pw}(D_{cp}) = \frac{\sqrt{\frac{E}{F_{yc}}}}{\left(0.54\frac{M_p}{R_h M_y} - 0.09\right)^2} \le \lambda_{rw}\left(\frac{D_{cp}}{D_c}\right)$$

$$M_p = \left(\left[(18'' \times 1'') \times 50 \times (51.5'')\right] + \left[(51 \times 0.75)(50) \times \left(\frac{51}{2}\right)\right]\right) \times 2$$




$$= 190237 \text{ k-in}$$

$$= 15853 \text{ k-ft}$$

$$M_y = S_x \cdot F_{yf} = 3112 \times 50 = 155600 \text{ k-in} = 12967 \text{ k-ft}$$

$$\lambda_{pw}(D_{cp}) = \frac{\sqrt{\frac{29{,}000}{50}}}{\left(0.54\frac{15853}{(12967)(1.0)} - 0.09\right)^2}$$

$$= \frac{24.08}{0.3251} = 74.07 < \lambda_{rw}\left(\frac{D_{cp}}{D_c}\right)$$

$$\lambda_{rw} = 5.7\sqrt{\frac{E}{F_{yc}}} = 5.7\sqrt{\frac{29{,}000}{50}} = 137.3$$

$$\lambda_{rw}\left(\frac{D_{cp}}{D_c}\right) = 137.3$$

$$\frac{2D_{cp}}{t_w} = \frac{2 \times 51}{0.75} = 136 > \lambda_{pw}(D_{cp})$$

$$\boxed{\text{web is non-compact}}$$

$$\lambda_w = \frac{2D_c}{t_w} = \frac{2 \times 51}{0.75} = 136 < \lambda_{rw} = 137.3$$

The web plasticfication factor

$$R_{pc} = \left[1 - \left(1 - \frac{R_h M_{yc}}{M_p}\right)\left(\frac{\lambda_w - \lambda_{pw(D_c)}}{\lambda_{rw} - \lambda_{pw(D_c)}}\right)\right]\frac{M_p}{M_{yc}} \le \frac{M_p}{M_{yc}}$$

$$\lambda_{pw(D_c)} = \lambda_{pw(D_{cp})}\left(\frac{D_c}{D_{cp}}\right) \le \lambda_{pw}$$

$\because D_c = D_{cp} \quad \therefore \lambda_{pw(D_c)} = \lambda_{pw(D_{cp})}$

$$R_{pc} = \left[1 - \left(1 - \frac{(1)(12967)}{15853}\right)\left(\frac{136 - 74.07}{137.3 - 74.07}\right)\right]\frac{15853}{12967}$$

$$= \left[1 - 0.182 \times 0.979\right] 1.222 = 1.004 < 1.222 \text{ o.k}$$

$$M_{nc(FLB)} = R_{pc} M_{yc} = (1.004) \times 12967 = 13020^{k\text{-}ft}$$

<u>Calculate Lateral Torsional Buckling Resistance</u>

In negative moment regions the bottom compression flange is braced by the cross Frame.

$$L_p = 1.0\, r_1 \sqrt{\frac{E}{F_{yc}}}$$

$$r_1 = \frac{b_{fc}}{\sqrt{12 \times \left(1 + \frac{1}{3}\frac{D_c t_w}{b_{fc} t_{fc}}\right)}} = \frac{18}{\sqrt{12 \times \left(1 + \frac{1}{3}\left(\frac{51 \times 0.75}{18 \times 1}\right)\right)}} = 3.98^{in}$$

$$L_p = 1 \times 3.98 \sqrt{\frac{29{,}000}{50}} = 95.85^{in}$$

FHW
5/6/09

LA River Bridge

Strength Limit State, Negative Moment



$$L_r = 1.95 r_t \frac{E}{F_{yr}} \sqrt{\frac{J}{S_{xc} h}} \sqrt{1 + \sqrt{1 + 6.76\left(\frac{F_{yr}}{E}\frac{S_{xc} h}{J}\right)^2}}$$

$$J = \frac{D t_w^3}{3} + \frac{b_{fc} t_{fc}^3}{3}\left(1 - 0.63\frac{t_{fc}}{b_{fc}}\right) + \frac{b_{ft} t_{ft}^3}{3}\left(1 - 0.63\frac{t_{ft}}{b_{ft}}\right)$$

$$= \frac{102(0.75)^3}{3} + 2\frac{(18)(1)^3}{3}\left(1 - 0.63\frac{1}{18}\right) = 25.9 \text{ in}^3$$

$$F_{yr} = \text{Smaller of } \left(0.7F_{yc},\ \frac{R_h F_{yr} S_{xt}}{S_{xc}},\ F_{yw}\right) > 0.5F_{yc}$$

$$0.7F_{yc} = 0.7(50) = 35$$

$$\frac{R_h F_{yr} S_{xt}}{S_{xc}} = 1 \times (50) \times 1 = 50$$

$$F_{yw} = 50$$

$\Rightarrow 35^{ksi} > 0.5F_{yc} = 25^{ksi}$

Use $F_{yr} = 35^{ksi}$

$$L_r = 1.95 \times (3.98)\frac{29{,}000}{35}\sqrt{\frac{25.9}{(3112)(103)}} \times \sqrt{1 + \sqrt{1 + 6.76\left(\frac{35}{29{,}000}\frac{(3112)(103)}{25.9}\right)^2}} = 364.9 \text{ in}$$

$0.0899$

$= 6.312$

$L_b$ = Max 23.51' (287 in), Min. 11.64' (140 in)

$$L_p < L_b < L_r$$

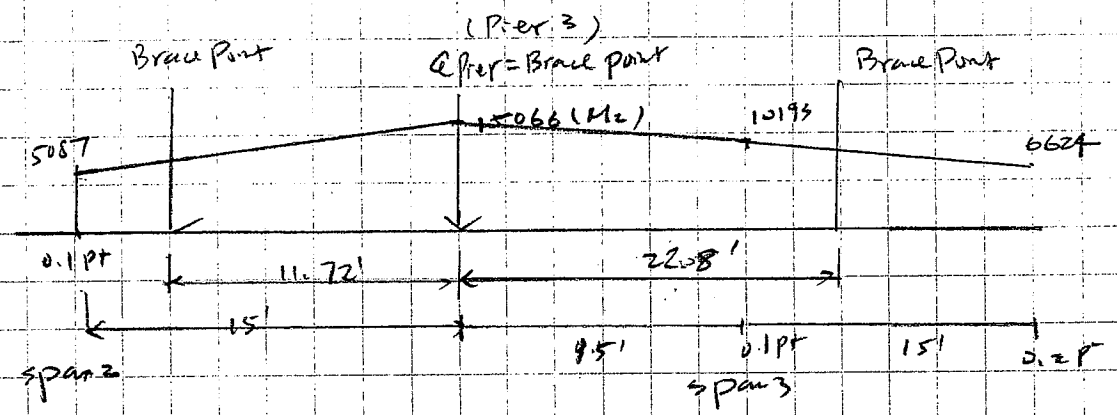
FHW LA River Bridge 46

5/6/09 Strength limit state, Negative Moment

$L_p < L_b < L_r$

$$M_{nc(LTB)} = C_b \left[1 - \left(1 - \frac{F_{br} S_{xc}}{R_{pc} M_{yc}}\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] R_{pc} M_{yc} \leq R_{pc} M_{yc}$$

$$C_b = 1.75 - 1.05\left(\frac{M_1}{M_2}\right) + 0.3\left(\frac{M_1}{M_2}\right)^2 \leq 2.3$$




for span 2 side

$$M_1 = 5087 + \frac{(15 - 11.72)}{15} \times (15066 - 5087) = 7269 \text{ k-ft}$$

for span 3 side

$$M_1 = 6624 + \frac{(15 - 7.8)}{15} \times (10193 - 6624) = 8337 \text{ k-ft}$$

$$C_b = 1.75 - 1.05\left(\frac{7269}{15066}\right) + 0.3\left(\frac{7269}{15066}\right)^2 = 1.31$$

$$\text{or} = 1.75 - 1.05\left(\frac{8337}{15066}\right) + 0.3\left(\frac{8337}{15066}\right)^2 = 1.26 \leftarrow \text{use}$$

$$M_{nc(LTB)} = 1.26\left[1 - \left(1 - \frac{(35)(-3112)}{(1.004) \times (2967) \times (12)}\right)^{0.697}\right]\left(\frac{140 - 95.85}{364.9 - 95.85}\right)^{0.165} R_{pc} M_{yc}$$

$$= 1.197 > (1) \times R_{pc} M_{yc}$$

$$M_{nc(LTB)} = R_{pc} M_{yc}$$

$\therefore$ Flexural Resistance $M_{nc} = R_{pc} M_{yc} = 13020$ k-ft.

$M_u = 15066^{k\text{-}ft} > \phi_f M_{nc} = 13020^{k\text{-}ft}$ N.G

15% over

Make Top flange and Bottom flange thicker

use $t_f = 1\frac{1}{2}''$ for Both Top & Bottom

recalculate $M_{nc}$

$$\lambda_f = \frac{b_{fc}}{2t_{fc}} = \frac{18''}{2\times 1.5} = 6 < \lambda_{pf}$$

$$\therefore M_{nc} = R_{pc} M_{yc}$$

check web compaction.

$$M_p = \left[(18\times 1.5)\times 50 \times 51.75 + (51\times 0.75)\times 50 \times \frac{51}{2}\right]\times 2$$

$$= 237263^{k\text{-}in} = 19772^{k\text{-}ft}$$

$$M_y = S\times F_{yf} = 4018 \times 50$$

$$= 200900^{k\text{-}in} = 16742^{k\text{-}ft}$$

$$\lambda_{pw(D_{cp})} = \frac{\sqrt{\frac{29{,}000}{50}}}{\left(0.54\frac{19772}{(16742)(1.2)} - 0.09\right)^2} = \frac{24.08}{0.3} = 80.26$$

$$\lambda_w = \frac{2D_{cp}}{t_w} = 73.6 > \lambda_{pw(D_{cp})}$$

web is non-compact.

web plastic-friction factor

$$R_{pc} = \left[1 - \left(1 - \frac{(1)(16742)}{(19772)}\right)\left(\frac{136 - 80.26}{137.3 - 80.26}\right)\right] \frac{19772}{16742}$$

$$= (1 - 0.153 \times 0.977) \times 1.181$$

$$= 1.004 < 1.181 \quad \text{O.K}$$

$$M_{Nc} = R_{pc} M_{yc} = 1.004 \times 16742^{k\text{-}ft}$$

$$= 16811^{k\text{-}ft} > \phi M_u = 15066^{k\text{-}ft}$$

11% over OK.

AASHTO 6.10.4.2.2

Service II loads apply. For Both steel flanges of non-composite sections.

$$f_f + \frac{f_l}{2} \leq 0.80 R_h f_{yf}$$

Also.

$$f_c \leq F_{crw}$$

check loads at span 1, 1.0 PT, Factored load at Service II.

$M_{DC1} = -3254^{k\text{-}ft}$

$M_{DC2} + M_{DW} = -504 - 696 = -1200^{k\text{-}ft}$

$M_{(LL+IM)HL93} = -5109^{k\text{-}ft}$

$$F_{crw} = \frac{0.9EK}{\left(\frac{D}{t_w}\right)^2} \leq \text{smaller of} \begin{pmatrix} R_h F_{yc} \\ F_{yw}/0.7 \end{pmatrix}$$

$$K = \frac{9}{\left(\frac{D_c}{D}\right)^2} = \frac{9}{\left(\frac{51}{102}\right)^2} = 36$$

$$F_{crw} = \frac{0.9(29{,}000)\times 36}{\left(\frac{102}{0.75}\right)^2} = 50.8^{ksi}$$

$$\left.\begin{aligned} R_h F_{yc} &= (1)(50) = 50 \\ \frac{F_{yw}}{0.7} &= \frac{50}{0.7} = 71.4 \end{aligned}\right\} \Rightarrow \text{use } 50^{ksi}$$

So $F_{crw} = 50^{ksi}$

$$f_f = \frac{M_{DC1} + M_{DC2} + M_{DW} + M_{LL(Fatigue)}}{S_{NC}} \leq 0.80 R_h F_{yf} = 0.8 \times 1.0 \times (50) = 40^{ksi}$$

$$= \frac{|-3254 - 1200 - 5109| \times (12)}{4018} = 27.9^{ksi} < 40^{ksi} \quad OK$$

$$f_c = f_f = 27.9^{ksi} < F_{crw} = 50^{ksi} \quad O.K.$$

AASHTO c. 6.10.2.1.1

$$\frac{D}{t_w} = \frac{102}{0.75} = 136 < 150$$

∴ maximum transverse stiffner spacing up to

$$3D = 3 \times 102 = 306^{in} = 25.5'$$

$$\frac{2 D t_w}{(b_{fc} t_{fc} + b_{ft} t_{ft})} = \frac{2 \times (102) \times 0.75}{18 + 18} = 4.25$$

or

$$= \frac{2(102)(0.75)}{(18 \times 1.5 + 18 \times 1.5)} = 2.83$$

$> 2.5$

$$\therefore V_n = V_p \left[ C + \frac{0.87(1-C)}{\left(\sqrt{1+\left(\frac{d_o}{D}\right)^2} + \frac{d_o}{D}\right)} \right]$$ ← Intermediate stiffner.

$$V_p = 0.58 F_{yw} D t_w$$

$$= 0.58 \times (50)(102)(0.75) = 2218^{kips}$$

$d_o \rightarrow$ transverse stiffner spacing,

use $d_o = 10'$ to design $= 120^{in}$

$$K = 5 + \frac{5}{\left(\frac{d_o}{D}\right)^2} = 5 + \frac{5}{\left(\frac{120}{102}\right)^2} = 8.61$$

$$1.12\sqrt{\frac{EK}{F_{yw}}} = 1.12\sqrt{\frac{(29000)(8.61)}{50}} = 79.15$$

$$1.4\sqrt{\frac{EK}{F_{yw}}} = 98.94$$

$$\frac{D}{t_w} = \frac{102}{0.75} = 136 > 1.4\sqrt{\frac{EK}{F_{yw}}}$$

$$C = \frac{1.57}{\left(\frac{D}{t_w}\right)^2}\left[\frac{Ek}{F_{yw}}\right]$$

$$= \frac{1.57}{\left(\frac{102}{0.75}\right)^2}\left[\frac{29{,}000(8.61)}{50}\right] = 0.4238$$

$$V_n = 22.18\left[0.424 + \frac{\overset{0.184}{0.87(1-0.424)}}{\sqrt{1+\left(\frac{120}{102}\right)^2}+\left(\frac{120}{102}\right)}\right]$$

$$= 22.18 \times 0.608 = 1350^{\text{kips}}$$ > strength II shear $V_u = 719^{\text{kips}}$

Recalculate $V_n$ for $d_o = 6.5' = 78^{\text{in}}$

$$K = 5 + \frac{5}{\left(\frac{78}{102}\right)^2} = 13.55$$

$$C = \frac{1.57}{136^2}\left[\frac{29{,}000 \times 13.56}{50}\right] = 0.667$$

$$V_n = 22.18\left[0.667 + \frac{\overset{0.143}{0.87(1-0.667)}}{\sqrt{1+\left(\frac{78}{102}\right)^2}+\left(\frac{78}{102}\right)}\right]$$

$$= 22.18 \times 0.81 = 1797^{\text{kips}}$$

Transverse stiffner @ 10' is adequate ✓

Stiffner Requirements AASHO 6.10.11.1.2

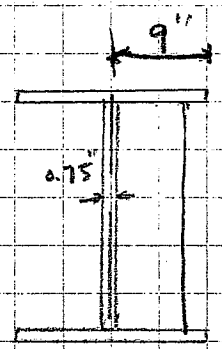
Projecting width

$$b_t \geq 2.0 + \frac{D}{30} = 2.0 + \frac{102}{30} = \underline{5.4}\ in$$

and

$$16 t_p > b_t \geq b_f/4$$

$$16\left(\tfrac{3}{8}\right) = 6 > b_t \geq 18/4 = 4.5\ in$$




Intermediate stiffner use

Plate $5\frac{1}{2}" \times \frac{3}{8}"$ #

Bearing stiffner use

Plate $8" \times \frac{3}{4}"$ #

$4 t_w = 4 \times 0.75 = 3\ in$
$6 t_w = 6 \times 0.75 = 4.5\ in$

(to match with existing structure)

Cross Frame stiffner (as connection plate) will be detailed at cross Frame design

CA 6.10.11.2.1

Each stiffner shall be either finish (mill or grind) to bear plus a fillet weld against the flange through which it receives its load or attached to that flange by a full penetration groove weld.

Moment of Inertia of Transverse stiffners should satisfy

$$I_t \geq \text{smaller of} \begin{cases} b t_w^3 J \\ \dfrac{D^4 \rho_t^{1.3}}{40} \left( \dfrac{F_{yw}}{E} \right)^{1.5} \end{cases}$$

$$J = 2.5 \left( \frac{D}{d_o} \right)^3 - 2.0 > 0.5$$

$$= 2.5 \left( \frac{102}{120} \right)^3 - 2.0 = -0.464 < 0.5$$

$$\therefore J = 0.5$$

$$b = \text{smaller of } (D, d_o) = 102^{in}$$

$$\rho_t = \text{larger of } \left( \frac{F_{yw}}{F_{crs}}, 1 \right)$$

$$F_{crs} = \frac{0.31 E}{\left( \frac{b_t}{t_p} \right)^2} \leq F_{ys} = 36^{ksi} \quad \left( \begin{matrix} \text{stiffner} \\ M270, A36 \end{matrix} \right)$$

$$F_{crs} = \frac{0.31 (29{,}000)}{\left( \frac{5.5}{0.375} \right)^2} = 41.79 \Rightarrow F_{crs} = 36^{ksi}$$

$$\frac{F_{yw}}{F_{crs}} = \frac{50}{36} = 1.39 \Rightarrow \rho_t = 1.39$$

$$b t_w^3 J = 102 (0.75)^3 \times 0.5 = 21.5^{in^4}$$

$$\frac{D^4 \rho_t^{1.3}}{40} \left( \frac{F_{yw}}{E} \right)^{1.5} = \frac{102^4 \cdot (1.39)^{1.3}}{40} \left( \frac{50}{29000} \right)^{1.5}$$

$$= 297.3^{in^4}$$

$$\therefore I_t > 21.5^{in^4}$$

LA River Bridge

Transverse Stiffner



$$I_t = 2\left[\frac{(5.5)^3(0.375)}{12} + (5.5)(0.375)\left[\frac{5.5}{2} + \frac{0.75}{2}\right]^2\right]$$

$$= 2(5.2 + 20.15) = 50.68 \text{ in}^4 > 21.5 \text{ in}^4$$

OK.

AASHTO 6.10.11.2

Design Bearing stiffener @ Bent 2.

Projecting width

$$b_f \le 0.48\, t_p \sqrt{\frac{E}{F_{ys}}}$$

$$0.48\, t_p \sqrt{\frac{E}{F_{ys}}} = 0.48 \times (0.75)\sqrt{\frac{29000}{36}} = 10.21^{in}$$

use 8" ok

check Bearing Resistance

$$(R_{sb})_r = \phi_b (R_{sb})_n \qquad \phi_b = 1.0$$

$(R_{sb})_n = 1.4\, A_{pn}\, F_{ys}$, Assume 1" cope

$$A_{pn} = 2(8-1)(0.75) = 10.5\ in^2$$

$(R_{sb})_r = 1.4 \times 10.5 \times 36 = 529^{kips}$ > Abut 1 >
Abut 5 ok

at Pier use 3 pairs, 8" x 3/4" plate

$$3 \times 529 = 1587^{kips} > 1124^{kip} \text{ ok.}$$

check Axial Resistance

$$P_r = \phi_c P_n$$

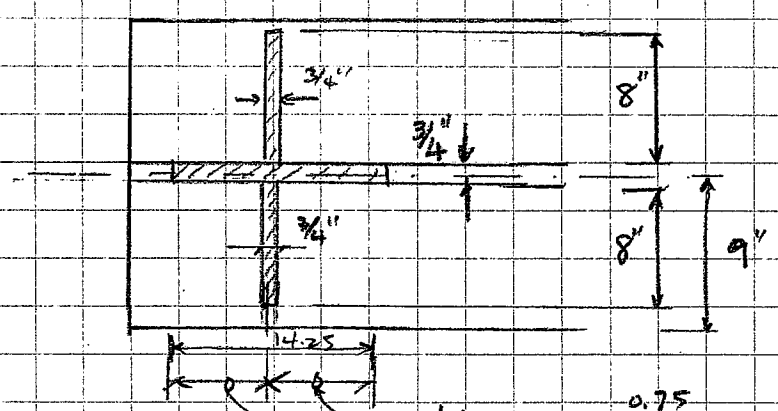
$$P_n = 0.66^{\lambda} F_y A_s \quad \text{for } \lambda \le 2.25$$

$$= \frac{0.88\, F_y A_s}{\lambda} \quad \text{for } \lambda > 2.25$$

$$\lambda = \left(\frac{kl}{r_s \pi}\right)^2 \frac{F_y}{E}$$

Effective column section includes 2 stiffner plate and a centrally located strip of web not more than 9 tw on each side of stiffners. If more than one pair of stiffners is used, the effective column section shall consist of all stiffner elements, plus a centrally located strip of web extending not more than 9 tw on each side of the outer projecting elements of the group

For Abutment




$t/2 + 9tw = \frac{0.75}{2} + 9(0.75) = 7.125''$

Stiffner Area $= 2 \times 8 \times 0.75 = 12 \text{ in}^2$

web Area $= 2 \times (7.125) \times 0.75 = 10.6875 \text{ in}^2$

$A_g = A_{TOT} = 12 + 10.6875 = 22.68 \text{ in}^2$

The radius of gyration shall be computed about the mid-thickness of the web, and the effective length shall be taken as 0.75D, where D is the web depth

$I_{xc} = \frac{1}{12}(14.125) \times (0.75)^3 + \frac{1}{12}(0.75)(16.625)^3$

$= 0.496 + 287.19 = 287.68 \text{ in}^4$

$$r_s = \sqrt{\frac{I_{x-x}}{A_{TOT}}} = \sqrt{\frac{287.68}{22.68}} = 3.56^{in}$$

use K = 0.75 for the weld end connection.

unbraced length for the bearing stiffner

$$l = D = 102^{in}$$

$$\frac{Kl}{r_s} = \frac{0.75 \times 102}{3.56} = 21.48 < 120 \text{ OK}$$

$$\lambda = \left(\frac{Kl}{r_s \pi}\right)^2 \frac{F_y}{E}$$

$$= \left(\frac{0.75\,(102)}{3.56\,\pi}\right)^2 \times \frac{36}{29000}$$

$$= 0.058 < 2.25$$

$$\therefore P_n = 0.66^{\lambda} F_y A_s$$

$$= 0.66^{0.058}\,(36)\,(22.68) = 797^{kips}$$

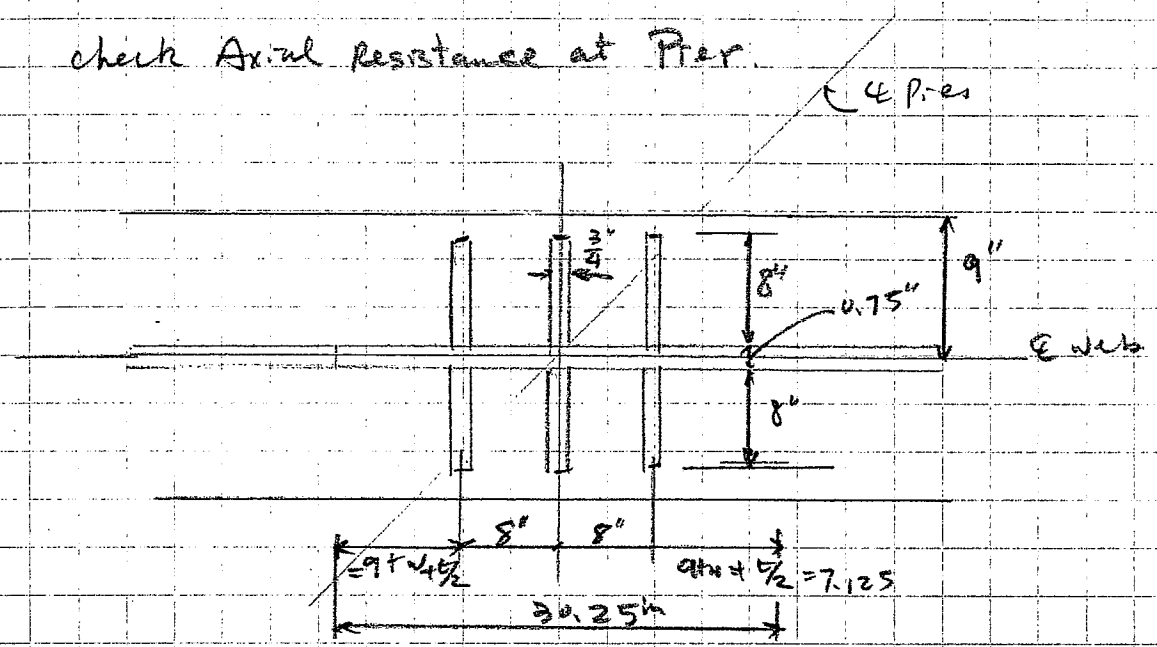
$$P_r = \phi_c P_n = 0.9\,(797) = 717^{kip} > 496.25$$

@ Abutment OK

Abutment Bearing stiffner

use 2 8" x 3/4" PL

check Axial Resistance at Pier




stiffner Area = $6 \times 8 \times 0.75 = 36 \text{ in}^2$

web Area = $30.25 \times 0.75 = 22.6875 \text{ in}^2$

$A_s = 58.68 \text{ in}^2$

$I_{xx} = \frac{1}{12} \times (30.25)(0.75)^3 + \frac{1}{12}(0.75)(16.625)^3 \times 3$

$= 1.06 + 861.57 = 862.63$

$r_s = \sqrt{\frac{I_{xx}}{A_s}} = \sqrt{\frac{862.63}{58.68}} = 3.83 \text{ in}$

$\frac{Kl}{r} = \frac{0.75 \times 102}{3.83} = 19.97 < 120$ ok

$\lambda = \left(\frac{Kl}{r_s \pi}\right)^2 \frac{F_y}{E} = \left(\frac{0.75 \times (102)}{3.83 \times \pi}\right)^2 \times \frac{36}{29{,}000}$

$= 0.05 \times 2.25$

$\therefore P_n = 0.66^{0.55}(36)(58.68) = 2069 \text{ kips}$

$P_r = 0.9 P_n = 1862 \text{ kips} > 1120 \text{ kips}$ o.k

use 3-pairs 8" x 3/4" plate

Bearing stiffner to web welds.

shear resistance of Fillet weld.

$$R_r = 0.6\,\phi_{e2}\,F_{exx}$$

using E70XX weld metal. $F_{exx} = 70^{ksi}$

$$\phi_{e2} = 0.8$$

$$R_r = (0.6)(0.8)(70) = 33.6^{ksi}$$

Total length of welds.

$$L = 102' - 6'' = 96^{in}$$

Total shear resistance of welds connecting the bearing stiffner to the web is

Assuming $t_w = 1/4''$

throat dist. $= 0.707 \times 0.25 = 0.17675^{in}$

$$V_r = 4 \times (\text{throat dist.}) \times L \times R_r$$

$$= 4 \times 0.17675 \times 96^{in} \times 33.6$$

$$= 2280^{ksi} > P_n \text{ at Pier } 1120^{kips}$$

Use Pier 4.

Top of Deck = 128.64'

BOF = 76.589 + 4.5 (Footing Depth) + 2 (cover) = 83.089

Height = 128.64 − 83.089 = 45'

$$V_{DZ} = 2.5\, V_o \left(\frac{V_{30}}{V_B}\right) \ln\left(\frac{Z}{Z_o}\right)$$

$Z = 45'$, $Z_o = 3.28$ (suburban)

$V_{30} = V_B = 100$ mph

$V_o = 10.9$ mph

$$V_{DZ} = 2.5\,(10.9)\left(\frac{100}{100}\right) \ln\left(\frac{45}{3.28}\right)^{=2.62}$$

$= 71.4$ mph

$$P_D = P_B \frac{V_{DZ}^2}{10{,}000} = (0.05)\left(\frac{71.4^2}{10{,}000}\right) = 0.025 \text{ ksf}$$

[The total depth of the structure w/ Barrier 736

10' + 3' = 13' (use 10' girder for conservative purpose)]

Wind load acting on girder span:

$$WS_{girder} = P_D(13) = 0.025 \times 13 = 0.33 \text{ kip/ft} > 0.3 \text{ k/ft}$$

Wind load acting on the bottom flange

$$WS_{bf} = \frac{W_{S\,girder}\,(d/2)}{13'} = 0.33 \times \frac{(108''+4'')/12 \times 1/2}{13'}$$

$= 0.118$ kips/ft

Wind load acting on top of girder

$$WS_{tf} = 0.33 - 0.118 = 0.212 \text{ k/ft}$$

Cross Frame spacing $L_b = 23.5^{ft}$ wind induced moment applied on the bottom of the exterior girder is estimated.

$$M_{WS} = \frac{W S_{bf} L_b^2}{10} = \frac{0.118 \times (23.5)^2}{10} = 6.52^{k-ft}$$

for bottom flange, wind induced lateral stresses is

$$f_\ell = \frac{M_{WS}}{\frac{t_f b_f^2}{6}}$$

$$= \frac{6.52 \times 12}{\frac{1}{6} \times (18)^2 \times 1} = 1.45^{ksi}$$

Strength V load case controls

$$1.25\,DC + 1.5\,DW + 1.35(LL + IM)_{HL-93} + 0.4\,WS$$

span 3 @ 0.5 pt from maximum positive moment.

$1.25 M_{DC1} = 2924^{k-ft}$

$1.25 M_{DC2} = 420^{k-ft}$

$1.5 M_{DW} = 756^{k-ft}$

$1.35 (LL+IM)_{HL-93} = 1.35 \times 4906 = 6623^{k-ft}$.

$M_u = 2924 + 420 + 756 + 6623 = 10725^{k-ft}$.

$$M_u + \frac{1}{3} f_\ell S_{xt} \leq \phi_f M_n$$

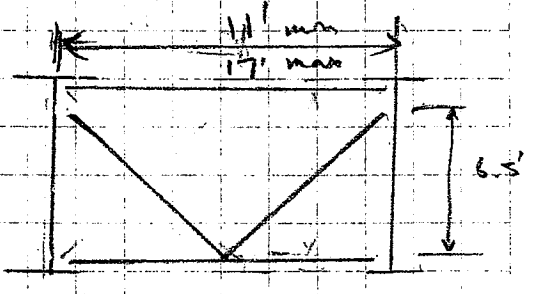
$$S_{xt} = \frac{M_{yt}}{F_{yt}} = \frac{17174^{k-ft} \text{ (from earlier calcs)} \times 12}{50} = 4122\ in^3$$

$$M_u + \frac{1}{3} f_\ell S_{xt} = 10725 + \frac{1}{3}(1.45)\frac{4122}{12}$$

$$= 10891^{k-ft} < \phi M_N = (1)(22291)^{k-ft}$$

Calculate Forces acting on the cross Frame. VOID Details

Wind load forces in the top strut is assumed to be zero. The diagonals will transfer the wind load directly into deck slab.




The horizontal wind forces applied to the brace points are assumed to be carried fully by the bottom strut in the exterior bays.

For strength III, the factor wind load forces acting on the bottom strut is

$$P_u = 1.4 W S_{bf} L_b = 1.4 \times (0.118) \times 23.5' = 3.88 \text{ kips}$$

Factored force acting on the diagonal

$$P_u = \frac{3.88}{\cos\phi} = 4.5 \text{ kips}$$

Bottom Strut, choose Angle L 6 x 6 x 1/2"

Check limiting effective slenderness ratio.

AASHTO 4.6.2.5 $K = 1.0$ (for angle, regardless of end cond.)

$L_z = 5.5' = 66''$ $r_z = 1.18$ in

$$\frac{KL_z}{r_z} = \frac{(1)(66)}{1.18} = 56 < 140 \quad \text{OK}$$

$L_y = L_x = 17' = 204$ in (For out-of-plane buckling) (11' min to 17' max)

$$\frac{KL_y}{r_y} = \frac{(1)(204)}{1.86} = 109.7 < 140 \quad \text{OK}$$

Design bottom strut in accordance with AISC E5.

$$\frac{L}{r_x} = \frac{11 \times 12}{1.86} = 70.96 < 80 \quad \text{or (upto 12')}$$

$$\frac{L}{r_x} = \frac{17 \times 12}{1.86} = 109.7 > 80$$

For $\frac{L}{r_x} < 80$ sections

$$\frac{Kl}{r} = 72 + 0.75\frac{L}{r_x} = 72 + 0.75\frac{144}{1.86} = 130$$

From AISC Table B4.1

$$\lambda = \frac{b}{t} = \frac{6}{0.5} = 12$$

$$\lambda_r = 0.45\sqrt{\frac{E}{F_y}} = 0.45\sqrt{\frac{29{,}000}{36}} = 12.8$$

Section is noncompact.

For a non-compact section in accordance with AISC E3., the nominal compressive strength is calculated:

$$P_n = F_{cr} A_g$$

$$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29000}{36}} = 133.7 > \frac{Kl}{r} = 130$$

$$\therefore F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y$$

$$F_e = \frac{\pi^2 E}{\left(\frac{Kl}{r}\right)^2} = \frac{\pi^2 \times 29000}{(130)^2} = 16.94 \text{ ksi}$$

$$F_{cr} = \left[0.658^{\left(\frac{36}{16.94}\right)}\right](36) = 14.8 \text{ ksi}$$

$$P_n = 14.8 \times 5.77 = 85.34 \text{ kips}$$

$$P_u = 288 \text{ kips} < \phi_c P_n = 0.9 \times 85.34 = 76.81 \quad \text{OK}$$

For sections, $\frac{L}{r_x} > 80$

$$\frac{KL}{r} = 32 + 1.25\frac{L}{r_x} = 32 + 109.7 = 141.7 < 200$$

$$\frac{KL}{r} > 4.71\sqrt{\frac{E}{F_y}} = 133.7$$

$$F_{cr} = 0.877\,F_e$$

$$F_e = \frac{\pi^2 \times 29000}{(141.7)^2} = 14.25^{ksi}$$

$$F_{cr} = 0.877 \times (14.25) = 12.5^{ksi}$$

$$P_n = 12.5 \times 5.77 = 72.13^{kips}$$

$$P_u = 288^{kips} < \phi_c P_n = 0.9 \times 72.13 = 64.9^{ksi} \text{ OK}$$

Estimate seismic forces acting on the Frame, each cross Frame have 25' Distributed Length of super structure

DL = 3.0 K/ft X 25 x ½ (Acting to the bottom) x ½ (Distribution factor)

= 18.75 kips

ARS., 0.6 g. at Peak (Assume) 1.58

EQ Force = 1.58 x 18.75 = 30 kips < $\phi_c P_n$ = 64.9 ok

Choose Diagonal L 6x6x½

Diagonal tension force $\frac{30}{\cos\phi}$ = 39 kips
0.763

* Note Seismic force Acting on the frame is preliminary, which needs to be verified w/ structure Analysis.

Bottom strut Connection Design. Design connection forces.

$P_u = 39^{kips}$

Determine nominal shear resistance per bolt.

$R_n = 0.48 A_b F_{ub} N_s$

$A_b = \left(\frac{0.75}{2}\right)^2 \pi = 0.442 \text{ in}^2$

$F_{ub} = 120^{ksi}$ ; $N_s = 1$ (single shear)

$R_n = 0.48 A_b F_{ub} N_s = 0.48(0.442)(120)(1) = 25.5^{kips}$

Calculate design bearing strength for each bolt on stiffner.

Since the clear end distance is less than 2d,

The nominal bearing resistance

$R_n = 1.2 L_c t F_u$

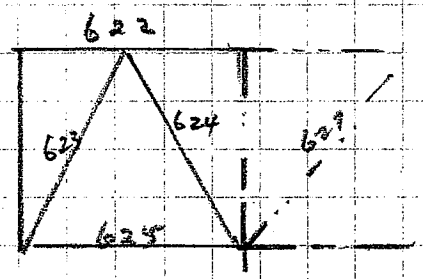
Existing stiffner plate 7 x 4 x 7/16 (Angle)

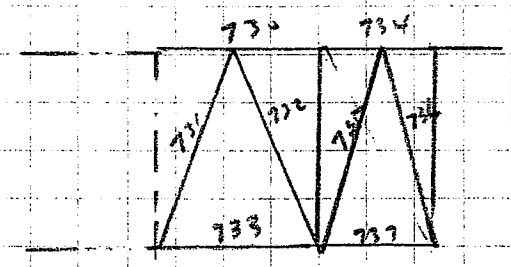
New stiffner plate 7 x 4 x 3/8

Stiffner material is A36 $F_u = 58^{ksi}$

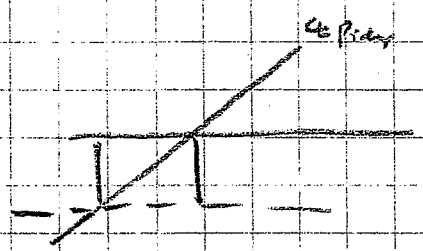
Girder Bracing forces at ℄ Pier, under seismic Forces
( Force From SAP 2000 )

Pier 2




| | Number | Axial Load | | Number | Axial Load |
|---|---|---|---|---|---|
| | 622 | 414 | | 1606 | 351 |
| | 623 | 693 | | 1607 | 584 |
| Pier 2 | 624 | 674 | Pier 3 | 1608 | 568 |
| | 625 | 103 | | 1609 | 88 |
| | 730 | 392 | | 1642 | 335 |
| | 731 | 664 | | 1643 | 558 |
| | 732 | 682 | | 1644 | 593 |
| | 733 | 100 | | 1645 | 84 |
| | 734 | 285 | | 1646 | 243 |
| | 735 | 630 | | 1647 | 531 |
| | 736 | 630 | | 1648 | 531 |
| | 737 | 3 | | 1649 | 3 |




we place 2 cross Frames at ℄ Pier

Each diagonal will take 350 kips max.

use Double L 8 x 6 x 1"

$A_s = 26 \text{ in}^2$ $L = \sqrt{5.5^2 + 7.83^2} = 9.6' = 115 \text{ in}$

$r_{min} = 2.49$

$\frac{KL}{r_{min}} = \frac{1 \times 115}{2.49} = 46.1 < 140$ O.K.

AISC E4.

$P_n = F_{cr} A_g$

$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29{,}000}{36}} = 133.7$

$\frac{KL}{r} < 4.71\sqrt{\frac{E}{F_y}}$

$\therefore F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y$

$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r_y}\right)^2} = \frac{\pi^2 \times 29{,}000}{\left(\frac{1 \times 115}{2.95}\right)^2} = 135$

46.1

$F_{cr} = \left[0.658^{\left(\frac{36}{135}\right)}\right](36) = 32.2 \text{ ksi}$

0.894

$P_n = 32.2 \text{ ksi} \times 26 \text{ in}^2 = 837 \text{ kips}$

$\phi_c P_n = 0.9 \times P_n = 753 \text{ kips}$

Diagonal # of Bolts required, 7/8", A325 Bolt

$R_n = 0.48 A_b F_{ub} N_s$

$= 0.48 \times \left(\frac{\frac{7}{8}^2 \times \pi}{4}\right) \times 120 = 34.6^{kip}/_{Bolt}$

Need minimum 10 bolts for 350 kip Axial load

Bottom strut. Axial Load = 100 kips

Need minimum 4 bolts.

Top strut. Axial load = 414 kips

Need minimum 12 Bolts

use A325, threaded HS Rod, 3/4"

$R_n = 0.38\left(\frac{0.75^2 \times \pi}{4}\right) \times 120 = 25.2^{kip}$

$414/25.2 = 16.4 \leftarrow$ Need 18 bolts

For South Bound Traffic, slope range from +2% to -1%

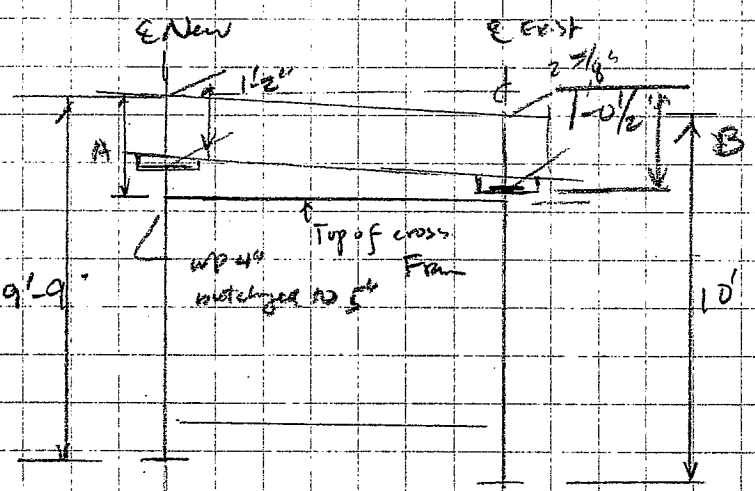
(1) Frame Type A

11' Girder spacing.

The cross slope difference

$11' \times 0.02 = +2\frac{5}{8}''$

$11' \times 0.01 = -1\frac{3}{8}''$

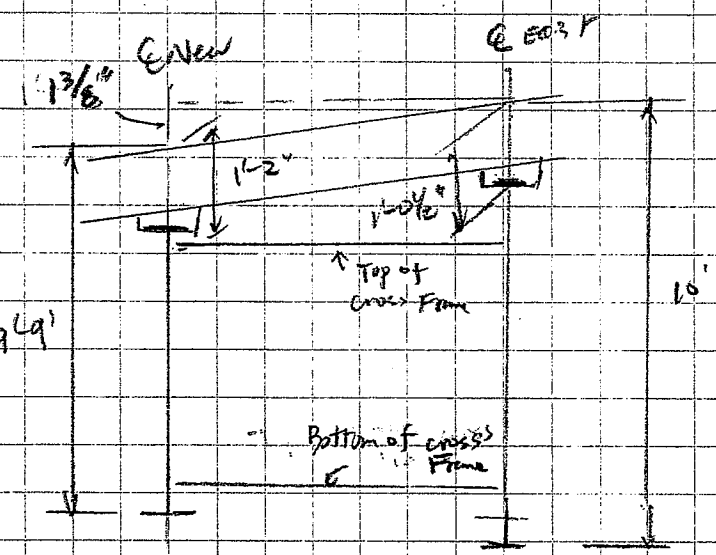



for 2% cross slope

$A = (1'-2'') + 4'' - (1\frac{1}{2}'') = 1'-4\frac{1}{2}''$

$B = (1'-0\frac{1}{2}'') + 2\frac{5}{8}'' = (1'-3\frac{1}{8}'')$

A > B OK




for 1%, No Problem

South Bound Traffic

(a) Frame Type B.

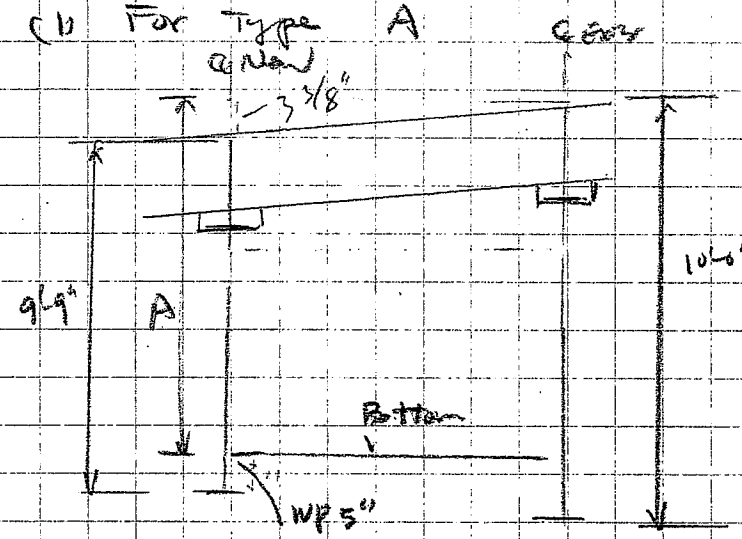
for Bottom of Bottom cross Frame, 2% cross slope

$11'0" + (2\frac{5}{8}") - (14 - 4") = 9'-4\frac{5}{8}"$

Comparing with 9'-9" should be ok.

For North Bound Traffic, slope range from -2.5% to +1.5%

(b) For Type A




$11' \times 0.025 = 3.3" = 3\frac{3}{8}"$

$11' \times 0.095 = 1\frac{5}{8}"$

Top of cross Frame, OK

For Bottom of Bottom cross Frame

$A = (9'-9") - (1") - (1") + 3\frac{3}{8}" = 9'-10\frac{3}{8}"$ < 10'-0"

spacing

Bottom Flange thickness

Compared to 10'-0" subtracts angle thickness $\frac{3}{4}"$

$9'-11\frac{1}{4}"$

move wp point to 5" ←

For +1%, 5" at top is ok, see Page 1

FKW

8/10/10

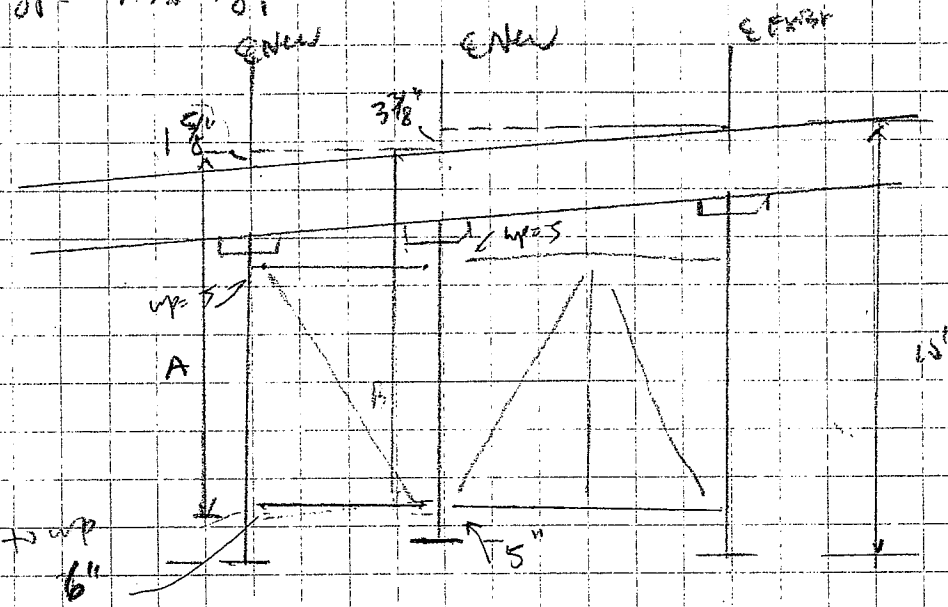
LA River Bridge

Cross Frame Detailing



For North Bound Traffic

(2) Type A, & Type C.

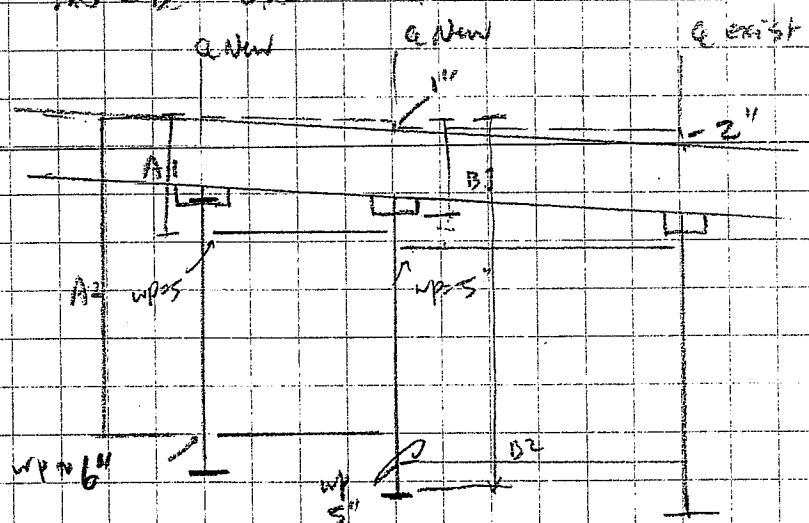



$5.5 \times 0.1375 = 0.1375 = 1\frac{5}{8}''$

$A = (9'-9') + (1\frac{5}{8}'') - 1'' = -6'' + 3'' = 9'-6\frac{5}{8}''$

compare to $(9'-9') - (1'') = 9'-8''$ is

$A < B$ OK




$5.5 \times 0.015 = 1''$

$11 \times 0.015 = 2''$

$A1 = (1'-2'') + 5'' - (1\frac{1}{2}'') = 1'-5\frac{1}{2}''$ A1 > B1 OK

$B1 = (1'-2'') + 1 = 1'-3''$

$A2 = (9'-9'') - 1'' - 7'' + 3'' = 9'-4''$

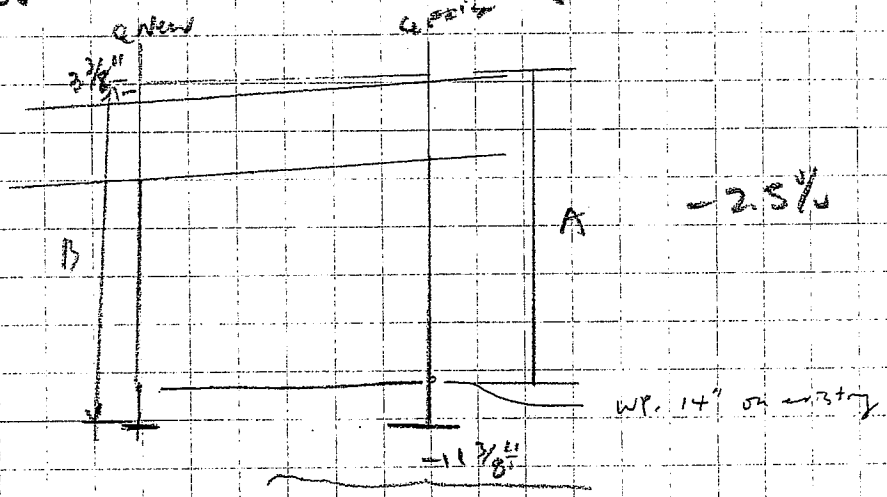
$B2 = (9'-9'') + 1'' - 1'' = 9'-9''$ B2 > A2 OK

FHW
8/10/10.

LA River Bridge
Cross Frame Detailing



(3) Type B, -2.5% cross slope

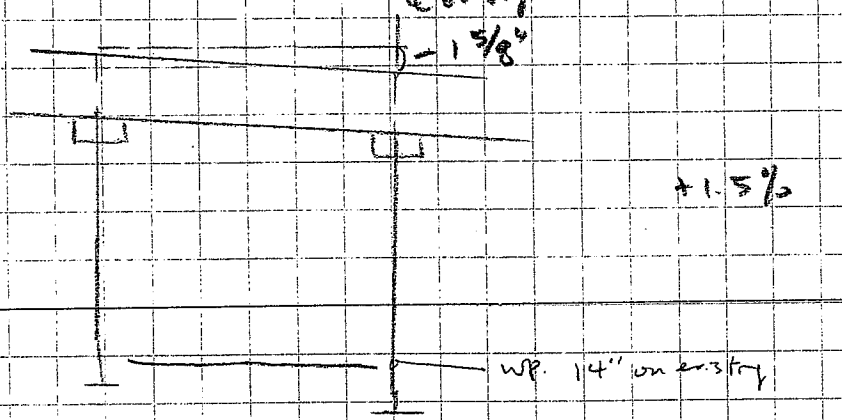



$A = (10'-0") - (5/8") - (3/4") - 14" + 4" = 9'-0\,5/8"$

cover plate angle

B > A OK

$B = (9'-9") + (3\,3/8") - 1" = 9'-11\,3/8"$




$A = (10'-0") - (5/8") - (3/4") - 14" + 4" + 1\,5/8" = 9'-2\,1/4"$

$B = (9'-9") - 1" = 9'-8"$

B > A OK

FHW
8/10/00

LA River Bridge
Cross Frame Detailing



(4) Type B1
look at Page 3 Type C cross Frame,
make up @ 6" for the exterior New Girder.

Try 1" outer and inner splice plate for bottom flange

3/4" outer and inner splice plate for Top flange

1/2" web splice plate.

use A325 d= 7/8" Bolt

(CA 6.13.1)

Factored resistance of splices shall not be less than 100% of the factored flexural, shear or axial resistance of the smaller member section at the strength limit state.

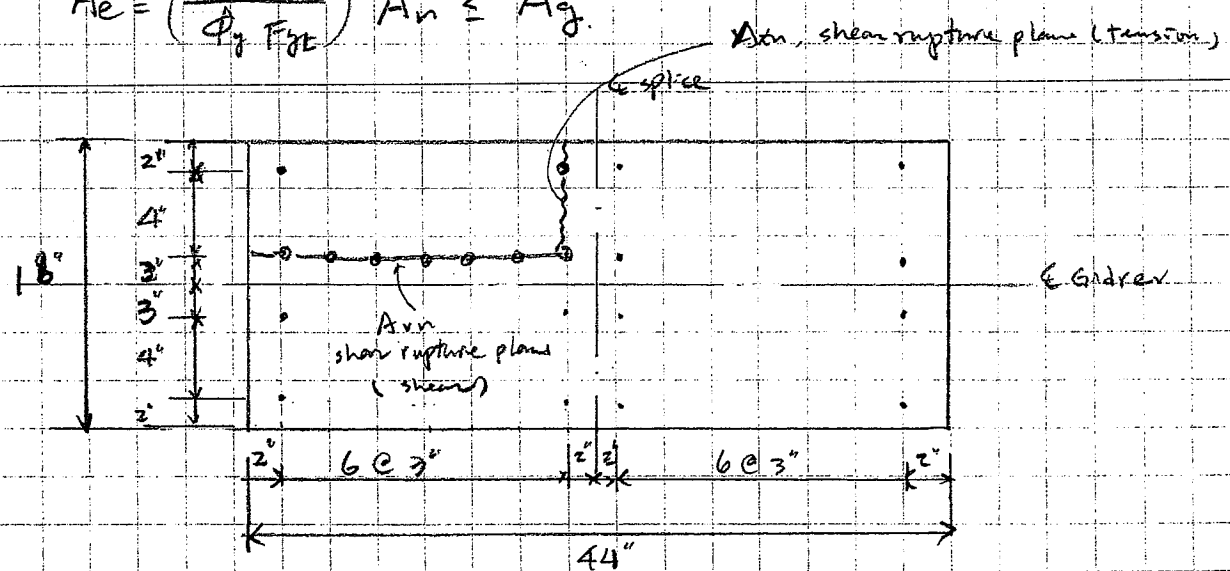
Design Bottom flange splice

$F_{cf} = \alpha \phi_f F_{yf}$ (CA 6.13.6.1.4c-1)

splice plate resistance is defined as

$P_{cu} = A_e F_{cf}$, $A_e$ is the smaller effective area on either side of the splices.

$$A_e = \left(\frac{\phi_u F_u}{\phi_y F_{yt}}\right) A_n \leq A_g$$




$A_n = [18 - 4(7/8" + 1/8")] \times 1"$ (thickness) $= 14 \text{ in}^2$

$A_g = 18" \times 1" = 18 \text{ in}^2$

$$\therefore A_e = \left(\frac{\phi_u F_u}{\phi_y F_{yt}}\right) A_n$$

$$= \left(\frac{0.8 \times 65}{0.95 \times 50}\right) \times 14 = 15.33 < 18 \text{ in}^2$$

$$\therefore P_{cu} = A_e F_{cf} = A_e \alpha \phi_f F_{yf}$$

$$= 15.33 \times (1) \times (1) \times (50) = 766.5 \text{ kips}$$

CA 6.13.2.7 the nominal shear resistance of a high strength Bolt is

$$R_n = 0.48 A_b F_{ub} N_s$$

$$A_b = \left(\frac{0.875}{2}\right)^2 \times \pi = 0.6 \text{ in}^2$$

$F_{ub} = 120$ ksi $N_s = 2$ for Double shear.

$$R_n = 0.48 \times 0.6 \times 120 \times 2 = 69.1 \text{ kips} \leftarrow \underline{\text{Control}}$$

the nominal Bolt Bearing resistance is taken as

AASHTO. 6.13.2.9.

the clear distance between Bolts is.

$$3'' - 1'' = 2''$$

the clear Bolt edge distance is

$$(2'' - 1/8'') - 1/2'' = 1.375 \text{ in} < 2d.$$

$$\therefore R_n = 1.2 L_c t F_u$$

$$= 1.2 (1.375) \times (1) \times 65 = 107 \text{ kips}$$

No filler plate is required

Bolts needed $N = \frac{P_{cu}}{\phi_s R_n} = \frac{766.5}{(0.8)(69.1)} = 13.8$ bolts

Provide 28 Bolts o.k.

(or splice plate can be reduced)

<u>Check slip resistance of Bolts</u>

AASHTO 6.13.6.1.4c

Bolted connections for flange splices shall be designed as slip critical connections for the flange design force. As a minimum, the design force for the flange under consideration shall be taken as the Service II design stress, $F_s$, times smaller gross flange area on either side of the splice.

$$F_s = \frac{f_s}{R_h}$$

Factored Service II moment

span 3. Point 0.2 is the controlling case

$M_{DC1} = 336^{k\text{-}ft}$ $M_{DC2} = 75^{k\text{-}ft}$ $M_{DW} = 79^{kips\text{-}ft}$

$+M_{(LL+IM)} = 4227^{k\text{-}ft}$ $-M_{(LL+IM)} = -3621^{k\text{-}ft}$

$+M_u = 336 + 75 + 79 + 4227 = \underline{4717}^{k\text{-}ft}$

$-M_u = 336 + 75 + 79 - 3621 = -3131^{k\text{-}ft}$

$F_s = \frac{f_s}{R_h} = \frac{M_u/S_{NCL}}{L_0} = \frac{(4717)(12)/3112}{1} = 18.1^{ksi}$

$R_u = F_s \times A_g = 18.1^{ksi} \times 18^{in^2} = 326^{kips}$

$R_r = R_N$ (nominal resistance as specified in AASHTO 6.13.2.8

$R_n = K_h K_s N_s P_t$

$N_s = 2$ , $P_t$ = minimum required bolt tension specified in Table 1 = 39 kips.
[28 kips for 3/4" Bolt]

$K_h$ = hole size factor specified in Table 2 = 1

$K_s$ = surface condition factor specified in Table 3 = 0.5

$R_n = 1 \times (0.5)(2) \times 39 = 39^{kips}$ per Bolt

Bolt needed = $\frac{R_u}{R_n} = \frac{326^{kip}}{39} = 8.35$ Bolts
(11.64 Bolts for 3/4" Bolts)

provide 28 bolts ok

Check Tensile Resistance of splice plate
AASHTO 6.8.2.1

$P_r = \phi_y P_{ny} = \phi_y F_y A_g$
$P_r = \phi_u P_{nu} = \phi_u F_u A_n U$ } lesser of the two

$P_r = 0.95(18 \times 1 + 2 \times 8 \times 1) \times (50) = 1615$ or

$P_r = 0.8(65) \times (14 + 12) \times 1 = 1352^{kip} > P_u = 767^{kips}$

Check Block shear rupture resistance (AASHTO 6.13.4) of splice plate

If $A_{tn} \geq 0.58 A_{vn}$ Then

$R_r = \phi_{bs}(0.58 F_y A_{vg} + F_u A_{tn})$ otherwise

$R_r = \phi_{bs}(0.58 F_u A_{vn} + F_y A_{tg})$

$A_{tn}$ = net Area along the plane resisting tension stress

$A_{vn}$ = net Area along the plane resisting shear stress

Inner plate and outer plate.

$A_{tn} = 2[6 - 1.5 \times 1] \times 1 = 9 \text{ in}^2$

$A_{vn} = 2[20 - 6.5 \times 1] \times 1 = 27$

$A_{tn} < 0.58 A_{vn} = 15.66 \text{ in}^2$

$\therefore A_{tn} < 0.58 A_{vn}$, so $R_r = \phi_{bs}(0.58 F_u A_{vn} + F_y A_{tg})$

$A_{tg} = 2 \times (6) \times 1 = 12 \text{ in}^2$

$R_r = 0.8[0.58(65)(27) + 50(12)] = 1294^{kips}$

$> \frac{P_{cu}}{2} = 767/2 = 384^{kips}$

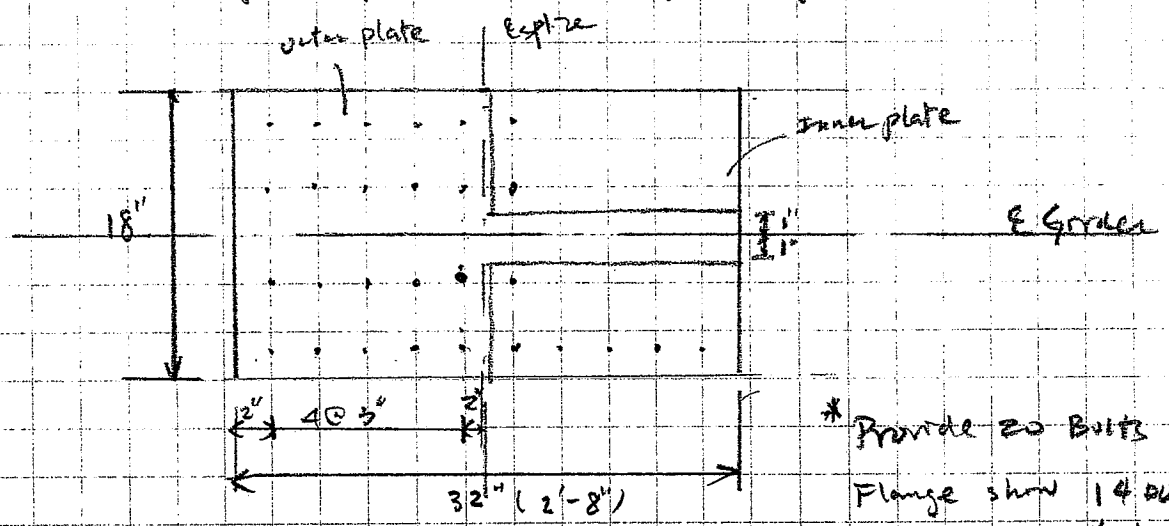
Check Block shear rupture resistance of Bottom flange

Since Bottom Flange thickness 1" is same as splice plate the force will be the same

$R_r = 1294^{kips} > P_{cu} = 767^{kips}$ OK.

Design Top flange splices

From the bottom flange splice calculations, the trial splice plate thickness and number of Bolts seems too conservative. Reduce the splice plate to the following




* Provide 20 Bolts
Flange show 14 bolts needed.

Try 32" x 18" x 3/4" splice plate

Check Tensile resistance of splice plate

$P_r = \phi_u P_{nu} = \phi_u F_u A_{nu}$ controls

Inner Plate $A_n = 2[8 - 2(1)] \times 0.75 = 9 \text{ in}^2$ ← controls

outer plate $A_n = [18 - 4(1)] \times 0.75 = 10.5 \text{ in}^2$

$P_r = 0.8 \times (65) \times (9) \times (1) = 468 \text{ kips} > \frac{P_{cu}}{2}$

$= 766.5/2 = 383 \text{ kips}$

check Block shear rupture resistance of splice plate

Inner Plate

$A_{tn} = 2[6 - 1.5 \times (1)] \times 3/4 = 6.75 \text{ in}^2$

$A_{vn} = 2[14 - 4.5 \times (1)] \times 3/4 = 14.25 \text{ in}^2$

$A_{tn} = 6.75\ in^2 < 0.58\ A_{vn} = 0.58 \times 14.25 = 8.27$

$R_r = \phi_{bs}\,(0.58\,F_u\,A_{vn} + F_y\,A_{tg})$

$A_{tg} = 2 \times (6) \times 0.75 = 9\ in^2$

$R_r = 0.8\,[0.58\,(65)\,(14.25) + (50)\,(9)]$

$= 789.8 > \frac{P_{cu}}{2} = 766\,\frac{7}{2} = 383.3\ kips$

outer plate is same as inner plate.

## Design web splice

Try web splice plate shown on right.

CA 6.13.6.1.4b

shear $V_{uw} = \phi_v V_n$

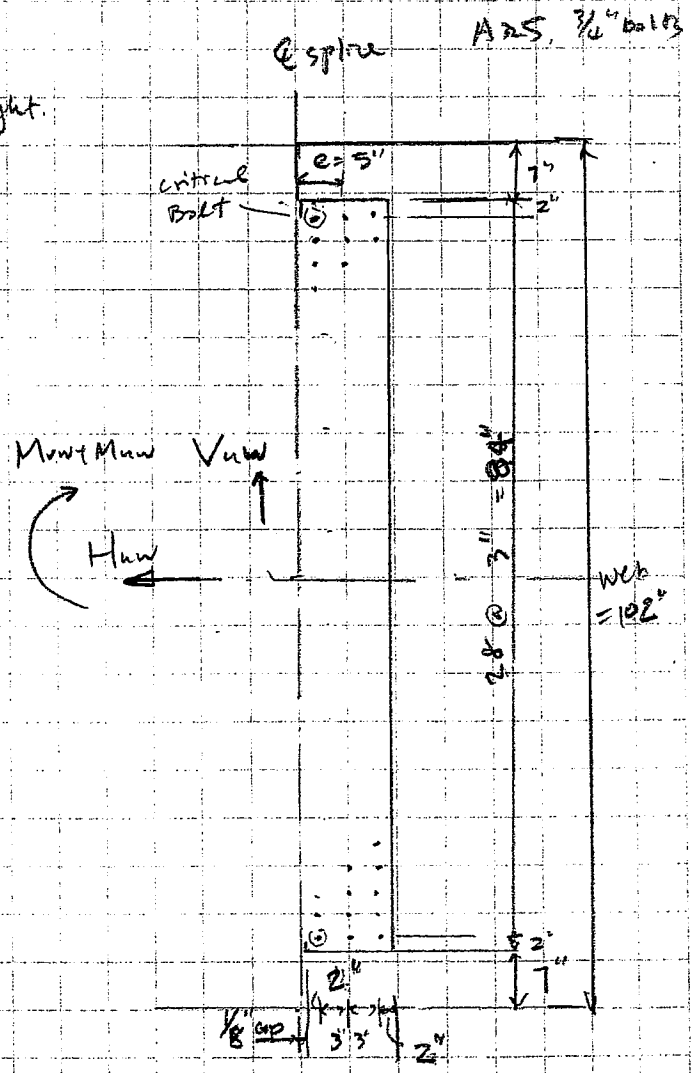
$= (1)\,(1350\ kips)$

($V_n$ calculated earlier)

moment induced by eccentrically loaded shear

$M_{vw} = V_{uw}(e) = (1350 \times 5)$

$= 6750\ k\text{-}in$



Check the flange splice by using A325. d = 3/4" Bolt.

$$A_n = [18 - 4(3/4" + 1/8")] \times 1" = 14.5 \text{ in}^2$$

$$A_g = 18 \times 1 = 18 \text{ in}^2$$

$$A_e = \left(\frac{\phi_u F_u}{\phi_y F_{yx}}\right) A_n$$

$$= \left(\frac{0.8 \times 65}{0.95 \times 50}\right) \times 14.5 = 15.87 \text{ in}^2 < 18 \text{ in}^2$$

$$\therefore P_{cu} = A_e F_{cf} = A_e \alpha \phi_f F_{yf}$$

$$= 15.87 \times (1)(1 \times (50) = 794^{kips} \rightarrow \text{say } 800^{kips}$$

CA 6.13.2.7 the nominal shear resistance of high strength bolt

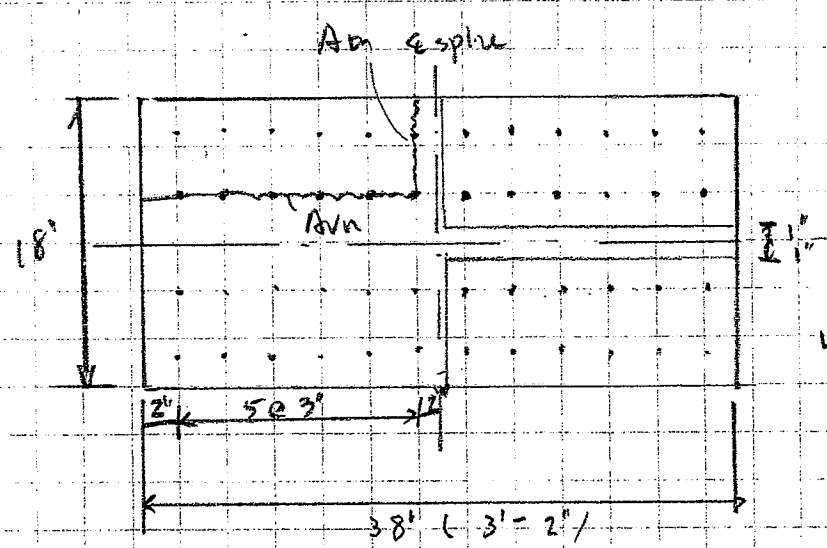
$$R_n = 0.48 A_b F_{ub} N_s$$

$$A_b = (0.75/2)^2 \times \pi = 0.442 \text{ in}^2$$

$$R_n = 0.48 \times 0.442 \times 120 \times 2 = 50.89^{kips} \leftarrow \text{control}$$

Bolts needed $N = \dfrac{P_{cu}}{\phi_s R_n} = \dfrac{800}{(0.8)(50.89)} = 19.65$ bolts

Provide 24 Bolts (20% extra)




use 38" x 18" x 3/4" splice plate

Try 38" x 18" x 3/4" splice plate

check Tensile resistance of splice plate

$P_r = \phi_u P_{nu} = \phi_u F_u A_n U$ controls

Inner Plate

$A_n = 2(8 - 2(3/4" + 1/8")) \times 0.75 = 9.375 \text{ in}^2$ ← control

7/8

outer plate

$A_n = [18 - 4(0.875)] \times 0.75 = 10.875 \text{ in}^2$

$P_r = 0.8 \times (65) \times (9.375) \times 1 = 488 \text{ kips} > \frac{P_{cu}}{2} = \frac{800}{2} = 400 \text{ kips}$

check Block shear rupture resistance of splice plate

Inner Plate

$A_{tn} = 2[6 - 1.5 \times (0.875)] \times 0.75 = 7.03 \text{ in}^2$

$A_{vn} = 2[(15 + 1.875) - 5.5(0.875)] \times 0.75 = 18.09 \text{ in}^2$

$A_{tn} < 0.58 A_{vn}$

$\therefore R_r = \phi_{bs}(0.58 F_u A_{vn} + F_y A_{tg})$

$A_{tg} = 2 \times 6 \times 0.75 = 9 \text{ in}^2$

$R_r = 0.8[0.58(65)(18.09) + 50 \times 9]$

$= 905 \text{ kips} > \frac{P_{cu}}{2} = 400 \text{ kips}$

Outer plate is same as Inner plate.

Moment resisted by web.

Case 1. positive Bending

At the strength limit state, the smaller section at the point of splice is composite compact section.

CA 6.13.6.1.4b-1a.

$$M_{nw} = \phi_f \frac{t_w F_{yw}}{4} (D^2 - 4 y_0^2)$$

$y_0$, distance from the mid-depth of the web to the plastic neutral axis.

$y_0 > D/2$, all web is in tension.

Therefore $M_{nw} = 0$.

case 2. Negative Bending.

At the strength limit state, the smaller section at the point of splice is non compact section.

$$M_{nw} = \phi_f \frac{t_w D^2}{12} (F_{nc} + F_{yw})$$

$F_{nc} = F_{yw} = 50^{ksi}$. $\phi_f = 1$.

$D = 102^{in}$ $t_w = 0.75^{in}$

$$M_{nw} = (1) \times \frac{0.75 \times 102^2}{12} (50+50) = 65025^{kip-in}$$

Horizontal force resulted in web.

case 1 positive Bending

Since the whole web is in tension CA-C6.13.6.1.4b-2a is no longer valid, use the following equation.

$$H_{uw} = \phi_f (t_w D F_{yw})$$
$$= (1.0)(0.75 \times 1.2 \times 50) = 3825^{kip}$$

case 2. negative Bending

$$H_{uw} = \phi_f \frac{t_w D}{2} (F_{yw} - F_{nc}) = 0.$$

Web design force summary.

case 1. - positive Bending

Moment = $M_{uw} + M_{vw} = 0 + 6750 = 6750^{k\text{-}in}$

Horizontal Force = $H_{uw} = 3825^{kips}$

Vertical Force = $V_{uw} = 1350^{kips}$

case 2 - Negative Bending

Moment = $M_{uw} + M_{vw} = 65025 + 6750 = 71775^{kips\text{-}in}$

Horizontal force = $H_{uw} = 0$

vertical force = $V_{uw} = 1350^{kips}$

Nominal shear resistance for A325 - 3/4" bolt in double shear

$R_n = 50.89^{kips}$ ← controls

nominal bearing resistance for each bolt hole on web

$R_n = 1.2 L_c t F_u$ (AASHTO 6.13.2.9-2)

$= 1.2 \times (1.875 - 0.875/2) \times 0.625 \times 65$

$= 70^{kips}$

The most stressed bolts are the corner Bolts. check the shear force R on the top right Bolt

$I_p = \Sigma x^2 + \Sigma y^2$

$= (2)(3)(42^2 + 39^2 + 36^2 + 33^2 + \overset{9135}{30^2} + 27^2 + 24^2 + 21^2 + 18^2 + 15^2 + 12^2 + 9^2 + 6^2 + 3^2) + 2 \times (29) \times (3^2)$

$= 54810 + 522 = 55332$

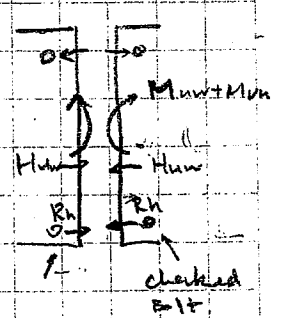
case 1 - Positive Bending, Lower right corner

$R_v = \frac{V_{uw}}{(3 \times 29)} = \frac{1350}{87} = 15.52^{kips}$ (↑)

$R_h = \frac{H_{uw}}{(3 \times 29)} = \frac{3825}{87} = 43.97^{kips}$ (←)

$R_x = \frac{M_u y}{I_p} = \frac{6750^{k\cdot in} \times 42^{in}}{55332} = 5.12^{kips}$ (←)

$R_y = \frac{M_u x}{I_p} = \frac{6750 \times 3}{55332} = 0.366^{kips}$ (↑)




$R_{bolt} = \sqrt{(R_h + R_x)^2 + (R_v + R_y)^2}$

$= \sqrt{(43.97 + 5.12)^2 + (15.52 + 0.366)^2} = \sqrt{2409.8 + 252.365}$

$= 51.6^{kips} > \phi_s R_n$ N.G.

$\phi_s R_n = (0.8)(50.89) = 40.71^{kips}$

case 2 — Negative Bending

$$R_x = \frac{M_u y}{I_p} = \frac{71775 \times 42}{55332} = 54.48^{kips} \quad (\rightarrow)$$

$$R_y = \frac{M_u x}{I_p} = \frac{71775 \times 3}{55332} = 3.89^{kips} \quad (\downarrow)$$

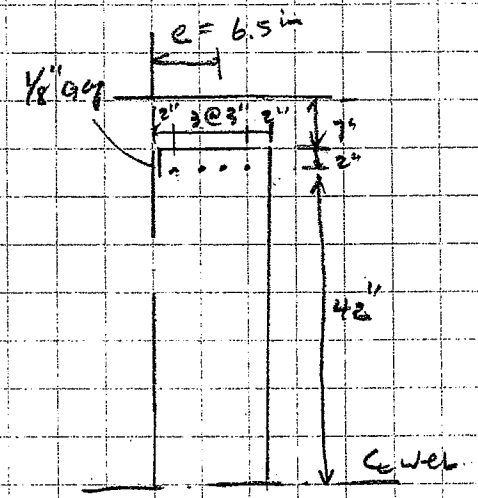
$$R_v = \frac{1350}{87} = 15.52^{kips} \quad (\downarrow)$$

$$R_h = 0$$

$$R_{bolt} = \sqrt{54.48^2 + (15.52 + 3.89)^2} = \sqrt{2968 + 376.7}$$

$$= 57.83^{kips} > \phi_s R_n \quad \underline{\underline{N.G}}$$

<u>2nd trial, add 1 row of Bolts</u>




$$M_{uw} = 1350 \times 6.5^{in} = 8775^{kip\text{-}in}$$

case 1 - Positive Bending

moment $M_u = M_{uuw} + M_{uw}$
$= 0 + 8775 = 8775^{k\text{-}in}$

Horizontal force $H_{uw} = 3825^{kip}$

vertical shear $V_{uw} = 1350^{kips}$

case 2 - Negative Bending

moment $M_u = M_{uuw} + M_{uw} = 65025 + 8775 = 73800^{kip\text{-}in}$

Horizontal $H_{uw} = 0$

vertical $V_{uw} = 1350^{kips}$

$I_p = (2)(4)(9135) + (2)(29 \times 4.5^2 + 29 \times 0.5^2)$

$= 73080 + 1189 = 74269$

case 1 - Positive Bending, lower right corner.

$R_x = \frac{M_u y}{I_p} = \frac{8775 \times 42}{74269} = 4.96^{kips} \ (\leftarrow)$

$R_y = \frac{M_u x}{I_p} = \frac{8775 \times 4.5}{74269} = 0.354^{kips} \ (\uparrow)$

$R_v = \frac{V_{uw}}{(4 \times 29)} = \frac{1350}{116} = 11.6^{kips} \ \uparrow$

$R_h = \frac{H_{uw}}{4 \times 29} = \frac{3825}{116} = 32.97 \ (\leftarrow)$

$R_{bolt} = \sqrt{(32.97 + 4.96)^2 + (11.6 + 0.354)^2}$

$= \sqrt{1439 + 142.9} = 39.77^{kips} < \phi_b R_N = 40.71^{kips}$ ok

case 2 - Negative Bending, lower right corner

$R_x = \frac{73800 \times 42}{74269} = 41.73 \ (\rightarrow)$

$R_y = \frac{73800 \times 4.5}{74269} = 2.98^{kips} \ (\downarrow)$

$R_v = \frac{1350}{116} = 11.6^{kip} \ (\downarrow)$

$R_{bolt} = \sqrt{41.73^2 + (11.6 + 2.98)^2} = 44.2^{kips} > \phi_b R_N = 40.71^{kips}$

8.5% overstress

use 7/8" A325 Bolts.

shear resistance of A325, – 7/8" bolt in double shear

$R_n = 69.1^{kips}$

nominal bearing resistance for each bolt hole on web.

$R_n = 1.2 L_c t F_u$

$= 1.2(1.875 - 0.5)(0.625) \times 65 = 67.3^{kips}$ ← Control.

$\phi_{bb} R_N = 0.8 \times (67.3) = 53.84^{kips}$

$R_{bolt} = 44.2^{kips} < \phi_{bb} R_N$ OK.

<u>Check slips of Bolts – Service Limit State II</u>

$+M_u = 4227^{k\text{-}ft}$ $-M_u = -3191^{k\text{-}ft}$

span 3, 0.2 pt

$V_{DC1} = 86.5^{kips}$ $V_{DC2} = 13.7^{kips}$

$V_{DW} = 18.38^{kips}$ $[(LL+IM)_{HL-93}] = 1.09^{kips}$

$V_u = V_{DC1} + V_{DC2} + V_{DW} + (1.3) DF_v (LL+IM)_{HL-93}$

$= 86.5 + 13.7 + 18.4 + 1.3 \times 1.09 = 260.3^{kips}$ ↓

moment due to eccentrically loaded shear

$M_{vu} = V_u \times e = 260.3 \times (6.5) = 1692^{k\text{-}in}$

CA 6.13.6.1.4b-14

$M_{uw} = \frac{t_w D^2}{12} |f_s - f_{vs}|$

$H_{uw} = \frac{t_w D}{2} (f_s + f_{vs})$

$f_s$: maximum flexural stress due to Load combination Service II at the extreme fiber of the flange under consideration for the smaller section at the point of splice (positive for tension, negative for compression)

$$f_s = \frac{M_u}{S_{NCT}} = \frac{4227 \times 12}{3112} = 16.3^{ksi} \text{ (compression)}$$

$$f_{os} = \frac{M_u}{S_{NCL}} = \frac{4227 \times 12}{3112} = +16.3^{ksi} \text{ (Tension)}$$

$$M_{uw} = \frac{(0.75) \times (102)^2}{12} \left| -16.3 - 16.3 \right| = 21198^{k\text{-}in}$$

$$H_{uw} = \frac{(0.75)(102)}{2}(-16.3 + 16.3) = 0$$

$$R_x = \frac{(M_{vw} + M_{uw})\,y}{I_p} = \frac{(1692 + 21198) \times 42}{74269} = 13^{kips} \ (\leftarrow)$$

$$R_y = \frac{(M_{vu} + M_{uw})\,y}{I_p} = \frac{(1692 + 21198) \times 4.5}{74269} = 1.4^{kips} \ (\uparrow)$$

$$R_v = \frac{V_u}{4 \times 29} = \frac{260}{116} = 2.24 \ (\uparrow)$$

$$R_h = \frac{H_{uw}}{4 \times 29} = 0$$

$$R_{bolt} = \sqrt{(1.4 + 2.24)^2 + 13^2} = 13.5^{kips}$$

slip resistance of one bolt is

$$R_N = K_h K_s N_s P_t \quad \text{(AASHTO 6.13.2.8-1)}$$

$$= 39^{kips} \ (7/8'' \text{ Bolt}) > 13.5^{kips} \quad \text{OK}$$

Web splice Plate Design / check

Plate size: $\frac{1}{2}'' \times 26'' \times 88''$

- Yielding on the gross section.

$V_r = \phi_v \, 0.58 A_g F_w$

$A_g = 88'' \times \frac{1}{2}'' = 44 \text{ in}^2$, $\phi_v = 1.0$, $F_w = 50$

$V_r = (1)(0.58)(2 \times 44)(50) = 2552^{kips} > V_{uw} = 1350^{kips}$

- Fracture on net section

$A_n = 2[88 - 2 \times 1.)] \times \frac{1}{2} = 59 \text{ in}^2$

$V_r = \phi_u (0.58 F_u A_n)$

$= (0.8)(0.58)(65)(59) = 1779^{kips} > V_{uw} = 1350^{kips}$

- Block shear rupture

$A_{tn} = 2[11 - 3.5(1.0)][\frac{1}{2}] = 7.5 \text{ in}^2$

$A_{vn} = 2[86 - 2 \times 8.5(1.0)](\frac{1}{2}) = 57.5 \text{ in}^2$

$A_{tn} < 0.58 A_{vn} = 33.35 \text{ in}^2 \Rightarrow$ use AASHTO 6.13.4-2

$A_{tg} = 2(11)(\frac{1}{2}) = 11 \text{ in}^2$

$R_r = \phi_{bs}(0.58 F_u A_{vn} + F_y A_{tg})$

$= (0.8)(0.58 \times 65 \times \overset{2168}{57.5} + 50 \times \overset{550}{11})$

$= 2174^{kips} > V_{uw} = 1350^{kips}$ ok

Check splice plate flexure resistance.

$$I_w = 2\left[\frac{(1/2)(88)^3}{12}\right] = 56789 \text{ in}^4$$

$$f_b = \frac{M_u y}{I_w} = \frac{73800 \times 44}{56789} = 57.1 \text{ ksi} > F_y = 50 \text{ ksi} \quad \text{N.G.}$$

Try splice plate thickness $\underline{5/8''}$

$$I_w = 2\left[\frac{(\frac{5}{8})(88)^3}{12}\right] = 70986 \text{ in}^4$$

$$f_b = \frac{73800 \times 44}{70986} = 45.74 < F_y = 50 \text{ ksi}$$

Final web splice plate.

$$\underline{\underline{5/8'' \times 26'' \times 88''}}$$

FHW
10/6/10

LA River Bridge
Hinge Resign



— Hinge Pin & Hinge Plate will resist dead load and live load vertically. (strength limit state)

DL = 1780 kip  WS = 288 kip

1780/12 girders = 148 kip/girder  288/12 = 24 kips/girder

HL-93 = 125 kips (unfactor)

P-15 = 250 kips (unfactor)

Provide 9" pin

Strength I

= 1.25 x 148 + 1.5 x 24 + 1.75 x 125 = 440 kip

strength II

= 1.25 x 148 + 1.5 x 24 + 1.35 x 250 = 559 kip ← control

each pin will have 280 kips applied shear

9" pin shear resistance

$R_n = 0.38 A_b F_{ub}$

$A_b = \frac{1}{4}\pi D^2 = \frac{1}{4}\pi (9)^2 = 63.61 \text{ in}^2$

$F_{ub} = 0.38 \times (63.61)(58) = 1400 \text{ kips}$ ✓

Hinge plate 1" x 1'-10" x 6'-6" - 2 of them

(1) Plate under Tension

$\phi_y F_y A_g$ or $\phi_u F_u A_n U$ (6.8.2.1-2)

$F_y = 36$ ksi $A_g = 22" \times 1" = 22"$

$F_u = 58$ ksi $A_n = [22 - (9 + 0.125)] \times 1" = 12.875"$

$\phi_y = 0.95$ $\phi_u = 0.8$

$\phi_y F_y A_g = 0.95 \times 36 \times 22" = 752$ kips

$\phi_u F_u A_n U = 0.8 \times 58 \times 12.875 \times 1 = 597$ kips ← control

2 Hinge plates $2 \times 597 = 1195 > 559$ kips ok

(2) Shear

$R_r = \phi_v R_n$ (6.13.5.3-1)

$R_n = 0.58 A_g F_y$ (6.13.5.3-2)

$R_n = 0.58 (22)(36) = 459$ kips

$\phi_v = 1$. $R_r = R_n = 459$ kips

$2 \times 459 = 918$ kips $> 559$ kips ok

## Distribution Factor for Fatigue Limit State

$DF_m=0.06+(S/14)^{0.4}*(S/L)^{0.3}*(K_g/12Lt_s^3)^{0.1}$

$DF_v=0.36+S/25$

$K_g$ = 4534476 $in^4$

ts = 10 in

MPF = 1.2 (Multiplle lane presence factor)

S= 11 ft

| Span | L= (ft) | $DF_m$ | $DF_v$ |
|---|---|---|---|
| 1* | 133.5 | 0.45 | 0.8 |
| 1 & 2** | 141.75 | 0.44 | 0.8 |
| 2* | 150 | 0.43 | 0.8 |
| 2 & 3** | 150 | 0.43 | 0.8 |
| 3* | 150 | 0.43 | 0.8 |
| 3 & 4** | 141.75 | 0.44 | 0.8 |
| 4* | 133.5 | 0.45 | 0.8 |

S= 17 ft

| Span | L= (ft) | $DF_m$ | $DF_v$ |
|---|---|---|---|
| 1* | 133.5 | 0.59 | 1.04 |
| 1 & 2** | 141.75 | 0.58 | 1.04 |
| 2* | 150 | 0.56 | 1.04 |
| 2 & 3** | 150 | 0.56 | 1.04 |
| 3* | 150 | 0.56 | 1.04 |
| 3 & 4** | 141.75 | 0.58 | 1.04 |
| 4* | 133.5 | 0.59 | 1.04 |

## Fatigue I Limit State - Moment and Shear

Because the effective width is about 10', conservatively assume the exterir girder will take 1 full truck veicle load.

Fatigue I (HL-93 Truck) = 1.75*(DF)*(LL+IM)$_{HL-93}$

= 1.75/1.2, Remove Multiple Lane Importance factor

1.458333333 1.458333333

| Span | Point | Unfactored Fatigue Moment (LL+IM) HL-93, One design Truck | | Factored Fatigue Moment (LL+IM) HL-93, One design Truck | | | 1.75*(DF)*(LL+IM)$_{HL-93}$ | | |
|---|---|---|---|---|---|---|---|---|---|
| | | +M (K-ft) | -M (K-ft) | +M (K-ft) | -M (K-ft) | M_sr (K-ft) | +M (K-ft) | -M (K-ft) | M_sr (K-ft) |
| | 0.0 | 6.17 | -52.31 | 9.00 | -76.29 | 85.28 | 4.02 | -34.11 | 38.13 |
| | 0.1 | 740.13 | 0 | 1079.36 | 0.00 | 1079.36 | 482.56 | 0.00 | 482.56 |
| | 0.2 | 1249.34 | 0 | 1821.95 | 0.00 | 1821.95 | 814.57 | 0.00 | 814.57 |
| | 0.3 | 1551 | 0 | 2261.88 | 0.00 | 2261.88 | 1011.25 | 0.00 | 1011.25 |
| 1 | 0.4 | 1588.56 | 0 | 2316.65 | 0.00 | 2316.65 | 1035.74 | 0.00 | 1035.74 |
| | 0.5 | 1570.64 | 0 | 2290.52 | 0.00 | 2290.52 | 1024.06 | 0.00 | 1024.06 |
| | 0.6 | 1306.02 | 0 | 1904.61 | 0.00 | 1904.61 | 851.53 | 0.00 | 851.53 |
| | 0.7 | 824.34 | 0 | 1202.16 | 0.00 | 1202.16 | 537.47 | 0.00 | 537.47 |
| | 0.8 | 93.36 | -5.67 | 136.15 | -8.27 | 144.42 | 60.87 | -3.70 | 64.57 |
| | 0.9 | 1.76 | -821.3 | 2.57 | -1197.73 | 1200.30 | 1.15 | -535.49 | 536.64 |
| | 1.0 | 0 | -1741.04 | 0.00 | -2539.02 | 2539.02 | 0.00 | -1135.16 | 1135.16 |
| | 0.0 | 5.49 | -1637.02 | 8.01 | -2387.32 | 2395.33 | 3.43 | -1024.16 | 1027.60 |
| | 0.1 | 860.52 | -1466.34 | 1254.93 | -2138.41 | 3393.34 | 538.36 | -917.38 | 1455.75 |
| | 0.2 | 1466 | -1259.65 | 2137.92 | -1836.99 | 3974.91 | 917.17 | -788.07 | 1705.24 |
| | 0.3 | 1839.42 | -1052.97 | 2682.49 | -1535.58 | 4218.07 | 1150.79 | -658.77 | 1809.56 |
| 2 | 0.4 | 1923.65 | -846.29 | 2805.32 | -1234.17 | 4039.50 | 1203.49 | -529.46 | 1732.95 |
| | 0.5 | 1899.77 | -636.91 | 2770.50 | -928.83 | 3699.33 | 1188.55 | -398.47 | 1587.01 |
| | 0.6 | 1625.71 | -432.93 | 2370.83 | -631.36 | 3002.18 | 1017.09 | -270.85 | 1287.94 |
| | 0.7 | 1098.25 | -226.24 | 1601.61 | -329.93 | 1931.55 | 687.09 | -141.54 | 828.64 |
| | 0.8 | 306.03 | -19.56 | 446.29 | -28.53 | 474.82 | 191.46 | -12.24 | 203.70 |
| | 0.9 | 187.12 | -729.59 | 272.88 | -1063.99 | 1336.87 | 117.07 | -456.45 | 573.52 |
| | 1.0 | 393.8 | -1790.54 | 574.29 | -2611.20 | 3185.50 | 246.37 | -1120.21 | 1366.58 |
| | 0.0 | 332.14 | -1704.93 | 484.37 | -2486.36 | 2970.73 | 207.80 | -1066.65 | 1274.45 |
| | 0.1 | 895.84 | -1528.11 | 1306.43 | -2228.49 | 3534.93 | 560.46 | -956.03 | 1516.49 |
| | 0.2 | 1594.04 | -1351.28 | 2324.64 | -1970.62 | 4295.26 | 997.27 | -845.40 | 1842.67 |
| | 0.3 | 2087.83 | -1174.46 | 3044.75 | -1712.75 | 4757.51 | 1306.20 | -734.77 | 2040.98 |
| 3 | 0.4 | 2356.55 | -997.64 | 3436.64 | -1454.89 | 4891.53 | 1474.32 | -624.15 | 2098.47 |
| | 0.5 | 2385.77 | -820.81 | 3479.25 | -1197.01 | 4676.26 | 1492.60 | -513.52 | 2006.12 |
| | 0.6 | 2356.55 | -971.74 | 3436.64 | -1417.12 | 4853.76 | 1474.32 | -607.95 | 2082.27 |
| | 0.7 | 2087.73 | -1143.98 | 3044.61 | -1668.30 | 4712.91 | 1306.14 | -715.70 | 2021.84 |
| | 0.8 | 1594.04 | -1316.21 | 2324.64 | -1919.47 | 4244.11 | 997.27 | -823.46 | 1820.73 |
| | 0.9 | 895.84 | -1488.45 | 1306.43 | -2170.66 | 3477.09 | 560.46 | -931.21 | 1491.68 |
| | 1.0 | 63.31 | -1660.68 | 92.33 | -2421.83 | 2514.15 | 39.61 | -1038.97 | 1078.57 |
| | 0.0 | 0 | -1741.04 | 0.00 | -2539.02 | 2539.02 | 0.00 | -1135.16 | 1135.16 |
| | 0.1 | 1.76 | -821.3 | 2.57 | -1197.73 | 1200.30 | 1.15 | -535.49 | 536.64 |
| | 0.2 | 93.36 | -5.67 | 136.15 | -8.27 | 144.42 | 60.87 | -3.70 | 64.57 |
| | 0.3 | 824.34 | 0 | 1202.16 | 0.00 | 1202.16 | 537.47 | 0.00 | 537.47 |
| 4 | 0.4 | 1306.02 | 0 | 1904.61 | 0.00 | 1904.61 | 851.53 | 0.00 | 851.53 |
| | 0.5 | 1570.64 | 0 | 2290.52 | 0.00 | 2290.52 | 1024.06 | 0.00 | 1024.06 |
| | 0.6 | 1588.56 | 0 | 2316.65 | 0.00 | 2316.65 | 1035.74 | 0.00 | 1035.74 |
| | 0.7 | 1551 | 0 | 2261.88 | 0.00 | 2261.88 | 1011.25 | 0.00 | 1011.25 |
| | 0.8 | 1249.34 | 0 | 1821.95 | 0.00 | 1821.95 | 814.57 | 0.00 | 814.57 |
| | 0.9 | 740.13 | 0 | 1079.36 | 0.00 | 1079.36 | 482.56 | 0.00 | 482.56 |
| | 1.0 | 6.17 | -52.31 | 9.00 | -76.29 | 85.28 | 4.02 | -34.11 | 38.13 |

Fatigue I Limit state

1.75/1.2, Remove Multiple Lane Importance factor

For Shear

| Span | Point | Unfactored Fatigue Shear (LL+IM) HL-93, One design Truck +V (Kt) | Unfactored Fatigue Shear (LL+IM) HL-93, One design Truck -V (K) | 1.458333333 Factored Fatigue Sheat (LL+IM) HL-93, One design Truck +V (K-ft) | 1.458333333 Factored Fatigue Sheat (LL+IM) HL-93, One design Truck -V (K-ft) | V_sr (K) | $1.75*(DF)*(LL+IM)_{HL-93}$ +V (K-ft) | $1.75*(DF)*(LL+IM)_{HL-93}$ -V (K-ft) |
|---|---|---|---|---|---|---|---|---|
| | 0.0 | 67.42 | 0 | 98.32 | 0.00 | 98.32 | 78.66 | 0.00 |
| | 0.1 | 58.71 | -4.53 | 85.62 | -6.61 | 92.23 | 68.50 | -5.29 |
| | 0.2 | 48.52 | -9.06 | 70.76 | -13.21 | 83.97 | 56.61 | -10.57 |
| | 0.3 | 38.33 | -16.99 | 55.90 | -24.78 | 80.68 | 44.72 | -19.82 |
| 1 | 0.4 | 28.14 | -26.85 | 41.04 | -39.16 | 80.19 | 32.83 | -31.33 |
| | 0.5 | 18.15 | -37.03 | 26.47 | -54.00 | 80.47 | 21.18 | -43.20 |
| | 0.6 | 9.63 | -47.22 | 14.04 | -68.86 | 82.91 | 11.24 | -55.09 |
| | 0.7 | 5.1 | -57.41 | 7.44 | -83.72 | 91.16 | 5.95 | -66.98 |
| | 0.8 | 0.58 | -67.6 | 0.85 | -98.58 | 99.43 | 0.68 | -78.87 |
| | 0.9 | 0 | -73.83 | 0.00 | -107.67 | 107.67 | 0.00 | -86.14 |
| | 1.0 | 0 | -78.68 | 0.00 | -114.74 | 114.74 | 0.00 | -91.79 |
| | 0.0 | 69.44 | 0 | 101.27 | 0.00 | 101.27 | 81.01 | 0.00 |
| | 0.1 | 60.79 | 0 | 88.65 | 0.00 | 88.65 | 70.92 | 0.00 |
| | 0.2 | 50.86 | -7.8 | 74.17 | -11.38 | 85.55 | 59.34 | -9.10 |
| | 0.3 | 40.92 | -17.74 | 59.68 | -25.87 | 85.55 | 47.74 | -20.70 |
| 2 | 0.4 | 30.99 | -27.67 | 45.19 | -40.35 | 85.55 | 36.16 | -32.28 |
| | 0.5 | 21.05 | -37.61 | 30.70 | -54.85 | 85.55 | 24.56 | -43.88 |
| | 0.6 | 13.78 | -47.55 | 20.10 | -69.34 | 89.44 | 16.08 | -55.48 |
| | 0.7 | 13.78 | -57.48 | 20.10 | -83.83 | 103.92 | 16.08 | -67.06 |
| | 0.8 | 13.78 | -67.42 | 20.10 | -98.32 | 118.42 | 16.08 | -78.66 |
| | 0.9 | 13.78 | -74.41 | 20.10 | -108.51 | 128.61 | 16.08 | -86.81 |
| | 1.0 | 13.78 | -79.22 | 20.10 | -115.53 | 135.63 | 16.08 | -92.42 |
| | 0.0 | 71.65 | -11.48 | 104.49 | -16.74 | 121.23 | 83.59 | -13.39 |
| | 0.1 | 64.46 | -11.48 | 94.00 | -16.74 | 110.75 | 75.20 | -13.39 |
| | 0.2 | 56.18 | -11.48 | 81.93 | -16.74 | 98.67 | 65.54 | -13.39 |
| | 0.3 | 47.9 | -14.78 | 69.85 | -21.55 | 91.41 | 55.88 | -17.24 |
| 3 | 0.4 | 39.62 | -23.06 | 57.78 | -33.63 | 91.41 | 46.22 | -26.90 |
| | 0.5 | 30.25 | -31.34 | 44.11 | -45.70 | 89.82 | 35.29 | -36.56 |
| | 0.6 | 23.06 | -39.62 | 33.63 | -57.78 | 91.41 | 26.90 | -46.22 |
| | 0.7 | 14.78 | -47.9 | 21.55 | -69.85 | 91.41 | 17.24 | -55.88 |
| | 0.8 | 11.79 | -56.18 | 17.19 | -81.93 | 99.12 | 13.76 | -65.54 |
| | 0.9 | 11.79 | -64.46 | 17.19 | -94.00 | 111.20 | 13.76 | -75.20 |
| | 1.0 | 11.79 | -71.65 | 17.19 | -104.49 | 121.68 | 13.76 | -83.59 |
| | 0.0 | 78.68 | 0 | 114.74 | 0.00 | 114.74 | 91.79 | 0.00 |
| | 0.1 | 73.83 | 0 | 107.67 | 0.00 | 107.67 | 86.14 | 0.00 |
| | 0.2 | 67.6 | -0.58 | 98.58 | -0.85 | 99.43 | 78.87 | -0.68 |
| | 0.3 | 57.41 | -5.1 | 83.72 | -7.44 | 91.16 | 66.98 | -5.95 |
| 4 | 0.4 | 47.22 | -9.63 | 68.86 | -14.04 | 82.91 | 55.09 | -11.24 |
| | 0.5 | 37.03 | -18.15 | 54.00 | -26.47 | 80.47 | 43.20 | -21.18 |
| | 0.6 | 26.85 | -28.14 | 39.16 | -41.04 | 80.19 | 31.33 | -32.83 |
| | 0.7 | 16.99 | -38.33 | 24.78 | -55.90 | 80.68 | 19.82 | -44.72 |
| | 0.8 | 9.06 | -48.52 | 13.21 | -70.76 | 83.97 | 10.57 | -56.61 |
| | 0.9 | 4.53 | -58.71 | 6.61 | -85.62 | 92.23 | 5.29 | -68.50 |
| | 1.0 | 0 | -67.4 | 0.00 | -98.29 | 98.29 | 0.00 | -78.63 |

$V_u = V_{dc1}+V_{dc2}+V_{dw}+1.75(DF_V)(LL+IM)_{HL-93}$

| Span | Point | Dead Load DC1 V_dc1 (K) | DC2 V_dc2 (K) | DW V_dw (K) | Factored Fatigue Sheat (LL+IM) HL-93, One design Truck +V (K-ft) | -V (K-ft) | $1.75*(DF)*(LL+IM)_{HL-93}$ +V (K-ft) | -V (K-ft) | +Vu (K-ft) | -Vu (K-ft) |
|---|---|---|---|---|---|---|---|---|---|---|
| | 0.0 | 104.28 | 17.69 | 22.37 | 98.32 | 0.00 | 78.66 | 0.00 | 223.00 | 144.34 |
| | 0.1 | 80.15 | 12.51 | 17.24 | 85.62 | -6.61 | 68.50 | -5.29 | 178.40 | 104.62 |
| | 0.2 | 56.12 | 7.17 | 12.17 | 70.76 | -13.21 | 56.61 | -10.57 | 132.07 | 64.89 |
| | 0.3 | 27.62 | 3.64 | 5.99 | 55.90 | -24.78 | 44.72 | -19.82 | 81.97 | 17.43 |
| 1 | 0.4 | 3.59 | -1.7 | 0.92 | 41.04 | -39.16 | 32.83 | -31.33 | 35.64 | -28.52 |
| | 0.5 | -25.87 | -3.66 | -5.61 | 26.47 | -54.00 | 21.18 | -43.20 | -13.97 | -78.34 |
| | 0.6 | -49.9 | -9 | -10.69 | 14.04 | -68.86 | 11.24 | -55.09 | -58.36 | -124.68 |
| | 0.7 | -78.48 | -10.98 | -16.95 | 7.44 | -83.72 | 5.95 | -66.98 | -100.46 | -173.39 |
| | 0.8 | -102.51 | -16.32 | -22.03 | 0.85 | -98.58 | 0.68 | -78.87 | -140.18 | -219.73 |
| | 0.9 | -128.74 | -19.83 | -27.54 | 0.00 | -107.67 | 0.00 | -86.14 | -176.11 | -262.25 |
| | 1.0 | -152.77 | -25.17 | -32.61 | 0.00 | -114.74 | 0.00 | -91.79 | -210.55 | -302.34 |
| | 0.0 | 137.67 | 24.24 | 29.69 | 101.27 | 0.00 | 81.01 | 0.00 | 272.61 | 191.60 |
| | 0.1 | 114.97 | 19.04 | 24.65 | 88.65 | 0.00 | 70.92 | 0.00 | 229.58 | 158.66 |
| | 0.2 | 88.69 | 14.17 | 18.89 | 74.17 | -11.38 | 59.34 | -9.10 | 181.09 | 112.65 |
| | 0.3 | 61.69 | 8.17 | 13.19 | 59.68 | -25.87 | 47.74 | -20.70 | 130.79 | 62.35 |
| 2 | 0.4 | 31.21 | 4.43 | 6.58 | 45.19 | -40.35 | 36.16 | -32.28 | 78.38 | 9.94 |
| | 0.5 | 4.21 | -1.57 | 0.88 | 30.70 | -54.85 | 24.56 | -43.88 | 28.08 | -40.36 |
| | 0.6 | -27.61 | -4.55 | -5.98 | 20.10 | -69.34 | 16.08 | -55.48 | -22.06 | -93.62 |
| | 0.7 | -58.61 | -7.73 | -12.6 | 20.10 | -83.83 | 16.08 | -67.06 | -62.86 | -146.00 |
| | 0.8 | -85.61 | -13.73 | -18.3 | 20.10 | -98.32 | 16.08 | -78.66 | -101.56 | -196.30 |
| | 0.9 | -114.25 | -18.1 | -24.37 | 20.10 | -108.51 | 16.08 | -86.81 | -140.64 | -243.53 |
| | 1.0 | -141.25 | -24.1 | -30.07 | 20.10 | -115.53 | 16.08 | -92.42 | -179.34 | -287.84 |
| | 0.0 | 133.92 | 23.97 | 28.96 | 104.49 | -16.74 | 83.59 | -13.39 | 270.44 | 173.46 |
| | 0.1 | 112.12 | 18.66 | 24.02 | 94.00 | -16.74 | 75.20 | -13.39 | 230.00 | 141.41 |
| | 0.2 | 86.52 | 13.74 | 18.38 | 81.93 | -16.74 | 65.54 | -13.39 | 184.18 | 105.25 |
| | 0.3 | 59.52 | 7.74 | 12.68 | 69.85 | -21.55 | 55.88 | -17.24 | 135.82 | 62.70 |
| 3 | 0.4 | 29.34 | 4 | 6.18 | 57.78 | -33.63 | 46.22 | -26.90 | 85.74 | 12.62 |
| | 0.5 | 2.34 | -2 | 0.48 | 44.11 | -45.70 | 35.29 | -36.56 | 36.11 | -35.74 |
| | 0.6 | -29.01 | -4.91 | -6.31 | 33.63 | -57.78 | 26.90 | -46.22 | -13.33 | -86.45 |
| | 0.7 | -59.02 | -7.93 | -12.83 | 21.55 | -69.85 | 17.24 | -55.88 | -62.54 | -135.66 |
| | 0.8 | -86.02 | -13.93 | -18.53 | 17.19 | -81.93 | 13.76 | -65.54 | -104.73 | -184.02 |
| | 0.9 | -112.55 | -17.87 | -24.34 | 17.19 | -94.00 | 13.76 | -75.20 | -141.01 | -229.96 |
| | 1.0 | -139.55 | -23.87 | -30.04 | 17.19 | -104.49 | 13.76 | -83.59 | -179.71 | -277.05 |
| | 0.0 | 138.28 | 25.14 | 29.98 | 114.74 | 0.00 | 91.79 | 0.00 | 285.19 | 193.40 |
| | 0.1 | 118.91 | 20.62 | 25.7 | 107.67 | 0.00 | 86.14 | 0.00 | 251.37 | 165.23 |
| | 0.2 | 94.88 | 15.28 | 20.62 | 98.58 | -0.85 | 78.87 | -0.68 | 209.65 | 130.10 |
| | 0.3 | 80.49 | 12.17 | 17.04 | 83.72 | -7.44 | 66.98 | -5.95 | 176.68 | 103.75 |
| 4 | 0.4 | 56.46 | 6.83 | 11.97 | 68.86 | -14.04 | 55.09 | -11.24 | 130.35 | 64.03 |
| | 0.5 | 24.72 | 3.6 | 5.37 | 54.00 | -26.47 | 43.20 | -21.18 | 76.89 | 12.52 |
| | 0.6 | 0.69 | -1.74 | 0.29 | 39.16 | -41.04 | 31.33 | -32.83 | 30.57 | -33.59 |
| | 0.7 | -28.96 | -4.08 | -6.17 | 24.78 | -55.90 | 19.82 | -44.72 | -19.39 | -83.93 |
| | 0.8 | -56.38 | -6.62 | -12.19 | 13.21 | -70.76 | 10.57 | -56.61 | -64.62 | -131.80 |
| | 0.9 | -80.41 | -11.96 | -17.26 | 6.61 | -85.62 | 5.29 | -68.50 | -104.35 | -178.13 |
| | 1.0 | -104.26 | -14.26 | -22.6 | 0.00 | -98.29 | 0.00 | -78.63 | -141.12 | -219.75 |

From SAP 2000, use for Design Load, Fatigue II Load

Fatigue Limit II - Moment, Left Exterior Girder

| Distance (ft) | M3 (Kip-ft) |
|---|---|
| 0 | 1.8798 |
| 8.99 | 1157.7559 |
| 8.99 | 1134.2931 |
| 17.98 | 1660.5849 |
| 17.98 | 1638.7454 |
| 25.8167 | 2172.229 |
| 25.8167 | 2155.7435 |
| 33.6533 | 2370.6338 |
| 33.6533 | 2356.9296 |
| 41.49 | 2284.6785 |
| 41.49 | 2282.334 |
| 49.3267 | 2348.8299 |
| 49.3267 | 2357.0689 |
| 57.1633 | 2197.7909 |
| 57.1633 | 2208.7625 |
| 65 | 1825.7134 |
| 65 | 1838.5474 |
| 72.8367 | 1556.6282 |
| 72.8367 | 1576.3565 |
| 80.6733 | 1115.9485 |
| 80.6733 | 1137.6404 |
| 88.51 | 671.7782 |
| 88.51 | 676.7947 |
| 98.505 | 334.3067 |
| 98.505 | 337.5556 |
| 108.5 | 322.2581 |
| 108.5 | 21.9685 |
| 116.8333 | 309.8801 |
| 116.8333 | 308.6425 |
| 125.1667 | 569.2243 |
| 125.1667 | 567.3923 |
| 133.5 | 856.7296 |
| 133.5 | 779.3379 |
| 142.49 | 679.0605 |
| 142.49 | 678.9831 |
| 151.48 | 889.6447 |
| 151.48 | 878.2532 |
| 159.0667 | 1458.4696 |
| 159.0667 | 1436.0628 |
| 166.6533 | 1872.6053 |
| 166.6533 | 1853.6662 |
| 174.24 | 2066.1954 |
| 174.24 | 2054.6432 |
| 181.8267 | 2306.2716 |
| 181.8267 | 2300.1807 |
| 189.4133 | 2367.0775 |
| 189.4133 | 2362.0628 |
| 197 | 2224.1617 |
| 197 | 2227.3398 |

Fatigue Limit II - Moment, Left Exterior Girder

| Distance (ft) | M3 (Kip-ft) |
|---|---|
| 204.5867 | 2189.4195 |
| 204.5867 | 2201.0526 |
| 212.1733 | 1963.6808 |
| 212.1733 | 1973.4851 |
| 219.76 | 1571.4286 |
| 219.76 | 1588.2897 |
| 227.3467 | 1246.0658 |
| 227.3467 | 1267.867 |
| 234.9333 | 822.3888 |
| 234.9333 | 834.1356 |
| 242.52 | 476.9742 |
| 242.52 | 484.8859 |
| 250.51 | 330.1077 |
| 250.51 | 329.1254 |
| 258.5 | 458.7072 |
| 258.5 | 34.2448 |
| 266.8333 | 316.3181 |
| 266.8333 | 315.2293 |
| 275.1667 | 570.6644 |
| 275.1667 | 567.3322 |
| 283.5 | 881.3844 |
| 283.5 | 789.6415 |
| 292.49 | 704.3456 |
| 292.49 | 705.1135 |
| 301.48 | 849.0359 |
| 301.48 | 838.5001 |
| 309.0667 | 1479.711 |
| 309.0667 | 1454.8871 |
| 316.6533 | 1973.3523 |
| 316.6533 | 1950.8381 |
| 324.24 | 2265.7302 |
| 324.24 | 2249.786 |
| 331.8267 | 2617.475 |
| 331.8267 | 2604.6324 |
| 339.4133 | 2799.3483 |
| 339.4133 | 2787.1087 |
| 347 | 2784.538 |
| 347 | 2780.7903 |
| 354.5867 | 2923.5931 |
| 354.5867 | 2926.2925 |
| 362.1733 | 2884.6481 |
| 362.1733 | 2886.8763 |
| 369.76 | 2642.1156 |
| 369.76 | 2650.7649 |
| 377.3467 | 2556.477 |
| 377.3467 | 2571.0975 |
| 384.9333 | 2308.3595 |
| 384.9333 | 2322.6694 |
| 392.52 | 1899.1812 |
| 392.52 | 1913.189 |

Fatigue Limit II - Moment, Left Exterior Girder

| Distance (ft) | M3 (Kip-ft) |
|---|---|
| 400.716 | 1573.4298 |
| 400.716 | 1591.5829 |
| 408.912 | 1047.719 |
| 408.912 | 1067.0636 |
| 417.108 | 505.3905 |
| 417.108 | 512.4591 |
| 425.304 | 359.1701 |
| 425.304 | 361.0516 |
| 433.5 | 413.2861 |
| 433.5 | 384.3073 |
| 441.8333 | 551.6566 |
| 441.8333 | 555.104 |
| 450.1667 | 754.0054 |
| 450.1667 | 759.8232 |
| 458.5 | 1018.4733 |
| 458.5 | 69.9598 |
| 468.495 | 1099.2044 |
| 468.495 | 1085.1738 |
| 478.49 | 1786.5048 |
| 478.49 | 1776.7826 |
| 486.3267 | 2414.5365 |
| 486.3267 | 2413.1616 |
| 494.1633 | 2741.0768 |
| 494.1633 | 2736.3922 |
| 502 | 2788.3573 |
| 502 | 2793.1748 |
| 509.8367 | 3037.3406 |
| 509.8367 | 3048.8713 |
| 517.6733 | 3050.3324 |
| 517.6733 | 3062.0046 |
| 525.51 | 2808.19 |
| 525.51 | 2829.6231 |
| 533.3467 | 2784.1084 |
| 533.3467 | 2813.067 |
| 541.1833 | 2466.1206 |
| 541.1833 | 2492.6298 |
| 549.02 | 1864.3143 |
| 549.02 | 1893.8419 |
| 558.01 | 1263.2883 |
| 558.01 | 1287.0091 |
| 567 | 4.1383 |

For SAP2000, use For Design Load
Fatigue II Load

Fatigue Limit II - Shear, Left Exterior Girder

| Distance (ft) | X/L | +V2 (Kip) | -V2 (Kip) | $V_{sr}$ (Kip) | Use Value X/L | Use Value $V_{sr}$ (Kip) |
|---|---|---|---|---|---|---|
| 0.00 | | 3.03 | -158.40 | 161.43 | 0.00 | 161.43 |
| 8.99 | 0.07 | 9.92 | -130.78 | 140.69 | 0.10 | 130.84 |
| 8.99 | | 14.05 | -116.97 | 131.02 | 0.20 | 125.22 |
| 17.98 | 0.13 | 22.56 | -98.43 | 120.98 | 0.30 | 130.52 |
| 17.98 | | 21.37 | -107.32 | 128.69 | 0.40 | 133.12 |
| 25.82 | 0.19 | 29.89 | -95.32 | 125.22 | 0.50 | 143.39 |
| 25.82 | | 39.64 | -83.15 | 122.79 | 0.60 | 160.81 |
| 33.65 | 0.25 | 48.91 | -72.73 | 121.64 | 0.70 | 185.69 |
| 33.65 | | 58.44 | -62.98 | 121.42 | 0.80 | 222.06 |
| 41.49 | 0.31 | 67.46 | -57.94 | 125.41 | | |
| 41.49 | | 55.69 | -74.83 | 130.52 | | |
| 49.33 | 0.37 | 64.05 | -65.30 | 129.35 | | |
| 49.33 | | 74.28 | -55.42 | 129.70 | | |
| 57.16 | 0.43 | 84.40 | -46.95 | 131.35 | | |
| 57.16 | | 97.69 | -38.86 | 136.55 | | |
| 65.00 | 0.49 | 108.36 | -35.02 | 143.39 | | |
| 65.00 | | 94.94 | -44.71 | 139.65 | | |
| 72.84 | 0.55 | 104.87 | -37.00 | 141.87 | | |
| 72.84 | | 117.30 | -29.03 | 146.33 | | |
| 80.67 | 0.60 | 130.68 | -22.15 | 152.83 | | |
| 80.67 | | 145.19 | -15.62 | 160.81 | | |
| 88.51 | 0.66 | 157.68 | -12.61 | 170.29 | | |
| 88.51 | | 157.46 | -12.16 | 169.61 | | |
| 98.51 | 0.74 | 181.42 | -8.84 | 190.26 | | |
| 98.51 | | 195.64 | -6.12 | 201.76 | | |
| 108.50 | 0.81 | 218.95 | -3.11 | 222.06 | | |
| 108.50 | | 2.88 | -49.21 | 52.10 | | |
| 116.83 | 0.88 | 80.93 | -38.98 | 119.92 | | |
| 116.83 | | 90.99 | -35.36 | 126.35 | | |
| 125.17 | 0.94 | 110.54 | -35.21 | 145.75 | | |
| 125.17 | | 119.86 | -35.13 | 154.98 | | |
| 133.50 | 1.00 | 149.27 | -35.11 | 184.38 | | |
| 133.50 | | 11.08 | -203.88 | 214.96 | | |
| 142.49 | 0.06 | 10.89 | -179.25 | 190.14 | | |
| 142.49 | | 10.72 | -166.11 | 176.83 | 0.20 | 142.43 |
| 151.48 | 0.12 | 13.28 | -145.13 | 158.42 | 0.30 | 135.16 |
| 151.48 | | 11.83 | -144.30 | 156.13 | 0.40 | 134.71 |
| 159.07 | 0.17 | 16.77 | -131.16 | 147.92 | 0.50 | 140.98 |
| 159.07 | | 23.93 | -117.72 | 141.65 | 0.60 | 150.63 |
| 166.65 | 0.22 | 31.38 | -105.57 | 136.94 | 0.70 | 181.69 |
| 166.65 | | 39.70 | -93.89 | 133.59 | 0.80 | 219.08 |
| 174.24 | 0.27 | 46.35 | -86.63 | 132.98 | | |
| 174.24 | | 38.59 | -99.61 | 138.20 | | |
| 181.83 | 0.32 | 45.31 | -88.96 | 134.27 | | |
| 181.83 | | 54.14 | -77.99 | 132.13 | | |
| 189.41 | 0.37 | 63.22 | -68.26 | 131.48 | | |
| 189.41 | | 73.75 | -58.99 | 132.74 | | |
| 197.00 | 0.42 | 82.32 | -54.10 | 136.42 | | |
| 197.00 | | 68.40 | -68.27 | 136.67 | | |
| 204.59 | 0.47 | 76.68 | -59.43 | 136.11 | | |

Fatigue Limit II - Shear, Left Exterior Girder

| Distance (ft) | X/L | +V2 (Kip) | -V2 (Kip) | $V_{sr}$ (Kip) | Use Value X/L | Use Value $V_{sr}$ (Kip) |
|---|---|---|---|---|---|---|
| 204.59 | | 87.06 | -50.26 | 137.32 | | |
| 212.17 | 0.52 | 98.01 | -42.18 | 140.19 | | |
| 212.17 | | 110.13 | -34.51 | 144.64 | | |
| 219.76 | 0.58 | 119.95 | -30.77 | 150.72 | | |
| 219.76 | | 108.63 | -37.89 | 146.52 | | |
| 227.35 | 0.63 | 118.88 | -31.07 | 149.95 | | |
| 227.35 | | 131.23 | -23.52 | 154.75 | | |
| 234.93 | 0.68 | 143.88 | -17.01 | 160.89 | | |
| 234.93 | | 157.57 | -15.91 | 173.48 | | |
| 242.52 | 0.73 | 169.60 | -15.94 | 185.55 | | |
| 242.52 | | 173.71 | -16.20 | 189.91 | | |
| 250.51 | 0.78 | 188.20 | -16.24 | 204.44 | | |
| 250.51 | | 203.60 | -16.27 | 219.87 | | |
| 258.50 | 0.83 | 217.43 | -16.29 | 233.72 | | |
| 258.50 | | 2.88 | -49.22 | 52.10 | | |
| 266.83 | 0.89 | 83.06 | -39.05 | 122.12 | | |
| 266.83 | | 92.91 | -38.39 | 131.30 | | |
| 275.17 | 0.94 | 112.15 | -38.36 | 150.50 | | |
| 275.17 | | 121.34 | -38.37 | 159.71 | | |
| 283.50 | 1.00 | 150.67 | -38.70 | 189.37 | | |
| 283.50 | | 9.82 | -213.26 | 223.08 | | |
| 292.49 | 0.06 | 9.35 | -190.22 | 199.57 | | |
| 292.49 | | 9.06 | -177.86 | 186.92 | 0.20 | 155.74 |
| 301.48 | 0.12 | 12.49 | -158.27 | 170.75 | 0.30 | 148.66 |
| 301.48 | | 10.71 | -158.40 | 169.11 | 0.40 | 146.02 |
| 309.07 | 0.17 | 15.20 | -145.96 | 161.15 | 0.50 | 146.11 |
| 309.07 | | 21.88 | -133.15 | 155.03 | 0.60 | 149.98 |
| 316.65 | 0.22 | 28.83 | -121.50 | 150.34 | 0.70 | 156.66 |
| 316.65 | | 36.65 | -110.25 | 146.90 | 0.80 | 163.36 |
| 324.24 | 0.27 | 42.71 | -103.41 | 146.12 | | |
| 324.24 | | 33.95 | -118.39 | 152.34 | | |
| 331.83 | 0.32 | 39.86 | -108.00 | 147.87 | | |
| 331.83 | | 47.86 | -97.12 | 144.98 | | |
| 339.41 | 0.37 | 56.09 | -87.31 | 143.40 | | |
| 339.41 | | 65.66 | -77.80 | 143.46 | | |
| 347.00 | 0.42 | 73.07 | -72.84 | 145.91 | | |
| 347.00 | | 56.34 | -92.23 | 148.57 | | |
| 354.59 | 0.47 | 63.17 | -82.71 | 145.88 | | |
| 354.59 | | 72.12 | -72.60 | 144.72 | | |
| 362.17 | 0.52 | 81.56 | -63.71 | 145.27 | | |
| 362.17 | | 92.11 | -55.39 | 147.50 | | |
| 369.76 | 0.58 | 99.99 | -51.40 | 151.38 | | |
| 369.76 | | 80.63 | -69.53 | 150.15 | | |
| 377.35 | 0.63 | 88.19 | -60.87 | 149.07 | | |
| 377.35 | | 97.87 | -51.93 | 149.80 | | |
| 384.93 | 0.68 | 107.81 | -44.21 | 152.03 | | |
| 384.93 | | 118.79 | -36.79 | 155.58 | | |
| 392.52 | 0.73 | 127.27 | -33.45 | 160.72 | | |
| 392.52 | | 109.90 | -47.84 | 157.74 | | |
| 400.72 | 0.78 | 122.40 | -37.00 | 159.40 | | |

Fatigue Limit II - Shear, Left Exterior Girder

| Distance (ft) | X/L | +V2 (Kip) | -V2 (Kip) | $V_{sr}$ (Kip) | Use Value X/L | Use Value $V_{sr}$ (Kip) |
|---|---|---|---|---|---|---|
| 400.72 | | 130.77 | -30.72 | 161.49 | | |
| 408.91 | 0.84 | 147.17 | -20.14 | 167.31 | | |
| 408.91 | | 157.33 | -15.15 | 172.47 | | |
| 417.11 | 0.89 | 177.52 | -12.92 | 190.44 | | |
| 417.11 | | 189.21 | -12.94 | 202.16 | | |
| 425.30 | 0.95 | 210.59 | -13.02 | 223.61 | | |
| 425.30 | | 221.95 | -13.07 | 235.01 | | |
| 433.50 | 1.00 | 242.98 | -13.24 | 256.22 | | |
| 433.50 | | 32.82 | -141.28 | 174.10 | | |
| 441.83 | 0.06 | 32.79 | -117.03 | 149.82 | 0.20 | 131.71 |
| 441.83 | | 32.81 | -106.07 | 138.87 | 0.30 | 141.75 |
| 450.17 | 0.12 | 33.87 | -86.28 | 120.16 | 0.40 | 139.35 |
| 450.17 | | 42.92 | -78.89 | 121.81 | 0.50 | 141.69 |
| 458.50 | 0.19 | 63.86 | -67.85 | 131.71 | 0.60 | 136.48 |
| 458.50 | | 1.94 | -50.16 | 52.10 | 0.70 | 142.00 |
| 468.50 | 0.26 | 9.11 | -143.92 | 153.04 | 0.80 | 139.22 |
| 468.50 | | 13.33 | -128.24 | 141.57 | 0.90 | 142.97 |
| 478.49 | 0.34 | 22.12 | -108.35 | 130.46 | 1.00 | 178.71 |
| 478.49 | | 22.48 | -121.04 | 143.52 | | |
| 486.33 | 0.40 | 29.98 | -109.37 | 139.35 | | |
| 486.33 | | 38.36 | -97.24 | 135.60 | | |
| 494.16 | 0.45 | 46.34 | -86.48 | 132.81 | | |
| 494.16 | | 54.71 | -76.45 | 131.16 | | |
| 502.00 | 0.51 | 64.53 | -71.42 | 135.95 | | |
| 502.00 | | 48.68 | -93.01 | 141.69 | | |
| 509.84 | 0.57 | 55.33 | -82.39 | 137.72 | | |
| 509.84 | | 63.88 | -70.81 | 134.69 | | |
| 517.67 | 0.63 | 73.19 | -62.15 | 135.34 | | |
| 517.67 | | 84.83 | -53.44 | 138.27 | | |
| 525.51 | 0.69 | 93.14 | -48.86 | 142.00 | | |
| 525.51 | | 70.81 | -63.80 | 134.61 | | |
| 533.35 | 0.75 | 78.28 | -53.41 | 131.68 | | |
| 533.35 | | 88.55 | -44.80 | 133.34 | | |
| 541.18 | 0.81 | 99.28 | -36.51 | 135.80 | | |
| 541.18 | | 111.42 | -27.80 | 139.22 | | |
| 549.02 | 0.87 | 121.41 | -21.96 | 143.36 | | |
| 549.02 | | 109.59 | -22.19 | 131.78 | | |
| 558.01 | 0.93 | 130.11 | -13.34 | 143.45 | | |
| 558.01 | | 144.47 | -9.70 | 154.16 | | |
| 567.00 | 1.00 | 174.46 | -4.25 | 178.71 | | |

Fatigue II. ADDT = 20

$N = (365)(75)(1)(.28) \times 20 = 438{,}000 < N^{th}$

$Z_r = \alpha d^2$ (CA 6.10.10.2-16)

$\alpha = 34.5 - 4.28 \log N$ (AASHTO 6.10.10.2-2)

$= 34.5 - 4.28 \log(438000) = 10.35$

$p \leq \frac{n Z_r}{V_{sr}}$ (AASHTO 6.10.10.1.2.1)

$Z_r = 10.35 (0.875)^2 = 7.93^{kips}$

## Pitch For Shear Connector

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | n= | 4 | | | | | |
| | | $Z_r$= | 4.21 | | | $Z_r$= | 7.93 | |
| | | | 0.0098 | | | | 0.0098 | |
| | | Fatigue 1 - HL-93 Truck For Infinite Life | | | Fatigue 2 - P-9 Truck for finite Life | | | |
| Span | X/L | $V_f$ | $V_{sr}$=0.0098$V_f$ | p=n$Z_r$/$V_{sr}$ | $V_f$ | $V_{sr}$=0.0098$V_f$ | p=n$Z_r$/$V_{sr}$ | |
| | 0 | 78.66 | 0.77 | 21.85 | 161.43 | 1.58 | 20.05 | 18" |
| | 0.1 | 73.78 | 0.72 | 23.29 | 130.84 | 1.28 | 24.74 | |
| | 0.2 | 67.18 | 0.66 | 25.58 | 125.22 | 1.23 | 25.85 | |
| | 0.3 | 64.54 | 0.63 | 26.62 | 130.52 | 1.28 | 24.80 | 24" |
| 1 | 0.4 | 64.16 | 0.63 | 26.78 | 133.12 | 1.30 | 24.31 | |
| | 0.5 | 64.38 | 0.63 | 26.69 | 143.39 | 1.41 | 22.57 | 18" |
| | 0.6 | 66.33 | 0.65 | 25.91 | 160.81 | 1.58 | 20.13 | |
| | 0.7 | 72.93 | 0.71 | 23.56 | 185.69 | 1.82 | 17.43 | 12" |
| | 0.8 | 79.54 | 0.78 | 21.60 | 222.06 | 2.18 | 14.58 | 14" |
| | 0.2 | 68.44 | 0.67 | 25.11 | 142.43 | 1.40 | 22.72 | |
| | 0.3 | 68.44 | 0.67 | 25.11 | 135.16 | 1.32 | 23.95 | |
| | 0.4 | 68.44 | 0.67 | 25.11 | 134.71 | 1.32 | 24.03 | 18" |
| 2 | 0.5 | 68.44 | 0.67 | 25.11 | 140.98 | 1.38 | 22.96 | |
| | 0.6 | 71.55 | 0.70 | 24.02 | 150.63 | 1.48 | 21.49 | |
| | 0.7 | 83.14 | 0.81 | 20.67 | 181.69 | 1.78 | 17.81 | 12" |
| | 0.8 | 94.73 | 0.93 | 18.14 | 219.08 | 2.15 | 14.77 | 14" |
| | 0.2 | 78.94 | 0.77 | 21.77 | 155.74 | 1.53 | 20.78 | |
| | 0.3 | 73.13 | 0.72 | 23.50 | 148.66 | 1.46 | 21.77 | |
| | 0.4 | 73.13 | 0.72 | 23.50 | 146.02 | 1.43 | 22.17 | |
| 3 | 0.5 | 71.86 | 0.70 | 23.91 | 146.11 | 1.43 | 22.15 | 18" |
| | 0.6 | 73.13 | 0.72 | 23.50 | 149.98 | 1.47 | 21.58 | |
| | 0.7 | 73.13 | 0.72 | 23.50 | 156.66 | 1.54 | 20.66 | |
| | 0.8 | 79.30 | 0.78 | 21.67 | 163.36 | 1.60 | 19.81 | |
| | 0.2 | 79.54 | 0.78 | 21.60 | 131.71 | 1.29 | 24.57 | |
| | 0.3 | 72.93 | 0.71 | 23.56 | 141.75 | 1.39 | 22.83 | |
| | 0.4 | 66.33 | 0.65 | 25.91 | 139.35 | 1.37 | 23.23 | |
| 4 | 0.5 | 64.38 | 0.63 | 26.69 | 141.69 | 1.39 | 22.84 | 18" |
| | 0.6 | 64.16 | 0.63 | 26.78 | 136.48 | 1.34 | 23.72 | |
| | 0.7 | 64.54 | 0.63 | 26.62 | 142.00 | 1.39 | 22.79 | |
| | 0.8 | 67.18 | 0.66 | 25.58 | 139.22 | 1.36 | 23.25 | |
| | 0.9 | 73.78 | 0.72 | 23.29 | 142.97 | 1.40 | 22.64 | |
| | 1.00 | 78.63 | 0.77 | 21.85 | 178.71 | 1.75 | 18.11 | |

Based upon this,
provide pitch @ 12", 18" and 24"

Check for strength

For straight bridge

$$n = \frac{P}{Q_n} \quad \text{(AASHTO 6.10.10.4.1-2)}$$

$$P = P_p$$

$$P_p = \text{smaller} \begin{cases} 0.85 f_c' b_s t_s \\ A_s F_y \end{cases} \quad \text{(AASHTO 6.10.10.4.2-2 & 3)}$$

$$\left.\begin{array}{l} 0.85 \times (3.6)(120)(10'') = 3672 \\ 112.5 \times (50) = 5625 \end{array}\right\} \rightarrow 3672^{kips}$$

The factored shear resistance of a 7/8" shear stud

$$Q_n = 0.5 A_{sc} \sqrt{f_c' E_c} \leq A_{sc} F_u$$

(AASHTO 6.10.10.4.3-1)

$$E_c = w^{2/3} 33 \sqrt{f_c'}$$

$$= (150)^{2/3} 33 \sqrt{3600} = 3640^{ksi}$$

$$Q_n = 0.5 \frac{(0.875)^2}{4} \pi \sqrt{(3640)(3.6)} = \underline{\underline{34.4}}^{kips} \leftarrow \text{use}$$

$$A_{sc} F_u = \frac{\pi}{4}(0.875)^2 (60) = 36.1^{kips}$$

$$n = \frac{3672^{kips}}{\phi_{sc} Q_n} = \frac{3675}{0.85(34.4)} = \underline{\underline{126}}$$

satisfy this requirement.

(the number of provided shear connectors, between the point of maximum positive moment and each adjacent of zero moment, shall be greater than 'n' (126)]

Shear Connectors at Point of Contraflexure

$$n_{ac} = \frac{A_s f_{sr}}{Z_r} \quad \text{(AASHTO 6.10.10.3-1)}$$

$f_{sr}$ is fatigue stress range in the slab reinforcement over the interior support under the Fatigue I Load combination for infinite fatigue life.

| Bent | $M_{sr}$ (K-ft) | $S_{NSRV}$ ($in^3$) | $f_{sr}$ (ksi) | $A_s$ ($in^2$) | $Z_r$ (kip) | $n_{ac}$ |
|---|---|---|---|---|---|---|
| 2 | 1135 | 4668 | $\frac{1135 \times 12}{4668} = 2.92$ | 12 | 4.21 | 8.3 |
| 3 | 1367 | 4668 | 3.51 | 12 | 4.21 | 10. |
| 4 | 1135 | 4668 | 2.92 | 12 | 4.21 | 8.3 |

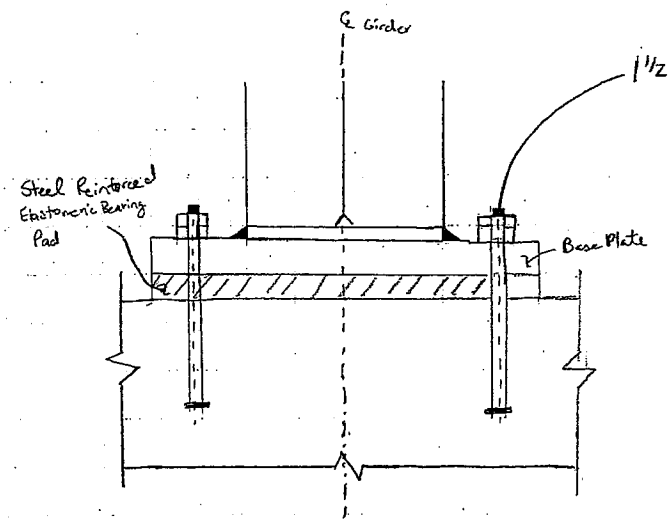
placed within a distance equal to 1/3 of the effective concrete deck width on each side of the point of permanent load contraflexure.

( $n_{ac}$ is additional connectors)

effective concrete deck width ⇒ 14'

$11/3 = 3.67'$ or $5.67/3 = 1.87'$

Checking Capacity of A325 HS Bolts 07-LA-710-17.30




- Bolts are in single shear
- Outside of the shear plane

Shear

Available shear strength from AASHTO 6.13.2.7

eq. 6.13.2.7-1 $R_n = 0.48 A_b F_{ub} N_s$ ✓ where $A_b = (1.5/2)^2 \pi = 1.77 in^2$ ✓

$F_{ub} = 105$ ksi ✓ from 6.4.3.1

$N_s = 1$

$R_n = 0.48 (1.77 in^2)(105 ksi)(1) = 89.21 \times 8$ bolts $= \underline{713.66}$ kips ✓

Longitudinal shear Shear from SAP2000 looking @ girder links (seismic no skew) ✓

| | Shear Pier 2 - (Kips) Long | Trans | Shear - Pier 3 (Kips) Long | Trans OK ✓ | Shear - Pier 4 (Kips) |
|---|---|---|---|---|---|
| South 12 | 311.39 | 361.96 | 228.15 | 304.32 | 235.93 |
| | 41.40 | 974.81 | 16.16 | 824.78 | 20.55 |
| | 525.03 | 902.22 | 429.31 | 758.29 | 408.07 |
| | 1229.91 | 807.56 | 983.32 | 677.89 | 984.04 |
| | 1667.64 | 979.29 | 1353.62 | 822.08 | 1340.92 |
| | 2208.19 | 913.38 | 1768.13 | 766.12 | 1818.47 |
| | 1962.01 | 778.82 | 1578.96 | 653.62 | 1607.19 |
| | 1581.71 | 791.57 | 1272.20 | 644.43 | 1288.93 |
| | 819.45 | 899.27 | 671.89 | 755.49 | 651.89 |
| | 265.96 | 953.56 | 189.35 | 801.99 | 176.48 |
| | 105.58 | 623.27 | 72.17 | 525.21 | 84.51 |
| South 1 | 255.44 | 347.83 | 205.70 | 792.31 | 204.24 |
| Avg. | 914.48 Kips | 777.80 | 732.26 Kips | | 735.10 Kips |

↑ Transverse

% off = 28% 8.9% higher % off = 32 % off = 32%

## Tensile Strength of Steel

From ACI Code 318 D.5.1.2

$N_{sa} = n A_{se,N} f_{uta}$

Steel Strength of Anchor in Tension

where n = number of anchors in tension = 4

$A_{se,N}$ = eff. cross-sectional area of a single anchor

$= \pi r^2 = \pi (0.75)^2 = 1.77 \text{ in}^2$

$f_{uta}$ = smaller of $1.9 f_{ya}$ or 125 ksi

$= 1.9(81 \text{ ksi}) = 153.90 \text{ ksi}$

use 125 ksi ✓

$N_{sa} = 4(1.77)(125)$

$= 885 \text{ K}$

Steel strength of anchor group in tension = 885 kips

## Pullout strength of a single headed stud anchor (begins to fail)

From ACI Code 318 D.5.3.4

$N_p = 8 A_{brg} f_c'$ where $f_c' = 3250$ psi

$A_{brg}$ = bearing area of headed or anchor bolt

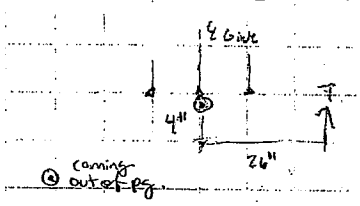
assume anchor bolt is embeded 1 ft.

$A_{brg} = 2\pi r \cdot L = 2\pi(0.75 \text{ in})(12 \text{ in}) = 56.55 \text{ in}^2$

$N_p = 8(56.55 \text{ in}^2)(3250) = 1470.3$ kips per bolt.

$N_{p\,Total} = 4 \text{ bolts} \times 1470.3 = 5{,}881.20$ kips per assembly

## Tensile Force




$T = 4V/26''$

Bearing Pad Ht.

spacing of Bolt

Pier 2: $T = 914.48 \times 4''/26'' = 140.69 \text{ K} < 885 \text{ K} < 1470.3 \text{ K}$

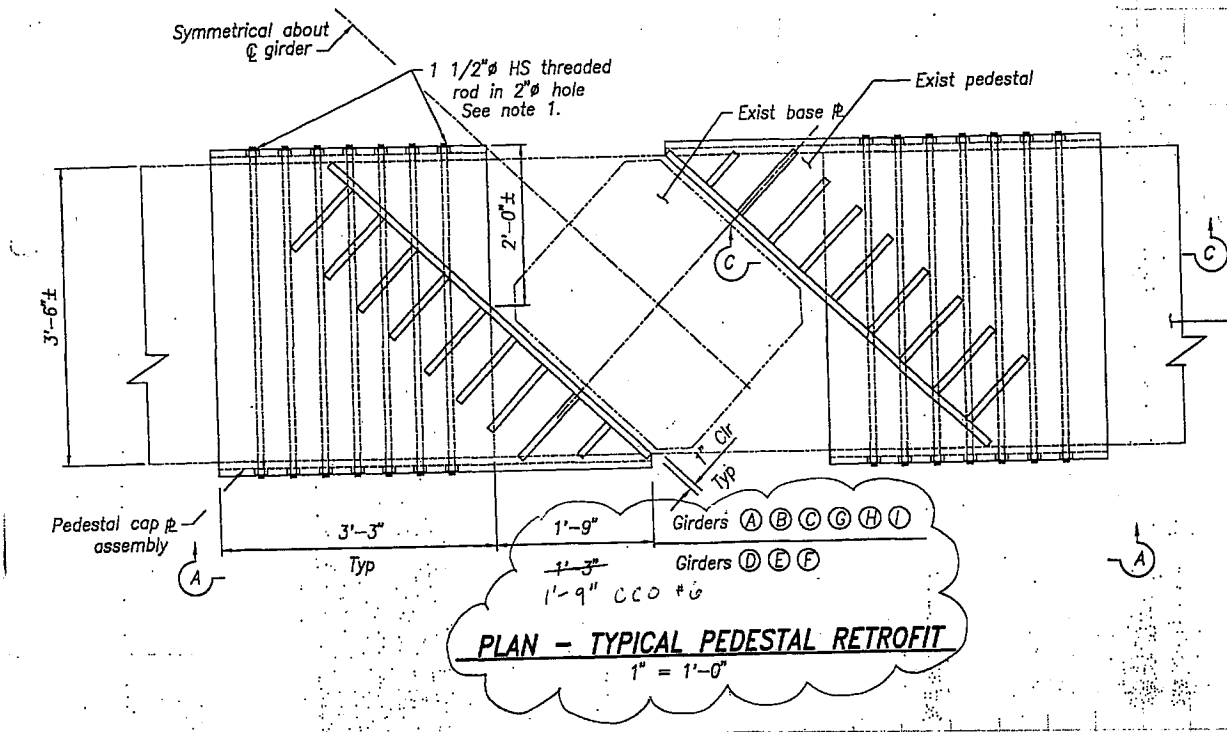
Pier 3: $T = 732.26 \times 4''/26'' = 112.66 \text{ K} < 885 \text{ K}$

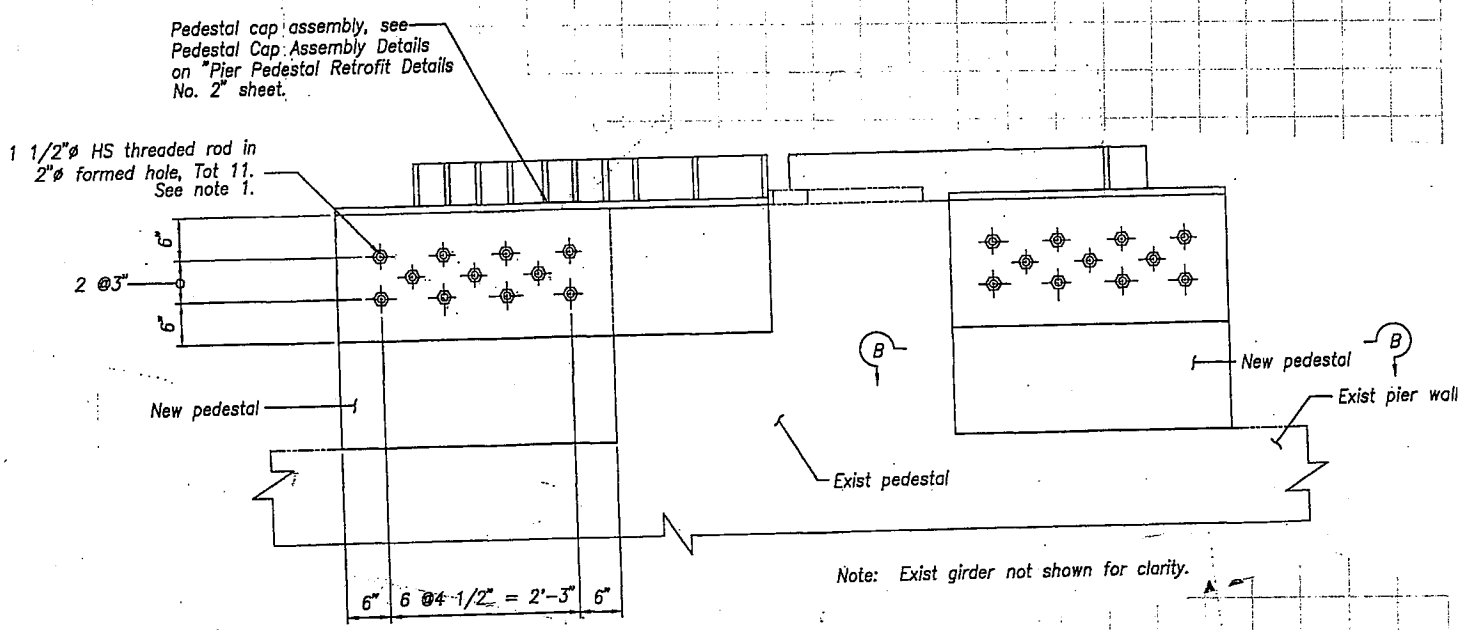
Pier 4: $T = 735.10 \times 4''/26'' = 113.09 \text{ K} < 885 \text{ K}$

FHW LA River Bridge 113

10/5/09 Pier wall stiffner plate capacity

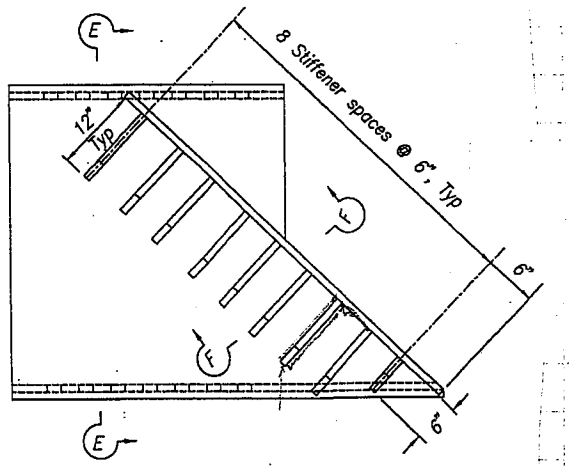
Calculate Pier wall stiffner Plate (retrofitted in '96) capacity.



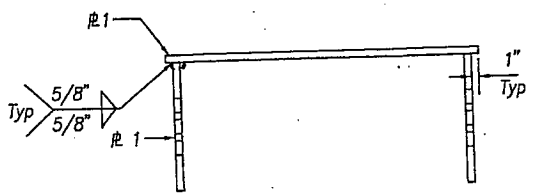



FHW LA River Bridge 114

10/5/09 Pier Wall stiffner Plate capacity.

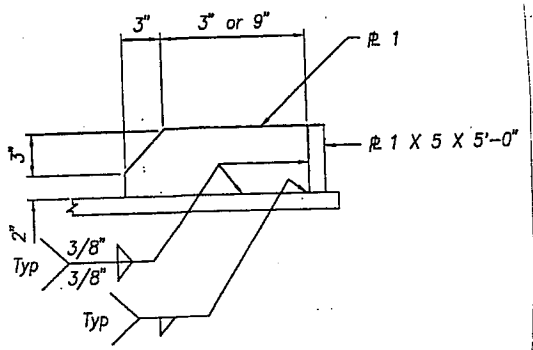



**PLAN – PEDESTAL CAP ASSEMBLY**
1" = 1'-0"




Note: Stiffeners not shown, for clarity.

**SECTION E–E**
1" = 1'-0"




**SECTION F–F**
No Scale

(1) check cover Plate and side plate welding capacity

welding 5/8" x 5'-0", Assume E-70 xx wheel metal

$F_{exx} = 70^{ksi}$

$R_r = 0.6\,\phi_{ee}\,F_{exx} = 0.6 \times 0.8 \times 70 = 33.6^{ksi}$

effective length $L = 60''$

effective thickness $t_e = 0.707 \times 5/8'' = 0.4419$

shear resistance of welds

$V_r = (33.6) \times 60'' \times (0.4419) \times 4$ (sides)

$= 3563^{kips}$

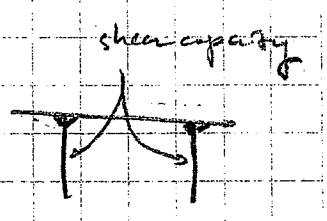
(2) Allowable tensile capacity of plates

Plate thickness $t = 1''$

$P_r = \phi_y F_y A_g$

$= 0.95 \times 36 \times (60'' \times 1'') = 2052^{kips}$

(3) Allowable shear capacity of plates




$R_r = \phi_v R_n = \phi_v (0.58 A_g F_y)$

$= 1.0\,(0.58)\,(36) \times (60 \times 1) = 1253^{kips}$

← Control

From Seismic Run, Max Transverse EQ force = 1000 kips.

PORTLAND BOLT & MANUFACTURING COMPANY
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LIVE CHAT need help?



# ASTM A449

**SCOPE**

ASTM A449 covers **headed bolts**, **rods**, and **anchor bolts** in diameters ranging from 1/4" through 3" inclusive. It is a medium strength bolt manufactured from a medium carbon or alloy steel that develops its mechanical values through a heat treating process. It is intended for general engineering applications.

**QUICK LINKS**

- Thread Pitch Chart
- Ask the Expert
- Bolt Torque Chart

ASTM A449 is virtually identical in chemistry and strength to **ASTM A325** and SAE J429 grade 5. However, A449 is more flexible in the sense that it **covers a larger diameter range** and is not restricted by a specific configuration.

**TYPES**

**TYPE 1** Plain carbon steel, carbon boron steel, alloy steel, or alloy boron steel.

**TYPE 2** Withdrawn 2003

**TYPE 3** Weathering steel.

**MECHANICAL PROPERTIES**

| Size | Tensile, ksi | Yield, ksi | Elong. %, min | RA %, min |
|---|---|---|---|---|
| 1/4 - 1 | 120 min | 92 min | 14 | 35 |
| 1-1/8 - 1-1/2 | 105 min | 81 min | 14 | 35 |
| 1-5/8 - 3 | 90 min | 58 min | 14 | 35 |

**RECOMMENDED NUTS AND WASHERS**

| Nuts | | | Washers |
|---|---|---|---|
| Plain | | Galvanized | |
| 1/4 - 1-1/2 | 1-5/8 - 3 | 1/4 - 3 | |
| A563B Hex | A563A Heavy Hex | A563DH Heavy Hex | F436 |

Note: Nuts of other grades having proof load stresses greater than the specified grade are suitable. The **ASTM A563 Nut Compatibility Chart** has a complete list of specifications.



Portland Bolt & Manufacturing Co.
3441 NW Guam Street, Portland, OR 97210
© 1996-2009

(4) 1½" φ HS threaded rod capacity.

ASTM A449 Rods, $F_u = 105$ ksi

$A_b = \frac{\pi D^2}{4} = \frac{\pi (1.5)^2}{4} = 1.767$

AISC, Table J3.2 nominal Tensile stress

$F_{nt} = 0.75 F_u = 0.75 \times 105 = 78.75$ ksi

each rod has $P = 1.767 \times 78.75 = 139$ kips

11 Rods on one side.

$11 \times 139 = 1530$ kips

Welds from bottom of flange to Pedestal base plate

$R_r = 0.6 \phi_{e2} F_{exx}$

$= (0.6) \times (0.8) \times (70^{ksi}) = 33.6^{ksi}$

The length of the welds

Provide 3/4" fillet at Abutment

$(2)(0.707) \times (3/4") \times 18" \times 33.6 = 641^{kips}$

Provide 3/4" fillet at Pier

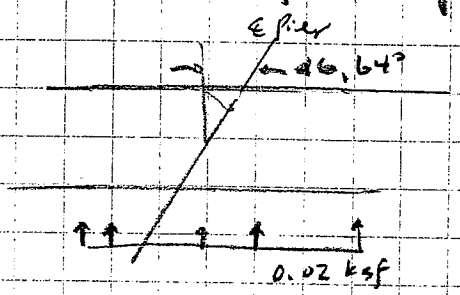
$(2)(0.707)(3/4") \times 52" \times 33.6 = 1140^{kips} > 914^{kips}$

Damage Load

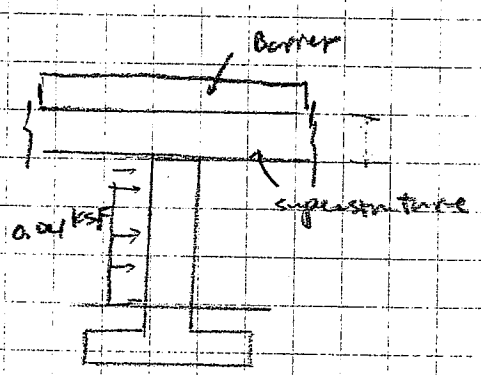
Seismic 2

## Loads acting on the Pier Bottom (Longitudinal Direction)

1. Wind load from superstructure




plan




$0.02 \times 150'$ (long) $\times 10'$ (deep) $= 30^{kips}$ (↑)

resolve into Pier wall

Force acting on Top of Pier (longitudinal)

$30 \times \sin(46.64) = 21.8$ ←

Force acting Top of Pier (Transverse)

$30 \times \cos(46.64) = 20.6^{kips}$

Distribution

$21.8 \times 1/12 = 2^{kips}$ (partial)

$2 \times 36 = 72^{k\text{-}ft}$

2. Wind load from substructure

In longitudinal Direction.

$0.04 \times 36$ (Pier height) $\times 7.5' = 11^{kip}$ ( 5.5 Top / 5.5 Bottom )

Moment estimated.

$5.5 \times 36/2 = 100^{k\text{-}ft}$

Transverse Direction Designed as shear wall

Strem Force

Assume HWL = 99.4

Bottom of Pier wall 74.10 Footing Depth = 3.0'

108 − 74 = 34 ft (water submerged height)

weight of water, acting on top of ftg

62.4 #/ft³ × 34 ft × (3.75' × 7.5') × 2 = 120 kips

Dead load From SAP 2000, Bottom of Pier wall

Mlong = 1325 k-ft
Paxial = 8035
} DC Load case

This is for 226' long Pier wall

proportion it.

Mlong = 1325 × 7.5/226 = 44 k-ft
Paxial 8035 × 7.5/226 = 267 kips
} DC case

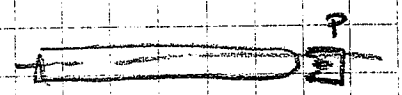
For DW case

Mlong = 1038 × 7.5/226 = 35 k-ft
Paxial = 754 × 7.5/226 = 25
} DW case

Stream Pressure AASHTO 3.7.3

$p = \frac{C_D V^2}{1000}$ $C_D = 0.7$ $V = 15$ ft/s

(ksf)




$p = \frac{0.7 \times 15^2}{1000} = 0.16$ ksf

Water submerged face

$108 - 77.10 = 31$ ft

stream force =

$0.16$ ksf $\times 31 \times 3.5' = 18$ kips

Design the Transverse Direction

Braking force AASHTO 3.6.4

25% of the weight of Design Tadem of Truck axel load

$0.25 \times (32) = 8$ kips ← use

5% of the design Truck plus lane load or
design Tadem plus lane load

design Truck $32 + 32 + 8 = 72$ kips

lane load $0.64$ k/ft $\times 44' = 28$

$0.05(72 + 28) = 5$ kips

Acting at roadway level

the moment to the top of the Pier

$= 5 \times 10' = 50$ k-ft.

122

WinYield
Pier 3.
For all service loads
check

```
*************************************************************************
*                          WinYield                                     *
*                     LRFD / LFD Specification                          *
*                       Column Design Program                           *
*              California Department of Transportation                  *
*                -- For Caltrans In House Use Only --                   *
*          Copyright (c) Caltrans 2007, All Rights Reserved             *
*                           3.0.10                                      *
*                          03.30.10                                     *
*************************************************************************
```

| | | |
|---|---|---|
| Column Type | = 3 | (Rectangular W/Fillets) |
| Concrete Loops | = 1 | |
| Total No. of Concrete Coord. | = 8 | |
| Steel Rebar Pattern | = 5 | (Rows) |
| Number of Steel Rebar Rows | = 2 | |
| Total No. of Steel Rebars | = 14 | |
| Plot Type | = 0 | (No Plot) |
| Design Type | = 2 | (Check) |
| Design Specification | = CLRFD | |

CLRFD = AASHTO LRFD Bridge Design Specifications, Third Edition 2004,
with 2005 & 2006 Interims, as Ammended by Caltrans v3.06.01

Percent Steel Limits = (0.30 % Min. 4.00 % Max.)

```
*************
* Bent Data *
*************************************************************************
```

| | | |
|---|---|---|
| Number of Columns in Bent | = | 1 |
| Out to Out Distance (Diameter) of Spiral | = | 0.00 in |
| Distance from Top Column Plastic Hinge to Center of Gravity of Superstructure | = | 0.00 Ft |
| Center to Center Spacing of Columns | = | 0.00 Ft |

```
***********************
* Material Properties *
*************************************************************************
```

| | | | |
|---|---|---|---|
| Specified Concrete Compressive Strength | - f'c | = | 4000.0 Psi |
| Youngs Modulus for Concrete | - Ec | = | 3604996.5 Psi |
| Youngs Modulus for Steel Rebars | - Es | = | 29000000.0 Psi |
| Concrete Ultimate Compressive Strain | - Eo | = | 0.00300 |
| Rebar Specified Minimum Yield Strength | - fy | = | 60000.0 Psi |

123

Rectangular Cross-Section (With Fillets) (in)

| Hx | Hy | Fx | Fy |
|---|---|---|---|
| 90.00 | 42.00 | 0.00 | 0.00 |

Concrete Coordinates (in)

| Coord | X | Y |
|---|---|---|
| 1 | 45.00 | 21.00 |
| 2 | -45.00 | 21.00 |
| 3 | -45.00 | -21.00 |
| 4 | 45.00 | -21.00 |

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Row Pattern

| X1 (in) | Y1 (in) | X2 (in) | Y2 (in) | Area (in^2) | Bars | Row |
|---|---|---|---|---|---|---|
| -42.00 | 19.00 | 42.00 | 19.00 | 1.27 | 7 | 1 |
| -42.00 | -19.00 | 42.00 | -19.00 | 1.27 | 7 | 2 |

Steel Rebar Coordinates (in)

| Coord | X | Y |
|---|---|---|
| 1 | -42.00 | 19.00 |
| 2 | -28.00 | 19.00 |
| 3 | -14.00 | 19.00 |
| 4 | 0.00 | 19.00 |
| 5 | 14.00 | 19.00 |
| 6 | 28.00 | 19.00 |
| 7 | 42.00 | 19.00 |
| 8 | -42.00 | -19.00 |
| 9 | -28.00 | -19.00 |
| 10 | -14.00 | -19.00 |
| 11 | 0.00 | -19.00 |
| 12 | 14.00 | -19.00 |
| 13 | 28.00 | -19.00 |
| 14 | 42.00 | -19.00 |

The Main Longitudinal Steel is Assumed to be # 10 Bars.

125

```
**************************
* Initial Reference Data *
**************************************************************************
Po = .85f'c(Ag-As) + As fy

Total Area of Section              Ag =     26.25 Ft^2
Nominal Axial Load Strength
at Zero Eccentricity               Po =  13858.35 Kip
Total Reinforcement Area           As =     17.78 in^2
Percent Steel                         =      0.47 %

Concrete Moment of Inertia about Y-Axis  Iyc =  123.0469 Ft^4
Concrete Moment of Inertia about X-Axis  Ixc =   26.7969 Ft^4

Steel    Moment of Inertia about Y-Axis  Iys =    0.6722 Ft^4
Steel    Moment of Inertia about X-Axis  Ixs =    0.3095 Ft^4


*******************************
* Column Load Data (Kip,K-Ft) *
***************************************************************
```

Load Name:
Footing Data File: YES
Type Footing: PILE

| Column Effective Length Factors Ky (Tran) | Kx (Long) | Percent Impact H-Load | Percent Impact P-Load | Column Length (Ft) | Steel 1=Tie 0=Spiral | Location 1=Top 0=Bottom | Ductility Factor (Z) |
|---|---|---|---|---|---|---|---|
| 2.10 | 1.00 | 25.0 | 33.0 | 30.39 | 1 | 0 | 1.00 |

Column End Conditions (TopY TopX BotY BotX) (1=Fix 0=Pin) 0011

LRFD Seismic Zone = 3 or 4

126

```
************************************************
* LRFD Limit State Loads - Unfactored (Kip,K-Ft) *
****************************************************************************
```

Note: Loads are Combined According To Their Sign.

| | Case-1 Maximum Transverse | | | | |
|---|---|---|---|---|---|
| | P-Truck PLL+IM | H-Truck HLL+IM | Lane Load LL | Centrifugal PC-Truck | HC-Truck |
| My-Tran | 0. | 0. | 0. | 0. | 0. |
| Mx-Long | 128. | 35. | 0. | 0. | 0. |
| P-Axial | 3. | 1. | 0. | 0. | 0. |

| | Case-2 Maximum Longitudinal | | | | |
|---|---|---|---|---|---|
| | P-Truck PLL+IM | H-Truck HLL+IM | Lane Load LL | Centrifugal PC-Truck | HC-Truck |
| My-Tran | 0. | 0. | 0. | 0. | 0. |
| Mx-Long | 228. | 136. | 0. | 0. | 0. |
| P-Axial | 2. | 1. | 0. | 0. | 0. |

| | Case-3 Maximum Axial | | | | |
|---|---|---|---|---|---|
| | P-Truck PLL+IM | H-Truck HLL+IM | Lane Load LL | Centrifugal PC-Truck | HC-Truck |
| My-Tran | 0. | 0. | 0. | 0. | 0. |
| Mx-Long | 46. | 32. | 0. | 0. | 0. |
| P-Axial | 3. | 2. | 0. | 0. | 0. |

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```
*************************************************
* LRFD Limit State Loads - Unfactored (Kip,K-Ft) *
*************************************************
```

Note: Loads are Combined According To Their Sign.

| | DC | DW | PS | PL | BR |
|---|---|---|---|---|---|
| | Dead Load | Added Dead Load | Prestress Sec Mom | Pedestrian Live Load | Braking Force |
| My-Tran | 0. | 0. | 0. | 0. | 0. |
| Mx-Long | 44. | 35. | 0. | 0. | 50. |
| P-Axial | 257. | 25. | 0. | 0. | 0. |

| | WA | WS-Wind Struc Horiz | | WL-Wind on Live Load | |
|---|---|---|---|---|---|
| | Stream Pressure | Case-1 Max Tran | Case-2 Max Long | Case-1 Max Tran | Case-2 Max Long |
| My-Tran | 0. | 0. | 0. | 0. | 0. |
| Mx-Long | 577. | 0. | 72. | 0. | 100. |
| P-Axial | 0. | 0. | 0. | 0. | 0. |

| | WV | FR | CR | SH | TU |
|---|---|---|---|---|---|
| | Wind Struc Vertical | Friction | Creep | Shrinkge | Temp Uniform |
| My-Tran | 0. | 0. | 0. | 0. | 0. |
| Mx-Long | 0. | 0. | 0. | 0. | 45. |
| P-Axial | 0. | 0. | 0. | 0. | 0. |

| | TG | SE | EQ | EQ |
|---|---|---|---|---|
| | Temp Gradient | Settlement | Case-1 Max Tran | Case-2 Max Long |
| My-Tran | 0. | 0. | 0. | 0. |
| Mx-Long | 0. | 0. | 0. | 0. |
| P-Axial | 0. | 0. | 0. | 0. |
| Pro Spc Factors | 0.00 | 1.00 | 0.00 | |

(EQ Load Factor: For Live Load,PL,BR)

| | IC Ice Collision | | LRFD |
|---|---|---|---|
| | Case-1 Max Tran | Case-2 Max Long | Arbritrary Load |
| My-Tran | 0. | 0. | 0. |
| Mx-Long | 0. | 0. | 0. |
| P-Axial | 0. | 0. | 0. |

Type of Superstructure = Non-Segmental
Column Moment of Inertia = I-gross

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```
****************************************
* LRFD Load Factors for Column Design *
****************************************************************************
```

(LRFD Tables 3.4.1-1, 3.4.1-2)

Note: Loads are Combined According To Their Sign.

| Limit State | :---DC--: Max | Min | :---DW--: Max | Min | :---PS--: Max | Min | :---CR--: Max | Min | :---SH--: Max | Min |
|---|---|---|---|---|---|---|---|---|---|---|
| Str-I | 1.25 | 0.90 | 1.50 | 0.65 | 0.50 | 0.50 | 0.50 | 0.50 | 0.50 | 0.50 |
| Str-II | 1.25 | 0.90 | 1.50 | 0.65 | 0.50 | 0.50 | 0.50 | 0.50 | 0.50 | 0.50 |
| Str-III | 1.25 | 0.90 | 1.50 | 0.65 | 0.50 | 0.50 | 0.50 | 0.50 | 0.50 | 0.50 |
| Str-IV | 1.50 | 0.90 | 1.50 | 0.65 | 0.50 | 0.50 | 1.50 | 0.90 | 1.50 | 0.90 |
| Str-V | 1.25 | 0.90 | 1.50 | 0.65 | 0.50 | 0.50 | 0.50 | 0.50 | 0.50 | 0.50 |
| Ext-I | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Ext-II | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Ser-I | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |

| Limit State | PLL Trck | HLL Trck | HLL Lane | P-CE Cent | H-CE Cent | PL | BR |
|---|---|---|---|---|---|---|---|
| Str-I | 0.00 | 1.75 | 1.75 | 0.00 | 1.75 | 1.75 | 1.75 |
| Str-II | 1.35 | 0.00 | 0.00 | 1.35 | 0.00 | 0.00 | 0.00 |
| Str-III | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Str-IV | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Str-V | 0.00 | 1.35 | 1.35 | 0.00 | 1.35 | 1.35 | 1.35 |
| Ext-I | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Ext-II | 0.00 | 0.50 | 0.50 | 0.00 | 0.50 | 0.50 | 0.50 |
| Ser-I | 0.00 | 1.00 | 1.00 | 0.00 | 1.00 | 1.00 | 1.00 |

| Limit State | WA | WS | WL | WV | FR | TU | TG | SE | EQ | IC | CT | CV |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Str-I | 1.00 | 0.00 | 0.00 | 0.00 | 1.00 | 0.50 | 0.00 | 1.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Str-II | 1.00 | 0.00 | 0.00 | 0.00 | 1.00 | 0.50 | 0.00 | 1.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Str-III | 1.00 | 1.40 | 0.00 | 1.40 | 1.00 | 0.50 | 0.00 | 1.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Str-IV | 1.00 | 0.00 | 0.00 | 0.00 | 1.00 | 0.50 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Str-V | 1.00 | 0.40 | 1.00 | 0.00 | 1.00 | 0.50 | 0.00 | 1.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Ext-I | 1.00 | 0.00 | 0.00 | 0.00 | 1.00 | 0.00 | 0.00 | 0.00 | 1.00 | 0.00 | 0.00 | 0.00 |
| Ext-II | 1.00 | 0.00 | 0.00 | 0.00 | 1.00 | 0.00 | 0.00 | 0.00 | 0.00 | 1.00 | 1.00 | 1.00 |
| Ser-I | 1.00 | 0.30 | 1.00 | 0.00 | 1.00 | 1.00 | 0.50 | 1.00 | 0.00 | 0.00 | 0.00 | 0.00 |

```
**********************
* Load Designations *
**************************************************************************
```

(LRFD 3.3.2)

Permanent Loads
---

DC = Dead Load of Structural Components and Nonstructural Attachments (Dead Load)
DW = Dead Load of Wearing Surfaces and Utilities (Added Dead Load).
PS = Secondary Forces from Post-Tensioning.
CR = Creep.
SH = Shrinkage.

Transient Loads
---

PLL = Permit Live Load + IM
HLL = H-Design Truck/Tandem Live Load + IM.
LL = H-Design Lane Live Load (No IM).
P-CE = Centrifugal Permit Live Load (No IM).
H-CE = Centrifugal H-Design Truck/Tandem Live Load (No IM).
PL = Pedestrian Live Load. (No IM)
BR = Vehicular Braking Force (No IM).
WA = Water Load and Stream Pressure.
WS = Wind Load on Structure - Horizontal.
WL = Wind on Live Load (No IM).
WV = Wind Load on Structure - Vertical.
FR = Friction.
TU = Uniform Temperature.
TG = Temperature Gradient.
SE = Settlement Live Load.
EQ = Earthquake.
IC = Ice Collision.
CT = Vehicular (Truck or Train) Collision Force.
CV = Vessel Collision Force.
IM = Vehicular Dynamic Load Allowance (Percent Impact).

Note: EQ, IC, CT, CV are Applied One at a Time.

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*************************************************************
* Controlling Load Factor Combinations for Permanent Loads *
*************************************************************
```

1 = Max Load Factor Used
2 = Min Load Factor Used

Bd = Ratio Max Factored Permanent Moments to Max Factored Total Moment

| Limit State | Load Case | DC | DW | PS | CR | SH | Load Factors DC | DW | PS | CR | SH | Bd Tran | Bd Long |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Str-I | 1 | 1 | 1 | 1 | 1 | 1 | 1.25 | 1.50 | 0.50 | 0.50 | 0.50 | 0.00 | 0.13 |
| Str-I | 2 | 1 | 1 | 1 | 1 | 1 | 1.25 | 1.50 | 0.50 | 0.50 | 0.50 | 0.00 | 0.10 |
| Str-I | 3 | 1 | 1 | 1 | 1 | 1 | 1.25 | 1.50 | 0.50 | 0.50 | 0.50 | 0.00 | 0.13 |
| Str-I | 1a | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.50 | 0.50 | 0.00 | 0.08 |
| Str-I | 2a | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.50 | 0.50 | 0.00 | 0.06 |
| Str-I | 3a | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.50 | 0.50 | 0.00 | 0.08 |
| Str-II | 1 | 1 | 1 | 1 | 1 | 1 | 1.25 | 1.50 | 0.50 | 0.50 | 0.50 | 0.00 | 0.12 |
| Str-II | 2 | 1 | 1 | 1 | 1 | 1 | 1.25 | 1.50 | 0.50 | 0.50 | 0.50 | 0.00 | 0.11 |
| Str-II | 3 | 1 | 1 | 1 | 1 | 1 | 1.25 | 1.50 | 0.50 | 0.50 | 0.50 | 0.00 | 0.14 |
| Str-II | 1a | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.50 | 0.50 | 0.00 | 0.07 |
| Str-II | 2a | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.50 | 0.50 | 0.00 | 0.06 |
| Str-II | 3a | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.50 | 0.50 | 0.00 | 0.09 |
| Str-III | 1 | 1 | 1 | 1 | 1 | 1 | 1.25 | 1.50 | 0.50 | 0.50 | 0.50 | 0.00 | 0.15 |
| Str-III | 2 | 1 | 1 | 1 | 1 | 1 | 1.25 | 1.50 | 0.50 | 0.50 | 0.50 | 0.00 | 0.13 |
| Str-III | 3 | 1 | 1 | 1 | 1 | 1 | 1.25 | 1.50 | 0.50 | 0.50 | 0.50 | 0.00 | 0.13 |
| Str-III | 1a | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.50 | 0.50 | 0.00 | 0.09 |
| Str-III | 2a | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.50 | 0.50 | 0.00 | 0.08 |
| Str-III | 3a | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.50 | 0.50 | 0.00 | 0.08 |
| Str-IV | 1 | 1 | 1 | 1 | 1 | 1 | 1.50 | 1.50 | 0.50 | 1.50 | 1.50 | 0.00 | 0.17 |
| Str-IV | 2 | 1 | 1 | 1 | 1 | 1 | 1.50 | 1.50 | 0.50 | 1.50 | 1.50 | 0.00 | 0.17 |
| Str-IV | 3 | 1 | 1 | 1 | 1 | 1 | 1.50 | 1.50 | 0.50 | 1.50 | 1.50 | 0.00 | 0.17 |
| Str-IV | 1a | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.90 | 0.90 | 0.00 | 0.09 |
| Str-IV | 2a | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.90 | 0.90 | 0.00 | 0.09 |
| Str-IV | 3a | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.90 | 0.90 | 0.00 | 0.09 |
| Str-V | 1 | 1 | 1 | 1 | 1 | 1 | 1.25 | 1.50 | 0.50 | 0.50 | 0.50 | 0.00 | 0.13 |
| Str-V | 2 | 1 | 1 | 1 | 1 | 1 | 1.25 | 1.50 | 0.50 | 0.50 | 0.50 | 0.00 | 0.10 |
| Str-V | 3 | 1 | 1 | 1 | 1 | 1 | 1.25 | 1.50 | 0.50 | 0.50 | 0.50 | 0.00 | 0.11 |
| Str-V | 1a | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.50 | 0.50 | 0.00 | 0.08 |
| Str-V | 2a | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.50 | 0.50 | 0.00 | 0.06 |
| Str-V | 3a | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.50 | 0.50 | 0.00 | 0.07 |
| Ext-I | 1 | 1 | 1 | 1 | 1 | 1 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 0.00 | 0.12 |
| Ext-I | 2 | 1 | 1 | 1 | 1 | 1 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 0.00 | 0.12 |
| Ext-I | 3 | 1 | 1 | 1 | 1 | 1 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 0.00 | 0.12 |
| Ext-I | 1a | 2 | 2 | 2 | 2 | 2 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 0.00 | 0.12 |
| Ext-I | 2a | 2 | 2 | 2 | 2 | 2 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 0.00 | 0.12 |
| Ext-I | 3a | 2 | 2 | 2 | 2 | 2 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 0.00 | 0.12 |
| Ext-II | 1 | 1 | 1 | 1 | 1 | 1 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 0.00 | 0.11 |
| Ext-II | 2 | 1 | 1 | 1 | 1 | 1 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 0.00 | 0.11 |
| Ext-II | 3 | 1 | 1 | 1 | 1 | 1 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 0.00 | 0.11 |
| Ext-II | 1a | 2 | 2 | 2 | 2 | 2 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 0.00 | 0.11 |
| Ext-II | 2a | 2 | 2 | 2 | 2 | 2 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 0.00 | 0.11 |
| Ext-II | 3a | 2 | 2 | 2 | 2 | 2 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 0.00 | 0.11 |

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**********************************************
* Moment Magnification and Buckling Equations *
**********************************************
(LRFD 4.5.3.2.2; 5.7.4.3)

Note: Column Assumed to be Unbraced Against Sidesway.

Mu  = Mag x Factored Moment          (LRFD 4.5.3.2.2b-1)

              1.0
Mag = ------------------             (LRFD 4.5.3.2.2b-4)
       1 - (Pu /.75 Pc)

       Pi^2 EI
Pc  = ---------                      (LRFD 4.5.3.2.2b-5)
       (KL)^2

Approximate EI for Cracked Section is Larger of:

EI    = (Ec Ic / 5 + Es Is) / (1+Bd) (LRFD 5.7.4.3-1)

EI    = (Ec Ic / 2.5) / (1+Bd)       (LRFD 5.7.4.3-2)

Magy  = Moment Magnifcation Factor about Y-Axis
Magx  = Moment Magnifcation Factor about X-Axis
Pcy   = Critical Buckling Load about Y-Axis
Pcx   = Critical Buckling Load about X-Axis

Ky    = Effective Length Factor about Y-Axis    =     2.10
Kx    = Effective Length Factor about X-Axis    =     1.00
KyL/r = Slenderness Ratio about Y-Axis          =    29.48
KxL/r = Slenderness Ratio about X-Axis          =    30.08

Iys   = Steel    Moment of Inertia about Y-Axis =     0.6722 Ft^4
Ixs   = Steel    Moment of Inertia about X-Axis =     0.3095 Ft^4
Iyc   = Concrete Moment of Inertia about Y-Axis =   123.0469 Ft^4
Ixc   = Concrete Moment of Inertia about X-Axis =    26.7969 Ft^4

Pn = Nomial Axial Resistance at Unintended Eccentricity
Pn = .80 (.85f'c(Ag-As) + As fy)  Tied Column

Pn =     11087.  Kip

Pr = Factored Axial Resistance
Pr = Phi x Pn

Pu = Factored Axial Force
Pu <= Pr
```

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*************************************************
* Moment Magnification and Buckling Calculations *
*************************************************
```

Note: Axial Load Pu is Calculated at Column Top for Buckling Calculations

| Limit State | Ca | Moment :-Magnification-: Tran Magy | Long Magx | Comb Mag | Approximate EI For :---Cracked Section---: EIy (K-Ft^2) | EIx (K-Ft^2) | Critical :---Buckling---: Pcy (Kip) | Pcx (Kip) | Axial Load Pu (Kip) |
|---|---|---|---|---|---|---|---|---|---|
| Str-I | 1 | 1.005 | 1.005 | 1.005 | 25550412. | 4943327. | 61915. | 52827. | 211. |
| Str-I | 2 | 1.005 | 1.005 | 1.005 | 25550412. | 5039607. | 61915. | 53856. | 211. |
| Str-I | 3 | 1.005 | 1.005 | 1.005 | 25550412. | 4939924. | 61915. | 52791. | 213. |
| Str-I | 1a | 1.002 | 1.002 | 1.002 | 25550412. | 5166884. | 61915. | 55216. | 100. |
| Str-I | 2a | 1.002 | 1.002 | 1.002 | 25550412. | 5233803. | 61915. | 55931. | 100. |
| Str-I | 3a | 1.002 | 1.002 | 1.002 | 25550412. | 5164479. | 61915. | 55191. | 101. |
| Str-II | 1 | 1.005 | 1.005 | 1.005 | 25550412. | 4958454. | 61915. | 52989. | 213. |
| Str-II | 2 | 1.005 | 1.005 | 1.005 | 25550412. | 5031332. | 61915. | 53768. | 212. |
| Str-II | 3 | 1.005 | 1.005 | 1.005 | 25550412. | 4881944. | 61915. | 52171. | 213. |
| Str-II | 1a | 1.002 | 1.002 | 1.002 | 25550412. | 5177540. | 61915. | 55330. | 102. |
| Str-II | 2a | 1.002 | 1.002 | 1.002 | 25550412. | 5228135. | 61915. | 55871. | 101. |
| Str-II | 3a | 1.002 | 1.002 | 1.002 | 25550412. | 5123087. | 61915. | 54748. | 102. |
| Str-III | 1 | 1.005 | 1.005 | 1.005 | 25550412. | 4829918. | 61915. | 51615. | 209. |
| Str-III | 2 | 1.005 | 1.005 | 1.005 | 25550412. | 4910796. | 61915. | 52480. | 209. |
| Str-III | 3 | 1.005 | 1.005 | 1.005 | 25550412. | 4910796. | 61915. | 52480. | 209. |
| Str-III | 1a | 1.002 | 1.002 | 1.002 | 25550412. | 5085252. | 61915. | 54344. | 98. |
| Str-III | 2a | 1.002 | 1.002 | 1.002 | 25550412. | 5143785. | 61915. | 54969. | 98. |
| Str-III | 3a | 1.002 | 1.002 | 1.002 | 25550412. | 5143785. | 61915. | 54969. | 98. |
| Str-IV | 1 | 1.005 | 1.006 | 1.006 | 25550412. | 4776062. | 61915. | 51040. | 244. |
| Str-IV | 2 | 1.005 | 1.006 | 1.006 | 25550412. | 4776062. | 61915. | 51040. | 244. |
| Str-IV | 3 | 1.005 | 1.006 | 1.006 | 25550412. | 4776062. | 61915. | 51040. | 244. |
| Str-IV | 1a | 1.001 | 1.002 | 1.002 | 25550412. | 5085252. | 61915. | 54344. | 68. |
| Str-IV | 2a | 1.001 | 1.002 | 1.002 | 25550412. | 5085252. | 61915. | 54344. | 68. |
| Str-IV | 3a | 1.001 | 1.002 | 1.002 | 25550412. | 5085252. | 61915. | 54344. | 68. |
| Str-V | 1 | 1.005 | 1.005 | 1.005 | 25550412. | 4920606. | 61915. | 52584. | 211. |
| Str-V | 2 | 1.005 | 1.005 | 1.005 | 25550412. | 5063505. | 61915. | 54112. | 211. |
| Str-V | 3 | 1.005 | 1.005 | 1.005 | 25550412. | 4996794. | 61915. | 53399. | 212. |
| Str-V | 1a | 1.002 | 1.002 | 1.002 | 25550412. | 5150777. | 61915. | 55044. | 99. |
| Str-V | 2a | 1.002 | 1.002 | 1.002 | 25550412. | 5250088. | 61915. | 56105. | 99. |
| Str-V | 3a | 1.002 | 1.002 | 1.002 | 25550412. | 5204309. | 61915. | 55616. | 101. |
| Ext-I | 1 | 1.000 | 1.000 | 1.000 | 25550412. | 4966243. | 61915. | 53072. | 162. |
| Ext-I | 2 | 1.000 | 1.000 | 1.000 | 25550412. | 4966243. | 61915. | 53072. | 162. |
| Ext-I | 3 | 1.000 | 1.000 | 1.000 | 25550412. | 4966243. | 61915. | 53072. | 162. |
| Ext-I | 1a | 1.000 | 1.000 | 1.000 | 25550412. | 4966243. | 61915. | 53072. | 162. |
| Ext-I | 2a | 1.000 | 1.000 | 1.000 | 25550412. | 4966243. | 61915. | 53072. | 162. |
| Ext-I | 3a | 1.000 | 1.000 | 1.000 | 25550412. | 4966243. | 61915. | 53072. | 162. |
| Ext-II | 1 | 1.004 | 1.004 | 1.004 | 25550412. | 4998935. | 61915. | 53421. | 163. |
| Ext-II | 2 | 1.004 | 1.004 | 1.004 | 25550412. | 5033417. | 61915. | 53790. | 163. |
| Ext-II | 3 | 1.004 | 1.004 | 1.004 | 25550412. | 4997842. | 61915. | 53410. | 163. |
| Ext-II | 1a | 1.004 | 1.004 | 1.004 | 25550412. | 4998935. | 61915. | 53421. | 163. |
| Ext-II | 2a | 1.004 | 1.004 | 1.004 | 25550412. | 5033417. | 61915. | 53790. | 163. |
| Ext-II | 3a | 1.004 | 1.004 | 1.004 | 25550412. | 4997842. | 61915. | 53410. | 163. |

```
*************************************************
* LRFD Factored Loads for Column CHECK (Kip,K-Ft) *
*************************************************************************
```

Loads Have Been Combined According to Their Sign.

Applied Factored Moments are Magnified for Slenderness,
Except for Seismic Moments and Arbritrary Moment(s).

Length = 30.39 Ft f'c = 4.00 Ksi fy = 60.00 Ksi
Steel = 0.47 % As = 17.78 in^2

Mn = Nominal Flexural Resistance
Mr = Factored Flexural Resistance
Mr = Phi x Mn

Pn = Nominal Axial Resistance
Pr = Factored Axial Resistance
Pr = Phi x Pn

Mu = Factored Moment
Mu <= Mr

Pu = Factored Axial Load
Pu = Pr (Set by Program)

| Limit State | Ca | :----- Applied LRFD Factored------: Trans Muy | Long Mux | Comb Mu | Axial Pu=Pr | Capacity (Phi*Mn) Mr | Phi | Maximum Rebar Strain | Ratio Mr/Mu | |
|---|---|---|---|---|---|---|---|---|---|---|
| Str-I | 1 | 0. | 860. | 860. | 361. | 2137. | 0.900 | 0.04034 | 2.48 | OK |
| Str-I | 2 | 0. | 1038. | 1038. | 361. | 2137. | 0.900 | 0.04034 | 2.06 | OK |
| Str-I | 3 | 0. | 855. | 855. | 362. | 2140. | 0.900 | 0.04027 | 2.50 | OK |
| Str-I | 1a | 0. | 813. | 813. | 249. | 1962. | 0.900 | 0.04476 | 2.41 | OK |
| Str-I | 2a | 0. | 990. | 990. | 249. | 1962. | 0.900 | 0.04476 | 1.98 | OK |
| Str-I | 3a | 0. | 807. | 807. | 251. | 1965. | 0.900 | 0.04468 | 2.43 | OK |
| Str-II | 1 | 0. | 885. | 885. | 363. | 2140. | 0.900 | 0.04025 | 2.42 | OK |
| Str-II | 2 | 0. | 1020. | 1020. | 361. | 2138. | 0.900 | 0.04030 | 2.10 | OK |
| Str-II | 3 | 0. | 773. | 773. | 363. | 2140. | 0.900 | 0.04025 | 2.77 | OK |
| Str-II | 1a | 0. | 837. | 837. | 252. | 1966. | 0.900 | 0.04466 | 2.35 | OK |
| Str-II | 2a | 0. | 972. | 972. | 250. | 1963. | 0.900 | 0.04472 | 2.02 | OK |
| Str-II | 3a | 0. | 726. | 726. | 252. | 1966. | 0.900 | 0.04466 | 2.71 | OK |
| Str-III | 1 | 0. | 711. | 711. | 359. | 2134. | 0.900 | 0.04040 | 3.00 | OK |
| Str-III | 2 | 0. | 812. | 812. | 359. | 2134. | 0.900 | 0.04040 | 2.63 | OK |
| Str-III | 3 | 0. | 812. | 812. | 359. | 2134. | 0.900 | 0.04040 | 2.63 | OK |
| Str-III | 1a | 0. | 663. | 663. | 248. | 1959. | 0.900 | 0.04483 | 2.95 | OK |
| Str-III | 2a | 0. | 764. | 764. | 248. | 1959. | 0.900 | 0.04483 | 2.56 | OK |
| Str-III | 3a | 0. | 764. | 764. | 248. | 1959. | 0.900 | 0.04483 | 2.56 | OK |
| Str-IV | 1 | 0. | 723. | 723. | 423. | 2234. | 0.900 | 0.03806 | 3.09 | OK |
| Str-IV | 2 | 0. | 723. | 723. | 423. | 2234. | 0.900 | 0.03806 | 3.09 | OK |
| Str-IV | 3 | 0. | 723. | 723. | 423. | 2234. | 0.900 | 0.03806 | 3.09 | OK |
| Str-IV | 1a | 0. | 663. | 663. | 248. | 1959. | 0.900 | 0.04483 | 2.96 | OK |
| Str-IV | 2a | 0. | 663. | 663. | 248. | 1959. | 0.900 | 0.04483 | 2.96 | OK |
| Str-IV | 3a | 0. | 663. | 663. | 248. | 1959. | 0.900 | 0.04483 | 2.96 | OK |
| Str-V | 1 | 0. | 826. | 826. | 360. | 2136. | 0.900 | 0.04035 | 2.59 | OK |
| Str-V | 2 | 0. | 1093. | 1093. | 360. | 2136. | 0.900 | 0.04035 | 1.96 | OK |
| Str-V | 3 | 0. | 952. | 952. | 361. | 2138. | 0.900 | 0.04030 | 2.25 | OK |
| Str-V | 1a | 0. | 778. | 778. | 249. | 1961. | 0.900 | 0.04477 | 2.52 | OK |
| Str-V | 2a | 0. | 1044. | 1044. | 249. | 1961. | 0.900 | 0.04477 | 1.88 | OK |
| Str-V | 3a | 0. | 904. | 904. | 250. | 1963. | 0.900 | 0.04472 | 2.17 | OK |
| Ext-I | 1 | 0. | 656. | 656. | 282. | 2471. | 1.163 | 0.04607 | 3.77 | OK |
| Ext-I | 2 | 0. | 656. | 656. | 282. | 2471. | 1.163 | 0.04607 | 3.77 | OK |
| Ext-I | 3 | 0. | 656. | 656. | 282. | 2471. | 1.163 | 0.04607 | 3.77 | OK |
| Ext-I | 1a | 0. | 656. | 656. | 282. | 2471. | 1.163 | 0.04607 | 3.77 | OK |
| Ext-I | 2a | 0. | 656. | 656. | 282. | 2471. | 1.163 | 0.04607 | 3.77 | OK |
| Ext-I | 3a | 0. | 656. | 656. | 282. | 2471. | 1.163 | 0.04607 | 3.77 | OK |
| Ext-II | 1 | 0. | 701. | 701. | 283. | 2014. | 0.900 | 0.04339 | 2.87 | OK |
| Ext-II | 2 | 0. | 752. | 752. | 283. | 2014. | 0.900 | 0.04339 | 2.68 | OK |
| Ext-II | 3 | 0. | 700. | 700. | 283. | 2015. | 0.900 | 0.04337 | 2.88 | OK |
| Ext-II | 1a | 0. | 701. | 701. | 283. | 2014. | 0.900 | 0.04339 | 2.87 | OK |
| Ext-II | 2a | 0. | 752. | 752. | 283. | 2014. | 0.900 | 0.04339 | 2.68 | OK |
| Ext-II | 3a | 0. | 700. | 700. | 283. | 2015. | 0.900 | 0.04337 | 2.88 | OK |

NOTE: Both The Unreduced Seismic Moments and Axial Loads are Divided
by the Response Modification Factor R.

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*********************************************************************
* Column Service Loads (Kip,K-Ft) and Maximum Working Stresses (Psi ) *
*********************************************************************
```

Young s Modulus for Concrete - Ee = 3604997. Psi
Modular Ratio (Es/Ec) - N = 8.0

| | | :---- Applied Service Loads ---: | | | | :----- Maximum Stress ------: | | |
|---|---|---|---|---|---|---|---|---|
| | | Trans | Long | Comb | Axial | Conc | Steel | Steel |
| Load | Ca | My | Mx | M | P | Comp | Ten | Comp |
| Ser-I | 1 | 0. | 786. | 786. | 283. | 712. | 14047. | 4737. |
| Ser-I | 2 | 0. | 1009. | 1009. | 283. | 930. | 21711. | 6023. |
| Ser-I | 3 | 0. | 905. | 905. | 284. | 829. | 18060. | 5431. |

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```
*********************************************
* Service Loads for Footing Design (Kip,K-Ft) *
* (Impact Removed)                            *
***********************************************************************
```

| Limit State | Ca | Trans My | Long Mx | Comb M | Axial P |
|---|---|---|---|---|---|
| Ser-I | 1 | 0. | 779. | 779. | 283. |
| Ser-I | 2 | 0. | 981. | 981. | 283. |
| Ser-I | 3 | 0. | 898. | 898. | 284. |

```
*********************************************
* Factored Loads for Footing Design (Kip,K-Ft) *
* (Impact Removed)                             *
***********************************************************************
```

| Limit State | Ca | Trans My | Long Mx | Comb M | Axial P |
|---|---|---|---|---|---|
| Str-I | 1 | 0. | 844. | 844. | 360. |
| Str-I | 2 | 0. | 985. | 985. | 360. |
| Str-I | 3 | 0. | 839. | 839. | 362. |
| Str-I | 1a | 0. | 798. | 798. | 249. |
| Str-I | 2a | 0. | 940. | 940. | 249. |
| Str-I | 3a | 0. | 794. | 794. | 250. |
| Str-II | 1 | 0. | 837. | 837. | 362. |
| Str-II | 2 | 0. | 938. | 938. | 361. |
| Str-II | 3 | 0. | 754. | 754. | 362. |
| Str-II | 1a | 0. | 792. | 792. | 251. |
| Str-II | 2a | 0. | 893. | 893. | 250. |
| Str-II | 3a | 0. | 709. | 709. | 251. |
| Str-III | 1 | 0. | 707. | 707. | 359. |
| Str-III | 2 | 0. | 808. | 808. | 359. |
| Str-III | 3 | 0. | 808. | 808. | 359. |
| Str-III | 1a | 0. | 662. | 662. | 248. |
| Str-III | 2a | 0. | 763. | 763. | 248. |
| Str-III | 3a | 0. | 763. | 763. | 248. |
| Str-IV | 1 | 0. | 718. | 718. | 423. |
| Str-IV | 2 | 0. | 718. | 718. | 423. |
| Str-IV | 3 | 0. | 718. | 718. | 423. |
| Str-IV | 1a | 0. | 662. | 662. | 248. |
| Str-IV | 2a | 0. | 662. | 662. | 248. |
| Str-IV | 3a | 0. | 662. | 662. | 248. |
| Str-V | 1 | 0. | 812. | 812. | 360. |
| Str-V | 2 | 0. | 1050. | 1050. | 360. |
| Str-V | 3 | 0. | 938. | 938. | 361. |
| Str-V | 1a | 0. | 767. | 767. | 249. |
| Str-V | 2a | 0. | 1005. | 1005. | 249. |
| Str-V | 3a | 0. | 893. | 893. | 250. |
| Ext-I | 1 | 0. | 656. | 656. | 282. |
| Ext-I | 2 | 0. | 656. | 656. | 282. |
| Ext-I | 3 | 0. | 656. | 656. | 282. |
| Ext-I | 1a | 0. | 656. | 656. | 282. |
| Ext-I | 2a | 0. | 656. | 656. | 282. |
| Ext-I | 3a | 0. | 656. | 656. | 282. |
| Ext-II | 1 | 0. | 695. | 695. | 282. |
| Ext-II | 2 | 0. | 735. | 735. | 282. |
| Ext-II | 3 | 0. | 694. | 694. | 283. |
| Ext-II | 1a | 0. | 695. | 695. | 282. |
| Ext-II | 2a | 0. | 735. | 735. | 282. |
| Ext-II | 3a | 0. | 694. | 694. | 283. |

```
**************************************************
* Plastic Moment (1.3 X Nominal Moment) (Kip,K-Ft) *
**************************************************************************
(LRFD 3.10.9.4.3b)

f'c =     4.00 Ksi          fy  =    60.00 Ksi
Eo  =  0.00300              Ast =      17.78 in^2
```

| Angle | Neutral Axis Loc Angle | Dist-D | Trans Mny | Long Mnx | Comb Mn | Axial Pn | P-Balance (Approx) |
|---|---|---|---|---|---|---|---|
| 90.0 | 90.0 | -1.678 | 16551. | 0. | 16551. | 8314. | 6443. |
| | 90.0 | -0.850 | 17756. | 0. | 17756. | 6928. | |
| | 90.0 | -0.046 | 17775. | 0. | 17775. | 5542. | |
| | 90.0 | 0.755 | 16511. | 0. | 16511. | 4159. | |
| | 90.0 | 1.559 | 13953. | 0. | 13953. | 2773. | |
| | 90.0 | 2.375 | 10072. | 0. | 10072. | 1387. | |
| | 90.0 | 3.027 | 6063. | 0. | 6063. | 283. | |
| | 90.0 | 3.204 | 4878. | 0. | 4878. | 1. | |
| | 90.0 | 3.555 | 1768. | 0. | 1768. | -692. | |
| | 90.0 | 3.816 | 614. | 0. | 614. | -940. | |
| 75.0 | 36.4 | -1.278 | 13969. | 3742. | 14462. | 8316. | 6306. |
| | 36.1 | -0.751 | 14822. | 3970. | 15344. | 6930. | |
| | 36.9 | -0.249 | 14797. | 3964. | 15319. | 5544. | |
| | 39.5 | 0.290 | 13836. | 3706. | 14324. | 4158. | |
| | 43.7 | 0.905 | 11960. | 3204. | 12381. | 2772. | |
| | 51.7 | 1.710 | 8969. | 2403. | 9285. | 1387. | |
| | 63.9 | 2.599 | 5624. | 1507. | 5822. | 283. | |
| | 66.0 | 2.846 | 4596. | 1231. | 4758. | 1. | |
| | 83.0 | 3.523 | 1749. | 470. | 1811. | -692. | |
| | 0.0 | 0.000 | ********** | ********** | ********** | ********** | |
| 60.0 | 23.3 | -1.018 | 10285. | 5935. | 11874. | 8316. | 6306. |
| | 23.8 | -0.632 | 10872. | 6275. | 12553. | 6930. | |
| | 24.2 | -0.282 | 10879. | 6280. | 12562. | 5544. | |
| | 23.8 | 0.149 | 10271. | 5929. | 11860. | 4159. | |
| | 24.3 | 0.618 | 8939. | 5160. | 10322. | 2773. | |
| | 24.6 | 1.176 | 6821. | 3937. | 7876. | 1387. | |
| | 21.1 | 1.682 | 4355. | 2514. | 5029. | 283. | |
| | 18.7 | 1.798 | 3586. | 2069. | 4140. | 1. | |
| | 5.2 | 1.798 | 1371. | 790. | 1582. | -692. | |
| | 0.0 | 0.000 | ********** | ********** | ********** | ********** | |

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```
*************************************************************
* Plastic Moment Values (Kip,K-Ft) Ast =      17.78 (in^2) *
*************************************************************
```

| Angle | Neutral Axis Loc Angle | Dist-D | Trans Mny | Long Mnx | Comb Mn | Axial Pn | P-Balance (Approx) |
|---|---|---|---|---|---|---|---|
| 45.0 | 16.6 | -0.968 | 7147. | 7143. | 10104. | 8316. | 6583. |
| | 18.3 | -0.620 | 7598. | 7593. | 10742. | 6930. | |
| | 16.9 | -0.195 | 7650. | 7648. | 10818. | 5545. | |
| | 16.1 | 0.196 | 7211. | 7210. | 10197. | 4159. | |
| | 14.2 | 0.611 | 6243. | 6243. | 8829. | 2773. | |
| | 11.0 | 1.102 | 4575. | 4575. | 6470. | 1387. | |
| | 6.2 | 1.486 | 2761. | 2759. | 3903. | 283. | |
| | 4.4 | 1.563 | 2225. | 2221. | 3144. | 1. | |
| | 1.0 | 1.665 | 810. | 806. | 1142. | -694. | |
| | 0.0 | 0.000 | ********** | ********** | ********** | ********** | |
| 30.0 | 12.9 | -0.983 | 4548. | 7869. | 9089. | 8316. | 6098. |
| | 11.0 | -0.520 | 4870. | 8433. | 9738. | 6930. | |
| | 11.0 | -0.136 | 4923. | 8525. | 9844. | 5544. | |
| | 10.7 | 0.262 | 4622. | 8005. | 9244. | 4159. | |
| | 8.9 | 0.703 | 3900. | 6755. | 7800. | 2773. | |
| | 5.8 | 1.164 | 2788. | 4830. | 5577. | 1387. | |
| | 2.6 | 1.503 | 1629. | 2820. | 3257. | 283. | |
| | 1.8 | 1.568 | 1301. | 2251. | 2600. | 1. | |
| | 0.5 | 1.657 | 479. | 817. | 947. | -692. | |
| | 0.0 | 0.000 | ********** | ********** | ********** | ********** | |
| 15.0 | 5.3 | -0.861 | 2244. | 8277. | 8576. | 8314. | 6582. |
| | 5.8 | -0.470 | 2487. | 8949. | 9288. | 6929. | |
| | 5.7 | -0.084 | 2448. | 9132. | 9455. | 5545. | |
| | 5.4 | 0.341 | 2264. | 8447. | 8745. | 4158. | |
| | 4.3 | 0.788 | 1883. | 7027. | 7275. | 2773. | |
| | 2.5 | 1.230 | 1325. | 4946. | 5120. | 1387. | |
| | 1.1 | 1.525 | 761. | 2840. | 2941. | 283. | |
| | 0.7 | 1.577 | 606. | 2260. | 2340. | 1. | |
| | 0.2 | 1.650 | 222. | 821. | 850. | -692. | |
| | 0.0 | 0.000 | ********** | ********** | ********** | ********** | |
| 0.0 | 0.0 | -0.817 | 0. | 8438. | 8438. | 8314. | 6581. |
| | 0.0 | -0.430 | 0. | 9224. | 9224. | 6928. | |
| | 0.0 | -0.026 | 0. | 9367. | 9367. | 5542. | |
| | 0.0 | 0.418 | 0. | 8595. | 8595. | 4159. | |
| | 0.0 | 0.862 | 0. | 7143. | 7143. | 2773. | |
| | 0.0 | 1.292 | 0. | 4999. | 4999. | 1387. | |
| | 0.0 | 1.540 | 0. | 2844. | 2844. | 283. | |
| | 0.0 | 1.582 | 0. | 2263. | 2263. | 1. | |
| | 0.0 | 1.647 | 0. | 823. | 823. | -692. | |
| | 0.0 | 1.768 | 0. | 460. | 460. | -861. | |

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```
*****************
* Bent Analysis *
*****************************************************************************
```

Note: Equations are Referenced to Caltrans LFD Bridge Design Specification, September 2004.

All Equations are in US Format.

Design Assumptions:

1. All Columns are the Same i.e., Prismatic, Equal Length, Equal Spacing.
2. Plastic Hinges form at the Top and Bottom of the Columns.
3. The Distance Between the Plastic Hinges is Taken as the Column Length.
4. The Over-Turning Force Acts Through the C.G. of the SuperStructure at a Distance 'Cgs' Above the Top of the Column.
5. The Columns are Numbered from Left to Right and the OverTurning Force Acts to the Right.

```
************************
* Transverse Direction *
*****************************************************************************
```

Loads Resulting from Plastic Hinging of Column No. 1

| | | | | |
|---|---|---|---|---|
| Top of Column | Axial Force | P = | 162. | Kip |
| | Plastic Moment | Mp = | 0. | K-Ft |
| Bottom of Column | Axial Force | P = | 282. | Kip |
| | Plastic Moment | Mp = | 6059. | K-Ft |
| Design Shear Force: | | Vu = | 199. | Kip |

```
***************************
* Longitudinal Direction *
*****************************************************************************
```

Loads Resulting from Plastic Hinging of Column No. 1

| | | | | |
|---|---|---|---|---|
| Top of Column | Axial Force | P = | 162. | Kip |
| | Plastic Moment | Mp = | 0. | K-Ft |
| Bottom of Column | Axial Force | P = | 282. | Kip |
| | Plastic Moment | Mp = | 2842. | K-Ft |
| Design Shear Force: | | Vu = | 94. | Kip |

Displacement Capacity and Ductility
Inside Plastic Hinge zone w/o screw

Pier 2

Dead Load = 7414.73 kips
EQ Load = 814.76 k
DL + EQ = 8229.49 k

H = 30.39'
L = 232' = 2784"
$f_{ye}$ = 68 ksi
$d_{bl}$ = 1.128 in #9 bar

Capacity

From SAP 2000

$\phi_y$ idealized = 8.988 E-05 rad/in
$\phi_u$ = 0.002623 rad/in
$M_{p-long}$ = 815014 k-in

$L = 30.39 * 12 = 364.68$ in

$L_p = 0.08L + 0.5 f_{ye} * d_{bl} \geq 0.3 f_{ye} * d_{bl}$

$L_p = 0.08(364.68) + 0.5(68)(1.128) \geq 0.3(68)(1.128)$

$= 67.52 \geq 23.01$

$L_p = 67.52$ in

$\phi_p = \phi_u - \phi_y = 0.002623 - 8.988E-05 = 2.53E-3$

$\theta_p = L_p * \phi_p = 67.52 \text{ in} * 2.53E-3 = 0.17$ rad

$\Delta_p = \theta_p * (L - 0.5L_p)$

$= 0.17 + (364.68 - 67.58(0.5))$

$= 56.25$ in

$\Delta Y\text{-col} = (L^2/3) * \phi_y = (364.68^2/3) * 8.988E-05 = 3.98$

$\Delta c = \Delta Y\text{-col} + \Delta p = 3.98 + 56.25 = 60.23$

$\mu C = \Delta c / \Delta Y\text{-col} = 60.23 / 3.98 = 15.13"$

Demand

From SAP

$\Delta D = 7.5405$

$\Delta Y = 3.984$

$\mu D = \Delta D / \Delta Y = 7.5405 / 3.984 = 1.89"$

$\mu D < \mu C$ ✓

**Displacement Capacity and Ductility Demand**

Whole PW- Inside Plastic Hinge Zone w/o scew - Spacing 6"

| Pier | Dead Load (kips) | Height | Total Length of Pier (ft) | Total Length of Pier(in) | Total EQ Load (k) | DL + EQ (k) | φ_Y (idealized) (rad/in) | φ_u (rad/in) | Mp-long (k-in) | Mp-long (k-ft) | Icrack |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 2 | 7414.73 | 30.39 | 226 | 2712 | 814.76 | 8229.49 | 8.991E-05 | 2.62E-03 | 797567 | 66464 | 2460572 |
| 3 | 7758.71 | 36.04 | 225.5 | 2706 | 310.364 | 8069.074 | 8.98E-05 | 2.62E-03 | 792015 | 66001 | 2429502 |
| 4 | 7853.672 | 34.2 | 226 | 2712 | 873.33 | 8727.002 | 9.16E-05 | 2.42E-03 | 800685 | 66724 | 2423809 |

| Pier | L (in) | LP (in) | LP >= 3 f_ye d_bl | φ_Y (rad/in) | φ_u (rad/in) | φ_p (rad/in) | θ_p (rad) | Δp (in) | ΔY-col (in) | Δc (in) | μC |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 2 | 364.68 | 67.5264 | true | 0.00008991 | 0.002623 | 0.002533 | 0.1710504 | 56.60347 | 3.985755 | 60.58922 | 15.2014405 |
| 3 | 432.48 | 72.9504 | true | 0.00008984 | 0.002622 | 0.002532 | 0.1847221 | 73.15083 | 5.601193 | 78.75203 | 14.0598662 |
| 4 | 410.4 | 71.184 | true | 0.00009163 | 0.00242 | 0.002328 | 0.1657427 | 62.12169 | 5.144357 | 67.26604 | 13.075694 |

f_ye= 68 ksi

d_bl= 1.128 in #9 bar

**Equations SDC 3.1.3 and SDC 7.6.2(a)**

Lp=0.08L+ 0.5f_ye*d_bl >= 0.3f_ye*d_bl

φ_p= φ_u - φ_Y

θp= Lp*φ_p

Δp= θp* (L-0.5Lp)

ΔY-col= ($L^2$/3) *φ_Y

Δc=ΔY-col + Δp

μC= Δc/ΔY-col

μD= ΔD/ΔY

Displacement Ductility Demand

| Pier | ΔD | ΔY | μD | μC > μD |
|---|---|---|---|---|
| 2 | 7.5405 | 3.985755 | 1.891862 | TRUE |
| 3 | 7.8167 | 5.601193 | 1.395542 | TRUE |
| 4 | 7.6322 | 5.144357 | 1.483606 | TRUE |

**Displacement Capacity and Ductility Demand**

Whole PW- Inside Plastic Hinge Zone w/scew - Spacing 6"

| Pier | Dead Load (kips) | Height | Total Length of Pier (ft) | Total Length of Pier(in) | Total EQ Load (k) | DL +EQ (k) | φ_Y (idealized) (rad/in) | φ_u (rad/in) | Mp(k-in) | Icrack |
|---|---|---|---|---|---|---|---|---|---|---|
| 2 | 7414.73 | 30.39 | 226 | 2712 | 1165.606 | 8580.336 | 8.99E-05 | 2.62E-03 | 821003 | 2532558 |
| 3 | 7758.71 | 36.04 | 225.5 | 2706 | 246.442 | 8005.152 | 8.99E-05 | 2.62E-03 | 790961 | 2441886 |
| 4 | 7853.672 | 34.2 | 226 | 2712 | 1245.971 | 9099.643 | 9.17E-05 | 2.45E-03 | 865065 | 2437414 |

65913 k-ft/226 = 291 k-ft/ft × 7.5 = 2187

~~2758/226 × 7.5 = 257 kips~~

| Pier | L (in) | LP (in) | LP >= 3.1_ye d_bl | φ_Y (rad/in) | φ_u (rad/in) | φ_p (rad/in) | θ_p (rad) | Δp (in) | ΔY-col (in) | Δc (in) | μC |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 2 | 364.68 | 67.5264 | true | 0.00008993 | 0.002624 | 0.002534 | 0.17111662 | 56.6253658 | 3.986642 | 60.612 | 15.2037752 |
| 3 | 432.48 | 72.9504 | true | 0.00008985 | 0.002623 | 0.002533 | 0.18479431 | 73.1794321 | 5.601817 | 78.7812 | 14.0635181 |
| 4 | 410.4 | 71.184 | true | 0.00009167 | 0.002452 | 0.00236 | 0.16801773 | 62.9743896 | 5.146603 | 68.121 | 13.2361076 |

f_ye= 68 ksi

d_bl= 1.128 in #9 bar

**Equations SDC 3.1.3 and SDC 7.6.2(a)**

Lp=0.08L+ 0.5f_ye*d_bl >= 0.3f_ye*d_bl

φ_p= φ_u - φ_Y

θp= Lp*φ_p

Δp= θp* (L-0.5Lp)

ΔY-col= (L²/3) *φ_Y

Δc=ΔY-col + Δp

✓ μC= Δc/ΔY-col

✓ μD= ΔD/ΔY

$\mu_D = \frac{\Delta D}{\Delta_y}$ ≤ 5 for pier wall.

$\mu_c = \frac{\Delta_c}{\Delta_y}$

Displacement Ductility Demand

| Pier | ΔD | ΔY | μD | μC>μD |
|---|---|---|---|---|
| 2 | 5.643 | 3.986642 | 1.415477 | TRUE |
| 3 | 5.8812 | 5.601817 | 1.049874 | TRUE |
| 4 | 5.7497 | 5.146603 | 1.117183 | TRUE |

From SAP 2000

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**Displacement Capacity and Ductility Demand**

Whole PW- Outside Plastic Hinge Zone w/o scew - Spacing 12"

| Pier | Dead Load (kips) | Height | Total Length of Pier (ft) | Total Length of Pier (in) | Total EQ Load (k) | DL +EQ (k) | φ_Y (idealized) (rad/in) | φ_u (rad/in) | Mp-long (k-in) | Icrack |
|---|---|---|---|---|---|---|---|---|---|---|
| 2 | 7414.73 | 30.39 | 226 | 2712 | 814.76 | 8229.49 | 9.15E-05 | 2.48E-03 | 792489 | 2403748 |
| 3 | 7758.71 | 36.04 | 225.5 | 2706 | 310.364 | 8069.074 | 9.14E-05 | 2.48E-03 | 787154 | 2389121 |
| 4 | 7853.672 | 34.2 | 226 | 2712 | 873.33 | 8727.002 | 9.16E-03 | 2.46E-03 | 800685 | 2423809 |

| Pier | L (in) | LP (in) | LP >= 3*f_ye d_bl | φ_Y (rad/in) | φ_u (rad/in) | φ_p (rad/in) | θ_p (rad) | Δp (in) | ΔY-col (in) | Δc (in) | μC |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 2 | 364.68 | 67.5264 | true | 0.00009145 | 0.002475 | 0.002384 | 0.1609526 | 53.2619 | 4.054024 | 57.3159 | 14.1380325 |
| 3 | 432.48 | 72.9504 | true | 0.00009139 | 0.002478 | 0.002387 | 0.1741042 | 68.94608 | 5.69783 | 74.6439 | 13.1004105 |
| 4 | 410.4 | 71.184 | true | 0.009163 | 0.002462 | -0.006701 | -0.477004 | -178.785 | 514.4357 | 335.651 | 0.65246406 |

f_ye= 68 ksi

d_bl= 1.128 in #9 bar

**Equations SDC 3.1.3 and SDC 7.6.2(a)**

Lp=0.08L+ 0.5f_ye*d_bl >= 0.3f_ye*d_bl

φ_p= φ_u - φ_Y

θp= Lp*φ_p

Δp= θp* (L-0.5Lp)

ΔY-col= ($L^2$/3) *φ_Y

Δc=ΔY-col + Δp

μC= Δc/ΔY-col

μD= ΔD/ΔY

Displacement Ductility Demand

| Pier | ΔD | ΔY | μD | μC>μD |
|---|---|---|---|---|
| 2 | 7.5405 | 4.05402 | 1.860004 | TRUE |
| 3 | 7.8167 | 5.69783 | 1.371873 | TRUE |
| 4 | 7.6322 | 514.436 | 0.014836 | TRUE |

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**Displacement Capacity and Ductility Demand**

Whole PW- Outside Plastic Hinge Zone w/scew - Spacing 12"

| Pier | Dead Load (kips) | Height | Total Length of Pier (ft) | Total Length of Pier (in) | Total EQ Load (k) | DL + EQ (k) | φ_Y (idealized) (rad/in) | φ_u (rad/in) | Mp(k-in) | Icrack |
|---|---|---|---|---|---|---|---|---|---|---|
| 2 | 7414.73 | 30.39 | 226 | 2712 | 1165.606 | 8580.336 | 9.16E-05 | 2.47E-03 | 798664 | 2417527 |
| 3 | 7758.71 | 36.04 | 225.5 | 2706 | 246.442 | 8005.152 | 9.15E-05 | 2.48E-03 | 786853 | 2386691 |
| 4 | 7853.672 | 34.2 | 226 | 2712 | 1245.971 | 9099.643 | 9.17E-05 | 2.45E-03 | 805502 | 2437414 |

| Pier | L (in) | LP (in) | LP >= 3*f_ye*d_bl | φ_Y (rad/in) | φ_u (rad/in) | φ_p (rad/in) | θ_p (rad) | Δp (in) | ΔY-col (in) | Δc (in) | μC |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 2 | 364.68 | 67.5264 | true | 0.00009164 | 0.002465 | 0.002373 | 0.1602645 | 53.0342 | 4.062447 | 57.09665 | 14.0547426 |
| 3 | 432.48 | 72.9504 | true | 0.00009145 | 0.00248 | 0.002389 | 0.1742457 | 69.00212 | 5.701571 | 74.7037 | 13.1023011 |
| 4 | 410.4 | 71.184 | true | 0.00009167 | 0.002452 | 0.00236 | 0.1680177 | 62.97439 | 5.146603 | 68.12099 | 13.2361076 |

f_ye= 68 ksi

d_bl= 1.128 in #9 bar

**Equations SDC 3.1.3 and SDC 7.6.2(a)**

Lp=0.08L+ 0.5f_ye*d_bl >= 0.3f_ye*d_bl

φ_p= φ_u - φ_Y

θp= Lp*φ_p

Δp= θp* (L-0.5Lp)

ΔY-col= ($L^2$/3) *φ_Y

Δc=ΔY-col + Δp

μC= Δc/ΔY-col

μD= ΔD/ΔY

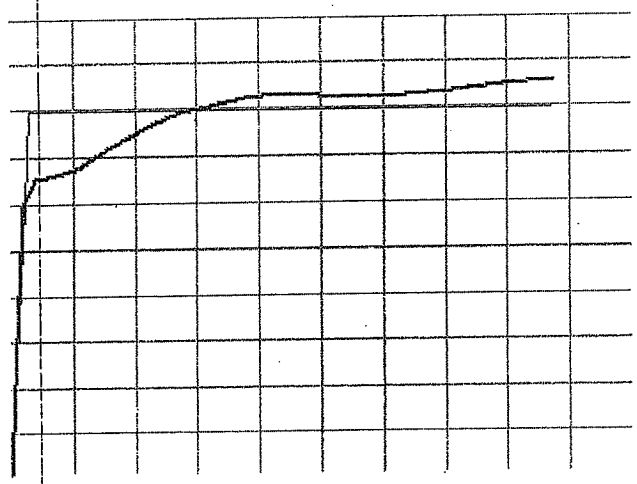
Displacement Ductility Demand

| Pier | ΔD | ΔY | μD | μC>μD |
|---|---|---|---|---|
| 2 | 5.643 | 4.062447 | 1.389064 | TRUE |
| 3 | 5.8812 | 5.701571 | 1.031505 | TRUE |
| 4 | 5.7497 | 5.146603 | 1.117183 | TRUE |

PW2-long

## MOMENT CURVATURE *(M-ϕ)* GRAPH - Kip-in

*Axial Load* = -8230
*Moment Angle* = 0.



### Results For Exact-Integration

$\phi_{y(Initial)}$ = 6.778E-05
$M_y$ = 601234
$\phi_{y(Idealized)}$ = 8.991E-05
$M_p$ = 797567
$I_{crack}$ = 2460572
$\phi_{concrete}$ = N/A
$M_{concrete}$ = N/A
$\phi_{steel}$ = 2.623E-03
$M_{steel}$ = 859566

| Concrete | Neutral | Steel | Tendon Strain | Concrete | Steel | Steel | Prestress | Net Force | Curvature | Moment |
|---|---|---|---|---|---|---|---|---|---|---|

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| Strain | Axis | Strain | | Compression | Compression | Tension | Force | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| -2.237E-05 | 0. | -2.237E-05 | 0. | -7931 | -294.5025 | 0. | 0. | -8225 | 0 | 2.534E-12 |
| 2.591E-04 | 10.6761 | 7.056E-04 | 0. | 11764 | 1117 | 4645.1444 | 0. | -8236 | 0.0000251 | 308688 |
| -5.165E-04 | 12.7696 | 1.895E-03 | 0. | -18773 | -1928 | 12478 | 0. | -8223 | 0.00006275 | 595923 |
| -7.138E-04 | 14.6806 | 3.628E-03 | 0. | -19820 | -2049 | 13620 | 0. | -8249 | 0.0001113 | 648940 |
| -8.980E-04 | 15.8892 | 5.855E-03 | 0. | -20066 | -1789 | 13620 | 0. | -8235 | 0.0001757 | 656528 |
| -1.096E-03 | 16.6328 | 8.551E-03 | 0. | -20526 | -1327 | 13620 | 0. | -8233 | 0.000251 | 661688 |
| -1.323E-03 | 17.0971 | 0.0117 | 0. | -21463 | -755.9217 | 13944 | 0. | -8275 | 0.0003389 | 678294 |
| -1.589E-03 | 17.3818 | 0.0153 | 0. | -22742 | -156.713 | 14628 | 0. | -8271 | 0.0004393 | 706206 |
| -1.900E-03 | 17.5592 | 0.0193 | 0. | -24085 | 0. | 15843 | 0. | -8242 | 0.0005522 | 736064 |
| -2.267E-03 | 17.655 | 0.0238 | 0. | -25441 | 0. | 17220 | 0. | -8221 | 0.0006777 | 768151 |
| -2.688E-03 | 17.7043 | 0.0287 | 0. | -26535 | 0. | 18317 | 0. | -8218 | 0.0008158 | 791811 |
| -3.171E-03 | 17.7184 | 0.034 | 0. | -27608 | 0. | 19090 | 0. | -8213 | 0.0009664 | 806638 |
| -3.740E-03 | 17.6892 | 0.0397 | 0. | -27851 | 0. | 19626 | 0. | -8225 | 0.00113 | 821608 |
| -4.434E-03 | 17.6033 | 0.0457 | 0. | -27792 | 0. | 19590 | 0. | -8202 | 0.001305 | 831607 |
| -5.636E-03 | 17.2264 | 0.0518 | 0. | -24695 | -2061 | 18518 | 0. | -8238 | 0.001493 | 825570 |
| -6.871E-03 | 16.9447 | 0.0586 | 0. | -21651 | -5480 | 18908 | 0. | -8223 | 0.001694 | 824295 |
| -7.993E-03 | 16.8102 | 0.0653 | 0. | -19656 | -7859 | 19269 | 0. | -8246 | 0.001908 | 829477 |
| -9.076E-03 | 16.7458 | 0.0729 | 0. | -18186 | -9694 | 19656 | 0. | -8223 | 0.002134 | 838075 |
| -0.0102 | 16.7199 | 0.081 | 0. | -17063 | -11182 | 20026 | 0. | -8218 | 0.002372 | 848163 |
| -0.0112 | 16.7174 | 0.0896 | 0. | -16199 | -12408 | 20411 | 0. | -8195 | 0.002623 | 859566 |

| Curvature | Moment Actual | Moment Ideal | Area b/w curves |
|---|---|---|---|
| 6.778E-05 | 601234 | 601234 | 0. |
| 8.991E-05 | 624609 | 797567 | 1.9141 |
| 1.130E-04 | 648940 | 797567 | 3.7044 |
| 1.757E-04 | 656528 | 797567 | 9.0884 |
| 2.510E-04 | 661688 | 797567 | 10.4261 |
| 3.389E-04 | 678294 | 797567 | 11.2078 |
| 4.393E-04 | 706206 | 797567 | 10.574 |
| 5.522E-04 | 736064 | 797567 | 8.6331 |
| 6.777E-04 | 768151 | 797567 | 5.7052 |
| 8.158E-04 | 791811 | 797567 | 2.4278 |
| 9.664E-04 | 806638 | 797567 | -0.2496 |
| 1.130E-03 | 821608 | 797567 | -2.7012 |
| 1.305E-03 | 831607 | 797567 | -5.1026 |
| 1.493E-03 | 825570 | 797567 | -5.84 |
| 1.694E-03 | 824295 | 797567 | -5.4951 |
| 1.908E-03 | 829477 | 797567 | -6.2554 |
| 2.134E-03 | 838075 | 797567 | -8.1799 |
| 2.372E-03 | 848163 | 797567 | -10.862 |
| 2.623E-03 | 859566 | 797567 | -14.131 |
| | | | SUM = 4.8639 |

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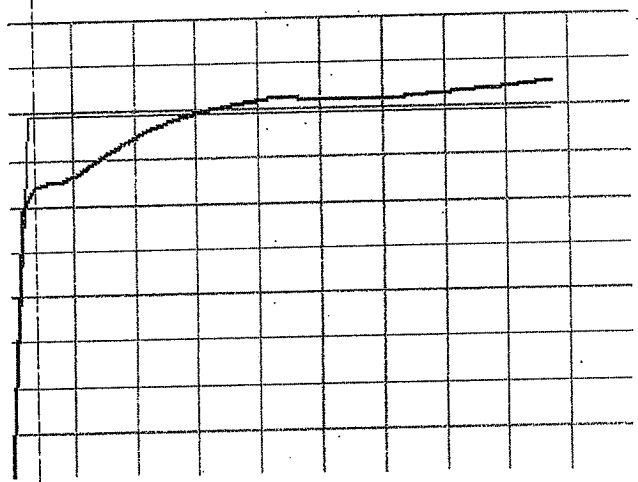
PW3-long

## MOMENT CURVATURE *(M-φ)* GRAPH - Kip-in

*Axial Load* = -8069
*Moment Angle* = 0.



### Results For Exact-Integration

$\phi_{y(Initial)}$ = 6.772E-05
$M_y$ = 596867
$\phi_{y(Idealized)}$ = 8.986E-05
$M_p$ = 792015 Originally used 786892 for $M_p$ in calcs. Difference = 0.6%
$I_{crack}$ = 2444947
$\phi_{concrete}$ = N/A
$M_{concrete}$ = N/A
$\phi_{steel}$ = 2.623E-03
$M_{steel}$ = 853640

| Concrete Strain | Neutral Axis | Steel Strain | Tendon Strain | Concrete Compression | Steel Compressi on | Steel Tension | Prestress Force | Net Force | Curvature | Moment |
|---|---|---|---|---|---|---|---|---|---|---|

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| -2.198E-05 | 0. | -2.198E-05 | 0. | -7776 | -288.0879 | 0. | 0. | -8064 | 0 | 1.125E-11 |
| -2.576E-04 | 10.7358 | 7.071E-04 | 0. | 11607 | 1102 | 4634498 | 0. | -8075 | 0.0000251 | 305696 |
| -5.144E-04 | 12.8025 | 1.898E-03 | 0. | -18593 | -1906 | 12436 | 0. | -8062 | 0.00006275 | 591702 |
| -7.108E-04 | 14.7067 | 3.631E-03 | 0. | -19627 | -2020 | 13560 | 0. | -8087 | 0.000113 | 643906 |
| -8.943E-04 | 15.9101 | 5.859E-03 | 0. | -19877 | -1757 | 13560 | 0. | -8074 | 0.0001757 | 651402 |
| -1.092E-03 | 16.6504 | 8.555E-03 | 0. | -20310 | -1292 | 13560 | 0. | -8072 | 0.000251 | 656513 |
| -1.317E-03 | 17.1125 | 0.0117 | 0. | -21278 | -718.3458 | 13883 | 0. | -8113 | 0.0003389 | 673060 |
| -1.583E-03 | 17.3957 | 0.0153 | 0. | -22558 | 116.0928 | 14564 | 0. | -8110 | 0.0004393 | 700875 |
| -1.893E-03 | 17.5719 | 0.0193 | 0. | -23902 | 0. | 15820 | 0. | -8081 | 0.0005522 | 730643 |
| -2.259E-03 | 17.6672 | 0.0238 | 0. | -25261 | 0. | 17200 | 0. | -8061 | 0.0006777 | 762641 |
| -2.679E-03 | 17.7165 | 0.0287 | 0. | -26358 | 0. | 18302 | 0. | -8056 | 0.0008158 | 786182 |
| -3.160E-03 | 17.7303 | 0.0341 | 0. | -27195 | 0. | 19083 | 0. | -8052 | 0.0009664 | 801009 |
| -3.726E-03 | 17.7011 | 0.0397 | 0. | -27693 | 0. | 19629 | 0. | -8064 | 0.00113 | 815963 |
| -4.416E-03 | 17.6168 | 0.0458 | 0. | -27676 | 0. | 19621 | 0. | -8055 | 0.001305 | 826330 |
| -5.605E-03 | 17.2468 | 0.0518 | 0. | -24665 | -1852 | 18438 | 0. | -8079 | 0.001493 | 820446 |
| -6.846E-03 | 16.9591 | 0.0588 | 0. | -21592 | -5296 | 18826 | 0. | -8063 | 0.001694 | 818892 |
| -7.970E-03 | 16.8219 | 0.0654 | 0. | -19589 | -7678 | 19185 | 0. | -8082 | 0.001908 | 823874 |
| -9.056E-03 | 16.7553 | 0.07297 | 0. | -18116 | -9518 | 19571 | 0. | -8064 | 0.002134 | 832432 |
| -0.0101 | 16.7279 | 0.081 | 0. | -16989 | -11009 | 19939 | 0. | -8059 | 0.002372 | 842432 |
| -0.0112 | 16.7244 | 0.0896 | 0. | -16119 | -12283 | 20322 | 0. | -8030 | 0.002623 | 853640 |

| Curvature | Moment Actual | Moment Ideal | Area b/w curves |
|---|---|---|---|
| 6.772E-05 | 596867 | 596867 | 0. |
| 8.986E-05 | 619891 | 792015 | 119055 |
| 1.130E-04 | 643906 | 792015 | 3.6977 |
| 1.757E-04 | 651402 | 792015 | 9.0588 |
| 2.510E-04 | 656513 | 792015 | 10.3959 |
| 3.389E-04 | 673060 | 792015 | 11.1772 |
| 4.393E-04 | 700875 | 792015 | 10.547 |
| 5.522E-04 | 730643 | 792015 | 8.6132 |
| 6.777E-04 | 762641 | 792015 | 5.6944 |
| 8.158E-04 | 786182 | 792015 | 2.4302 |
| 9.664E-04 | 801009 | 792015 | -0.238 |
| 1.130E-03 | 815963 | 792015 | -2.6873 |
| 1.305E-03 | 826330 | 792015 | -5.1186 |
| 1.493E-03 | 820446 | 792015 | -5.9062 |
| 1.694E-03 | 818892 | 792015 | -5.5531 |
| 1.908E-03 | 823874 | 792015 | -6.2658 |
| 2.134E-03 | 832432 | 792015 | -8.1638 |
| 2.372E-03 | 842432 | 792015 | -10.8299 |
| 2.623E-03 | 853640 | 792015 | -14.0616 |
| | | | SUM= 4.6958 |

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L. Sanchez-Manzo

07-202101



Pier Wall 2 North Shear Check

For Pier Wall 2 North

From Section 3.6.6.1 - Shear Capacity of Pierwalls in weak Direction
Look to 3.6.2 & 3.6.3

$V_c = v_c \times A_e$ where $A_g = (3.5' * 27.58') * 144\,in^2/1ft^2 = 13,900.32\,in^2$

$A_e = 0.8\,A_g$

$A_e = 11,120.26\,in^2$

- Inside the plastic hinge zone
  $v_c = \text{factor } 1 \times \text{factor } 2 \times \sqrt{f'_c} \leq 4\sqrt{f'_c}$
- Outside the plastic hinge zone
  $v_c = 3 \times \text{factor } 2 \times \sqrt{f'_c} \leq 4\sqrt{f'_c}$

where $f'_c$ = greater of { 1.3 f'c or 5000 psi }

1.3(3250) = 4225 < 5000 psi

∴ use 5000 psi

where

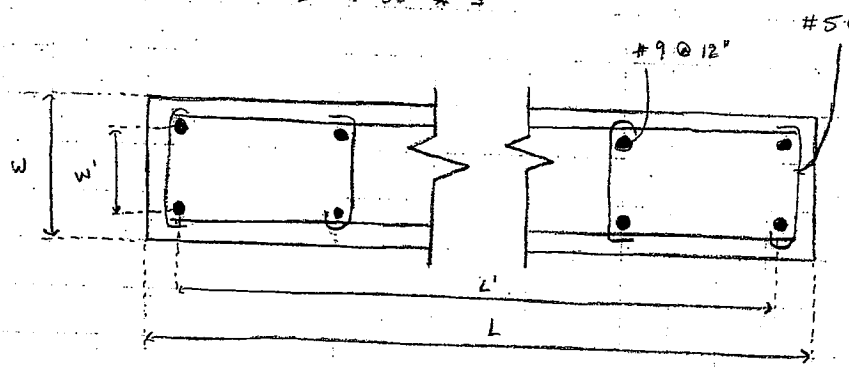
$$\text{factor } 1 = 0.3 \leq \frac{\rho_s f_{yh}}{0.150} + 3.67 - \mu_d < 3$$

$f_{yh} = 60$ ksi

# bars = 29

where $\rho_s$ = volume of steel / volume of concrete

$$= \frac{A_b * \#\text{bars} * w}{L' * w' * s}$$




#TO 6-5

$w = 42"$ $L = 27.58'$

$w' = 42" - 2(2")_{cover} - 2(0.625"\,\phi\,\#5) - 2(1.128\,\phi\,\#9/2) = 35.62"$

$L' = L - 2(2")_{cover} - 2(0.625"\,\phi\,\#5/2) = 326.34"$

Inside plastic hinge: $\rho_s = \dfrac{0.31in^2 * (29) * 42"}{(326.34")(35.62")(6")} = 0.0054$

factor 1 = $0.3 \leq \frac{\rho_s f_{yh}}{0.150} + 3.67 - \mu d < 3$ where $\mu d = 1.859$ for Pier 2

$= 0.3 \leq \frac{0.0054(60\,ksi)}{0.150} + 3.67 - 1.859 < 3$

$= 0.3 \leq 3.97 < 3$ so use $f_1 = 3$

factor 2 = $1 + \frac{P_c}{2000 * A_g} < 1.5$ where $P_c = 448,456$ lbs

$= 1 + \frac{448,456}{2000 * 13,900.32} < 1.5$

$= 1.02 < 1.5$ so $f_2 = 1.02$

Inside the plastic hinge zone

$v_c = f1 \times f_2 \times \sqrt{f'c} \leq 4\sqrt{f'c} = 3 \times 1.02 \times \sqrt{5000} \leq 4\sqrt{5000}$

$216.37 \leq 282.84$

$= 216.37$ psi

Outside the plastic hinge zone

$v_c = 3 \times f_2 \times \sqrt{f'c} \leq 4\sqrt{f'c}$

$v_c = 216.37$ psi

$V_c = v_c \times A_e = 216.37\,psi * 11,120.26\,in^2 \div 1000 = 2,406.09$ kips

SDC. • Inside plastic hinge zone

3.6.3 $V_s = \left(\frac{A_v f_{yh} D'}{s}\right) \leq 8\sqrt{f'c}\, A_e$

3.6.5.1

3.6.5.2 where $A_{v\,min} = 0.025 * \frac{D's}{f_{yh}} = 0.025 * \frac{(35.62'')(6'')}{60}$

$= 0.09\,in^2$

$A_{v\,used} = 0.31\,in^2 * 29$ bars $= 8.99\,in^2$

$V_s = \frac{A_v f_{yh} D'}{s} = \frac{8.99\,in^2(60\,ksi)(35.62'')}{6''} = 3,202.24$ Kips $\leq 8\sqrt{5000}(11,120.26) \div 1000$

$\leq 6,290.51$ Kips

$V_n = V_c + V_s$

$= 2,406.09\,K + 3,202.24\,K = 5,608.33\,K$

$\phi V_n = 0.85(5,608.33) = 4,767.08\,K$ capacity

Demand

$$V_o = \frac{M_o}{H}$$

per SDC 4.3.1 $M_o = 1.2 M_p = 1.2(815014) = 978,016.8$ K-in

$$V_o = \frac{978,016.8 \text{ Kip-in}}{30.39' \times 12\text{in}/1\text{ft}} = 2,681.85 \text{ Kips} < 4,767.08 \text{ K}$$

for Total Length of Pier Wall.
Actual Pier shear force, much smaller

• Outside Plastic hinge zone

$$V_s = \left(\frac{A_v f_{yh} D'}{s}\right) \leq 8\sqrt{f'_c}\, A_c$$

where $A_{v\,min} = \frac{0.025 * D' s}{f_{yh}} = 0.025 * \frac{(35.62")(12")}{60} = 0.18 \text{ in}^2$

$A_{v\text{-used}} = 8.99 \text{ in}^2$

$$V_s = \frac{A_v f_{yh} D'}{s} = \frac{8.99 \text{ in}^2 (60 \text{ ksi})(35.62")}{12"} = 1,601.12 \text{ Kips}$$

$V_c = 216.37 * 11,120.26 \text{ in}^2 / 1000 = 2,406.09$ Kips

$V_n = V_c + V_s = 2,406.09 + 1,601.12$ Kips $= 4,007.21$

$\phi V_n = 0.85(4,007.21) = 3,406.13$ Kips capacity

Demand $V_o = \frac{M_o}{H} = \frac{1.2 M_p}{H} = \frac{1.2(810650 \text{ K-in})}{30.39' * 12"/\text{ft}} = \frac{972,780}{364.68} = 2,667.49$ K

$< 3,406.13$ K

∴ OK

Shear Capacity in the Strong Direction

$\phi V_n^{pw} > V_u^{pw}$ where $\phi = 0.85$ SDC 3.6.6.2

SDC 2.3.2.2.

Pier walls with fixed-fixed end conditions shall be designed for the least value of

1. unreduced elastic ARS demand
2. 130% of the ultimate capacity of the foundation

Note: foundation not designed yet so will check this condition at a later time.

Type D Soil
Mag. 7.25 ± 0.25

• Calculating Vu from ARS Curve

period - transverse = 0.60162 - from SAP 2000

a = 1.42g from ODC Appendix B.

Vu = m * a

where m = [(3.5' * 27.58' * 30.39')(0.150 kcf)] / 32.2 ft/s + 448.456 k/k

= 13.67 + 13.93 = 27.60 ks/ft

Vu = m * a = 27.68 * (1.42 * 32.2) = 1,261.85 K

• Checking capacity outside of the hinge zone.

• BDS 8.16.6.9.2

Vn = vn Ac

where $v_n = 2\sqrt{f_c'} + \rho_h f_y < 8\sqrt{f_c'}$

where $\rho_h$ = horizontal shear reinforcement area / gross concrete area of vertical section

$= \frac{0.31 in^2 * 29 \text{ bars}}{12" * (30.36 * 12in/ft)} = \frac{8.99}{4371.84} = 2.06E-3$

$v_n = 2\sqrt{5} \text{ ksi} + 2.06E-3(60 \text{ ksi}) < 8\sqrt{5} \text{ ksi}$

= 4.47 + 0.12 = 4.59 ksi

Ac = 35.62" * 326.34" = 11,624.23 in

Vn = 53,355.22 Kips

$\phi V_n$ = 45,351.94 K > Vu = 1,261.85 K ∴ OK

**Shear Capacity in Longitudinal (Weak) Direction**

Givens:

Ab= 0.31 in²

fyh= 60 ksi

f'c= greater of {1.3*3250 psi or 5000 psi

= 5000 psi

w= 42 in

w'= 42"- 4"- 2*(.625)- 2*(1.128/2) 35.622 in

#5 Spacing inside plastic hinge zone = 6 in

#5 Spacing outside plastic hinge zone : 12 in

Av-min (inside) = 0.08906 in²

Av-min (outside)= 0.17811 in²

| Pier | Girder Location | Height (ft) | Length (ft) | Length (in) | L' (in) | Ag_long (in) | Ae (in) | # of #5 ties along L' | ρS_inside plastic hinge | μD | Factor 1 | 0.3<= F1<3 | Pc (lbs) | Factor 2 | F2<1.5 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 2 | North | 30.39 | 27.58 | 330.96 | 326.34 | 13900.32 | 11120.3 | 29 | 0.005413 | 1.89186 | 3.94353 | 3 | 448,456 | 1.0161 | 1.01613 |
| | South | 30.39 | 34.79 | 417.48 | 412.86 | 17534.16 | 14027.3 | 36 | 0.005312 | 1.89186 | 3.90288 | 3 | 600,651 | 1.0171 | 1.01713 |
| 3 | North | 36.04 | 24.42 | 293.04 | 288.42 | 12307.68 | 9846.14 | 26 | 0.005492 | 1.39554 | 4.47109 | 3 | 437,148 | 1.0178 | 1.01776 |
| | South | 36.04 | 34.94 | 419.28 | 414.66 | 17609.76 | 14087.8 | 36 | 0.005289 | 1.39554 | 4.38998 | 3 | 579,753 | 1.0165 | 1.01646 |
| 4 | North | 34.2 | 27.52 | 330.24 | 325.62 | 13870.08 | 11096.1 | 29 | 0.005425 | 1.48361 | 4.35657 | 3 | 459,815 | 1.0166 | 1.01658 |
| | South | 34.2 | 35.1 | 421.2 | 416.58 | 17690.4 | 14152.3 | 36 | 0.005264 | 1.48361 | 4.29216 | 3 | 613,892 | 1.0174 | 1.01735 |

Inside Plastic Hinge

| Pier | Girder Location | vc_inside plastic hinge (psi) | vc<= 4√f'c | Vc_inside (kips) | Av used (in²) | Vs_inside (kips) | ΦVn (kips) | Mp (kips-in) | Mo (kips-in) | Vo (kips) | Capacity> Demand |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 2 | North | 215.554 | 215.55 | 2397.015 | 8.99 | 3202.418 | 4759.52 | 797567 | 957080.4 | 2624.44 | TRUE |
| | South | 215.765 | 215.77 | 3026.613 | 11.16 | 3975.415 | 5951.72 | 797567 | 957080.4 | 2624.44 | TRUE |
| 3 | North | 215.899 | 215.9 | 2125.776 | 8.06 | 2871.133 | 4247.37 | 792015 | 950418 | 2197.6 | TRUE |
| | South | 215.624 | 215.62 | 3037.669 | 11.16 | 3975.415 | 5961.12 | 792015 | 950418 | 2197.6 | TRUE |
| 4 | North | 215.648 | 215.65 | 2392.847 | 8.99 | 3202.418 | 4755.98 | 800685 | 960822 | 2341.18 | TRUE |
| | South | 215.813 | 215.81 | 3054.251 | 11.16 | 3975.415 | 5975.22 | 800685 | 960822 | 2341.18 | TRUE |

Outside Plastic Hinge

| Pier | Girder Location | $\upsilon c$ outside plastic hinge (psi) | $\upsilon c <= 4\sqrt{f'c}$ | $Vc$ outside (kips) | Av-used ($in^2$) | $Vs$ outside (kips) | $\Phi Vn$ (kips) | Mp (kips-in) | Mo (kips-in) | Vo (kips) | Capacity > Demand |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 2 | North | 215.554 | 215.55 | 2397.015 | 8.99 | 1601.209 | 3398.49 | 792489 | 950986.8 | 2607.73 | TRUE |
| | South | 215.765 | 215.77 | 3026.613 | 11.16 | 1987.708 | 4262.17 | 792489 | 950986.8 | 2607.73 | TRUE |
| 3 | North | 215.899 | 215.9 | 2125.776 | 8.06 | 1435.567 | 3027.14 | 787154 | 944584.8 | 2184.11 | TRUE |
| | South | 215.624 | 215.62 | 3037.669 | 11.16 | 1987.708 | 4271.57 | 787154 | 944584.8 | 2184.11 | TRUE |
| 4 | North | 215.648 | 215.65 | 2392.847 | 8.99 | 1601.209 | 3394.95 | 800685 | 960822 | 2341.18 | TRUE |
| | South | 215.813 | 215.81 | 3054.251 | 11.16 | 1987.708 | 4285.66 | 800685 | 960822 | 2341.18 | TRUE |

Equations:

$\rho s\_{inside\ plastic\ hinge} = (Ab*\#\ bars*w)/(L' * w' * s)$

$\mu D$ obtained from previous calculation.

$Ae = 0.8*Ag$

Inside Plastic Hinge

$\upsilon c = factor\ 1 * factor\ 2 * \sqrt{f'c} <= 4\sqrt{f'c}$

Outside Plastic Hinge

$\upsilon c = 3 * factor\ 2 * \sqrt{f'c} <= 4\sqrt{f'c}$

$factor\ 1 = 0.3 <= (\rho s*fyh)/0.15 + 3.67 - \mu d < 3$

$factor\ 2 = 1 + Pc/(2000*Ag) < 1.5$

$Vc = \upsilon c * Ae$

$Vs = ((Av*fyh*D')/s) <= 8\sqrt{f'c} * Ae$

$Av\text{-}min = 0.025 * D' * s / fyh$

$Av\text{-}used = Ab*\#bars$

$Vn = Vc + Vs$

$\Phi = 0.85$

$Vo = Mo/H$ ✓

$Mo = 1.2Mp$

## Shear Capacity in Transverse (Strong) Direction- Outside plastic hinge

period-transverse= 0.6016 from SAP2000
a=1.42g= 45.724

| Pier | Girder Location | m (k-s/ft) | Vu (kips) | Ac (in²) | ρh | υu (ksi) | υu < 8*√5 | ΦVn (kips) | Capacity>Demand |
|---|---|---|---|---|---|---|---|---|---|
| 2 | North | 27.5928 | 1261.7 | 11624.71 | 0.0021 | 4.595395 | 4.59539 | 45407.1 | TRUE |
| | South | 35.8918 | 1641.1 | 14706.72 | 0.0026 | 4.625147 | 4.62515 | 57817.6 | TRUE |
| 3 | North | 27.9254 | 1276.9 | 10273.92 | 0.0016 | 4.565319 | 4.56532 | 39868.2 | TRUE |
| | South | 38.5358 | 1762 | 14770.84 | 0.0022 | 4.601159 | 4.60116 | 57768.5 | TRUE |
| 4 | North | 29.6254 | 1354.6 | 11599.06 | 0.0018 | 4.581663 | 4.58166 | 45171.5 | TRUE |
| | South | 38.637 | 1766.6 | 14839.23 | 0.0023 | 4.608101 | 4.6081 | 58123.6 | TRUE |

Equations:
Vu= m*a
υu= 2*√f'c + ρh*fy < 8√f'c
ρh=horizontal shear reinforcement area/gross concrete area of vertical section
Ac=w'*L'
Vn= υu*Ac

Longitudinal Reinforcement Calculations

SDC 3.7.1

Max Longitudinal Reinforcement =0.04*Ag

SDC 3.7.2

Minimum Longitudinal Reinforcement =0.005 *Ag

| Pier | Girder Location | Height (ft) | Thicknes s (ft) | Ag (in²) | Ag max (in²) | Ag min (in²) | Ag max/ LF (in²) | Ag min/ LF (in²) |
|---|---|---|---|---|---|---|---|---|
| 2 | North | 30.39 | 3.5 | 15316.56 | 612.6624 | 76.5828 | 1.68 | 0.21 |
| | South | 30.39 | 3.5 | 15316.56 | 612.6624 | 76.5828 | 1.68 | 0.21 |
| 3 | North | 36.04 | 3.5 | 18164.16 | 726.5664 | 90.8208 | 1.68 | 0.21 |
| | South | 36.04 | 3.5 | 18164.16 | 726.5664 | 90.8208 | 1.68 | 0.21 |
| 4 | North | 34.2 | 3.5 | 17236.8 | 689.472 | 86.184 | 1.68 | 0.21 |
| | South | 34.2 | 3.5 | 17236.8 | 689.472 | 86.184 | 1.68 | 0.21 |

| | |
|---|---|
| Inside Plastic Hinge Zone using #5 @ 6" | Ag=0.62 in² |
| Outside Plastic Hinge Zone using #5 @12" | Ag=0.31 in² |

Ag is within limits.

Use WINFOUT to calculate the pile loads due to longitudinal moment. For Transverse moment, reduced pile loads, see hand calcs below.

Service I controls the Design of Pile Load, under the Service I, Transverse load at the bottom of the pier.

HL-93 Transverse shear 132 kips

132 x 4.5' (Depth of Ftg) / (226-4) = 2.67

HL-93 Transverse shear will (increase/decrease) pile load 2.67 ↑ K

stream force acting as shear in Transverse direction

stream force = 18 kips

18 x 4.5 / (226-4) = 0.36 kips

Use 5 kips for force in Transverse direction

Report to Geotech

Service Load

compression 104 + 5 = 109 say 110 kips

Strength Limit

compression 125 + 10 = 135

Extreme Limit state only at Pier 2 and Pier 4

From Pier 2 & Pier 4 Run under plastic moment at Longitudinal direction.

Max pile load. 119 kips

Max pile uplift 57 kips

Lateral load $V_p = \frac{M_p \times 1.2}{30.9} = 85^{kips}/6^{piles} = 14^{kips}/pile$

the maximum shear in transverse direction that the pile can take is the shear transver through Bolt + ½ of the pier wall mass

From SAP2000, South 2 Girders

$362 + 975 = 1337^{kips}$ (shear at Top)

½ of Pier wt

$w = 25 \times 3.5 \times (114.5 - 80.6) \times 0.15 \times 1/2 = 202^{kips}$ (30.9)

$ARS \times w = 0.6 \times 202 = 121$

Total shear $= 1337 + 121 = 1458^{kips}$

Total No of piles = 35

Lateral shear $1458/35 = 41.6^{kips}/pile$

(without considering PTG cap passive capacity)

Also, For entire structure, The Transverse force

735 kips × 12 girders = 8820 kips

½ of Pier Wall

$w = 226 \times 3.5 \times (111.5 - 80.5) \times 0.15 \times 1/2 = 1833$ kips

Total Shear = 8820 + 1833 = 10653 kip

$10653 \times 4.5 / (232 - 4) = 210$

$210/5 = 42$ kips compression or tension.

DL contribution per pile = 47 kip/pile. < 47 − 42 = 5

pile load in transverse direction not control.

Live Load. From SAP 2000 Model. LL Full Sec.

Strength I Live Load - (HL93 + IM) 33% IM

Strength II Live - (P15 + IM) 25% IM

Bottom of Pier (Pier 3)

| | HL93 | P15 |
|---|---|---|
| Max Longitudinal Moment | 4280 k-ft | 5300 k-ft |
| Longitudinal shear | 134 kips | 195 |
| Max transverse Moment | 3156 k-ft | 7695 k-ft |
| Transverse shear | 132 kips | 232 kips |
| Max P | 112 kip | 115 kip |

Input to WinFoot

$4280/226 \times 7.5 = 1.42$ - HL93

$5300/226 \times 7.5 = 1.76$ - P15

$W5/226 \doteq 1$

Transverse moment of live load is not realistic because pier wall is modelled as a "stick"

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Pier 2 & Pier 4
Ftg

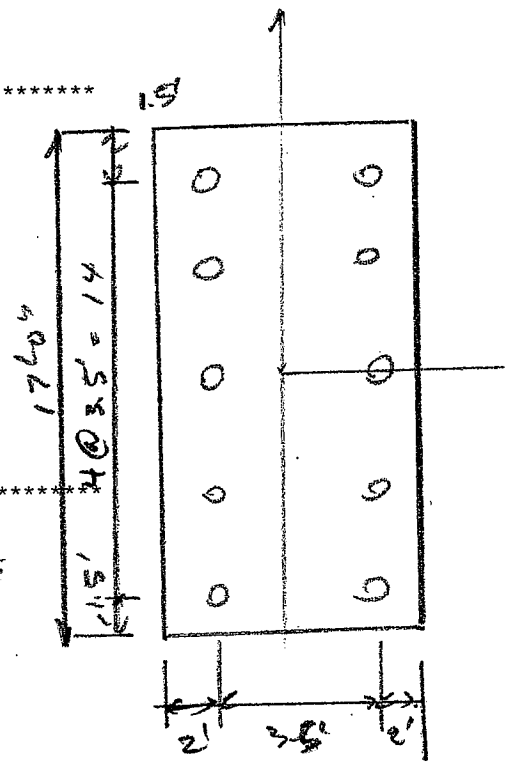
```
*************************************************************************
*                          ATTENTION                                    *
*                 Beta LRFD / Production LFD Specification              *
*                            WinFoot                                    *
*                 Pile and Spread Footing Design Program                *
*                 California Department of Transportation               *
*                   -- For Caltrans In House Use Only --                *
*           Copyright (c) Caltrans 2007, All Rights Reserved            *
*                          Version 3.5.12                               *
*                            03.30.10                                   *
*                                                                       *
*************************************************************************
```

InPut File Name:
D:\Frank Wei\Project\EA07-202100 - LA River Bridge\Pier\WinFoot\pier2-4_f

CLRFD = AASHTO LRFD Bridge Design Specifications, Fourth Edition 2007,
as Ammended by Caltrans, December 2008.

```
***************
* Column Data *
*************************************************************************
```

| | | |
|---|---|---|
| Column Width X-Dir (Tran) | = | 7.50 Ft |
| Column Depth Y-Dir (Long) | = | 3.50 Ft |
| Column Area | = | 26.25 Ft^2 |
| Equivalent Width X-Dir (Tran) | = | 7.50 Ft |
| Equivalent Depth Y-Dir (Long) | = | 3.50 Ft |
| H-Live Load Percent Impact | = | 33.00 Percent |
| P-Live Load Percent Impact | = | 25.00 Percent |
| End Condition about X-Axis | = | FIX |
| End Condition about Y-Axis | = | FIX |
| Number of Columns in Bent | = | 1 Columns |
| Design Specification | = | CLRFD |




```
****************
* Footing Data *
*************************************************************************
```

| | | |
|---|---|---|
| Type of Footing Analysis | = | CHECK |
| Footing Length X-Dir (Tran) | = | 7.50 Ft |
| Footing Width Y-Dir (Long) | = | 17.00 Ft |
| Min Footing Depth Considered | = | 4.50 Ft |
| Max Footing Depth Considered | = | 4.50 Ft |
| Type of Footing | = | PILE |

Area = 127.5

```
*******************************
* Footing Material Properties *
*************************************************************************
```

| | | | |
|---|---|---|---|
| Specified Concrete Compressive Strength | (fc) | = | 3250.0 Psi |
| Specified Rebar Minimum Yield Strength | (fy) | = | 60000.0 Psi |
| Unit Weight of Concrete | | = | 150.0 Pcf |
| Modular Ratio (Es/Ec) | (N) | = | 8.9 |
| Expected Concrete Compressive Strength | (fce) | = | 4000.0 Psi |
| Expected Rebar Yield Strength | (fye) | = | 68000.0 Psi |

Pier Wall
= 3.5' x 2.5'

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```
***********************************
* Top Flexure Reinforcement Data *
***********************************************************************

Number of Bars  X-Dir (Tran)  =   17
Number of Bars  Y-Dir (Long)  =    8
Bar Size in X-Dir (Tran)      =    6
Bar Size in Y-Dir (Long)      =    7
Top Bar Clearance             =    3.00 in
Direction of Top Bars         =    Y Direction

*************************************
* Bottom Flexure Reinforcement Data *
***********************************************************************

Number of Bars  X-Dir (Tran)  =   34
Number of Bars  Y-Dir (Long)  =    8
Bar Size in X-Dir (Tran)      =    9
Bar Size in Y-Dir (Long)      =    9
Bottom Bar Clearance          =    6.00 in
Direction of Bottom Bars      =    Y Direction

****************************
* Shear Reinforcement Data *
***********************************************************************

Stirrup Bar Size              =    5
Stirrup Spacing X-Dir (Tran)  =   12.00 in
Stirrup Spacing Y-Dir (Long)  =   12.00 in

*************
* Soil Data *
***********************************************************************

Original Ground Elevation     (OG) =    81.35 Ft
Final Grade Elevation         (FG) =    81.35 Ft
Bottom of Footing Elevation (BOF) =    76.10 Ft

Depth of Initial Soil Overburden  =     5.25 Ft
Depth of Final Soil Cover         =     0.75 Ft

Soil Unit Weight                  =   120.00 Pcf
LRFD Exposure Factor (Ge)         =     1.00

***************************
* Pile Footing Cost Data *
***********************************************************************

Contract Item                          Price  Unit
-------------                        -------- -----
Structure Excavation (Bridge)      =    45.00 $/Cy
Structure Backfill   (Bridge)      =    30.00 $/Cy
Structural Concrete Bridge Footing =   260.00 $/Cy
Bar Reinforcement (Bridge)         =     0.60 $/lb
Furnish Piling                     =    15.00 $/Ft
Drive Piles                        =  1100.00 $ Ea
```

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```
*************
* Pile Data *
*****************************************************************************

Total Number of Piles              =  10
Total Number of Rows               =   5
Diameter of Pile                   =   12.0 in
Capacity of Pile (Service)         =   70.0 Tons
Consider Tension in Piles?         =    YES
Length of One Pile                 =   40.0 Ft
Static  Tension/Compression Ratio  =   40.0 Percent
Dynamic Tension/Compression Ratio  =   50.0 Percent


********************
* Pile Layout Data *
*****************************************************************************
```

| Row No | No of Piles | Y-Coord (Ft) | X-Spacing (Ft) |
|---|---|---|---|
| 1 | 2 | 0.00 | 3.50 |
| 2 | 2 | 3.50 | 3.50 |
| 3 | 2 | -3.50 | 3.50 |
| 4 | 2 | 7.00 | 3.50 |
| 5 | 2 | -7.00 | 3.50 |

```
***********************************
* Moment of Inertia of Pile Group *
*****************************************************************************

Ixx =    245.00 Pile-Ft^2
Iyy =     30.63 Pile-Ft^2
```

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```
************************
* Pile Coordinates (Ft) *
************************************************************************
```

| Pile | X-Coord | Y-Coord |
|---|---|---|
| 1 | -1.75 | 7.00 |
| 2 | 1.75 | 7.00 |
| 3 | -1.75 | 3.50 |
| 4 | 1.75 | 3.50 |
| 5 | -1.75 | 0.00 |
| 6 | 1.75 | 0.00 |
| 7 | -1.75 | -3.50 |
| 8 | 1.75 | -3.50 |
| 9 | -1.75 | -7.00 |
| 10 | 1.75 | -7.00 |

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**********
* LRFD Limit State Loads - Unfactored (Kip, K-FT) *
**********

Seismic Analysis Specifications:
Seismic Zone = 3
Response Modification Factor (R/2) = 1.00

Note: Loads are Combined According To Their Sign.

| | Case-1 Maximum Transverse | | | | |
|---|---|---|---|---|---|
| | P-Truck PLL+IM | H-Truck HLL+IM | Lane Load LL | Centrifugal PC-Truck | Centrifugal HC-Truck |
| My-Tran | 0. | 0. | 0. | 0. | 0. |
| Mx-Long | 0. | 0. | 0. | 0. | 0. |
| P-Axial | 1. | 1. | 0. | 0. | 0. |

| | Case-2 Maximum Longitudinal | | | | |
|---|---|---|---|---|---|
| | P-Truck PLL+IM | H-Truck HLL+IM | Lane Load LL | Centrifugal PC-Truck | Centrifugal HC-Truck |
| My-Tran | 0. | 0. | 0. | 0. | 0. |
| Mx-Long | 176. | 142. | 0. | 0. | 0. |
| P-Axial | 1. | 1. | 0. | 0. | 0. |

← controlling loading case

| | Case-3 Maximum Axial | | | | |
|---|---|---|---|---|---|
| | P-Truck PLL+IM | H-Truck HLL+IM | Lane Load LL | Centrifugal PC-Truck | Centrifugal HC-Truck |
| My-Tran | 0. | 0. | 0. | 0. | 0. |
| Mx-Long | 0. | 0. | 0. | 0. | 0. |
| P-Axial | ~~1.35~~ | ~~1.20~~ | 0. | 0. | 0. |

↳ No effect to the final results because it is not controlling loading case

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**********
* LRFD Limit State Loads - Unfactored (Kip, K-FT) *
**********

Note: Loads are Combined According To Their Sign.

| | DC | DW | PS | PL | BR |
|---|---|---|---|---|---|
| | Dead Load | Added Dead Load | Prestress Sec Mom | Pedestrian Live Load | Braking Force |
| My-Tran | 0. | 0. | 0. | 0. | 0. |
| Mx-Long | 44. | 35. | 0. | 0. | 80. |
| P-Axial | 257. | 25. | 0. | 0. | 8. |

| | WA | WS-Wind Struc Horiz | | WL-Wind on Live Load | |
|---|---|---|---|---|---|
| | Stream Pressure | Case-1 Max Tran | Case-2 Max Long | Case-1 Max Tran | Case-2 Max Long |
| My-Tran | 0. | 0. | 0. | 0. | 0. |
| Mx-Long | 0. | 0. | 72. | 0. | 100. |
| P-Axial | 120. | 0. | 0. | 0. | 0. |

| | WV | FR | CR | SH | TU |
|---|---|---|---|---|---|
| | Wind Struc Vertical | Friction | Creep | Shrinkge | Temp Uniform |
| My-Tran | 0. | 0. | 0. | 0. | 0. |
| Mx-Long | 0. | 0. | 0. | 0. | 45. |
| P-Axial | 0. | 0. | 0. | 0. | 0. |

| | TG | SE | EQ | EQ |
|---|---|---|---|---|
| | Temp Gradient | Settlement | Case-1 Max Tran | Case-2 Max Long |
| My-Tran | 0. | 0. | 0. | 0. |
| Mx-Long | 0. | 0. | 0. | 0. |
| P-Axial | 0. | 0. | 0. | 0. |
| Pro Spc Factors | 0.00 | 0.00 | 0.00 | |

(EQ Load Factor: For H-Live Load,PL,BR)

| | IC Ice Collision | | LRFD |
|---|---|---|---|
| | Case-1 Max Tran | Case-2 Max Long | Arbritrary Load |
| My-Tran | 0. | 0. | 0. |
| Mx-Long | 0. | 0. | 0. |
| P-Axial | 0. | 0. | 0. |

| | Resistance Factors | |
|---|---|---|
| | Soil Phi | Piles Phi |
| Strength | 0.50 | 0.75 |
| Seismic | 1.00 | 1.00 |

Type of Superstructure = Non-Segmental
Column Moment of Inertia = I-gross

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```
*****************************************************
* Plastic Moments (1.3 X Nominal Moment)(Kip, K-FT) *
*****************************************************
```

(LRFD 3.10.9.4.3b)

| | Mp1 Max Tran | Mp2 75-Degree | Mp3 60-Degree | Mp4 45-Degree | Mp5 30-Degree | Mp6 15-Degree | Mp7 Max Long |
|---|---|---|---|---|---|---|---|
| MPy | 0. | 0. | 0. | 0. | 0. | 0. | 0. |
| MPx | 0. | 0. | 0. | 0. | 0. | 0. | 2206 ≒ 2187 |
| P | 0. | 0. | 0. | 0. | 0. | 0. | 216. |
| Vp | 0. | 0. | 0. | 0. | 0. | 0. | 80. |

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**********
* LRFD Load Factors for Footing Design *
**********
(LRFD Tables 3.4.1-1, 3.4.1-2)

Note: Loads are Combined According To Their Sign.

| Limit | :---DC--: | | :---DW--: | | :---PS--: | | :---EV--: | | :---CR--: | | :---SH--: | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| State | Max | Min | Max | Min | Max | Min | Max | Min | Max | Min | Max | Min |
| Str-I | 1.25 | 0.90 | 1.50 | 0.65 | 0.50 | 0.50 | 1.35 | 0.90 | 0.50 | 0.50 | 0.50 | 0.50 |
| Str-II | 1.25 | 0.90 | 1.50 | 0.65 | 0.50 | 0.50 | 1.35 | 0.90 | 0.50 | 0.50 | 0.50 | 0.50 |
| Str-III | 1.25 | 0.90 | 1.50 | 0.65 | 0.50 | 0.50 | 1.35 | 0.90 | 0.50 | 0.50 | 0.50 | 0.50 |
| Str-IV | 1.50 | 0.90 | 1.50 | 0.65 | 0.50 | 0.50 | 1.35 | 0.90 | 1.50 | 0.90 | 1.50 | 0.90 |
| Str-V | 1.25 | 0.90 | 1.50 | 0.65 | 0.50 | 0.50 | 1.35 | 0.90 | 0.50 | 0.50 | 0.50 | 0.50 |
| Ext-I | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Ext-II | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Ser-I | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Perm Ld | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |

| Limit State | PLL Trck | HLL Trck | HLL Lane | P-CE Cent | H-CE Cent | PL | BR |
|---|---|---|---|---|---|---|---|
| Str-I | 0.00 | 1.75 | 1.75 | 0.00 | 1.75 | 1.75 | 1.75 |
| Str-II | 1.35 | 0.00 | 0.00 | 1.35 | 0.00 | 0.00 | 0.00 |
| Str-III | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Str-IV | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Str-V | 0.00 | 1.35 | 1.35 | 0.00 | 1.35 | 1.35 | 1.35 |
| Ext-I | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Ext-II | 0.00 | 0.50 | 0.50 | 0.00 | 0.50 | 0.50 | 0.50 |
| Ser-I | 0.00 | 1.00 | 1.00 | 0.00 | 1.00 | 1.00 | 1.00 |
| Perm Ld | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |

| Limit State | WA | WS | WL | WV | FR | TU | TG | SE | EQ | IC | CT | CV |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Str-I | 1.00 | 0.00 | 0.00 | 0.00 | 1.00 | 0.50 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Str-II | 1.00 | 0.00 | 0.00 | 0.00 | 1.00 | 0.50 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Str-III | 1.00 | 1.40 | 0.00 | 1.40 | 1.00 | 0.50 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Str-IV | 1.00 | 0.00 | 0.00 | 0.00 | 1.00 | 0.50 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Str-V | 1.00 | 0.40 | 1.00 | 0.00 | 1.00 | 0.50 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Ext-I | 1.00 | 0.00 | 0.00 | 0.00 | 1.00 | 0.00 | 0.00 | 0.00 | 1.00 | 0.00 | 0.00 | 0.00 |
| Ext-II | 1.00 | 0.00 | 0.00 | 0.00 | 1.00 | 0.00 | 0.00 | 0.00 | 0.00 | 1.00 | 1.00 | 1.00 |
| Ser-I | 1.00 | 0.30 | 1.00 | 0.00 | 0.00 | 1.00 | 0.50 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Perm Ld | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |

**********
* Load Designations *
**********
(LRFD 3.2.2)

Permanent Loads
---
DC = Dead Load of Structural Components and Nonstructural Attachments (Dead Load)
DW = Dead Load of Wearing Surfaces and Utilities (Added Dead Load).
PS = Secondary Forces from Post-Tensioning.
CR = Creep.
SH = Shrinkage.

Transient Loads
---
PLL = Permit Live Load + IM
HLL = H-Design Truck/Tandem Live Load + IM.
LL = H-Design Lane Live Load (No IM).
P-CE = Centrifugal Permit Live Load (No IM).
H-CE = Centrifugal H-Design Truck/Tandem Live Load (No IM).
PL = Pedestrian Live Load. (No IM)
BR = Vehicular Braking Force (No IM).
WA = Water Load and Stream Pressure.
WS = Wind Load on Structure - Horizontal.
WL = Wind on Live Load (No IM).
WV = Wind Load on Structure - Vertical.
FR = Friction.
TU = Uniform Temperature.
TG = Temperature Gradient.
SE = Settlement Live Load.
EQ = Earthquake.
IC = Ice Collision.
CT = Vehicular (Truck or Train) Collision Force.
CV = Vessel Collision Force.
IM = Vehicular Dynamic Load Allowance (Percent Impact).

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**********
* Controlling Load Factor Combinations for Permanent Loads *
**********

1 = Max Load Factor Used
2 = Min Load Factor Used

| Limit State | Load Case | DC | DW | PS | EV | CR | SH | Load Factors DC | DW | PS | EV | CR | SH |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Str-I | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 1.25 | 1.50 | 0.50 | 1.35 | 0.50 | 0.50 |
| Str-I | 2 | 1 | 1 | 1 | 1 | 1 | 1 | 1.25 | 1.50 | 0.50 | 1.35 | 0.50 | 0.50 |
| Str-I | 3 | 1 | 1 | 1 | 1 | 1 | 1 | 1.25 | 1.50 | 0.50 | 1.35 | 0.50 | 0.50 |
| Str-I | 1a | 2 | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.90 | 0.50 | 0.50 |
| Str-I | 2a | 2 | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.90 | 0.50 | 0.50 |
| Str-I | 3a | 2 | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.90 | 0.50 | 0.50 |
| Str-II | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 1.25 | 1.50 | 0.50 | 1.35 | 0.50 | 0.50 |
| Str-II | 2 | 1 | 1 | 1 | 1 | 1 | 1 | 1.25 | 1.50 | 0.50 | 1.35 | 0.50 | 0.50 |
| Str-II | 3 | 1 | 1 | 1 | 1 | 1 | 1 | 1.25 | 1.50 | 0.50 | 1.35 | 0.50 | 0.50 |
| Str-II | 1a | 2 | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.90 | 0.50 | 0.50 |
| Str-II | 2a | 2 | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.90 | 0.50 | 0.50 |
| Str-II | 3a | 2 | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.90 | 0.50 | 0.50 |
| Str-III | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 1.25 | 1.50 | 0.50 | 1.35 | 0.50 | 0.50 |
| Str-III | 2 | 1 | 1 | 1 | 1 | 1 | 1 | 1.25 | 1.50 | 0.50 | 1.35 | 0.50 | 0.50 |
| Str-III | 3 | 1 | 1 | 1 | 1 | 1 | 1 | 1.25 | 1.50 | 0.50 | 1.35 | 0.50 | 0.50 |
| Str-III | 1a | 2 | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.90 | 0.50 | 0.50 |
| Str-III | 2a | 2 | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.90 | 0.50 | 0.50 |
| Str-III | 3a | 2 | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.90 | 0.50 | 0.50 |
| Str-IV | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 1.50 | 1.50 | 0.50 | 1.35 | 1.50 | 1.50 |
| Str-IV | 2 | 1 | 1 | 1 | 1 | 1 | 1 | 1.50 | 1.50 | 0.50 | 1.35 | 1.50 | 1.50 |
| Str-IV | 3 | 1 | 1 | 1 | 1 | 1 | 1 | 1.50 | 1.50 | 0.50 | 1.35 | 1.50 | 1.50 |
| Str-IV | 1a | 2 | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.90 | 0.90 | 0.90 |
| Str-IV | 2a | 2 | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.90 | 0.90 | 0.90 |
| Str-IV | 3a | 2 | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.90 | 0.90 | 0.90 |
| Str-V | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 1.25 | 1.50 | 0.50 | 1.35 | 0.50 | 0.50 |
| Str-V | 2 | 1 | 1 | 1 | 1 | 1 | 1 | 1.25 | 1.50 | 0.50 | 1.35 | 0.50 | 0.50 |
| Str-V | 3 | 1 | 1 | 1 | 1 | 1 | 1 | 1.25 | 1.50 | 0.50 | 1.35 | 0.50 | 0.50 |
| Str-V | 1a | 2 | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.90 | 0.50 | 0.50 |
| Str-V | 2a | 2 | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.90 | 0.50 | 0.50 |
| Str-V | 3a | 2 | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.90 | 0.50 | 0.50 |
| Ext-I | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Ext-I | 2 | 1 | 1 | 1 | 1 | 1 | 1 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Ext-II | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Ext-II | 2 | 1 | 1 | 1 | 1 | 1 | 1 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Ext-II | 3 | 1 | 1 | 1 | 1 | 1 | 1 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Ext-II | 1a | 2 | 2 | 2 | 2 | 2 | 2 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Ext-II | 2a | 2 | 2 | 2 | 2 | 2 | 2 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Ext-II | 3a | 2 | 2 | 2 | 2 | 2 | 2 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Ser-I | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Ser-I | 2 | 1 | 1 | 1 | 1 | 1 | 1 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Ser-I | 3 | 1 | 1 | 1 | 1 | 1 | 1 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Perm Ld | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |

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**********
* NET TOTAL Service Loads for Settlement Check (Kip, K-Ft) *
**********

| Limit State | Case | Applied Loads Tran My | Long Mx | Comb M | Axial Pnet | Min Pile Load | Max Pile Load |
|---|---|---|---|---|---|---|---|
| Ser-I | 1 | 0. | 204. | 204. | 426. | 37. | 48. |
| Ser-I | 2 | 0. | 432. | 432. | 426. | 30. | 55. |
| Ser-I | 3 | 0. | 326. | 326. | 426. | 33. | 52. |

506
− 80
426

Note:
Pnet = Pgross − Poverburden.
Poverburden = 80. Kip. ✓

(81.35−76.10) × 7.5 × 17 × 0.12 = 80 Kips ✓

Total = (257 + 25 + 95) + 8 + 120 + 1 = 506
Permanent Transient

**********
* NET PERMANENT Service Loads for Settlement Check (Kip, K-Ft) *
**********

| Limit State | Case | Applied Loads Tran My | Long Mx | Comb M | Axial Pnet | Min Pile Load | Max Pile Load |
|---|---|---|---|---|---|---|---|
| Perm Ld | 1 | 0. | 79. | 79. | 297. | 27. | 32. |

257 + 25 + 95 − 80
= 297 Kips ✓

Note: Limit State "Perm Ld" = Permanent Loads Only.

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```
*************************************************************
* GROSS Service Loads (Ser) for Footing Design (Kip, K-Ft) *
*************************************************************
```

Note: Ratio = Capacity (Comp or Uplift) / (Max or Min) Pile Load

| Cap Uplift | Cap Comp |
|---|---|
| -56. | 140. |

| Limit State | Case | Applied Loads Tran My | Long Mx | Comb M | Axial Pgross | Min Pile Load | Ratio | Max Pile Load | Ratio | |
|---|---|---|---|---|---|---|---|---|---|---|
| Ser-I | 1 | 0. | 204. | 204. | 506. | 45. | | 56. | 2.48 | OK |
| Ser-I | 2 | 0. | 432. | 432. | 506. | 38. | | 63. | 2.22 | OK |
| Ser-I | 3 | 0. | 326. | 326. | 506. | 41. | | 60. | 2.34 | OK |

The Following are Incluced in the Applied Loads Above:

Weight of Cover = 9. Kip ✓
Weight of Footing = 86. Kip
Total Weight = 95. Kip

65+5=70 kips
↑
Transverse Contribution

For Limit State Ser-I Case 2 The Pile Loads at Each Corner Are:

| | |
|---|---|
| 63. | 63. |
| 38. | 38. |

Note: Pile Structural Capacity Check for Ser-I Limit State is not Required per LRFD Code.
However, Check is Performed for Information Only.

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**********
* Factored Loads (Fac) for Footing Design (Kip, K-Ft) *
**********

Note: Ratio = Capacity (Comp or Uplift) / (Max or Min) Pile Load

Phi = 0.75

| Cap Uplift | Cap Comp |
|---|---|
| -84. | 210. |

| Limit State | Case | Applied Loads Tran My | Long Mx | Comb M | Axial Pgross | Min Pile Load | Ratio | Max Pile Load | Ratio | |
|---|---|---|---|---|---|---|---|---|---|---|
| Str-I | 1 | 0. | 270. | 270. | 614 | 54. | | 69. | 3.04 | OK |
| Str-I | 2 | 0. | 457. | 457. | 614 | 48. | | 74 | 2.82 | OK |
| Str-I | 3 | 0. | 270. | 270. | 614 | 54. | | 69. | 3.04 | OK |
| Str-I | 1a | 0. | 225. | 225. | 469. | 40. | | 53. | 3.94 | OK |
| Str-I | 2a | 0. | 412. | 412. | 469. | 35. | | 59. | 3.58 | OK |
| Str-I | 3a | 0. | 225. | 225. | 469. | 40. | | 53. | 3.94 | OK |
| Str-II | 1 | 0. | 130. | 130. | 600. | 56. | | 64. | 3.30 | OK |
| Str-II | 2 | 0. | 320. | 320. | 600. | 51. | | 69. | 3.04 | OK |
| Str-II | 3 | 0. | 130. | 130. | 600. | 56. | | 64. | 3.30 | OK |
| Str-II | 1a | 0. | 85. | 85. | 454. | 43. | | 48. | 4.39 | OK |
| Str-II | 2a | 0. | 275. | 275. | 454. | 38. | | 53. | 3.94 | OK |
| Str-II | 3a | 0. | 85. | 85. | 454. | 43. | | 48. | 4.39 | OK |
| Str-III | 1 | 0. | 130. | 130. | 599. | 56. | | 64. | 3.30 | OK |
| Str-III | 2 | 0. | 231. | 231. | 599. | 53. | | 66. | 3.16 | OK |
| Str-III | 3 | 0. | 231. | 231. | 599. | 53. | | 66. | 3.16 | OK |
| Str-III | 1a | 0. | 85. | 85. | 453. | 43. | | 48. | 4.40 | OK |
| Str-III | 2a | 0. | 186. | 186. | 453. | 40. | | 51. | 4.15 | OK |
| Str-III | 3a | 0. | 186. | 186. | 453. | 40. | | 51. | 4.15 | OK |
| Str-IV | 1 | 0. | 141. | 141. | 684. | 64. | | 72. | 2.90 | OK |
| Str-IV | 2 | 0. | 141. | 141. | 684. | 64. | | 72. | 2.90 | OK |
| Str-IV | 3 | 0. | 141. | 141. | 684. | 64. | | 72. | 2.90 | OK |
| Str-IV | 1a | 0. | 85. | 85. | 453. | 43. | | 48. | 4.40 | OK |
| Str-IV | 2a | 0. | 85. | 85. | 453. | 43. | | 48. | 4.40 | OK |
| Str-IV | 3a | 0. | 85. | 85. | 453. | 43. | | 48. | 4.40 | OK |
| Str-V | 1 | 0. | 238. | 238. | 610 | 54. | | 68. | 3.10 | OK |
| Str-V | 2 | 0. | 511. | 511. | 610 | 46. | | 76. | 2.78 | OK |
| Str-V | 3 | 0. | 367. | 367. | 610 | 51. | | 72. | 2.94 | OK |
| Str-V | 1a | 0. | 193. | 193. | 465. | 41. | | 52. | 4.04 | OK |
| Str-V | 2a | 0. | 466. | 466. | 465. | 33. | | 60. | 3.51 | OK |
| Str-V | 3a | 0. | 322. | 322. | 465. | 37. | | 56. | 3.77 | OK |
| Ext-II | 1 | 0. | 119. | 119. | 502. | 47. | | 54. | 3.92 | OK |
| Ext-II | 2 | 0. | 172. | 172. | 502. | 45. | | 55. | 3.81 | OK |
| Ext-II | 3 | 0. | 119. | 119. | 502. | 47. | | 54. | 3.92 | OK |
| Ext-II | 1a | 0. | 119. | 119. | 502. | 47. | | 54. | 3.92 | OK |
| Ext-II | 2a | 0. | 172. | 172. | 502. | 45. | | 55. | 3.81 | OK |
| Ext-II | 3a | 0. | 119. | 119. | 502. | 47. | | 54. | 3.92 | OK |
| Arb-I | 1 | 0. | 0. | 0. | 0. | 0. | | 0. | 0.00 | OK |

76+10=86 kips

The Following are Incluced in the Applied Loads Above:

Weight of Cover = 9. Kip
Weight of Footing = 86. Kip
Total Weight = 95. Kip

For Limit State Str-V Case 2 The Pile Loads at Each Corner Are:

| | |
|---|---|
| 76. | 76. |
| 46. | 46. |

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```
*****************************************************
* Earthquake Loads (Eq) for Footing Design (Kip, K-Ft) *
*****************************************************************************
```

Note: Ratio = Capacity (Comp or Uplift) / (Max or Min) Pile Load

ARS Unreduced Seismic (VII) & Overstrength Demand Moments (Md)

Phi = 1.00

| Cap Uplift | Cap Comp |
|---|---|
| -140. | 280. |

| Limit State | Case | Applied Loads Tran My | Long Mx | Comb M | Axial Pgross | Min Pile Load | Ratio | Max Pile Load | Ratio | |
|---|---|---|---|---|---|---|---|---|---|---|
| Ext-I | 1 | 0. | 0. | 0. | 95. | | | | | NA |
| Ext-I | 2 | 0. | 0. | 0. | 95. | | | | | NA |
| Md1 | 1 | 0. | 0. | 0. | 95. | 0. | | 0. | 0.00 | OK |
| Md2 | 1 | 0. | 0. | 0. | 95. | 0. | | 0. | 0.00 | OK |
| Md3 | 1 | 0. | 0. | 0. | 95. | 0. | | 0. | 0.00 | OK |
| Md4 | 1 | 0. | 0. | 0. | 95. | 0. | | 0. | 0.00 | OK |
| Md5 | 1 | 0. | 0. | 0. | 95. | 0. | | 0. | 0.00 | OK |
| Md6 | 1 | 0. | 0. | 0. | 95. | 0. | | 0. | 0.00 | OK |
| Md7 | 1 | 0. | 3079. | 3079. | 311. | -57. | 2.46 | 119. | 2.35 | OK |

The Following are Incluced in the Applied Loads Above:

Weight of Cover = 9. Kip
Weight of Footing = 86. Kip
Total Weight = 95. Kip

For Limit State Md7 Case 1 The Pile Loads at Each Corner Are:

| | |
|---|---|
| 119. | 119. |
| -57. | -57. |

Calculation of Footing Overstrength Demand Moments - Md:

Md = Mo + (Vo x D) + (Np x Mpile)

Md = Footing Demand Moment
Mp = Column Plastic Moment
Vp = Column Plastic Shear
Mo = Column Overstrength Moment = 1.20 x Mp
Vo = Column Overstrength Shear = 1.20 x Vp
D = Foooting Depth
Np = Number of Piles
Mpile = Moment at Top of Pile = 0.

Controlling Nominal Resistance

Compression 119. Kip (Limit State Md7 Case 1 )
Tension -57. Kip (Limit State Md7 Case 1 )

```
*****************************************
* Design of Bottom Flexural Reinforcement *
*************************************************************************
```

Key to Types of Loads:

```
Ser-I    = Gross Service Load - Crack Control (LRFD 5.7.3.4)
Fac      = Factored Loads, Variable Phi
 Str-I   = H-Loads
 Str-II  = P-Loads
 Str-III = No H-Loads, Wind > 55 mph
 Str-IV  = No H-Loads, Hign DL to LL Ratio
 Str-V   = H-Loads, Wind=55 mph
 Ext-II  = Ice/Train,Vehicle/Vessel Collision
 Arb-I   = Arbritrary User Defined Load
Eq       = Ext-I & Plastic Earthquake Loads
Min1     = Minimum Reinforcement (LRFD 5.7.3.3.2)
           Based on 1.2 X The Cracking Moment
Min2     = Waiver of Minimum Reinforcement (LRFD 5.7.3.3.2)
           Based on Steel Area One Third Greater than Required by Analysis
```

Footing Depth = 4.50 Ft

| Bar Size | Bar Dir | Load Type | Limit State | Ca | Depth Effective (in) | Applied Moment (K-Ft) | Phi | Max Rebar Strain | Req Area of Steel (in^2) | No Bars |
|---|---|---|---|---|---|---|---|---|---|---|
| 9 | X | Ser | | | 46.13 | 0. | | | 0.0 | 0 |
| 9 | Y | Ser | Ser-I | 2 | 47.38 | 729. | | | 8.3 | 9 |
| 9 | X | Fac | | | 46.13 | 0. | 0.900 | 0.00000 | 0.0 | 0 |
| 9 | Y | Fac | Str-V | 2 | 47.38 | 868. | 0.900 | 0.11857 | 4.1 | 5 |
| 9 | X | Eq | | | 46.13 | 0. | 1.000 | 0.00000 | 0.0 | 0 |
| 9 | Y | Eq | Md7 | 1 | 47.38 | 1383. | 1.000 | 0.11510 | 5.2 | 6 |
| 9 | X | Min1 | | | 46.13 | 6613. | 0.900 | 0.03032 | 33.1 | 34 |
| 9 | Y | Min1 | | | 47.38 | 2917. | 0.900 | 0.03223 | 14.2 | 15 |
| 9 | X | Min2 | | | 46.13 | 0. | 0.900 | 0.00000 | 0.0 | 0 |
| 9 | Y | Min2 | | | 47.38 | 1224. | 0.750 | -0.00317 | 6.9 | 7 |

********** CONTROL **********

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 9 | X | Min1 | | | 46.13 | 6613. | | | 33.1 | 34 |
| 9 | Y | Ser | Ser-I | 2 | 47.38 | 729. | | | 8.3 | 9 |

Note: (LRFD 5.13.3.5)
In the short Y-direction, ( 5.1 in^2) of reinforcemtment shall be distributed uniformly over a band width equal to the length of short side of the footing ( 7.5 Ft) and centered on the column.
The remainder of the reinforcement in the short Y-direction, ( 3.2 in^2) shall be distributed uniformly outside the center band width of the footing.

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```
****************************************
* Design of Top Flexural Reinforcement *
****************************************************************************
```

Key to Types of Loads:

```
Ser-I    = Gross Service Load - Crack Control (LRFD 5.7.3.4)
Fac      = Factored Loads, Variable Phi
 Str-I   = H-Loads
 Str-II  = P-Loads
 Str-III = No H-Loads, Wind > 55 mph
 Str-IV  = No H-Loads, Hign DL to LL Ratio
 Str-V   = H-Loads, Wind=55 mph
 Ext-II  = Ice/Train,Vehicle/Vessel Collision
 Arb-I   = Arbritrary User Defined Load
Eq       = Ext-I & Plastic Earthquake Loads
Min1     = Minimum Reinforcement (LRFD 5.7.3.3.2)
           Based on 1.2 X The Cracking Moment
Min2     = Waiver of Minimum Reinforcement (LRFD 5.7.3.3.2)
           Based on Steel Area One Third Greater than Required by Analysis
Min3     = Minimum Reinforcement (Shrinkage & Temperature) (LRFD 5.10.8)
```

Footing Depth = 4.50 Ft

| Bar Size | Bar Dir | Load Type | Limit State | Ca | Depth Effective (in) | Applied Moment (K-Ft) | Phi | Max Rebar Strain | Req Area of Steel (in^2) | No Bars |
|---|---|---|---|---|---|---|---|---|---|---|
| 6 | X | Ser | | | 49.56 | 0. | | | 0.0 | 0 |
| 7 | Y | Ser | | | 50.50 | 0. | | | 0.0 | 0 |
| 6 | X | Fac | | | 49.56 | 0. | 0.900 | 0.00000 | 0.0 | 0 |
| 7 | Y | Fac | | | 50.50 | 0. | 0.900 | 0.00000 | 0.0 | 0 |
| 6 | X | Eq | Md7 | 1 | 49.56 | 0. | 1.000 | 0.00000 | 0.0 | 0 |
| 7 | Y | Eq | Md7 | 1 | 50.50 | 773. | 1.000 | 0.20013 | 2.7 | 5 |
| 6 | X | Min1 | | | 49.56 | 6613. | 0.900 | 0.03568 | 30.7 | 70 |
| 7 | Y | Min1 | | | 50.50 | 2917. | 0.900 | 0.03722 | 13.3 | 23 |
| 6 | X | Min2 | | | 49.56 | 0. | 0.900 | 0.00000 | 0.0 | 0 |
| 7 | Y | Min2 | | | 50.50 | 683. | 0.750 | -0.00318 | 3.6 | 7 |
| 6 | X | Min3 | | | 49.56 | 1751. | 0.900 | 0.14687 | 7.9 | 18 |
| 7 | Y | Min3 | | | 50.50 | 1078. | 0.900 | 0.10817 | 4.8 | 8 |

********** CONTROL **********

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 6 | X | Min3 | | | 49.56 | 1751. | | | 7.9 | 18 |
| 7 | Y | Min3 | | | 50.50 | 1078. | | | 4.8 | 8 |

Note: (LRFD 5.13.3.5)
In the short Y-direction, ( 2.9 in^2) of reinforcemtment shall be distributed uniformly over a band width equal to the length of short side of the footing ( 7.5 Ft) and centered on the column.
The remainder of the reinforcement in the short Y-direction, ( 1.9 in^2) shall be distributed uniformly outside the center band width of the footing.

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```
************************************
* Final Results for Footing CHECK *
***************************************************************************

Footing Length =    7.50 Ft
Footing Width  =   17.00 Ft
Input Depth    =    4.50 Ft

*********************
* Pile Load Results *
***************************************************************************

     Control      Min Pile  Capacity        Control      Max Pile  Capacity
Load Limit           Load    Uplift        Limit            Load      Comp
Type State   Ca     (Kip)     (Kip)        State    Ca     (Kip)     (Kip)
---- ------- -- --------- ---------        -------- -- --------- ---------
Ser  Ser-I   2        38.      -56. OK     Ser-I    2        63.       140. OK
Fac  Str-V   2a       33.      -84. OK     Str-V    2        76.       210. OK
Eq   Md7     1       -57.     -140. OK     Md7      1       119.       280. OK

*****************
* Shear Results *
***************************************************************************
(LRFD 5.8.3.3, 5.8.3.4.1 Simplified Procedure)

Ratio = Shear Capacity/Shear Applied

Phi            =     0.90
Footing Depth  =     4.50 Ft
Perimeter (Bo) =     0.00 Ft

     Control              Depth          Shear            Shear
Load Limit             Effective        Applied         Capacity   Ratio
Type State   Ca            (in)          (Kip)           (Kip)
---- ------- --     ------------    -----------     ------------   -----
Fac                 Dvy =   42.6    VY =    128.    VUY =    840.   6.56 OK
Eq                  Dvy =   42.6    VY =    220.    VUY =    840.   3.82 OK

Note: Effective Shear Depth "Dvx" & "Dvy" is Taken as the Greater of (LRFD 5.8.2.9):
      .72h  = .72 x Footing   Depth
      .90dx = .90 x Effective Depth for Bars in "X" Transverse  Direction.
      .90dy = .90 x Effective Depth for Bars in "Y" Longitudinal Direction.

Note: Dvp = (Dvx + Dvy) / 2

* Note: The Shear Plane at Distance "dx" is Outside the Footing Dimesnion.
   Therefore, NO (Vx) or (Vp) Shear Calculations are Made.

************************************
* Top and Bottom Flexure Results *
***************************************************************************
Footing Depth =     4.50 Ft

                  Control       Eff   Applied     Bar    Area
Bar    Bar Load Limit        Depth    Moment   Space     Req Bars Bars      Error
Size   Dir Type State    Ca   (in)    (K-Ft)    (in)  (in^2)  Req InPut     Code
----   --- ---- -------  -- ------ --------- ------- ------- ---- -----     -----
Top
  6    X   Min3                49.6     1751.    12.3     7.9   18    17 NG**    4
  7    Y   Min3                50.5     1078.    11.9     4.8    8     8 OK

Bottom
  9    X   Min1                46.1     6613.     6.0    33.1   34    34 OK      4
  9    Y   Ser  Ser-I   2     47.4      729.    11.8     8.3    9     8 NG**
```

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```
*********************************
* Bottom Flexural Reinforcement *
*********************************************************************************
```

(LRFD 5.3.7.4)

(Area of Steel Required for Different Bar Sizes)

| Bar Size | Bar Dir | Cont Load Type | Actual Bar Stress (Psi ) | Acutal Bar Space (in) | Allow Bar Space (in) | Eff Depth (in) | Req Steel (in^2) | No Bars | Foot Depth (Ft) | Error Code |
|---|---|---|---|---|---|---|---|---|---|---|
| 5 | X | Min1 | 0. | 1.90 | 8.22 | 46.97 | 32.5 | 105 | 4.50 | 14 |
| 5 | Y | Min2 | 27170. | 3.79 | 8.96 | 47.66 | 6.9 | 23 | | |
| 6 | X | Min1 | 0. | 2.66 | 12.00 | 46.68 | 32.7 | 75 | 4.50 | 14 |
| 6 | Y | Min2 | 27565. | 5.54 | 8.40 | 47.56 | 6.9 | 16 | | |
| 7 | X | Min1 | 0. | 3.65 | 12.00 | 46.50 | 32.8 | 55 | 4.50 | 24 |
| 7 | Y | Ser | 27003. | 7.55 | 8.68 | 47.50 | 6.9 | 12 | | |
| 8 | X | Min1 | 0. | 4.80 | 12.00 | 46.31 | 33.0 | 42 | 4.50 | 4 |
| 8 | Y | Ser | 24704. | 9.21 | 10.53 | 47.44 | 7.6 | 10 | | |
| 9 | X | Min1 | 0. | 5.96 | 12.00 | 46.13 | 33.1 | 34 | 4.50 | 4 |
| 9 | Y | Ser | 21791. | 10.34 | 13.52 | 47.38 | 8.3 | 9 | | |
| 10 | X | Min1 | 0. | 7.56 | 12.00 | 45.84 | 33.4 | 27 | 4.50 | 4 |
| 10 | Y | Ser | 19411. | 11.79 | 16.54 | 47.28 | 9.1 | 8 | | |
| 11 | X | Min1 | 0. | 9.35 | 12.00 | 45.56 | 33.6 | 22 | 4.50 | 4 |
| 11 | Y | Ser | 18137. | 13.73 | 18.36 | 47.19 | 9.9 | 7 | | 3 |
| 14 | X | Min1 | 0. | 13.07 | 12.00 | 45.18 | 33.9 | 16 | 4.50 | 34 |
| 14 | Y | Ser | 14811. | 16.42 | 25.16 | 47.06 | 11.6 | 6 | | 3 |
| 18 | X | Min1 | 0. | 24.44 | 12.00 | 44.25 | 34.7 | 9 | 4.50 | 34 |
| 18 | Y | Ser | 12657. | 27.17 | 30.77 | 46.75 | 15.3 | 4 | | 34 |

Note: The Designer must Check the Error Code for Adaquate Development Length and Bar Spacing.

Error Codes:
---

1 = Bar Spacing LESS than The AASHTO Minimum. (LRFD 5.10.3.1)
2 = Bar Spacing LESS than The Preferred CALTRANS Minimum. (Br. Des. Details)
3 = Bar Spacing MORE than The AASHTO Maximum (LRFD 5.10.8)
4 = Inadequate Development Length. (LRFD 5.11.2.1)

```
******************************
* Top Flexural Reinforcement *
******************************************************************************
```

(LRFD 5.3.7.4)

(Area of Steel Required for Different Bar Sizes)

| Bar Size | Bar Dir | Cont Load Type | Actual Bar Stress (Psi ) | Acutal Bar Space (in) | Allow Bar Space (in) | Eff Depth (in) | Req Steel (in^2) | No Bars | Foot Depth (Ft) | Error Code |
|---|---|---|---|---|---|---|---|---|---|---|
| 4 | X | Min3 | 0. | 5.20 | 5.20 | 50.16 | 7.8 | 39 | 4.50 | 4 |
| 4 | Y | Min2 | 0. | 4.64 | 6.95 | 50.72 | 3.6 | 19 | | |
| 5 | X | Min3 | 0. | 8.22 | 8.22 | 49.97 | 7.8 | 25 | 4.50 | 4 |
| 5 | Y | Min2 | 0. | 7.57 | 10.41 | 50.66 | 3.6 | 12 | | |
| 6 | X | Min3 | 0. | 11.60 | 12.00 | 49.68 | 7.9 | 18 | 4.50 | 4 |
| 6 | Y | Min2 | 0. | 10.39 | 12.00 | 50.56 | 3.6 | 9 | | |
| 7 | X | Min3 | 0. | 11.59 | 12.00 | 49.50 | 10.8 | 18 | 4.50 | 4 |
| 7 | Y | Min3 | 0. | 11.86 | 12.00 | 50.50 | 4.8 | 8 | | |

Note: The Designer must Check the Error Code for Adaquate Development Length and Bar Spacing.

Error Codes:
---
1 = Bar Spacing LESS than The AASHTO Minimum. (LRFD 5.10.3.1)
2 = Bar Spacing LESS than The Preferred CALTRANS Minimum. (Br. Des. Details)
3 = Bar Spacing MORE than The AASHTO Maximum (LRFD 5.10.8)
4 = Inadequate Development Length. (LRFD 5.11.2.1)

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**********
* Minimum, Preferred C. to C. Bar Spacing & Development Length *
**********

Assume Maximum Coarse Size Aggregate = 1.5 in

The Clear Distance between Parallel Bars Shall NOT be less than
1.5 Bar Diameters (Nominal), 1.5 Times the Maximum Size Course Aggregate,
nor 1.5 inches. (LRFD 5.10.3.1)

Preferred Center to Center Spacing Based on Bridge Design Details Page 13-20

| Bar Size | Bar Area (in^2) | Nominal Diameter (in) | Min Clear Distance (in) | Deformed Diameter (in) | Minimum Spacing (in) | Preferred Spacing (in) | Development Length (in) Spc<6" | Spc>=6" |
|---|---|---|---|---|---|---|---|---|
| 4 | .20 | .500 | 2.250 | .563 | 2.813 | 3.250 | 12 | 12 |
| 5 | .31 | .625 | 2.250 | .688 | 2.938 | 3.500 | 15 | 12 |
| 6 | .44 | .750 | 2.250 | .875 | 3.125 | 3.750 | 19 | 15 |
| 7 | .60 | .875 | 2.250 | 1.000 | 3.250 | 4.000 | 26 | 20 |
| 8 | .79 | 1.000 | 2.250 | 1.125 | 3.375 | 4.250 | 34 | 27 |
| 9 | 1.00 | 1.128 | 2.250 | 1.250 | 3.500 | 4.500 | 42 | 34 |
| 10 | 1.27 | 1.270 | 2.250 | 1.438 | 3.688 | 4.750 | 54 | 43 |
| 11 | 1.56 | 1.410 | 2.250 | 1.625 | 3.875 | 5.000 | 66 | 53 |
| 14 | 2.25 | 1.693 | 2.539 | 1.875 | 4.414 | 5.500 | 90 | 72 |
| 18 | 4.00 | 2.257 | 3.386 | 2.500 | 5.886 | 6.000 | 116 | 93 |

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**********
* Bar Reinforcing Steel *
**********

| Bar Type | Bar Size | No Bar | Length (Ft) | Weight (Lb/Ft) | Weight (Lb) |
|---|---|---|---|---|---|
| Top | | | | | |
| X-Bar | 6 | 17 | 7.0 | 1.502 | 179. |
| Y-Bar | 7 | 8 | 16.5 | 1.502 | 270. |
| Bottom | | | | | |
| X-Bar | 9 | 34 | 7.0 | 3.400 | 809. |
| Y-Bar | 9 | 8 | 16.5 | 3.400 | 449. |
| Stirrup | 5 | 202 | 5.1 | 1.043 | 1071. |

Note: Stirrups are Required in a Band Extending 6 inches
Maximum from The Column Reinforcement to
44.3 inches from The Face of The Column in The "X" Direction and
46.8 inches from The Face of The Column in The "Y" Direction.

**********
* Quantities and Estimated Cost - Pile Footing *
**********

| Contract Item | Unit | Quantity | Price | Amount |
|---|---|---|---|---|
| Structure Excavation (Br) | Cy | 35.1 | 45.00 | 1579. |
| Structure Backfill (Br) | Cy | 13.1 | 30.00 | 394. |
| Structure Concrete, Br Ftg | Cy | 21.3 | 260.00 | 5525. |
| Bar Reinforcement (Br) | Lb | 2777.5 | 0.60 | 1667. |
| Furnish Piling | Ft | 400.0 | 15.00 | 6000. |
| Drive Piles | Ea | 10 | 1100.00 | 11000. |
| Total Cost | | | | 26164. |

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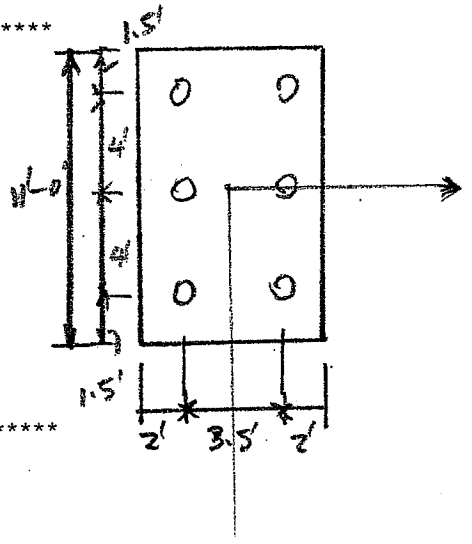
Pier 3 Ftg.

```
*************************************************************************
*                          ATTENTION                                    *
*                Beta LRFD / Production LFD Specification               *
*                            WinFoot                                    *
*                Pile and Spread Footing Design Program                 *
*                California Department of Transportation                *
*                  -- For Caltrans In House Use Only --                 *
*           Copyright (c) Caltrans 2007, All Rights Reserved            *
*                          Version 3.5.12                               *
*                            03.30.10                                   *
*                                                                       *
*************************************************************************
InPut File Name:
D:\Frank Wei\Project\EA07-202100 - LA River Bridge\Pier\WinFoot\pier3_fra

CLRFD = AASHTO LRFD Bridge Design Specifications, Fourth Edition 2007,
        as Ammended by Caltrans, December 2008.

***************
* Column Data *
*************************************************************************

Column Width X-Dir (Tran)        =     7.50 Ft
Column Depth Y-Dir (Long)        =     3.50 Ft
Column Area                      =    26.25 Ft^2
Equivalent Width X-Dir (Tran)    =     7.50 Ft
Equivalent Depth Y-Dir (Long)    =     3.50 Ft
H-Live Load Percent Impact       =    33.00 Percent
P-Live Load Percent Impact       =    25.00 Percent
End Condition about X-Axis       =      FIX
End Condition about Y-Axis       =      FIX
Number of Columns in Bent        =        1 Columns
Design Specification             = CLRFD

****************
* Footing Data *
*************************************************************************

Type of Footing Analysis         =  CHECK
Footing Length X-Dir (Tran)      =     7.50 Ft
Footing Width  Y-Dir (Long)      =    11.00 Ft
Min Footing Depth Considered     =     3.00 Ft
Max Footing Depth Considered     =     3.00 Ft
Type of Footing                  =   PILE

*******************************
* Footing Material Properties *
*************************************************************************

Specified Concrete Compressive Strength (fc) =   3250.0 Psi
Specified Rebar Minimum Yield Strength  (fy) =  60000.0 Psi
Unit Weight of Concrete                      =    150.0 Pcf
Modular Ratio (Es/Ec)                   (N)  =      8.9

Expected Concrete Compressive Strength (fce) =   4000.0 Psi
Expected Rebar Yield Strength          (fye) =  68000.0 Psi
```

Pier wall width
3.5' x 7.5'



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```
************************************
* Top Flexure Reinforcement Data *
**************************************************************************

Number of Bars  X-Dir (Tran)  =   12
Number of Bars  Y-Dir (Long)  =    8
Bar Size in X-Dir (Tran)      =    6
Bar Size in Y-Dir (Long)      =    6
Top Bar Clearance             =    3.00 in
Direction of Top Bars         =    Y Direction

**************************************
* Bottom Flexure Reinforcement Data *
**************************************************************************

Number of Bars  X-Dir (Tran)  =   12
Number of Bars  Y-Dir (Long)  =    8
Bar Size in X-Dir (Tran)      =    8
Bar Size in Y-Dir (Long)      =    9
Bottom Bar Clearance          =    6.00 in
Direction of Bottom Bars      =    Y Direction

****************************
* Shear Reinforcement Data *
**************************************************************************

Stirrup Bar Size              =    9
Stirrup Spacing X-Dir (Tran)  =   12.00 in
Stirrup Spacing Y-Dir (Long)  =   12.00 in

*************
* Soil Data *
**************************************************************************

Original Ground Elevation     (OG) =    79.20 Ft
Final Grade Elevation         (FG) =    79.20 Ft
Bottom of Footing Elevation (BOF) =    74.10 Ft

Depth of Initial Soil Overburden  =     5.10 Ft
Depth of Final Soil Cover         =     2.10 Ft

Soil Unit Weight                  =   120.00 Pcf
LRFD Exposure Factor (Ge)         =     1.00

**************************
* Pile Footing Cost Data *
**************************************************************************

Contract Item                           Price  Unit
-------------                         -------- -----
Structure Excavation (Bridge)       =    45.00 $/Cy
Structure Backfill   (Bridge)       =    30.00 $/Cy
Structural Concrete Bridge Footing  =   260.00 $/Cy
Bar Reinforcement (Bridge)          =     0.60 $/lb
Furnish Piling                      =    15.00 $/Ft
Drive Piles                         =  1100.00 $ Ea
```

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```
*************
* Pile Data *
*****************************************************************************

Total Number of Piles                =   6
Total Number of Rows                 =   3
Diameter of Pile                     =   12.0 in
Capacity of Pile (Service)           =   70.0 Tons
Consider Tension in Piles?           =    YES
Length of One Pile                   =   20.0 Ft
Static  Tension/Compression Ratio    =   40.0 Percent
Dynamic Tension/Compression Ratio    =   50.0 Percent


********************
* Pile Layout Data *
*****************************************************************************
```

| Row No | No of Piles | Y-Coord (Ft) | X-Spacing (Ft) |
|---|---|---|---|
| 1 | 2 | 0.00 | 3.50 |
| 2 | 2 | 4.00 | 3.50 |
| 3 | 2 | -4.00 | 3.50 |

```
***********************************
* Moment of Inertia of Pile Group *
*****************************************************************************

Ixx =      64.00 Pile-Ft^2
Iyy =      18.38 Pile-Ft^2
```

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```
*************************
* Pile Coordinates (Ft) *
*************************
```

| Pile | X-Coord | Y-Coord |
|---|---|---|
| 1 | -1.75 | 4.00 |
| 2 | 1.75 | 4.00 |
| 3 | -1.75 | 0.00 |
| 4 | 1.75 | 0.00 |
| 5 | -1.75 | -4.00 |
| 6 | 1.75 | -4.00 |

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**********
* LRFD Limit State Loads - Unfactored (Kip, K-FT) *
**********

Seismic Analysis Specifications:
Seismic Zone = 3
Response Modification Factor (R/2) = 1.00

Note: Loads are Combined According To Their Sign.

| | Case-1 Maximum Transverse | | | | |
|---|---|---|---|---|---|
| | P-Truck PLL+IM | H-Truck HLL+IM | Lane Load LL | Centrifugal PC-Truck | Centrifugal HC-Truck |
| My-Tran | 0. | 0. | 0. | 0. | 0. |
| Mx-Long | 0. | 0. | 0. | 0. | 0. |
| P-Axial | 1. | 1. | 0. | 0. | 0. |

| | Case-2 Maximum Longitudinal | | | | |
|---|---|---|---|---|---|
| | P-Truck PLL+IM | H-Truck HLL+IM | Lane Load LL | Centrifugal PC-Truck | Centrifugal HC-Truck |
| My-Tran | 0. | 0. | 0. | 0. | 0. |
| Mx-Long | 176. | 142. | 0. | 0. | 0. |
| P-Axial | 1. | 1. | 0. | 0. | 0. |

← controlling loading case

| | Case-3 Maximum Axial | | | | |
|---|---|---|---|---|---|
| | P-Truck PLL+IM | H-Truck HLL+IM | Lane Load LL | Centrifugal PC-Truck | Centrifugal HC-Truck |
| My-Tran | 0. | 0. | 0. | 0. | 0. |
| Mx-Long | 0. | 0. | 0. | 0. | 0. |
| P-Axial | ~~1.~~ 35 | ~~1.~~ 20 | 0. | 0. | 0. |

↳ No effect on the final results because it is not controlling loading case.

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**********
* LRFD Limit State Loads - Unfactored (Kip, K-FT) *
**********

Note: Loads are Combined According To Their Sign.

| | DC | DW | PS | PL | BR |
|---|---|---|---|---|---|
| | Dead Load | Added Dead Load | Prestress Sec Mom | Pedestrian Live Load | Braking Force |
| My-Tran | 0. | 0. | 0. | 0. | 0. |
| Mx-Long | 44. | 35. | 0. | 0. | 80. |
| P-Axial | 257. | 25. | 0. | 0. | 8. |

| | WA | WS-Wind Struc Horiz | | WL-Wind on Live Load | |
|---|---|---|---|---|---|
| | Stream Pressure | Case-1 Max Tran | Case-2 Max Long | Case-1 Max Tran | Case-2 Max Long |
| My-Tran | 0. | 0. | 0. | 0. | 0. |
| Mx-Long | 0. | 0. | 72. | 0. | 100. |
| P-Axial | 120. | 0. | 0. | 0. | 0. |

| | WV | FR | CR | SH | TU |
|---|---|---|---|---|---|
| | Wind Struc Vertical | Friction | Creep | Shrinkge | Temp Uniform |
| My-Tran | 0. | 0. | 0. | 0. | 0. |
| Mx-Long | 0. | 0. | 0. | 0. | 45. |
| P-Axial | 0. | 0. | 0. | 0. | 0. |

| | TG | SE | EQ | EQ |
|---|---|---|---|---|
| | Temp Gradient | Settlement | Case-1 Max Tran | Case-2 Max Long |
| My-Tran | 0. | 0. | 0. | 0. |
| Mx-Long | 0. | 0. | 0. | 0. |
| P-Axial | 0. | 0. | 0. | 0. |
| Pro Spc Factors | 0.00 | 0.00 | 0.00 | |

(EQ Load Factor: For H-Live Load,PL,BR)

| | IC Ice Collision | | LRFD |
|---|---|---|---|
| | Case-1 Max Tran | Case-2 Max Long | Arbritrary Load |
| My-Tran | 0. | 0. | 0. |
| Mx-Long | 0. | 0. | 0. |
| P-Axial | 0. | 0. | 0. |

| | Resistance Factors | |
|---|---|---|
| | Soil Phi | Piles Phi |
| Strength | 0.50 | 0.75 |
| Seismic | 1.00 | 1.00 |

Type of Superstructure = Non-Segmental
Column Moment of Inertia = I-gross

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```
**************************************************
* Plastic Moments (1.3 X Nominal Moment)(Kip, K-FT) *
**************************************************************************
(LRFD 3.10.9.4.3b)
```

| | Mp1 Max Tran | Mp2 75-Degree | Mp3 60-Degree | Mp4 45-Degree | Mp5 30-Degree | Mp6 15-Degree | Mp7 Max Long |
|---|---|---|---|---|---|---|---|
| MPy | 0. | 0. | 0. | 0. | 0. | 0. | 0. |
| MPx | 0. | 0. | 0. | 0. | 0. | 0. | 0. |
| P | 0. | 0. | 0. | 0. | 0. | 0. | 0. |
| Vp | 0. | 0. | 0. | 0. | 0. | 0. | 0. |

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**********
* LRFD Load Factors for Footing Design *
**********
(LRFD Tables 3.4.1-1, 3.4.1-2)

Note: Loads are Combined According To Their Sign.

| Limit | :---DC--: | | :---DW--: | | :---PS--: | | :---EV--: | | :---CR--: | | :---SH--: | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| State | Max | Min | Max | Min | Max | Min | Max | Min | Max | Min | Max | Min |
| Str-I | 1.25 | 0.90 | 1.50 | 0.65 | 0.50 | 0.50 | 1.35 | 0.90 | 0.50 | 0.50 | 0.50 | 0.50 |
| Str-II | 1.25 | 0.90 | 1.50 | 0.65 | 0.50 | 0.50 | 1.35 | 0.90 | 0.50 | 0.50 | 0.50 | 0.50 |
| Str-III | 1.25 | 0.90 | 1.50 | 0.65 | 0.50 | 0.50 | 1.35 | 0.90 | 0.50 | 0.50 | 0.50 | 0.50 |
| Str-IV | 1.50 | 0.90 | 1.50 | 0.65 | 0.50 | 0.50 | 1.35 | 0.90 | 1.50 | 0.90 | 1.50 | 0.90 |
| Str-V | 1.25 | 0.90 | 1.50 | 0.65 | 0.50 | 0.50 | 1.35 | 0.90 | 0.50 | 0.50 | 0.50 | 0.50 |
| Ext-I | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Ext-II | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Ser-I | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Perm Ld | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |

| Limit State | PLL Trck | HLL Trck | HLL Lane | P-CE Cent | H-CE Cent | PL | BR |
|---|---|---|---|---|---|---|---|
| Str-I | 0.00 | 1.75 | 1.75 | 0.00 | 1.75 | 1.75 | 1.75 |
| Str-II | 1.35 | 0.00 | 0.00 | 1.35 | 0.00 | 0.00 | 0.00 |
| Str-III | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Str-IV | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Str-V | 0.00 | 1.35 | 1.35 | 0.00 | 1.35 | 1.35 | 1.35 |
| Ext-I | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Ext-II | 0.00 | 0.50 | 0.50 | 0.00 | 0.50 | 0.50 | 0.50 |
| Ser-I | 0.00 | 1.00 | 1.00 | 0.00 | 1.00 | 1.00 | 1.00 |
| Perm Ld | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |

| Limit State | WA | WS | WL | WV | FR | TU | TG | SE | EQ | IC | CT | CV |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Str-I | 1.00 | 0.00 | 0.00 | 0.00 | 1.00 | 0.50 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Str-II | 1.00 | 0.00 | 0.00 | 0.00 | 1.00 | 0.50 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Str-III | 1.00 | 1.40 | 0.00 | 1.40 | 1.00 | 0.50 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Str-IV | 1.00 | 0.00 | 0.00 | 0.00 | 1.00 | 0.50 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Str-V | 1.00 | 0.40 | 1.00 | 0.00 | 1.00 | 0.50 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Ext-I | 1.00 | 0.00 | 0.00 | 0.00 | 1.00 | 0.00 | 0.00 | 0.00 | 1.00 | 0.00 | 0.00 | 0.00 |
| Ext-II | 1.00 | 0.00 | 0.00 | 0.00 | 1.00 | 0.00 | 0.00 | 0.00 | 0.00 | 1.00 | 1.00 | 1.00 |
| Ser-I | 1.00 | 0.30 | 1.00 | 0.00 | 0.00 | 1.00 | 0.50 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| Perm Ld | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |

**********
* Load Designations *
**********
(LRFD 3.2.2)

Permanent Loads
---
DC = Dead Load of Structural Components and Nonstructural Attachments (Dead Load)
DW = Dead Load of Wearing Surfaces and Utilities (Added Dead Load).
PS = Secondary Forces from Post-Tensioning.
CR = Creep.
SH = Shrinkage.

Transient Loads
---
PLL = Permit Live Load + IM
HLL = H-Design Truck/Tandem Live Load + IM.
LL = H-Design Lane Live Load (No IM).
P-CE = Centrifugal Permit Live Load (No IM).
H-CE = Centrifugal H-Design Truck/Tandem Live Load (No IM).
PL = Pedestrian Live Load. (No IM)
BR = Vehicular Braking Force (No IM).
WA = Water Load and Stream Pressure.
WS = Wind Load on Structure - Horizontal.
WL = Wind on Live Load (No IM).
WV = Wind Load on Structure - Vertical.
FR = Friction.
TU = Uniform Temperature.
TG = Temperature Gradient.
SE = Settlement Live Load.
EQ = Earthquake.
IC = Ice Collision.
CT = Vehicular (Truck or Train) Collision Force.
CV = Vessel Collision Force.
IM = Vehicular Dynamic Load Allowance (Percent Impact).

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**********
* Controlling Load Factor Combinations for Permanent Loads *
**********

1 = Max Load Factor Used
2 = Min Load Factor Used

| Limit State | Load Case | DC | DW | PS | EV | CR | SH | DC | DW | PS | EV | CR | SH |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Load Factors | | | | | | | | | | | |
| Str-I | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 1.25 | 1.50 | 0.50 | 1.35 | 0.50 | 0.50 |
| Str-I | 2 | 1 | 1 | 1 | 1 | 1 | 1 | 1.25 | 1.50 | 0.50 | 1.35 | 0.50 | 0.50 |
| Str-I | 3 | 1 | 1 | 1 | 1 | 1 | 1 | 1.25 | 1.50 | 0.50 | 1.35 | 0.50 | 0.50 |
| Str-I | 1a | 2 | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.90 | 0.50 | 0.50 |
| Str-I | 2a | 2 | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.90 | 0.50 | 0.50 |
| Str-I | 3a | 2 | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.90 | 0.50 | 0.50 |
| Str-II | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 1.25 | 1.50 | 0.50 | 1.35 | 0.50 | 0.50 |
| Str-II | 2 | 1 | 1 | 1 | 1 | 1 | 1 | 1.25 | 1.50 | 0.50 | 1.35 | 0.50 | 0.50 |
| Str-II | 3 | 1 | 1 | 1 | 1 | 1 | 1 | 1.25 | 1.50 | 0.50 | 1.35 | 0.50 | 0.50 |
| Str-II | 1a | 2 | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.90 | 0.50 | 0.50 |
| Str-II | 2a | 2 | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.90 | 0.50 | 0.50 |
| Str-II | 3a | 2 | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.90 | 0.50 | 0.50 |
| Str-III | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 1.25 | 1.50 | 0.50 | 1.35 | 0.50 | 0.50 |
| Str-III | 2 | 1 | 1 | 1 | 1 | 1 | 1 | 1.25 | 1.50 | 0.50 | 1.35 | 0.50 | 0.50 |
| Str-III | 3 | 1 | 1 | 1 | 1 | 1 | 1 | 1.25 | 1.50 | 0.50 | 1.35 | 0.50 | 0.50 |
| Str-III | 1a | 2 | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.90 | 0.50 | 0.50 |
| Str-III | 2a | 2 | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.90 | 0.50 | 0.50 |
| Str-III | 3a | 2 | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.90 | 0.50 | 0.50 |
| Str-IV | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 1.50 | 1.50 | 0.50 | 1.35 | 1.50 | 1.50 |
| Str-IV | 2 | 1 | 1 | 1 | 1 | 1 | 1 | 1.50 | 1.50 | 0.50 | 1.35 | 1.50 | 1.50 |
| Str-IV | 3 | 1 | 1 | 1 | 1 | 1 | 1 | 1.50 | 1.50 | 0.50 | 1.35 | 1.50 | 1.50 |
| Str-IV | 1a | 2 | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.90 | 0.90 | 0.90 |
| Str-IV | 2a | 2 | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.90 | 0.90 | 0.90 |
| Str-IV | 3a | 2 | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.90 | 0.90 | 0.90 |
| Str-V | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 1.25 | 1.50 | 0.50 | 1.35 | 0.50 | 0.50 |
| Str-V | 2 | 1 | 1 | 1 | 1 | 1 | 1 | 1.25 | 1.50 | 0.50 | 1.35 | 0.50 | 0.50 |
| Str-V | 3 | 1 | 1 | 1 | 1 | 1 | 1 | 1.25 | 1.50 | 0.50 | 1.35 | 0.50 | 0.50 |
| Str-V | 1a | 2 | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.90 | 0.50 | 0.50 |
| Str-V | 2a | 2 | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.90 | 0.50 | 0.50 |
| Str-V | 3a | 2 | 2 | 2 | 2 | 2 | 2 | 0.90 | 0.65 | 0.50 | 0.90 | 0.50 | 0.50 |
| Ext-I | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Ext-I | 2 | 1 | 1 | 1 | 1 | 1 | 1 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Ext-II | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Ext-II | 2 | 1 | 1 | 1 | 1 | 1 | 1 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Ext-II | 3 | 1 | 1 | 1 | 1 | 1 | 1 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Ext-II | 1a | 2 | 2 | 2 | 2 | 2 | 2 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Ext-II | 2a | 2 | 2 | 2 | 2 | 2 | 2 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Ext-II | 3a | 2 | 2 | 2 | 2 | 2 | 2 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Ser-I | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Ser-I | 2 | 1 | 1 | 1 | 1 | 1 | 1 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Ser-I | 3 | 1 | 1 | 1 | 1 | 1 | 1 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Perm Ld | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |

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**********
* NET TOTAL Service Loads for Settlement Check (Kip, K-Ft) *
**********

| Limit State | Case | Applied Loads Tran My | Long Mx | Comb M | Axial Pnet | Min Pile Load | Max Pile Load |
|---|---|---|---|---|---|---|---|
| Ser-I | 1 | 0. | 204. | 204. | 412. | 56. | 81. |
| Ser-I | 2 | 0. | 432. | 432. | 412. | 42. | 96. |
| Ser-I | 3 | 0. | 326. | 326. | 412. | 48. | 89. |

Note:
Pnet = Pgross - Poverburden.
Poverburden = 50. Kip

**********
* NET PERMANENT Service Loads for Settlement Check (Kip, K-Ft) *
**********

| Limit State | Case | Applied Loads Tran My | Long Mx | Comb M | Axial Pnet | Min Pile Load | Max Pile Load |
|---|---|---|---|---|---|---|---|
| Perm Ld | 1 | 0. | 79. | 79. | 283. | 42. | 52. |

Note: Limit State "Perm Ld" = Permanent Loads Only.

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**********
* GROSS Service Loads (Ser) for Footing Design (Kip, K-Ft) *
**********

Note: Ratio = Capacity (Comp or Uplift) / (Max or Min) Pile Load

| Cap Uplift | Cap Comp |
|---|---|
| -56. | 140. |

| Limit State | Case | Applied Loads Tran My | Long Mx | Comb M | Axial Pgross | Min Pile Load | Ratio | Max Pile Load | Ratio | |
|---|---|---|---|---|---|---|---|---|---|---|
| Ser-I | 1 | 0. | 204. | 204. | 462. | 64. | | 90. | 1.56 | OK |
| Ser-I | 2 | 0. | 432. | 432. | 462. | 50. | | 104. ✓ | 1.35 | OK |
| Ser-I | 3 | 0. | 326. | 326. | 462. | 57. | | 97. | 1.44 | OK |

The Following are Incluced in the Applied Loads Above:

Weight of Cover = 14. Kip
Weight of Footing = 37. Kip
Total Weight = 51. Kip

For Limit State Ser-I Case 2 The Pile Loads at Each Corner Are:

| | |
|---|---|
| 104. | 104. |
| 50. | 50. |

Note: Pile Structural Capacity Check for Ser-I Limit State is not Required per LRFD Code.
However, Check is Performed for Information Only.

Ser I

Axial,

$$\frac{DC}{257} + \frac{DW}{25} + \frac{HL\text{-}93}{1} + \frac{BR}{8} + \frac{WA}{120} + \frac{WS}{0.3\times 0} + \frac{WL}{0} + \frac{TU}{0} = 411$$

$$411 + 51 = 462 \text{ Kips}$$

Moment

$$\frac{DC}{44} + \frac{DW}{35} + \frac{HL\text{-}93}{\left(\frac{142}{1.33}\right)} + \frac{BR}{80} + \frac{WA}{0} + \frac{WS}{0.3\times 72} + \frac{WL}{100} + \frac{TU}{45} = 432 \checkmark$$

$$\frac{437\times 4}{64} = 27$$

$$\frac{462}{6} = 77$$

] 104 Kips

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**********
* Factored Loads (Fac) for Footing Design (Kip, K-Ft) *
**********

Note: Ratio = Capacity (Comp or Uplift) / (Max or Min) Pile Load

Phi = 0.75

| Cap Uplift | Cap Comp |
|---|---|
| -84. | 210. |

| Limit State | Case | Applied Loads Tran My | Long Mx | Comb M | Axial Pgross | Min Pile Load | Ratio | Max Pile Load | Ratio | |
|---|---|---|---|---|---|---|---|---|---|---|
| Str-I | 1 | 0. | 270. | 270. | 560. | 76. | | 110. | 1.91 | OK |
| Str-I | 2 | 0. | 457. | 457. | 560. | 65. | | 122. | 1.72 | OK |
| Str-I | 3 | 0. | 270. | 270. | 560. | 76. | | 110. | 1.91 | OK |
| Str-I | 1a | 0. | 225. | 225. | 429. | 58. | | 86. | 2.46 | OK |
| Str-I | 2a | 0. | 412. | 412. | 429. | 46. | | 97. | 2.16 | OK |
| Str-I | 3a | 0. | 225. | 225. | 429. | 58. | | 86. | 2.46 | OK |
| Str-II | 1 | 0. | 130. | 130. | 545. | 83. | | 99. | 2.12 | OK |
| Str-II | 2 | 0. | 320. | 320. | 545. | 71. | | 111. | 1.89 | OK |
| Str-II | 3 | 0. | 130. | 130. | 545. | 83. | | 99. | 2.12 | OK |
| Str-II | 1a | 0. | 85. | 85. | 415. | 64. | | 74. | 2.82 | OK |
| Str-II | 2a | 0. | 275. | 275. | 415. | 52. | | 86. | 2.43 | OK |
| Str-II | 3a | 0. | 85. | 85. | 415. | 64. | | 74. | 2.82 | OK |
| Str-III | 1 | 0. | 130. | 130. | 544. | 83. | | 99. | 2.12 | OK |
| Str-III | 2 | 0. | 231. | 231. | 544. | 76. | | 105. | 2.00 | OK |
| Str-III | 3 | 0. | 231. | 231. | 544. | 76. | | 105. | 2.00 | OK |
| Str-III | 1a | 0. | 85. | 85. | 414. | 64. | | 74. | 2.83 | OK |
| Str-III | 2a | 0. | 186. | 186. | 414. | 57. | | 81. | 2.61 | OK |
| Str-III | 3a | 0. | 186. | 186. | 414. | 57. | | 81. | 2.61 | OK |
| Str-IV | 1 | 0. | 141. | 141. | 618. | 94. | | 112. | 1.88 | OK |
| Str-IV | 2 | 0. | 141. | 141. | 618. | 94. | | 112. | 1.88 | OK |
| Str-IV | 3 | 0. | 141. | 141. | 618. | 94. | | 112. | 1.88 | OK |
| Str-IV | 1a | 0. | 85. | 85. | 414. | 64. | | 74. | 2.83 | OK |
| Str-IV | 2a | 0. | 85. | 85. | 414. | 64. | | 74. | 2.83 | OK |
| Str-IV | 3a | 0. | 85. | 85. | 414. | 64. | | 74. | 2.83 | OK |
| Str-V | 1 | 0. | 238. | 238. | 556. | 78. | | 108. | 1.95 | OK |
| Str-V | 2 | 0. | 511. | 511. | 556. | 61. | | 125. | 1.69 | OK |
| Str-V | 3 | 0. | 367. | 367. | 556. | 70. | | 116. | 1.82 | OK |
| Str-V | 1a | 0. | 193. | 193. | 426. | 59. | | 83. | 2.53 | OK |
| Str-V | 2a | 0. | 466. | 466. | 426. | 42. | | 100. | 2.10 | OK |
| Str-V | 3a | 0. | 322. | 322. | 426. | 51. | | 91. | 2.31 | OK |
| Ext-II | 1 | 0. | 119. | 119. | 458. | 69. | | 84. | 2.51 | OK |
| Ext-II | 2 | 0. | 172. | 172. | 458. | 66. | | 87. | 2.41 | OK |
| Ext-II | 3 | 0. | 119. | 119. | 458. | 69. | | 84. | 2.51 | OK |
| Ext-II | 1a | 0. | 119. | 119. | 458. | 69. | | 84. | 2.51 | OK |
| Ext-II | 2a | 0. | 172. | 172. | 458. | 66. | | 87. | 2.41 | OK |
| Ext-II | 3a | 0. | 119. | 119. | 458. | 69. | | 84. | 2.51 | OK |
| Arb-I | 1 | 0. | 0. | 0. | 0. | 0. | | 0. | 0.00 | OK |

125+10=135

The Following are Incluced in the Applied Loads Above:

Weight of Cover = 14. Kip
Weight of Footing = 37. Kip
Total Weight = 51. Kip

For Limit State Str-V Case 2 The Pile Loads at Each Corner Are:

| | |
|---|---|
| 125. | 125. |
| 61. | 61. |

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**********
* Earthquake Loads (Eq) for Footing Design (Kip, K-Ft) *
**********

Note: Ratio = Capacity (Comp or Uplift) / (Max or Min) Pile Load

ARS Unreduced Seismic (VII) & Overstrength Demand Moments (Md)

Phi = 1.00

| Cap Uplift | Cap Comp |
|---|---|
| -140. | 280. |

| Limit State | Case | Applied Loads Tran My | Long Mx | Comb M | Axial Pgross | Min Pile Load | Ratio | Max Pile Load | Ratio | |
|---|---|---|---|---|---|---|---|---|---|---|
| Ext-I | 1 | 0. | 0. | 0. | 51. | | | | | NA |
| Ext-I | 2 | 0. | 0. | 0. | 51. | | | | | NA |
| Md1 | 1 | 0. | 0. | 0. | 51. | 0. | | 0. | 0.00 | OK |
| Md2 | 1 | 0. | 0. | 0. | 51. | 0. | | 0. | 0.00 | OK |
| Md3 | 1 | 0. | 0. | 0. | 51. | 0. | | 0. | 0.00 | OK |
| Md4 | 1 | 0. | 0. | 0. | 51. | 0. | | 0. | 0.00 | OK |
| Md5 | 1 | 0. | 0. | 0. | 51. | 0. | | 0. | 0.00 | OK |
| Md6 | 1 | 0. | 0. | 0. | 51. | 0. | | 0. | 0.00 | OK |
| Md7 | 1 | 0. | 0. | 0. | 51. | 0. | | 0. | 0.00 | OK |

The Following are Incluced in the Applied Loads Above:

Weight of Cover = 14. Kip
Weight of Footing = 37. Kip
Total Weight = 51. Kip

For Limit State Case The Pile Loads at Each Corner Are:

0. 0.

4. 0.

Calculation of Footing Overstrength Demand Moments - Md:

Md = Mo + (Vo x D) + (Np x Mpile)

Md = Footing Demand Moment
Mp = Column Plastic Moment
Vp = Column Plastic Shear
Mo = Column Overstrength Moment = 1.20 x Mp
Vo = Column Overstrength Shear = 1.20 x Vp
D = Foooting Depth
Np = Number of Piles
Mpile = Moment at Top of Pile = 0.

Controlling Nominal Resistance

Compression 166. Kip (Limit State Str-V Case 2 )
Tension 0. Kip (Limit State Case )

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```
*****************************************
* Design of Bottom Flexural Reinforcement *
*****************************************************************************
```

Key to Types of Loads:

```
Ser-I    = Gross Service Load - Crack Control (LRFD 5.7.3.4)
Fac      = Factored Loads, Variable Phi
 Str-I   = H-Loads
 Str-II  = P-Loads
 Str-III = No H-Loads, Wind > 55 mph
 Str-IV  = No H-Loads, Hign DL to LL Ratio
 Str-V   = H-Loads, Wind=55 mph
 Ext-II  = Ice/Train,Vehicle/Vessel Collision
 Arb-I   = Arbritrary User Defined Load
Eq       = Ext-I & Plastic Earthquake Loads
Min1     = Minimum Reinforcement (LRFD 5.7.3.3.2)
           Based on 1.2 X The Cracking Moment
Min2     = Waiver of Minimum Reinforcement (LRFD 5.7.3.3.2)
           Based on Steel Area One Third Greater than Required by Analysis
```

Footing Depth = 3.00 Ft

| Bar Size | Bar Dir | Load Type | Limit State | Ca | Depth Effective (in) | Applied Moment (K-Ft) | Phi | Max Rebar Strain | Req Area of Steel (in^2) | No Bars |
|---|---|---|---|---|---|---|---|---|---|---|
| 8 | X | Ser | | | 28.19 | 0. | | | 0.0 | 0 |
| 9 | Y | Ser | Ser-I | 2 | 29.38 | 431. | | | 8.6 | 9 |
| 8 | X | Fac | | | 28.19 | 0. | 0.900 | 0.00000 | 0.0 | 0 |
| 9 | Y | Fac | Str-V | 2 | 29.38 | 513. | 0.900 | 0.07564 | 3.9 | 4 |
| 8 | X | Eq | | | 28.19 | 0. | 1.000 | 0.00000 | 0.0 | 0 |
| 9 | Y | Eq | | | 29.38 | 0. | 1.000 | 0.00000 | 0.0 | 0 |
| 8 | X | Min1 | | | 28.19 | 1902. | 0.900 | 0.02477 | 15.7 | 20 |
| 9 | Y | Min1 | | | 29.38 | 1297. | 0.900 | 0.02728 | 10.2 | 11 |
| 8 | X | Min2 | | | 28.19 | 0. | 0.900 | 0.00000 | 0.0 | 0 |
| 9 | Y | Min2 | | | 29.38 | 0. | 0.900 | 0.00000 | 0.0 | 0 |

********** CONTROL **********

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 8 | X | Min1 | | | 28.19 | 1902. | | | 15.7 | 20 |
| 9 | Y | Min1 | | | 29.38 | 1297. | | | 10.2 | 11 |

Note: (LRFD 5.13.3.5)
In the short Y-direction, ( 8.3 in^2) of reinforcemtment shall be distributed uniformly over a band width equal to the length of short side of the footing ( 7.5 Ft) and centered on the column.
The remainder of the reinforcement in the short Y-direction, ( 1.9 in^2) shall be distributed uniformly outside the center band width of the footing.

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```
****************************************
* Design of Top Flexural Reinforcement *
****************************************************************************
```

Key to Types of Loads:

```
Ser-I    = Gross Service Load - Crack Control (LRFD 5.7.3.4)
Fac      = Factored Loads, Variable Phi
 Str-I   = H-Loads
 Str-II  = P-Loads
 Str-III = No H-Loads, Wind > 55 mph
 Str-IV  = No H-Loads, Hign DL to LL Ratio
 Str-V   = H-Loads, Wind=55 mph
 Ext-II  = Ice/Train,Vehicle/Vessel Collision
 Arb-I   = Arbritrary User Defined Load
Eq       = Ext-I & Plastic Earthquake Loads
Min1     = Minimum Reinforcement (LRFD 5.7.3.3.2)
           Based on 1.2 X The Cracking Moment
Min2     = Waiver of Minimum Reinforcement (LRFD 5.7.3.3.2)
           Based on Steel Area One Third Greater than Required by Analysis
Min3     = Minimum Reinforcement (Shrinkage & Temperature) (LRFD 5.10.8)
```

Footing Depth = 3.00 Ft

| Bar Size | Bar Dir | Load Type | Limit State | Ca | Depth Effective (in) | Applied Moment (K-Ft) | Phi | Max Rebar Strain | Req Area of Steel (in^2) | No Bars |
|---|---|---|---|---|---|---|---|---|---|---|
| 6 | X | Ser | | | 31.68 | 0. | | | 0.0 | 0 |
| 6 | Y | Ser | | | 32.56 | 0. | | | 0.0 | 0 |
| 6 | X | Fac | | | 31.68 | 0. | 0.900 | 0.00000 | 0.0 | 0 |
| 6 | Y | Fac | | | 32.56 | 0. | 0.900 | 0.00000 | 0.0 | 0 |
| 6 | X | Eq | | | 31.68 | 0. | 1.000 | 0.00000 | 0.0 | 0 |
| 6 | Y | Eq | | | 32.56 | 0. | 1.000 | 0.00000 | 0.0 | 0 |
| 6 | X | Min1 | | | 31.68 | 1902. | 0.900 | 0.03245 | 13.8 | 32 |
| 6 | Y | Min1 | | | 32.56 | 1297. | 0.900 | 0.03453 | 9.2 | 21 |
| 6 | X | Min2 | | | 31.68 | 0. | 0.900 | 0.00000 | 0.0 | 0 |
| 6 | Y | Min2 | | | 32.56 | 0. | 0.900 | 0.00000 | 0.0 | 0 |
| 6 | X | Min3 | | | 31.68 | 742. | 0.900 | 0.08999 | 5.3 | 12 |
| 6 | Y | Min3 | | | 32.56 | 509. | 0.900 | 0.09474 | 3.5 | 8 |

********** CONTROL **********

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 6 | X | Min3 | | | 31.68 | 742. | | | 5.3 | 12 |
| 6 | Y | Min3 | | | 32.56 | 509. | | | 3.5 | 8 |

Note: (LRFD 5.13.3.5)
In the short Y-direction, ( 2.9 in^2) of reinforcemtment shall be distributed uniformly over a band width equal to the length of short side of the footing ( 7.5 Ft) and centered on the column.
The remainder of the reinforcement in the short Y-direction, ( 0.7 in^2) shall be distributed uniformly outside the center band width of the footing.

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```
************************************
* Final Results for Footing CHECK *
***************************************************************************

Footing Length =    7.50 Ft
Footing Width  =   11.00 Ft
Input Depth    =    3.00 Ft

*********************
* Pile Load Results *
***************************************************************************

     Control      Min Pile  Capacity        Control      Max Pile  Capacity
Load Limit           Load    Uplift        Limit            Load      Comp
Type State   Ca     (Kip)     (Kip)        State    Ca     (Kip)     (Kip)
---- ------- -- --------- ---------        -------- -- --------- ---------
Ser  Ser-I   2        50.      -56. OK     Ser-I    2       104.      140. OK
Fac  Str-V   2a       42.      -84. OK     Str-V    2       125.      210. OK
Eq                  4.     -140. OK                      0.      280. OK

*****************
* Shear Results *
***************************************************************************
(LRFD 5.8.3.3, 5.8.3.4.1 Simplified Procedure)

Ratio = Shear Capacity/Shear Applied

Phi             =     0.90
Footing Depth   =     3.00 Ft
Perimeter (Bo)  =     0.00 Ft

     Control              Depth          Shear           Shear
Load Limit             Effective        Applied        Capacity   Ratio
Type State   Ca            (in)          (Kip)          (Kip)
---- ------- --     ------------    -----------    ------------   -----
Fac                 Dvy =   26.4    VY =     126.    VUY =   1136.   9.04 OK
Eq                  Dvy =   26.4    VY =       0.    VUY =   1136.   0.00 OK

Note: Effective Shear Depth "Dvx" & "Dvy" is Taken as the Greater of (LRFD 5.8.2.9):
      .72h  = .72 x Footing   Depth
      .90dx = .90 x Effective Depth for Bars in "X" Transverse  Direction.
      .90dy = .90 x Effective Depth for Bars in "Y" Longitudinal Direction.

Note: Dvp = (Dvx + Dvy) / 2

* Note: The Shear Plane at Distance "dx" is Outside the Footing Dimesnion.
   Therefore, NO (Vx) or (Vp) Shear Calculations are Made.

************************************
* Top and Bottom Flexure Results *
***************************************************************************
Footing Depth =     3.00 Ft

                  Control       Eff   Applied     Bar    Area
Bar    Bar Load Limit        Depth    Moment   Space     Req Bars Bars       Error
Size   Dir Type State    Ca   (in)    (K-Ft)    (in)  (in^2)  Req InPut      Code
----   --- ---- -------  -- ------ --------- ------- ------- ---- -----      -----
Top
  6    X   Min3                31.7      742.    11.4     5.3   12    12 OK      4
  6    Y   Min3                32.6      509.    11.9     3.5    8     8 OK

Bottom
  8    X   Min1                28.2     1902.    11.4    15.7   20    12 NG**    4
  9    Y   Min1                29.4     1297.    11.8    10.2   11     8 NG**
```

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```
*********************************
* Bottom Flexural Reinforcement *
*********************************************************************************
(LRFD 5.3.7.4)
```

(Area of Steel Required for Different Bar Sizes)

| Bar Size | Bar Dir | Cont Load Type | Actual Bar Stress (Psi ) | Acutal Bar Space (in) | Allow Bar Space (in) | Eff Depth (in) | Req Steel (in^2) | No Bars | Foot Depth (Ft) | Error Code |
|---|---|---|---|---|---|---|---|---|---|---|
| 5 | X | Min1 | 0. | 2.56 | 12.00 | 28.97 | 15.3 | 50 | 3.00 | 14 |
| 5 | Y | Min1 | 18462. | 2.60 | 16.35 | 29.66 | 10.1 | 33 | | 1 |
| 6 | X | Min1 | 0. | 3.57 | 12.00 | 28.68 | 15.4 | 36 | 3.00 | 24 |
| 6 | Y | Min1 | 17965. | 3.61 | 16.84 | 29.56 | 10.2 | 24 | | 2 |
| 7 | X | Min1 | 0. | 5.00 | 12.00 | 28.50 | 15.5 | 26 | 3.00 | 4 |
| 7 | Y | Min1 | 18615. | 5.19 | 15.60 | 29.50 | 10.2 | 17 | | |
| 8 | X | Min1 | 0. | 6.57 | 12.00 | 28.31 | 15.6 | 20 | 3.00 | 4 |
| 8 | Y | Min1 | 18535. | 6.91 | 15.51 | 29.44 | 10.2 | 13 | | |
| 9 | X | Min1 | 0. | 8.32 | 12.00 | 28.13 | 15.8 | 16 | 3.00 | 4 |
| 9 | Y | Min1 | 17386. | 8.28 | 17.20 | 29.38 | 10.2 | 11 | | |
| 10 | X | Min1 | 0. | 10.38 | 12.00 | 27.84 | 15.9 | 13 | 3.00 | 4 |
| 10 | Y | Min1 | 16812. | 10.32 | 17.92 | 29.28 | 10.3 | 9 | | 4 |
| 11 | X | Min1 | 0. | 12.44 | 12.00 | 27.56 | 16.1 | 11 | 3.00 | 34 |
| 11 | Y | Ser | 17626. | 13.73 | 16.15 | 29.19 | 10.3 | 7 | | 34 |
| 14 | X | Min1 | 0. | 17.73 | 12.00 | 27.18 | 16.4 | 8 | 3.00 | 34 |
| 14 | Y | Ser | 14440. | 16.42 | 22.26 | 29.06 | 12.1 | 6 | | 34 |
| 18 | X | Min1 | 0. | 30.88 | 12.00 | 26.25 | 17.0 | 5 | 3.00 | 34 |
| 18 | Y | Ser | 10027. | 20.38 | 36.82 | 28.75 | 16.1 | 5 | | 34 |

Note: The Designer must Check the Error Code for Adaquate
Development Length and Bar Spacing.

Error Codes:
---
1 = Bar Spacing LESS than The AASHTO Minimum. (LRFD 5.10.3.1)
2 = Bar Spacing LESS than The Preferred CALTRANS Minimum. (Br. Des. Details)
3 = Bar Spacing MORE than The AASHTO Maximum (LRFD 5.10.8)
4 = Inadequate Development Length. (LRFD 5.11.2.1)

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```
******************************
* Top Flexural Reinforcement *
******************************************************************************
```

(LRFD 5.3.7.4)

(Area of Steel Required for Different Bar Sizes)

| Bar Size | Bar Dir | Cont Load Type | Actual Bar Stress (Psi ) | Acutal Bar Space (in) | Allow Bar Space (in) | Eff Depth (in) | Req Steel (in^2) | No Bars | Foot Depth (Ft) | Error Code |
|---|---|---|---|---|---|---|---|---|---|---|
| 4 | X | Min3 | 0. | 7.84 | 7.84 | 32.16 | 3.4 | 17 | 3.00 | 4 |
| 4 | Y | Min3 | 0. | 9.27 | 9.27 | 32.72 | 2.0 | 10 | | |

Note: The Designer must Check the Error Code for Adaquate Development Length and Bar Spacing.

Error Codes:
---
1 = Bar Spacing LESS than The AASHTO Minimum. (LRFD 5.10.3.1)
2 = Bar Spacing LESS than The Preferred CALTRANS Minimum. (Br. Des. Details)
3 = Bar Spacing MORE than The AASHTO Maximum (LRFD 5.10.8)
4 = Inadequate Development Length. (LRFD 5.11.2.1)

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```
*************************************************************
* Minimum, Preferred C. to C. Bar Spacing & Development Length *
*****************************************************************************************
```

Assume Maximum Coarse Size Aggregate = 1.5 in

The Clear Distance between Parallel Bars Shall NOT be less than
1.5 Bar Diameters (Nominal), 1.5 Times the Maximum Size Course Aggregate,
nor 1.5 inches. (LRFD 5.10.3.1)

Preferred Center to Center Spacing Based on Bridge Design Details Page 13-20

| Bar Size | Bar Area (in^2) | Nominal Diameter (in) | Min Clear Distance (in) | Deformed Diameter (in) | Minimum Spacing (in) | Preferred Spacing (in) | Development Length (in) Spc<6" | Spc>=6" |
|---|---|---|---|---|---|---|---|---|
| 4 | .20 | .500 | 2.250 | .563 | 2.813 | 3.250 | 12 | 12 |
| 5 | .31 | .625 | 2.250 | .688 | 2.938 | 3.500 | 15 | 12 |
| 6 | .44 | .750 | 2.250 | .875 | 3.125 | 3.750 | 19 | 15 |
| 7 | .60 | .875 | 2.250 | 1.000 | 3.250 | 4.000 | 26 | 20 |
| 8 | .79 | 1.000 | 2.250 | 1.125 | 3.375 | 4.250 | 34 | 27 |
| 9 | 1.00 | 1.128 | 2.250 | 1.250 | 3.500 | 4.500 | 42 | 34 |
| 10 | 1.27 | 1.270 | 2.250 | 1.438 | 3.688 | 4.750 | 54 | 43 |
| 11 | 1.56 | 1.410 | 2.250 | 1.625 | 3.875 | 5.000 | 66 | 53 |
| 14 | 2.25 | 1.693 | 2.539 | 1.875 | 4.414 | 5.500 | 90 | 72 |
| 18 | 4.00 | 2.257 | 3.386 | 2.500 | 5.886 | 6.000 | 116 | 93 |

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**********
* Bar Reinforcing Steel *
**********

| Bar Type | Bar Size | No Bar | Length (Ft) | Weight (Lb/Ft) | Weight (Lb) |
|---|---|---|---|---|---|
| Top | | | | | |
| X-Bar | 6 | 12 | 7.0 | 1.502 | 126. |
| Y-Bar | 6 | 8 | 10.5 | 1.502 | 126. |
| Bottom | | | | | |
| X-Bar | 8 | 12 | 7.0 | 2.670 | 224. |
| Y-Bar | 9 | 8 | 10.5 | 3.400 | 286. |
| Stirrup | 9 | 120 | 4.8 | 3.400 | 1972. |

Note: Stirrups are Required in a Band Extending 6 inches
Maximum from The Column Reinforcement to
26.3 inches from The Face of The Column in The "X" Direction and
28.8 inches from The Face of The Column in The "Y" Direction.

**********
* Quantities and Estimated Cost - Pile Footing *
**********

| Contract Item | Unit | Quantity | Price | Amount |
|---|---|---|---|---|
| Structure Excavation (Br) | Cy | 23.3 | 45.00 | 1050. |
| Structure Backfill (Br) | Cy | 12.1 | 30.00 | 364. |
| Structure Concrete, Br Ftg | Cy | 9.2 | 260.00 | 2383. |
| Bar Reinforcement (Br) | Lb | 2734.2 | 0.60 | 1641. |
| Furnish Piling | Ft | 120.0 | 15.00 | 1800. |
| Drive Piles | Ea | 6 | 1100.00 | 6600. |
| Total Cost | | | | 13837. |

Foundation Load Summary

The loads summarized in the following pages are used for communications between structure and geotechnical engineers. Geotechnical branch used these loads to analyze and revise design pile length.

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**EA: 07-202101**
**Bridge Name: LA River**
**Bridge No.: 53-0828**

General Foundation Information to be sent from **SD** to **GS**

| Foundation Design Data Sheet | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Support No. | Design Method | Pile Type | Finished Grade Elevation | Cut-off Elevation (ft) | Pile Cap Size | | Permissible Settlement Under Service Load * | Number of Piles per Support |
| | | | | | B (ft) | L (ft) | | |
| Abutment 1 | WSD | Driven Class | 108.5 (North) 107.8 (South) | 104.75 (North) 104.75 (South) | Varies See Pile Layout Sheet | Varies See Pile Layout Sheet) | 1 in | 16 (North) 18 (South) |
| Pier 2 | LRFD | Driven Class | 82.2 (North) 82 (South) | 76.25 (North) 76.25 (South) | 17 (North) 17 (South) | 18 (North) 25 (South) | 1 in | 25 (North) 35 (South) |
| Pier 3 | LRFD | Driven Class | 80.1 (North) 79.8 (South) | 74.75 (North) 74.75 (South) | 11 (North) 11 (South) | 21.5 (North) 28.5 (South) | 1 in | 18 (North) 24 (South) |
| Pier 4 | LRFD | Driven Class | 82.1 (North) 82.4 (South) | 76.75 (North) 76.75 (South) | 17 (North) 17 (South) | 18 (North) 25 (South) | 1 in | 25 (North) 35 (South) |
| Abutment 5 | WSD | Driven Class | 108 (North) 107.6 (South) | 104.75 (North) 104.75 (South) | Varies See Pile Layout Sheet | Varies See Pile Layout Sheet | 1 in | 12 (North) 25 (South) |

* MTD 4-1:Based on CALTRANS' current practice, the total permissible settlement is one inch for multi-span structures with continuous spans or multi-column bents, one inch for single span structures with diaphragm abutments, and two inches for single span structures with seat abutments. Different permissible settlement under service loads may be allowed if a structural analysis verifies that required level of serviceability is met.

Design Loads to be sent from **SD** to **GS**

| Foundation Design Loads | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Support No. | Service-1 Limit State (kips) | | | Strength Limit State (Controlling Group, kips) | | | | Extreme Event Limit State (Controlling Group, kips) | | | |
| | Total Load | | Permanent Loads | Compression | | Tension | | Compression | | Tension | |
| | Per Support | Max Per Pile | Per Support | Per Support | Max Per Pile | Per Support | Max Per Pile | Per Support | Max Per Pile | Per Support | Max Per Pile |
| Abut 1 | 1060 (North) 1190 (South) | 115 (North) 115 (South) | 760 (North) 855 (South) | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A |
| Pier 2 | 1250 (North) 1730 (South) | 70 (North) 70 (South) | 905 (North) 1255 (South) | 1640 (North) 2280 (South) | 85 (North) 85 (South) | N/A | 0 | 1465 (North) 1945 (South) | 120 (North) 120 (South) | N/A | -60 (North) -60 (South) |
| Pier 3 | 1370 (North) 1750 (South) | 110 (North) 110 (South) | 955 (North) 1220 (South) | 1745 (North) 2270 (South) | 135 (North) 135 (South) | N/A | 0 | 1370 (North) 1750 (South) | N/A (North) N/A (South) | N/A | N/A (North) N/A (South) |
| Pier 4 | 1250 (North) 1730 (South) | 70 (North) 70 (South) | 905 (North) 1255 (South) | 1640 (North) 2280 (South) | 85 (North) 85 (South) | N/A | 0 | 1465 (North) 1945 (South) | 120 (North) 120 (South) | N/A | -60 (North) -60 (South) |
| Abut 5 | 995 (North) 2065 (South) | 130 (North) 130 (South) | 780 (North) 1640 (South) | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A |

**Note:**

1. **For Abutment piles, please provide pile lateral capacity with ¼" displacement.**
2. **For Pier wall piles, please provide pile lateral capacity with ¼" displacement, ½" displacement, 1" displacement, 2" displacement and 3" displacement. Under seismic loading, I will need 40 kips/pile lateral load capacity in pier wall transverse direction.**
3. **Please also provide recommendation for the retaining walls adjacent to the abutment 1 and abutment 5. I am using Standard Caltrans Type 1 wall. Please let me know whether the spread footing foundation works.**

FHW LA River Bridge

8/19/10 Pier 2 & 4 Footing Load
– Service

The loads is in proportion to the overall length.

For Pier 2, use Footing 7.5 ft for calculation

For Pier 2 North length of footing = 18'-0"

$$\text{Ratio} = \frac{18}{7.5} = 2.4$$

Permanent Load

$$= 257 + 25 + 95 = 377 \times 2.4 = 905 \text{ kips}$$

Total Load

$$= 520 \times 2.4 = 1250$$

For Pier 2 South, length of footing = 25'

$$\text{Ratio} = 25/7.5 = 3.33$$

Permanent Load = $377 \times 3.33 = 1255$ kips

Total Load

$$= 520 \times 3.33 = 1730 \text{ kips}$$

For Pier 3 North.

$$\text{Ratio} = \frac{21.5}{7.5} = 2.87$$

Permanent load

$$= 257 + 25 + 51 = 333 \times 2.87 = 955 \text{ kips}$$

Total

$$= 476 \times 2.87 = 1370 \text{ kips}$$

For Pier 3 South

$$\text{Ratio} = \frac{27.5}{7.5} = 3.67$$

Permanent load

$$= 333 \times 3.67 = 1220 \text{ kips}$$

Total load

$$= 476 \times 3.67 = 1750 \text{ kips}$$

FRW

8/17/10

LA River Bridge

Pier 2, 3 & 4 load
– Strength



For Strength I

Axial load:

$$1.25\underbrace{(257)}_{DL} + 1.5\underbrace{(25)}_{DW} + 1.75\underbrace{(8)}_{BR} + 1\underbrace{(120)}_{WA}$$

$$+ 1.25\underbrace{(95)}_{DL\ (Soil + FTG)} + 1.75\underbrace{(11)}_{LL}$$

$= 613.25 \leftarrow 614^{kips}$ for 7.5 ft.

For Pier 2 North, Ratio = 2.4

Permanent load

$= 1.25 \times 257 + 1.5 \times 25 + 1.25(95) = 478 \times 2.4 = 1146^{kips}$

Total Load (Str IV controls)

$684 \times 2.4 = 1640^{kips}$

Pier 2 South. Ratio 3.33

Total load

$684 \times 3.33 = 2280^{kips}$

Pier 3 North, Ratio = 2.87

Total load

$618 \times 2.87 = 1774^{kips}$

Pier 3 South, Ratio = 3.67

Total load

$= 618 \times 3.67 = 2270^{kips}$

FHW
8/17/10

LA River Bridge
Pier Footing loads
Extreme load



Pier 2 North

Total = 1250 + 216 = 1466 kips
(Total Axial load) (EQ Axial)

Pier 2 South

Total = 1730 + 216 = 1946 kips
(Total Axial Ld) (EQ Axial)

Pier 3 Extreme state - form hinge

Pier 3 North
Total = 1370 + 216 = 1586 kips

Pier 3 South
Total = 1750 + 216 = 1966 kips

Report N/A because Pier 3 has no retrofit work.

FHW
8/17/10

LA River Bridge

Abutment Ftg Load
– Service

206

Abutment 1N, permanent Load, per support,

$\Sigma F = (19.1^{k}/LF \times 3.9)$ ↑ surcharge LL $= 15.23^{k}/LF$ ← from RW hand calcs

From WINABUT run from Abut 1 north, see attached page below

$\Sigma F = 48^{k}/LF$

$15.23^{k}/ft \times 23'$ (RW length) $+ 48^{k}/ft \times 8.5'$ (Abut length)

$= 758^{kips}$ say $760^{kips}$ for 16 piles ($47.5^{k}/pile$)

Abut 1 N, Total Load, per support

$\Sigma F = 73^{kip}/LF$ for Abut 1 North (From WINABUT RUN)

$19.1 \times 23 + 73 \times 8.5 = 1060^{kips}$ ($66^{k}/pile$)

* use same data, conservatively estimate Loads for Abut 1 south (South height lower Than North height)

Abut 1 S, Permanent Load, per support

$15.23^{k}/ft \times 1.5$ (RW length) $+ 48^{k}/ft \times 24'$

$= 1327^{kips}$ ($74^{k}/pile$) for 18 piles

too conservative!!

use $47.5^{k}/pile \times 18^{piles} = 855^{kips}$

use $66^{kips} \times 18^{piles} = 1190^{kips}$

(approximation 5%)

Abut 1 South, Permanent Load per support.

Permanent Load

Abutment WT, Ftg WT, Pedestal WT, From Quantity estimate

Ftg = 998 $m^3$
Abut others = 1000 $m^3$ } ≐ 2000 $m^3$

2000 × 0.15 = 300 kips

Soil WT, from Quantity calcs

1945 + 655 = 2600 $m^3$ − 998 $m^3$ − (591 − 175) = 836 ] 2100 $m^3$
10.5 × 53 × 22.8 = 1276
2100 × 0.1 = 252 kips

Dead Load from superstructure: 121 × 2 = 242 kips

Permanent Load = 300 + 252 + 242 = 794 kips vs. 855 estimate
OK

208

Abut 1 North

```
*****************************************
* MOMENTS ABOUT TOE OF FOOTING (K,Ft,K/Ft) * (SERVICE)
*****************************************************************************
```

| | FORCE | ARM | MOMENT |
|---|---|---|---|
| **LATERAL EARTH LOADS (BACKFILL TO TOP OF BACKWALL)** | | | |
| EARTH PRESSURE (KA =0.300) | 32.2 | 6.30 | 203. |
| EARTH PRESSURE (KA =0.225) | 24.1 | 6.30 | 152. |
| SURCHARGE PRESSURE (KA =0.300) | 6.8 | 9.46 | 64. |
| SURCHARGE PRESSURE (KA =0.225) | 5.1 | 9.46 | 48. |
| EARTHQUAKE | 0.0 | 0.00 | 0. |
| **SUPERSTRUCTURE LOADS** | | | |
| DEAD LOAD | 4.5 | 5.38 | -24. |
| H-LIVE LOAD | 0.0 | 5.38 | 0. |
| P-LIVE LOAD | 0.0 | 5.38 | 0. |
| BEARING PAD SHEAR | 0.0 | 16.91 | 0. |
| **OTHER LOADS** | | | |
| OTHER LATERAL LOAD | 20.8 | 10.08 | 210. |
| OTHER VERTICAL LOAD | 251.7 | 3.13 | -788. |
| **ABUTMENT DEAD LOADS** | | | |
| BACKWALL | 2.9 | 6.72 | -20. |
| SEAT | 29.2 | 6.35 | -185. |
| EXTRA SEAT PORTION (HIGH CANTILEVER) | 0.0 | 7.70 | 0. |
| FOOTING | 28.1 | 6.00 | -169. |
| FOOTING KEY | 0.0 | 0.00 | 0. |
| WINGWALL TOP RECTANGULAR SECTION | 0.0 | 6.54 | 0. |
| WINGWALL BOT TRIANGULAR SECTION | 0.0 | 6.93 | 0. |
| WINGWALL EXTRA SECTION (HIGH CANTILEVER) | 0.0 | 7.70 | 0. |
| BARRIER | 0.0 | 5.57 | 0. |
| SIDE KEYS | 0.0 | 5.38 | 0. |
| | 60.2 | | |
| **SOIL DEAD LOADS** | | | |
| SOIL ON HEEL (BACKFILL TO TOP OF BACKWALL) | 42.3 | 9.85 | -417. |
| EXTRA SOIL ON HEEL (HIGH CANTILEVER) | 0.0 | 7.70 | 0. |
| SOIL ON TOE | 0.0 | 2.50 | 0. |
| SURCHARGE ON HEEL | 5.2 | 9.85 | -51. |

Permanent.

Total
= Permanent + LL + LLsurcharge
= 240 + 119 + 5.2
= 364 kips for 5'
73 k/lf

| | | | |
|---|---|---|---|
| Abut WT | 4.5 + 60.2 = 64.7 | → 13 k/ft | 12' HT: 9 k/ft |
| soil WT | 42.3 | 8.46 k/ft | 5.8 k/ft |
| DL from superstructure | 104 | | 8.6 |
| Pedestal | 29 | 5.8 k/ft | 4 kips |
| | 240 kips for 5' Abutment | | |
| | 48 k/lf | | 27.4 k/ft |

13.41 HT

(24'long) TOTAL

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FHW
8/17/10

LA River Bridge
Abutment Pile Load
Service



Abut 5 S permanent Load, per support.

For RW.

$\Sigma F = 35.27 - 5.16 = 30$ K/ft ← From RW hand calc.

From WinAbut run,

$\Sigma F = 51$ K/ft

$51 \times 19.5$ (Abut Length) $+ 30 \times 21.5$ (RW Length)

$= 995 + 645 = 1640$ kips for 25 piles $\Rightarrow 65$ kips/pile

Abut 5 S, Total load, per support

$35.27 \times 21.5 + 67 \times 19.5 = 2065$ kips $\Rightarrow 82.6$ kips/pile

Abut 5 North permanent Load, per support

Estimation, use 65 kips/pile

$65 \times 12 = 780$ kips

Abut 5 North Total Load, per support

$83 \times 12 = 995$ kips

210

Abut 5 South

```
*****************************************
* MOMENTS ABOUT TOE OF FOOTING (K,Ft,K/Ft) * (SERVICE)
*****************************************************************************
```

| | FORCE | ARM | MOMENT |
|---|---|---|---|
| **LATERAL EARTH LOADS (BACKFILL TO TOP OF BACKWALL)** | | | |
| EARTH PRESSURE (KA =0.300) | 111.5 | 8.00 | 892. |
| EARTH PRESSURE (KA =0.225) | 83.6 | 8.00 | 669. |
| SURCHARGE PRESSURE (KA =0.300) | 18.6 | 12.00 | 223. |
| SURCHARGE PRESSURE (KA =0.225) | 13.9 | 12.00 | 167. |
| EARTHQUAKE | 0.0 | 0.00 | 0. |
| **SUPERSTRUCTURE LOADS** | | | |
| DEAD LOAD | 5.4 | 5.38 | -29. |
| H-LIVE LOAD | 0.0 | 5.38 | 0. |
| P-LIVE LOAD | 0.0 | 5.38 | 0. |
| BEARING PAD SHEAR | 0.0 | 23.00 | 0. |
| **OTHER LOADS** | | | |
| OTHER LATERAL LOAD | 34.7 | 15.05 | 522. |
| OTHER VERTICAL LOAD | 466.2 | 3.07 | -1431. |
| **ABUTMENT DEAD LOADS** | | | |
| BACKWALL | 4.6 | 6.92 | -32. |
| SEAT | 123.5 | 6.54 | -808. |
| EXTRA SEAT PORTION (HIGH CANTILEVER) | 0.0 | 8.09 | 0. |
| FOOTING | 67.4 | 6.92 | -466. |
| FOOTING KEY | 0.0 | 0.00 | 0. |
| WINGWALL TOP RECTANGULAR SECTION | 41.7 | 10.96 | -457. |
| WINGWALL BOT TRIANGULAR SECTION | 0.0 | 10.01 | 0. |
| WINGWALL EXTRA SECTION (HIGH CANTILEVER) | 0.0 | 8.09 | 0. |
| BARRIER | 3.2 | 9.80 | -32. |
| SIDE KEYS | 0.0 | 5.38 | 0. |
| | 240 | | |
| **SOIL DEAD LOADS** | | | |
| SOIL ON HEEL (BACKFILL TO TOP OF BACKWALL) | 159.2 | 10.96 | -1745. |
| EXTRA SOIL ON HEEL (HIGH CANTILEVER) | 0.0 | 8.09 | 0. |
| SOIL ON TOE | 0.0 | 2.50 | 0. |
| SURCHARGE ON HEEL | 14.8 | 10.96 | -162. |

Total

Permanent+LL+LLsurcharge

= 660 + 195 + 15
(103+95)

= 870 / 13 = 67 K/LF

Permanent

Abut WT 240 − 5.4 = 245 kips

Soil WT 159 kips

DL from superst 94+79 = 173

Rdwaite 83

660 kips for 13' abut

⇒ 51 K/ft

Abut 5 North, permanent load, per support

From Quantity Calcs.

Abut 5 North Ftg = 402 + 96 = 498 $FT^3$

Abut 5 North stem = 1590 $FT^3$

Abut 5 North DL = 0.15 ( 498 + 1590 ) = 313 kips

Abut 5 North Excavation = 4209 $FT^3$

Backfill = 4209 − 402 − (600 + 112 + 325) = 2770 $FT^3$

Soil WT = 0.12 × 2770 = 332 kips

DL from superstructure = 121 kips

Permanent Load = 313 + 332 + 121 = 766 kips vs 780 kips estimate OK

FUW LA River Bridge 2/2

3/15/10 Deflection

- Girder Deflections will be calculated using SAP2000 model.

<u>First stage</u> – calculate the Deflection under girder dead load only.

1. create SAP 2000 Beam models for the girder length
   A North Girder (South Girder 1 is same)
   B South Girder 2
2. Apply section property of the girder; Girder Sectional Properties only
3. Release moment at hinge
4. Apply Self weight of the girder
5. Need to add diaphragm, steel cross Frame, stiffener wt., say 30 #/LF.

SAP 2000 Model Name

Def_Girder_DL.SDB

Def_Girder_DL_S_EXT.SDB

Second stage

Deck slab Dead Load Reflection.

- Deck Load, use different effective width
- Permanent steel Deck Form 10% additional
- 10% deflection added to account for deflection induced by concrete shrinkage effects.
- use girder section properties only

With Girders

1. effective width (11'-0" span, 4'-6" overhang)
   $w = 1.25$ k/ft
2. Haunch $= 0.05$ k/ft
3. Permanent steel Deck Form 0.13 k/ft

Additional Load from First stage

$1.25 + 0.05 + 0.13 = 1.43$ k/ft

SAP2000 Model Name, Def_ Deckslab.xls.DB

Results Need to $\times 1.1$ to account for shrinkage

Second stage

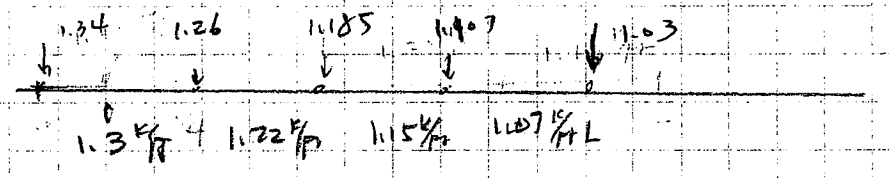
South Interior Girder

1. effective width

$10'' \times (5.5' + 2.75') \times 0.15 = 1.03$ k/ft = span 2, 3, 4

$10'' \times (5.5' + 5.25') \times 0.15 = 1.34$ k/ft

For span 1




2. Haunch = 0.05 k/ft

3. Permanent Steel Deck Form

$0.1 \times (1.3 + 0.05) = 0.135$

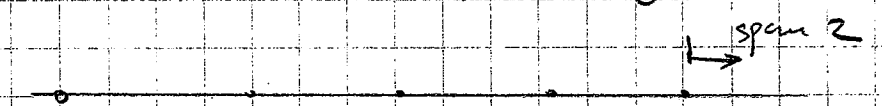
$0.1 \times (1.22 + 0.05) = 0.127$

$0.1 \times (1.15 + 0.05) = 0.12$

$0.1 \times (1.07 + 0.05) = 0.112$

$0.1 \times (1.03 + 0.05) = 0.108$

4. additional load from first stage




| | | | | |
|---|---|---|---|---|
| 1.30 | 1.22 | 1.15 | 1.07 | 1.03 |
| 0.05 | 0.05 | 0.05 | 0.05 | 0.05 |
| 0.135 | 0.127 | 0.12 | 0.112 | 0.108 |
| 1.485 | 1.397 | 1.32 | 1.232 | 1.188 |

SAP2000 Model Name Def_Deckslab_SInt_DL_SDB

Second Stage, South Exterior Girder

1. Effective width

$10'' \times (3 + 5.5/2) \times 0.15 = 0.92^k/ft$ – span 2, 3, 4

span 1,

at 0 pt $10'' \times (4.22 + 5.15/2) \times 0.15 = 0.87$

1/4 span $> 0.85^k/ft$

at 1/4 pt $10'' \times [(4.22 - \frac{(4.22-3)}{4} \times 1) + 5.5/2] \times 0.15 = 0.83$

$> 0.815^k/ft$

1/2 span at 1/2 pt $10'' \times [(4.22 - (\frac{4.22-3}{4} \times 2) + 5.5/2] \times 0.15 = 0.80$

$> 0.78^k/ft$

3/4 span

at 3/4 pt $10'' \times [4.22 - (\frac{4.22-3}{4}) \times 3 + 5.5/2] \times 0.15 = 0.76$

$> 0.74^k/ft$

at 1.0 pt $10'' \times [3 + (5.5/2)] \times 0.15 = 0.72$

2. Haunch $= 0.05^k/ft$

3. Permanent steel Deck Form

$0.1 \times (0.85 + 0.05) = 0.09$

$0.1 \times (0.82 + 0.05) = 0.087$

$0.1 \times (0.78 + 0.05) = 0.083$

$0.1 \times (0.76 + 0.05) = 0.081$

4. Additional load from first stage

span 2 →

| | | | | |
|---|---|---|---|---|
| 0.85 | 0.82 | 0.78 | 0.74 | 0.72 |
| 0.05 | 0.05 | 0.05 | 0.05 | 0.05 |
| 0.09 | 0.087 | 0.083 | 0.081 | 0.09 |
| 0.99 | 0.95 | 0.91 | 0.87 | 0.86 |

SAP 2000 model name Def_ Deck slab_SEXT_DL.SAP

FHW
3/15/10

# LA River Bridge
## Deflection



**Third stage**

Added Dead Load deflection.

- SAP 2000, Girder WT, concrete WT all set to zero
- Barrier load applied as line load into DEAD LOAD CASE
- wearing surface as WS Load

Deflection due to

- Barrier WT, pound walls, utilities, and future overlay.
- This dead load is resisted by the long-term composite section girder.

SAP 2000 Model Name. Def_ composite _ AddDL. ( 151.5 section
Def _ composite _ AddDL_R ( 145' section

North Girder

(1) look at DL+WS Load combination.

(2) Result look at 1/2 p.r.

| X/L | span 1 ΔU (in) | X/L | span 2 ΔU (in) | X/L | span 3 ΔW | X/L | span 4 ΔW |
|---|---|---|---|---|---|---|---|
| 0.07 | ~~-0.003~~ -0.003 | 0.06 | — (WD) see next page | 0.06 | -0.03 (WD) | 0.06 | — (WD) |
| 0.13 | ~~-0.007~~ ~~-0.006~~ | 0.12 | -0.02 | 0.12 | -0.06 | 0.12 | 0.01 |
| 0.19 | ~~-0.009~~ ~~-0.008~~ | 0.17 | -0.04 | 0.17 | -0.08 | 0.19 | 0.01 |
| 0.25 | ~~-0.01~~ ~~-0.010~~ | 0.22 | -0.06 | 0.22 | -0.11 | 0.26 | -0.02 |
| 0.31 | ~~-0.013~~ ~~-0.011~~ | 0.27 | -0.08 | 0.27 | -0.13 | 0.34 | -0.05 |
| 0.37 | ~~-0.013~~ ~~-0.012~~ | 0.32 | -0.09 | 0.32 | -0.15 | 0.40 | -0.08 |
| 0.43 | ~~-0.013~~ ~~-0.012~~ | 0.37 | -0.10 | 0.37 | -0.16 | 0.45 | -0.09 |
| 0.49 | ~~-0.013~~ ~~-0.012~~ | 0.42 | -0.11 | 0.42 | -0.17 | 0.51 | -0.10 |
| 0.55 | ~~-0.012~~ ~~-0.010~~ | 0.47 | -0.11 | 0.47 | -0.18 | 0.57 | -0.11 |
| 0.60 | ~~-0.009~~ ~~-0.009~~ | 0.52 | -0.11 | 0.52 | -0.18 | 0.63 | -0.11 |
| 0.66 | ~~-0.007~~ ~~-0.007~~ | 0.58 | -0.11 | 0.58 | -0.17 | 0.69 | -0.11 |
| 0.74 | ~~-0.003~~ ~~-0.004~~ | 0.63 | -0.10 | 0.63 | -0.16 | 0.75 | -0.09 |
| 0.81 | -0.01 | 0.68 | -0.08 | 0.68 | -0.15 | 0.81 | -0.08 |
| 0.88 | — | 0.73 | -0.06 | 0.73 | -0.13 | 0.87 | -0.06 |
| 0.94 | — | 0.78 | -0.04 | 0.78 | -0.10 | 0.93 | -0.03 |
| 1.00 | — | 0.83 | -0.01 | 0.84 | -0.08 | 1.00 | — |
| | | 0.89 | 0.01 | 0.89 | -0.05 | | |
| | | 0.94 | 0.01 | 0.95 | -0.03 | | |
| | | 1.00 | — | 1.00 | — | | |

FHW LA River Bridge 217

3/17/10 Deflection

Sixth Girder, Interior Girder

(1) look at DL+WS load comb.

(2) look at U2 Pir.

| Span 1 | | Span 2 | | Span 3 | | Span 4 | |
|---|---|---|---|---|---|---|---|
| X/L | ΔU (m) | X/L | ΔU | X/L | ΔU | X/L | ΔU |
| 0.07 | — | 0.06 | — | 0.06 | -0.01 | 0.06 | — |
| 0.13 | -0.01 | 0.12 | -0.01 | 0.12 | -0.01 | 0.12 | — |
| 0.19 | -0.01 | 0.17 | -0.01 | 0.17 | -0.01 | 0.19 | — |
| 0.25 | -0.01 | 0.22 | -0.01 | 0.22 | -0.02 | 0.26 | — |
| 0.31 | -0.01 | 0.27 | -0.02 | 0.27 | -0.02 | 0.34 | -0.01 |
| 0.37 | -0.01 | 0.32 | -0.02 | 0.32 | -0.02 | 0.40 | -0.01 |
| 0.43 | -0.01 | 0.37 | -0.02 | 0.37 | -0.02 | 0.45 | -0.01 |
| 0.49 | -0.01 | 0.42 | -0.02 | 0.42 | -0.02 | 0.51 | -0.01 |
| 0.55 | -0.01 | 0.47 | -0.02 | 0.47 | -0.02 | 0.57 | -0.01 |
| 0.60 | -0.01 | 0.52 | -0.02 | 0.52 | -0.02 | 0.63 | -0.01 |
| 0.66 | -0.01 | 0.58 | -0.02 | 0.58 | -0.02 | 0.69 | -0.01 |
| 0.74 | — | 0.63 | -0.01 | 0.63 | -0.02 | 0.75 | -0.01 |
| 0.81 | — | 0.68 | -0.01 | 0.68 | -0.02 | 0.81 | -0.01 |
| 0.88 | — | 0.73 | -0.01 | 0.73 | -0.02 | 0.87 | -0.01 |
| 0.94 | — | 0.78 | — | 0.78 | -0.01 | 0.93 | — |
| 1.00 | — | 0.83 | — | 0.83 | -0.01 | 1.00 | — |
| | | 0.89 | — | 0.89 | -0.01 | | |
| | | 0.94 | — | 0.95 | -0.01 | | |
| | | 1.00 | — | 1.00 | — | | |

PTW
3/17/10

LA River Bridge
Deflection

South Girder, Exterior Girder

| Span 1 | | Span 2 | | Span 3 | | Span 4 | |
|---|---|---|---|---|---|---|---|
| X/L | ΔU (in) | X/L | ΔU | X/L | ΔU | X/L | ΔU |
| 0.07 | — | 0.06 | — | 0.06 | -0.01 | 0.06 | — |
| 0.13 | -0.01 | 0.12 | -0.01 | 0.12 | -0.1 | 0.12 | — |
| 0.19 | -0.01 | 0.17 | -0.01 | 0.17 | -0.01 | 0.19 | — |
| 0.25 | -0.01 | 0.22 | -0.01 | 0.22 | -0.02 | 0.26 | — |
| 0.31 | -0.01 | 0.27 | -0.01 | 0.27 | -0.02 | 0.34 | -0.01 |
| 0.37 | -0.01 | 0.32 | -0.02 | 0.32 | -0.02 | 0.40 | -0.01 |
| 0.43 | -0.01 | 0.37 | -0.02 | 0.37 | -0.02 | 0.45 | -0.01 |
| 0.49 | -0.01 | 0.42 | -0.02 | 0.42 | -0.02 | 0.51 | -0.01 |
| 0.55 | -0.01 | 0.47 | -0.02 | 0.47 | -0.02 | 0.57 | -0.01 |
| 0.60 | -0.01 | 0.52 | -0.02 | 0.53 | -0.02 | 0.63 | -0.01 |
| 0.66 | -0.01 | 0.58 | -0.01 | 0.58 | -0.02 | 0.69 | -0.01 |
| 0.74 | — | 0.63 | -0.01 | 0.63 | -0.02 | 0.75 | -0.01 |
| 0.81 | — | 0.68 | -0.01 | 0.68 | -0.02 | 0.81 | -0.01 |
| 0.88 | — | 0.73 | -0.01 | 0.73 | -0.02 | 0.87 | -0.01 |
| 0.94 | — | 0.78 | — | 0.78 | -0.01 | 0.93 | — |
| 1.00 | — | 0.83 | — | 0.83 | -0.01 | 1.00 | — |
| | | 0.89 | — | 0.89 | -0.01 | | |
| | | 0.94 | — | 0.95 | — | | |
| | | 1.00 | — | 1.00 | — | | |

FHW L.A. River Bridge 219

3/29/10 Deck Drain Calculation

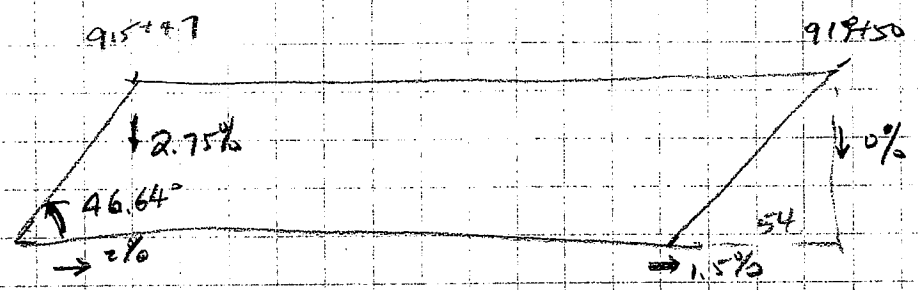
Deck width between Rails

$(74'-6") - (3'-6") = 71'-0"$ Min

$78' - (3'-6") = 74-6"$ Max } L

skew = 46.64°

Structure Length = 403'




n (mannings) = 0.016 (PCC with broom finish)

$$Q = (K/n)\, S_x^{5/3}\, S^{1/2}\, T^{8/3}$$

K = 0.56 $S_x$, say = 2% (0.02 ft/ft), S = 1.5% (0.015 ft/ft)

T = width of spread, say 10' shoulder

$$Q = \left(\frac{0.56}{0.016}\right)(0.02)^{1.67}(0.015)^{0.5}(10)^{2.67}$$

$$= 35 \times (0.00145) \times (0.12247) \times (467.7)$$

$$= 2.9 \text{ ft}^3/\text{s}$$

Flow depth at edge of drain

$$d = (10 - 1.5) \times 0.02 = 0.17 \text{ ft}$$

Flow bypassing grate

$$Q_b = \left(\frac{0.56}{0.016}\right)(0.02)^{1.67}(0.015)^{0.5}(10-1.5)^{2.67} = 1.88 \text{ ft}^3/\text{s}$$

Flow with grate width

$$Q_W = 2.9 - 1.88 = 1.02 \text{ ft}^3/\text{s}$$

check $L_b$ length for clear opening.

depth at face of curb

$$d = TS_x = 10(0.02) = 0.2$$

$$L_b = \frac{V}{2}(d + d_b)^{1/2}$$

$$V = \frac{Q}{A} = \frac{Q}{\frac{1}{2}\phi(d+d_b)} = \frac{1.02\ ft^3/s}{\frac{1}{2}(1.5)(0.17+0.2)} = 3.67$$

$$L_b = \frac{3.67}{2}(0.17+0.2)^{1/2} = \underline{1.11}\ ft < 1.37\ ft\ OK.$$

check capacity of drain outlet using equation (3).

$$d = 0.522 + 0.2 = 0.742$$

$$Q_o = 5.37\,A\,(d)^{1/2}$$

$$= 5.37 \times (0.194)(0.742)^{1/2} = \underline{0.90}\ ft^3/s < Q = 1.02\ ft^3/s$$

bypass flow $Q_b = 2.9 - 0.9 = 2.0\ ft^3/s$

Determine maximum distance to 1st Drain from begin bridge

$$Q = 0.000115\,LW$$

$$L = \frac{2.9\ ft^3/s}{0.000115 \times (71+74.5)/2} = 347\ ft.$$

Determine maximum spacing between intermediate drains.

$$L = \frac{0.9}{0.000115(71+74.5)/2} = 108\ ft \rightarrow \text{say } 110\ ft.$$

Drain number needed $\frac{464}{110} = 3.67 \rightarrow$ say 4.

Check Drain pipe capacity.

$$V = \frac{1.49}{n} R^{2/3} S_0^{1/2}$$

$$Q = VA$$

For Full circular pipe $R = D/4$.

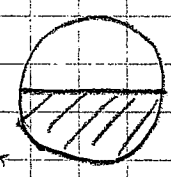
For $D = 8''$. $R = \frac{8}{4} = 2''$

$n = 0.015$ $S_0 = 0.02$

$$V = \frac{1.49}{0.015}\left(\frac{2}{12}\right)^{2/3}(0.02)^{1/2} = 4.25 \text{ ft/s}$$

For Half Full circular pipe.

$$R = \frac{\text{Area of section flow } (A)}{\text{wetted perimeter } (P_w)}$$



$$P_w = \frac{\pi \times D}{2} \qquad A = \left(\tfrac{1}{4}\pi D^2\right) \times \tfrac{1}{2} = \tfrac{1}{8}\pi D^2$$

$$R = \frac{\frac{1}{8}\pi D^2}{\frac{1}{2}\pi D} = \frac{1}{4} D.$$

$\therefore$ For half full $V = 4.25$ ft/s $> 3$ ft/s

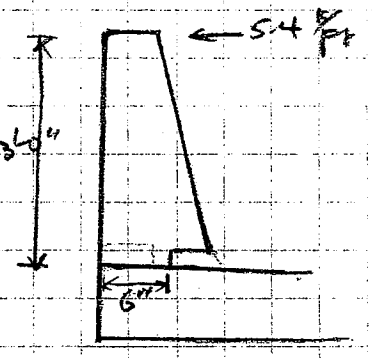
self cleaning

$$Q = VA = 4.25 \times \frac{1}{4}\pi \times \left(\frac{8}{12}\right)^2 = 1.48 \text{ ft}^3/\text{s}$$

FHW LA River Bridge 222

4/1/10 Barrier design @ Deck Drain opening

Moment @ base of Barrier




$5.4\ k/ft \times 3 = 16.2\ k\text{-}ft/ft$

CT, extreme event II

$\gamma_p = 1.0$

$M_n = A_s f_y (d - a/2)$ , $M_u = \phi M_n$ , $\phi = 0.9$

$d = 3''$ $f_y = 60{,}000$ psi. $a = \dfrac{A_s f_y}{0.85 f_c' b}$

$a = \dfrac{A_s \times 60{,}000}{0.85 \times 3{,}250 \times 12''} = 1.81 A_s$

$A_s = 1.2\ in^2/ft$ #7 @ 6" ← Can not be provided due to limited opening

$a = 2.17$

$M_n = 1.2 \times 60{,}000 \left(4 - \dfrac{2.17}{2}\right) = 210\ k\text{-}in/ft$

Demand

$16.2\ k\text{-}ft/ft / 0.9 = 18\ k\text{-}ft/ft = 216\ k\text{-}in/ft$

Provide #5 L @ 4" $A_s = 0.92\ in^2/ft$

Then Distributed outside the opening

Also, make horizontal bar #5.

Drainage - Bracing Design

- All vertical loads are carried by hangers
- Use UBC code for design.

Dead Loads

8" pipe . 29 PLF

water in pipe: $\frac{\pi}{4} \times \left(\frac{8}{12}\right)^2 \times 62.5$ PCF

$= 21.8$ PLF

51 PLF

$C_a = \frac{1.58}{2.5} = 0.632 \approx 0.6$ use $C_a = 0.6$

For attachments UBC

$F_p = 4.0 \times C_a \times I \times W_p$ (1632.2)

$\Rightarrow F_p = 4 \times 0.6 \times 1.0 \times W_p = 2.4 W_p$

Determine spacing. use ½" φ MEA for anchorage

$\phi V_n = 1.5$ kips

$\frac{\phi V_n}{F_p} = \frac{1.5}{2.4 \times 0.051} = 12.25'$

So need to provide 1 - ½" φ Bolt every 12.5'.
~~or~~ use 1 - ¾" φ bolt every 20'
or use 2 - ½" φ bolt every 20'

every 25' wt

$25 \times 0.051 = 1.275^{k/o} < 1.450^{k/o}$ ok

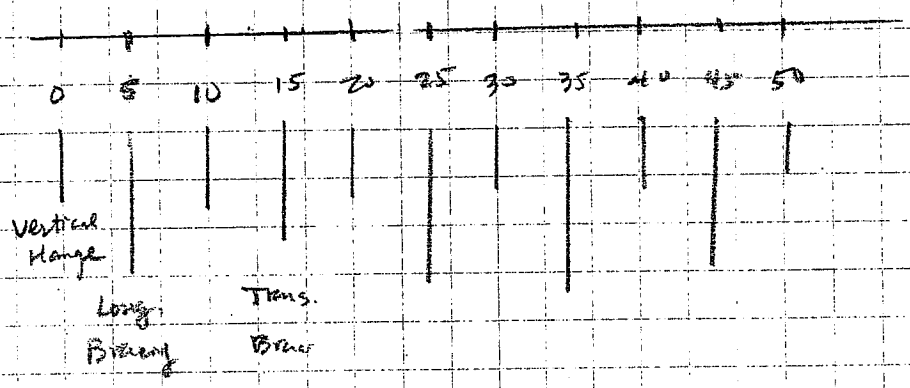
FMW

9/23/10

LA River Bridge

Bridge Deck Drainage






Have ½" φ Anchor Bolt every 5'. OK.

calculations, the PHBT is the nearest and controlling fault for this site. The $r_{rup}$ for the PHBT is about 5.8 km. The median or the design PBA and PGA at the site should be taken as 0.6g and 0.54g, respectively, with an MCE of moment magnitude M=7.25.

The project site is not located within any CGS designated Earthquake Fault Zone (EFZ) or directly underlain by any known fault considered active for bridge design. Therefore, the subject bridge site is not considered prone to surface fault rupture hazard.

Based on the available As-Builts, the Soil Profile Type for the proposed widening bridge may be classified as Type D. Standard Caltrans ARS corresponding to a PBA of 0.6g and a moment magnitude M = 7.25 were modified to account for the fault rupture directivity effect. The recommended ARS Curve and the coordinates of spectral accelerations are presented in Figure 1 and Table 2 below, respectively.

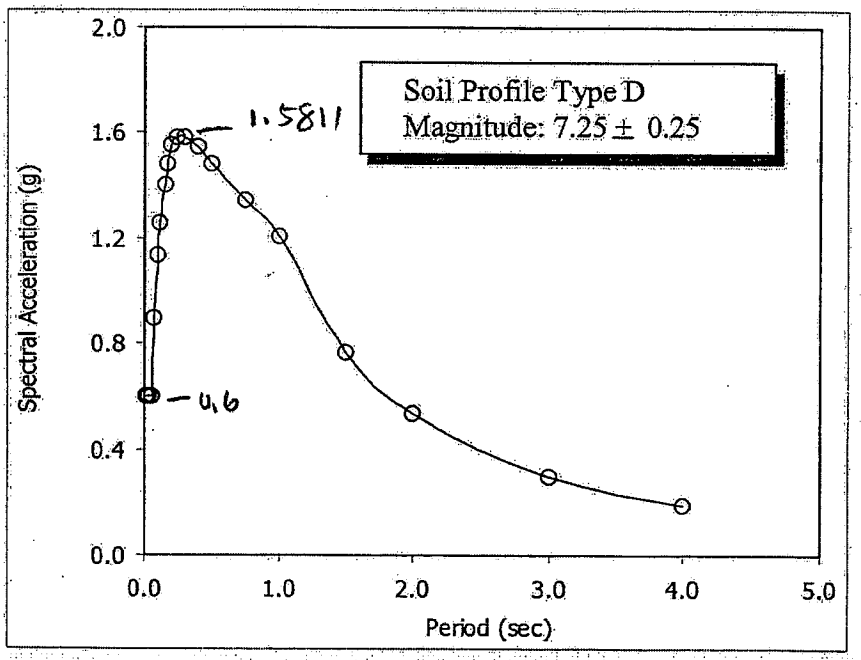



**Figure 1 ARS curve for the site**

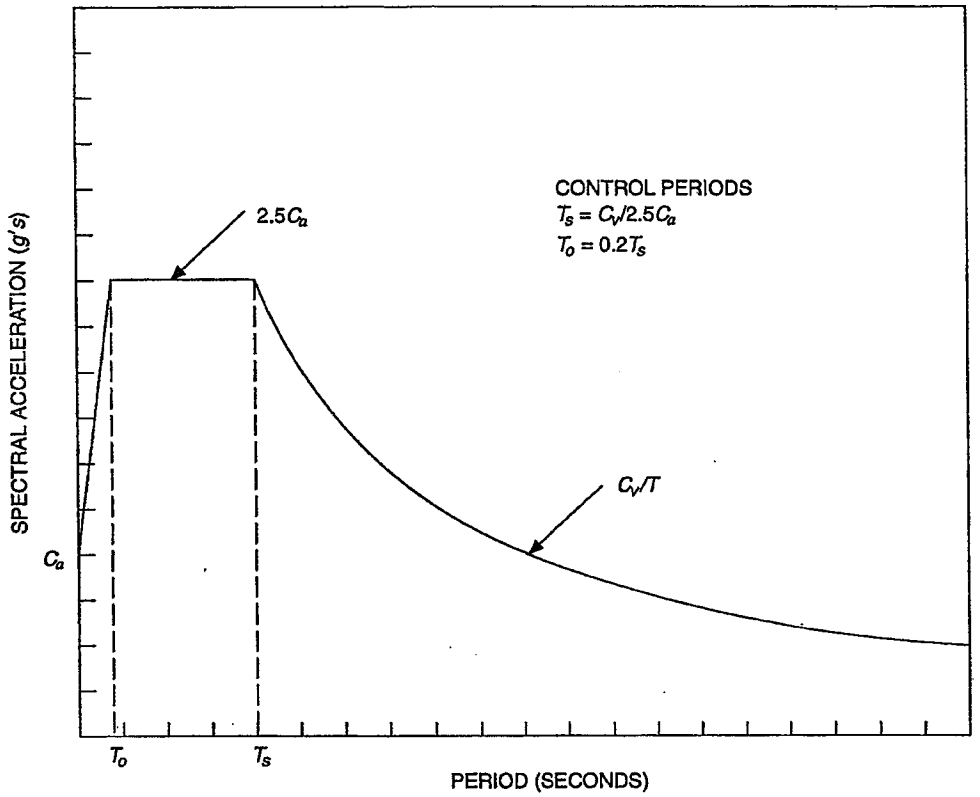
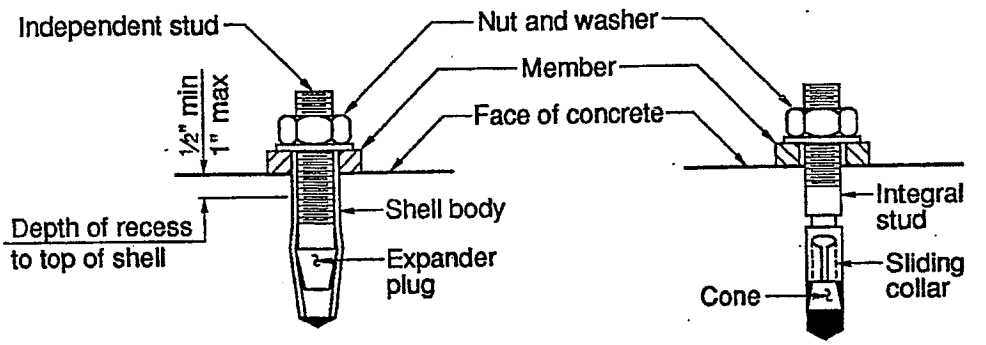



**FIGURE 16-3—DESIGN RESPONSE SPECTRA**




Typical Shell-Type MEA

Typical Stud-Type MEA

**Figure A.1 – Common Types of Mechanical Expansion Anchors (MEA)**

**Table A.1 – Design Data for Shell and Stud-Type Mechanical Expansion Anchors**

| Stud diameter, inches | Shear strength, kips | Tensile strength, kips |
|---|---|---|
| ¼ | 0.4 | 0.4 |
| ⅜ | 0.8 | 1.0 |
| ½ | 1.5 | 1.1 |
| ⅝ | 2.1 | 2.1 |
| ¾ | 2.4 | 2.4 |

The designer should take the following items in consideration when selecting Mechanical Expansion Anchors:

a) Design strengths shown in Table A.1 are for static load conditions only; when dynamic loading governs or for critical applications such as installations over traffic, resin capsule anchors, or grouted or bonded anchors are recommended.
b) Design strengths shown above are based on normal weight concrete having $f'_c = 4000$ psi; for $f'_c = 3250$ psi, multiply values by 0.85; for $f'_c = 5000$ psi multiply values by 1.17. For light weight concrete and other special conditions consult with Translab.
c) If a single MEA is used to hold an attachment, the design strengths allowed in Table A.1 should be reduced by one-half.

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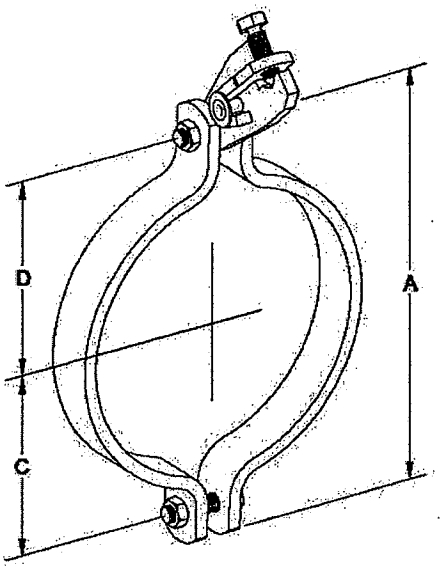
# Fig. 4L
# Longitudinal "In-Line" Sway Brace Attachment

c UL US LISTED
FM APPROVED




**Size Range** — 2" through 8" IPS.

**Material** — Carbon Steel

**Function** — For bracing pipe against sway and seismic disturbance.

**Approvals** — Underwriter's Laboratories Listed in the USA **(UL)** and Canada **(cUL)** 2" - 8". Approved by Factory Mutual Engineering **(FM)**, 2" - 8" pipe.

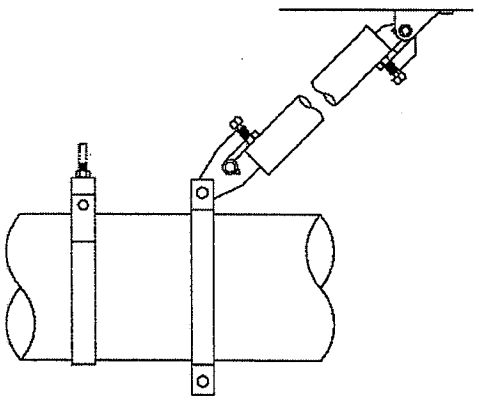
**Installation Instructions** — The Fig. 4L is the "braced pipe" attachment component of a longitudinal sway brace assembly. It is intended to be combined with the "bracing pipe" and TOLCO structural attachment component to form a complete bracing assembly. NFPA 13 and/or OSHPD guidelines should be followed.

**To Install** — Place the Fig. 4L over the pipe to be braced and tighten bolts. Then engage "bracing pipe" into jaw opening and tighten set bolt until hex head snaps off. Jaw attachment can pivot for adjustment to proper brace angle.

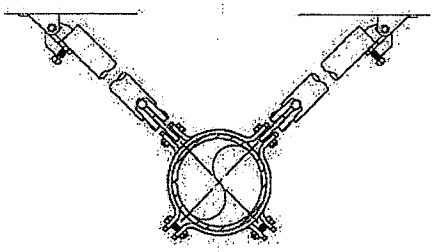
**Finish** — Plain

**Note** — Available in Electro-Galvanized and HDG finish.

**Order By** — Figure number, pipe size and finish.



Longitudinal Brace



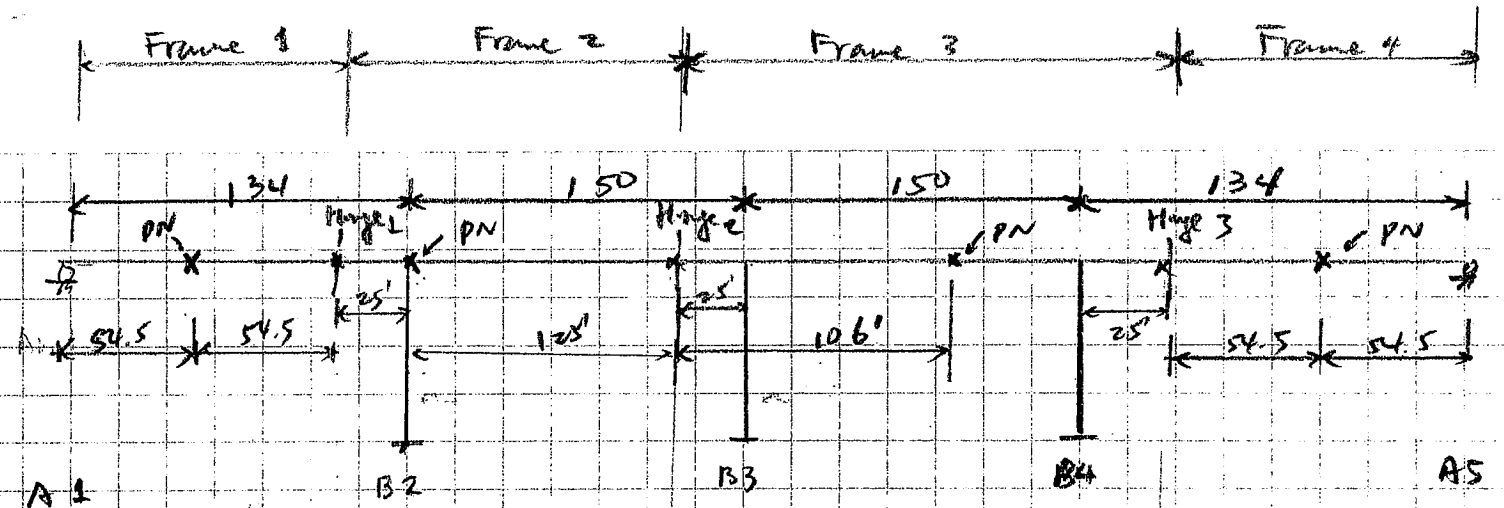
4-Way Riser Brace
(Plan view)

**Dimensions • Weights**

| Sizes | A | C | D | Bolt Size | Max. Rec. Load Lbs. (cULus) | *Max Rec. Load Lbs. (FM) | Approx. Wt./100 |
|---|---|---|---|---|---|---|---|
| 2½ | 6 7/16 | 2½ | 2¾ | 1/2 | 2015 | 3000 | 253 |
| 3 | 7 | 2¾ | 3 1/16 | 1/2 | 2015 | 1550 | 268 |
| 4 | 8½ | 3⅜ | 3 11/16 | 1/2 | 2015 | 1550 | 348 |
| 5 | 9¾ | 3⅞ | 4⅜ | 1/2 | 2015 | 1450 | 380 |
| 6 | 11½ | 5 | 5⅛ | 1/2 | 2015 | 1450 | 640 |
| 8 | 13¼ | 6 11/16 | 6⅛ | 1/2 | 2015 | 1450 | 728 |

* Load shown is allowable with brace installed, between 30° - 90°. No reduction of load based on brace angle is required.
FM approved when used with 1", 1¼", 1½" or 2" Sch. 40 brace pipe.

TOLCO® brand bracing components are designed to be compatible **ONLY** with other TOLCO® brand bracing components, resulting in a Listed seismic bracing assembly. **DISCLAIMER** — NIBCO does **NOT** warrant against the failure of TOLCO® brand bracing components, in the instance that such TOLCO® brand bracing components are used in combination with products, parts or systems which are not manufactured or sold under the TOLCO® brand. NIBCO shall **NOT** be liable under any circumstance for any direct or indirect, incidental or consequential damages of any kind, including but not limited to loss of business or profit, where non-TOLCO brand bracing components have been, or are used.




| | | | |
|---|---|---|---|
| I $(ft)^4$ | $\frac{1}{12} \times 145 \times (3.5)^3 = 518$ | 518 | 518 |
| L (ft) | 30.4' | 36' | 34.2' |
| P @ 1" sidewaay (kips) | $\frac{108 I}{(\frac{L}{10})^3} = 1991$ | 1199 | 1399 |
| D (ft) distance from ℄ member of frame | 25' | 25' | 175' |
| $\frac{P \times D}{100}$ | $\frac{1991 \times 25}{100} = 498$ | $\frac{1199 \times 25}{100} = 300$ | $\frac{1399 \times 175}{100} = 2448$ |
| X (Point of PN no movement) $\frac{\Sigma (P \times D)/100}{\Sigma P}$ | $\frac{498 \times 100}{1991} = 25'$ | $\frac{(300 + 2448)}{(1199 + 1399)} \times 100 = 106'$ | |

TATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

# JOINT MOVEMENTS CALCULATIONS [a]

S-D 0129 (Rev. 7/98)

**Note:** Specific Instructions are included as footnotes.

| A | DISTRICT | COUNTY | ROUTE | PO | BRIDGE NAME AND NUMBER |
|---|---|---|---|---|---|
| 02101 | 7 | LA | 710 | 17.3 | Los Angeles River Bridge #53-0828 |

| YPE STRUCTURE | TYPE ABUTMENT | TYPE EXPANSION (50 MM ELASTOMERIC PADS, ETC.) |
|---|---|---|
| Steel | | |

TEMPERATURE EXTREMES (from preliminary report)

| | | | Type of Structure | | | | 2 THERMAL MOVEMENT (inches/100 feet) | | ANTICIPATED SHORTENING (inches/100 feet) | | 3 MOVEMENT FACTOR (inches/100 feet) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Maximum | 112 | °F | Steel | Range ( | 60 | °F) X (0.0000065 X 1200) = | 0.47 | + | 0.00 | = | 0.47 |
| Minimum | 24 | °F | Concrete (Conventional) | Range ( | 0 | °F) X (0.0000060 X 1200) = | 0.00 | + | 0.06 | = | |
| | | | Concrete (Pretensioned) | Range ( | 0 | °F) X (0.0000060 X 1200) = | 0.00 | + | 0.12 | = | |
| Range | 88 | °F | Concrete (Post tensioned) | Range ( | 0 | °F) X (0.0000060 X 1200) = | 0.00 | + | 0.63 | = | |

| EM 1 DESIGNER | Date: | ITEM 2 CHECKED BY | Date: |
|---|---|---|---|
| rank Wei | 4/12/2010 | | |

o be filled in by Office of Structures Design [b] — To be filled in by SR: [c] — Date:

| Location | Skew (degrees) do not use in calcs. | 4 Contributory Length (ft) | Calculated Movement 3 X 4 /100 (in) | Movement Rating (mm) | Seal Type A,B or other or Open Joint | Seal Width Limits [d]: Catalog Number | W 1 max (mm) | 5 W 2 min @ max Temp. (mm) | Groove (saw cut) Width or Installation Width [e]: Structure Temp. °F | 6 Adjust from Max. Temp °/ 1 X 2 X 4 /100 (mm) | Width at Temp Listed W = 5 + 6 (mm) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| butment 1 | 46.6 | 55 | 0.26 | | AP | | | | | | |
| | | | | | | | | | | | |
| inge 1, Span 1 | 46.6 | 55 | 0.26 | | | | | | | | |
| inge 1, Span 2 | 46.6 | 25 | 0.12 | | | | | | | | |
| inge 1 total | | | 0.37 | | AP | | | | | | |
| | | | | use 1" open Jt. | | | | | | | |
| inge 2, Span 2 | 46.6 | 125 | 0.59 | match existing | as well | | | | | | |
| inge 2, Span 3 | 46.6 | 106 | 0.50 | | | | | | | | |
| inge 2 total | | | 1.08 | | AP | | | | | | |
| | | | | | | | | | | | |
| nge 3, Span 3 | 46.6 | 94 | 0.44 | | | | | | | | |
| nge 3, Span 4 | 46.6 | 55 | 0.26 | | | | | | | | |
| nge 3 total | 46.6 | | 0.70 | | AP | | | | | | |
| | | | | | | | | | | | |
| butment 5 | 46.6 | 55 | 0.26 | | AP | | | | | | |
| | | | | | | | | | | | |

* Joint seal size increased to fit existing opening (W1).

Project Designer: Send to RE or SR with report.

Show line drawing of structure on reverse side; show points of no movement and contributory lengths.

etain copy for design calculations file.

RE or SR: Complete and return to structure construction with final report.

[d] Type B information from Translab reports.

[e] Groove width adjustment based on °= *(maximum temperature extreme)* **minus** *(superstructure temperature).*

[f] Measure superstructure temperature by placing bulb of concrete thermometer **152 mm** into expansion joint.

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FMW
10/5/10

LA River Bridge
Bearing Pad

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$1.66GD = 1.66 \times (0.08) \times 13.3 = 1.78^{ksi} \leq 1.6^{ksi}$

↑ overm.

$0.66GS = 0.66 \times (0.08) \times 13.3 = 0.638^{ksi}$

Pad compressive stress criterion

Temperature movement

$1.5 \times 0.0000065 \times 60^{\circ} \times 12 \times 125 = 0.88''$ (Horizontal movement)

Minimum Pad Thickness $= 2 \times 0.88 = 1.76'' < 2''$ ok

Compressive Deflection by inspection ok

Actual Design shear force

$$\frac{169 \times (24 \times 30) \times 0.88''}{2'' \times 1000} = 53.5^{(kips)}$$

$\frac{PL}{5} = 340/5 = 68^{kips}$ (allowable) okay

At Pier Wall

Bearing Pad size 24" x 30" x 2"

Service load DC1 + DC2 + DW

$(152.77 + 132.67 + 25.17 + 24.24 + \underbrace{32.61 + 29.69}_{62.3^{kips}})$

$= 402^{kips}$

HL-93 (LL+IM) = $145^{kips}$

$\sigma_s = \frac{402}{24 \times 30} = 558^{psi}$ (with wearing surface)

$= \frac{340}{24 \times 30} = 472^{psi}$ (without wearing surface)

> 200 psi OK

$\sigma_L = \frac{145}{24 \times 30} = 201^{psi}$

G = shear modulus of elastomers (ksi)

= 0.08 ~ 0.175 ksi

S. shape factor of the thickest layer of the Bearing.

Assuming 1/2" thickness layer.

$S = \frac{LW}{2h(L+W)} = \frac{24 \times 30}{2 \times 1/2 \times (24+30)}$

$= 13.3 > 7.5$ OK

At Abutment. Bearing Pad size 18" x 18" x 1½"

Temperature movement =

$1.5 \times 0.00000065 \times 600 \times 12 \times 55 = 0.386''$

minimum pad thickness = $2 \times 0.386 = 0.77 < 1.5$ ok.

Service load DC1 + DC2 + DW

$= 104.28 + 17.69 + \underbrace{22.37}_{WS} = 144.34^{kips}$

HL-93 (LL + IM) = $120^{kips}$

$\sigma_D = \dfrac{144.34}{18 \times 18} = 446^{psi}$ (with wearing surface)

$= \dfrac{122}{18 \times 18} = 376^{psi} > 200^{psi}$ ok

(without wearing surface)

$\sigma_L = \dfrac{120}{18 \times 18} = 0.370^{ksi}$

$S = \dfrac{18 \times 18}{2 \times 1\frac{1}{2} \times (18 + 18)} = 9 > 7.5$ ok

$1.66\,GS = 1.66 \times (0.08) \times 9 = 1.19^{ksi} < 1.6^{ksi}$

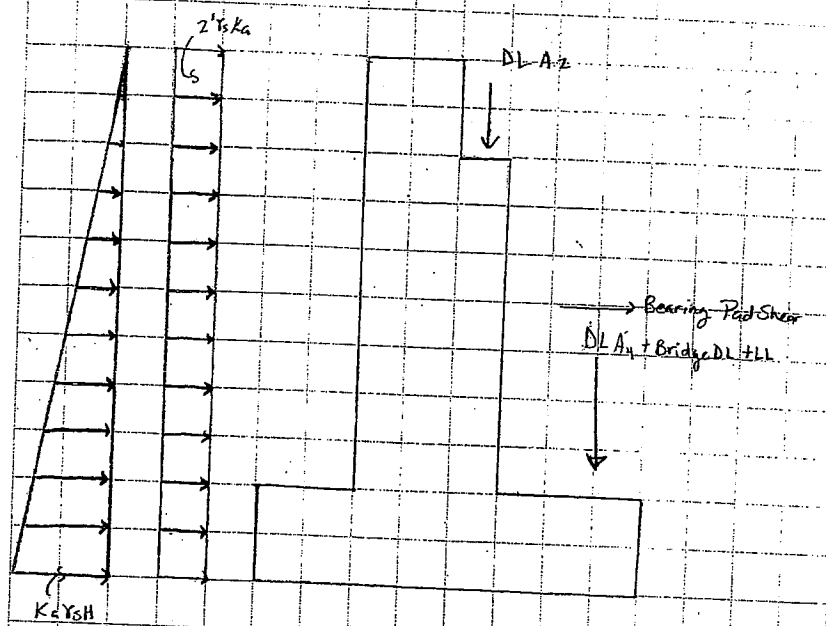
use

$0.66\,GS = 0.66 \times (0.08) \times 9 = 0.475^{ksi} > 0.370^{ksi}$ ok.

Actual Design shear force

$\dfrac{169 \times (18 \times 18) \times 0.386}{1.5'' \times 1000} = 14^{kips}$

$\dfrac{DL}{S} = 122/5 = 24.4 > 14^{kips}$ ok.




Please see sheet 1 for area ($A_i$) locations.

DL of $A_2$

$A_2 = (2' \times 1.75') + (1 \times .75) + ((1' \times 1')/2) = 3.50 + .75 + 0.$
$= 4.75\ ft^2$

$DL\ A_2 = 4.75 \times 0.15 \times 6.25'$
$= 4.45$ Kips

DL of $A_4$

$A_4 = 7.58' \times 5' = 37.90\ ft^2$

$DL\ A_4 = 37.90 \times 5' \times 0.15$
$= 28.43$ Kips

- In Win Abut applied DL of $A_2$ as Dead Load of structure.
- Applied actual dead load of bridge, live load (unfactored), the pedestals weight and 2ft of soil next to pedestal as other vertical load.

DL of Soil next to pedestal

$= 0.63' \times 2' \times 0.120\ kcf \times 2$
$= 0.30\ K$

Input for Win Abut

↳ DL = 4.45 Kips

↳ Other VL = 28.43K + 91.502K + 90.423 + 0.3K
= 210.66K

Rev. 1, Other VL = 28.43K + 104 + 119 + 0.3 = 251.73K

Bearing Pad Shear

$F_s = 20\%\ DL$
$= 0.2(91.502)$
$= 18.30$ Kips

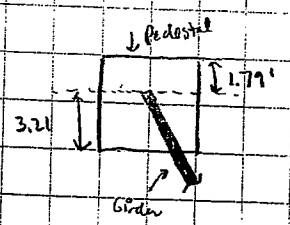
* Applied as other lateral load.

Rev. 1 = $0.2(104)$
$= 20.80$ Kips

To calculate centroid for DL+LL & pedestal.

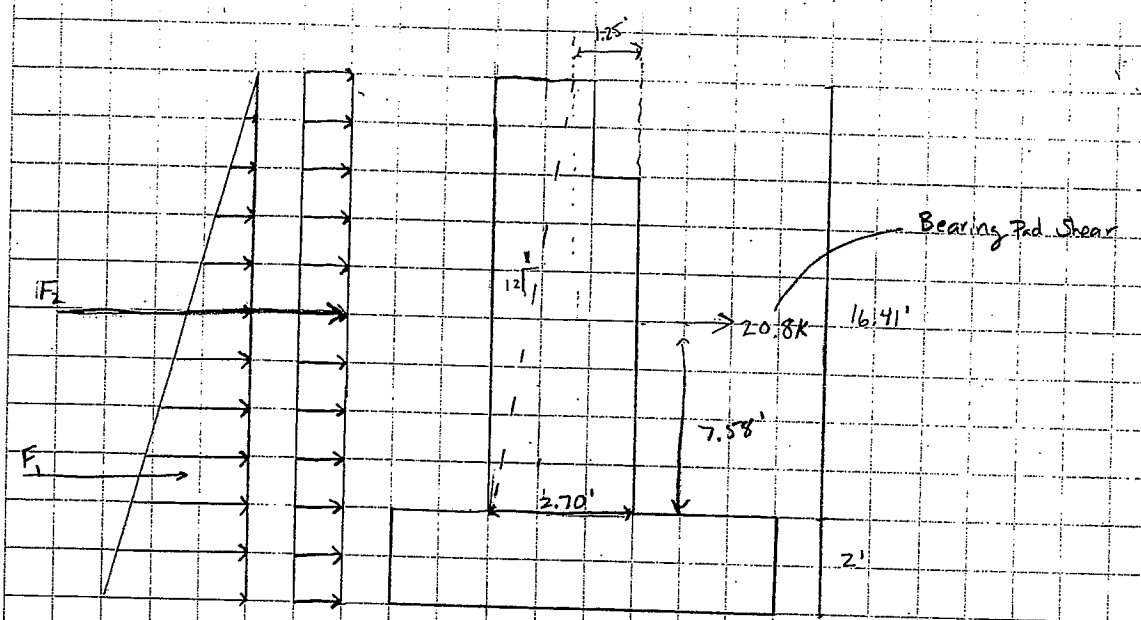
| | Force | Rev. 1 | Arm |
|---|---|---|---|
| DL+LL | 181.93 Kips | 223 | 3.21' |
| Pedestal | 28.43 Kips | 28.43 | 2.5' |
| | ΣF = 210.36 Kips | 251.43 | |

| Moment Arm | Rev. 1 |
|---|---|
| 584.00 K-ft | 715.83 K-ft |
| 71.08 K-ft | 71.08 K-ft |
| ΣM = 655.08 K-ft | ΣM = 786.91 |




$\frac{\Sigma M}{\Sigma F} = \frac{655.08}{210.36} = 3.11'$ from toe of footing

Rev. 1 $\frac{\Sigma M}{\Sigma F} = \frac{786.91K}{251.43} = 3.13'$ from toe of footing.




Calculating Vertical Stem Reinforcement

@ Top of Footing $H = 16.41'$

$F_1 = \frac{1}{2}\gamma K_a H^2 = \frac{1}{2}(0.120)(0.3)(16.41)^2 = 4.85$ K/LF

$F_2 = 2'\gamma K_a H = 2'(0.120)(0.3)(16.41) = 1.18$ K/LF

$\circlearrowleft M = 4.85$ K/LF $(16.41/3) + 1.18(16.41/2) + (20.8K/6.25' \times 7.58') = 26.53 + 9.68 + 25.23$
$= 61.44$ K-ft/LF

Try #9 @ 12" $jd = 26.87''$ from excel calc sheet

$f_s = \frac{M}{A_s jd} \Rightarrow A_s = \frac{M}{f_s jd} = \frac{61.44 \text{ K-ft/LF } (12\text{in}/1\text{ft})}{24 \text{ ksi } (27.87'')} = 1.10 \text{ in}^2/\text{LF}$

ok to use #9 @ 12" since keeping the bearing pad shear into account is very conservative.

@ 2.5 ft from top of footing $H = 16.41 - 2.5' = 13.91'$

$d_c = [(13.91/12) + 1.25] \times 12\text{in}/1\text{ft} = 28.91$

$F_1 = 0.5(0.120)(0.3)(13.91)^2 = 3.48$ K/LF

$F_2 = 2(0.12)(0.3)(13.91) = 1.0$ K/LF

$\circlearrowleft M = 3.48(13.91/3) + 1.0(13.91/2) + (20.8/6.25)(7.58 - 2.5)$
$= 16.14 + 6.96 + 16.92 = 40.02$ K-ft/LF

Try #9 @ 12"
$jd = 23.51''$ from excel file

$A_s = \frac{M}{f_s jd} = \frac{40.02 \times 12\text{in}/1\text{ft}}{(24)(23.51'')} = 0.85 \text{ in}^2/\text{LF}$

use #9 @ 12"

@ 5ft from top of footing $H = 16.41' - 5' = 11.41'$

$F_1 = \frac{1}{2}\gamma K_a H^2 = \frac{1}{2}(0.120)(0.3)(11.41)^2 = 2.34$ K/LF

$F_2 = 2'\gamma K_a H = 2'(0.120)(0.3)(11.41) = 0.82$ K/LF

$\circlearrowleft M = 2.34(11.41/3) + 0.82(11.41/2) + (20.8/6.25)(7.85-5)$

$= 8.90 + 4.68 + 9.49 = 23.07$ K-ft/LF

Try #9 @ 12" $d_t = 26.41$ $jd = 21.11$ from excel calc

$A_s = \frac{M}{f_s jd} = \frac{23.07 \times 12in/ft}{(24)(21.11)} = 0.55\ in^2/LF$

use #7 @ 12"

@ 10 ft from top of footing $H = 16.41' - 10' = 6.41'$

$F_1 = \frac{1}{2}\gamma K_a H^2 = \frac{1}{2}(0.120)(0.3)(6.41)^2 = 0.74$ K/LF

$F_2 = 2'\gamma K_a H = 2(0.120)(0.3)(6.41) = 0.46$ K/LF

$\circlearrowleft M = 0.74(6.41/3) + 0.46(6.41/2) = 1.58 + 1.47 = 3.05$ K-ft/LF

Try #7 @ 12" $d_t = 21.41''$ $jd = 16.73''$ from excel calc.

$A_s = \frac{M}{f_s jd} = \frac{3.05 k\text{-}ft/LF \times 12in/ft}{(24 ksi)(16.73'')} = 0.09\ in^2/LF$

@ 6ft from top of footing $H = 16.41' - 6' = 10.41'$

$F_1 = \frac{1}{2}\gamma K_a H^2 = \frac{1}{2}(0.120)(0.3)(10.41)^2 = 1.95$ K/LF

$F_2 = 2\gamma K_a H = 2(0.120)(0.3)(10.41) = 0.75$ K/LF

$\circlearrowleft M = 1.95(10.41/3) + 0.75(10.41/3) + (7.58' - 6')(20.8k/6.25')$

$= 6.77$ K-ft/LF $+ 2.60$ K-ft/LF $+ 5.26$ K-ft/LF $= 14.63$ K-ft/LF

Try #7 @ 12" $d_t = 25.41''$ $jd = 20.59in$

$A_s = \frac{M}{f_s jd} = \frac{14.63 K\text{-}ft/LF \times 12in/ft}{(24)(20.59)} = 0.36\ in^2/LF$

use #6 @ 12"

@ 7 ft. from top of footing $H = 16.41' - 7' = 9.41'$

$F_1 = \frac{1}{2}\gamma K_a H^2 = \frac{1}{2}(0.120)(0.3)(9.41)^2 = 1.59$

$F_2 = 2\gamma K_a H = 2(0.120)(0.3)(9.41) = 0.68$

$\circlearrowleft M = 1.59(9.41/3) + 0.68(9.41/2) + (7.58' - 7')(20.8/6.25')$

$= 4.99 + 3.20 + 0.31 = 8.50$ K-ft/LF

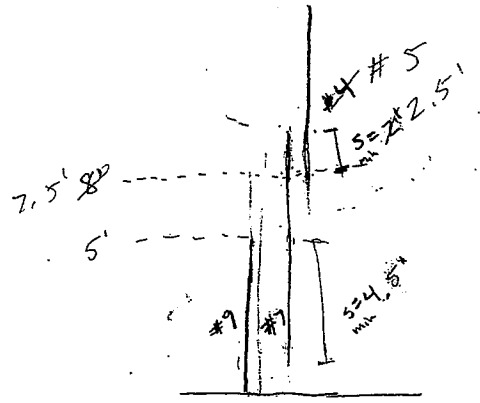
Try #6 @ 12" $d_t = 24.41$ $jd = 19.85$ from excel calc.

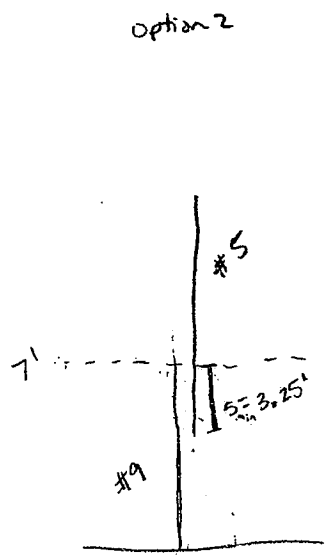
$A_s = \frac{M}{f_s jd} = \frac{8.50 K\text{-}ft/LF \times 12in/ft}{(24)(19.85)} = 0.21\ in^2/LF$ use #5 @ 12"

10ft As = 0.09 in²/LF

7ft As = 0.21 in²/LF
6ft As = 0.38 in²/LF
5 ft As = 0.55 in²/LF #7
2.5 ft As = 0.85 in²/LF
0ft As = 1.10 in²/LF #9

#9 ld = 3.17'
#7 ld = 2.08'
#5 ld = 1.50'




#9 - L = 5'
#7 - L = 9.5' start @ ~~0.5'~~ 0'
~~#4~~ #5 - L = 8.41' start @ ~~8'~~ 7.5'

#9 : L = 9' start bottom
#5 : L = 12.65' start @ 3.75'

#9 & #7 splice = 60 * 0.875 = 52.50" or 4.38'

#7 & #4 splice = 0.5 * 45 = 22.5" or 1.88'

#9 & #5 splice = 60 * 0.625 = 37.5" or 3.13'

#7 & #5 splice = 45 * 0.625" = 28.13" or 2.34'

```
*************************************************************************
*                              WinAbut                                  *
*                     ABUTment Design Program                           *
*                        (SS) SHORT SEAT                                *
*                      (HC) HIGH CANTILEVER                             *
*              California Department of Transportation                  *
*                   -- For CALTRANS Use Only --                         *
*              Graphical User Interface / PC Version                    *
*                          Version 1.1.5                                *
*                            12.05.03                                   *
*                                                                       *
*************************************************************************
```

```
****************
* GENERAL DATA *
*************************************************************************
```

| | | | |
|---|---|---|---|
| TYPE ABUTMENT | (HIGH CANTILEVER) | = | HC |
| TYPE SUPERSTRUCTURE | (CIP/PS?) | = | NO |
| TYPE FOOTING | | = | PILE |
| TYPE ANALYSIS | | = | DESIGN |
| TYPE DESIGN | (SERVICE LOAD DESIGN) | = | SLD |

MATERIAL PROPERTIES:
---

| | | | | |
|---|---|---|---|---|
| ULTIMATE CONCRETE COMPRESSIVE STRESS | (FC) | = | 3.60 | Ksi |
| YIELD STRESS FOR STEEL REBARS | (FY) | = | 60.00 | Ksi |
| ALLOWABLE CONCRETE STRESS (% FC) | | = | 40.00 | % |
| | | = | 1.44 | Ksi |
| ALLOWABLE STEEL STRESS (% FY) | | = | 40.00 | % |
| | | = | 24.00 | Ksi |
| YOUNGS MODULUS FOR CONCRETE | (EC) | = | 3420.0 | Ksi |
| YOUNGS MODULUS FOR STEEL REBARS | (ES) | = | 29000.0 | Ksi |
| MODULAR RATIO (ES/EC) | (N) | = | 8.48 | |
| CONCRETE UNIT WEIGHT | | = | 150.00 | Pcf |

BACKFILL SOIL DATA:
---

| | | | | |
|---|---|---|---|---|
| DEPTH OF FILL ON TOE | | = | 0.00 | Ft |
| DEPTH OF FILL ON HEEL | | = | 16.41 | Ft |
| DEPTH OF SURCHAGE ON HEEL | | = | 2.00 | Ft |
| ENVIRONMENTAL FACTOR | (Z) | = | 170.00 | K/in |
| COEF OF ACTIVE EARTH PRESSURE | (KA) | = | 0.30 | |
| SOIL UNIT WEIGHT | | = | 120.00 | Pcf |

```
*******************
* STRUCTURAL DATA *
*******************************************************************************

ABUTMENT DATA:
--------------
LENGTH                                        =     5.00 Ft
HEIGHT FROM TOP OF FOOTING                    =    16.41 Ft
BACKWALL THICKNESS                            =     1.95 Ft
SEAT WIDTH                                    =     0.75 Ft
DISTANCE FROM FOOTING TOE TO SEAT FACE        =     5.00 Ft
DISTANCE FROM SEAT FACE TO C.L. OF BEARING =     0.38 Ft
BACKWALL HEIGHT ABOVE SEAT LEVEL              =     2.00 Ft
WEIGHT OF EXTERIOR SHEAR KEYS                 =     0.00 K
UNIFORM WEIGHT OF BARRIER                     =     0.00 K/Ft
VERTICAL DISTANCE FROM SEAT TO FILLET         =     0.00 Ft
STEM THICKNESS                                =     2.70 Ft

WING WALL DATA:
---------------
NUMBER                                        =        0 WALLS
LENGTH                                        =     0.00 Ft
THICKNESS                                     =     0.00 Ft
END WALL DIMENSION                            =     0.00 Ft

FOOTING DATA:
-------------
LENGTH                                        =     6.25 Ft
WIDTH                                         =    12.00 Ft
DEPTH                                         =     2.50 Ft
```

```
*************
* PILE DATA *
*************************************************************************

VERTICAL PILE CAPACITY - SERVICE          =    70.00 Tons
LATERAL  PILE CAPACITY - SERVICE          =     9.00 K
LATERAL  PILE CAPACITY - ULTIMATE         =    40.00 K
PILE DIAMETER                             =    12.00 in
BATTER FACTOR                             =   1:3.00

PILE LAYOUT DATA (Sorted) (Ft):
-------------------------------
ROW    DISTANCE   NO. OF PILES   NO. OF PILES
NO.    TO TOE     IN ROW         BATTERED
---    --------   ------------   ------------
  1       1.50          2             2
  2       5.83          2             0
  3      10.50          2             0

TOTAL NUMBER OF PILES                     =    6
DISTANCE FROM TOE TO C.G. OF PILES (CG) =    5.94 Ft
MOMENT OF INERTIA OF PILE GROUP    (IX) =   81.04 Pile-Ft^2


***********************
* REBAR AND LOAD DATA *
*************************************************************************

REBAR LOCATION          BAR SIZE   NO. OF BARS   COVER(in)
--------------          --------   -----------   ---------
BACKWALL                    7          13          2.00
STEM                        9           8          2.00
FOOTING TOP                 5           5          3.00
FOOTING BOTTOM (HEEL)      11           5          6.00
FOOTING BOTTOM (TOE )      11          15          6.00


SERVICE LOAD DATA (K ):
-----------------------

         TOTAL :- LIVE LOAD -:  TOTAL      LATERAL   OTHER     OTHER
         DEAD  TOTAL   1.15P+H  BEARING    EARTH     LATERAL   VERTICAL
         LOAD  H-LOAD  P-LOAD   PAD SHEAR  PRESSURE  LOAD      LOAD
         ----- ------  -------  ---------  --------  -------   --------
LOADS      5.     0.      0.        0.                 21.       252.

FACTORS   1.30   1.30    1.30      1.30      1.69      1.30      1.30

DISTANCE ABOVE SEAT TO OTHER LATERAL  LOAD (Ft) =      -6.83
DISTANCE FROM  HEEL TO OTHER VERTICAL LOAD (Ft) =                 8.87

CONSIDER OTHER LOADS IN LOAD GROUPS (Y/N)              12345     12345
                                                       YYYYN     YNNYN
```

```
*****************************************
* MOMENTS ABOUT TOE OF FOOTING (K,Ft,K/Ft) * (SERVICE)
*****************************************************************************
```

| LATERAL EARTH LOADS (BACKFILL TO TOP OF BACKWALL) | FORCE | ARM | MOMENT |
|---|---|---|---|
| EARTH PRESSURE (KA =0.300) | 32.2 | 6.30 | 203. |
| EARTH PRESSURE (KA =0.225) | 24.1 | 6.30 | 152. |
| SURCHARGE PRESSURE (KA =0.300) | 6.8 | 9.46 | 64. |
| SURCHARGE PRESSURE (KA =0.225) | 5.1 | 9.46 | 48. |
| EARTHQUAKE | 0.0 | 0.00 | 0. |
| **SUPERSTRUCTURE LOADS** | | | |
| DEAD LOAD | 4.5 | 5.38 | -24. |
| H-LIVE LOAD | 0.0 | 5.38 | 0. |
| P-LIVE LOAD | 0.0 | 5.38 | 0. |
| BEARING PAD SHEAR | 0.0 | 16.91 | 0. |
| **OTHER LOADS** | | | |
| OTHER LATERAL LOAD | 20.8 | 10.08 | 210. |
| OTHER VERTICAL LOAD | 251.7 | 3.13 | -788. |
| **ABUTMENT DEAD LOADS** | | | |
| BACKWALL | 2.9 | 6.72 | -20. |
| SEAT | 29.2 | 6.35 | -185. |
| EXTRA SEAT PORTION (HIGH CANTILEVER) | 0.0 | 7.70 | 0. |
| FOOTING | 28.1 | 6.00 | -169. |
| FOOTING KEY | 0.0 | 0.00 | 0. |
| WINGWALL TOP RECTANGULAR SECTION | 0.0 | 6.54 | 0. |
| WINGWALL BOT TRIANGULAR SECTION | 0.0 | 6.93 | 0. |
| WINGWALL EXTRA SECTION (HIGH CANTILEVER) | 0.0 | 7.70 | 0. |
| BARRIER | 0.0 | 5.57 | 0. |
| SIDE KEYS | 0.0 | 5.38 | 0. |
| | 60.2 | | |
| **SOIL DEAD LOADS** | | | |
| SOIL ON HEEL (BACKFILL TO TOP OF BACKWALL) | 42.3 | 9.85 | -417. |
| EXTRA SOIL ON HEEL (HIGH CANTILEVER) | 0.0 | 7.70 | 0. |
| SOIL ON TOE | 0.0 | 2.50 | 0. |
| SURCHARGE ON HEEL | 5.2 | 9.85 | -51. |

Permanent.

Total
= Permanent + LL + Liveload
= 240 + 119 + 5.2
= 364 kips for 5'
73 k/LF

Abut WT. 45 + 60.2 = 647
Soil WT. 42.3
DL from superstructure 104
Pedestal 29
240 kips for 5' Abutment
48 k/LF

242

```
*****************************************************
* SERVICE PILE LOADS (SER) FOR ABUTMENT DESIGN (K,Ft) *
*****************************************************
```

Note: Toe & Heel Piles Loads and Capacities are along the Vertical.

| GRP | CA | :-- VERTICAL --: TOTAL | ECCEN | :-- LATERAL --: TOTAL | CAP | :----- HEEL ----: MAX | CAP | :----- TOE -----: MAX | CAP | |
|---|---|---|---|---|---|---|---|---|---|---|
| IH | 1 | 364. | 2.71 | 60. | 130. | 5. | 140. | 115. | 133. | OK |
| IH | 2 | 364. | 2.53 | 50. | 128. | 9. | 140. | 111. | 133. | OK |
| IH | 3 | 359. | 2.63 | 53. | 128. | 7. | 140. | 111. | 133. | OK |
| IH | 4 | 359. | 2.48 | 45. | 126. | 10. | 140. | 109. | 133. | OK |
| II | 1 | 112. | 2.48 | 60. | 77. | 3. | 140. | 34. | 133. | OK |
| II | 2 | 112. | 1.88 | 50. | 74. | 7. | 140. | 30. | 133. | OK |
| III | 1 | 108. | 2.56 | 60. | 103. | 2. | 210. | 33. | 199. | OK |
| III | 2 | 108. | 1.94 | 50. | 101. | 6. | 210. | 29. | 199. | OK |
| IV | 1* | 364. | 2.71 | 60. | 144. | 5. | 175. | 115. | 166. | OK |
| IV | 2 | 364. | 2.53 | 50. | 142. | 9. | 175. | 111. | 166. | OK |
| IV | 3 | 359. | 2.63 | 53. | 142. | 7. | 175. | 111. | 166. | OK |
| IV | 4 | 359. | 2.48 | 45. | 140. | 10. | 175. | 109. | 166. | OK |
| V | 1 | 316. | 2.86 | 0. | 149. | 2. | 210. | 102. | 199. | OK |

SUMMARY OF CONTROLLING LOAD CASES
(RATIO = CAP / LOAD)

| GRP | CA | DESCRIPTION | LOAD | CAP | RATIO | |
|---|---|---|---|---|---|---|
| II | 1 | TOTAL LATERAL LOAD | 60. | 77. | 1.28 | OK |
| IH | 4 | MAX HEEL LOAD | 10. | 140. | 14.47 | OK |
| IH | 1 | MAX TOE LOAD | 115. | 133. | 1.16 | OK |

Controlling Design Loading

| Grp | Ca | Description | Load |
|---|---|---|---|
| IH | 1 | Max Toe Load | 121. |

Note: Allowable Overstress = 1.00

Loading is Along the Batter.

```
*********************************************
* STEEL REQUIRED - SERVICE LOAD DESIGN (SLD) *
*********************************************************************

KEY TO REBAR LOCATIONS (BAR-LOC)
--------------------------------
1 = BACKWALL
2 = STEM
3 = FOOTING TOP
4 = FOOTING BOTTOM (HEEL)
5 = FOOTING BOTTOM (TOE)
```

| BAR LOC | LOADS GRP | CA | APPLIED MOMENT (K-Ft) | :- STRESS-: FC (Ksi) | FS (Ksi) | EFF DEPTH (in) | STEEL AREA (in^2) | BAR SIZE | BAR SPACE (in) | BAR REQ | ERR CODE |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | | | 159. | 0.74 | 23.3 | 20.9 | 4.2 | 7 | 9.2 | 7 | |
| 2 | | | 339. | 0.73 | 21.1 | 29.8 | 7.0 | 9 | 9.1 | 7 | |
| 3 | II | 1 | -107. | 0.40 | 23.1 | 26.7 | 2.2 | 5 | 11.7 | 7 | |
| 4 | MIN | | 0. | 0.00 | 0.0 | 23.2 | 7.8 | 11 | 17.3 | 5 | |
| 5 | IH | 1 | 774. | 1.47 | 22.4 | 23.2 | 20.3 | 11 | 5.8 | 13 | |

** ERROR CONCRETE STRESS (1.47) > ALLOWABLE CONCRETE STRESS (1.44) ! - Difference of 2% ∴ ok

ERROR CODES:

1 = BAR SPACING LESS THAN AASHTO MINIMUM. (CODE 8.21)
2 = BAR SPACING LESS THAN PREFERRED CALTRANS MINIMUM. (BR. DES. DET.)
3 = BAR SPACING MORE THAN THE AASHTO MAXIMUM ( 18. in). (CODE 8.20)

```
*****************************************************************
* STEM MOMENT ENVELOPE AND REQUIRED STEEL AREAS (BAR CUT-OFFS) *
*****************************************************************
(APPLIED MOMENT - GROUP IV CASE 1)
(BACKFILL, SURCHARGE, BEARING PAD SHEAR AND OTHER LATERAL LOAD)
```

| DIST FROM FOOTING TOP (Ft) | APPLIED MOMENT (K-Ft) | :- STRESS --: FC (Ksi) | FS (Ksi) | STEEL AREA (in^2) | BAR SIZE | BAR SPACE (in) | BARS REQ | ERROR CODE |
|---|---|---|---|---|---|---|---|---|
| 0.00 | 339. | 0.73 | 21.1 | 7.0 | 9 | 9.1 | 7 | |
| 1.00 | 289. | 0.66 | 20.9 | 6.0 | 9 | 10.9 | 6 | |
| 2.00 | 243. | 0.60 | 21.0 | 5.0 | 9 | 13.7 | 5 | |
| 3.00 | 200. | 0.54 | 21.4 | 4.0 | 9 | 18.3 | 4 | 3 |

```
*************************************************************************
*                              WinAbut                                  *
*                      ABUTment Design Program                          *
*                         (SS) SHORT SEAT                               *
*                       (HC) HIGH CANTILEVER                            *
*              California Department of Transportation                  *
*                   -- For CALTRANS Use Only --                         *
*              Graphical User Interface / PC Version                    *
*                           Version 1.1.5                               *
*                             12.05.03                                  *
*                                                                       *
*************************************************************************


****************
* GENERAL DATA *
*************************************************************************


TYPE ABUTMENT              (HIGH CANTILEVER) =      HC
TYPE SUPERSTRUCTURE                (CIP/PS?) =      NO
TYPE FOOTING                                 =    PILE
TYPE ANALYSIS                                =   CHECK
TYPE DESIGN            (LOAD FACTOR DESIGN) =     LFD


MATERIAL PROPERTIES:
--------------------
ULTIMATE CONCRETE COMPRESSIVE STRESS  (FC) =     3.60 Ksi
YIELD STRESS FOR STEEL REBARS         (FY) =    60.00 Ksi
ALLOWABLE CONCRETE STRESS (% FC)           =    40.00 %
                                           =     1.44 Ksi
ALLOWABLE STEEL STRESS    (% FY)           =    40.00 %
                                           =    24.00 Ksi
YOUNGS MODULUS FOR CONCRETE           (EC) =   3420.0 Ksi
YOUNGS MODULUS FOR STEEL REBARS       (ES) =  29000.0 Ksi
MODULAR RATIO (ES/EC)                  (N) =     8.48
CONCRETE UNIT WEIGHT                       =   150.00 Pcf


BACKFILL SOIL DATA:
-------------------
DEPTH OF FILL ON TOE                       =     0.00 Ft
DEPTH OF FILL ON HEEL                      =    16.41 Ft
DEPTH OF SURCHAGE ON HEEL                  =     2.00 Ft
ENVIRONMENTAL FACTOR                   (Z) =   170.00 K/in
COEF OF ACTIVE EARTH PRESSURE         (KA) =     0.30
SOIL UNIT WEIGHT                           =   120.00 Pcf
```

```
*******************
* STRUCTURAL DATA *
*******************************************************************************

ABUTMENT DATA:
--------------
LENGTH                                        =     6.25 Ft
HEIGHT FROM TOP OF FOOTING                    =    16.41 Ft
BACKWALL THICKNESS                            =     1.95 Ft
SEAT WIDTH                                    =     0.75 Ft
DISTANCE FROM FOOTING TOE TO SEAT FACE        =     5.00 Ft
DISTANCE FROM SEAT FACE TO C.L. OF BEARING    =     0.38 Ft
BACKWALL HEIGHT ABOVE SEAT LEVEL              =     2.00 Ft
WEIGHT OF EXTERIOR SHEAR KEYS                 =     0.00 K
UNIFORM WEIGHT OF BARRIER                     =     0.00 K/Ft
VERTICAL DISTANCE FROM SEAT TO FILLET         =     0.00 Ft
STEM THICKNESS                                =     2.70 Ft

WING WALL DATA:
---------------
NUMBER                                        =        0 WALLS
LENGTH                                        =     0.00 Ft
THICKNESS                                     =     0.00 Ft
END WALL DIMENSION                            =     0.00 Ft

FOOTING DATA:
-------------
LENGTH                                        =     6.25 Ft
WIDTH                                         =    12.00 Ft
DEPTH                                         =     2.50 Ft
```

```
*************
* PILE DATA *
*************************************************************************

VERTICAL PILE CAPACITY - SERVICE            =    70.00 Tons
LATERAL  PILE CAPACITY - SERVICE            =     9.00 K
LATERAL  PILE CAPACITY - ULTIMATE           =    40.00 K
PILE DIAMETER                               =    12.00 in
BATTER FACTOR                               =   1:3.00

PILE LAYOUT DATA (Sorted) (Ft):
-------------------------------
ROW     DISTANCE    NO. OF PILES   NO. OF PILES
NO.     TO TOE      IN ROW         BATTERED
---     --------    ------------   ------------
  1        1.50          2             2
  2        5.83          2             0
  3       10.50          2             0

TOTAL NUMBER OF PILES                       =    6
DISTANCE FROM TOE TO C.G. OF PILES (CG) =    5.94 Ft
MOMENT OF INERTIA OF PILE GROUP    (IX) =   81.04 Pile-Ft^2


***********************
* REBAR AND LOAD DATA *
*************************************************************************

REBAR LOCATION          BAR SIZE   NO. OF BARS   COVER(in)
--------------          --------   -----------   ---------
BACKWALL                    7           7          2.00
STEM                        9           7          2.00
FOOTING TOP                 5           7          3.00
FOOTING BOTTOM (HEEL)      11           9          6.00
FOOTING BOTTOM (TOE )      11          14          6.00


SERVICE LOAD DATA (K ):
-----------------------

          TOTAL :- LIVE LOAD -:  TOTAL      LATERAL   OTHER     OTHER
          DEAD  TOTAL   1.15P+H  BEARING    EARTH     LATERAL   VERTICAL
          LOAD  H-LOAD  P-LOAD   PAD SHEAR  PRESSURE  LOAD      LOAD
          ----- ------  -------  ---------  --------  -------   --------
LOADS       5.     0.       0.         0.                18.       441.

FACTORS   1.00   1.30     1.30       1.30      1.69     1.00      1.00

DISTANCE ABOVE SEAT TO OTHER LATERAL  LOAD (Ft) =      -6.83
DISTANCE FROM  HEEL TO OTHER VERTICAL LOAD (Ft) =                 8.83

CONSIDER OTHER LOADS IN LOAD GROUPS (Y/N)              12345     12345
                                                       YYYYN     YNNYN
```

```
*****************************************
* MOMENTS ABOUT TOE OF FOOTING (K,Ft,K/Ft) * (SERVICE)
*****************************************************************************
```

| LATERAL EARTH LOADS (BACKFILL TO TOP OF BACKWALL) | FORCE | ARM | MOMENT |
|---|---|---|---|
| EARTH PRESSURE (KA =0.300) | 40.2 | 6.30 | 254. |
| EARTH PRESSURE (KA =0.225) | 30.2 | 6.30 | 190. |
| SURCHARGE PRESSURE (KA =0.300) | 8.5 | 9.46 | 80. |
| SURCHARGE PRESSURE (KA =0.225) | 6.4 | 9.46 | 60. |
| EARTHQUAKE | 0.0 | 0.00 | 0. |
| **SUPERSTRUCTURE LOADS** | | | |
| DEAD LOAD | 4.5 | 5.38 | -24. |
| H-LIVE LOAD | 0.0 | 5.38 | 0. |
| P-LIVE LOAD | 0.0 | 5.38 | 0. |
| BEARING PAD SHEAR | 0.0 | 16.91 | 0. |
| **OTHER LOADS** | | | |
| OTHER LATERAL LOAD | 18.3 | 10.08 | 184. |
| OTHER VERTICAL LOAD | 440.5 | 3.17 | -1396. |
| **ABUTMENT DEAD LOADS** | | | |
| BACKWALL | 3.7 | 6.72 | -25. |
| SEAT | 36.5 | 6.35 | -232. |
| EXTRA SEAT PORTION (HIGH CANTILEVER) | 0.0 | 7.70 | 0. |
| FOOTING | 28.1 | 6.00 | -169. |
| FOOTING KEY | 0.0 | 0.00 | 0. |
| WINGWALL TOP RECTANGULAR SECTION | 0.0 | 6.54 | 0. |
| WINGWALL BOT TRIANGULAR SECTION | 0.0 | 6.93 | 0. |
| WINGWALL EXTRA SECTION (HIGH CANTILEVER) | 0.0 | 7.70 | 0. |
| BARRIER | 0.0 | 5.57 | 0. |
| SIDE KEYS | 0.0 | 5.38 | 0. |
| **SOIL DEAD LOADS** | | | |
| SOIL ON HEEL (BACKFILL TO TOP OF BACKWALL) | 52.9 | 9.85 | -521. |
| EXTRA SOIL ON HEEL (HIGH CANTILEVER) | 0.0 | 7.70 | 0. |
| SOIL ON TOE | 0.0 | 2.50 | 0. |
| SURCHARGE ON HEEL | 6.4 | 9.85 | -64. |

**********
* SERVICE PILE LOADS (SER) FOR ABUTMENT DESIGN (K,Ft) * No Need to check
**********
Note: Toe & Heel Piles Loads and Capacities are along the Vertical. Loads are Factor Load.

| GRP | CA | :-- VERTICAL --: TOTAL | ECCEN | -- LATERAL --: TOTAL | CAP | ----- HEEL ----: MAX | CAP | ----- TOE -----: MAX | CAP | |
|---|---|---|---|---|---|---|---|---|---|---|
| IH | 1 | 573. | 2.60 | 67. | 172. | 12. | 140. | 177.* | 133. | NG* |
| IH | 2 | 573. | 2.46 | 55. | 169. | 16. | 140. | 173.* | 133. | NG* |
| IH | 3 | 566. | 2.54 | 59. | 169. | 14. | 140. | 173.* | 133. | NG* |
| IH | 4 | 566. | 2.42 | 48. | 167. | 17. | 140. | 170.* | 133. | NG* |
| II | 1 | 132. | 2.04 | 67. | 79. | 7. | 140. | 37. | 133. | OK |
| II | 2 | 132. | 1.41 | 55. | 75. | 12. | 140. | 32. | 133. | OK |
| III | 1 | 128. | 2.09 | 67. | 105. | 6. | 210. | 36. | 199. | OK |
| III | 2 | 128. | 1.44 | 55. | 102. | 11. | 210. | 31. | 199. | OK |
| IV | 1 | 573. | 2.60 | 67. | 186. | 12. | 175. | 177.* | 166. | NG* |
| IV | 2 | 573. | 2.46 | 55. | 183. | 16. | 175. | 173.* | 166. | NG* |
| IV | 3 | 566. | 2.54 | 59. | 183. | 14. | 175. | 173.* | 166. | NG* |
| IV | 4 | 566. | 2.42 | 48. | 181. | 17. | 175. | 170.* | 166. | NG* |
| V | 1 | 513. | 2.71 | 0. | 189. | 7. | 210. | 162. | 199. | OK |

SUMMARY OF CONTROLLING LOAD CASES
(RATIO = CAP / LOAD)

| GRP | CA | DESCRIPTION | LOAD | CAP | RATIO | |
|---|---|---|---|---|---|---|
| II | 1 | TOTAL LATERAL LOAD | 67. | 79. | 1.17 | OK |
| IH | 4 | MAX HEEL LOAD | 17. | 140. | 8.15 | OK |
| IH | 1 | MAX TOE LOAD | 177. | 133. | 0.75 | NG* |

Controlling Design Loading

| Grp | Ca | Description | Load |
|---|---|---|---|
| IH | 1 | Max Toe Load | 187. |

Note: Allowable Overstress = 1.00

Loading is Along the Batter.

250

```
*****************************************************
* FACTORED PILE LOADS (FAC) FOR ABUTMENT DESIGN (K,Ft) *
*****************************************************************************
PHI = .75
Note: Toe & Heel Piles Loads and Capacities are along the Vertical.
```

| GRP | CA | :-- VERTICAL --: TOTAL | ECCEN | :-- LATERAL --: TOTAL | CAP | :----- HEEL ----: MAX | CAP | :----- TOE -----: MAX | CAP | |
|---|---|---|---|---|---|---|---|---|---|---|
| IH | 1 | 573. | 3.01 | 101. | 275. | -1.* | 0. | 190. | 199. | NG* |
| IH | 2 | 573. | 2.76 | 80. | 271. | 7. | 210. | 182. | 199. | OK |
| IH | 3 | 566. | 2.85 | 86. | 271. | 4. | 210. | 183. | 199. | OK |
| IH | 4 | 566. | 2.66 | 69. | 268. | 10. | 210. | 177. | 199. | OK |
| IP | 1 | 132. | 2.39 | 101. | 200. | 4. | 210. | 39. | 199. | OK |
| IP | 2 | 132. | 1.32 | 80. | 196. | 12. | 210. | 32. | 199. | OK |
| IP | 3 | 126. | 1.63 | 86. | 196. | 9. | 210. | 32. | 199. | OK |
| IP | 4 | 126. | 0.78 | 69. | 193. | 15. | 210. | 26. | 199. | OK |
| II | 1 | 132. | 3.79 | 101. | 205. | -6.* | 0. | 49. | 199. | NG* |
| II | 2 | 132. | 2.72 | 80. | 201. | 2. | 210. | 42. | 199. | OK |
| III | 1 | 128. | 3.90 | 101. | 204. | -7. | -84. | 49. | 199. | OK |
| III | 2 | 128. | 2.79 | 80. | 200. | 1. | 210. | 41. | 199. | OK |
| IV | 1 | 573. | 3.01 | 101. | 275. | -1.* | 0. | 190. | 199. | NG* |
| IV | 2 | 573. | 2.76 | 80. | 271. | 7. | 210. | 182. | 199. | OK |
| IV | 3 | 566. | 2.85 | 86. | 271. | 4. | 210. | 183. | 199. | OK |
| IV | 4 | 566. | 2.66 | 69. | 268. | 10. | 210. | 177. | 199. | OK |
| V | 1 | 513. | 2.71 | 0. | 261. | 7. | 210. | 162. | 199. | OK |

SUMMARY OF CONTROLLING LOAD CASES
(RATIO = CAP / LOAD)

| GRP | CA | DESCRIPTION | LOAD | CAP | RATIO | |
|---|---|---|---|---|---|---|
| IP | 1 | TOTAL LATERAL LOAD | 101. | 200. | 1.98 | OK |
| II | 1 | MAX HEEL LOAD | -6. | 0. | 0.00 | NG* ← OK. |
| IV | 1 | MAX TOE LOAD | 190. | 199. | 1.05 | OK |

Actual Footing size is larger than input.
SLD shows No uplift also.

```
*****************************************
* STEEL REQUIRED - LOAD FACTOR DESIGN (LFD) *
*****************************************************************************
```

BACKWALL, D = 20.9 (in)

| LOAD TYPE | LOADS GRP | CA | MOMENT (K-Ft) | NEUT AXIS (in) | STEEL STRESS (Ksi) | STEEL AREA (in^2) | BAR SIZE | BAR SPACE (in) | BARS REQ | ERR CODE | BAR IN |
|---|---|---|---|---|---|---|---|---|---|---|---|
| SER | | | 140. | 3.38 | 29.4 | 2.9 | 7 | 11.7 | 5 | | |
| FAC | | | 141. | 0.46 | | 1.5 | 7 | 11.7 | 3 | | |
| MIN1 | | | 308. | 1.03 | | 3.3 | 7 | 11.7 | 6 | | |
| MIN2 | | | 187. | 0.62 | | 2.0 | 7 | 11.7 | 4 | | |
| MIN3 | | | 277. | 0.92 | | 3.0 | 7 | 11.7 | 5 | | |
| MAX | | | 2302. | 9.28 | | 30.2 | 7 | 11.7 | 51 | | |
| ---- CONTROL ---- | | | | | | | | | | | |
| MIN3 | | | 277. | 0.92 | | 3.0 | 7 | 17.5 | 5 | | 7 OK |

STEM, D = 29.8 (in)

| LOAD TYPE | LOADS GRP | CA | MOMENT (K-Ft) | NEUT AXIS (in) | STEEL STRESS (Ksi) | STEEL AREA (in^2) | BAR SIZE | BAR SPACE (in) | BARS REQ | ERR CODE | BAR IN |
|---|---|---|---|---|---|---|---|---|---|---|---|
| SER | | | 365. | 5.42 | 29.3 | 5.3 | 9 | 11.6 | 6 | | |
| FAC | | | 521. | 1.22 | | 4.0 | 9 | 11.6 | 4 | | |
| MIN1 | | | 590. | 1.38 | | 4.5 | 9 | 11.6 | 5 | | |
| MIN2 | | | 691. | 1.62 | | 5.3 | 9 | 11.6 | 6 | | |
| MIN3 | | | 655. | 1.54 | | 5.0 | 9 | 11.6 | 5 | | |
| MAX | | | 4671. | 13.22 | | 43.0 | 9 | 11.6 | 43 | | |
| ---- CONTROL ---- | | | | | | | | | | | |
| SER | | | 365. | 5.42 | 29.3 | 5.3 | 9 | 13.9 | 6 | | 7 OK |

```
*******************************************
* STEEL REQUIRED - LOAD FACTOR DESIGN (LFD) *
*******************************************************************************
```

FOOT TOP, D = 26.7 (in)

| LOAD TYPE | LOADS GRP | CA | MOMENT (K-Ft) | NEUT AXIS (in) | STEEL STRESS (Ksi) | STEEL AREA (in^2) | BAR SIZE | BAR SPACE (in) | BARS REQ | ERR CODE | BAR IN |
|---|---|---|---|---|---|---|---|---|---|---|---|
| SER | III | 1 | -114. | 3.00 | 31.9 | 1.7 | 5 | 11.7 | 6 | | |
| FAC | III | 1 | -187. | 0.48 | | 1.6 | 5 | 11.7 | 6 | | |
| MIN1 | | | -506. | 1.33 | | 4.3 | 5 | 11.7 | 14 | | |
| MIN2 | | | -249. | 0.64 | | 2.1 | 5 | 11.7 | 7 | | |
| MIN3 | | | -185. | 0.48 | | 1.6 | 5 | 11.7 | 5 | | |
| MAX | | | -3744. | 11.83 | | 38.5 | 5 | 11.7 | 125 | | |
| CONTROL | | | | | | | | | | | |
| MIN2 | | | -249. | 0.64 | | 2.1 | 5 | 11.7 | 7 | | 7 OK |

FOOT BOT (HEEL), D = 23.2 (in)

| LOAD TYPE | LOADS GRP | CA | MOMENT (K-Ft) | NEUT AXIS (in) | STEEL STRESS (Ksi) | STEEL AREA (in^2) | BAR SIZE | BAR SPACE (in) | BARS REQ | ERR CODE | BAR IN |
|---|---|---|---|---|---|---|---|---|---|---|---|
| SER | V | 1 | 20. | 1.47 | 23.9 | 1.6 | 11 | 37.5 | 1 | 3 | |
| FAC | V | 1 | 20. | 0.06 | | 1.6 | 11 | 37.5 | 1 | 3 | |
| MIN1 | | | 506. | 1.54 | | 5.0 | 11 | 8.7 | 4 | | |
| MIN2 | | | 27. | 0.08 | | 1.6 | 11 | 37.5 | 1 | 3 | |
| MIN3 | | | 778. | 2.40 | | 7.8 | 11 | 8.7 | 5 | | |
| MAX | | | 2832. | 10.29 | | 33.5 | 11 | 8.7 | 22 | | |
| CONTROL | | | | | | | | | | | |
| MIN3 | | | 778. | 2.40 | | 7.8 | 11 | 17.3 | 5 | | 9 OK |

FOOT BOT (TOE), D = 23.2 (in)

| LOAD TYPE | LOADS GRP | CA | MOMENT (K-Ft) | NEUT AXIS (in) | STEEL STRESS (Ksi) | STEEL AREA (in^2) | BAR SIZE | BAR SPACE (in) | BARS REQ | ERR CODE | BAR IN |
|---|---|---|---|---|---|---|---|---|---|---|---|
| SER | IH | 1 | 1211. | 8.18 | 36.0 | 19.7 | 11 | 5.3 | 13 | | |
| FAC | IH | 1 | 1300. | 4.15 | | 13.5 | 11 | 5.3 | 9 | | |
| MIN1 | | | 506. | 1.54 | | 5.0 | 11 | 5.3 | 4 | | |
| MIN2 | | | 1685. | 5.53 | | 18.0 | 11 | 5.3 | 12 | | |
| MIN3 | | | 778. | 2.40 | | 7.8 | 11 | 5.3 | 5 | | |
| MAX | | | 2832. | 10.29 | | 33.5 | 11 | 5.3 | 22 | | |
| CONTROL | | | | | | | | | | | |
| SER | IH | 1 | 1211. | 8.18 | 36.0 | 19.7 | 11 | 5.8 | 13 | | 14 OK |

```
*****************************************************
* SUMMARY - STEEL REQUIRED - LOAD FACTOR DESIGN (LFD) *
*****************************************************************************
```

KEY TO TYPES OF LOADS:

SER = SERVICE LOADS
FAC = FACTORED LOADS PHI = 0.90
MIN1 = MINIMUM REINFORCEMENT (ART 8.17.1.1)
BASED ON 1.2 X THE CRACKING MOMENT
MIN2 = WAIVER OF MINIMUM REINFORCEMENT (ART 8.17.1.2)
BASED ON STEEL AREA ONE THIRD GREATER THAN REQUIRED BY ANALYSIS
MIN3 = MINIMUM REINFORCEMENT (TEMPERATURE) (ART 8.20)
BASED ON 18" MAXIMUM BAR SPACING
MAX = MAXIMUM REINFORCEMENT (ART 8.16.3.1)
BASED ON .75 X BALANCED STEEL RATIO

KEY TO REBAR LOACATIONS (BAR-LOC)

1 = BACKWALL
2 = STEM
3 = FOOTING TOP
4 = FOOTING BOTTOM (HEEL)
5 = FOOTING BOTTOM (TOE)

| BAR LOC | LOAD TYPE | LOADS GRP | CA | MOMENT (K-Ft) | NEUT AXIS (in) | EFF DEPTH (in) | STEEL AREA (in^2) | BAR SIZE | BAR SPC (in) | BARS REQ | ERR CODE | BARS IN | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | MIN3 | | | 277. | 0.92 | 20.9 | 3.0 | 7 | 17.5 | 5 | | 7 | OK |
| 2 | SER | | | 365. | 5.42 | 29.8 | 5.3 | 9 | 13.9 | 6 | | 7 | OK |
| 3 | MIN2 | | | -249. | 0.64 | 26.7 | 2.1 | 5 | 11.7 | 7 | | 7 | OK |
| 4 | MIN3 | | | 778. | 2.40 | 23.2 | 7.8 | 11 | 17.3 | 5 | | 9 | OK |
| 5 | SER | IH | 1 | 1211. | 8.18 | 23.2 | 19.7 | 11 | 5.8 | 13 | | 14 | OK |

ERROR CODES:

1 = BAR SPACING LESS THAN AASHTO MINIMUM. (CODE 8.21)
2 = BAR SPACING LESS THAN PREFERRED CALTRANS MINIMUM. (BR. DES. DET.)
3 = BAR SPACING MORE THAN THE AASHTO MAXIMUM ( 18. in). (CODE 8.20)

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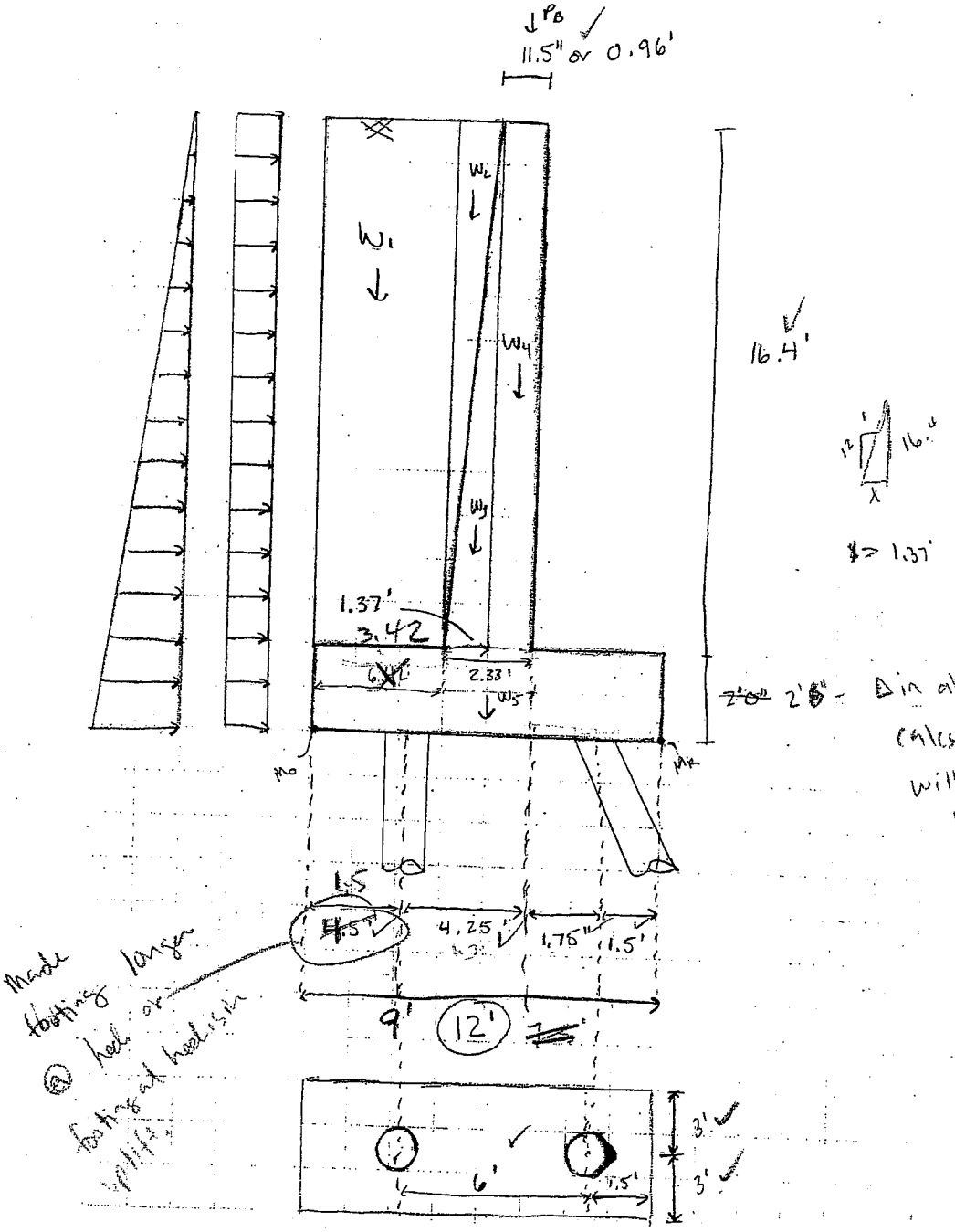
```
*****************************************************************
* STEM MOMENT ENVELOPE AND REQUIRED STEEL AREAS (BAR CUT-OFFS) *
*****************************************************************
(APPLIED MOMENT - GROUP IV CASE 1)
(BACKFILL, SURCHARGE, BEARING PAD SHEAR AND OTHER LATERAL LOAD)
```

| DIST FROM FOOTING TOP (Ft) | APPLIED MOMENT (K-Ft) | NUET AXIS (in) | STEEL AREA (in^2) | BAR SIZE | BAR SPACE (in) | BARS REQ | ERR CODE |
|---|---|---|---|---|---|---|---|
| 0.00 | 521. | 1.22 | 4.0 | 9 | 23.3 | 4 | 3 |

1-14-09 RW Check @ Abut #1 North

255

07-LA-710-17.3




Made footing longer @ heel or footing at heel is in uplift

Assumptions/Given:

$\gamma_s = 0.120$ kcf

$K_a = 0.30$

$\gamma_c = 0.150$ kcf

Concrete Barr = 0.4 K/ft

$f_s = 24{,}000$ psi per BDS 8.15.2.2

$f_y = 60{,}000$ psi

$f'_c = 3250$ psi




| Force (Kips/LF) | Arm (ft) | Moment (K-ft/LF) |
|---|---|---|
| $2 \times 0.12 \times 16.4 = 3.9$ K/ft | $9 - 3.42/2 = 7.29$ | ~~-28.1~~ |
| $P_B = 0.4$ K/ft | $3.25' + (0.96'/2) = 3.73'$ | -1.49 K-ft/LF |
| $W_1 = 3.42' \times 0.120$ Kcf $\times 16.4'$ = ~~12.03~~ 6.73 K/LF | $5.29' + (3.42'/2)$ = ~~8.79'~~ 7.29 | ~~-111.02~~ K-ft/LF -49.06 |
| $W_2 = (1.37' \times 0.120$ Kcf $\times 16.4')/2$ = 1.35 K/LF | $3.25' + 0.96' + 1.37'(2/3)$ = 5.12' | -6.91 K-ft/LF |

sundry

| Force (Kips / LF) | Arm (ft) | Moment (K-ft/LF) |
|---|---|---|
| $W_3 = (1.37' + 0.150 kcf + 16.4')/2$ = 1.69 K/LF ✓ | 3.25' + 0.96' + 1.37(1/3) = 4.67' ✓ | -7.87 K-ft/LF |
| $W_4 = 0.96' + 0.150 kcf + 16.4'$ = 2.36 K/LF ✓ | 3.25' + (0.96/2) = 3.73' ✓ | -8.80 K-ft/LF |
| $W_5 = 2' + \cancel{12}\,9' + 0.150 kcf$ = ~~3.6~~ 2.07 K/LF | ~~12'~~/2 = 6' 9' 4.5 | ~~-21.6~~ K-ft/LF -12.15 |
| $\Sigma F$ = ~~22.03~~ 19.13 K/LF | | $\Sigma M$ = -~~157.69~~ 115 K-ft/LF |

$$C.G. = \frac{\Sigma M}{\Sigma F} = \frac{\cancel{157.69}\ 115}{\cancel{22.03}\ 19.13} = \cancel{7.16'}\ 6.01' \text{ from toe}$$

$M_R$ = ~~157.69~~ 115 K-ft/LF

Calculating Mo

| Force (Kips /LF) | Arm (ft) | Moment (K-ft/LF) |
|---|---|---|
| Lateral Load = $F_1$ | | |
| $F_1 = 1/2\, K_a \gamma_s H^2$ | | |
| $= 1/2(0.12)(0.3)(16.4 + \cancel{2}\ 2.5)^2$ = ~~6.09~~ 6.43 Kips/LF ✓ | $(16.4 + \cancel{2}\ 2.5)(1/3)$ = ~~6.13~~ 6.3 ✓ | ~~40.5~~ ~~37.33~~ K-ft/LF |
| $F_2 = 2 K_a \gamma_s H$ | | |
| $= 2(0.3)(0.120)(16.4 + \cancel{2}\ 2.5)$ = ~~1.31~~ 1.36 Kips/LF | $(16.4 + \cancel{2}\ 2.5)(1/2)$ = ~~9.20~~ 9.45 ✓ | ~~12.14~~ K-ft/LF 12.85 |
| | | $\Sigma M$ = ~~49.47~~ 53.35 K-ft/LF |

Mo = 49.47 K-ft/LF

$M_R > M_o$ ∴ OK.

~~Need Factor of Safety = 1.5 per BDS 5.2.2.3~~

~~$M_R/M_o$ = 3.19 > 1.5 ∴ OK~~

No overturning Issue.

Calculating C.G. of pile group & I pile group.

$$C.G. = \frac{1.5' + 7.5}{2} = 4.5' \checkmark \text{ from toe}$$

$$I_{\text{pile group}} = 1(4.5' - 1.5')^2 + 1(4.5' - 7.5')^2$$
$$= 9 + 9 = 18 \text{ ft}^2 \checkmark$$

Converting the loading to place Moment @ C.G. of pile group

$$M = 49.47 \text{ k-ft/LF} - (22.03 \text{ k/LF})(7.16' - 4.5') = -9.13 \text{ K-ft/LF}$$
$$M = 19.19 \times (6' - 4.5') = 28.7 \text{ k-ft/ft}$$

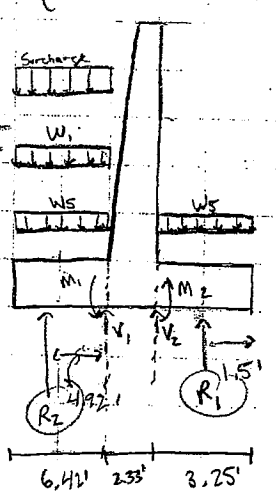
$$R = \frac{\Sigma V}{N} + \frac{Mc}{I} \qquad \text{O.T moment} = 52.35 - 28.7 = 24.65 \text{ k-ft/ft}$$

$$R_{\text{row \#1}} = \frac{19.19}{2} \times 6 + \frac{(-9.13)(4.5' - 1.5')}{18 \text{ ft}^2} \times 6 = 11.02 + 1.52 = 9.50 \text{ K/LF}$$
$$57 \quad 24.65 \quad 82 \text{ kips}$$

$$R_{\text{row \#2}} = \frac{19.19}{2} \times 6 - \frac{(-9.13)(4.5 - 7.5)}{18} \times 6 = 11.02 - 1.52 = 12.54 \text{ K/LF}$$
$$57 \quad 24.65 \quad 32.35 \text{ kips}$$

(use 11.5 kips compression bolts at Abut 1)

| W | Arm | Moment |
|---|---|---|
| 6.73 | 3.42/2 | 11.5 |
| 3.9 | 3.42/2 | 6.7 |
| 18.2 | | |




Calculating $M_1$

$$M_1 = 18.2 \times 6 - 32.35 \times (3.42 - 1.5) = 47.1 \text{ k-ft}$$

$$\circlearrowleft + \Sigma M_1 = -M_1 - (2' + 0.120 \text{ kcf} \times 6.42')(6.42'/2) \text{ or } 3.8 \text{ k-ft}$$
$$- (2' + 0.15 \times 6.42')(6.42'/2)$$
$$- (12.63)(6.42/2) + 12.54(4.92)$$
$$M_1 = -4.95 - 16.181 - 40.54 + 61.70$$
$$= 10.03 \text{ K-ft/LF}$$
$$V_1 = -1.54 - 1.93 - 12.63 + 12.54$$
$$= -3.56 \text{ K/LF}$$

Calculating $M_2$ $\quad \circlearrowleft + \Sigma M_2 = -M_2 - (2' \times 0.15 + 3.25)(3.25/2) + 9.50(3.25 - 1.5)$

$$M_2 = -1.59 + 16.63 = 15.04 \text{ K-ft/LF}$$
$$V_2 = -.98 + 9.50 = 8.52 \text{ K/LF} \times$$

$$M_2 = R1 \times 1.75' = 93.5 \times 1.75' = 163.625 \text{ k-ft @ 6 ft}$$
$$\text{or } 27.3 \text{ k-ft/ft}$$

01-202101

1-14-09 RW Check @ Abut #1 North #258

07-LA-710-17.30

Assuming #5 bars

For Top Reinforcement

$d = (2.5' * 12in/1ft) - 3'' - (0.625/2) = 26.68''$

$d' = 3'' + (1.128/2) = 3.564''$

For Bottom Reinforcement

$d = (2.5' * 12in/1ft) - 6'' - (0.625/2) = 23.38''$

$d' = 6'' + (1.128/2) = 6.564''$

$n = \frac{29 \times 10^3 ksi}{57,000\sqrt{3250}} = 8.92$

jd = 19.45 inches from excel ws jd use 23.75 at top

20.81 at bottom

For Top Reinforcement @ Heel

$f_s = \frac{M}{A_s jd} \Rightarrow A_s = \frac{M_1}{f_s jd} = \frac{28 \text{ k-ft/LF}(12in/1ft)}{(24)(23.75)} = 0.16 \text{ in}^2/LF$

Use #5 @12"

$v_{all} = 0.54 ksi$

$v = \frac{V}{b_w jd} = \frac{3.56 \times 6ft}{(12'')(19.45)} = 0.09 \text{ ksi} < 0.54$ ∴ ok

For Bottom Reinforcement @ Toe

$f_s = \frac{M}{A_s jd} \Rightarrow A_s = \frac{M_2}{f_s jd} = \frac{27.3 \text{ (k-ft/LF)}(12in/1ft)}{(24)(20.81)} = 0.65 \text{ in}^2/LF$

Use #6 @12"

$v = \frac{V}{b_w jd} = \frac{8.52 \times 6ft}{(12'')(19.45)} = 0.22 \text{ ksi} < 0.54 \text{ ksi}$ ∴ ok

#8 @ 12

Longitudinal Reinforcement

Per BDS 4.4.11.5.4 $A_s \geq 0.005 A_g$

$A_{s_{min}} = (12' \times 2') \, 12in/1ft \times 0.005 = 1.44 in^2$

$A_{s_{min}}/12' = 0.12 in^2/LF$

Per BDS 15.6.5.5 need $0.20 in^2/LF$

So use #5 @12in

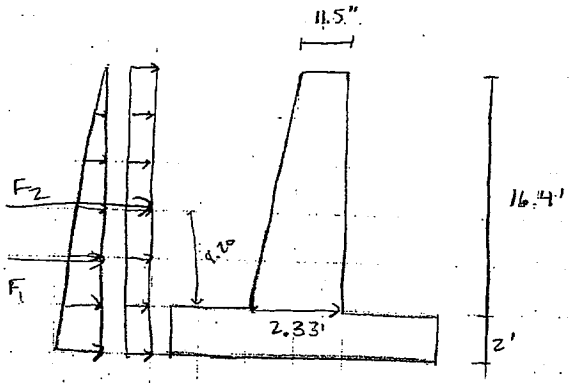
? Try #7 @ 12" $A_s = 0.60$ $d_T = 2'\times12''/ft - 3 - 0.875/2 = 20.56''$

$a = \frac{0.60(60{,}000)}{0.85(3250)(12)} = 1.09$ in $c = 1.09/0.85 = 1.28$

$\epsilon_t = 0.003\left(\frac{20.56 - 1.28}{1.28}\right) = 0.05 > 0.005 \therefore$ ok

$M_n = 0.60(60{,}000)(20.56 - 1.09/2)\times12\times1000 = 63.05$ k-ft > 62.70 ∴ ok

Use #7 @ 12" Top & Bottom




Vertical Stem Reinforcement @ top of footing => H = 16.4' ✓

$F_1 = \frac{1}{2}\gamma K_a H^2 = \frac{1}{2}(0.120\,ksf)(0.3)(16.4')^2 = 4.84$ k/LF ✓

$F_2 = 2'\gamma K_a H = 2'(0.120\,ksf)(0.3)(16.4') = 1.18$ k/LF ✓

$\circlearrowleft M = 4.84(16.4/3) + 1.18(16.4/2) = 26.46 + 9.68 = 36.14$ k-ft/LF

$V = 4.84 + 1.18 = 6.02$ k/LF

Try #9 @ 12" $ld = 25.4''$ & $jd = 23.56''$ from excel calc sheet

$f_s = \frac{M}{A_s jd} \Rightarrow A_s = \frac{M}{f_s jd} = \frac{36.14\text{ k-ft/LF}\times12}{(24)(23.56)} = 0.77\,in^2$

Use #8 @ 12" ✓

Vertical Stem Reinforcement @ 5.5 ft above footing

$F_1 = \frac{1}{2}(0.120\,ksf)(0.3)(16.4-5.5)^2 = 2.14$ k/LF

$F_2 = 2'(0.120)(0.3)(16.4-5.5) = 0.78$ k/LF

$\circlearrowleft M = 2.14((16.4-5.5)/3) + 0.78((16.4-5.5)/2) = 7.78 + 4.25$

$= 12.03$ k-ft/LF

Try #8 @ 12" 12 | 16.4-5.5 x = 0.91' or 10.9"

$d_T = 10.9 + 1.5 = 12.4''$ & jd = 18.43 from excel calc sheet

$A_s = \frac{M}{f_s jd} = \frac{12.03\times12}{(24)(18.43)} = 0.33\,in^2$ Use #6 @ 12"

Vertical Stem Reinforcement @ 11 ft above footing

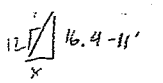
$F_1 = 1/2(0.120)(0.3)(16.4-11)^2 = 0.52$ K/LF

$F_2 = 2(0.120)(0.3)(16.4-11) = 0.39$ K/LF

$\circlearrowleft M = 0.52((16.4-11)/3)) + 0.39((16.4-11)/2)$

$= 0.94 + 1.05 = 1.99$ k-ft/LF

Try # 6 @ 12 in




$x = 5.4''$ $d_t = 5.4'' + 11.5 = 16.90''$ $jd = 13.56''$

$A_s = \frac{M}{f_s jd} = \frac{1.99 \times 12}{(24)(13.56'')} = 0.07$ in²/LF use #3 @ 12"

Vertical Stem Reinforcement @ 7' above footing.

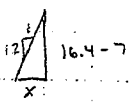
$F_1 = 1/2(0.120)(0.3)(16.4-7)^2 = 1.59$ K/LF

$F_2 = 2(0.120)(0.3)(16.4-7) = 0.68$ K/LF

$\circlearrowleft M = 1.59((16.4-7)/3) + 0.68((16.4-7)/2) = 4.98 + 3.20$

$= 8.18$ K-ft/LF

Try # 8 @ 12 in




$x = 9.40''$ $d_t = 9.4 + 11.5 = 20.90''$

$jd = 16.99''$

$A_s = \frac{M}{f_s jd} = \frac{8.18 \times 12}{(24)(16.99'')} = 0.24$ in²/LF use # 5 @ 12

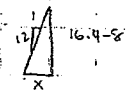
V.S.R @ 8' a.f.

$F_1 = 1/2(0.120)(0.3)(16.4-8)^2 = 1.27$ K/LF

$F_2 = 2(0.120)(0.3)(16.4-8) = 0.60$ K/LF

$\circlearrowleft M = 1.27((16.4-8)/3) + 0.60((16.4-8)/2) = 3.56 + 2.52 = 6.08$ K-ft/LF

Try # 6 @ 12 in




$x = 8.4''$ from excel file $jd = 19.9$

$d_t = 8.4 + 11.5$

$A_s = \frac{M}{f_s jd} = \frac{6.08 \times 12}{24(19.9)} = 0.15$ in²/LF use #4 @ 12"

V.S.R @ 9' a.f.

$F_1 = 1/2(0.120)(0.3)(16.4-9)^2 = 0.99$

$F_2 = 2(0.120)(0.3)(16.4-9) = 0.53$

$\circlearrowleft M = 0.99((16.4-9)/3) + 0.53((16.4-9)/2) = 2.44 + 1.96 = 4.40$ K-ft/LF

Try # 6 @ 12 in $x = 7.40''$ from excel file $jd = 15.49''$ $d_t = 18.9$

$A_s = \frac{M}{f_s jd} = \frac{4.40 \times 12}{(24)(15.49)} = 0.14$ in²/LF use #4 @ 12"

V.S.R @ 10' a.f.

$F_1 = \frac{1}{2}(0.120)(0.3)(16.4-10)^2 = 0.74$ K/LF

$F_2 = 2'(0.120)(0.3)(16.4-10) = 0.46$ K/LF

$\circlearrowleft + M = 0.74((16.4-10)/3) + 0.46((16.4-10)/2) = 1.58 + 1.47 = 3.05$ K-ft/LF

Try #6 @ 12in  x = 6.40"  from excel file jd = 14.52"  dt = 17.90

$A_s = \frac{M}{f_s j d} = \frac{3.05 * 12}{(24)(14.52)} = 0.11 \text{ in}^2$  use #3 @ 12"

V.S.R @ 6' a.f.

$F_1 = \frac{1}{2}(0.120)(0.3)(16.4-6')^2 = 1.95$ K/LF

$F_2 = 2'(0.120)(0.3)(16.4-6) = 0.75$ K/LF

$\circlearrowleft + M = 1.95((16.4-6)/3) + 0.75((16.4-6)/2) = 6.76 + 3.90 = 10.66$ K-ft/LF

Try #8 @ 12"  x = 10.40"  from excel file jd = 17.95"  dt = 21.9

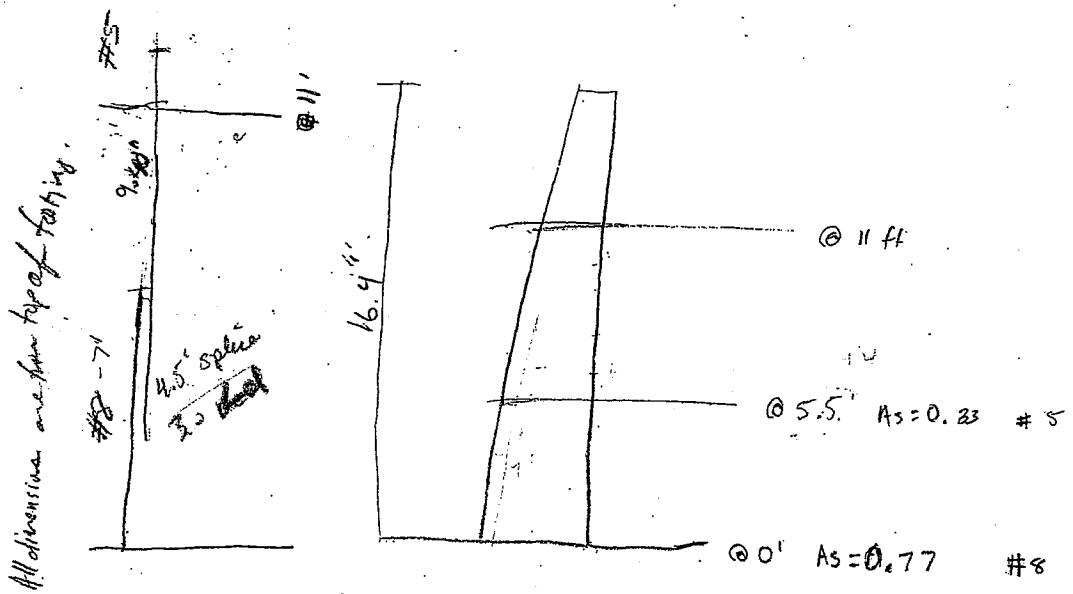
$A_s = \frac{M}{f_s j d} = \frac{10.66 * 12}{(24)(17.95)} = 0.30 \text{ in}^2$  use #3 @ 12"

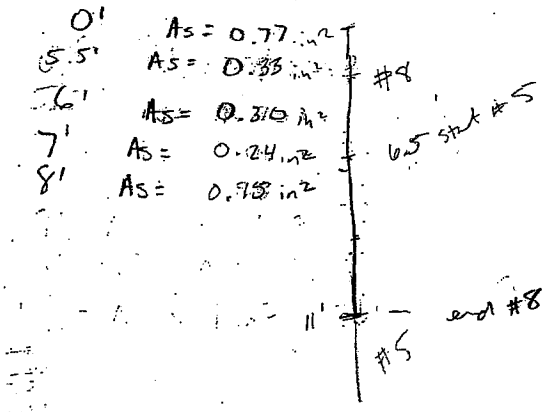
<u>Longitudinal Reinforcement</u>

From BDS 4.4.11.5.4  $A_s \geq 0.005 A_g$

$A_{s_{min}} = \frac{11.5'' + (2.33 * 12\text{in}/1\text{ft})}{2} * 0.005 = 0.10 \text{ in}^2/\text{LF}$

Per BDS 5.6.15.5 need 0.20 in² per foot so use #4 @ 12 in




| | | | | |
|---|---|---|---|---|
| #9 | As = 1.0 | ld = 3.17' | d = 1.128 | Lsplicing = 5.64' |
| #8 | As = 0.792 | ld = 2.5' | d = 1.0 | Lsplicing = 4.5' |
| #7 | As = 0.60 | ld = 2.05' | d = 0.875 | Lsplicing = 3.28' |
| #6 | As = 0.44 | ld = 1.83' | d = 0.750 | Lsplicing = 2.81' |
| #5 | As = 0.31 | ld = 1.50' | d = 0.625 | Lsplicing = 2.34' |

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12-10-09 Win Abut - Abutment 5 (Right) - south 07-LA-710-17.30

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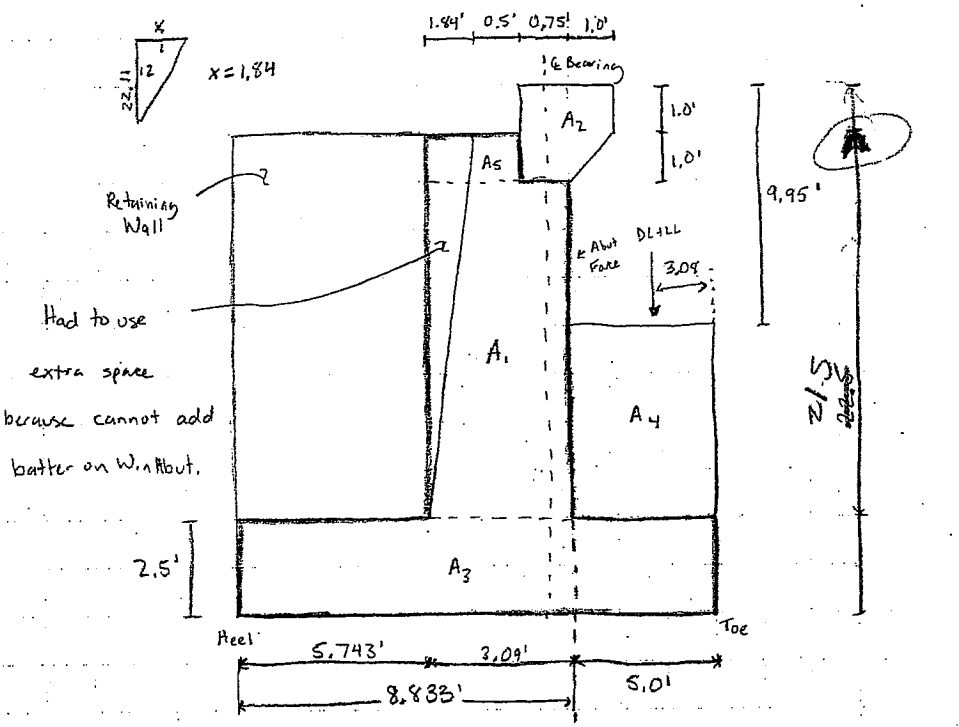
## From Abutment 5 Details - Right Side

Elevation View




— Actual Dimension

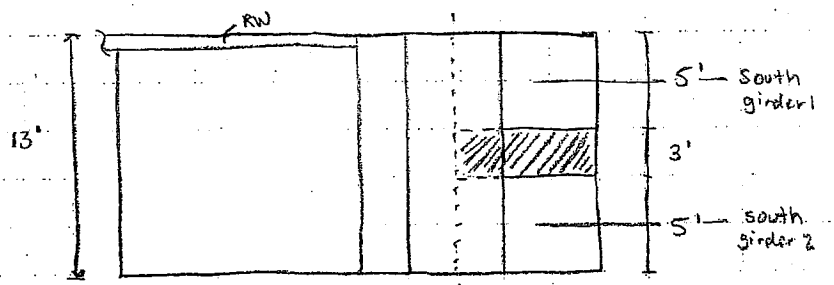
— Dimension used in Win Abut

○ Battered Pile 1:3

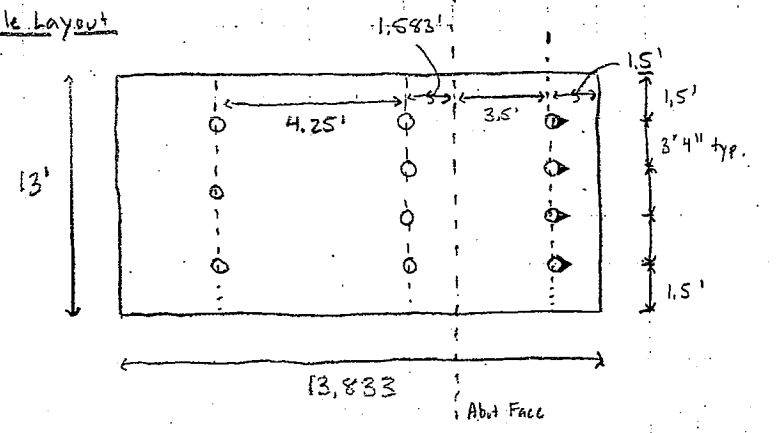
○ Vertical Pile

▨ 2ft of fill from top of footing.

Plan View




Pile Layout




Assumptions/Given:

$f'c = 3.6$ ksi

$fy = 60$ ksi

$Ka = 0.30$

$\gamma_{soil} = 0.12$ kcf

Piles: From BDS 4.5.6.5.1

Lateral Pile Cap = 13 kips

For Driven Pile Dia = 14 in.

Vertical Pile Cap = 70 ton.

Retaining Wall

- In Win Abut place as wingwall.

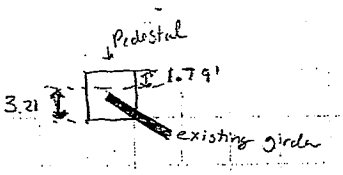
T = 2.25'

H = 21.5'

L = From ℄ of Bearing

∴ L = 8.833 - (0.75/2)

= 8.458

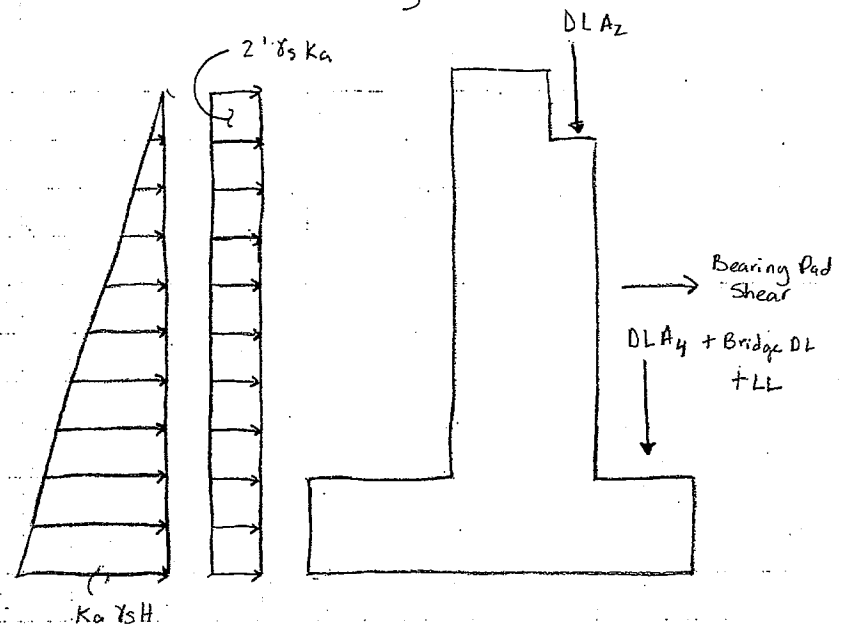



- To calculate centroid for DL+LL and pedestal.

| | P | Arm | Moment Arm |
|---|---|---|---|
| DL+LL | 173.37 + 198.01 = 371.38 | 3.21 | 1,192.13 |
| $P_{pedestal}$ | 82.875 | 2.5 | 207.188 |

$$\frac{\Sigma M_A}{\Sigma P} = \frac{1399.318}{454.255} = \boxed{3.08}$$

Win Abut Loading




Please see sheet 1 for area (An) locations.

• Loads obtained from SAP 2000.

Bridge Dead and Live Loads

@ South Girder 1

• use Joints 1391 & 7211

94.019 + 103.512

= 197.531 Kips

@ South Girder 2

• use Joints 1392 & 7212

79.347 + 94.499

= 173.846 Kips

DL of $A_2$

$A_2 = (1.25' \times 1') + (1' \times 1') + (1' \times 1' \times 0.5) = 2.75\ ft^2$

$DLA_2 = 2.75 \times 13 \times 0.15$

$= 5.363$ Kips

DL of $A_4$

$A_4 = 11.05' \times 5' = 55.2\ ft^2$

$DLA_4 = 55.25 \times 0.15 \times 10'$

$= 82.875$ Kips

DL of soil in b/w pedestals

$= 3' \times 5' \times 2' \times 0.12 = 3.6K$

Bearing Pad Shear

$F_s = 20\%\ DL$

$= 0.2(173.37K) = 34.67$ Kips

• Applied as other lateral load

• In Win Abut applied DL of $A_2$ and Dead load of structure.

• Applied actual dead load of bridge, live load (unfactored), the two pedestals weight and 2ft of soil in between pedestals as other vertical load.

Input for Win Abut

↳ DL = 5.4 K

↳ Other VL = 197.531 + 173.846 + 82.875 + 3.6 = 457.852

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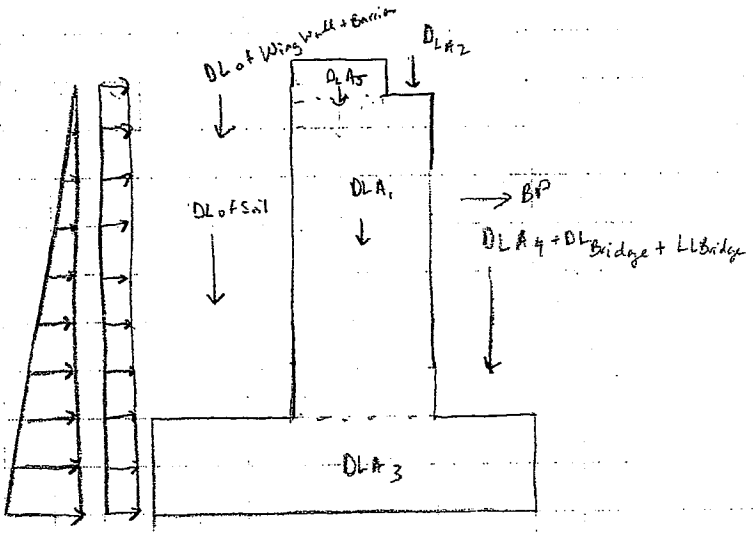
12-10-09 Win Abut - Abutment 5 Right 07-LA-710-17.30

Verify Win Abut Ouput

Group I Case I




For Case I $K_a = 100\%$ $K_a = 0.3$

From Sheet 2

$DL_{A_2} = 5.363$ k

$DL_{A_4} = 82.875$ k

$DL_{Bridge} + LL_{Bridge} =$ 371.377 k

DL of soil b/w pedestals = 3.6 k

Width = 13' - 2.25' (Length of WW) = 10.5'

Check Load Calculations

- Lateral Earth Loads

1. $K_a = 0.30$

$F = \frac{1}{2} K_a (\gamma)(H^2) \times W = \frac{1}{2}(0.3)(.12)(24)^2 \times 10.75' = 111.46$ k ✓ OK

2. $K_a = 0.225$

$F = \frac{1}{2}(0.225)(0.12)(24)^2 \times 10.75' = 83.59$ k ✓ OK

Surcharge

3. $K_a = 0.30$

$F = 2' K_a \gamma H \times W = 2(0.3)(.12)(24) \times 10.75 = 18.57$ k ✓ OK

4. $K_a = 0.225$

$F = 2'(0.225)(.12)(24) \times 10.75 = 13.93$ k ✓ OK

Abutment Dead Loads

5. Backwall = 1' × 13 ft × (1.84 + 0.5') × 0.15 = 4.56 k ✓ ok - $D_L A_5$
6. Seat = 3.09' × 21.5' × 13' × 0.15 = 129.55 k ✓ ok - $DLA_1$
7. Footing = 13.833' × 2.5' × 13' × 0.15 = 67.4 k ✓ ok - $DLA_3$
8. Wing Wall = 21.5' × 5.743' × 2.25' × 0.15 = 41.67 ✓ ok
9. Triangular section = (1' × 5.743' × 2.25' × 0.15)/2 = 0.97 ✓ ok
10. Barrier = (5.743' + 1.84' + 0.5) × 0.4 k/ft = 3.23 ✓ ok

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12-10-09 WinAbut - Abutment 5 (Right) 07-LA-710-17.30

~~Rev. 1~~ - 3-2-09 Rev. 2 - 3-18-10

## Soil Dead Loads

11. Soil on Heel = 21.5' × 5.743' × 10.75' × 0.12 = 159.28k ✓ OK

12. Surcharge on Heel = 2' × 5.743' × (.12 k/ft³) × 10.75 = 14.82 k ✓ OK.

## Checking Controlling Load Case - Case 1 Group 1

Vertical Loads

| Load # | Load (Kips) | Arm (from Toe in ft) | Moment (k-ft) |
|---|---|---|---|
| other VL | 457.852 | 3.08 | 1410.18 |
| DL $A_2$ | 5.363 | 5' + (0.75'/2) = 5.375' | 28.83 |
| 5. | 4.56 | 5' + 0.75' + (1.84' + 0.5')/2 = 6.92' | 31.56 |
| 6. | 129.55 | 5' + (3.09'/2) = 6.545' | 847.90 |
| 7. | 67.4 | 13.833'/2 = 6.917' | 466.21 |
| 8. | 41.67 | 5' + 3.09' + (5.743'/2) = 10.962 | 456.79 |
| 9. | 0.97 | 5' + 3.09' + (5.743'/3) = 10.00 | 9.7 |
| 10. | 3.23 | 5' + 0.75' + (5.743' + 1.84' + 0.5')/2 = 9.792' | 31.63 |
| 11. | 159.28 | 5' + 3.09' + (5.743'/2) = 10.962' | 1746.03 |
| 12. | 14.82 | 5' + 3.09' + (5.743'/2) = 10.962' | 162.46 |

$\Sigma L = 884.695$ kips $\boxed{15}$ ✓ $\Sigma M = 5{,}191.29$

C.G. of VL = $\frac{5191.29}{884.695}$ = 5.87' from Toe

13. Total Lateral Load = 111.46 + 18.57 + 34.67 (Bearing Pad Shear) = 164.7 ✓ OK $\boxed{13}$

Use 130.03 for Case 1 Group 1

| Load (Kips) | Arm (ft) | Moment (k-ft) |
|---|---|---|
| 111.46 | 24'/3 = 8' | 891.68 |
| 18.57 | 24'/2 = 12' | 222.84 |
| R1 34.67 | (24 - 8.95) = 15.05 | 521.78 |

R1 Note: WinAbut does not take into account bearing pad shear when calculating C.G. of Pile group. $\Sigma M = 1{,}636.30$

~~C.G. of Pile Group = $\frac{\Sigma M}{\Sigma P} = \frac{1636.30}{164.70} = 9.94'$~~

M = 1,636.30 - (884.695)(5.87' - 9.94') = 5,237.01 k-ft

~~$I_{pile\ group} = 4(9.94 - 1.5)^2 + 4(9.94 - 6.583)^2 + 3(9.94 - 10.83)^2$~~

~~= 284.93 + 45.08 + 2.38 = 332.39~~

see next sheet for Rev. 2

L. Sanchez - [illegible] 07-2021-01 #267

12-10-09 Wing Abut - Abut 5 (Right) 07-LA-710-17.30

Revision 1 Rev. 2 - 3/18/10

$R = P/N + Mc/I_{piles\ row}$

$R_{(row\ \#1)} = [884.695 \div 11] + \frac{1618.61}{\cancel{5237.01}} \cdot (\frac{5.89}{\cancel{9.94}} - 1.5) \div \frac{152.21}{\cancel{332.39}}$

$80.43 + \cancel{132.98}\ 46.04 = \cancel{213.41\ K}\ \boxed{126.48\ K}$

$R_{(row\ \#2)} = [884.695 \div 11] + \frac{1618.61}{\cancel{5237.01}} (\frac{5.89}{\cancel{9.94}} - 6.583) \div \frac{152.21}{\cancel{332.39}} =$

$80.43\ \cancel{+52.89}\ -7.37 = \cancel{133.32\ K}\ \boxed{73.06\ K}$

$R_{(row\ \#3)} = [884.695 \div 11] + \frac{1618.61}{\cancel{5237.01}} (\frac{5.89}{\cancel{9.94}}\ 10.83) \div \frac{152.21}{\cancel{332.39}} =$

$80.43\ \cancel{-14.02}\ -52.53 = \cancel{66.41\ K}\ \boxed{27.9\ K}$

Revision 2

C.G. of Pile Group $= [4(1.5') + 4(6.583') + 3(10.83')] \div 11$
$= 5.89$ from Pier

$I_{pile\ group} = 4(5.89 - 1.5)^2 + 4(5.89 - 6.583)^2 + 3(5.89 - 10.83)^2$
$= 77.08 + 1.92 + 73.21$
$= 152.21\ ft^2$

$M = 1{,}636.30 - (884.695)(5.87' - 5.89') = 1618.61$

01-202101

DATE 1/20/10

Rev. 1-3-1-10

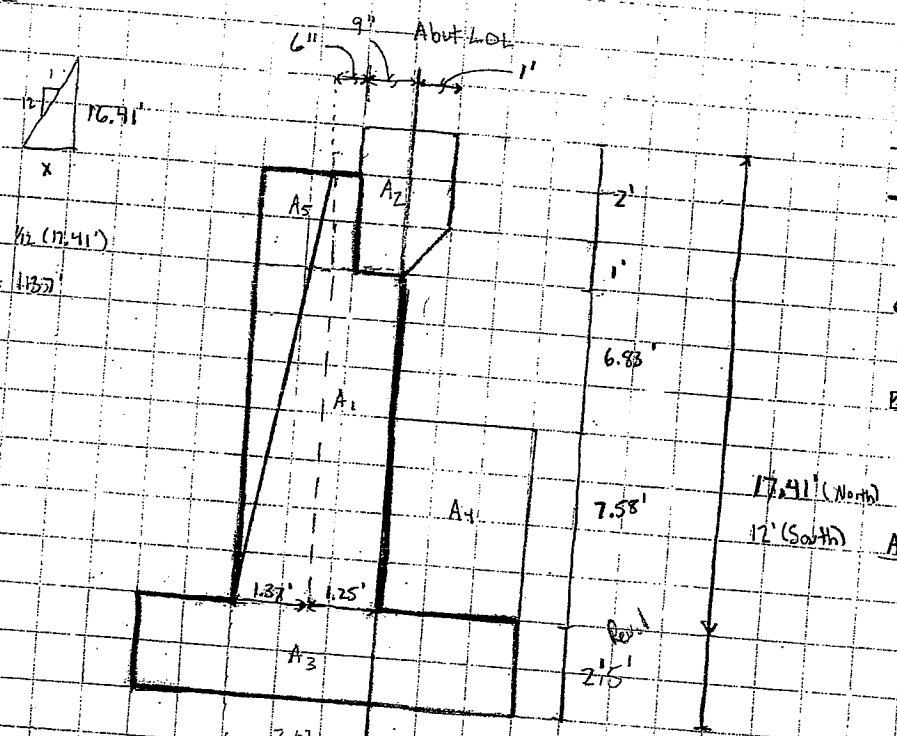
Win Abut - Abutment 1 North

(South similar)

#268

07-LA-710-17.i

*Not To scale




x = 1/12 (17.91')

x = 1.493'

— Actual Dimension

— Dimension used in Win A

Battered Pile 1:3

Vertical Pile

2' of fill from top of footi

Assumptions / Given:

f'c = 3.6 ksi

fy = 60 ksi

Ka = 0.30

γsoil = 0.120 kcf

γconc = 0.150 kcf

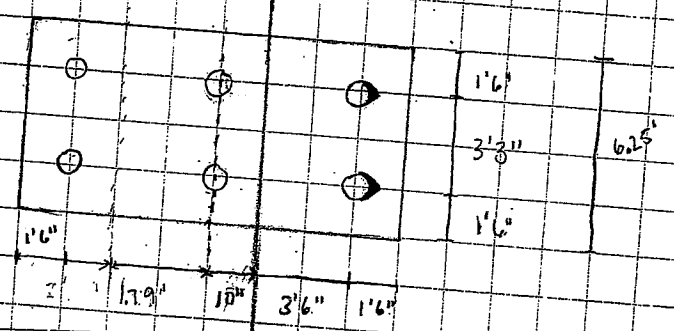
Piles: From Bps 4, 5, 6, 5.4

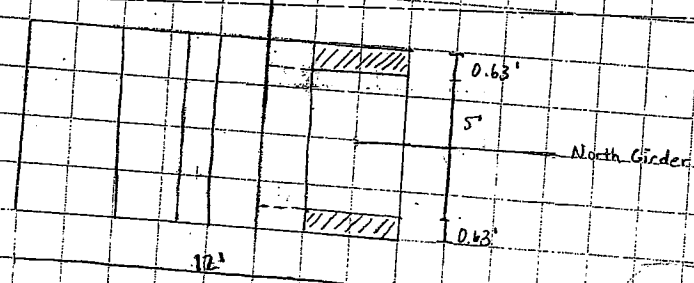
Lateral Pile Cap = 19 Kips

For Driven Pile Dia = 12"

Vertical Pile Cap = 70 tons

Pile Layout




North Girder 1 - Loads obtained from SAP 2000

Joint 1357 - Dead Load = 91.502 Kips

Joint 7177 - Live Load = 90.428 Kips

Total Load = 181.93 Kips

Rev. 1

From MDX

104 Kips ✓

119 Kips

223 Kips

P-15 Load

230 Kips

For check mode

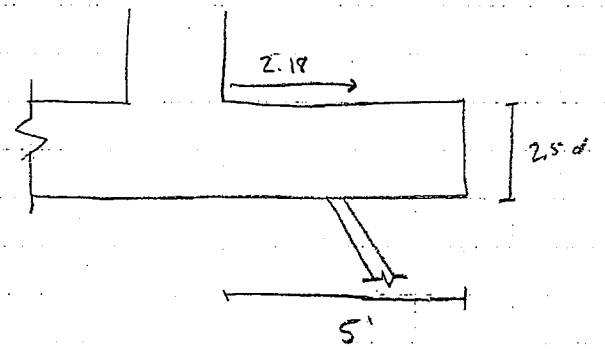
other VL = 1.25(104) + 1.35(230) = 130 + 310.5 = 440.50 Kips

Located 3.17' from toe.

Form 201101 08 GSD

L. Sanchez-Manzo 07-202101 #269 6

12-10-09 Check for Shear - Oneway 07-LA-710-17.30




deff = 2.18 from output page 6

Piles Total Shear = 131 k × 4
= 524 k

A = 2.5 × 13 = 32.5 ft²
= 4680 in²

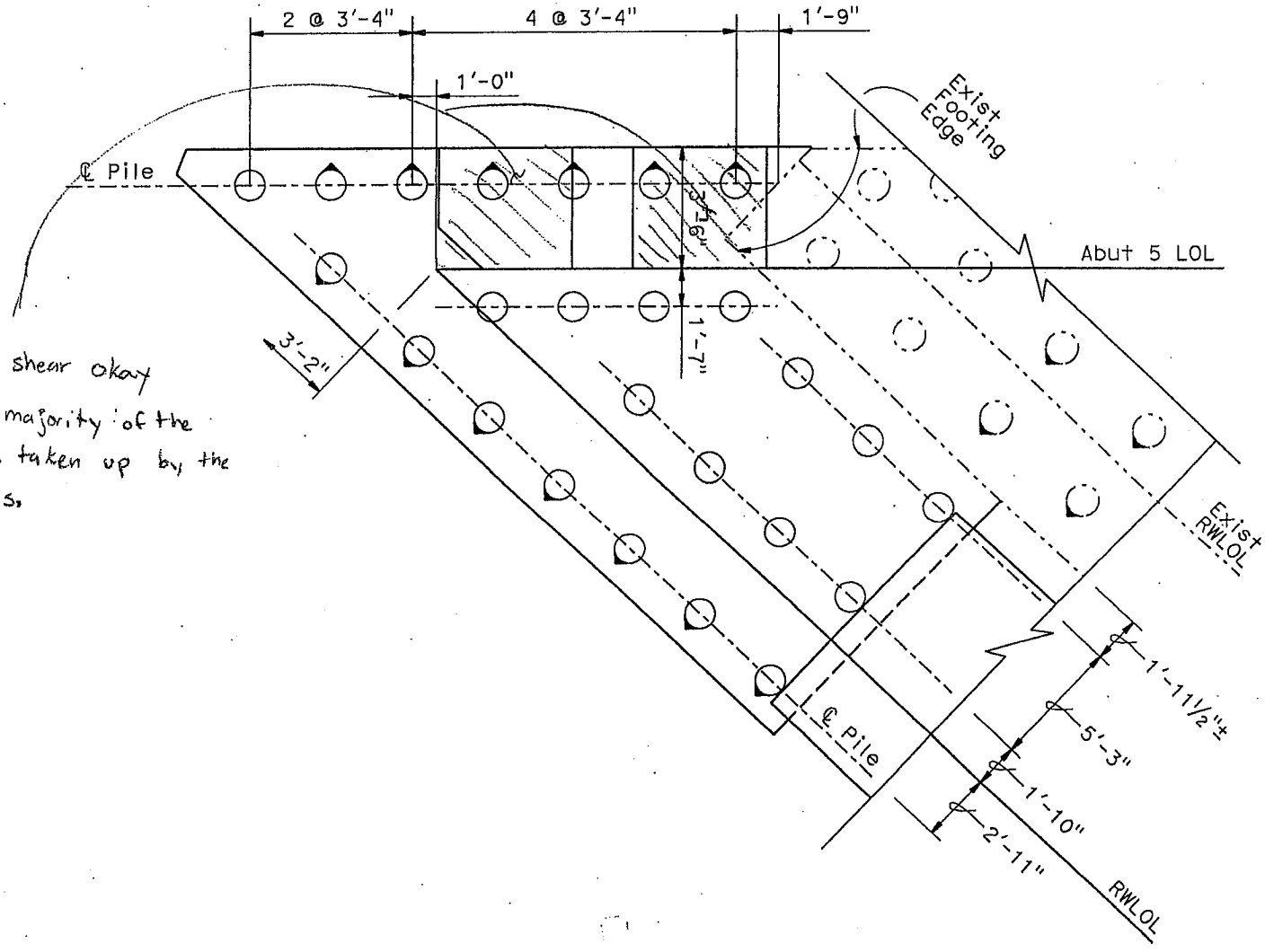
$\sigma_{actual} = \frac{524k}{4680} = 0.112$ ksi

$\sigma_{all} = 0.95\sqrt{f_c'} = 0.95\sqrt{3250}$
= 54 psi

$\sigma_{actual} > \sigma_{all}$ N.G.

Please see attached Figure 1.
with explanation.

Figure 1




- One-way shear okay because majority of the force is taken up by the pedestals.

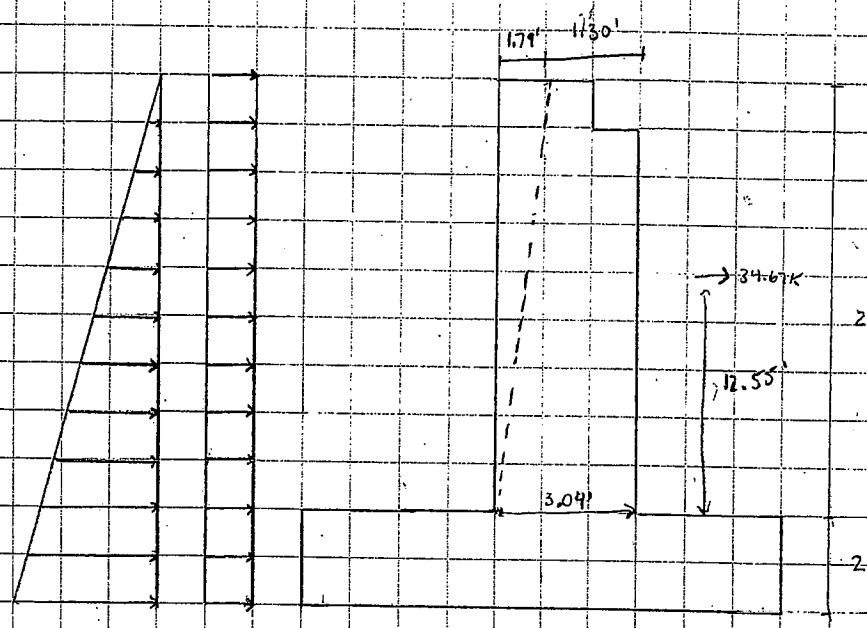
P-15 Truck Input

Joints 7211 & 7212
267 K & 277 K use larger

Other VL = (277) 1.35 + (173.37) 1.25 + 82.875 + 3.6 =
373.95 + 216.71 + 82.875 + 3.6
= 677.14 K

$$C.G. = \frac{\Sigma M}{\Sigma F} = \frac{(173.37 + 277)(3.21) + (82.875 + 3.6)(2.5)}{173.37 + 277 + 82.875 + 3.6}$$

$$= \frac{1445.69 + 216.19}{586.85} = 3.10' \text{ from toe.}$$




Bearing Pad Shear

1 girder DL = 94.019

46° 38' 26"

26" | 1/2 = 0.43 | 60"

38" | 1' | 60°

46.64°

DL/LF = 94.019 K cos 46.6° / 11

= 5.87 K/LF

$F_s$ = 0.2(5.87 K/LF) = 1.17 K/LF

Calculating Vertical Stem Reinforcement

@ top of footing H = 21.5'

$F_1 = \frac{1}{2}\gamma K_a H^2 = 0.5(0.120)(0.3)(21.5)^2 = 8.32$ K/LF

$F_2 = 2\gamma K_a H = 2(0.120)(0.3)(21.5) = 1.55$ K/LF

$\circlearrowleft M = 8.32(21.5/3) + 1.55(21.5/2) + (1.17 \text{ K/LF})(12.55')$

$= 59.63$ K-ft/LF + 16.66 K-ft/LF + 14.68 K-ft/LF

$= 90.97$ K-ft/LF

Try #11 @ 12" $d_t = 36.48''$ jd = 30.24" from excel calc

$f_s = \frac{M}{A_s jd} \Rightarrow A_s = \frac{M}{f_s jd} = \frac{90.97 \text{ k-ft/LF} * 12 \text{ in/ft}}{24 \text{ ksi } (30.24'')}$

$= 1.50 \text{ in}^2$

use #11 @ 12"

@ 7ft above top of footing $H = 21.5 - 7' = 14.50'$

$F_1 = \frac{1}{2}\gamma K_a H^2 = \frac{1}{2}(0.120)(0.3)(14.5)^2 = 3.78$ K/LF

$F_2 = 2'\gamma K_a H = 2(0.120)(0.3)(14.5) = 1.04$ K/LF

$\circlearrowleft M = 3.78(14.5/3) + 1.04(14.5/2) + (1.17\text{ K/LF})(12.55 - 7)$

$= 18.27 + 7.54 + 6.49 = 32.30$ K-ft/LF

Try #9 @ 12"

12 | 14.50 x

$x = 14.5''$ $d_t = 14.5'' + (1.30' * 12'') = 30.10''$ (15.6)

$jd = 24.65$

$A_s = \frac{M}{f_s jd} = \frac{32.30 * 12}{24(24.65)} = 0.66$ in²/LF use #8 @ 12"

@ 14 ft above top of footing $H = 21.5 - 14$ ft $= 7.5'$

$F_1 = \frac{1}{2}\gamma K_a H^2 = \frac{1}{2}(0.120)(0.3)(7.5)^2 = 1.01$ K/LF

$F_2 = 2'\gamma K_a H = 2(0.120)(0.3)(7.5) = 0.54$

$\circlearrowleft M = 1.01(7.5/3) + 0.54(7.5/2) = 2.53 + 2.03 = 4.56$ K-ft/LF

Try #7 @ 12" $d_t = 7.5'' + 15.6'' = 23.10$ in

$jd = 18.36$

$A_s = \frac{M}{f_s jd} = \frac{4.56 * 12}{(24)(18.36)} = 0.27$ in²/LF

use #5 @ 12"

@ 8ft above top of footing $H = 21.5 - 8' = 13.5'$

$F_1 = \frac{1}{2}\gamma K_a H^2 = \frac{1}{2}(0.120)(0.3)(13.5)^2 = 3.28$ K/LF

$F_2 = 2\gamma K_a H = 2(0.120)(0.3)(13.5) = 0.97$ K/LF

$\circlearrowleft M = 3.28(13.5/3) + 0.97(13.5/2) = 14.76 + 6.96 = 21.72$ K-ft/LF

Try #9 @ 12" $d_t = 13.5'' + 15.6'' = 29.10$ $jd = 23.69''$

$A_s = \frac{M}{f_s jd} = \frac{21.72 * 12}{(24)(23.69)} = 0.46$ in²/LF use #7 @ 12".

@ 9 ft above top of footing $H = 21.5 - 9' = 12.5'$

$F_1 = \frac{1}{2}\gamma K_a H^2 = \frac{1}{2}(0.120)(0.3)(12.5)^2 = 2.81$

$F_2 = 2\gamma K_a H = 2(0.120)(0.3)(12.5) = 0.90$

$\circlearrowleft M = 2.81(12.5/3) + 0.90(12.5/2) = 11.71 + 5.63 = 17.34$ K-ft/LF

Try #7 @ 12" $d_t = 12.5 + 15.6 = 28.10''$ $jd = 23.20$ $A_s = \frac{M}{f_s jd} = \frac{17.34 * 12}{24(23.20)} = 0.37$ in² use #6 @ 12"

OSP 98 101702

14ft $A_s = 0.27\ in^2/LF$ #5

9ft $A_s = 0.37\ in^2/LF$

8ft $A_s = 0.46\ in^2/LF$

7ft $A_s = 0.60\ in^2/LF$

6ft $A_s = 0.75\ in^2/LF$

5ft $A_s = 0.87\ in^2/LF$

4ft $A_s = 0.97\ in^2/LF$

2ft $A_s = 1.22\ in^2/LF$

0ft $A_s = 1.60\ in^2/LF$ #11

#11 $l_d = 5'$

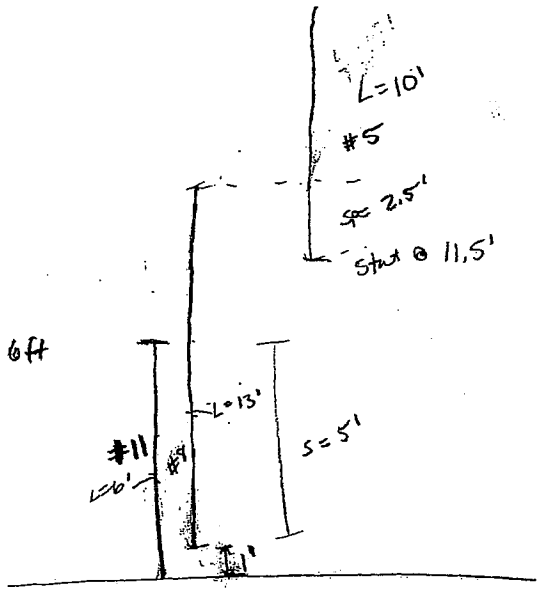
#9 $l_d = 3.17'$

#7 $l_d = 2.08'$

#6 $l_d = 1.83'$

#5 $l_d = 1.5'$

#8 $l_d = 2.5'$




Splice

#11 & #8 = 60" × 1.0 = 60" = 5'

#8 & #5 = 0.625 × 45 = 28.13" = 2.34'

```
*************************************************************************
*                              WinAbut                                  *
*                       ABUTment Design Program                         *
*                          (SS) SHORT SEAT                              *
*                        (HC) HIGH CANTILEVER                           *
*               California Department of Transportation                 *
*                     -- For CALTRANS Use Only --                       *
*               Graphical User Interface / PC Version                   *
*                            Version 1.1.5                              *
*                              12.05.03                                 *
*                                                                       *
*************************************************************************
```

```
****************
* GENERAL DATA *
*************************************************************************

TYPE ABUTMENT            (HIGH CANTILEVER) =      HC
TYPE SUPERSTRUCTURE              (CIP/PS?) =      NO
TYPE FOOTING                               =    PILE
TYPE ANALYSIS                              =  DESIGN
TYPE DESIGN          (SERVICE LOAD DESIGN) =     SLD

MATERIAL PROPERTIES:
--------------------
ULTIMATE CONCRETE COMPRESSIVE STRESS  (FC) =     3.60 Ksi
YIELD STRESS FOR STEEL REBARS         (FY) =    60.00 Ksi
ALLOWABLE CONCRETE STRESS (% FC)           =    40.00 %
                                           =     1.44 Ksi
ALLOWABLE STEEL STRESS    (% FY)           =    40.00 %
                                           =    24.00 Ksi
YOUNGS MODULUS FOR CONCRETE           (EC) =   3420.0 Ksi
YOUNGS MODULUS FOR STEEL REBARS       (ES) =  29000.0 Ksi
MODULAR RATIO (ES/EC)                  (N) =     8.48
CONCRETE UNIT WEIGHT                       =   150.00 Pcf

BACKFILL SOIL DATA:
-------------------
DEPTH OF FILL ON TOE                       =     0.00 Ft
DEPTH OF FILL ON HEEL                      =    21.50 Ft
DEPTH OF SURCHAGE ON HEEL                  =     2.00 Ft
ENVIRONMENTAL FACTOR                   (Z) =   170.00 K/in
COEF OF ACTIVE EARTH PRESSURE         (KA) =     0.30
SOIL UNIT WEIGHT                           =   120.00 Pcf
```

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```
*************
* PILE DATA *
*************************************************************************

VERTICAL PILE CAPACITY - SERVICE          =    70.00 Tons
LATERAL  PILE CAPACITY - SERVICE          =    10.00 K
LATERAL  PILE CAPACITY - ULTIMATE         =    40.00 K
PILE DIAMETER                             =    12.00 in
BATTER FACTOR                             =   1:3.00
```

PILE LAYOUT DATA (Sorted) (Ft):

| ROW NO. | DISTANCE TO TOE | NO. OF PILES IN ROW | NO. OF PILES BATTERED |
|---|---|---|---|
| 1 | 1.50 | 4 | 4 |
| 2 | 6.58 | 4 | 0 |
| 3 | 10.83 | 3 | 0 |

```
TOTAL NUMBER OF PILES                     =   11
DISTANCE FROM TOE TO C.G. OF PILES (CG)   =    5.89 Ft
MOMENT OF INERTIA OF PILE GROUP    (IX)   =  152.20 Pile-Ft^2

***********************
* REBAR AND LOAD DATA *
*************************************************************************
```

| REBAR LOCATION | BAR SIZE | NO. OF BARS | COVER(in) |
|---|---|---|---|
| BACKWALL | 8 | 12 | 2.00 |
| STEM | 11 | 12 | 3.00 |
| FOOTING TOP | 7 | 0 | 3.00 |
| FOOTING BOTTOM (HEEL) | 11 | 0 | 3.00 |
| FOOTING BOTTOM (TOE ) | 11 | 0 | 3.00 |

SERVICE LOAD DATA (K ):

| | TOTAL DEAD LOAD | :- LIVE LOAD -: TOTAL H-LOAD | 1.15P+H P-LOAD | TOTAL BEARING PAD SHEAR | LATERAL EARTH PRESSURE | OTHER LATERAL LOAD | OTHER VERTICAL LOAD |
|---|---|---|---|---|---|---|---|
| LOADS | 5. | 0. | 0. | 0. | | 35. | 466. |
| FACTORS | 1.30 | 1.30 | 1.30 | 1.30 | 1.69 | 1.30 | 1.30 |

```
DISTANCE ABOVE SEAT TO OTHER LATERAL  LOAD (Ft) =      -7.95
DISTANCE FROM  HEEL TO OTHER VERTICAL LOAD (Ft) =                10.76

CONSIDER OTHER LOADS IN LOAD GROUPS (Y/N)              12345     12345
                                                       YYYYN     YNNYN
```

 279

Abut 5 South

```
*****************************************
* MOMENTS ABOUT TOE OF FOOTING (K,Ft,K/Ft) * (SERVICE)
*****************************************************************************
```

| | FORCE | ARM | MOMENT |
|---|---|---|---|
| **LATERAL EARTH LOADS (BACKFILL TO TOP OF BACKWALL)** | | | |
| EARTH PRESSURE (KA =0.300) | 111.5 | 8.00 | 892. |
| EARTH PRESSURE (KA =0.225) | 83.6 | 8.00 | 669. |
| SURCHARGE PRESSURE (KA =0.300) | 18.6 | 12.00 | 223. |
| SURCHARGE PRESSURE (KA =0.225) | 13.9 | 12.00 | 167. |
| EARTHQUAKE | 0.0 | 0.00 | 0. |
| **SUPERSTRUCTURE LOADS** | | | |
| DEAD LOAD | 5.4 | 5.38 | -29. |
| H-LIVE LOAD | 0.0 | 5.38 | 0. |
| P-LIVE LOAD | 0.0 | 5.38 | 0. |
| BEARING PAD SHEAR | 0.0 | 23.00 | 0. |
| **OTHER LOADS** | | | |
| OTHER LATERAL LOAD | 34.7 | 15.05 | 522. |
| OTHER VERTICAL LOAD | 466.2 | 3.07 | -1431. |
| **ABUTMENT DEAD LOADS** | | | |
| BACKWALL | 4.6 | 6.92 | -32. |
| SEAT | 123.5 | 6.54 | -808. |
| EXTRA SEAT PORTION (HIGH CANTILEVER) | 0.0 | 8.09 | 0. |
| FOOTING | 67.4 | 6.92 | -466 |
| FOOTING KEY | 0.0 | 0.00 | 0. |
| WINGWALL TOP RECTANGULAR SECTION | 41.7 | 10.96 | -457. |
| WINGWALL BOT TRIANGULAR SECTION | 0.0 | 10.01 | 0. |
| WINGWALL EXTRA SECTION (HIGH CANTILEVER) | 0.0 | 8.09 | 0. |
| BARRIER | 3.2 | 9.80 | -32. |
| SIDE KEYS | 0.0 | 5.38 | 0. |
| | 240 | | |
| **SOIL DEAD LOADS** | | | |
| SOIL ON HEEL (BACKFILL TO TOP OF BACKWALL) | 159.2 | 10.96 | -1745. |
| EXTRA SOIL ON HEEL (HIGH CANTILEVER) | 0.0 | 8.09 | 0. |
| SOIL ON TOE | 0.0 | 2.50 | 0. |
| SURCHARGE ON HEEL | 14.8 | 10.96 | -162. |

Total

Permanent+LL+LLsurcharge

= 660 + 195 + 15
(143+95)

= 870 / 13 = 67 k/LF

Permanent

| | |
|---|---|
| Abut WT | 240 + 5.4 = 245 kips |
| Soil WT | 159 kips |
| DL from super | 94 + 79 = 173 |
| Pedestal | 83 |
| | 660 kips for 13' Abut |
| | ⇒ 51 k/ft |

 280

```
*****************************************************
* SERVICE PILE LOADS (SER) FOR ABUTMENT DESIGN (K,Ft) *
*****************************************************
```

Note: Toe & Heel Piles Loads and Capacities are along the Vertical.

| | | :-- VERTICAL --: | | :-- LATERAL --: | | ----- HEEL ----: | | ----- TOE -----: | | |
|---|---|---|---|---|---|---|---|---|---|---|
| GRP | CA | TOTAL | ECCEN | TOTAL | CAP | MAX | CAP | MAX | CAP | |
| IH | 1 | 886. | 1.91 | 165. | 283. | 26. | 140. | 129. | 133. | OK |
| IH | 2 | 886. | 1.60 | 132. | 272. | 35. | 140. | 121. | 133. | OK |
| IH | 3 | 871. | 1.78 | 146. | 275. | 29. | 140. | 124. | 133. | OK |
| IH | 4 | 871. | 1.52 | 118. | 267. | 36. | 140. | 117. | 133. | OK |
| II | 1 | 420. | 0.90 | 165. | 175. | 26. | 140. | 49. | 133. | OK |
| II | 2 | 420. | 0.24 | 132. | 165. | 35. | 140. | 41. | 133. | OK |
| III | 1 | 411. | 0.95 | 165. | 230. | 25. | 210. | 49. | 199. | OK |
| III | 2 | 411. | 0.27 | 132. | 219. | 34. | 210. | 41. | 199. | OK |
| IV | 1 | 886. | 1.91 | 165. | 310. | 26. | 175. | 129. | 166. | OK |
| IV | 2 | 886. | 1.60 | 132. | 299. | 35. | 175. | 121. | 166. | OK |
| IV | 3 | 871. | 1.78 | 146. | 303. | 29. | 175. | 124. | 166. | OK |
| IV | 4 | 871. | 1.52 | 118. | 294. | 36. | 175. | 117. | 166. | OK |
| V | 1 | 712. | 2.05 | 0. | 308. | 17. | 210. | 107. | 199. | OK |

SUMMARY OF CONTROLLING LOAD CASES
(RATIO = CAP / LOAD)

| GRP | CA | DESCRIPTION | LOAD | CAP | RATIO | |
|---|---|---|---|---|---|---|
| II | 1 | TOTAL LATERAL LOAD | 165. | 175. | 1.07 | OK |
| IH | 4 | MAX HEEL LOAD | 36. | 140. | 3.86 | OK |
| IH | 1 | MAX TOE LOAD | 129. | 133. | 1.03 | OK |

Controlling Design Loading

| Grp | Ca | Description | Load |
|---|---|---|---|
| IH | 1 | Max Toe Load | 136. |

Note: Allowable Overstress = 1.00

Loading is Along the Batter.

```
*********************************************
* STEEL REQUIRED - SERVICE LOAD DESIGN (SLD) *
*********************************************
```

KEY TO REBAR LOCATIONS (BAR-LOC)

1 = BACKWALL
2 = STEM
3 = FOOTING TOP
4 = FOOTING BOTTOM (HEEL)
5 = FOOTING BOTTOM (TOE)

| BAR LOC | LOADS GRP | CA | APPLIED MOMENT (K-Ft) | :- STRESS-: FC (Ksi) | FS (Ksi) | EFF DEPTH (in) | STEEL AREA (in^2) | BAR SIZE | BAR SPACE (in) | BAR REQ | ERR CODE |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | | | 436. | 0.60 | 23.0 | 25.5 | 9.5 | 8 | 13.7 | 12 | |
| 2 | | | 1255. | 0.81 | 22.5 | 33.3 | 21.8 | 11 | 11.6 | 14 | |
| 3 | IH | 1 | -369. | 0.52 | 22.7 | 26.5 | 7.8 | 7 | 12.6 | 13 | |
| 4 | V | 1 | 62. | 0.18 | 18.7 | 26.2 | 1.6 | 11 | 78.0 | 1 | 3 |
| 5 | IH | 1 | 1751. | 1.34 | 23.1 | 26.2 | 39.0 | 11 | 6.3 | 25 | |

ERROR CODES:

1 = BAR SPACING LESS THAN AASHTO MINIMUM. (CODE 8.21)
2 = BAR SPACING LESS THAN PREFERRED CALTRANS MINIMUM. (BR. DES. DET.)
3 = BAR SPACING MORE THAN THE AASHTO MAXIMUM ( 18. in). (CODE 8.20)

```
*****************************************************************
* STEM MOMENT ENVELOPE AND REQUIRED STEEL AREAS (BAR CUT-OFFS) *
*****************************************************************
```

(APPLIED MOMENT - GROUP IV CASE 1)
(BACKFILL, SURCHARGE, BEARING PAD SHEAR AND OTHER LATERAL LOAD)

| DIST FROM FOOTING TOP (Ft) | APPLIED MOMENT (K-Ft) | :- STRESS --: FC (Ksi) | FS (Ksi) | STEEL AREA (in^2) | BAR SIZE | BAR SPACE (in) | BARS REQ | ERROR CODE |
|---|---|---|---|---|---|---|---|---|
| 0.00 | 1255. | 0.81 | 22.5 | 21.8 | 11 | 11.6 | 14 | |
| 1.00 | 1119. | 0.77 | 23.3 | 18.7 | 11 | 13.7 | 12 | |
| 2.00 | 992. | 0.70 | 22.4 | 17.2 | 11 | 15.0 | 11 | |
| 3.00 | 872. | 0.67 | 23.9 | 14.0 | 11 | 18.8 | 9 | 3 |

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```
*************************************************************************
*                              WinAbut                                  *
*                     ABUTment Design Program                           *
*                        (SS) SHORT SEAT                                *
*                      (HC) HIGH CANTILEVER                             *
*              California Department of Transportation                  *
*                   -- For CALTRANS Use Only --                         *
*              Graphical User Interface / PC Version                    *
*                          Version 1.1.5                                *
*                            12.05.03                                   *
*                                                                       *
*************************************************************************
```

```
****************
* GENERAL DATA *
*************************************************************************
```

| | | | |
|---|---|---|---|
| TYPE ABUTMENT | (HIGH CANTILEVER) | = | HC |
| TYPE SUPERSTRUCTURE | (CIP/PS?) | = | NO |
| TYPE FOOTING | | = | PILE |
| TYPE ANALYSIS | | = | CHECK |
| TYPE DESIGN | (LOAD FACTOR DESIGN) | = | LFD |

MATERIAL PROPERTIES:
---

| | | | | |
|---|---|---|---|---|
| ULTIMATE CONCRETE COMPRESSIVE STRESS | (FC) | = | 3.60 | Ksi |
| YIELD STRESS FOR STEEL REBARS | (FY) | = | 60.00 | Ksi |
| ALLOWABLE CONCRETE STRESS (% FC) | | = | 40.00 | % |
| | | = | 1.44 | Ksi |
| ALLOWABLE STEEL STRESS (% FY) | | = | 40.00 | % |
| | | = | 24.00 | Ksi |
| YOUNGS MODULUS FOR CONCRETE | (EC) | = | 3420.0 | Ksi |
| YOUNGS MODULUS FOR STEEL REBARS | (ES) | = | 29000.0 | Ksi |
| MODULAR RATIO (ES/EC) | (N) | = | 8.48 | |
| CONCRETE UNIT WEIGHT | | = | 150.00 | Pcf |

BACKFILL SOIL DATA:
---

| | | | | |
|---|---|---|---|---|
| DEPTH OF FILL ON TOE | | = | 0.00 | Ft |
| DEPTH OF FILL ON HEEL | | = | 21.50 | Ft |
| DEPTH OF SURCHAGE ON HEEL | | = | 2.00 | Ft |
| ENVIRONMENTAL FACTOR | (Z) | = | 170.00 | K/in |
| COEF OF ACTIVE EARTH PRESSURE | (KA) | = | 0.30 | |
| SOIL UNIT WEIGHT | | = | 120.00 | Pcf |

```
*******************
* STRUCTURAL DATA *
*******************************************************************************

ABUTMENT DATA:
--------------
LENGTH                                        =    13.00 Ft
HEIGHT FROM TOP OF FOOTING                    =    21.50 Ft
BACKWALL THICKNESS                            =     2.34 Ft
SEAT WIDTH                                    =     0.75 Ft
DISTANCE FROM FOOTING TOE TO SEAT FACE        =     5.00 Ft
DISTANCE FROM SEAT FACE TO C.L. OF BEARING    =     0.38 Ft
BACKWALL HEIGHT ABOVE SEAT LEVEL              =     1.00 Ft
WEIGHT OF EXTERIOR SHEAR KEYS                 =     0.00 K
UNIFORM WEIGHT OF BARRIER                     =     0.40 K/Ft
VERTICAL DISTANCE FROM SEAT TO FILLET         =     0.00 Ft
STEM THICKNESS                                =     3.09 Ft

WING WALL DATA:
---------------
NUMBER                                        =        1 WALLS
LENGTH                                        =     8.46 Ft
THICKNESS                                     =     2.25 Ft
END WALL DIMENSION                            =    21.50 Ft

FOOTING DATA:
-------------
LENGTH                                        =    13.00 Ft
WIDTH                                         =    13.83 Ft
DEPTH                                         =     3.50 Ft
```

```
*************
* PILE DATA *
*************************************************************************
```

| | | | |
|---|---|---|---|
| VERTICAL PILE CAPACITY - SERVICE | = | 70.00 | Tons |
| LATERAL PILE CAPACITY - SERVICE | = | 10.00 | K |
| LATERAL PILE CAPACITY - ULTIMATE | = | 40.00 | K |
| PILE DIAMETER | = | 12.00 | in |
| BATTER FACTOR | = | 1:3.00 | |

PILE LAYOUT DATA (Sorted) (Ft):

| ROW NO. | DISTANCE TO TOE | NO. OF PILES IN ROW | NO. OF PILES BATTERED |
|---|---|---|---|
| 1 | 1.50 | 4 | 4 |
| 2 | 6.58 | 4 | 0 |
| 3 | 10.83 | 3 | 0 |

| | | |
|---|---|---|
| TOTAL NUMBER OF PILES | = | 11 |
| DISTANCE FROM TOE TO C.G. OF PILES (CG) | = | 5.89 Ft |
| MOMENT OF INERTIA OF PILE GROUP (IX) | = | 152.20 Pile-Ft^2 |

```
***********************
* REBAR AND LOAD DATA *
*************************************************************************
```

| REBAR LOCATION | BAR SIZE | NO. OF BARS | COVER(in) |
|---|---|---|---|
| BACKWALL | 8 | 13 | 2.00 |
| STEM | 11 | 14 | 3.00 |
| FOOTING TOP | 7 | 13 | 3.00 |
| FOOTING BOTTOM (HEEL) | 11 | 10 | 3.00 |
| FOOTING BOTTOM (TOE ) | 11 | 26 | 3.00 |

SERVICE LOAD DATA (K ):

| | TOTAL DEAD LOAD | :- LIVE TOTAL H-LOAD | LOAD -: 1.15P+H P-LOAD | TOTAL BEARING PAD SHEAR | LATERAL EARTH PRESSURE | OTHER LATERAL LOAD | OTHER VERTICAL LOAD |
|---|---|---|---|---|---|---|---|
| LOADS | 5. | 0. | 0. | 0. | | 35. | 677. |
| FACTORS | 1.00 | 1.30 | 1.30 | 1.30 | 1.69 | 1.00 | 1.00 |

| | | |
|---|---|---|
| DISTANCE ABOVE SEAT TO OTHER LATERAL LOAD (Ft) = | -7.95 | |
| DISTANCE FROM HEEL TO OTHER VERTICAL LOAD (Ft) = | | 10.73 |
| CONSIDER OTHER LOADS IN LOAD GROUPS (Y/N) | 12345 | 12345 |
| | YYYYN | YNNYN |

```
*****************************************
* MOMENTS ABOUT TOE OF FOOTING (K,Ft,K/Ft) * (SERVICE)
*****************************************************************************
```

| LATERAL EARTH LOADS (BACKFILL TO TOP OF BACKWALL) | FORCE | ARM | MOMENT |
|---|---|---|---|
| EARTH PRESSURE (KA =0.300) | 120.9 | 8.33 | 1008. |
| EARTH PRESSURE (KA =0.225) | 90.7 | 8.33 | 756. |
| SURCHARGE PRESSURE (KA =0.300) | 19.3 | 12.50 | 242. |
| SURCHARGE PRESSURE (KA =0.225) | 14.5 | 12.50 | 181. |
| EARTHQUAKE | 0.0 | 0.00 | 0. |
| **SUPERSTRUCTURE LOADS** | | | |
| DEAD LOAD | 5.4 | 5.38 | -29. |
| H-LIVE LOAD | 0.0 | 5.38 | 0. |
| P-LIVE LOAD | 0.0 | 5.38 | 0. |
| BEARING PAD SHEAR | 0.0 | 24.00 | 0. |
| **OTHER LOADS** | | | |
| OTHER LATERAL LOAD | 34.7 | 16.05 | 557. |
| OTHER VERTICAL LOAD | 677.1 | 3.10 | -2099. |
| **ABUTMENT DEAD LOADS** | | | |
| BACKWALL | 4.6 | 6.92 | -32. |
| SEAT | 123.5 | 6.54 | -808. |
| EXTRA SEAT PORTION (HIGH CANTILEVER) | 0.0 | 8.09 | 0. |
| FOOTING | 94.4 | 6.92 | -653. |
| FOOTING KEY | 0.0 | 0.00 | 0. |
| WINGWALL TOP RECTANGULAR SECTION | 41.7 | 10.96 | -457. |
| WINGWALL BOT TRIANGULAR SECTION | 0.0 | 10.01 | 0. |
| WINGWALL EXTRA SECTION (HIGH CANTILEVER) | 0.0 | 8.09 | 0. |
| BARRIER | 3.2 | 9.80 | -32. |
| SIDE KEYS | 0.0 | 5.38 | 0. |
| **SOIL DEAD LOADS** | | | |
| SOIL ON HEEL (BACKFILL TO TOP OF BACKWALL) | 159.2 | 10.96 | -1745. |
| EXTRA SOIL ON HEEL (HIGH CANTILEVER) | 0.0 | 8.09 | 0. |
| SOIL ON TOE | 0.0 | 2.50 | 0. |
| SURCHARGE ON HEEL | 14.8 | 10.96 | -162. |

 287

Loads are Factored work
AD Need to check MOS.

```
*****************************************************
* SERVICE PILE LOADS (SER) FOR ABUTMENT DESIGN (K,Ft) *
*****************************************************
```

Note: Toe & Heel Piles Loads and Capacities are along the Vertical.

| GRP | CA | :-- VERTICAL --: TOTAL | ECCEN | :-- LATERAL --: TOTAL | CAP | :----- HEEL ----: MAX | CAP | :----- TOE -----: MAX | CAP | |
|---|---|---|---|---|---|---|---|---|---|---|
| IH | 1 | 1124. | 2.15 | 175. | 339. | 24. | 140. | 172.* | 133. | NG* |
| IH | 2 | 1124. | 1.87 | 140. | 327. | 34. | 140. | 163.* | 133. | NG* |
| IH | 3 | 1109. | 2.02 | 156. | 331. | 28. | 140. | 166.* | 133. | NG* |
| IH | 4 | 1109. | 1.80 | 125. | 321. | 36. | 140. | 158.* | 133. | NG* |
| II | 1 | 447. | 1.17 | 175. | 184. | 24. | 140. | 56. | 133. | OK |
| II | 2 | 447. | 0.47 | 140. | 172. | 34. | 140. | 47. | 133. | OK |
| III | 1 | 438. | 1.21 | 175. | 239. | 23. | 210. | 55. | 199. | OK |
| III | 2 | 438. | 0.50 | 140. | 227. | 33. | 210. | 46. | 199. | OK |
| IV | 1 | 1124. | 2.15 | 175. | 367. | 24. | 175. | 172.* | 166. | NG* |
| IV | 2 | 1124. | 1.87 | 140. | 354. | 34. | 175. | 163. | 166. | OK |
| IV | 3 | 1109. | 2.02 | 156. | 358. | 28. | 175. | 166. | 166. | OK |
| IV | 4 | 1109. | 1.80 | 125. | 349. | 36. | 175. | 158. | 166. | OK |
| V | 1 | 950. | 2.15 | 0. | 359. | 20. | 210. | 145. | 199. | OK |

SUMMARY OF CONTROLLING LOAD CASES
(RATIO = CAP / LOAD)

| GRP | CA | DESCRIPTION | LOAD | CAP | RATIO | |
|---|---|---|---|---|---|---|
| II | 1 | TOTAL LATERAL LOAD | 175. | 184. | 1.05 | OK |
| IH | 4 | MAX HEEL LOAD | 36. | 140. | 3.87 | OK |
| IH | 1 | MAX TOE LOAD | 172. | 133. | 0.77 | NG* |

Controlling Design Loading

| Grp | Ca | Description | Load |
|---|---|---|---|
| IH | 1 | Max Toe Load | 181. |

Note: Allowable Overstress = 1.00

Loading is Along the Batter.

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```
*****************************************************
* FACTORED PILE LOADS (FAC) FOR ABUTMENT DESIGN (K,Ft) *
*****************************************************************************
PHI = .75
Note: Toe & Heel Piles Loads and Capacities are along the Vertical.
```

| GRP | CA | :-- VERTICAL --: TOTAL | ECCEN | :-- LATERAL --: TOTAL | CAP | :----- HEEL ----: MAX | CAP | :----- TOE -----: MAX | CAP | |
|---|---|---|---|---|---|---|---|---|---|---|
| IH | 1 | 1124. | 2.91 | 272. | 527. | -4.* | 0. | 197. | 199. | NG* |
| IH | 2 | 1124. | 2.44 | 213. | 511. | 13. | 210. | 181. | 199. | OK |
| IH | 3 | 1109. | 2.65 | 239. | 516. | 5. | 210. | 186. | 199. | OK |
| IH | 4 | 1109. | 2.27 | 188. | 503. | 19. | 210. | 173. | 199. | OK |
| IP | 1 | 447. | 1.85 | 272. | 394. | 14. | 210. | 64. | 199. | OK |
| IP | 2 | 447. | 0.67 | 213. | 379. | 31. | 210. | 49. | 199. | OK |
| IP | 3 | 432. | 1.14 | 239. | 383. | 23. | 210. | 53. | 199. | OK |
| IP | 4 | 432. | 0.16 | 188. | 371. | 37. | 210. | 41. | 199. | OK |
| II | 1 | 447. | 3.10 | 272. | 411. | -4.* | 0. | 81. | 199. | NG* |
| II | 2 | 447. | 1.91 | 213. | 395. | 13. | 210. | 65. | 199. | OK |
| III | 1 | 438. | 3.18 | 272. | 410. | -5. | -84. | 80. | 199. | OK |
| III | 2 | 438. | 1.97 | 213. | 395. | 12. | 210. | 65. | 199. | OK |
| IV | 1 | 1124. | 2.91 | 272. | 527. | -4.* | 0. | 197. | 199. | NG* |
| IV | 2 | 1124. | 2.44 | 213. | 511. | 13. | 210. | 181. | 199. | OK |
| IV | 3 | 1109. | 2.65 | 239. | 516. | 5. | 210. | 186. | 199. | OK |
| IV | 4 | 1109. | 2.27 | 188. | 503. | 19. | 210. | 173. | 199. | OK |
| V | 1 | 950. | 2.15 | 0. | 475. | 20. | 210. | 145. | 199. | OK |

SUMMARY OF CONTROLLING LOAD CASES
(RATIO = CAP / LOAD)

| GRP | CA | DESCRIPTION | | | LOAD | CAP | RATIO | |
|---|---|---|---|---|---|---|---|---|
| IP | 1 | TOTAL | LATERAL | LOAD | 272. | 394. | 1.45 | OK |
| II | 1 | MAX | HEEL | LOAD | -4. | 0. | 0.00 | NG* |
| IV | 1 | MAX | TOE | LOAD | 197. | 199. | 1.01 | OK |

OK.
Actual Pile size is larger than input.

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```
*******************************************
* STEEL REQUIRED - LOAD FACTOR DESIGN (LFD) *
*******************************************************************************
```

BACKWALL, D = 25.5 (in)

| LOAD TYPE | LOADS GRP | CA | MOMENT (K-Ft) | NEUT AXIS (in) | STEEL STRESS (Ksi) | STEEL AREA (in^2) | BAR SIZE | BAR SPACE (in) | BARS REQ | ERR CODE | BAR IN | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| SER | | | 436. | 4.20 | 28.5 | 7.6 | 8 | 12.6 | 10 | | | |
| FAC | | | 436. | 0.57 | | 3.8 | 8 | 12.6 | 5 | | | |
| MIN1 | | | 923. | 1.21 | | 8.2 | 8 | 12.6 | 11 | | | |
| MIN2 | | | 580. | 0.76 | | 5.1 | 8 | 12.6 | 7 | | | |
| MIN3 | | | 889. | 1.17 | | 7.9 | 8 | 12.6 | 10 | | | |
| MAX | | | 7135. | 11.33 | | 76.6 | 8 | 12.6 | 97 | | | |
| CONTROL | | | | | | | | | | | | |
| MIN3 | | | 889. | 1.17 | | 7.9 | 8 | 16.8 | 10 | | 13 | OK |

STEM, D = 33.3 (in)

| LOAD TYPE | LOADS GRP | CA | MOMENT (K-Ft) | NEUT AXIS (in) | STEEL STRESS (Ksi) | STEEL AREA (in^2) | BAR SIZE | BAR SPACE (in) | BARS REQ | ERR CODE | BAR IN | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| SER | | | 1255. | 7.04 | 28.0 | 17.4 | 11 | 11.6 | 12 | | | |
| FAC | | | 1821. | 1.84 | | 12.5 | 11 | 11.6 | 8 | | | |
| MIN1 | | | 1609. | 1.62 | | 11.0 | 11 | 11.6 | 8 | | | |
| MIN2 | | | 2409. | 2.46 | | 16.6 | 11 | 11.6 | 11 | | | |
| MIN3 | | | 2266. | 2.31 | | 15.6 | 11 | 11.6 | 10 | | | |
| MAX | | | 12128. | 14.77 | | 99.9 | 11 | 11.6 | 65 | | | |
| CONTROL | | | | | | | | | | | | |
| SER | | | 1255. | 7.04 | 28.0 | 17.4 | 11 | 13.7 | 12 | | 14 | OK |

```
*******************************************
* STEEL REQUIRED - LOAD FACTOR DESIGN (LFD) *
*******************************************************************************
```

FOOT TOP, D = 38.5 (in)

| LOAD TYPE | LOADS GRP | CA | MOMENT (K-Ft) | NEUT AXIS (in) | STEEL STRESS (Ksi) | STEEL AREA (in^2) | BAR SIZE | BAR SPACE (in) | BARS REQ | ERR CODE | BAR IN |
|---|---|---|---|---|---|---|---|---|---|---|---|
| SER | III | 1 | -426. | 4.31 | 27.6 | 5.0 | 7 | 12.6 | 9 | | |
| FAC | III | 1 | -656. | 0.56 | | 3.8 | 7 | 12.6 | 7 | | |
| MIN1 | | | -2064. | 1.80 | | 12.2 | 7 | 12.6 | 21 | | |
| MIN2 | | | -873. | 0.75 | | 5.1 | 7 | 12.6 | 9 | | |
| MIN3 | | | -1029. | 0.89 | | 6.0 | 7 | 12.6 | 10 | | |
| MAX | | | -16245. | 17.09 | | 115.6 | 7 | 12.6 | 193 | | |
| ---- CONTROL ---- | | | | | | | | | | | |
| MIN3 | | | -1029. | 0.89 | | 6.0 | 7 | 16.8 | 10 | | 13 OK |

FOOT BOT (HEEL), D = 38.2 (in)

| LOAD TYPE | LOADS GRP | CA | MOMENT (K-Ft) | NEUT AXIS (in) | STEEL STRESS (Ksi) | STEEL AREA (in^2) | BAR SIZE | BAR SPACE (in) | BARS REQ | ERR CODE | BAR IN |
|---|---|---|---|---|---|---|---|---|---|---|---|
| SER | V | 1 | 52. | 1.66 | 24.0 | 1.6 | 11 | 78.0 | 1 | 3 | |
| FAC | V | 1 | 52. | 0.05 | | 1.6 | 11 | 78.0 | 1 | 3 | |
| MIN1 | | | 2064. | 1.81 | | 12.3 | 11 | 16.7 | 8 | | |
| MIN2 | | | 70. | 0.06 | | 1.6 | 11 | 78.0 | 1 | 3 | |
| MIN3 | | | 2612. | 2.31 | | 15.6 | 11 | 16.7 | 10 | | |
| MAX | | | 15980. | 16.95 | | 114.6 | 11 | 16.7 | 74 | | |
| ---- CONTROL ---- | | | | | | | | | | | |
| MIN3 | | | 2612. | 2.31 | | 15.6 | 11 | 16.7 | 10 | | 10 OK |

FOOT BOT (TOE), D = 38.2 (in)

| LOAD TYPE | LOADS GRP | CA | MOMENT (K-Ft) | NEUT AXIS (in) | STEEL STRESS (Ksi) | STEEL AREA (in^2) | BAR SIZE | BAR SPACE (in) | BARS REQ | ERR CODE | BAR IN |
|---|---|---|---|---|---|---|---|---|---|---|---|
| SER | IV | 1 | 2319. | 8.91 | 31.7 | 24.9 | 11 | 6.0 | 16 | | |
| FAC | IH | 1 | 2668. | 2.36 | | 15.9 | 11 | 6.0 | 11 | | |
| MIN1 | | | 2064. | 1.81 | | 12.3 | 11 | 6.0 | 8 | | |
| MIN2 | | | 3525. | 3.14 | | 21.3 | 11 | 6.0 | 14 | | |
| MIN3 | | | 2612. | 2.31 | | 15.6 | 11 | 6.0 | 10 | | |
| MAX | | | 15980. | 16.95 | | 114.6 | 11 | 6.0 | 74 | | |
| ---- CONTROL ---- | | | | | | | | | | | |
| SER | IV | 1 | 2319. | 8.91 | 31.7 | 24.9 | 11 | 10.0 | 16 | | 26 OK |

```
*****************************************************
* SUMMARY - STEEL REQUIRED - LOAD FACTOR DESIGN (LFD) *
*****************************************************************************

KEY TO TYPES OF LOADS:

SER  = SERVICE LOADS
FAC  = FACTORED LOADS     PHI = 0.90
MIN1 = MINIMUM REINFORCEMENT (ART 8.17.1.1)
       BASED ON 1.2 X THE CRACKING MOMENT
MIN2 = WAIVER OF MINIMUM REINFORCEMENT (ART 8.17.1.2)
       BASED ON STEEL AREA ONE THIRD GREATER THAN REQUIRED BY ANALYSIS
MIN3 = MINIMUM REINFORCEMENT (TEMPERATURE) (ART 8.20)
       BASED ON 18" MAXIMUM BAR SPACING
MAX  = MAXIMUM REINFORCEMENT (ART 8.16.3.1)
       BASED ON .75 X BALANCED STEEL RATIO


KEY TO REBAR LOACATIONS (BAR-LOC)
---------------------------------
1 = BACKWALL
2 = STEM
3 = FOOTING TOP
4 = FOOTING BOTTOM (HEEL)
5 = FOOTING BOTTOM (TOE)
```

| BAR LOC | LOAD TYPE | LOADS GRP | CA | MOMENT (K-Ft) | NEUT AXIS (in) | EFF DEPTH (in) | STEEL AREA (in^2) | BAR SIZE | BAR SPC (in) | BARS REQ | ERR CODE | BARS IN | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | MIN3 | | | 889. | 1.17 | 25.5 | 7.9 | 8 | 16.8 | 10 | | 13 | OK |
| 2 | SER | | | 1255. | 7.04 | 33.3 | 17.4 | 11 | 13.7 | 12 | | 14 | OK |
| 3 | MIN3 | | | -1029. | 0.89 | 38.5 | 6.0 | 7 | 16.8 | 10 | | 13 | OK |
| 4 | MIN3 | | | 2612. | 2.31 | 38.2 | 15.6 | 11 | 16.7 | 10 | | 10 | OK |
| 5 | SER | IV | 1 | 2319. | 8.91 | 38.2 | 24.9 | 11 | 10.0 | 16 | | 26 | OK |

```
ERROR CODES:

1 = BAR SPACING LESS THAN AASHTO MINIMUM. (CODE 8.21)
2 = BAR SPACING LESS THAN PREFERRED CALTRANS MINIMUM. (BR. DES. DET.)
3 = BAR SPACING MORE THAN THE AASHTO MAXIMUM ( 18. in). (CODE 8.20)
```

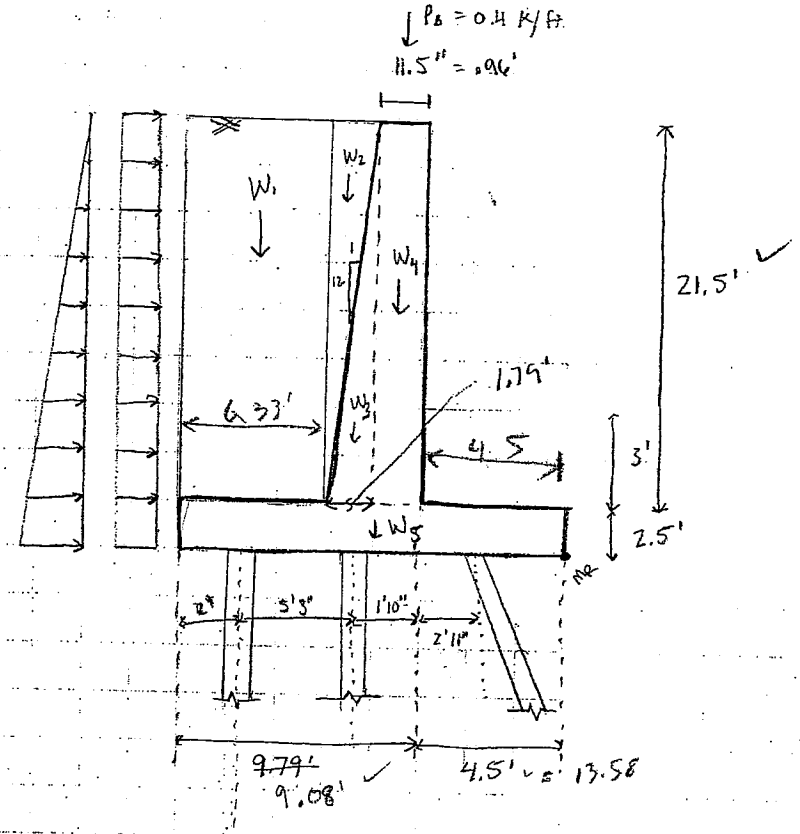
```
*****************************************************************
* STEM MOMENT ENVELOPE AND REQUIRED STEEL AREAS (BAR CUT-OFFS) *
*****************************************************************
(APPLIED MOMENT - GROUP IV CASE 1)
(BACKFILL, SURCHARGE, BEARING PAD SHEAR AND OTHER LATERAL LOAD)
```

| DIST FROM FOOTING TOP (Ft) | APPLIED MOMENT (K-Ft) | NUET AXIS (in) | STEEL AREA (in^2) | BAR SIZE | BAR SPACE (in) | BARS REQ | ERR CODE |
|---|---|---|---|---|---|---|---|
| 0.00 | 1821. | 1.84 | 12.5 | 11 | 21.5 | 8 | 3 |

L. Sanchez-Manzo 07- 202101 1 9

1/5/10 RW Check @ Abut 5 (South 07-LA-710-17.30

Rev. 1 - 3/17/10 293




Assumption/Given:

$\gamma_s$ = 0.120 kcf

$K_a$ = 0.30

$\gamma_c$ = 0.150 kcf

Concrete Barrier = 0.4 K/ft

$f_s$ = 24,000 psi

per BDS 8.15.2.2

$f_y$ = 60,000 psi

$f'_c$ = 3250 psi

| Force (Kips/LF) | Arm (ft) | Moment k-ft/LF |
|---|---|---|
| surcharge $P_s = 2 \times 21.5 \times 0.12 = 5.16$ | $13.58 - 6.33/2 = 10.415$ | $-53.75$ k-ft/LF |
| $P_B = 0.4$ k/ft | $4.5' + (11.5''/2) \times 1'/12'' = 4.98'$ | $-1.99$ k-ft/LF |
| $W_1 = (6.33') \times 21.5' \times 0.120$ = ~~18.16~~ 16.33 K/LF | $4.5' + (0.96' + 1.79') + (6.33/2)$ = ~~10.77~~ 10.42' | ~~-195.58~~ $-170.08$ k-ft/LF |
| $W_2 = (1.79 \times 21.5)/2 \times 0.120$ = 2.31 K/LF | $4.5' + (0.96') + (1.79'(2/3))$ = 6.65' | $-15.37$ k-ft/LF |
| $W_3 = (1.79') \times 21.5'/2 \times 0.15$ = 2.89 K/LF | $4.5' + (0.96) + (1.79'(1/3))$ = 6.06' | $-17.51$ k-ft/LF |

1/5/10

RW Check @ Abut 5

Rev. 1 3/17/10

| Force (kips/LF) | Arm (ft) | Moment (k-ft/LF) |
|---|---|---|
| $W_4 = (11.5''/12) * 21.5' * 0.15$ = 3.09 kips/LF ✓ | $4.5 + [(11.5/12)/2]$ = 4.98' ✓ | -15.39 k-ft/LF ✓ |
| $W_5 = (\cancel{9.79'}\ 9.08' + 4.5') * 2.5' * 0.15$ = ~~5.36~~ 5.09 kips/LF ✓ | $(\cancel{9.79}\ 9.08 + 4.5)/2$ = ~~7.15~~ 6.79' ✓ | ~~38.31~~ k-ft/LF -34.56 ✓ |
| $\Sigma F$ = ~~32.21~~ ~~30.11~~ 35.27 k/LF | | $\Sigma M$ = ~~-284.15~~ -254.9 k-ft/LF |
| C.G. $\frac{\Sigma M}{\Sigma F} = \frac{\cancel{284.15}\ 254.9}{\cancel{32.21}\ \cancel{30.11}\ 35.27} = \cancel{8.82'}\ \cancel{8.47'}\ 8.75'$ from toe | | $M_R$ = ~~-284.15~~ -254.9 ~~309~~ k-ft/LF |

Calculating Mo

| Force (kips/LF) | Arm (ft) | Moment (k-ft/LF) |
|---|---|---|
| Lateral Load = $P_1$<br>$P_1 = \frac{1}{2} K_a \gamma_s H^2$<br>$= \frac{1}{2}(0.3)(0.12)(21.5+2.5)^2$<br>= 10.37 k/LF ✓ | $H/3 = (21.5+2.5)/3$<br>= 8.0 ft ✓ | 82.96 k-ft/LF ✓ |
| Surcharge = $P_2 = 2 K_a \gamma_s H$<br>$= 2(0.3)(0.12)(21.5+2.5)$<br>= 1.73 k/LF ✓ | $H/2 = (21.5+2.5)/2$<br>= 12.0 ft ✓ | 20.76 k-ft/LF ✓ |
| | | $\Sigma M_o$ = 103.72 k-ft/LF ✓ |

$M_R > M_o$ ∴ ok. ✓

Need Factor of safety = 1.5 per ~~BDS~~ 5.2.2.3

$M_R / M_o = 2.74 > 1.5$ ∴ ~~ok~~ ok.

Converting the loading to place Moment @ C.G. of pile group.

$M = 103.72 - (\cancel{32.21}\ \cancel{30.11}\ 35.27)(\cancel{8.82}\ \cancel{8.47}\ 8.75 - 6.5) = \cancel{28.99}$ k-ft/LF

= 44.4 k-ft/LF ✗

$M = 35.27 \times (8.75 - 6.5) = 79.36$ k-ft/LF

O.T. moment $= 103.72 - 79.36 = 24.36$ k-ft/LF

Calculating C.G. of pile group and I pile group:

$$C.G. = \frac{(1'7") + (6'4") + (11'7")}{3}$$

$$= \frac{1.583' + 6.33' + 11.583'}{3} = 6.5' \checkmark$$

$$I_{pile\ group} = [1(6.5-1.58)^2 + 1(6.5-6.33)^2 + 1(6.5-11.583)^2]$$

$$= 24.21 + 0.029 + 25.84$$

$$= 50.08\ ft^2 \checkmark$$

$$R = \frac{\Sigma V}{N} + \frac{Mc}{I}$$

$$R_{row\#1} = \frac{\cancel{30.11}\ 35.27 \times 4'}{\cancel{32.21}\ 3} + \frac{\cancel{44.9}\ 24.36 \times 4}{\cancel{28.99}\ (6.5' - 1.58')} = \frac{}{50.08}$$

$$\cancel{10.74}\ 47.02 + \cancel{2.95}\ 4.86 = \cancel{13.59}\ 9.57\ K/LF \quad 57\ kips$$

$$R_{row\#2} = \frac{30.11}{\cancel{32.21}\ 3} + \frac{\cancel{28.99}\ (6.5' - 6.33')}{50.08}$$

$$10.04 + 44.4 = \cancel{14.4}\ K/LF$$

$$\cancel{10.74}\ 10.04 + \cancel{0.098}\ 0.151 = \cancel{10.84}\ K/LF$$

$$R_{row\#3} = \frac{\cancel{20.11}\ 35.27 \times 4}{\cancel{32.21}\ 3} + \frac{\cancel{44.9}\ 24.36}{\cancel{28.99}\ (6.5' - 11.583')} \times 4 = \frac{}{50.08}$$

$$= 10.19\ K/LF$$

$$\cancel{10.74} + \cancel{2.94} = \cancel{7.80}\ K/LF$$

$$\cancel{10.04}\ 47.02 - 4.51\ 9.9 = \cancel{5.53}\ K/LF \quad 37\ kips$$

Pile Reaction

$R_{row\#1} = 14.4\ K/LF \times 3'4" = 48K$

$R_{row\#2} = 10.19\ K/LF \times 3'4" = 33.97\ K$

$R_{row\#3} = 5.53\ K/LF \times 3'4" = 18.43\ K$

OK

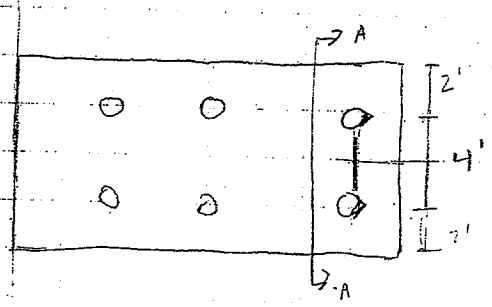
RW Check @ Abut 5
Rev. 1 - 3-17-10

07-LA-710-17.30

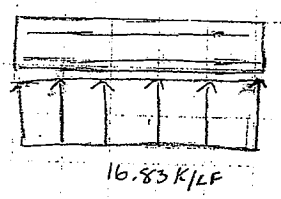
Longitudinal Reinforcement in Footing

Plan View

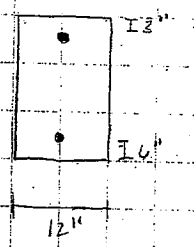
$R_i$ = 13.59 K/LF




A-A




$\therefore$ Moment for Top $= \frac{wL^2}{8} = \frac{\cancel{13.59}\ 14.4(4)^2}{8'} = \cancel{27.18}\ 28.8$ k-ft $/0.9 = \cancel{30.2}$ k-ft
$= 32$ k-ft




$b = 12"$ $\quad d_T = 2.5' \times 12 in/ft - 3" = 27"$
$d_B = 2.5' \times 12 in/ft - 6" = 24"$

Using #6 @ 12" $\quad A_s = 0.4$

Top $\quad a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.44(60{,}000)}{0.85(3250)(12")} = 0.796$ in

$c = a/\beta_1$ where $\beta_1 = 0.85$ for $f'_c < 4{,}000$ psi

$c = 0.796/0.85 = 0.937$

$\epsilon_t = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{27-0.937}{0.937}\right) = 0.0834 > 0.005$ in/in $\therefore$ ok

$M_n = A_s f_y \left(d - \frac{a}{2}\right) = 0.44(60{,}000)\left(27" - \frac{0.796}{2}\right) = 702{,}293$ lb-in

$\frac{703{,}309 \text{ lb-in}}{} \cdot \frac{1 ft}{12 in} \cdot \frac{1 kip}{1000 lb} = 58.52$ k-ft

Capacity $M_n = 58.52$ k-ft $>$ Demand $M = \cancel{30.20}\ 32$ k-ft $\therefore$ OK

Try #5 @ 12" $\quad A_s = 0.31$

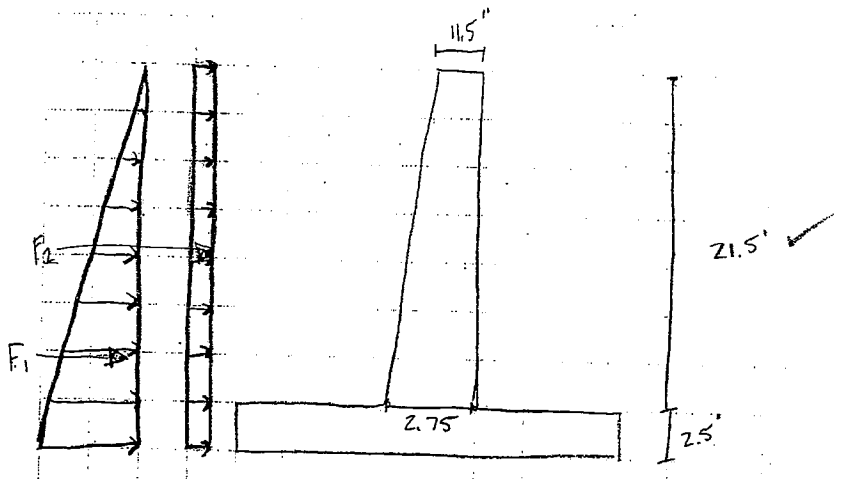
$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(0.31)(60{,}000)}{0.85(3250)(12")} = 0.561 \qquad c = a/\beta_1 = 0.561/0.85 = 0.660$

$\epsilon_t = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{27-0.66}{0.66}\right) = 0.120 > 0.005$ in/in $\therefore$ OK

$M_n = 0.31(60{,}000)(27" - 0.561/2) = 496{,}983$ lb-in $= 41.42$ k-ft $> \cancel{30.20}\ 32$ k-ft

Use #5 @ 12"

Stem Design at Top of FTG:




Vertical Stem Reinforcement @ top of footing => H = 21.5'

$F_1 = \frac{1}{2}\gamma K_a H^2 = \frac{1}{2}(0.120\,ksf)(0.3)(21.5^2) = 8.32$ Kips/LF

$F_2 = 2\gamma K_a * H = 2'(0.120\,ksf)(0.3)(21.5) = 1.55$ K/LF

$\circlearrowleft M = 8.32(21.5/3) + 1.55(21.5/2) = 59.63 + 16.66 = 76.29$ k-ft/LF

$V = 8.32 + 1.55 = 9.87$ K/LF

Try #11 @ 12" $d = (2.75' * 12''/ft) - 2'' - (1.410/2) = 30.3''$ $A_s = 1.56\,in^2$

$jd = d - z$ $n = 8.48$ $d = 30.3''$ $d' = 2'' + (1.41 * 0.5) = 2.71''$

$\rho = \frac{A_s}{bd} = 1.56/(12'' * 30.3'') = 0.004$ $\rho' = 0.31/(12'' * 30.3) = 0.0009$

$$K = \sqrt{2\rho n + (\rho n)^2} - \rho n = \sqrt{2(0.004)(8.48) + (0.004 * 8.48)^2} - (0.004 * 8.48)$$

$$= \sqrt{0.068 + 0.0012} - 0.034 = 0.23$$

$$z = \frac{\frac{1}{3}K^3 d + 2\rho' n d'(K - \frac{d'}{d})}{K^2 + 2\rho' n (K - d'/d)} =$$

$$= \frac{\frac{1}{3}(0.23)^3(30.3'') + 2(0.0009)(8.48)(2.71)(0.23 - 2.71/30.3)}{(0.23)^2 + 2(0.0009)(8.48)(0.23 - 2.71/30.3)}$$

$$= \frac{0.123 + 0.006}{0.053 + 0.0022} = 2.34''$$

$jd = d - z = 30.3'' - 2.34'' = 27.96''$

$f_s = \frac{M}{A_s jd} \Rightarrow A_s = \frac{M}{f_s jd} = \frac{76.29 * 12''/ft}{(24)(27.96'')} = 1.36\,in^2/LF$

$v = \frac{V}{b_o jd} = \frac{9.87\,K/LF * 4ft}{(12'')(27.96'')} = 0.118 < 0.54\,ksi$ OK

Use #11 @ 12"

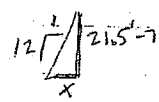
Vertical Stem Reinforcement @ 7 ft above footing

$P_1 = 1/2\,\gamma K_a H^2 = 1/2(0.120\,ksf)(0.3)(21.5'-7')^2 = 3.78$ K/LF

$P_2 = 2\gamma K_a H = 2'(0.120\,ksf)(0.3)(21.5-7) = 1.04$ K/LF

$\circlearrowleft M = 3.78[(21.5-7)/3] + 1.04[(21.5-7)/2] = 18.27 + 7.54 = 25.81$ K-ft/LF

$V = 4.82$ K/LF




$\frac{1}{12} = \frac{x}{21.5-7'} = x = 1.21'$ or $14.52''$ $d_t = 26.02'$

Assume #9 @ 12" $A_s = 1.00$

$d = (26.02'') - 2'' - (1.128 + 0.5) = 11.95''$ $d' = 2.56''$

$\rho = A_s/bd = 1.00/(12''*11.95'') = 0.007$ $\rho' = 0.31/(12''*11.95'') = 0.0022$

$K = \sqrt{2\rho n + (\rho n)^2} - \rho n = \sqrt{2(0.007)(8.48) + (0.007*8.48)^2} - (0.007*8.48)$

$= \sqrt{0.119 + 0.0035} - 0.059 = .291$

$z = \frac{1/3(0.291)^3(11.95) + 2(0.0022)(8.48)(2.56)(0.291 - 2.56/11.95)}{(0.291)^2 + 2(0.0022)(8.48)(0.291 - 2.56/11.95)} = 0.077$

$= \frac{0.098 + 0.11}{0.085 + 0.0029} = \cancel{2.37''}\ 1.19$

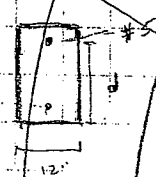
$jd = d - z = \cancel{11.95'' - 2.37''} = \cancel{9.58''}\ \cancel{10.53}\ 21.69''$

$A_s = M/f_s jd = \frac{25.81\,k\text{-}ft/LF * 12\,in/ft}{(24)(21.69'')} = 0.59\,in^2/LF$

use #7 @ 12"

$v = V/bjd = 4.82*4ft/(12'')(9.58) = 0.17 < 0.54\,ksi \therefore$ ok

Longitudinal Reinforcement - Max tension @ bottom of stem. L = 8ft




$b = 12''$ $d = (1.85*12\,in/1ft) - 2'' - (1.410 + 1.128) - 0.5(0.625)$

$= 17.19$

$w = (2'*K_a*\gamma*1) + (K_a\gamma*21.5*1'$

$= 0.072$ K/LF $+ 0.774$ K/LF $= 0.846$ K/LF

$M_{Bottom} = \frac{wL^2}{2} = \frac{(0.846\,K/LF)(8ft)^2}{2} = 27.07\,K\text{-}ft/0.9$

$= 30.08\,K\text{-}ft$

$M_{Top} = \frac{wL^2}{8} = \frac{(0.846)(8ft^2)}{8} = 6.77\,K\text{-}ft/0.9$

$= 7.52\,K\text{-}ft$

Checking for #5 @ min $A_s = 0.31\,in^2$

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.31(60{,}000)}{0.85(3600)(12'')} = 0.507''$

$c = a/\beta_1 = 0.507/0.85 = 0.596''$

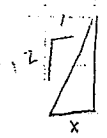
Vertical Stem @ 14 ft above footing

$P_1 = \frac{1}{2}\gamma K_a H^2 = \frac{1}{2}(0.120\ ksf)(0.3)(21.5' - 14')^2 = 1.01$

$P_2 = 2\gamma K_a H = 2'(0.120\ ksf)(0.3)(21.5 - 14) = 0.54$

$M_m = 1.01\ [(21.5' - 14')/3] + 0.54\ [(21.5 - 14)/2] = 2.53 + 2.03 = 4.56$ k-ft/LF

$V = 1.01 + 0.54 = 1.55$ k




$\frac{1}{12} = \frac{x}{7.5} = 0.625'$ or $7.5''$ $\quad dt = 7.5'' + 11.5 = 19''$

Assume #6 @ 12" $A_s = 0.44\ in^2$

$d = (7.5) - 2'' - (0.75/2) = 5.13$ $\quad d = 2 + 0.75/2 = 2.38''$

$\rho = A_s/bd = 0.44/(12''(5.13)) = 0.0072$ $\quad \rho' = 0.31/(12'' * 5.13) = 0.005$

$k = \sqrt{2\rho n + (\rho n)^2} - \rho n = \sqrt{2(0.0072)(8.48) + (0.0072 * 8.48)^2} - 0.0072 * 8.48$

$= \sqrt{0.122 + 0.0037} - 0.061 = 0.294$

$z = \frac{1/3(0.294)^3(5.13) + 2(0.005)(8.48)(2.38)(0.294 - 2.38/5.13)}{(0.294)^2 + 2(0.005)(8.48)(0.294 - 2.38/5.13)}$ $\quad -0.17$

$z = \frac{0.043 + 0.034}{0.086 + 0.014} = 0.77''$

$jd = d - z = 5.13 - 0.77'' = \cancel{4.36''}\ 15.59''$

$A_s = \frac{M}{f_s jd} = \frac{4.56\ k\text{-}ft/LF * 12\ in/ft}{(24)(15.59)} = 0.15\ in^2/LF$

use #4 @ 12"

$v = V/bjd = (1.55\ k)/[(12'')(4.36)] = 0.12 < 0.54\ ksi \therefore$ OK

Longitudinal Reinforcement

From BDS 4.4.11.5.4 $\quad A_s \geq 0.005 A_g$

$A_{s\,min} = \frac{[11.5'' + 33'']}{2} * 0.005 = 0.11\ in^2/LF$

Per 5.6.5.5 need 0.20 in² per foot. so use #4 @ 12 in.

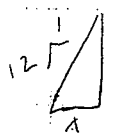
Vertical Stem Reinforcement @ 10 ft above footing.

$F_1 = \frac{1}{2}\gamma K_a H^2 = \frac{1}{2}(0.120\ ksf)(0.3)(21.5-10)^2 = 2.38$ K/LF

$F_2 = 2\gamma K_a H = 2'(0.120)(0.3)(21.5-10) = 0.83$ K/LF

$@M = 2.38((21.5-10)/3) + 0.83((21.5-10)/2) = 9.12 + 4.77 = 13.89$ K-ft/LF

Try #11.




$x = 0.95'$ or $11.5''$ $\quad dt = 11.5 + 11.5 = 23''$

$jd = 18.34''$ from excel calc sheet

$$As = \frac{13.89 \times 12}{(24)(18.34'')} = 0.38\ in^2/LF \quad \text{use \#6 @ 12''}$$

@ 11 ft above

$F_1 = \frac{1}{2}\gamma K_a H^2 = \frac{1}{2}(0.120)(0.3)(21.5-11)^2 = 1.98$ K/LF

$F_2 = 2(0.120)(0.3)(21.5-11) = 0.76$ K/LF

$@M = 1.98((21.5-11)/3) + 0.76((21.5-11)/2) = 6.93 + 3.99 = 10.92$ K-ft/LF

Try # 6 $\quad x = 10.5''$ $\quad d_t = 10.5 + 11.5 = 22''$ $\quad jd = 18.48$

$$As = \frac{10.92 \times 12}{(24)(18.48)} = 0.30\ in^2/LF \quad \text{use \#5 @ 12''}$$

@ 12 ft above. Try # 6 $\quad x = 9.5$ $\quad dt = 21$

$F_1 = 0.5(0.120)(0.3)(21.5-12)^2 = 1.62$

$F_2 = 2(0.120)(0.3)(21.5-12) = 0.68$

$@M = 1.62(9.5/3) + 0.68(9.5/2) = 5.13 + 3.23 = 8.36$

$jd = 17.52$ $\quad As = \frac{8.36 \times 12}{(24)(17.52)} = 0.24\ in^2/LF$ $\quad$ use #5 @ 12

@ 13 ft above Try # 5 $\quad x = 8.5''$ $\quad dt = 20$

$F_1 = 0.5(0.120)(0.3)(21.5-13)^2 = 1.3$ $\quad F_2 = 2(0.120)(0.3)(21.5-13) = 0.61$

$@M = 1.3(8.5/3) + 0.61(8.5/2) = 3.68 + 2.59 = 6.27$

$jd = 16.78''$ $\quad As = \frac{6.27(12)}{24(16.78)} = 0.19\ in^2/LF$ $\quad$ Use #4 @ 12''

@ Bott = As = 1.36 in²/LF

@ 3.5 ft As =

@ 7 ft a.f. As = 0.59 in² - #11 @ 4'

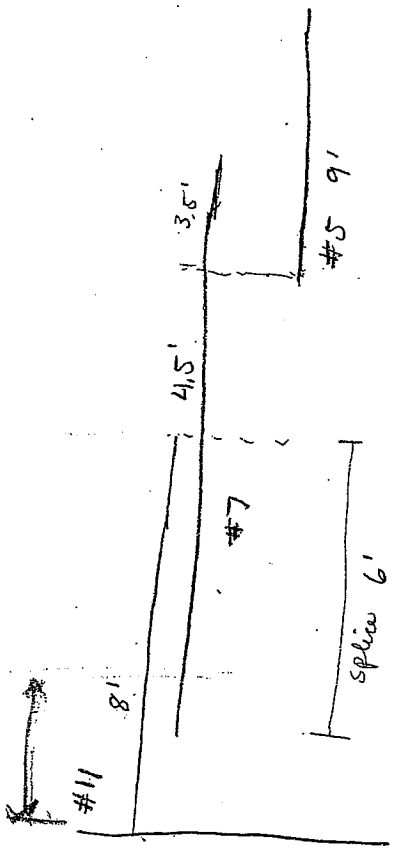
@ 10 ft a.f. As = 0.38 in²

@ 11 ft a.f. As = 0.30 in² - #5 5 ft

@ 12 ft a.f. As = 0.24 in²

@ 13 ft a.f. As = 0.19 in²

@ 14 ft a.f. As = 0.15 in²




ld of #11 = 5'' As = 1.56

ld of # 9 = 3.17' As = 1.00

ld of # 7 = 2.08' As = 0.60

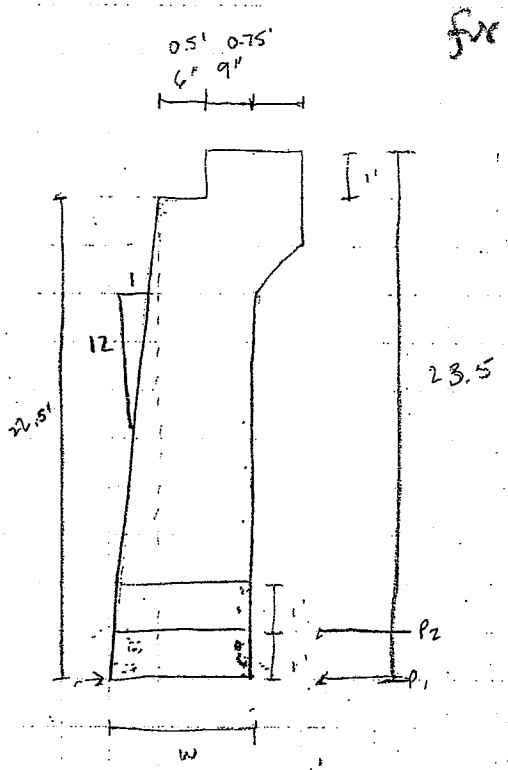
ld of #5 = 1.5'

B/w #11 & #9 Lap splice need 1.128*60 times /12 = 5.64'

B/w #9 & #7 Lap splice need 0.875*60/12 = 4.38'

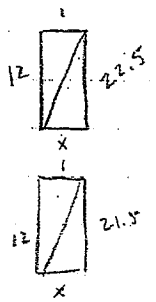
B/w #7 & #5 Lap splice 3.28'

# for Cantilever stem section




$f'c = 3600$ psi  $fy = 60{,}000$ psi

$L = 5.43'$

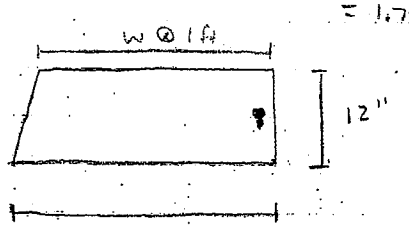



$\frac{1}{12} = \frac{x}{22.5'} \Rightarrow x = 1.875'$

$x = 1.792'$

w @ 0 ft from bottom $= 1.875' + 0.5' + 0.75' = 3.125'$

w @ 1 ft from bottom $= 1.792' + 0.5' + 0.75' = 3.042'$




Taking average $(3.125' + 3.042')/2 = 3.084'$ OR 37"

Pressure 1 $= \frac{1}{2}\gamma H^2 K_a = \frac{1}{2}(0.120)(22.5^2)(0.3) = 9.11$ K/LF

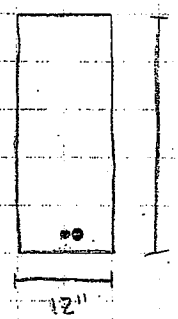
Pressure 2 $= \frac{1}{2}\gamma H^2 K_a = \frac{1}{2}(0.120)(21.5)^2(0.3) = 8.32$ K/LF

surcharge $= 2' \times \gamma \times K_a \times H = 0.072$ K/LF

Force $= ((9.11 + 8.32)$ K/LF $+ 0.072) \times 1.5 \times 5.43 = 71.53$

Applied Force @ end: $71.53$ K $(5.43'/2) = 194.20$ K-ft

Moment @ end $\equiv 194.20$ K-ft or $2330.4$ K-in




$b = 12"$  $d = 37" - 3"$ cover $= 34"$

Using 1 #11 w/ $A_s = 1.56$ in$^2$

& 1 #5 w/ $A_s = 0.31$ in$^2$

Total $A_s = 1.87$ in$^2$

$\beta_1 = 0.85$ for $f'c < 4{,}000$ psi

$$a = \frac{A_s \times f_y}{0.85 f_c \times b} = \frac{(1.87 \text{ in}^2)(60{,}000 \text{ psi})}{0.85(3600 \text{ psi})(12")} = 3.06"$$

$$c = a/\beta_1 = 3.06/0.85 = 3.60"$$

$$\epsilon_t = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{34 - 3.60}{3.60}\right) = 0.025$$

$0.025 > 0.005$ in/in

$\therefore$ OK section tension-controlled

$$M_n = A_s f_y \left(d - \frac{a}{2}\right) = 1.87(60{,}000)\left(34" - \frac{3.06}{2}\right) = 3{,}643{,}134 \text{ lb-in} = 3{,}643 \text{ K-in}$$

$$M_n = \frac{2{,}330.4}{0.9} = 2589.36 \text{ K-in} < 3{,}643 \text{ K-in}$$

$\therefore$ OK

Calcs By: L.Sanchez-Manzo



LA River Widening- Abutment 5 South Over Existing Footing

Given: ka= 0.3 Soil Unit Weight (kcf) = 0.12 L (ft)= 5.43

H of backfill(ft)= 22.5

| Equations | | | 2'*Ka*0.120*1' | 0.5*Ka*(H^2)*0.120 | 1.5*W | | Avg.P*L | (Avg.P*L*(L/2))/0.9 |
|---|---|---|---|---|---|---|---|---|
| | Distance from Bottom of Abutment (Ft) | Height- (ft) | Force from Surcharge (k/ft) | W=Force Applied at Location H (kips/ft) | Total W=Wt= Surcharge + W (kips/in) | Average Prusure (kip/in) | Force (kips) | Moment where Existing Abutment and Cantilever Abutment Connect |
| | 0 | 22.5 | 0.072 | 9.1125 | 13.77675 | | | |
| | 1 | 21.5 | 0.072 | 8.3205 | 12.58875 | 13.18275 | 858.98799 | 2591.280437 |
| | 2 | 20.5 | 0.072 | 7.5645 | 11.45475 | | | |
| | 3 | 19.5 | 0.072 | 6.8445 | 10.37475 | 10.91475 | 711.20511 | 2145.468749 |
| | 4 | 18.5 | 0.072 | 6.1605 | 9.34875 | | | |
| | 5 | 17.5 | 0.072 | 5.5125 | 8.37675 | 8.86275 | 577.49679 | 1742.115317 |
| | 6 | 16.5 | 0.072 | 4.9005 | 7.45875 | | | |
| | 7 | 15.5 | 0.072 | 4.3245 | 6.59475 | 7.02675 | 457.86303 | 1381.220141 |
| | 8 | 14.5 | 0.072 | 3.7845 | 5.78475 | | | |
| | 9 | 13.5 | 0.072 | 3.2805 | 5.02875 | 5.40675 | 352.30383 | 1062.783221 |
| | 10 | 12.5 | 0.072 | 2.8125 | 4.32675 | | | |
| | 11 | 11.5 | 0.072 | 2.3805 | 3.67875 | 4.00275 | 260.81919 | 786.8045565 |
| | 12 | 10.5 | 0.072 | 1.9845 | 3.08475 | | | |
| | 13 | 9.5 | 0.072 | 1.6245 | 2.54475 | 2.81475 | 183.40911 | 553.2841485 |
| | 14 | 8.5 | 0.072 | 1.3005 | 2.05875 | | | |
| | 15 | 7.5 | 0.072 | 1.0125 | 1.62675 | 1.84275 | 120.07359 | 362.2219965 |
| | 16 | 6.5 | 0.072 | 0.7605 | 1.24875 | | | |
| | 17 | 5.5 | 0.072 | 0.5445 | 0.92475 | 1.08675 | 70.81263 | 213.6181005 |
| | 18 | 4.5 | 0.072 | 0.3645 | 0.65475 | | | |
| | 19 | 3.5 | 0.072 | 0.2205 | 0.43875 | 0.54675 | 35.62623 | 107.4724605 |
| | 20 | 2.5 | 0.072 | 0.1125 | 0.27675 | | | |
| | 21 | 1.5 | 0.072 | 0.0405 | 0.16875 | 0.22275 | 14.51439 | 43.7850765 |
| | 22 | 0.5 | 0.072 | 0.0045 | 0.11475 | | | |
| | | | | | | | | |

Note: W multiplied by load factor of 1.5 from AASHTO LRFD Bridge Design Specifications.

Abutment 1 North Over Existing Footing

L (ft)= 3.2
H of backfill(ft)= 16.167

| Equations | | | 2'*Ka*0.120*1' | 0.5*Ka*(H^2)*0.120 | 1.5*W | | Avg.P*L | (Avg.P*L*(L/2))/0.9 |
|---|---|---|---|---|---|---|---|---|
| | Distance from Bottom of Abutment (Ft) | Height- (ft) | Force from Surcharge (k/ft) | W=Force Applied at Location H (kips/ft) | Total W=Wt= Surcharge + W (kips/in) | Average Prusure (kip/in) | Force (kips) | Moment where Existing Abutment and Cantilever Abutment Connect |
| | 0 | 16.167 | 0.072 | 4.704694002 | 7.165041003 | | | |
| | 1 | 15.167 | 0.072 | 4.140682002 | 6.319023003 | 6.742032003 | 258.8940289 | 460.2560514 |
| | 2 | 14.167 | 0.072 | 3.612670002 | 5.527005003 | | | |
| | 3 | 13.167 | 0.072 | 3.120658002 | 4.788987003 | 5.157996003 | 198.0670465 | 352.1191938 |
| | 4 | 12.167 | 0.072 | 2.664646002 | 4.104969003 | | | |
| | 5 | 11.167 | 0.072 | 2.244634002 | 3.474951003 | 3.789960003 | 145.5344641 | 258.7279362 |
| | 6 | 10.167 | 0.072 | 1.860622002 | 2.898933003 | | | |
| | 7 | 9.167 | 0.072 | 1.512610002 | 2.376915003 | 2.637924003 | 101.2962817 | 180.0822786 |
| | 8 | 8.167 | 0.072 | 1.200598002 | 1.908897003 | | | |
| | 9 | 7.167 | 0.072 | 0.924586002 | 1.494879003 | 1.701888003 | 65.35249932 | 116.182221 |
| | 10 | 6.167 | 0.072 | 0.684574002 | 1.134861003 | | | |
| | 11 | 5.167 | 0.072 | 0.480562002 | 0.828843003 | 0.981852003 | 37.70311692 | 67.0277634 |
| | 12 | 4.167 | 0.072 | 0.312550002 | 0.576825003 | | | |
| | 13 | 3.167 | 0.072 | 0.180538002 | 0.378807003 | 0.477816003 | 18.34813452 | 32.6189058 |
| | 14 | 2.167 | 0.072 | 0.084526002 | 0.234789003 | | | |
| | 15 | 1.167 | 0.072 | 0.024514002 | 0.144771003 | 0.189780003 | 7.287552115 | 12.9556482 |
| | 16 | 0.167 | 0.072 | 0.000502002 | 0.108753003 | | | |
| | | | | | | | | |

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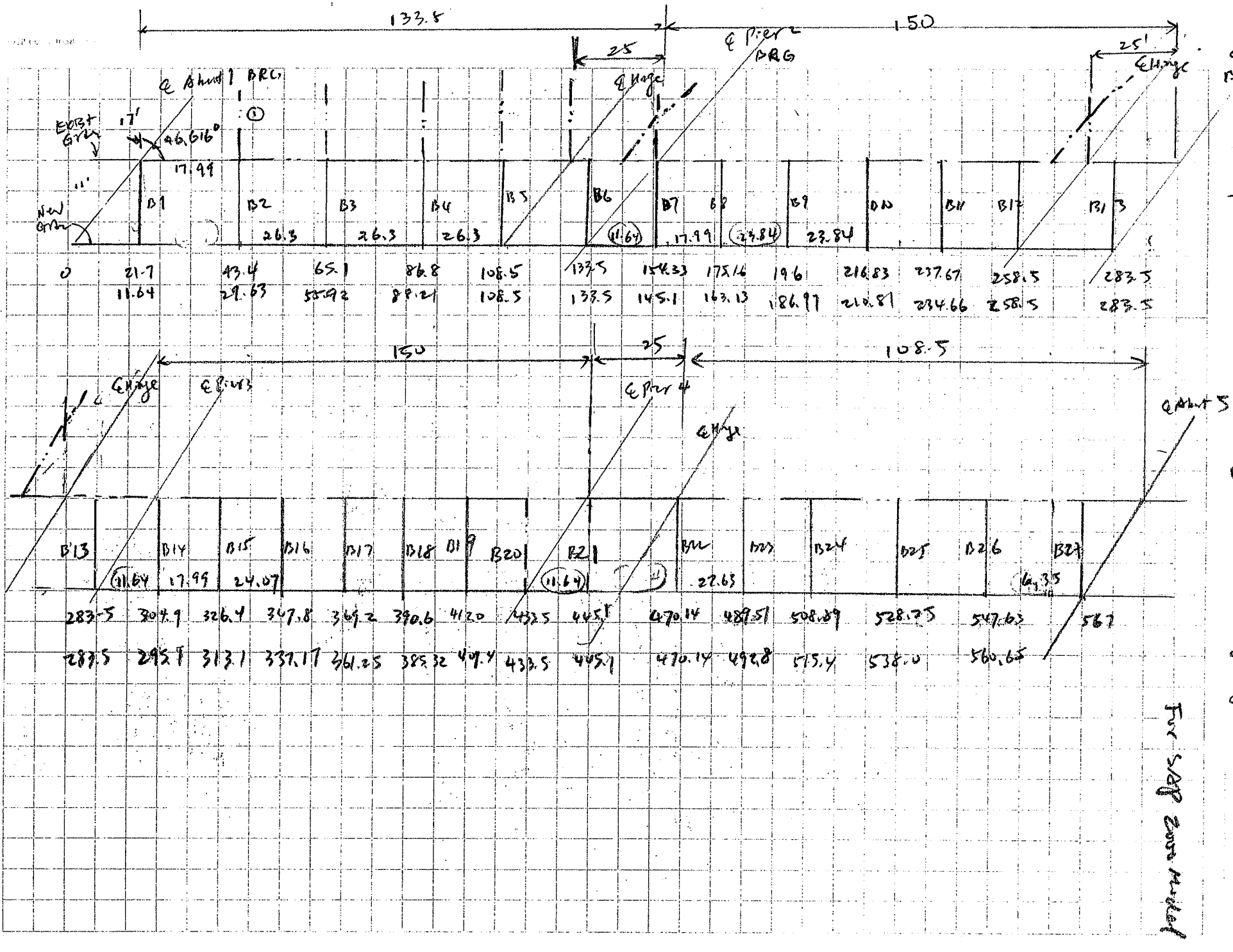
| Abutment 5 @ bot | | | | | |
|---|---|---|---|---|---|
| | As | d | a=(As*fy)/ (0.85f'c*b) | c=a/B1 | Mn=As*fy(d-a/2) /1000 (k-in) |
| | | | | | |
| #5 & #11 | 1.87 | 34 | 3.06 | 3.59 | 3643.38 |
| #5 & #10 | 1.58 | 34 | 2.58 | 3.04 | 3100.83 |
| #5 & #9 | 1.31 | 34 | 2.14 | 2.52 | 2588.28 |
| #5 & #8 | 1.1 | 34 | 1.80 | 2.11 | 2184.69 |
| #5 & #7 | 0.91 | 34 | 1.49 | 1.75 | 1815.81 |
| #5 | 0.31 | 34 | 0.51 | 0.60 | 627.69 |

| Abutment 1 @bot | | | | | |
|---|---|---|---|---|---|
| | As | d | a=(As*fy)/ (0.85f'c*b) | c=a/B1 | Mn=As*fy(d-a/2) /1000 (k-in) |
| | | | | | |
| #5 & #11 | 1.87 | 28 | 3.06 | 3.59 | 2970.18 |
| #5 & #10 | 1.58 | 28 | 2.58 | 3.04 | 2532.03 |
| #5 & #9 | 1.31 | 28 | 2.14 | 2.52 | 2116.68 |
| #5 & #8 | 1.1 | 28 | 1.80 | 2.11 | 1788.69 |
| #5 & #7 | 0.91 | 28 | 1.49 | 1.75 | 1488.21 |
| #5 | 0.31 | 28 | 0.51 | 0.60 | 516.09 |

307




MPHW
4/6/09

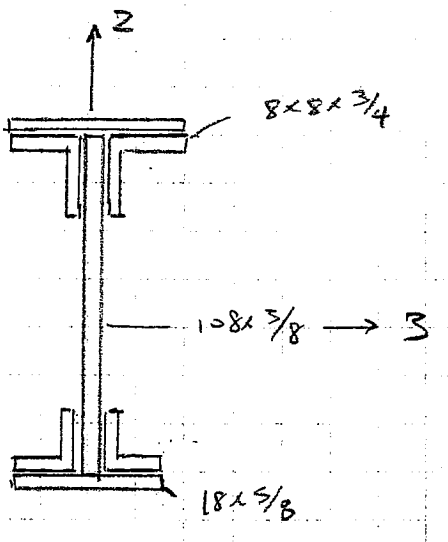
LA River Bridge - Prel'm. Bearing Layout

For SAP Zero Marked

308

FHW — LA River Bridge — 3.09

7/22/09 — Existing Bridge section Properties — SAP 2000 input




Area.

① Top cover plate: $18 \times 5/8 = 11.25\ in^2$

② 2-∠ 8×8×3/4: $11.4 \times 2 = 22.8\ in^2$

③ Web: $108 \times 5/8 = 67.5\ in^2$

④ 2-∠ 8×8×3/4: $11.4 \times 2 = 22.8\ in^2$

⑤ Bot cover Plate: $18 \times 5/8 = 11.25\ in^2$

N.A about Axis 3

| Item | Area | Arm | Moment |
|---|---|---|---|
| ① | 11.25 | $(108 + 5/8 + 5/8) - 5/8 \times \frac{1}{2} = 108.9375$ (= 109.25) | 1225.5 |
| ② | 22.8 | $109.25 - 5/8 - 0.72 = 107.905$ | 2460.7 |
| ③ | 67.5 | $5/8 + 108/2 = 54.625$ | 3687.2 |
| ④ | 22.8 | $5/8 + 0.72 = 1.345$ | 30.67 |
| ⑤ | 11.25 | $5/8 \times \frac{1}{2} = 0.3125$ | 3.52 |
| | 135.6 | | 7407.7 |

$y_{NA} = N.A = \frac{7407.7}{135.6} = 54.628$

| Item | $I_o$ | $(y_i - y_{NA})$ Dist to NA ($\bar{x}$) | $(A)(y_i - y_{NA})^2$ $A\bar{x}^2$ | $(I_{33})$ $I_o + A\bar{x}^2$ |
|---|---|---|---|---|
| ① | $\frac{1}{12} \times 18 \times (5/8)^3 = 0.367$ | $109.25 - 54.628 - 5/8 \times 1/2 = 54.31$ | $11.25 \times 54.31^2 = 33182$ | 33182 |
| ② | $69.9 \times 2 = 139.8$ | $109.25 - 5/8 - 0.72 - 54.628 = 53.277$ | $22.8 \times (53.27)^2 = 64716$ | 64856 |
| ③ | $\frac{1}{12} \times 5/8 \times 108^3 = 65610$ | $54.628 - 54 = 0.628$ | $67.5 \times 0.628^2 = 26.6$ | 65636 |
| ④ | $69.9 \times 2 = 139.8$ | $54.628 - 5/8 - 0.72 = 53.283$ | $22.8 \times 53.283^2 = 64731$ | 64871 |
| ⑤ | $\frac{1}{12} \times 18 \times (5/8)^3 = 0.367$ | $54.628 - 5/8 \times 1/2 = 54.315$ | $11.25 \times 54.315^2 = 33189$ | 33190 |
| | | | | $261735\ in^4$ |

FMW

7/22/09

# LA River Bridge



## Existing Bridge Section Properties
– SAP2000 output

symmetric to $I_{22}$. NA for $I_{22}$ should be at 0.

| Item | $I_o$ | Distance to N.A | $AY^2$ | $I_o + AY^2$ ($I_{22}$) |
|---|---|---|---|---|
| ① | $\frac{1}{12} \times \frac{5}{8} \times 18^3 = 303.75$ | 0. | 0 | 303.75 |
| ② | $69.9 \times 2 = 139.8$ | $\frac{5}{8} \times \frac{1}{2} \times 10.72 = 1.345$ | $(11.4 \times 1.345^2) \times 2 = 41.24$ | 181.05 |
| ③ | $\frac{1}{12} \times 108 \times \frac{5}{8}^3 = 2.20$ | 0 | 0 | 2.2 |
| ④ | $69.9 \times 2 = 139.8$ | $\frac{5}{8} \times \frac{1}{2} + 0.72 = 1.345$ | $11.4 \times 1.345^2 \times 2 = 41.24$ | 181.05 |
| ⑤ | $\frac{1}{12} \times \frac{5}{8} \times 18^3 = 303.75$ | 0 | 0 | 303.75 |
| | | | | 971.8 |

Torsional constant.

$\sum \frac{1}{3} b_i t_i^3$, b is the large dimension
t is the small dimension

$$J = \frac{1}{3}\left[\left(18 \times 0.75^3 + 8 \times 0.75^3 \times 4\right) \times 2 + 108 \times 0.625^3\right] = 22.9^{in}$$

Longitudinally, Abut 5 has less width, which provides less resistance, and Abut 5 spring stiffness is used in SAP2000.

$$K_{abut} = K_i \times w \times \left(\frac{h}{5.5}\right)$$

$$w = (74.5 + 70.5) = 145$$

$h = 9.9' \sim 10.5' \rightarrow$ use $10'$ as an average.

$$K_{abut} = 20 \frac{k/in}{ft} \times 139 \times \left(\frac{10}{5.5}\right) = 5272 \, k/in$$

$$P_{bw} = A_e \times 5^{ksf} \times \left(\frac{h_{bw}}{5.5}\right)$$

$$A_e = h_{bw} \times W_{bw} = 10 \times (145 - 4.5 - 1.5) = 1390 \, ft^2$$

$$P_{bw} = 1390 \times 5 \times \frac{10}{5.5} = 12636 \, kips$$

$$\Delta = \frac{P_{bw}}{K_{abut}} = \frac{12636}{5272} = 2.4 \, in$$

$\Delta_{gap} = 1''$ $\Delta_{total} = 3.4 \, in$

$$K_{eff} = \frac{12636}{3.4 \, in} = 3716 \, k/in \leftarrow \text{Input into SAP2000}$$

(K long)

$K_{Tran}$ use fix.

- WS is assign as area Pressure 0.035 K/ft² different loading case

- Permanent steel Deck Form = 0.13 K/ft convert into area will be,

$$\frac{0.13\ K/ft}{(7+7)ft} = 0.89\ lb/ft^2$$

- Barrier Load, 0.4 K/ft, add as line Load

- Barrier Load for Seismic model, Delete the line Load, and convert the line load as added mass.

$$0.4\ K/ft = 400\ lbf/ft = 400\ \frac{lbm}{ft}$$

- Convert permanent steel Deck and wearing surface into concrete Deck thickness to account for the mass.

WS = 35 lb/ft² Deck Form = 0.89 lb/ft²

Total = 36 lb/ft², equivalent thickness =

$$\frac{0.15 \cdot x}{12} = 0.036$$

$$\left(0.036 \times L \times 1(ft) = 0.15\ K/ft^3 \times L \times 1 \times \frac{x''}{12}\right)$$

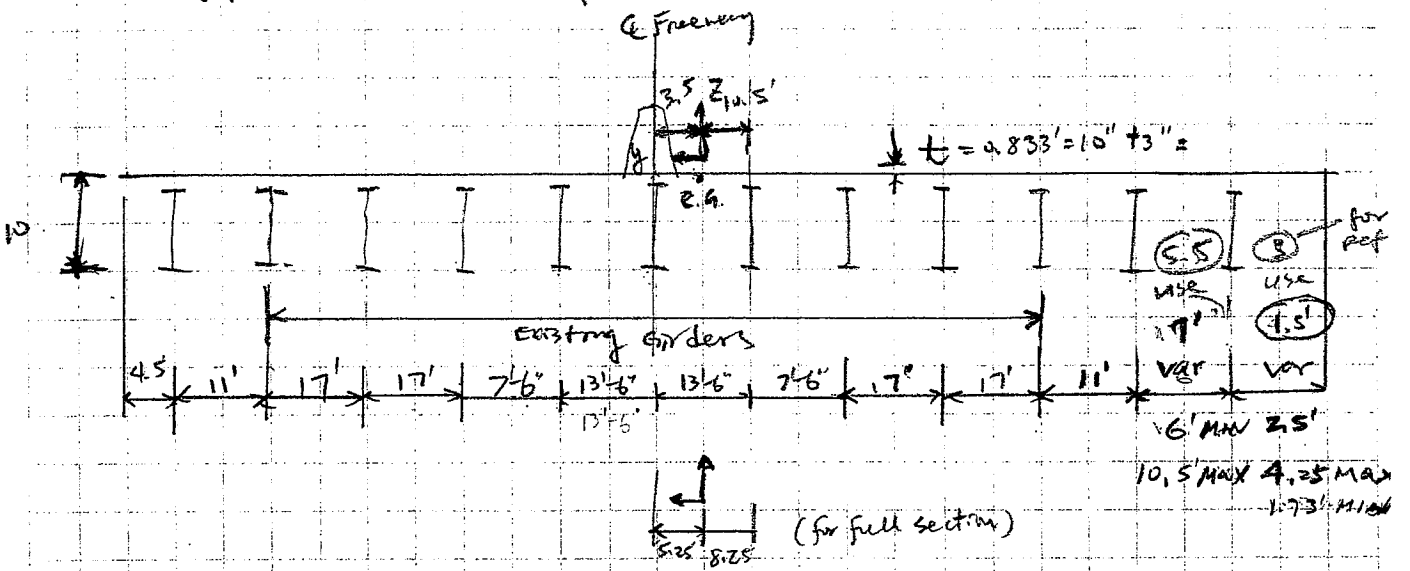
$$x \doteq 3''$$

- For seismic model place the column at C.G., which is same as layout line in the SAP 2000 model

FHW LA River Bridge 314

8/5/09 SAP 2000 model input

- Typical section Input for Seismic Model / Live Load




Total width of Bridge (From EOD TO EOD)

$4.5' + 11' + 17' + 17' + 7.5 + 13.5' + 13.5' + 7.5' + 17 + 17' + 11 + 7' + 4.5'$
(5.5) 3

$= 145'$

- Full section Total width of bridge

$4.5' + 11' + 17' + 17' + 7.5' + 13.5' + 13.5' + 7.5' + 17 + 17' + 11' + 10.5 + 4.5'$

$= 151.5'$

Total Live Load Lanes

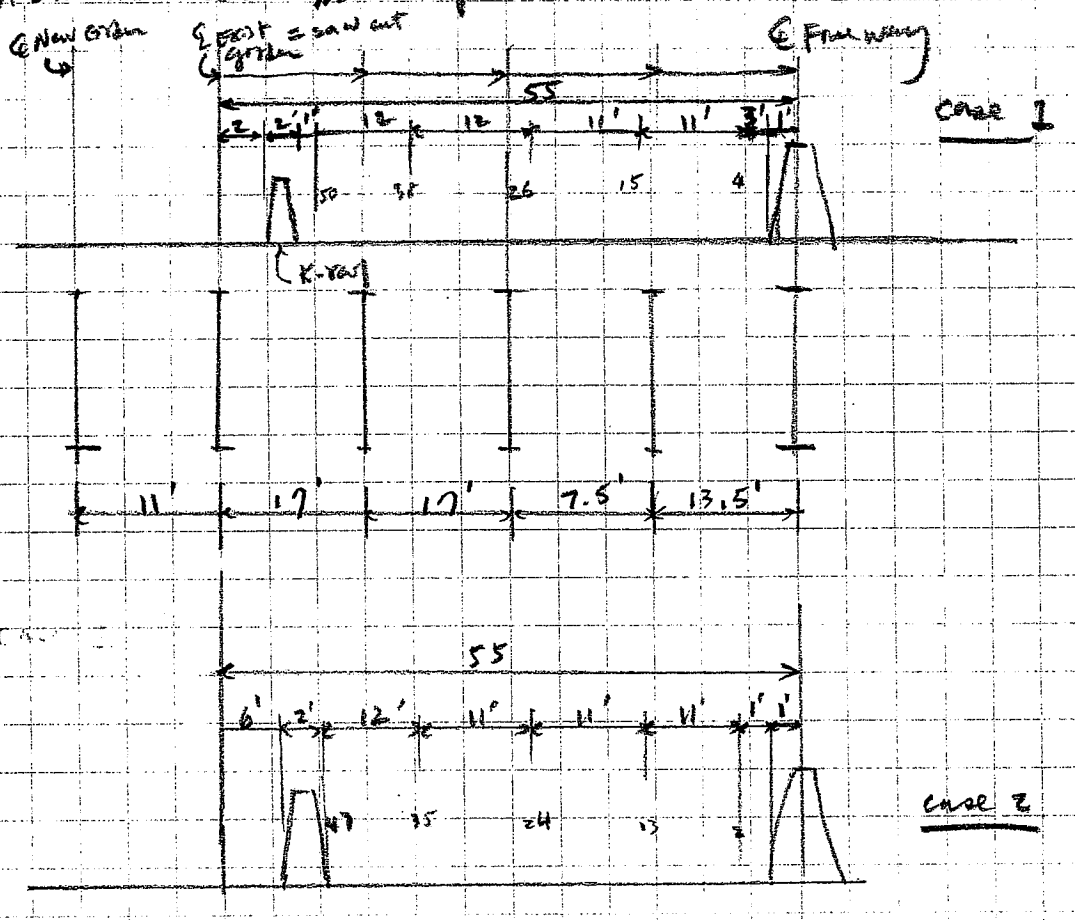
$= \frac{151.5' - 3' - 2'}{12} = 12.2$ say 12 Lanes, 6 Lanes each side

FHW
5/13/10

LA River Bridge
SAP 2000 model Input



To Run for Temporary condition, to obtain the live load during the Temporary Bridge construction, acting on the exterior girder, to check whether girder composite section is needed to resist the positive bending moment at mid-span.



Gamini Weeratunga/D12/Caltrans/CA Gov

09/21/2009 06:00 PM

To Frank Wei/HQ/Caltrans/CAGov@DOT
cc
bcc
Subject LA River Bridge, I-710; Spring Constants

Frank:

I found that the 15-kip/pile load produces lateral deflections in excess of 1 inch in both longitudinal and transverse directions due to low group efficiency. Therefore, I reduced the lateral load to 10 kips per pile. The deflections computed with this lateral load and the nominal vertical loads on piles are given below.

Lateral deflection in longitudinal or transverse directions = 0.74 inches.
Vertical displacement = 0.05 inches.

Total lateral load in either direction = 2210 kips
Total vertical load = 16,440 kips.

You can calculate the spring constants by dividing the above loads by the respective deflections.

Note that the lateral deflections in both directions are identical. Since the lateral load was applied to the cap in directions perpendicular to the pile axis, no vertical lateral loads were induced in the piles.

Gamini Weeratunga, G.E.
OGDS1
3351 Michelson Drive, Suite 100
Irvine, CA 92612
(949)440-3427

vertical spring stiffness = $\frac{16440 \text{ kip}}{0.05}$ = 328800 K/in

translational spring stiffness = $\frac{2210}{0.74}$ = 3000 K/in

– Abutment spring stiffness

$$K_{abut} = 20 \times w \times \frac{h}{5.5}$$

$w$ = entire Abutment width (EOD to EOD) = 145'

$h$ = height of superstructure + Pad thickness

= 9.75' + 0.25' = 10' = $h_{bw}$

$$K_{abut} = 20 \times 145' \times \frac{10'}{5.5} = 5272$$

$$P_{bw} = A_e \times 5\,ksf \times \frac{h_{bw}}{5.5}$$

$A_e$ = 10' × 145 = 1450

$$P_{bw} = 1450 \times 5 \times \frac{10}{5.5} = 13181 \text{ kips}$$

$\Delta_{gap}$ = 1"

$$\Delta = \frac{P_{bw}}{K_{abut}} = \frac{13181}{5272} = 2.5 \text{ in}$$

$$\Delta_{eff} = \Delta + \Delta_{gap} = 3.5 \text{ in}$$

$$K_{eff} = \frac{P_{bw}}{\Delta_{eff}} = \frac{13181}{3.5} = 3766 \text{ k/in}$$

use Half-Half approach with total 12 girders

each girder have

$$3766 \times \frac{1}{12} \times \frac{1}{2} \doteq 155 \text{ k/in}$$

if use Full-zero approach, each girder have

$$3766 \times \frac{1}{12} = 310 \text{ k/in}$$

- Abutment Transverse spring stiffness.

By SDC. say 40 K/in/pile Total 137 piles - Abut 5

137 x 40 = 5480 K/in

(45+49+22+10+10+11) = 137 piles

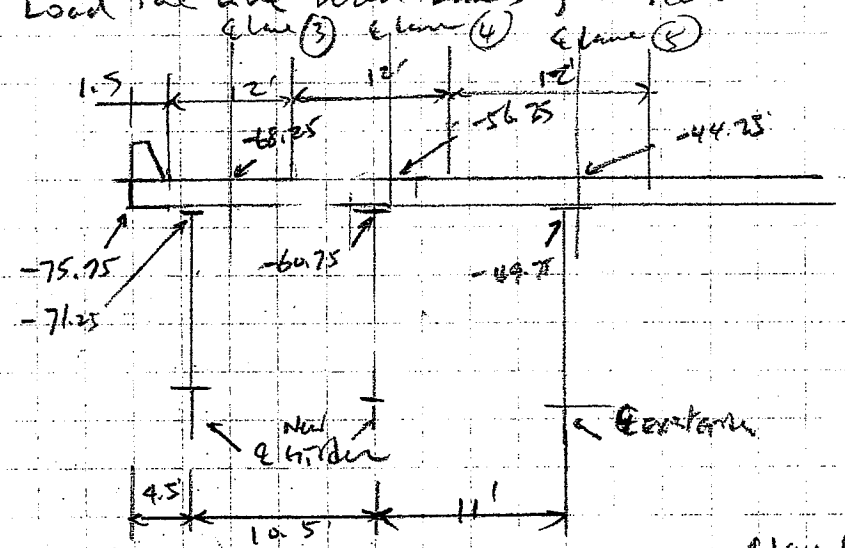
Abut 1 ⇒ 56 piles

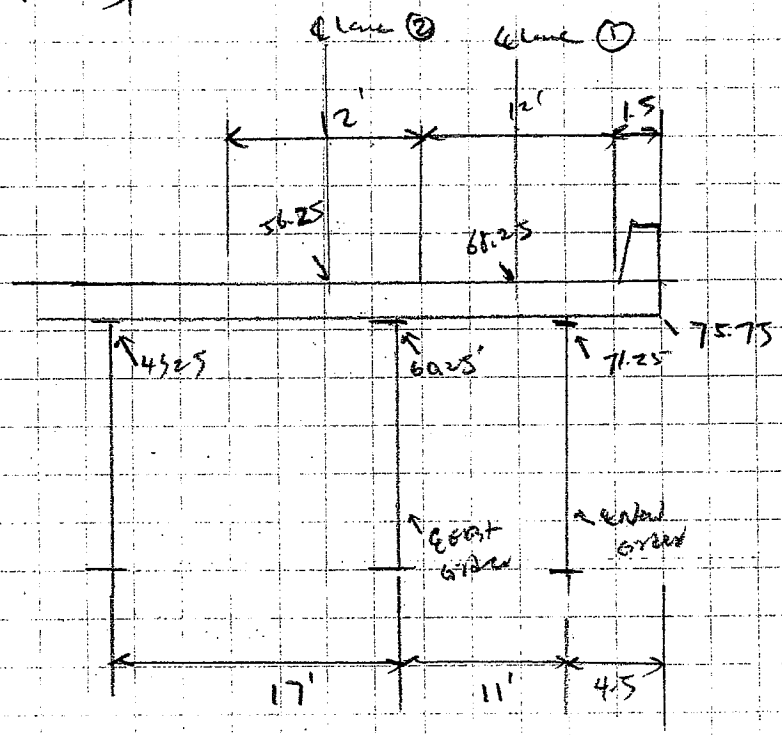
56 x 40 = 2240 K/in

FHW LA River Bridge 319

10/5/09 SAP 2000 model Live Load Input

Load the live load lanes for new girders only.



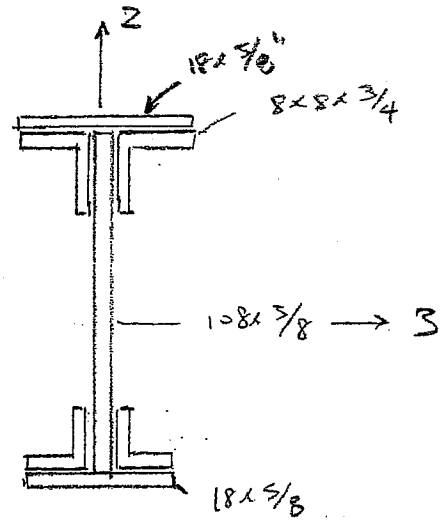



FHW — LA River Bridge — # 320

7/22/09

Existing Bridge section Properties
— SAP 2000 input

Check existing bridge flexural capacity during construction.




Area

① Top cover plate: $18 \times 5/8 = 11.25\ in^2$

② 2-∠ 8x8x3/4 $11.4 \times 2 = 22.8\ in^2$

③ Web $108 \times 5/8 = 67.5\ in^2$

④ 2-∠ 8x8x3/4 $11.4 \times 2 = 22.8\ in^2$

⑤ Bot cover plate $18 \times 5/8 = 11.25\ in^2$

135.6

N.A about Axis 3

| Item | Area | Arm ($y_i$) | Moment ($A_i y_i$) |
|---|---|---|---|
| ① | 11.25 | $(108 + 5/8 + 5/8) - 5/8 \times \frac{1}{2} = 109.375$ (109.25) | 1225.5 |
| ② | 22.8 | $109.25 - 5/8 - 0.72 = 107.905$ | 2460.7 |
| ③ | 67.5 | $5/8 + 108/2 = 54.625$ | 3687.2 |
| ④ | 22.8 | $5/8 + 0.72 = 1.345$ | 30.67 |
| ⑤ | 11.25 | $5/8 \times \frac{1}{2} = 0.3125$ | 3.52 |
| | 135.6 | | 7407.7 |

$$y_{NA} = N.A = \frac{7407.7}{135.6} = 54.628$$

| Item | $I_o$ | $y_i - y_{NA}$ Dist to NA ($x$) | $A(y_i - y_{NA})^2$ $AX^2$ | ($I_{33}$) $I_o + AX^2$ |
|---|---|---|---|---|
| ① | $\frac{1}{12} \times 18 \times (5/8)^3 = 0.367$ | $109.25 - 54.628 - 5/8 \times 1/2 = 54.31$ | $11.25 \times 54.31^2 = 33182$ | 33182 |
| ② | $69.9 \times 2 = 139.8$ | $109.25 - 5/8 - 0.72 - 54.628 = 53.277$ | $22.8 \times (53.27)^2 = 64716$ | 64856 |
| ③ | $\frac{1}{12} \times 5/8 \times 108^3 = 65610$ | $54.628 - 54 = 0.628$ | $67.5 \times 0.628^2 = 26.6$ | 65636 |
| ④ | $69.9 \times 2 = 139.8$ | $54.628 - 5/8 - 0.72 = 53.283$ | $22.8 \times 53.283^2 = 64731$ | 64871 |
| ⑤ | $\frac{1}{12} \times 18 \times (5/8)^3 = 0.367$ | $54.628 - 5/8 \times 1/2 = 54.315$ | $11.25 \times 54.315^2 = 33189$ | 33190 |
| | | | | 261735 in |

FHW

7/22/09

LA River Bridge

Existing Bridge Section Properties
– sap2000 output



symmetric to $I_{zz}$. NA for $I_{zz}$ should be at 0.

| Item | $I_o$ | Distance to N.A | $Ay^2$ | $I_o + Ay^2$ ($I_{zz}$) |
|---|---|---|---|---|
| ① | $\frac{1}{12} \times \frac{5}{8} \times 18^3 = 303.75$ | 0. | 0 | 303.75 |
| ② | $69.9 \times 2 = 139.8$ | $\frac{5}{8} \times \frac{1}{2} \times 0.72 = 1.345$ | $(11.4 \times 1.345^2) \times 2 = 41.24$ | 181.05 |
| ③ | $\frac{1}{12} \times 108 \times \frac{5}{8}^3 = 2.20$ | 0 | 0 | 2.2 |
| ④ | $69.9 \times 2 = 139.8$ | $\frac{5}{8} \times \frac{1}{2} + 0.72 = 1.345$ | $11.4 \times 1.345^2 \times 2 = 41.24$ | 181.05 |
| ⑤ | $\frac{1}{12} \times \frac{5}{8} \times 18^3 = 303.75$ | 0 | 0 | 303.75 |
| | | | | 971.8 |

Torsional constant.

$\sum \frac{1}{3} b_i t_i^3$, b is the large dimension
t is the small dimension

$$J = \frac{1}{3}\left[\left(18 \times 0.75^3 + 8 \times 0.75^3 \times 4\right) \times 2 + 108 \times 0.625^3\right] = 22.9^{in}$$

Check existing Bridge Section Capacity @ Mid girder, assuming non-composite section. check capacity for Temporary construction condition, At strength Limit state

use AASHTO Appendix A6. for Non-composite section

$y_{NCb} = 54.628$

$y_{NCT} = 108 + 2 \times 3/8 - 54.628 = 54.622$

$S_{NCb} = S_{NCT} = \frac{I_{NC}}{y_{NCb}} = \frac{I_{NC}}{y_{NCT}} = \frac{261735}{54.625} = 4791.5 \text{ in}^3$

Since As-Built Plans do not have structural steel designation, assume Grade 36 steel with

$F_y = 36$ ksi $\quad F_u = 58$ ksi

$\frac{2D_c}{t_w} < 5.7\sqrt{\frac{E}{F_{yc}}}$

$\frac{2 \times 54}{3/8} = 172.5$

$\lambda_{rw} = 5.7\sqrt{\frac{E}{F_{yc}}} = 5.7\sqrt{\frac{29{,}000}{36}} = 161.8$, Not satisfying

use AASHTO 6.10.8.

$f_{bu} + \frac{1}{3} f_l \le \phi_f F_{nc}$

$f_{bu} + \frac{1}{3} f_l \le \phi_f F_{nt}$

$f_\ell = 0$ for straight bridge.

$$f_{bu} \le \phi_f F_{nc} \quad \text{and} \quad f_{bu} \le \phi_f F_{nc}$$

For compression-flange flexural resistance

<u>Local Buckling Resistance</u>

$$\lambda_f = \frac{b_{fc}}{2t_{fc}} = \frac{18^{in}}{2\times(5/8+3/4)} = 6.54$$

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_{yc}}} = 0.38\sqrt{\frac{29{,}000}{36}} = 10.7$$

$$\lambda_f < \lambda_{pf} \Rightarrow \boxed{F_{nc} = R_b R_h F_{yc}}$$

$$R_b = 1 - \left(\frac{a_{wc}}{1200+300a_{wc}}\right)\left(\frac{2D_c}{t_w} - \lambda_{rw}\right) \le 1.0$$

$$a_{wc} = \frac{2D_c t_w}{b_{fc} t_{fc}} = \frac{2\times 54\times 0.625}{18\times(5/8+3/4)} = \frac{67.5}{24.75} = 2.73$$

$$R_b = 1 - \left(\frac{2.73}{1200+300\times 2.73}\right)(172.5 - 161.8) = 0.9855$$

$$R_h = \frac{12+\beta(3\rho-\rho^3)}{12+2\beta}$$

$$\beta = \frac{2D_n t_w}{A_{fn}} \quad \text{— AASHTO 6.10.1.10.1-2.}$$

$D_n = 54^{in}$, $t_w = 0.625$

$A_{fn} = 11.25 + 8\times 0.75\times 2 = 23.25^{in}$

$$\beta = \frac{2\times 54\times 0.625}{23.25} = 2.9$$

5/18/10 Existing Bridge section flexural capacity

$\rho$ = smaller of ( $F_{yw}/F_h$ or 1.0 ) = 1.0

$$R_h = \frac{12 + 2.9(3-1)}{12 + 2(2.9)} = 1 \text{ (homogeneous Built up Section)}$$

$F_{nc} = (0.9855) \times (1) \times 36 = 35.48$ ksi

$f_{bu}$ (compressive stresses) $= \dfrac{M_u}{S_{nc}}$

$M_u$ is from SAP 2000 Analysis. check critical mid span section at span 3, 0.5 pt.

DC1 = 2339 k-ft
DC2 = 336.1 k-ft
DW = 504
HL-93 = 1862 k-ft
P15 = 4000 k-ft

} Temporary construction, 1st interior girder load.

Strength I = 1.25 DC + 1.5 DW + 1.75 (HL-93)

Strength II = 1.25 DC + 1.5 DW + 1.35 (P15) ← control.

$= 1.25(2339 + \cancel{336.1}^{0}) + 1.5(\cancel{504}^{0}) + 1.35(4000)$
Temp (Temp.)

$= 8324$ k-ft

$$f_{bu} = \frac{8324 \times 12}{4791.5} = 20.8 \text{ ksi} < F_{nc} \text{ OK.}$$

## Lateral Torsional Buckling Resistance

From earlier calcs for the new girder

$L_p = 95.85$ , $L_r = 364.9$ , $L_b = 287$ in

$C_b = 1.26$

$$F_{nc} = C_b\left(1-\left(1-\frac{F_{yr}}{R_h F_{yc}}\right)\left[\frac{L_b-L_p}{L_r-L_p}\right]\right) R_b R_h F_{yc}$$

$F_{yr} = 0.7 F_{yc} = 0.7(36) = 25.2$

$$\frac{F_{yr}}{R_h F_{yc}} = \frac{25.2}{1 \times 36} = 0.7$$

$$F_{nc} = 1.26\left(1-(1-0.7)\left(\frac{287-95.85}{364.9-95.85}\right)\right) \times 0.985 \times 1 \times F_{yc}$$

$= 0.977(36) = 35.17$ ksi $> f_{bu} = 20.8$ ksi

## Tension flange flexural Resistance

$F_{nt} = R_h F_{yt} = 36$ ksi $> f_{bu}$

FHW LA River Bridge 326

6/28/10 Pier 3 pinning.

$M_D$ of each pier is as follows

Pier 2, $M_D = 1.4$

Pier 3, $M_D = 1.05$

Pier 4. $M_D = 1.12$

1996 Retrofit. Pier 3 did not have foundation retrofit. Pier 2 and pier 4 had foundation Retrofit. Per displacement ductility concept, Pier 3 Bottom of column is reaching Mp.

Pier 2 & Pier 4 New widening will match retrofitted footing size, and design for $M_D = 1.2 M_p$. Also adding Top footing rebars and footing stirrup.

Pier 3 footing has not been retrofitted before, Also, it does not have Top fto reinforcement. In the model, I will rerun to place a pin at Pier 3 bottom of column, and see how Pier 2, Pier 4 displacement demand becomes to

| Pier | $\Delta D$ (Displacement Demand) (in) | $\Delta Y$ | $\mu_D$ ($\frac{\Delta D}{\Delta y}$) SS |
|---|---|---|---|
| 2 | 7 | 4 | 1.75 |
| 3 | 8.4 | 5.6 | 1.5 |
| 4 | 8.4 | 5.1 | 1.57 |

Also $\Delta D < \Delta C$ OK.

Therefore, base upon SAP 2000 run, File Name

SC13m.2 load with skew — Pin Pier 3.

The displacement demand is acceptable Therefore

- Pier 3 Footing No need to retrofit.
- Pier 2 & Pier 4 Footing will be designed for $M_o = 1.2\ M_p$