AASHTO T-3 TRIAL DESIGN BRIDGE DESCRIPTION

State: *Arkansas*

Trial Design Designation: *AR-2 (100566 Arkansas #2)*

Bridge Name: *Mud Slough Ditch*

Superstructure Type: *90' Continuous Composite W-Beam Unit*

Span Length(s): *28' – 34' – 28'*

Substructure Type: *Trestle Pile Bent*

Foundation: *Steel Shell Friction Piles*

Abutments: *Integral Pile Bent*

Seismic Design Category (SDC): *"D"*

Seismic Design Strategy (Type 1, 2 or 3): *Type 1*

Design Spectral Acceleration at 1-second Period (S_{D1}): *0.533g*

Additional Description (Optional): ______________________________

P. TERRY E.I.
9/22/2006

Arkansas Highway and Transportation Department

MUD SLOUGH DITCH

Seismic Design

with

Proposed LRFD Guidelines

Current LRFD Code as Appendix

Table of Contents

Analysis Summary: Mud Slough Ditch

Comparison of Design Parameters - Mud Slough Ditch

Parameter	Current LRFD Specification	Proposed LRFD Guidelines
Seismic Classification	Zone 3	SDC D
Importance Category	Other	Life Safety
Expected yield stress of reinforcing	75 ksi	66 ksi (8.4.2)
Expected compressive concrete strength	4.5 ksi	5 ksi (8.4.4)
R-factor for columns	3.5 (multi-column bent)	1.8 for SDC D (4.3.3) *

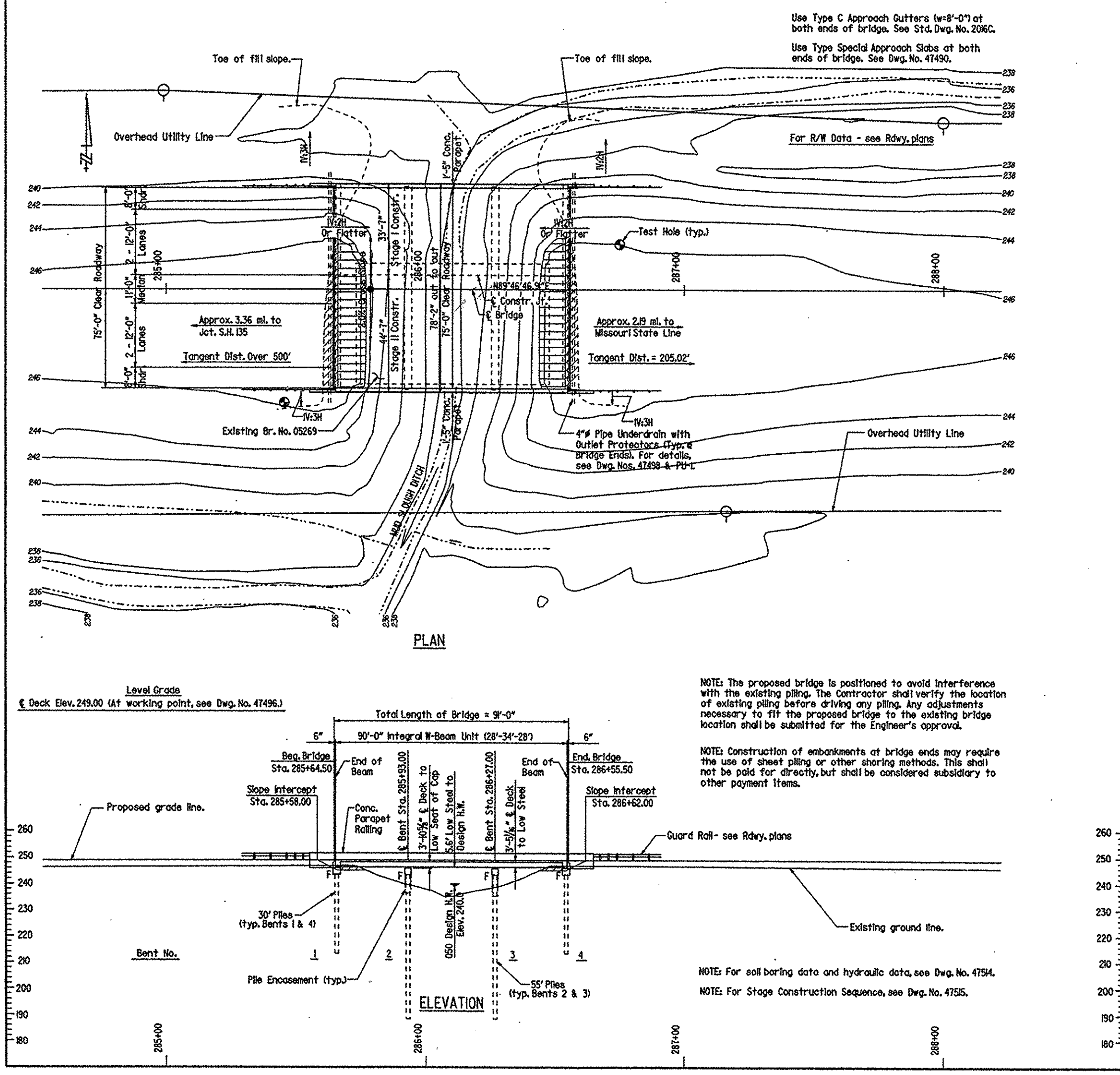
Comparison of Results - Mud Slough Ditch

Result	Current LRFD Specification	Proposed LRFD Guidelines
Elastic bent force (transverse)	141 kips	302 kips
Elastic bent force (longitudinal)	134 kips	281 kips
Bent displacement demand (transverse)	1.72 inches (R = 3.5)	1.88 inches (R = 1.8 *)
Bent displacement demand (longitudinal)	1.81 inches (R = 3.5)	2.01 inches (R = 1.8 *)
Bent displacement capacity (transverse)	N/A	4.53 inches (implicit) 18.45 inches (pushover)
Bent displacement capacity (longitudinal)	N/A	7.42 inches (implicit) 44.61 inches (pushover)
Elastic abutment force (transverse)	404 kips	866 kips
Elastic abutment force (longitudinal)	546 kips	1136 kips
Column spiral reinforcement	Steel Shell Pile	Steel Shell Pile
Superstructure to bent connection force		

* R_d based on R from "SDC C", since the "SDC D" R is less stringent.

No change in reinforcement. The current bridge was conservatively designed.
Reinforcement hooks do not count for development length. This may change some details.
Assuming no liquefaction, steel shell piles are at plastic capacity limit. See page 90.
Increased earthquake magnitude causes serious liquefaction concerns. See page 59.
Approach slabs are required by the new guidelines for this bridge. See page 94.

DATE REVISED	DATE FILMED	DATE REVISED	DATE FILMED	FED. ROAD DIST. NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
				6	ARK.			
				JOB NO.		100566	102	223
				07041		LAYOUT		47513



NOTE: The proposed bridge is positioned to avoid interference with the existing piling. The Contractor shall verify the location of existing piling before driving any piling. Any adjustments necessary to fit the proposed bridge to the existing bridge location shall be submitted for the Engineer's approval.

NOTE: Construction of embankments at bridge ends may require the use of sheet piling or other shoring methods. This shall not be paid for directly, but shall be considered subsidiary to other payment items.

NOTE: For soil boring data and hydraulic data, see Dwg. No. 47514.

NOTE: For Stage Construction Sequence, see Dwg. No. 47515.

GENERAL NOTES

BENCH MARK: #919 southwest corner of existing bridge, 35.01' Rt. of Sta. 285+72.32, Elev. 247.48.

CONSTRUCTION SPECIFICATIONS: Arkansas State Highway and Transportation Department Standard Specifications for Highway Construction, (2003 edition) with applicable supplemental specifications and special provisions. Section and Subsection refer to the Standard Construction Specification unless otherwise noted in the Plans.

DESIGN SPECIFICATIONS: AASHTO LRFD Bridge Design Specifications (2004 Edition).

LIVE LOADING: HL93

SEISMIC PERFORMANCE ZONE: 3

MATERIALS AND STRENGTHS:

Class S(AE) Concrete (superstructure)	f'c = 4,000 psi
Class S Concrete (substructure)	f'c = 3,500 psi
Reinforcing Steel (AASHTO M31 or M53, Gr. 60)	fy = 60,000 psi
Structural Steel (AASHTO M270, Gr. 36)	Fy = 36,000 psi
Structural Steel (AASHTO M270, Gr. 50W)	Fy = 50,000 psi

BORING LOGS: Boring logs may be obtained from the Programs and Contracts Division.

CONCRETE FILLED STEEL SHELL PILING: Piling for Bents 1 & 4 shall be 16" diameter concrete filled steel shell piles and shall be driven to a minimum safe bearing capacity of 30 tons per pile. Piling for Bents 2 & 3 shall be 18" diameter concrete filled steel shell piles and shall be driven to a minimum safe bearing capacity of 50 tons per pile. All piles shall be driven with an approved air, steam, or diesel hammer. All piling shall be driven to a minimum tip elevation of 220.0 or lower. Piling in end bents shall be driven after embankment to bottom of cap is in place. Length of piling shown are assumed for estimating quantities only. Actual lengths to be determined in the field. No payment will be made for cut-off or build-up. Test piles are not required but may be driven for the contractor's information in accordance with Subsection 805.08(g).

Water jetting or other methods approved by the Engineer may be needed to achieve the minimum pile penetration. Any cost associated with achieving the minimum pile penetration shall be included in the item "Steel Shell Piling (16" Dia.)" and "Steel Shell Piling (18" Dia.)".

PREBORING: Preboring is required for all piling at Bents 1 and 4 to a depth of 10' below bottom of cap. Prebored holes shall be 6" greater than the diameter of the pile and shall be backfilled with loose sand after piles are in place. The Contractor shall be responsible for keeping prebored holes free of debris prior to backfilling, which may require the use of temporary casings or other methods. Preboring and temporary casing if necessary will not be paid for directly but will be considered subsidiary to "Steel Shell Piling".

PILE ENCASEMENT: Pile encasement for Bents 2 & 3 shall extend 3 feet into the ground and to the bottom of the cap. See Dwg. No. 47488 for additional details.

PIPE UNDERDRAIN: One Pipe Underdrain with Outlet Protectors shall be installed behind each Bridge End in accordance with Section 611. Pipe Underdrains will not be paid for directly but shall be considered subsidiary to "Unclassified Excavation".

BRIDGE DECK: The concrete bridge deck shall be given a tine finish as specified for final finishing in Subsection 802.19 for Class 5 Tined Bridge Roadway Surface Finish.

DETAIL DRAWINGS:	DRAWING NO.
End Bents	47494
Int. Bents	47495
90' Integral W-Beam Unit	47496-49501
Elastomeric Bearings	47489
Type C Approach Gutters	2016C
Type Special Approach Slab	47490
Concrete Filled Steel Shell Piles	47488

EXISTING BRIDGE: The existing three-span bridge, No. 05269, (L.M. 6.05) is 75' long and 46.3' wide and consists of a concrete superstructure supported by a concrete pile substructure.

REMOVAL AND SALVAGE: After Stage 1 of the new bridge is open to traffic, the existing bridge, No. 05269, shall be removed in accordance with Section 205. All material from the existing bridge shall become the property of the Contractor.

MAINTENANCE OF TRAFFIC: See Roadway Plans.

STATE OF ARKANSAS REGISTERED PROFESSIONAL ENGINEER No. 4337 CHARLES P. BRAND 5/4/05

BRIDGE ENGINEER

SHEET 1 OF 2
LAYOUT OF BRIDGE OVER MUD SLOUGH DITCH
PARAGOULD - BIG SLOUGH DITCH (F)
GREENE COUNTY

ROUTE 412 SEC. 9

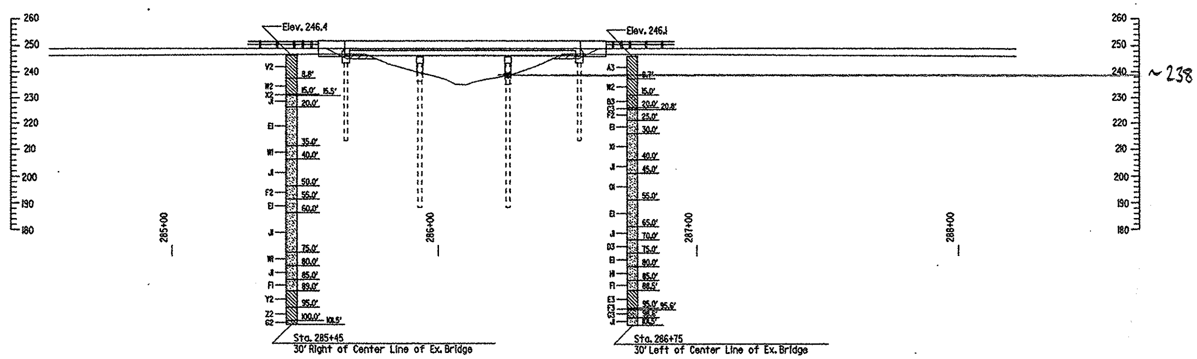
ARKANSAS STATE HIGHWAY COMMISSION

LITTLE ROCK, ARK.

DRAWN BY: K.W.Y. DATE: 9-20-04 FILENAME: b100566x4_11.dgn
CHECKED BY: MAH DATE: 4-28-05 SCALE: 1" = 20'
DESIGNED BY: KWY DATE: 8-04
BRIDGE NO. 07041 DRAWING NO. 47513

4

DATE REVISED	DATE FILMED	DATE REVISED	DATE FILMED	FED. ROAD DIST. NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
				6	ARK.			
				JOB NO.		100566	103	223
				07041		LAYOUT		47514



238
− 1.8(5.15)
228.73

HYDRAULIC DATA

FLOOD DESCRIPTION	FREQUENCY	DISCHARGE	*NATURAL WATER SURFACE ELEVATION	WATER SURFACE ELEV. WITH BACKWATER
	YEARS	CFS	FEET	FEET
Design	50	520	240.0	241.1
Base	100	590	240.1	241.4
Extreme	500	740	240.3	241.9
Overtopping	> 500	NA	NA	NA

* Unconstricted water surface without structure or roadway approaches.
Drainage area = 1.8 square miles.
Historical H.W. Elev. = 243.3

"N" VALUES

Sta. 285+45 - 30' Right of Center Line of Ex. Bridge	Sta. 286+75 - 30' Left of Center Line of Ex. Bridge
4.3- 5.3, N=6	4.2- 5.2, N=8
9.3- 10.3, N=5	9.2- 10.2, N=6
15.5- 16.5, N=16	15.5- 16.5, N=0
20.5- 21.5, N=33	20.5- 21.5, N=10
25.5- 26.5, N=36	25.5- 26.5, N=33
30.5- 31.5, N=33	30.5- 31.5, N=23
35.5- 36.5, N=19	35.5- 36.5, N=23
40.5- 41.5, N=14	40.5- 41.5, N=16
45.5- 46.5, N=14	45.5- 46.5, N=34
50.5- 51.5, N=9	50.5- 51.5, N=44
55.5- 56.5, N=45	55.5- 56.5, N=47
60.5- 61.5, N=25	60.5- 61.5, N=33
65.5- 66.5, N=26	65.5- 66.5, N=25
70.5- 71.5, N=28	70.5- 71.5, N=26
75.5- 76.5, N=28	75.5- 76.5, N=47
80.5- 81.5, N=24	80.5- 81.5, N=61
85.5- 86.5, N=63	85.5- 86.5, N=68
90.5- 91.5, N=8	90.5- 91.5, N=7
95.5- 96.5, N=2	95.5- 96.5, N=14
100.5-101.5, N=2	100.5-101.5, N=27

BORING LEGEND

E1-Wet, Dense, Gray Sand
F1-Wet, Very Dense, Gray Sand
H1-Wet, Very Dense, Gray Sand with Traces of Gravel
J1-Wet, Medium Dense, Gray Sand
Q1-Wet, Dense, Gray Sand with Traces of Gravel
W1-Wet, Medium Dense, Gray Sand with Traces of Gravel
X1-Wet, Medium Dense, Gray Sand with some Organic Matter
F2-Wet, Loose, Gray Sand
G2-Wet, Soft, Gray Sandy Clay
V2-Moist, Medium Stiff, Gray and Brown Clay with some Sand and Traces of Organic Matter
W2-Moist, Medium Stiff, Gray and Brown Clay
X2-Wet, Very Loose, Gray Sand with Clay Seams
Y2-Wet, Medium Stiff, Gray Clay with Traces of Gravel
Z2-Wet, Soft, Gray Clay
A3-Moist, Medium Stiff, Gray and Brown Clay with some Sand and Traces of Gravel
B3-Wet, Very Soft, Gray Silty Clay with some Organic Matter
C3-Wet, Very Soft, Gray Sandy, Silty Clay
D3-Wet, Medium Dense, Gray Sand with Traces of Organic Matter
E3-Wet, Medium Stiff, Gray Clay with some Sand
F3-Wet, Stiff, Gray Sandy Clay
G3-Wet, Medium Dense, Gray Clayey Sand

NOTE: Traces and seams of lignite were encountered in some of the borings and may be encountered in greater amounts at other locations within the project area.

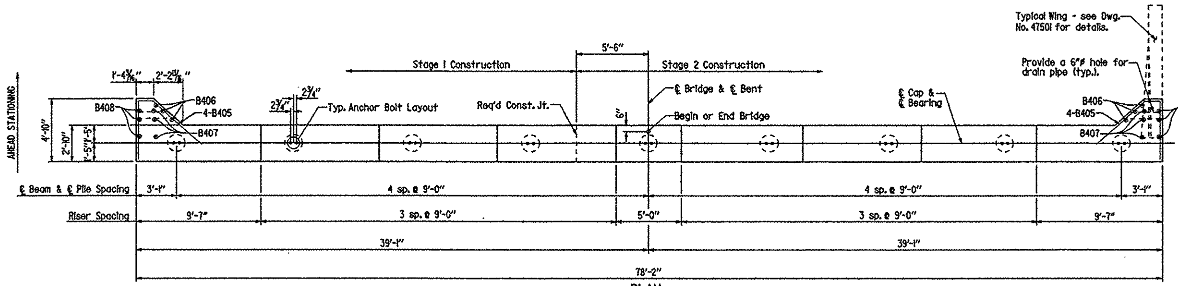
STATE OF ARKANSAS
REGISTERED PROFESSIONAL ENGINEER
No. 4337
4/28/05
CHARLES P. BRAND

BRIDGE ENGINEER

SHEET 2 OF 2
LAYOUT OF BRIDGE OVER
MUD SLOUGH DITCH
PARAGOULD - BIG SLOUGH DITCH (F)
GREENE COUNTY
ROUTE 412 SEC. 9
ARKANSAS STATE HIGHWAY COMMISSION
LITTLE ROCK, ARK.

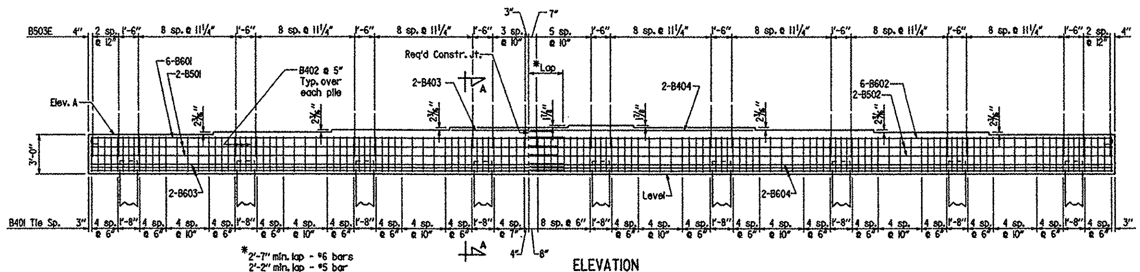
DRAWN BY: K.W.Y. DATE: 9-20-04 FILENAME: b100566x4_11.dgn
CHECKED BY: MAH DATE: 4-28-05 SCALE: 1" = 20'
DESIGNED BY: KWY DATE: 8-04
BRIDGE NO. 07041 DRAWING NO. 47514

DATE REVISED	DATE FILMED	DATE REVISED	DATE FILMED	FED. ROAD DIST. NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
				6	ARK.			
				JOB NO.		100566	83	223
				(1) 07039, 07041		END BENTS		47494



PLAN

Bent 4 Shown, Bent 1 Similar
Scale: ¼" = 1'-0"



ELEVATION

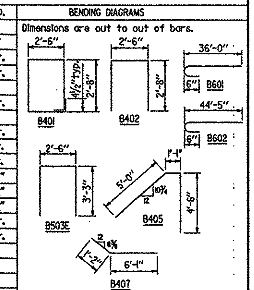
Looking Ahead Bent 4
Scale: ¼" = 1'-0"

BAR LIST-PER BENT

MARK	NO. REQ'D.	LENGTH	P.D.
B401	115	10'-8"	2"
B402	27	7'-8"	2"
B403	2	7'-7"	Str.
B404	2	16'-10"	Str.
B405	8	10'-6"	2"
B406	6	4'-2"	Str.
B407	6	7'-3"	2"
B408	6	8'-5"	Str.
B501	4	35'-7"	Str.
B502	4	44'-5"	Str.
B503E	79	8'-10"	2½"
B601	6	36'-8"	4½"
B602	6	45'-1"	4½"
B603	6	36'-0"	Str.
B604	6	44'-5"	Str.

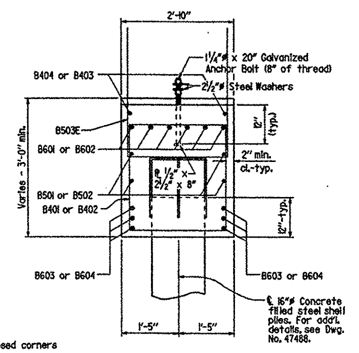
BENDING DIAGRAMS

Dimensions are out to out of bars.



Bars designated with an "E" suffix are epoxy coated.

NOTE: Use standard nuts for grade adjustment.

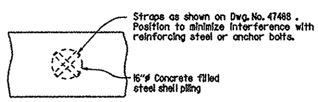


SECTION A-A

¾" = 1'-0"

TABLE OF VARIABLES

Bridge. No.	Elev. A
07039	249.26
07041	245.26



STRAP DETAIL

No Scale

GENERAL NOTES

All concrete shall be Class S and be poured in the dry. All exposed corners to be chamfered ¾" unless otherwise noted.

All reinforcing steel shall conform to AASHTO M31 or M53, Grade 60.

Granular backfill and pipe underdrain required behind cap, see Dwg. No. 47498.

For additional information, see Layout.

For details of steel shell piles, see Dwg. No. 47488.

BRIDGE ENGINEER

DETAILS OF END BENTS
LOCUST CREEK RELIEF
& MUD SLOUGH DITCH

ROUTE SEC.

ARKANSAS STATE HIGHWAY COMMISSION

LITTLE ROCK, ARK.

DRAWN BY: K.W.Y. DATE: 3-17-05 FILENAME: b100566x3.bl.dgn

CHECKED BY: MAH DATE: 4-28-05 SCALE: as noted

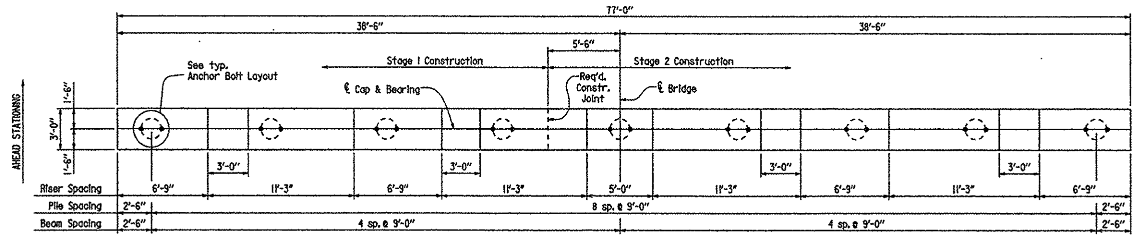
DESIGNED BY: KWY DATE: 8-04

BRIDGE NO. 07039, 07041 DRAWING NO. 47494

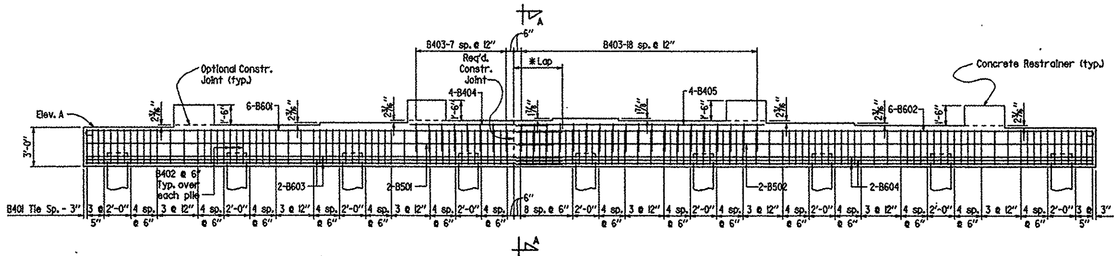
6

DATE REVISED	DATE FILMED	DATE REVISED	DATE FILMED	FED. ROAD DIST. NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
				6	ARK.			
				JOB NO.		100566	84	223

① 07039 & 07041 - INT. BENTS - 47495

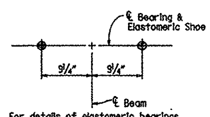


PLAN
Scale: 1/4" = 1'-0"



ELEVATION
Looking Ahead
Scale: 1/4" = 1'-0"

* 2'-7" min. lap - #6 bar
2'-2" min. lap - #5 bar
1'-9" min. lap - #4 bar

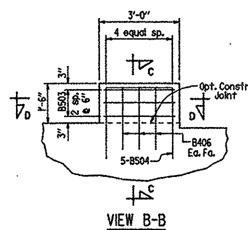


For details of elastomeric bearings, see Dwg. No. 47489.

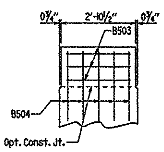
ANCHOR BOLT LAYOUT
No Scale

TABLE OF VARIABLES

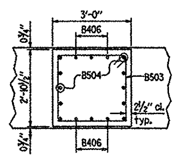
Bridge No.	Elev. A
07039	249.13
07041	245.13



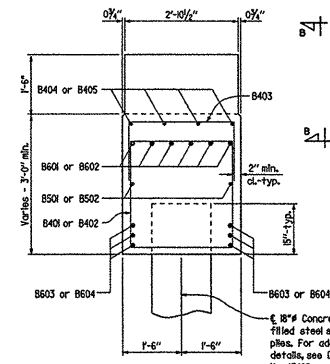
VIEW B-B
Scale: 1/2" = 1'-0"



SECTION C-C
Scale: 1/2" = 1'-0"



SECTION D-D
Scale: 1/2" = 1'-0"



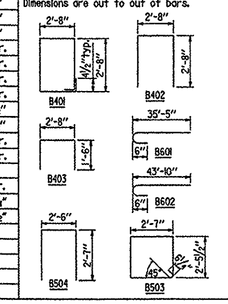
SECTION A-A
Scale: 3/4" = 1'-0"

BAR LIST-PER BENT

MARK	NO. REQ'D.	LENGTH	P.D.
B401	106	11'-0"	2"
B402	27	7'-10"	2"
B403	27	5'-6"	2"
B404	4	10'-3"	Str.
B405	4	19'-1"	Str.
B501	2	35'-0"	Str.
B502	2	43'-10"	Str.
B601	6	36'-1"	4½"
B602	6	44'-6"	4½"
B603	6	35'-5"	Str.
B604	6	43'-10"	Str.
B406	24	2'-4"	Str.
B503	12	10'-8"	3¾"
B504	20	7'-6"	2½"

BENDING DIAGRAMS

Dimensions are out to out of bars.



GENERAL NOTES

All concrete shall be Class S and be poured in the dry. All exposed corners to be chamfered ¾" unless otherwise noted.

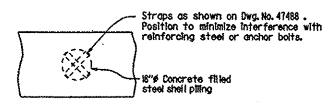
All reinforcing steel shall conform to AASHTO M31 or M53, Grade 60.

If anchor bolts are drilled into cap, top reinforcing bars and pile straps shall be properly placed to avoid damage.

For additional information, see Layout.

For elastomeric bearings, see Dwg. No. 47489.

For details of steel shell piles, see Dwg. No. 47488.



STRAP DETAIL
No Scale

STATE OF ARKANSAS REGISTERED PROFESSIONAL ENGINEER No. 4337 CHARLES P. BRAND 4/28/05

BRIDGE ENGINEER

DETAILS OF INT. BENTS LOCUST CREEK RELIEF & MUD SLOUGH DITCH

ROUTE 412 SEC. 9

ARKANSAS STATE HIGHWAY COMMISSION

LITTLE ROCK, ARK.

DRAWN BY: JWD DATE: 3-16-05 FILENAME: b100566x2_b2.dgn
CHECKED BY: MAH DATE: 4-28-05 SCALE: AS NOTED
DESIGNED BY: KWY DATE: 8-04

BRIDGE NO. 07039 & 07041 DRAWING NO. 47495

DATE REVISED	DATE FILMED	DATE REVISED	DATE FILMED	FED. ROAD DIST. NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
				6	ARK.			
				JOB NO.		100566	85	223
				① 07039, 07041		90' UNIT		47496

SLAB REINFORCING

Transverse:
S501E & S502E @ 14"o.c. bent up over beams
S601E & S602E @ 14"o.c. in top
S503E & S504E @ 14"o.c. in bottom
S505E @ 14" o.c. at both gutterlines (placed around ends of S501E & S502E, see "Reinforcing Plan & Deck Pouring Sequence", Dwg. No. 47498)

Longitudinal:
S401E as shown (12" max.)
S603E as shown over int. supports

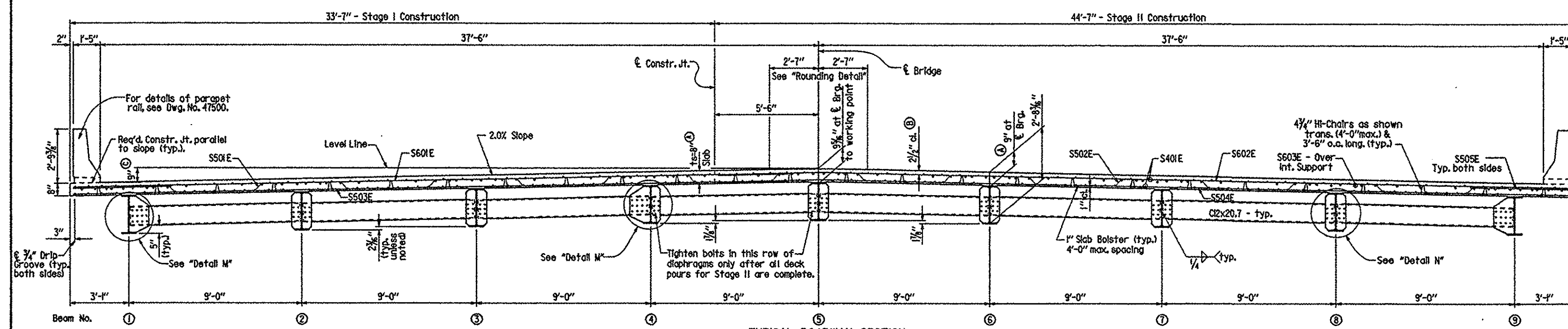
NOTE: At Contractor's Option, in lieu of providing bars S501E or S502E, one #5 bar top and bottom may be substituted for each bar. Payment for reinforcing will be based on the weight of bars S501E and S502E. Bars in top and bottom shall be Epoxy Coated.

NOTE: Class I Protective Surface Treatment shall be applied to the Roadway Surface and the Face and Top of Concrete Parapet Rail and Transition Rail.

Ⓐ See "Adustment for Slab and Haunch Thickness Tolerance."

Ⓑ Tolerance: Minus = 1/4"; Plus equal to the amount of slab thickening used to meet slab thickness tolerance. See "Adustment for Slab and Haunch Thickness Tolerance."

Ⓒ Working Point to Gutterline-See "Rounding Detail."



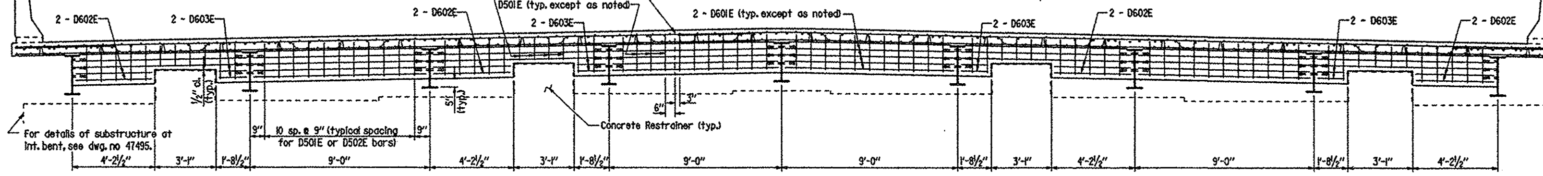
TYPICAL ROADWAY SECTION

Looking Ahead
Scale: 3/8" = 1'-0"

NOTE: Details shown in this section are similar to those shown in "Typical Roadway Section" except as shown.

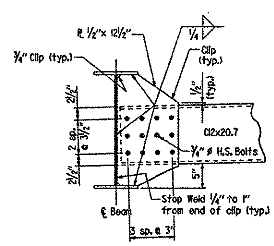
NOTE: Longitudinal restrainers are not shown for clarity.

NOTE: 1/2" polystyrene shall be used as a bond breaker between the concrete restrainer and the concrete diaphragm and may remain in place.



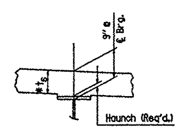
TYPICAL ROADWAY SECTION AT INT. BENTS

Looking Ahead
Scale: 3/8" = 1'-0"



DETAIL M

No Scale



NOTE: t_s = slab thickness as shown in "Typical Roadway Section."

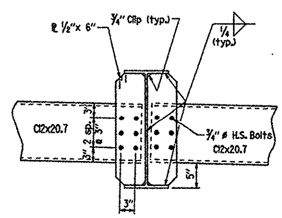
* Tolerance when removable deck forming is used is + 1/2", - 1/4". Haunch forming is required and shall be adjusted to maintain slab thickness tolerance.

ADJUSTMENT FOR SLAB AND HAUNCH THICKNESS TOLERANCE

No Scale

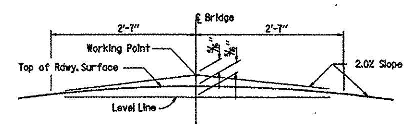
Haunch dimension may vary within the following limits to maintain the grade and slab thickness tolerance: Minimum - occurs when top flange contacts bottom reinforcing steel; Maximum - top flange thickness plus 1 3/4". No increase in concrete and structural steel quantities will be made to maintain tolerances.

Tolerances shown are applicable only when removable deck forming is used. See Std. Dwg. No. 14991 for tolerances when permanent steel deck forms are used. Payment for concrete shall be based on removable deck forming.



DETAIL N

No Scale



NOTE: Working Point matches Theoretical Roadway Grade.

ROUNDING DETAIL

No Scale

STATE OF ARKANSAS REGISTERED PROFESSIONAL ENGINEER No. 4337 4/28/05 CHARLES P. BRAND

BRIDGE ENGINEER

SHEET 1 OF 6
DETAILS OF 90' INTEGRAL W-BEAM UNIT LOCUST CREEK RELIEF & MUD SLOUGH DITCH

ROUTE SEC.

ARKANSAS STATE HIGHWAY COMMISSION

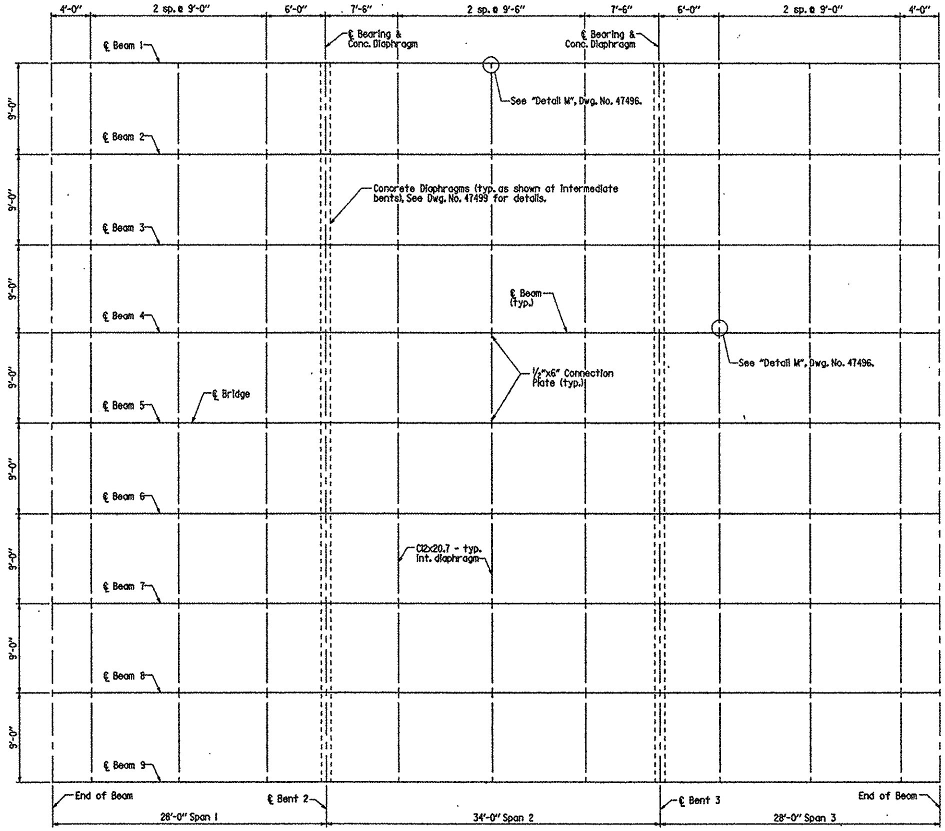
LITTLE ROCK, ARK.

DRAWN BY: K.W.Y. DATE: 2-9-05 FILENAME: bl00566x2_sl1.dgn
CHECKED BY: MAH DATE: 4-28-05 SCALE: as noted
DESIGNED BY: KWY DATE: 8-04

BRIDGE NO. 07039, 07041 DRAWING NO. 47496

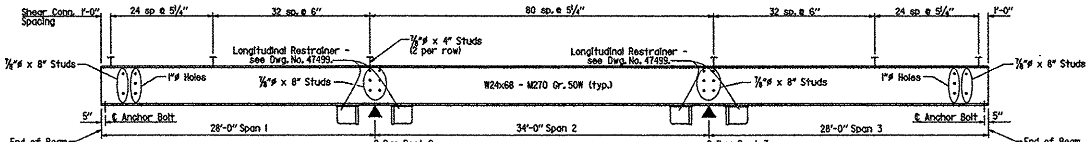
8

DATE REVISED	DATE FILMED	DATE REVISED	DATE FILMED	FED. ROAD DIST. NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
				6	ARK.			
				JOB NO.		100566	86	223
				(1) 07039, 07041		90' UNIT		47497



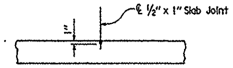
TYPICAL ROADWAY SECTION

⅜" = 1'-0"



TYP. BEAM ELEVATION

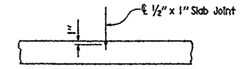
No Scale



Use Type 6 Joint Sealer. See Subsections 501.02(h) and 501.05(j). Joint Sealer shall be measured and paid for as Class S(AE) Concrete-Bridge. Slab joints shall extend to the outside edge of the deck slab. Slab joints shall be installed before the parapet railing is poured. If slab joints are to be sawed, they shall be sawed as soon as the concrete has sufficiently set to allow sawing of the joint without damage to the slab. Slab joints shall be placed at all pouring sequence construction joints and required slab joint locations.

SLAB JOINT DETAIL

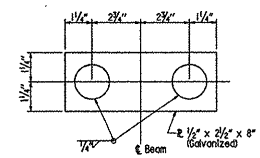
No Scale



½" X 1" Type 6 Joint Sealer. See Sections 501.02(h) and 501.05(j) of the Standard Specifications. Joint Sealer shall be measured and paid for as Class S(AE) Concrete. This joint shall be formed.

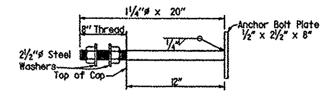
LONGITUDINAL CONSTRUCTION JOINT

No Scale



BENT 1 & 4 ANCHOR BOLT PLATE

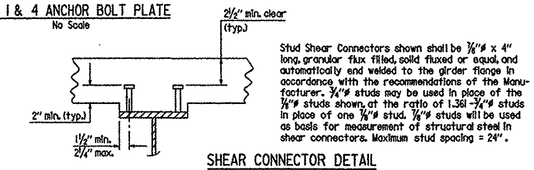
No Scale



BENT 1 & 4 ANCHOR BOLT DETAIL

No Scale

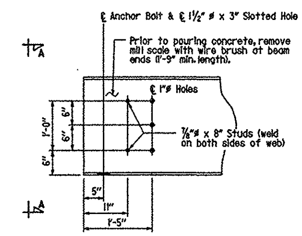
NOTE: Anchor bolts, nuts and washers shall conform to Subsection 807.07. All anchor bolts shall be grade 55.



Stud Shear Connectors shown shall be ⅞"ø x 4" long, granular flux filled, solid fluxed or equal, and automatically end welded to the girder flange in accordance with the recommendations of the Manufacturer. ¾"ø studs may be used in place of the ⅞"ø studs shown, at the ratio of 1.36 - ¾"ø studs in place of one ⅞"ø stud. ⅞"ø studs will be used as basis for measurement of structural steel in shear connectors. Maximum stud spacing = 24".

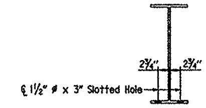
SHEAR CONNECTOR DETAIL

No Scale



DETAILS OF BEAM END (TYP.)

No Scale



VIEW A-A

No Scale

STATE OF ARKANSAS REGISTERED PROFESSIONAL ENGINEER No. 4337 CHARLES P. BRAND

BRIDGE ENGINEER

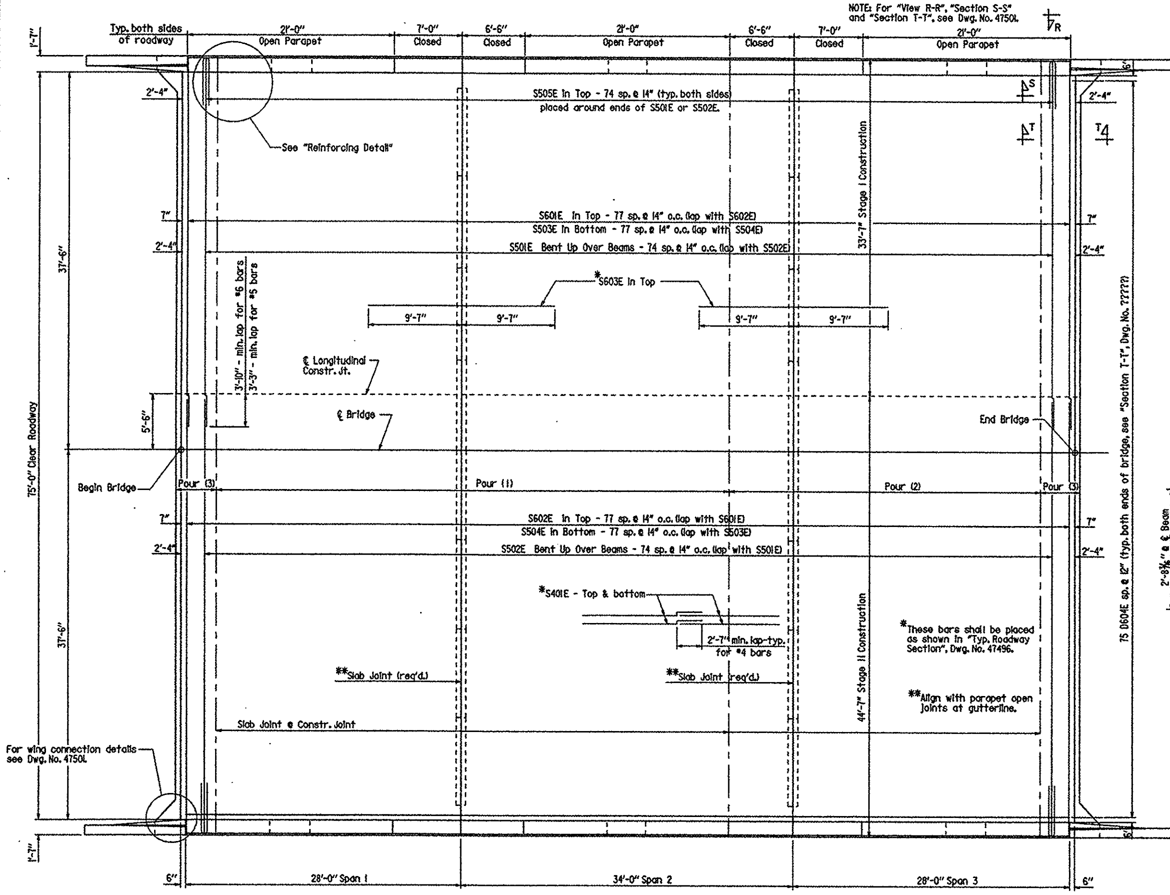
SHEET 2 OF 6
DETAILS OF 90' INTEGRAL
W-BEAM UNIT
LOCUST CREEK RELIEF
& MUD SLOUGH DITCH
ROUTE SEC.
ARKANSAS STATE HIGHWAY COMMISSION
LITTLE ROCK, ARK.

DRAWN BY: K.W.Y. DATE: 2-18-05 FILENAME: b100566x2.s12.dgn
CHECKED BY: MAH DATE: 4-25-05 SCALE: as noted
DESIGNED BY: KWY DATE: 8-04
BRIDGE NO. 07039, 07041 DRAWING NO. 47497

9

DATE REVISED	DATE FILMED	DATE REVISED	DATE FILMED	FED. ROAD DIST. NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
				6	ARK.			
				JOB NO.		100566	87	223
				① 07039, 07041		90' UNIT		47498

NOTE: For "View R-R", "Section S-S" and "Section T-T", see Dwg. No. 4750L

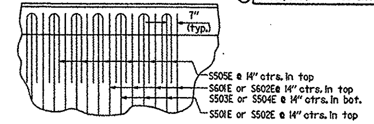


REINFORCING PLAN & DECK POURING SEQUENCE

3/8" = 1'-0"

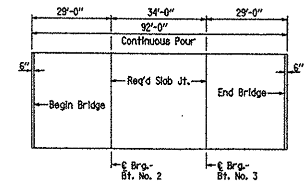
NOTE: Pour (1) must be placed before Pour (2) can be placed. Pour (2) must be placed before Pours (3) can be placed. 48 hours shall elapse between the end of a pour and the start of the next pour. 72 hours shall elapse between the end of a pour and the start of an adjacent pour. Any railing pours made before the entire slab unit has been placed must be approved by the Bridge Engineer. The Contractor must obtain approval from the Bridge Engineer for any deviations from the pouring sequence.

NOTE: The external load plates at Bents 2 & 3 shall be welded to the bottom beam flange prior to any deck pours.



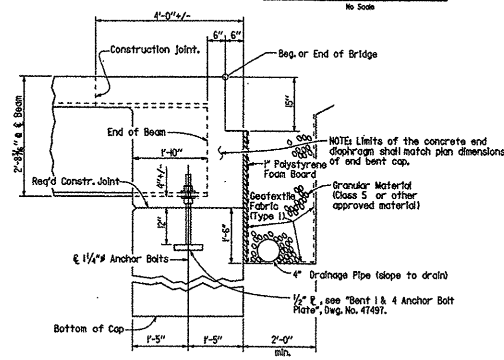
REINFORCING DETAIL

No Scale



ALTERNATE POURING SEQUENCE

No Scale



NOTE: For additional details of pipe underdrain see Std. Dwg. No. PU-1 and section 611 of the Standard Specifications. Pipe underdrains will not be measured or paid for separately, but will be considered subsidiary to the unit price bid for "Unclassified Excavation". 1" polystyrene foam board, filter fabric and granular material shall be considered subsidiary to the various bid items.

SECTION AT END BENT

No Scale

NOTE: Concrete diaphragms shall be poured monolithically with the bridge deck.

STATE OF ARKANSAS REGISTERED PROFESSIONAL ENGINEER No. 4337 CHARLES P. BRAND 4/28/05

BRIDGE ENGINEER

SHEET 3 OF 6

DETAILS OF 90' INTEGRAL W-BEAM UNIT

LOCUST CREEK RELIEF & MUD SLOUGH DITCH

ROUTE SEC.

ARKANSAS STATE HIGHWAY COMMISSION

LITTLE ROCK, ARK.

DRAWN BY: K.W.Y. DATE: 2-25-05 FILENAME: bl00566x2_sf3.dgn

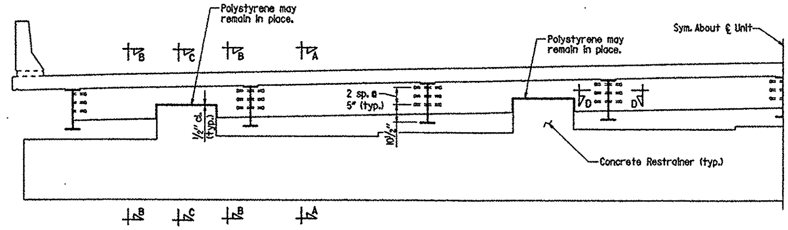
CHECKED BY: MAH DATE: 4-18-05 SCALE: as noted

DESIGNED BY: KWY DATE: 8-04

BRIDGE NO. 07039, 07041 DRAWING NO. 47498

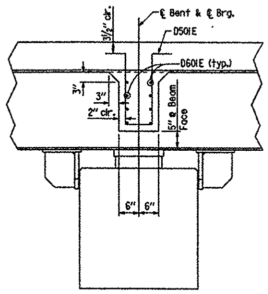
DATE REVISED	DATE FILMED	DATE REVISED	DATE FILMED	FED. ROAD DIST. NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
				6	ARK.			
				JOB NO.		100566	88	223
				07039, 07041		90' UNIT		47499

NOTE: ½" polystyrene shall be used as a bond breaker between the concrete restrainer and the concrete diaphragm and may remain in place.

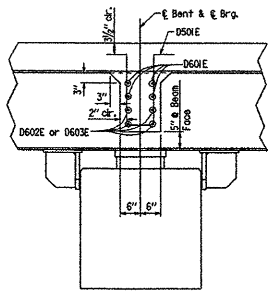


ROADWAY SECTION AT INT. BENTS SHOWING CONCRETE RESTRAINERS

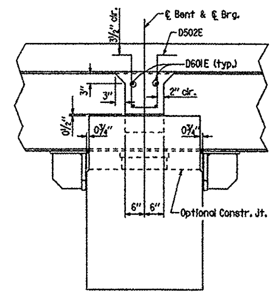
Looking Ahead
Scale: ⅜" = 1'-0"



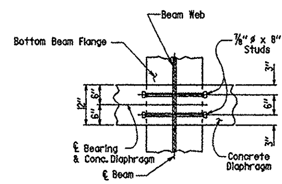
SECTION A-A
No Scale



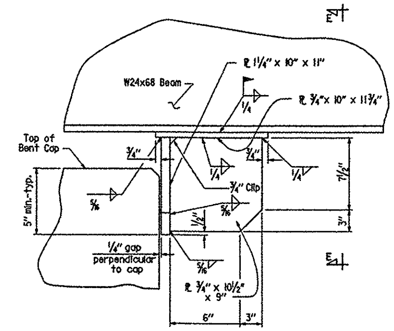
SECTION B-B
No Scale



SECTION C-C
No Scale

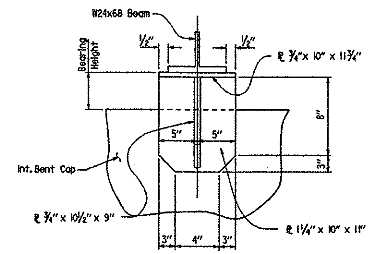


SECTION D-D
No Scale



LONGITUDINAL RESTRAINER DETAIL
No Scale

NOTE: Weld longitudinal restrainer after deck has been poured.



VIEW E-E
No Scale

WELD TABLE

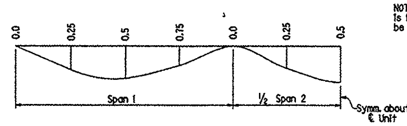
Material Thickness of Thicker Part Joined (Inches)	Minimum Size of Fillet Weld (Inches)	Single Pass Weld Must Be Used
To ¾" Inclusive	¼"	
Over ¾"	⅝"	

NOTE: When a fillet weld size, as shown on the plans, is larger than the minimum, the first pass shall be that specified for minimum size of fillet weld.

TABLE OF DEAD LOAD DEFLECTIONS (INCHES)

Span	Point of Deflection	Structural Steel		Structural Steel + Slab		Structural Steel + Slab + Parapet	
		Beams 1 & 9	Beams 2 - 8	Beams 1, 5 & 9	Beams 2-4, 6-8	Beams 1, 5 & 9	Beams 2-4, 6-8
1	0	0	0	0	0	0	0
1	0.25	0.007	0.008	0.073	0.087	0.075	0.088
1	0.5	0.008	0.009	0.089	0.105	0.091	0.107
1	0.75	0.004	0.005	0.043	0.050	0.044	0.051
2	1.0	0	0	0	0	0	0
2	1.25	0.005	0.006	0.052	0.061	0.053	0.062
2	1.5	0.009	0.010	0.093	0.110	0.095	0.112

NOTE: This table is symmetrical about the ℄ of Unit.



DEAD LOAD DEFLECTION DIAGRAM

NOTE: Camber for Dead Load Deflection plus Vertical curve ±¼" tolerance. Deflections shown are from a chord from ℄ Bearing to ℄ Bearing. Vertical curve corrections not included. Negative sign (-) indicates point above chord.

STATE OF ARKANSAS
REGISTERED PROFESSIONAL ENGINEER
No. 4327
4/28/05
CHARLES P. BRAND

BRIDGE ENGINEER

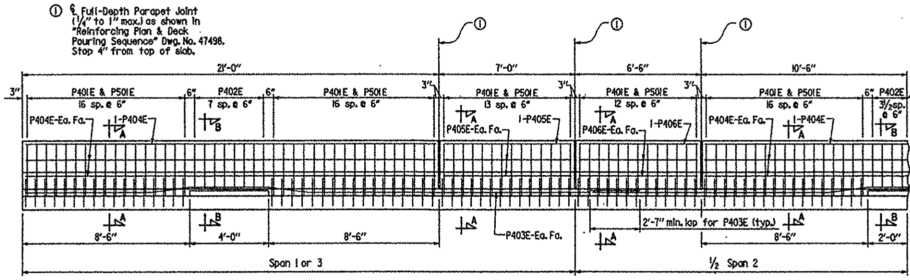
SHEET 4 OF 6
DETAILS OF 90' INTEGRAL
W-BEAM UNIT
LOCUST CREEK RELIEF
& MUD SLOUGH DITCH
ROUTE SEC.
ARKANSAS STATE HIGHWAY COMMISSION
LITTLE ROCK, ARK.

DRAWN BY: K.W.Y. DATE: 3-7-05 FILENAME: bl00566x2_s4.dgn
CHECKED BY: MAH DATE: 4-28-05 SCALE: as noted
DESIGNED BY: KWY DATE: 8-04
BRIDGE NO. 07039, 07041 DRAWING NO. 47499

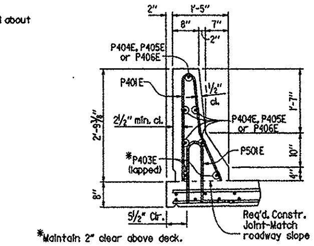
11

DATE REVISED	DATE FILMED	DATE REVISED	DATE FILMED	FED. ROAD DIST. NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
				6	ARK.			
				JOB NO.		100566	89	223
				① 07039, 07041		90' UNIT		47500

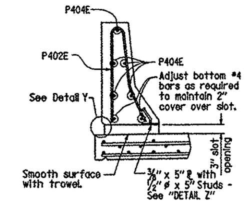
① ℄ Full-Depth Parapet Joint (1/4" to 1" max.) as shown in "Reinforcing Plan & Deck Pouring Sequence" Dwg. No. 47498. Stop 4" from top of slab.



DETAILS OF PARAPET RAIL
3/8" = 1'-0"



SECTION A-A
3/4" = 1'-0"



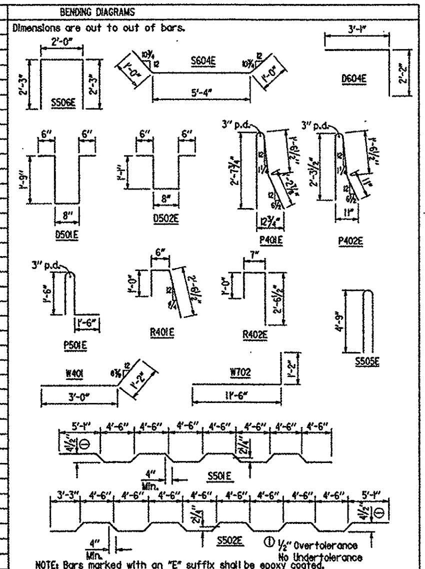
SECTION B-B
3/4" = 1'-0"

BAR LIST

MARK	NO. REQ'D.	LENGTH	P.D.
S401E	579	32'-1"	Str.
S402E	16	36'-0"	Str.
S403E	16	44'-5"	Str.
S501E	75	37'-4"	3"
S502E	75	45'-3"	3"
S503E	78	36'-8"	Str.
S504E	78	44'-5"	Str.
S505E	150	9'-11"	7"
S506E	158	6'-4"	2 1/2"
S601E	78	37'-3"	Str.
S602E	78	44'-5"	Str.
S603E	180	19'-2"	Str.
S604E	8	7'-4"	4 1/2"
D501E	144	4'-9"	2 1/2"
D502E	32	3'-5"	2 1/2"
D601E	96	8'-8	Str.
D602E	32	3'-10"	Str.
D603E	32	1'-4"	Str.
D604E	150	5'-1"	4 1/2"
P401E	312	5'-6"	2"
P402E	48	4'-10"	2"
P403E	12	31'-9"	Str.
P404E	30	20'-8"	Str.
P405E	20	6'-8"	Str.
P406E	20	6'-2"	Str.
P501E	312	4'-7"	3 3/4"
R401	16	3'-11"	2"
R402	16	4'-0"	2"
R403	24	9'-8"	Str.
R404	24	4'-0"	Str.
R601	32	5'-5"	Str.
R602	12	5'-0"	Str.
W401	20	4'-2"	2"
W402	20	5'-5"	Str.
W701	12	12'-10"	Str.
W702	24	12'-6"	5 1/4"

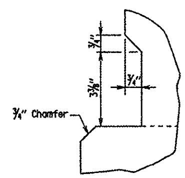
BENDING DIAGRAMS

Dimensions are out to out of bars.

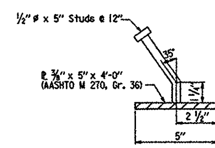


① 1/2" Overtolerance No Undertolerance

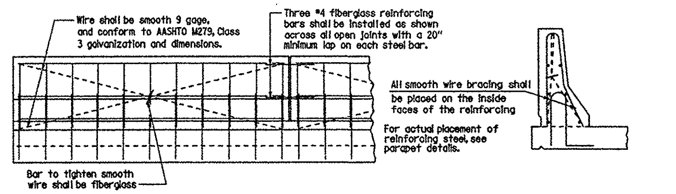
NOTE: Bars marked with an "E" suffix shall be epoxy coated.



DETAIL Y
No Scale



DETAIL Z
No Scale



All panels shall be braced as required to prevent racking. All open joints shall be sawed as soon as practical to a minimum width of 1/4". To control cracking before sawing all joints must be grooved before the concrete is set. Sawing of the joints must be controlled so it will follow the grooved joint.

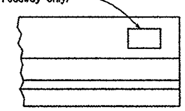
The extruded parapet shall conform to the horizontal and vertical lines shown on the plans or as directed by the Engineer and shall present a smooth, uniform appearance and texture.

DETAILS OF OPTIONAL SLIPFORMING OF CONCRETE PARAPET RAIL
No Scale

NOTE: The surfaces of the 3/8" plates which will not be in contact with concrete shall be painted with aluminum epoxy paint in accordance with Section 638, or as approved by the Engineer. Only one coat is required and shall be applied in the fabricator's shop. Painting will not be paid for directly, but will be considered subsidiary to "Structural Steel in Beam Spans (M270, Gr. 50W)."

Parapet studs shall be 5" long., granular flux filled, solid fluxed or equal, and automatically end welded to the plate. Studs and plates shall meet the requirements of Section 807 and shall be measured and paid for as "Structural Steel in Beam Spans (M270, Gr. 50W)."

Place Type D Bridge Name Plate on front face of span rail approx. 1'-6" from beginning of bridge. (Right side of roadway only)



NAME PLATE DETAIL
No Scale

STATE OF ARKANSAS
REGISTERED PROFESSIONAL ENGINEER
No. 4337
CHARLES P. BRAND

BRIDGE ENGINEER

SHEET 5 OF 6
DETAILS OF 90' INTEGRAL
W-BEAM UNIT
LOCUST CREEK RELIEF
& MUD SLOUGH DITCH
ROUTE SEC.
ARKANSAS STATE HIGHWAY COMMISSION
LITTLE ROCK, ARK.

DRAWN BY: K.W.Y. DATE: 3-4-05 FILENAME: bl00566x2_sl5.dgn
CHECKED BY: MAH DATE: 4-28-05 SCALE: as noted
DESIGNED BY: KWY DATE: 8-04
BRIDGE NO. 07039, 07041 DRAWING NO. 47500

12

DATE REVISED	DATE FILMED	DATE REVISED	DATE FILMED	FED. ROAD DIST. NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
				6	ARK.			
				JOB NO.		100566	90	223
				① 07039, 07041		90' UNIT		47501

GENERAL NOTES

CONSTRUCTION SPECIFICATIONS: Arkansas State Highway and Transportation Department Standard Specifications for Highway Construction, 2003 edition, with applicable supplemental specifications and special provisions. Unless otherwise noted, section and subsection numbers in the plans refer to the Standard Construction Specifications.

DESIGN SPECIFICATIONS: AASHTO LRFD Bridge Design Specifications, 2004 edition.

All concrete shall be Class S(AE) and shall be poured in the dry. All exposed corners to be chamfered ¾" unless otherwise noted.

All concrete shall be poured and screeded off prior to initial set. The concrete deck shall be finished in accordance with Subsection 802.19, Class 5 of the Standard Specifications. Movement of the finishing machine across new concrete shall be on planks placed on the surface and shall be prohibited for 72 hours after finishing the pour. Sufficient concrete must be placed ahead of the strike-off to fully load the beam. If a longitudinal strike-off is used, a vertical camber adjustment must be made in the strike-off to account for the future dead load deflection of the railing.

Concrete in bridge superstructure shall be placed and consolidated for the entire pour before any concrete has taken its initial set. This may require the use of a retarding agent.

Reinforcing steel shall conform to AASHTO M31 or M53, Grade 60. The reinforcing is to be accurately located in the forms and firmly held in place by steel wire supports, sufficient in number and size to prevent displacement during the course of construction. The wire supports will not be paid for directly but will be considered subsidiary to the item "Epoxy Coated Reinforcing Steel-Gr. 60".

All stud shear connectors shall be granular flux filled, solid fluxed, or equal, and shall be automatically end welded in accordance with recommendations of the manufacturer.

Field connections shall be bolted with ¾" high strength bolts unless otherwise noted. Bolt holes shall be 13/16"ϕ except that 15/16"ϕ holes may be used for connection of expansion devices, diaphragms and end struts if a washer is used under both the nut and head of the bolt.

Steel diaphragms shall be installed as beams are erected and shall be completely bolted, except as noted, prior to pouring of the concrete deck.

Drawings show general features of design only. Shop drawings shall be made in accordance with the specifications, submitted and approval secured before any fabrication is begun. Structural shapes of equal or greater strength may be substituted for shapes shown if approval is obtained from the Bridge Engineer. Payment will be made on the basis of shapes shown.

All Structural Steel shall be AASHTO M270, Gr. 50W, unless otherwise noted and shall be paid for at the unit price per pound bid for "Structural Steel in Beam Spans (M270, Gr. 50W)". M270, Gr. 50W, steel shall not be painted. All structural steel shall be cleaned in accordance with Subsection 807.04 unless otherwise noted. Structural steel completely embedded in concrete may be AASHTO M270, Gr. 36. See Dwg. No. 47489 for cleaning requirements of external load plates on elastomeric bearings.

All beams shall be blocked in their true position in the shop. The camber, length of sections and distance between bearings shall be measured with the beams in this position and this information shall become a part of the permanent record of the job. The component parts shall be match marked in this assembly and these marks shall be shown on the erection diagram. All beam dimensions are based on a temperature of 60°F. A tolerance of ±¼" is allowed for camber.

Beams are considered main load carrying members and shall meet the longitudinal Charpy V-Notch test specified in Section 807.05. All welding shall conform to Subsection 807.26. Welded connections shall be 5/16" fillet shop welds unless otherwise noted. All welding that is to be done during fabrication of structural steel, including temporary welds, shall be detailed on the shop drawings and submitted for approval. If the Contractor or Erector should want to make additional welds, whether temporary or permanent, he shall submit detailed drawings with formal request to the Bridge Engineer for approval.

Bearings shall be seated in accordance with Subsection 807.66.

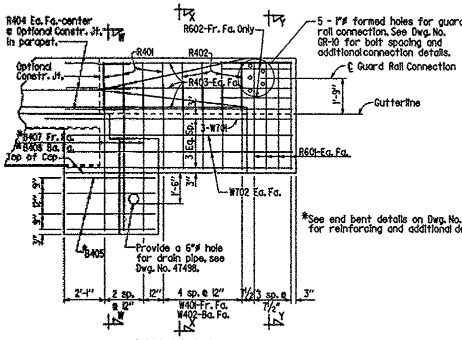
MATERIALS AND STRENGTHS:

Class S(AE) Concrete	f'c = 4,000 psi.
Reinforcing Steel (AASHTO M31 or M53, Gr. 60)	fy = 60,000 psi.
Structural Steel (AASHTO M270, Gr. 50W)	Fy = 50,000 psi.
Structural Steel (AASHTO M270, Gr. 36)	Fy = 36,000 psi.

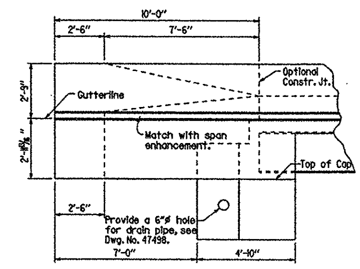
LOAD DISTRIBUTION:

Dead Load:	Beam No.	
A. To W-Beam	1, 9	756 plf + Wt. of Structural Steel
	2, 3, 6, 7, 8	900 plf + Wt. of Structural Steel
	4	800 plf + Wt. of Structural Steel
	5	725 plf + Wt. of Structural Steel
B. To Composite Beam	1, 2, 3, 4	410 plf **
	5, 6, 7, 8, 9	243 plf **

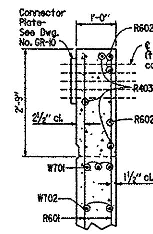
** Includes 200 plf future wearing surface



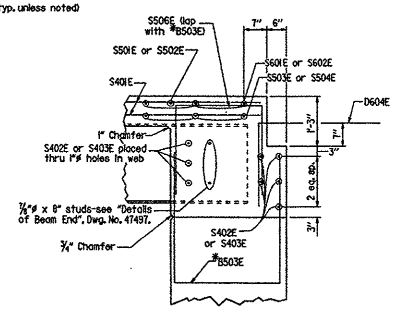
SECTION S-S
⅜" = 1'-0"



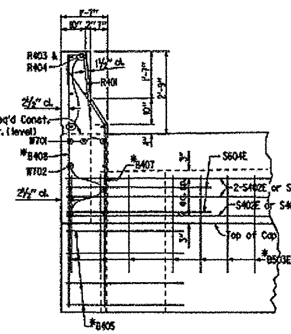
VIEW R-R
⅜" = 1'-0"



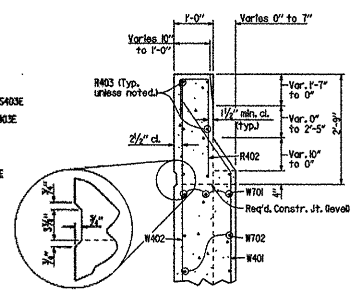
SECTION Y-Y
¾" = 1'-0"



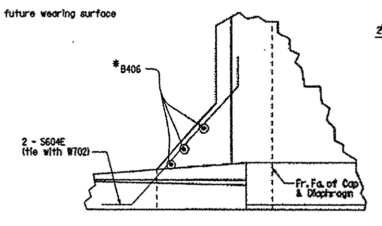
SECTION T-T
¾" = 1'-0"



SECTION W-W
No Scale



SECTION X-X
¾" = 1'-0"



WING CONNECTION
No Scale

STATE OF ARKANSAS
REGISTERED PROFESSIONAL ENGINEER
No. 4337
CHARLES P. BRAND
BRIDGE ENGINEER

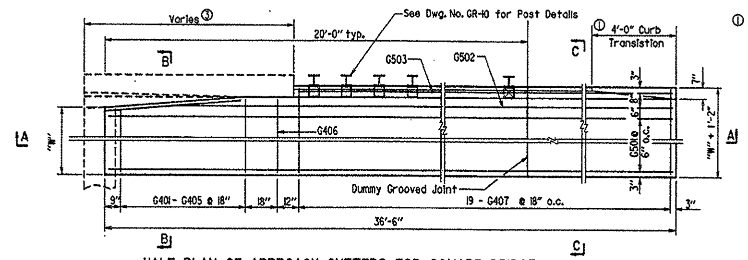
SHEET 6 OF 6
DETAILS OF 90' INTEGRAL W-BEAM UNIT
LOCUST CREEK RELIEF & MUD SLOUGH DITCH
ROUTE SEC.
ARKANSAS STATE HIGHWAY COMMISSION
LITTLE ROCK, ARK.

DRAWN BY: K.W.Y. DATE: 3-7-05 FILENAME: b100566x2_s6.dgn
CHECKED BY: MAH DATE: 4-26-05 SCALE: as noted
DESIGNED BY: KWY DATE: 8-04
BRIDGE NO. 07039, 07041 DRAWING NO. 47501

13

DATE REVISED	DATE FILMED	DATE REVISED	DATE FILMED	FED. ROAD DIST. NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
4-10-2003				6	ARK.			
				JOB NO.			107	

① TYPE C GUTTERS - 2016C

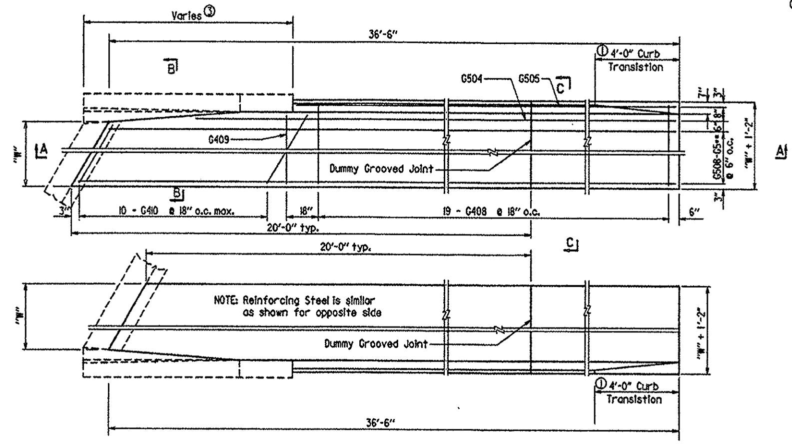


HALF PLAN OF APPROACH GUTTERS FOR SQUARE BRIDGE

① Construct gutter curb with height-transition as shown if drop inlet is not placed at end of gutter.

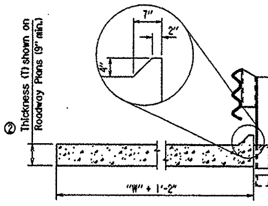
Construct gutter curb full height (no height-transition) if drop inlet is placed at end of gutter. Curb height transition placed on drop inlet. See drop inlet details.

③ Length Varies See End Bt. Details for Actual Length. Quantities Shown are for 10'-0" Transition Rail.

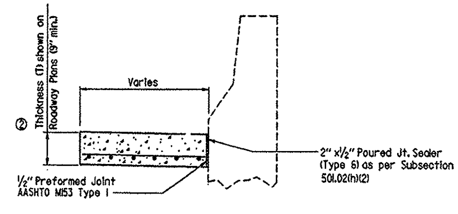


PLAN OF APPROACH GUTTERS FOR SKEWED BRIDGE

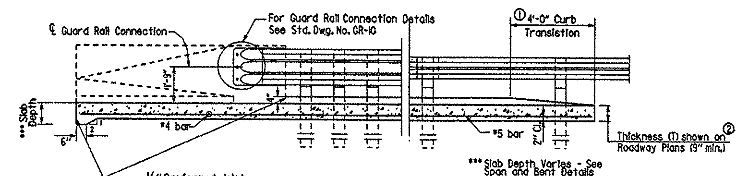
② Thickness shall match Approach Slab Thickness. Thickness shall be 9" if Approach Slab is not used.



SECTION C - C
N.T.S.



SECTION B - B
N.T.S.



SECTION A - A

***Special bar list required when skew angle exceeds 35° for W = 10'; 40° for W = 8'; 50° for W = 6'; or 60° for W = 4'.

***BAR LIST FOR ONE TYPE C GUTTER

Mark	No. Req'd. for Width "W"				Length	Square or Skewed
	4'-0"	6'-0"	8'-0"	10'-0"		
G401 - G405	1 each	1 each	1 each	1 each	"W"-3" to "W"+3"	Square
G406	1	1	1	1	"W"+3"	Square
G407	19	19	19	19	"W"+ 10"	Square
G408	19	19	19	19	"W"+ 10"	Skewed
G409	1	1	1	1	"W"+3"	Skewed
G410	10	10	10	10	*	Skewed
G501	8	12	16	20	36'-2"	Square
G502	1	1	1	1	31'-8"	Square
G503	1	1	1	1	27'-2"	Square
G504	1	1	1	1	*	Skewed
G505	1	1	1	1	*	Skewed
G508 - G5__ **	1 each	1 each	1 each	1 each	*	Skewed

* Bar Lengths vary with Skew.

** G515 for W = 4'
G519 for W = 6'
G523 for W = 8'
G527 for W = 10'

GENERAL NOTES

Concrete shall be Class S or Class S(AE) or mixture used for Portland Cement Concrete Pavement.

Reinforcement Steel shall conform to AASHTO M31or M53, Grade 60 (fy = 60,000 psi).

Approach Gutters will be measured and paid for in accordance with Section 504 of the Standard Specifications.

△ Revised and redrawn 4-10-2003. By KDH Ck. By: CJF

QUANTITIES FOR ONE SQUARE APPROACH GUTTER ③

"W" Width (ft.)	Reinforcing Steel (lbs.)	Concrete (cubic yards)				
		T=9"	T=10"	T=11"	T=12"	T=14 1/2"
4	439	5.19	5.75	6.31	6.88	8.25
6	623	7.24	8.02	8.80	9.59	11.52
8	807	9.28	10.29	11.30	12.32	14.79
10	991	11.33	12.56	13.79	15.03	18.06

STATE OF ARKANSAS
REGISTERED PROFESSIONAL ENGINEER
No. 4837
CHARLES P. BRAND

BRIDGE ENGINEER

DETAILS OF STANDARD
TYPE C APPROACH GUTTERS

ROUTE SEC.

ARKANSAS STATE HIGHWAY COMMISSION

LITTLE ROCK, ARK.

DRAWN BY: KDH DATE: 4-10-2003 FILENAME: B2016C.STD
CHECKED BY: CJF DATE: 4-10-2003 SCALE: 3/8" = 1'-0"
DESIGNED BY: STD. DATE:
BRIDGE NO. DRAWING NO. 2016C

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Step #

The seismic design of "Mud Slough Ditch" is divided into 11 steps, which are shown on this and the following sheets.

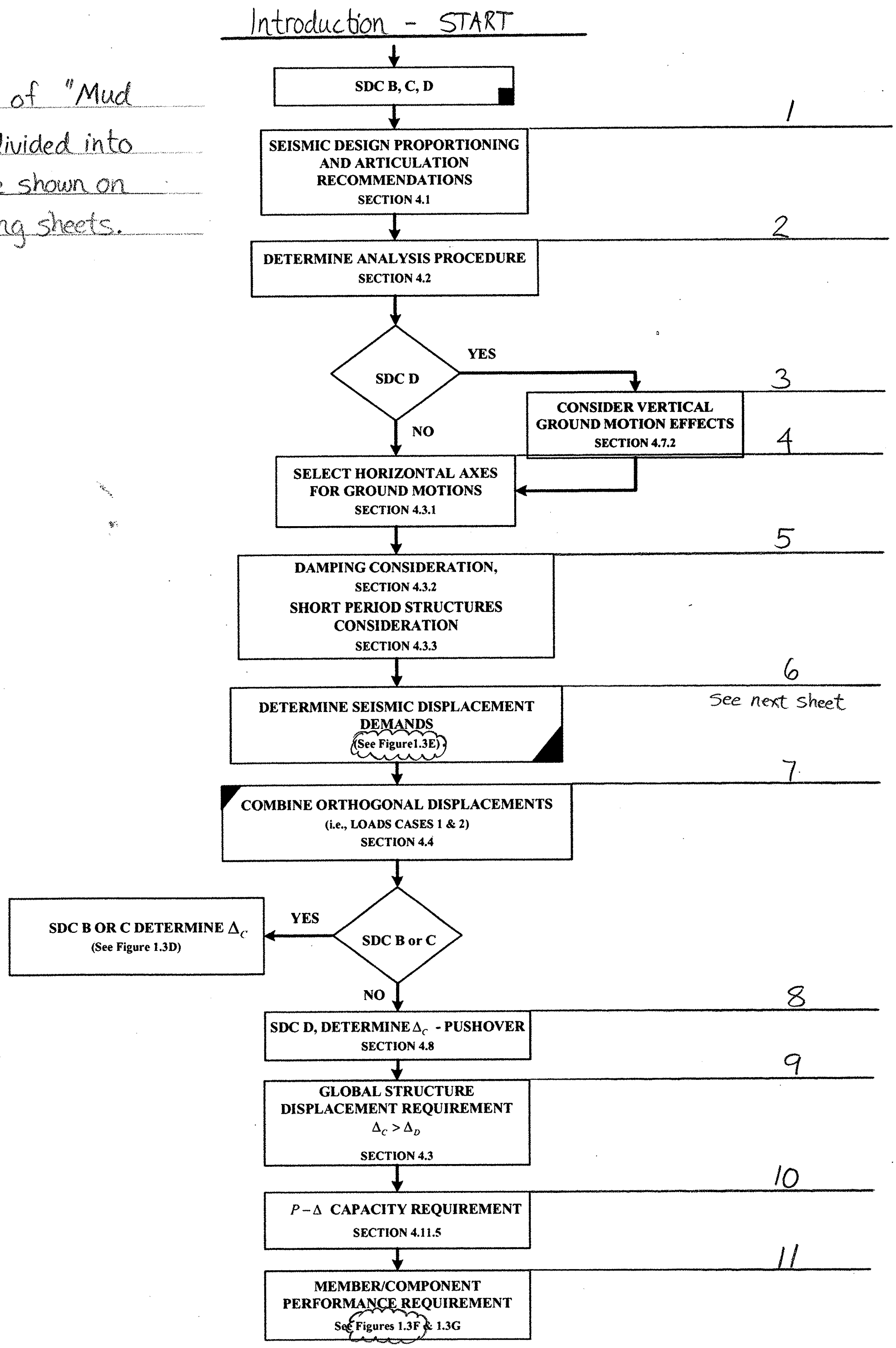


FIGURE 1.3C: Design Procedure Flow Chart C

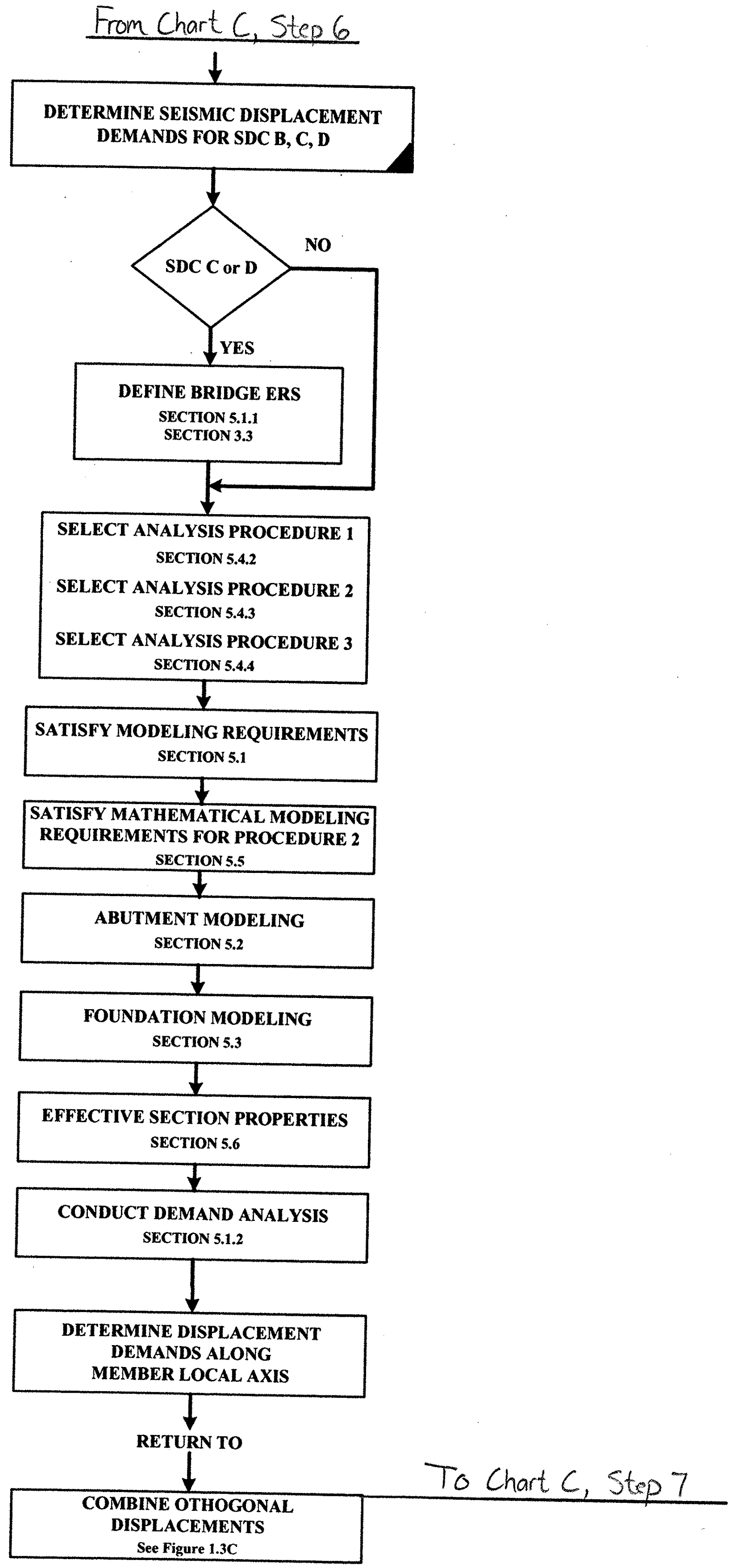


FIGURE 1.3E: Design Procedure Flow Chart E

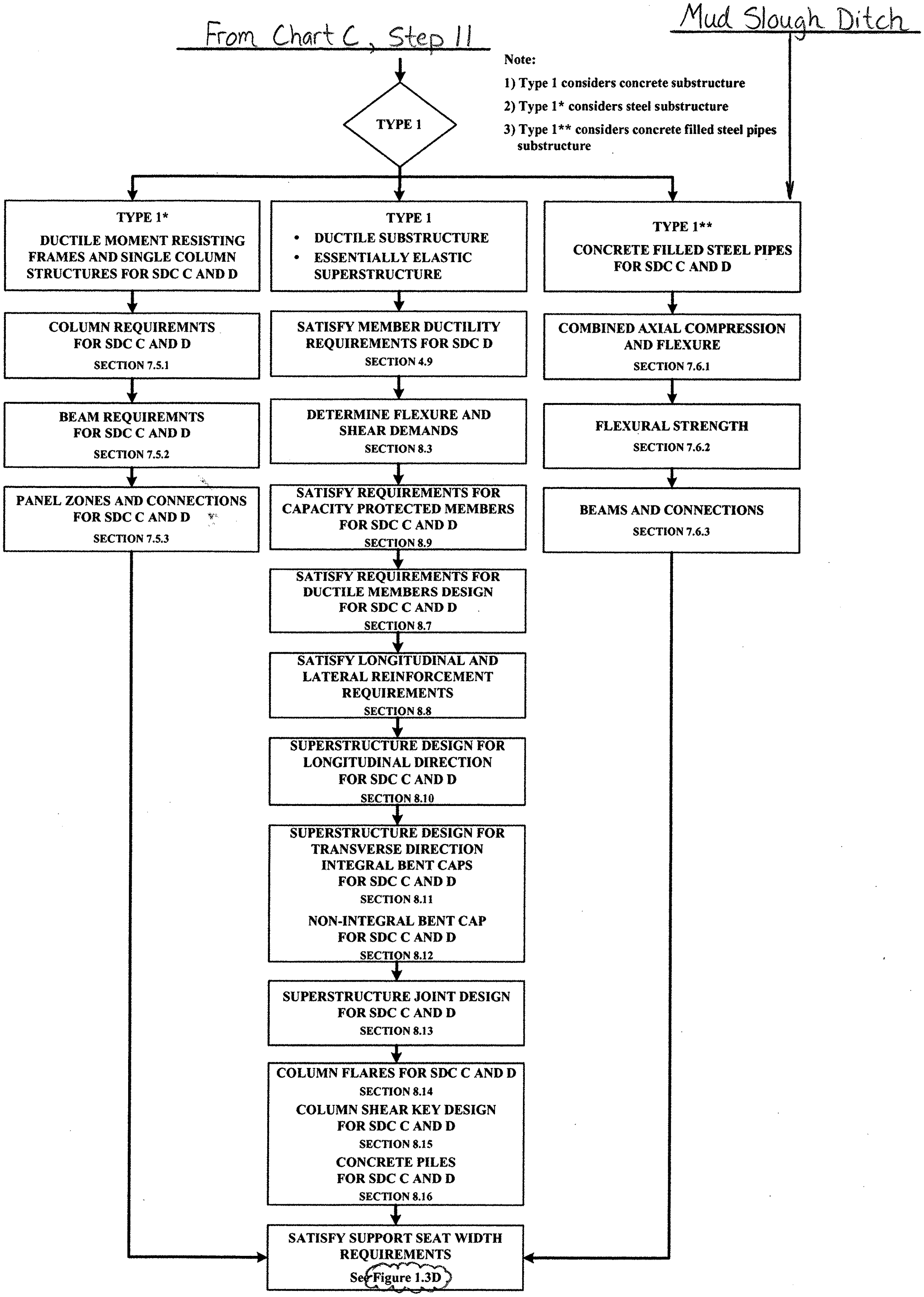


FIGURE 1.3F: Design Procedure Flow Chart F

From Chart F (Part of Chart C, Step 11)

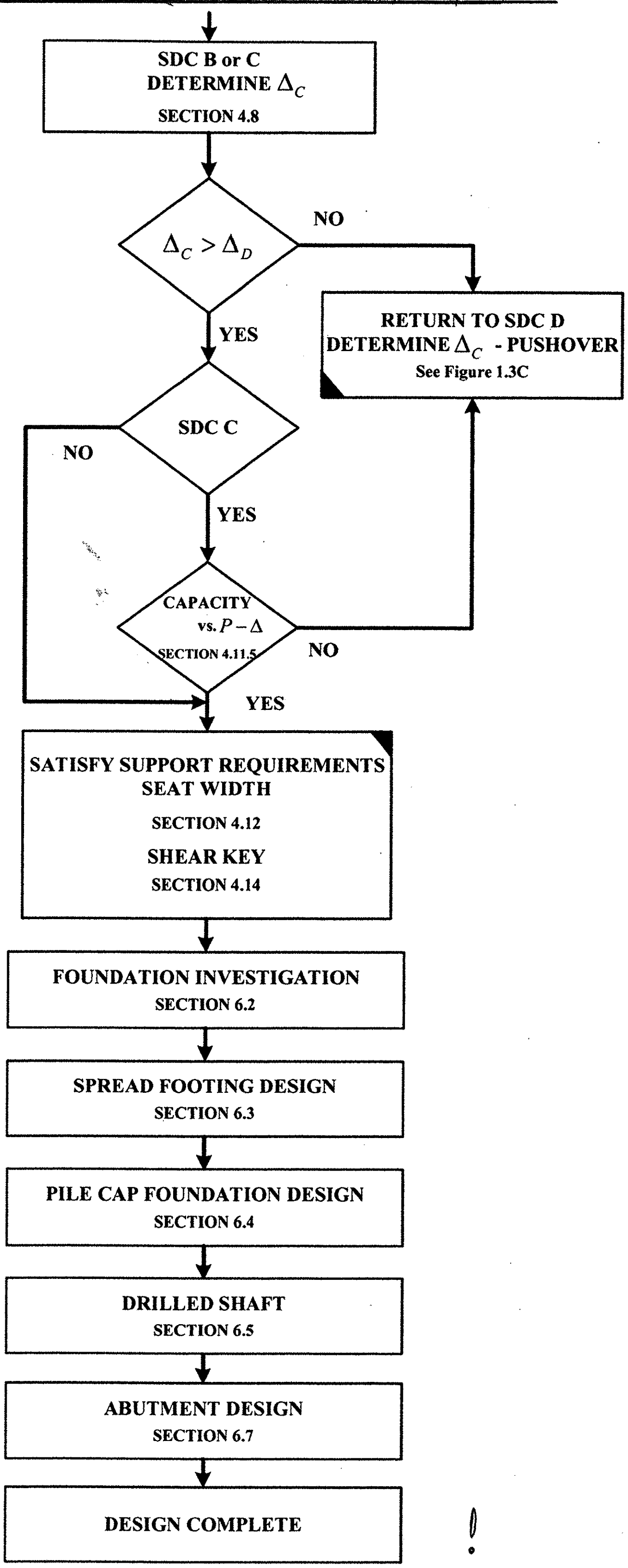


FIGURE 1.3D: Design Procedure Flow Chart D

Seismic Evaluation of 100566

Features
90' Three Span Continuous Spans (28' - 34' - 28')
No Skew
Composite W-Beam
Trestle Pile Bent (9 Columns)
Integral Pile Abutment
Steel Shell Piles
Jointless w/ Elasto Bearings @ Int. Bents
Existing Bridge located at Lat. = 36.0503   Long. = -90.3689
Old plans (2003 AASHTO) used A=0.226 which is Seismic Zone 3.

New Seismic Guidelines

Soil borings indicate the average blowcount for the top 100' of soil is 22, which is Site Class D (Guidelines Table 3.4.2-1).

With location and site class D known, the new guidelines software gives:

$$S_{DS} := 1.204 \qquad S_{D1} := 0.533$$

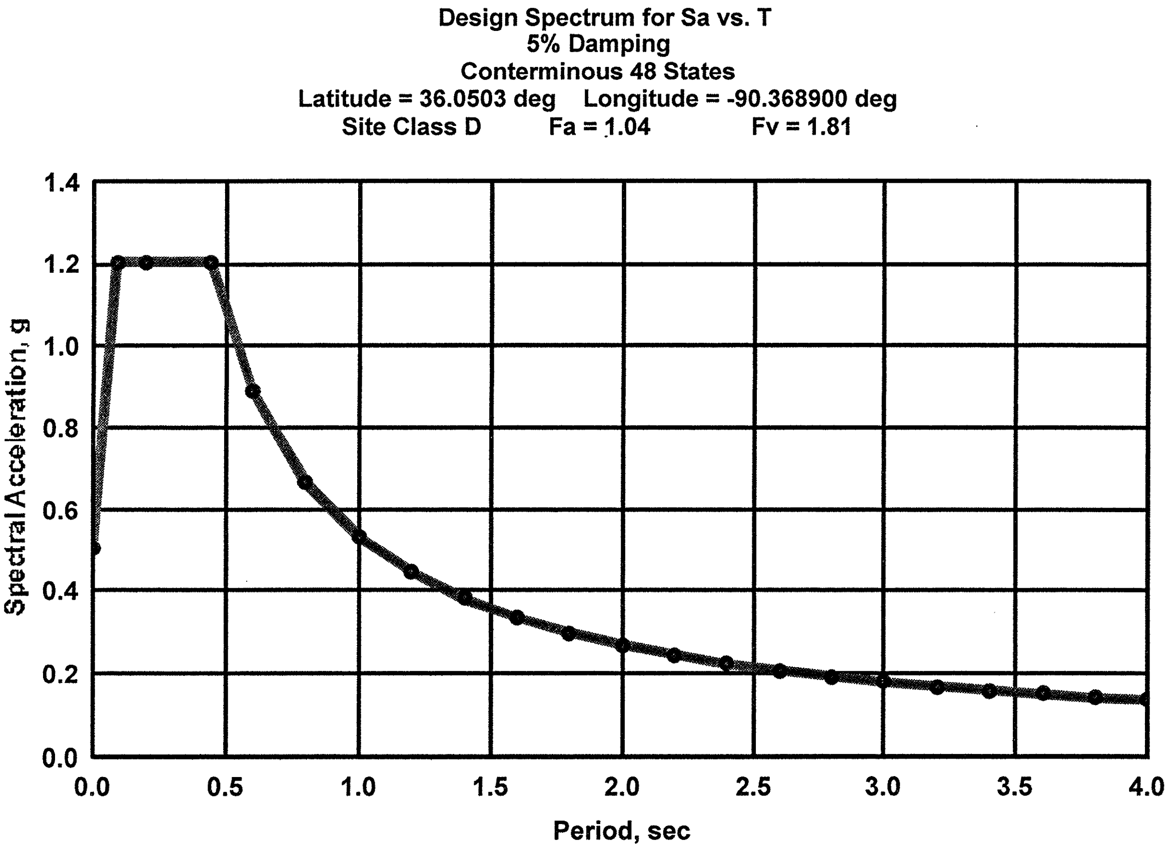
Since S_{D1} > 0.5, the bridge is **SDC D** (Table 3.5.1). The design requirements are as follows:

SDC D

a. Identification of ERS
b. Demand Analysis
c. Displacement Capacity required using Pushover Analysis (check P-Δ and seat width)
d. Capacity Design required including column shear requirement
e. SDC D level of detailing

Extreme Event 1 allows the designer to pick the gravity live load associated with an earthquake. For this project no live load will be assumed present during an earthquake.

See next page for a spectral analysis.



Spectrum ≈ A=0.48

Graph Data	
Period, sec	Sa, g
0.00	0.50
0.09	1.20
0.20	1.20
0.44	1.20
0.60	0.88
0.80	0.66
1.00	0.53
1.20	0.44
1.40	0.38
1.60	0.33
1.80	0.29
2.00	0.26
2.20	0.24
2.40	0.22
2.60	0.20
2.80	0.19
3.00	0.17
3.20	0.16
3.40	0.15
3.60	0.14
3.80	0.14
4.00	0.13

More information and definitions for fusing mechanism
" " T_F
" " M_w

Sectional Property Input for Seisab

$kip := 1000lb$

$ksi := \frac{kip}{in^2}$

MATERIALS $w_c := 0.145$ weight of concrete in kip/ft^3

$f_{c\_span} := 4$ $E_{c\_span} := 33000 \cdot w_c^{1.5} \cdot f_{c\_span}^{0.5} \cdot ksi$ $E_{c\_span} = 5.248 \times 10^5 \frac{kip}{ft^2}$

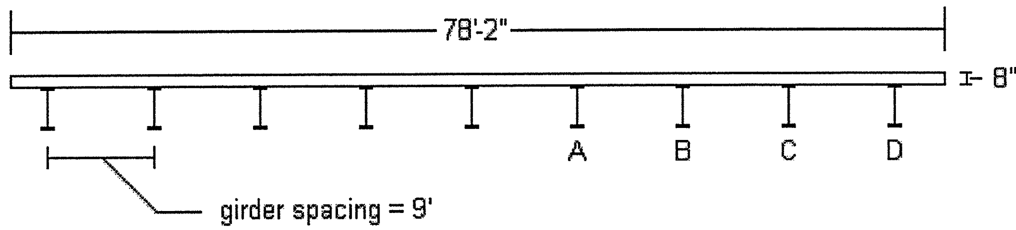
$f_{c\_sub} := 3.5$ $E_{c\_sub} := 33000 \cdot w_c^{1.5} \cdot f_{c\_sub}^{0.5} \cdot ksi$ $E_{c\_sub} = 4.909 \times 10^5 \frac{kip}{ft^2}$

$f_{c\_piles} := 3.5$ $E_{c\_piles} := 33000 \cdot w_c^{1.5} \cdot f_{c\_piles}^{0.5} \cdot ksi$ $E_{c\_piles} = 4.909 \times 10^5 \frac{kip}{ft^2}$

$f_s := 60ksi$ $E_s := 29000ksi$ $E_s = 4.176 \times 10^6 \frac{kip}{ft^2}$

$\nu := 0.2$ Poisson's ratio for steel is about 0.3 and for concrete 0.15. For concrete and steel compositely I will use 0.2.

Sectional Properties of **SPAN** for Seisab



$n := \dfrac{E_s}{E_{c\_span}}$ $n = 7.958$

n is about 8. For calculating sectional poperties, the concrete will be converted into steel by dividing its area by 8. The concrete centroid is still in the same position.

Steel Beam (W24x68):

$A_b := 20.1\text{in}^2$

$d_b := 23.75\text{in}$

$Ixo_b := 1830\text{in}^4$

$Iyo_b := 70.4\text{in}^4$

To find the composite properties of the span, most properties must be multiplied by the number of beams, 9.

Concrete Deck: $d_d := 8\text{in}$ $A_d := \dfrac{(78\text{ft} + 2\text{in})8\text{in}}{n}$

Element	Area	Moment Arm (y)	Ay	Ay²	Ixo
Deck	A_d	$MAy_d := \dfrac{d_b}{2} + 4\text{in}$	$Ay_d := A_d \cdot MAy_d$	$Ayy_d := A_d \cdot MAy_d^{\,2}$	$Ixo_d := \dfrac{(78\text{ft} + 2\text{in}) \cdot d_d^{\,3}}{12n}$
Beams	$9 \cdot A_b$	$MAy_b := 0\text{in}$	$Ay_b := 0\text{in}^3$	$Ayy_b := 0\text{in}^4$	$9 \cdot Ixo_b$
Total	$A_{tx} := A_d + 9 \cdot A_b$	$MAy_t := MAy_b + MAy_d$	$Ay_t := Ay_d + Ay_b$	$Ayy_t := Ayy_d + Ayy_b$	$Ixo_t := Ixo_d + 9 \cdot Ixo_b$

$I_{xs} := Ixo_t + Ayy_t$ $\quad I_{xs} = 2.591 \times 10^5\,in^4$ $\quad$ About centroid of the steel section.

Distance to centroid of composite section: $\quad \Delta y := \frac{Ay_t}{A_{tx}} \quad \Delta y = 13.32\,in$

The centroid is in the concrete deck since $\Delta y > d_b/2$. $\quad y_{bot} := \frac{d_b}{2} + \Delta y \quad y_{bot} = 25.195\,in$

y_{top} is the distance from the top of the concrete deck to the centroid. $\quad y_{top} := d_b + d_d - y_{bot} \quad y_{top} = 6.555\,in$

$I_x := I_{xs} - A_{tx}\cdot\Delta y^2 \quad I_x = 5.975 \times 10^4\,in^4 \quad I_{33s} := I_x \quad I_{33s} = 2.881\,ft^4$

Element	Area	Moment Arm	Ax	Ax²	Iyo
Deck	A_d	$MAx_d := 0in$	$Ax_d := A_d \cdot MAx_d$	$Axx_d := A_d \cdot MAx_d^2$	$Iyo_d := \frac{d_d\cdot(78ft + 2in)^3}{12n}$
Beam	A_b	$MAx_b := 0in$	$Ax_b := 0in^3$	$Axx_b := 0in^4$	Iyo_b
Beam A	$2A_b$	$MAx_{bA} := 9ft$	$Ax_{bA} := 0in^3$	$Axx_{bA} := 2\cdot A_b\cdot MAx_{bA}^2$	$2\cdot Iyo_b$
Beam B	$2A_b$	$MAx_{bB} := 18ft$	$Ax_{bB} := 0in^3$	$Axx_{bB} := 2\cdot A_b\cdot MAx_{bB}^2$	$2\cdot Iyo_b$
Beam C	$2A_b$	$MAx_{bC} := 27ft$	$Ax_{bC} := 0in^3$	$Axx_{bC} := 2\cdot A_b\cdot MAx_{bC}^2$	$2\cdot Iyo_b$
Beam D	$2A_b$	$MAx_{bD} := 36ft$	$Ax_{bD} := 0in^3$	$Axx_{bD} := 2\cdot A_b\cdot MAx_{bD}^2$	$2\cdot Iyo_b$
Total	$A_{ty} := A_d + 9\cdot A_b$				

$MAx_t := MAx_d + MAx_b + MAx_{bA} + MAx_{bB} + MAx_{bC} + MAx_{bD}$

$Ax_t := Ax_d + Ax_b + Ax_{bA} + Ax_{bB} + Ax_{bC} + Ax_{bD}$ $\quad Ax_t = 0\,in^3$

$Axx_t := Axx_d + Axx_b + Axx_{bA} + Axx_{bB} + Axx_{bC} + Axx_{bD}$ $\quad Axx_t = 1.407 \times 10^7\,in^4$

$Iyo_t := Iyo_d + 9Iyo_b$ $\quad Iyo_t = 6.914 \times 10^7\,in^4$

$I_y := Iyo_t + Axx_t$ $\quad I_y = 8.321 \times 10^7\,in^4$ $\quad I_{22s} := I_y$ $\quad I_{22s} = 4.013 \times 10^3\,ft^4$

$J := \dfrac{(78ft + 2in)\cdot d_d^{\,3}}{3}$ $\quad J = 1.601 \times 10^5\,in^4$ $\quad I_{11s} := J$ $\quad I_{11s} = 7.72\,ft^4$

SPAN WEIGHT

$parapet_{DL} := 5.2\cdot\dfrac{ft^3}{ft}\cdot\left(150\cdot\dfrac{lb}{ft^3}\right)\cdot 90ft$ $\quad parapet_{DL} = 70.2\,kip$

$diaphragm_{DL} := 9\cdot(9\cdot ft\cdot 8)\cdot 20.7\dfrac{lb}{ft}$ $\quad diaphragm_{DL} = 13.414\,kip$

$slab_{DL} := (78ft + 2in)\cdot(8in)\left(150\dfrac{lb}{ft^3}\right)\cdot 90ft$ $\quad slab_{DL} = 703.5\,kip$

$fws_{DL} := 2in\cdot 75ft\cdot 90ft\cdot 0.144\dfrac{kip}{ft^3}$ $\quad fws_{DL} = 162\,kip$

$steel_{DL} := 0.068\dfrac{kip}{ft}\cdot 9\cdot 90ft$ $\quad steel_{DL} = 55.08\,kip$

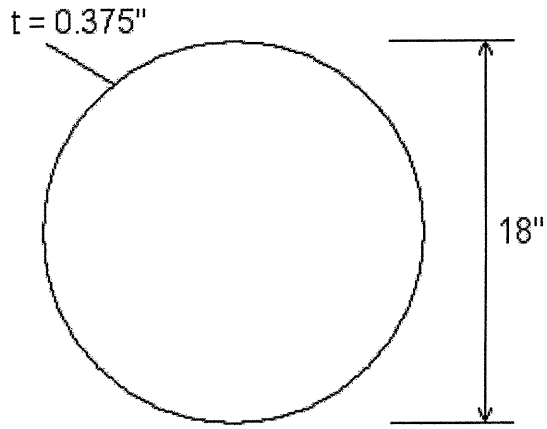
$total_{DL} := parapet_{DL} + diaphragm_{DL} + slab_{DL} + fws_{DL} + steel_{DL}$ $\quad total_{DL} = 1.004 \times 10^3\,kip$

The load input in seisab is most logical if a deck cross section area is entered and then misc loads are accounted for in the density of the deck.

$A_{deck} := A_d + 9\cdot A_b$ $\quad A_{deck} = 7.805\,ft^2$ $\quad$ Area of the deck and beams, with material stiffness transformed to steel.

$Density_{span} := \dfrac{total_{DL}}{A_{deck}\cdot 90ft}$ $\quad Density_{span} = 1.43\dfrac{kip}{ft^3}$ $\quad$ Replaces concrete weight of 0.15kcf

Elastic Section Properties of an 18" Pipe (Use for Seisab)



Assume a 1/16" reduction in wall thickness for corrosion.

$d := 18in - 2\left(\frac{1}{16}in\right)$ $\qquad$ $d = 17.875\,in$

$t := 0.4375in - \frac{1}{16}in$ $\qquad$ $t = 0.375\,in$

$d_{in} := d - 2t$ $\qquad$ $d_{in} = 17.125\,in$

$$I_{steel} := \left[\frac{\pi \cdot \left(\frac{d}{2}\right)^4}{4} - \frac{\pi \cdot \left(\frac{d_{in}}{2}\right)^4}{4}\right] \cdot n \qquad I_{steel} = 6.284 \times 10^3\,in^4$$

$$I_{conc} := \frac{\pi \cdot \left(\frac{d_{in}}{2}\right)^4}{4} \qquad I_{conc} = 4.222 \times 10^3\,in^4$$

See Guidelines C7.6 for the combined moment of inertia formula, p. 7-18. Also LRFD 6.9.5.

$I_{total} := I_{steel} + 0.4 I_{conc}$ $\qquad$ $I_{22p} := I_{total}$ $\quad$ $I_{33p} := I_{total}$ $\quad$ $I_{22p} = 0.384\,ft^4$

$A_{pipe} := \pi\left(\frac{d + d_{in}}{2}\right) \cdot t \cdot n$ $\qquad$ $A_{pipe} = 164.067\,in^2$ $\qquad$ Modified by n to change steel properties to concrete.

$A_{conc} := \pi \cdot \left(\frac{d_{in}}{2}\right)^2$ $\qquad$ $A_{conc} = 230.33\,in^2$

$A_{total} := A_{pipe} + A_{conc}$ $\qquad$ $A_{total} = 2.739\,ft^2$ $\qquad$ With concrete stiffness.

$J := 2 \cdot I_{total}$ $\qquad$ $I_{11p} := J$ $\qquad$ $I_{11p} = 0.769\,ft^4$

Elevation of Bridge Components

$Elev_{top} := 249ft - y_{top}$ $\qquad Elev_{top} = 248.454\,ft$

$Elev_{cap} := 245.26ft$

See Dwg. #47494

$Elev_{bearing} := 245.26\cdot ft + \left(5 + \frac{9}{16}\right)\cdot in$ $\qquad Elev_{bearing} = 245.724\,ft$

$Elev_{bot} := Elev_{cap} - 3ft - 15ft$ $\qquad Elev_{bot} = 227.26\,ft$

This is to the point of fixity, in actuality the piles end at elevation 188ft.

$Joint_{top} := 1.5ft$ Top end joint size: c.g. of cap to start of column.

$Elev_{abut} := Elev_{top}$ $\qquad Elev_{abut} = 248.454\,ft$

The bridge is flat, so the abutment c.g. is the same as a bent.

Seisab Summary for Cross Section Properties

Span

$A_{deck} = 7.805\,ft^2$

$A_{22} := 0$

$A_{33} := 0$

Torsion $I_{11s} = 7.72\,ft^4$

$I_{22s} = 4.013 \times 10^3\,ft^4$

$I_{33s} = 2.881\,ft^4$

Pile

$A_{total} = 2.739\,ft^2$

$A_{22} := 0$

$A_{33} := 0$

Torsion $I_{11p} = 0.769\,ft^4$

$I_{22p} = 0.384\,ft^4$

$I_{33p} = 0.384\,ft^4$

Cracked Span

$A_{deck} = 7.805\,ft^2$

$A_{22} := 0$

$A_{33} := 0$

The new guidelines (section 5.6) specify that cracked section properties should be used in a Seisab analysis. A simple recommendation is that 20% of torsional stiffness and 75% of flexural stiffness can be relied upon. Since the pile is steel it will not crack, and an analysis shows the cap uncracked.

Torsion $I_{11s} := 0.2 \cdot I_{11s}$ $\quad I_{11s} = 1.544\,ft^4$

$I_{22s} := 0.75 \cdot I_{22s}$ $\quad I_{22s} = 3.009 \times 10^3\,ft^4$

$I_{33s} := 0.75 I_{33s}$ $\quad I_{33s} = 2.161\,ft^4$

Cap

$A_{cap} := 9ft^2$

$A_{22} := 0$

$A_{33} := 0$

Torsion $I_{11c} := 11.408ft^4$

$I_{22c} := 6.75 \cdot 10^3 ft^4$

$I_{33c} := 6.75 \cdot 10^3 ft^4$

Special Cap

$A_{cap} := 9ft^2$

$A_{22} := 0$

$A_{33} := 0$

Torsion $I_{11c} := 0$

$I_{22c} := 0$

$I_{33c} := 6.75ft^4$

Note on span. Since it is composite (steel beams with concrete deck) its area has been modified so that it represents steel.

Note on pile. Since it is a steel tube filled with concrete, the steel tube area has been converted to concrete.

Abutment Stiffness and 16" Steel Shaft Pile Properties

The 16" steel shell pile properties and stiffness will be determined first. Then the geometry of the abutment, its nine (9) piles and backwall will be combined to find total stiffness.

$kip := 1000lb$ $ksi := \frac{kip}{in^2}$ $N := 9$ Number of Piles

$E_c := 3372ksi$ $E_s := 29000ksi$ $n := \frac{E_s}{E_c}$ $n = 8.6$

$d_s := 16in$ $r_s := \frac{d_s}{2}$ $t_s := 0.5in$

$I_{steel} := \frac{\pi \cdot r_s^4}{4} - \frac{\pi \cdot (r_s - t_s)^4}{4}$ $I_{steel} = 731.942\, in^4$ $I_{sc} := I_{steel} \cdot n$ $I_{sc} = 6.295 \times 10^3\, in^4$

The steel moment of inertia is converted into an equivalent concrete moment of intertia so it can be combined with the concrete that will fill the pipe.

$I_{conc} := \frac{\pi \cdot (r_s - t_s)^4}{4}$ $I_{conc} = 2.485 \times 10^3\, in^4$

$I_{total} := I_{sc} + 0.4 \cdot I_{conc}$ $I_{total} = 0.352\, ft^4$ See C7.6 for this type of formula, pg. 7-18
Similar language also found in LRFD 6.9.5.

$I_{22} := I_{total}$ $I_{33} := I_{total}$

$A_{pipe} := \pi \cdot r_s^2 - \pi \cdot (r_s - t_s)^2$ $A_{pc} := n \cdot A_{pipe}$ $A_{pc} = 209.393\, in^2$

$A_{conc} := \pi\, (r_s - t_s)^2$ $A_{conc} = 176.715\, in^2$

$A_{total} := A_{pc} + A_{conc}$ $A_{total} = 386.108\, in^2$ $A_{total} = 2.681\, ft^2$

$J := 2 \cdot I_{total}$ $J = 0.703\, ft^4$

LATERAL STIFFNESS

The soil around the pile will be assumed to be a medium dense sand, with a soil reaction modulus of 15 lb/in3.

$EI := E_c \cdot I_{total}$ $\boxed{EI = 2.458 \times 10^{10}\,lb \cdot in^2}$

The value of EI shown above is used in the chart for translational stiffness which yields:

$k_{11} := 7.5 \cdot 10^4 \dfrac{lb}{in}$ $k_{33} := k_{11}$ $k_{33} = 900 \dfrac{kip}{ft}$

The value of EI shown above is used in the chart for rotation stiffness which yields:

$k_{m1m1} := 6.2 \cdot 10^8 \cdot \dfrac{lb \cdot in}{rad}$ $k_{m1m1} = 5.167 \times 10^4 \dfrac{kip \cdot ft}{rad}$

The value of EI shown above is used in the chart for cross-coupling stiffness which yields:

$k_{m3m3} := 1.2 \cdot 10^7 \cdot lb$ $k_{m3m3} = 1.2 \times 10^4\,kip$

$k_{m2m2} := 0$

To find axial stiffness:

$\Delta := 1in$ $L := 18.6ft$ The length is found during seismic step 1.

$P := \dfrac{\Delta \cdot A_{total} \cdot E_c}{L}$ $P = 5.833 \times 10^3\,kip$ $P_m := \dfrac{P}{in}$

$k_{22} := P_m$ $k_{22} = 7 \times 10^4 \dfrac{kip}{ft}$ Axial stiffness for one pile.

Step 1 (vertical translation - group effect)

$k_{22} := k_{22} \cdot N \qquad k_{22} = 6.3 \times 10^5 \frac{kip}{ft}$

Step 2 (lateral stiffness)

Determine Elastic Seismic Forces and Displacements

$f_e := 15 \frac{ton}{ft^3} \qquad f_e = 17.361 \frac{lb}{in^3}$ The medium dense soil strength comes from the blue example books no. 5, on page 3-43.

$f_e := 15 \frac{lb}{in^3}$ Use this conservative value of fe.

$T_p := \left(\frac{E_c \cdot I_{total}}{f_e} \right)^{\frac{1}{5}} \qquad T_p = 69.645\,in$

$L_p := 30ft$ Length of pile at the abutments.

$\frac{L_p}{T_p} = 5.169$ This value is needed for a chart on page 3-45 of the blue example book no. 5.

$F_\Delta := 2.25$ The deflection coefficient is the same for L/T values between 5 and 10.

$\Delta := 1in$ Set deflection at 1 inch

$P := \frac{\Delta \cdot E_c \cdot I_{total}}{F_\Delta \cdot T_p^3} \qquad P = 32.336\,kip$

Therefore the translational stiffness for a single pile is:

$k_p := \frac{P}{\Delta} \qquad k_p = 32.336 \frac{kip}{in}$ Translational stiffness for one pile.

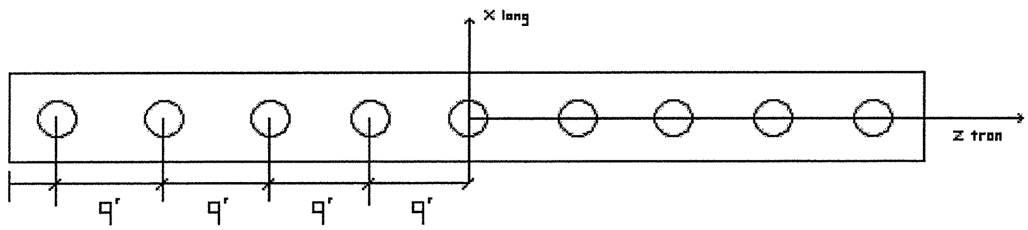
$k_{11} := k_p \cdot N \qquad k_{11} = 3.492 \times 10^3 \frac{kip}{ft}$

Step 3 (lateral stiffness - z translation)

$k_{33} := k_p \cdot N$ $\quad k_{33} = 3.492 \times 10^3 \frac{kip}{ft}$

Step 4 (Torsional Resistance)

Torsional Resistance is dependant on pile layout. The torsional resistance of a single pile is considered equal to zero.



$s := 9ft$ beam spacing

$$k_{55} = \sum_n k_p \cdot z_i^2 + k_p \cdot x_i^2$$

$$k_{55} := 2 \cdot s^2 \cdot k_p + 2 \cdot (2s)^2 \cdot k_p + 2(3s)^2 \cdot k_p + 2 \cdot (4s)^2 \cdot k_p$$

$$k_{55} = 1.886 \times 10^6 \frac{kip \cdot ft}{rad}$$

Step 5 (rocking rotational stiffness - about x axis)

The rocking rotational resistance is the axial stiffness times the distance of each pile squared from the center of rotation.

$$k_{pv} := \frac{k_{22}}{N}$$

$$k_{44} := 2 \cdot k_{pv} \cdot \left[s^2 + (2s)^2 + (3s)^2 + (4s)^2\right] \qquad k_{44} = 3.402 \times 10^8 \frac{kip \cdot ft}{rad}$$

Step 6 (rocking rotational stiffness - about z axis)

$k_{66} := 0$

Step 7 (Summary for the abutment bent, due to piles alone)

$k_{11} = 3.492 \times 10^{3} \frac{kip}{ft}$ Translation, x-axis

$k_{22} = 6.3 \times 10^{5} \frac{kip}{ft}$ Translation, y (vertical)

$k_{33} = 3.492 \times 10^{3} \frac{kip}{ft}$ Translation, z-axis

$k_{44} = 3.402 \times 10^{8} \frac{kip \cdot ft}{rad}$ Rocking about x-axis

$k_{55} = 1.886 \times 10^{6} \frac{kip \cdot ft}{rad}$ Torsion

$k_{66} = 0$ Rocking about z axis

Seisab gives an error if a value of zero is used for k66. Use Ix + Iy = Ip

$k_{66} := k_{11} + k_{33}$ $k_{66} = 6.985 \times 10^{3} \frac{kip}{ft}$

Find the soil stiffness, ks:

On page 4-15 of "Seismic Design of Bridges - Design Example No. 1", a stiffness per foot of wall is used, which **assumes an 8' backwall** with well-compacted backfill.

$k_b := 200\,\frac{kip}{in \cdot ft}$ Stiffness per foot of wall

$wall := 6ft$ The backwall is 6'-0" tall.

$k_6 := \frac{wall}{8ft} k_b$ $k_6 = 150\,\frac{kip}{in \cdot ft}$ For a 6' wall, assuming an even distribution of stress, the stiffness would be 150 kip/in.ft.

$k_{bs} := \frac{k_b + k_6}{2}$ $k_{bs} = 175\,\frac{kip}{in \cdot ft}$ However, the design guide recommends averaging the above number with the 8' backwall number.

Longitudinal (k11)

$k_s := k_{bs} \cdot (78ft + 2in)$ $k_s = 1.641 \times 10^5\,\frac{kip}{ft}$

$k_{piles} := 9 \cdot k_p$ $k_{piles} = 3.492 \times 10^3\,\frac{kip}{ft}$

The total abutment stiffness, realize that only one backwall can be in effect at a time due to the soil being in compression on one side, but being in tension (no help) on the other.

$k_{11} := \frac{k_s + 2 \cdot k_{piles}}{2}$ $k_{11} = 8.557 \times 10^4\,\frac{kip}{ft}$ Stiffness considering soil

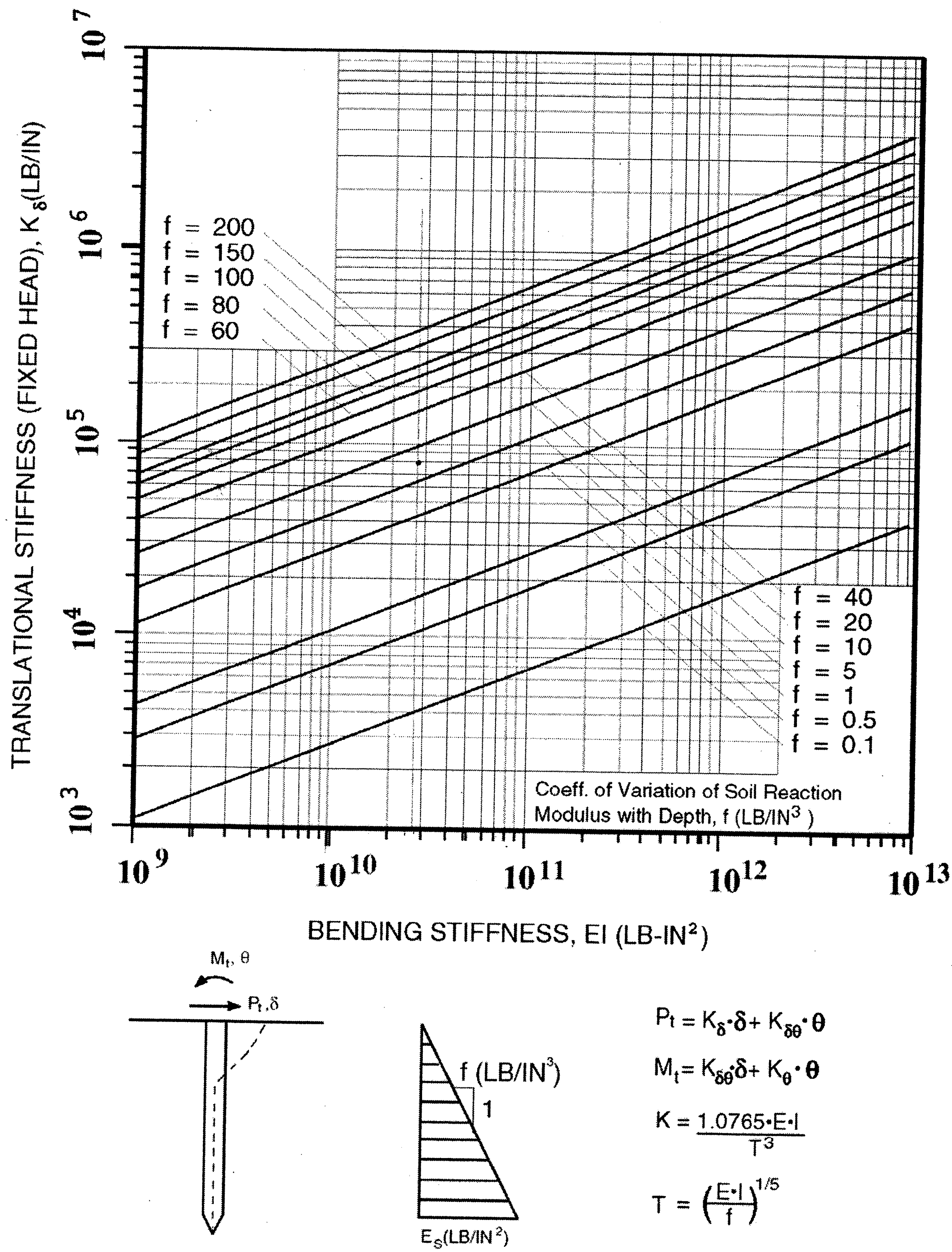
Transverse (k33)

The wingwall is only three (3) feet tall, and so its soil stiffness should be about half of the backwall.

$k_w := \frac{k_{bs}}{2} \cdot 10ft$ $k_w = 1.05 \times 10^4\,\frac{kip}{ft}$ Stiffness from one wingwall, 10' long.

Assume each wingwall is 50% effective, so just use one wingwall stiffness.

$k_{33} := \frac{k_w + 2k_{piles}}{2}$ $k_{33} = 8.742 \times 10^3\,\frac{kip}{ft}$ Stiffness considering soil



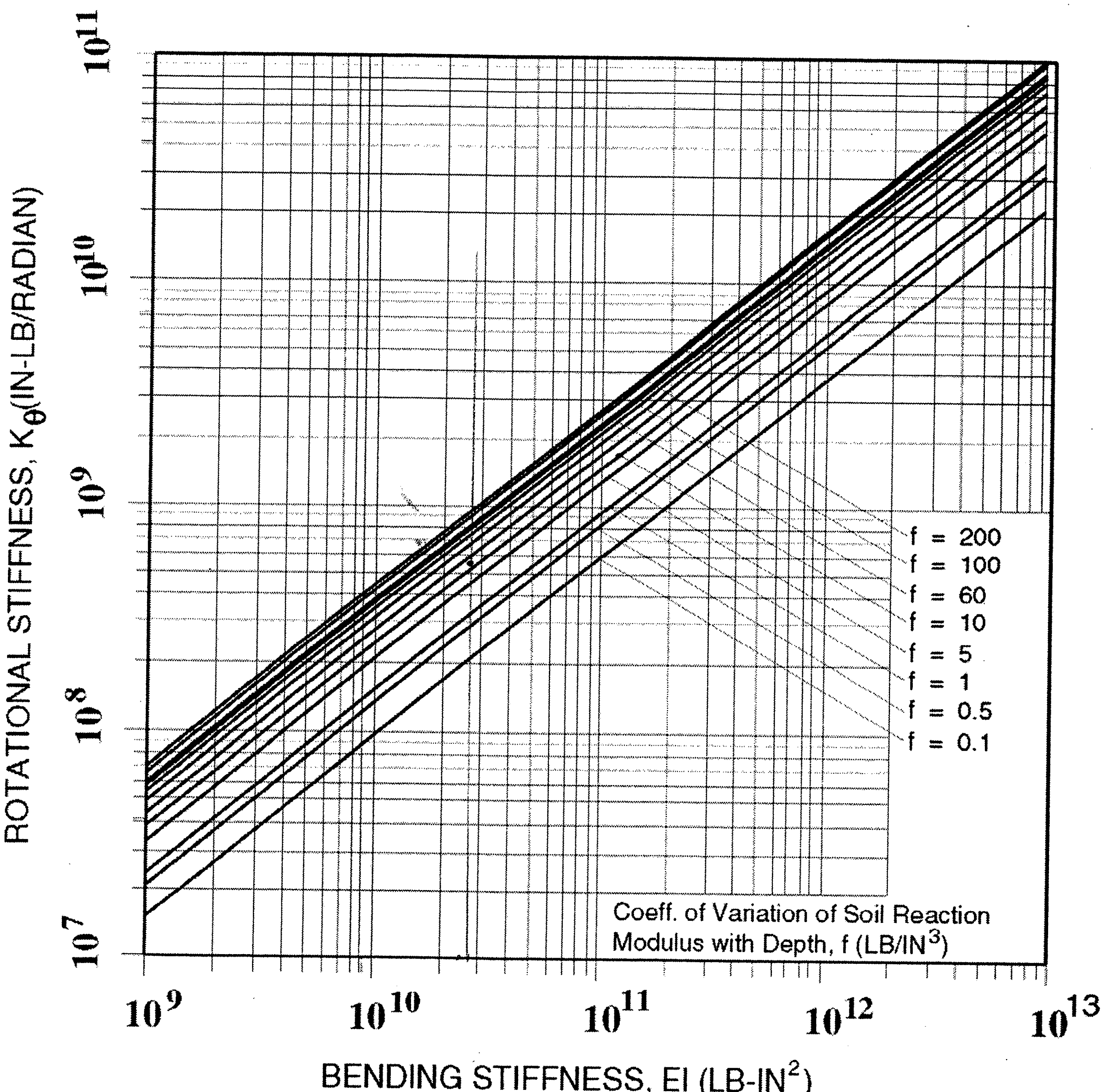
$$P_t = K_\delta \cdot \delta + K_{\delta\theta} \cdot \theta$$

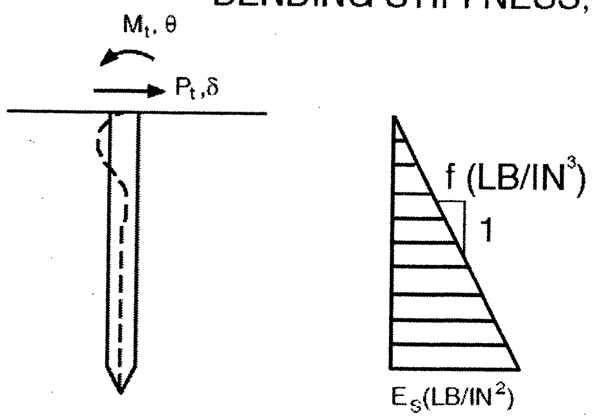
$$M_t = K_{\delta\theta} \cdot \delta + K_\theta \cdot \theta$$

$$K = \frac{1.0765 \cdot E \cdot I}{T^3}$$

$$T = \left(\frac{E \cdot I}{f}\right)^{1/5}$$

Figure C-5 **Coefficient for Lateral Pile-Head Stiffness for Fixed-Head Pile Lateral Stiffness (ATC, 1996)**





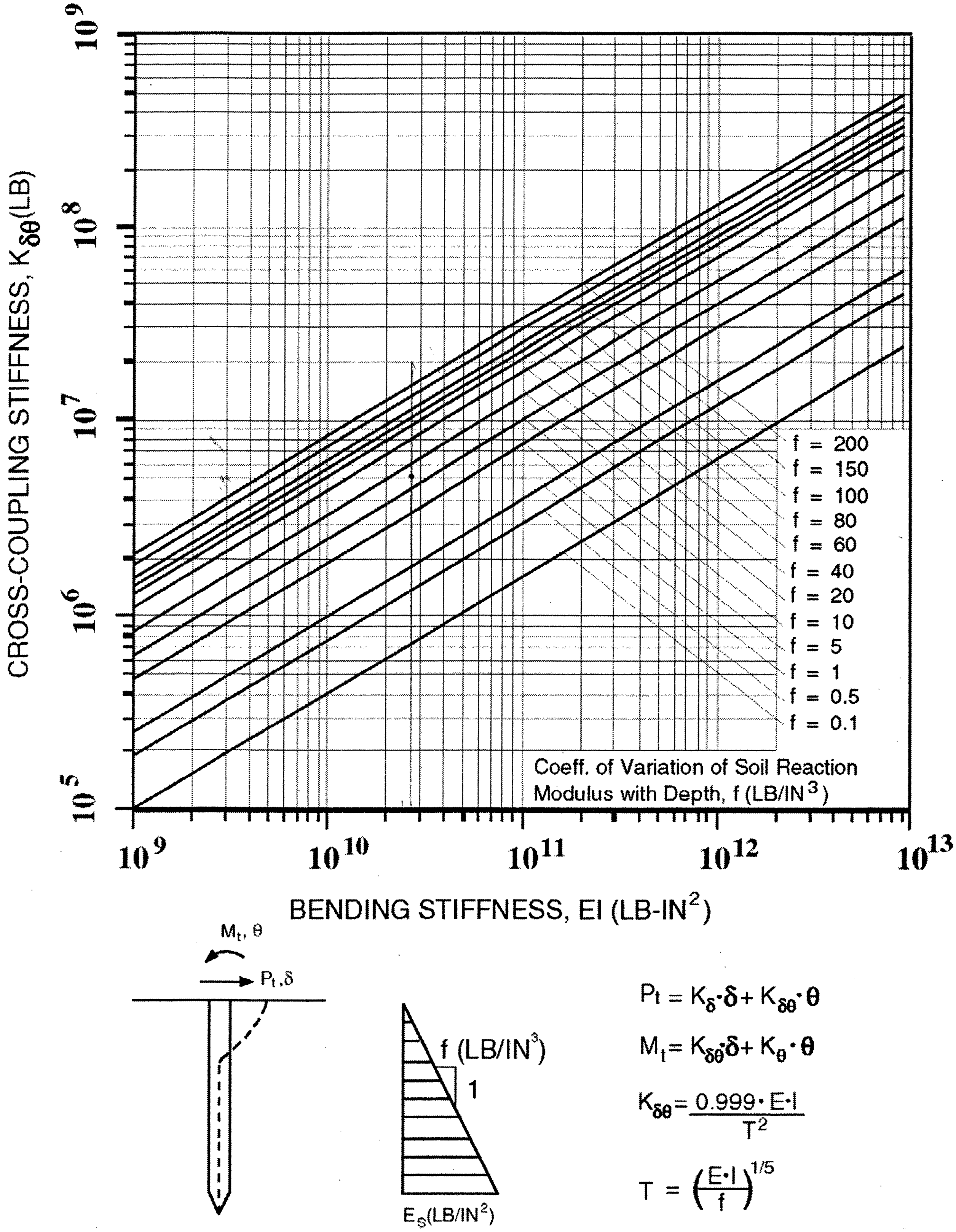
$$P_t = K_\delta \cdot \delta + K_{\delta\theta} \cdot \theta$$

$$M_t = K_{\delta\theta} \cdot \delta + K_\theta \cdot \theta$$

$$K = \frac{1.499 \cdot E \cdot I}{T^3}$$

$$T = \left(\frac{E \cdot I}{f}\right)^{1/5}$$

Figure C-6 **Coefficient for Pile Head Rotation (ATC, 1996)**



$$P_t = K_\delta \cdot \delta + K_{\delta\theta} \cdot \theta$$

$$M_t = K_{\delta\theta} \cdot \delta + K_\theta \cdot \theta$$

$$K_{\delta\theta} = \frac{0.999 \cdot E \cdot I}{T^2}$$

$$T = \left(\frac{E \cdot I}{f}\right)^{1/5}$$

Figure C-7 **Coefficient for Cross-Coupling Stiffness Term (ATC, 1996)**

Cap Section Properties

$w := 3\,ft \qquad h := 3\,ft$

$Area := w \cdot h \qquad Area = 9\,ft^2$

$$SMALLEST := \left| \begin{array}{l} x \leftarrow w \ \text{ if } \ w < h \\ x \leftarrow h \ \text{ otherwise} \\ \text{return } x \end{array} \right. \qquad SMALLEST = 3\,ft$$

$$b := \frac{SMALLEST}{2} \qquad a := \frac{Area}{2 \cdot SMALLEST} \qquad a = 1.5\,ft$$

$b = 1.5\,ft$

$$J := a \cdot b^3 \cdot \left[\frac{16}{3} - 3.36 \cdot \frac{b}{a} \cdot \left(1 - \frac{b^4}{12 \cdot a^4} \right) \right] \qquad I_{11} := J \qquad I_{11} = 11.408\,ft^4$$

$$I_x := \frac{w \cdot h^3}{12} \qquad I_y := \frac{h \cdot w^3}{12}$$

$I_{22} := I_x \cdot 1000 \qquad I_{33} := I_y \cdot 1000$

$I_{22} = 6.75 \times 10^3\,ft^4 \qquad I_{33} = 6.75 \times 10^3\,ft^4$

You should use a regular cap (as shown above) in cases where the cap can physically vibrate independently of the superstructure. Use a special cap for all other cases.

For this case, use the special cap... which is the same as a normal cap but with I22 and I33 set to large values (multiplied by 1000 above).

$kip := 1000 \cdot lb$ $I_g := I_x$

Cracked Cap Section Properties

The stiffness of a concrete beam is not elastic. Instead after a certain amount of moment is applied to the beam, the concrete cracks. This moment level is called the Cracking Moment, and after this point is reached, the beam continues to crack and the stiffness continues to drop.

$f_c := 3500$ $y_t := \frac{h}{2} - 3in$ Distance from centroid to extreme tension fiber

$f_r := 7.5 \cdot \sqrt{f_c} \cdot \frac{lb}{in^2}$ The modulus of rupture, or flexural tensile strength of concrete (p.48 macgregor)

$M_{cr} := \frac{f_r \cdot I_g}{y_t}$ $M_{cr} = 345.026\, kip \cdot ft$ $n := \frac{29000}{3500}$ $n = 8.286$

Determine the cracked moment of inertia

Area of #6 bar : $r := \frac{6in}{8 \cdot 2}$ $A := \pi \cdot r^2$ $A = 0.442\, in^2$

centroid of the section will be approximately in the middle.

$y_{top} := \frac{h}{2}$ $y_{bot} := \frac{h}{2}$ $I_g = 1.4 \times 10^5\, in^4$

$A_{sbot} := 6 \cdot A$ $A_{sn} := A_{sbot} \cdot n$ $A_{sn} = 21.963\, in^2$

$p := \frac{A_{sn}}{Area}$ $p = 0.017$

$k := \sqrt{2p \cdot n + (p \cdot n)^2} - p \cdot n$ $k = 0.408$

$d := h - 3in$ $c := k \cdot d$ $c = 13.458\, in$

Compression Zone	$A_{ccr} := c \cdot w$	$y_{ccr} := \frac{c}{2}$	$I_{ccr} := \frac{w \cdot c^4}{12}$	$Ay_c := A_{ccr} \cdot y_{ccr}^2$
Reinforcement	A_{sn}	$y_{scr} := d - 3in - c$		$Ay_s := A_{sn} \cdot y_{scr}^2$

$I_{cr} := Ay_c + Ay_s \qquad I_{cr} = 2.795 \times 10^4\,in^4$

Since the beams of the bridge are placed directly over the piers, there is no live load or even superstructure dead load that is supported by the cap. The only moment in the cap is therefore its own self weight.

The cap is basicly a continuous beam with 8 spans of 9 feet each.

$w_{dl} := Area \cdot 0.150\frac{kip}{ft^3} \qquad w_{dl} = 1.35\frac{kip}{ft}$

$M_a := 0.1 \cdot w_{dl} \cdot (9ft)^2 \qquad M_a = 10.935\,kip \cdot ft$

Since Ma < Mcr, the effective moment of inertia is uncracked, the full value.

$I_e := I_{cr} + (I_g - I_{cr}) \cdot \left(\frac{M_{cr}}{M_a}\right)^3 \qquad I_e = 3.519 \times 10^9\,in^4$ Greater than uncracked, not valid.

$I_x = 1.4 \times 10^5\,in^4$ Use Uncracked moment of inertia.

Seismic Design Proportioning and Articulation Recommendations - Step 1

Balanced Stiffness (4.1.1)

It is recommended that the ratio of **stiffness** between any two bents in a bridge be less than a factor of two. If a particular bent is much stiffer than the others, it will attract load and tend to get damaged in an earthquake.

For the bridge across Mud Slough Ditch, the two interior bents are the same, and the two end bents are the same. But piles of the interior and end bents differ in diameter and length to fixity, which will affect stiffness.

$kip := 1000 \cdot lb$ $ksi := \frac{kip}{in^2}$

Interior Bents

$E_s := 29000 \cdot ksi$ $E_c := 3372ksi$ $n := \frac{E_s}{E_c}$

$rad_{int} := 9 \cdot in$ $t_{int} := \frac{1}{2} in$ (assume 0.5" wall thickness)

$$I_{steeli} := \left[\frac{\pi \cdot rad_{int}^4}{4} - \frac{\pi \cdot (rad_{int} - t_{int})^4}{4}\right] \cdot n \qquad I_{steeli} = 9.058 \times 10^3\, in^4$$

$$I_{conci} := \frac{\pi \cdot (rad_{int} - t_{int})^4}{4} \qquad I_{conci} = 4.1 \times 10^3\, in^4$$

$$I_{int} := I_{steeli} + 0.4 \cdot I_{conci} \qquad I_{int} = 1.07 \times 10^4\, in^4$$

With the value of I found, the concrete modulus must be used for stiffness calculations.

Equivalent Length of Piles (Davisson and Robinson's Procedure, 1965)

$L_{ui} := 5.7ft$ Length of pile above ground. $n_h := 40 \cdot \frac{lb}{in^3}$ $\Delta := 1in$

$$T := \left(\frac{E_c \cdot I_{int}}{n_h}\right)^{0.2} \qquad T = 5.15\, ft$$

$$L_{eqi} := L_{ui} + 1.8T \qquad L_{eqi} = 14.971\, ft$$

$$P_{int} := \frac{\Delta \cdot 3 \cdot E_c \cdot I_{int}}{L_{eqi}^3} \qquad P_{int} = 1.866 \times 10^4\, lb$$

It takes 19,000 lbs to move an interior pile one inch laterally.

End Bents

$E_s := 29000 \cdot ksi$ $E_c := 3372ksi$ $n := \frac{E_s}{E_c}$

$dia_{end} := 8 \cdot in$ $t_{end} := \frac{1}{2} in$ (assume 0.5" wall thickness)

$$I_{steele} := \left[\frac{\pi \cdot dia_{end}^{4}}{4} - \frac{\pi \cdot (dia_{end} - t_{end})^{4}}{4} \right] \cdot n \qquad I_{steele} = 6.295 \times 10^{3}\, in^{4}$$

$$I_{conce} := \frac{\pi \cdot (dia_{end} - t_{end})^{4}}{4} \qquad I_{conce} = 2.485 \times 10^{3}\, in^{4}$$

$$I_{end} := I_{steele} + 0.4 \cdot I_{conce} \qquad I_{end} = 7.289 \times 10^{3}\, in^{4}$$

With the value of I found, the concrete modulus must be used for stiffness calculations.

Equivalent Length of Piles (Davisson and Robinson's Procedure, 1965)

$L_{ue} := 10ft$ Length of pile above ground. $n_h := 40 \cdot \frac{lb}{in^{3}}$ $\Delta := 1in$

$$T := \left(\frac{E_c \cdot I_{end}}{n_h} \right)^{0.2} \qquad T = 4.77\, ft$$

$$L_{eqe} := L_{ue} + 1.8T \qquad L_{eqe} = 18.586\, ft$$

$$P_{end} := \frac{\Delta \cdot 3 \cdot E_c \cdot I_{end}}{L_{eqe}^{3}} \qquad P_{end} = 6.646 \times 10^{3}\, lb$$

It takes 6650 lbs to move an end pile one inch laterally.

<u>Compare Stiffness of End and Interior Bents</u>

Tributary Mass

Each bent does not support the same amount of load. The difference in the load supported is part of the stiffness equation. Span lengths are 28' - 34' - 28' and the supports are fixed.

From winbeam, 11.7% of the load went into each end support. 38.3% of the load goes into each interior support.

$$k_{ratio} := \frac{P_{end} \cdot 38.3}{P_{int} \cdot 11.7} \qquad k_{ratio} = 1.166$$

$$\text{RESULT} := \left| \begin{array}{l} x \leftarrow \text{"Low End Bent stiffness (Fail)"} \quad \text{if } k_{ratio} < 0.5 \\ x \leftarrow \text{"Low Interior Bent stiffness (Fail)"} \quad \text{if } k_{ratio} > 2 \\ x \leftarrow \text{"PASS"} \quad \text{otherwise} \\ \text{return } x \end{array} \right.$$

$$\text{RESULT} = \text{"PASS"}$$

This bridge has a balanced stiffness.

Balanced Frame Geometry (4.1.2)

It is recommended that the ratio of **fundamental periods of vibration** for adjacent frames in the longitudinal and transverse directions satisfy the equation below. If the vibrational frequencies are significantly different, during vibration they will get out of sync which will lead to large relative displacements. This will increase the probability of longitudinal unseating and pounding between frames. It will also pass on extra seismic load to frames not designed for it.

$$\frac{T_{int}}{T_{end}} > 0.7$$

In this bridge, the interior bents are the same, as are the end bents. Thus the "adjacent frames" to consider are the interior bents vs. the end bents.

On a failed check, it is recommended to limit the bridge to five (5) frames. **Since this bridge only has four (4) bents, this check is not needed.**

PASS

Minimum Seat Width (C4.1.2)

The **current** AASHTO Division I-A requirement for minimum seat width is:

$L := 34\text{ft}$ Length of span

$H := 55\text{ft}$ Height of drilled shafts

$N := 1.5(0.2\text{ft} + .0017L + 0.0067H)$ $\qquad N = 0.939\,\text{ft}$ $\quad <$ 1'-10"

The above formula accounts for relative displacement due to motion of the piers, rotation of pier footings (where used), and deformation of the pier. A new minimum seat width equation is used in step 11.

PASS

Adjusting Dynamic Characteristics (4.1.3)

If either of the two cases above did not pass, here is a list of techniques to consider.

Use of oversized pile shafts.
Adjust effective column lengths (ie lower footings, isolation casing).
Use of modified end fixities.
Reduce and/or redistribute superstructure mass.
Vary the column cross section and longitudinal reinforcement ratios.
Add or relocate columns.
Modify the hinge/expansion joint layout.
Incorporate isolation bearings or dampers (ie response modification devices).
Rearticulation

Bridges in seimic category D must be carefully evaluated for individual member ductilities and capacity if they do not meet the requirements of 4.1.1, 4.1.2, and 4.1.3.

PASS

End Span Considerations (4.1.4)

If using single column bents, the superstructure rigidity can influence transverse stiffness. Be sure to consider this when calculating shear.

However, each bent on this bridge has nine piles.

PASS

Select Analysis Procedure for Seismic Demands - Step 2

Overall Guidelines (4.2)

Selection of an analysis method is determined by the regularity of the bridge - a function of the number of spans, and the distribution of weight and stiffness. Regular bridges have less than 7 spans, and no abrupt changes in weight, stiffness, or geometry. A regular bridge should also have balanced stiffness as computed in Step 1 (abutments excluded).

The definition of regular is presented in Table 4.2. For a four (4) span bridge such as this, it can be curved up to 90 degrees, have maximum spans twice as long as short, and have a maximum bent/pier stiffness ratio of 4.

$$span_{ratio} := \frac{34ft}{28ft}$$

$$SPAN_{RATIO} := \begin{array}{|l} s \leftarrow \text{"PASS"} \quad \text{if } 0.5 < span_{ratio} < 2 \\ s \leftarrow \text{"FAIL"} \quad \text{otherwise} \\ \text{return } s \end{array}$$

$$curve := 0$$

$$CURVE := \begin{array}{|l} c \leftarrow \text{"PASS"} \quad \text{if } -90 < curve < 90 \\ c \leftarrow \text{"FAIL"} \quad \text{otherwise} \\ \text{return } c \end{array}$$

$SPAN_{RATIO} = \text{"PASS"}$ $\quad CURVE = \text{"PASS"}$

$span_{ratio} = 1.214$

Since both interior bents are the same:

$stiffness_{ratio} := 1$

$$STIFFNESS_{RATIO} := \begin{array}{|l} x \leftarrow \text{"PASS"} \quad \text{if } 0.25 < stiffness_{ratio} < 4 \\ x \leftarrow \text{"FAIL"} \quad \text{otherwise} \\ \text{return } x \end{array}$$

$STIFFNESS_{RATIO} = \text{"PASS"}$

Our bridge over Mud Slough Ditch is not curved, has a max span of 34' with a min of 28', and the interior bents have the same stiffness. Thus it is a "regular" bridge.

Now that the bridge is classified as "regular", an analysis procedure can be read from Table 4.1. This results in a choice, either use Procedure 1 or 2. It is probably better to use procedure 1, since procedure 2 is used for the most complicated bridges. Procedure 1 is an **Equivalent Static Analysis** which is covered in **section 5.4.2**.

Special Requirements for Curved Bridges (4.2.1)

Not applicable.

Limitations and Special Requirements (4.2.2)

For important bridges, essential bridges, bridges near earthquake faults, or when using seismic isolation, a nonlinear time history is required. A nonlinear time history is not required for this bridge.

Consider Vertical Ground Motion Effects (4.7.2) - Step 3

This section is for bridges in Seismic Design Category D AND located within six (6) miles of a fault. These bridges are required to have 25% of their superstructure steel continuous over the whole length. This can be accomplished using service couplers capable of handling 125% of the rebar yield strength. Lapping of rebar to fill this requirement is not allowed.

Vertical ground motion design requirements DO NOT apply for steel girders. So we are OK.

PASS

Select Horizontal Axes for Ground Motions (4.3.1) - Step 4

Global seismic displacement demands are determined independantly along two perpendicular axes, using the procedure found in section 4.2 (Equivalent Static Analysis). These perpendicular axes are typically the longitudinal and transverse axes. The axes for a curved bridge are generally aligned on a chord between two abutments.

We will use the longitudinal and transverse axes.

Damping Consideration (4.3.2) - Step 5

Up to 10% damping can be used for bridges that have substantial energy dissipation at the abutments. The following characteristics are typically good indicators that higher damping is justified.

Total bridge length is less than 300 feet.
Abutments are designed for sustained soil mobilization.
Supports are normal or slight skew.
The superstructure is continuous without hinges or expansion joints.

The bridge over Mud Slough Ditch has all the above qualities except for number 2. The abutment piles on our bridge are surrounded by sand to let it deflect during an earthquake. Also the approach slabs are not connected to the backwall. These two items were intentionally provided so that the bridge could easily deflect and therefore bring less force into the foundations.

But to mobilize the soil (4.3.2), the frame should be RIGID and NOT respond in a FLEXIBLE manner. If the abutments were rigid, they would mobilize the soil and allow up to 10% damping which would lower the earthquake loads. And so our bridge, with flexible methods in place to reduce earthquake loads, does not qualify for the increased damping.

USE 5% Damping.

Short Period Structures Consideration (4.3.3)

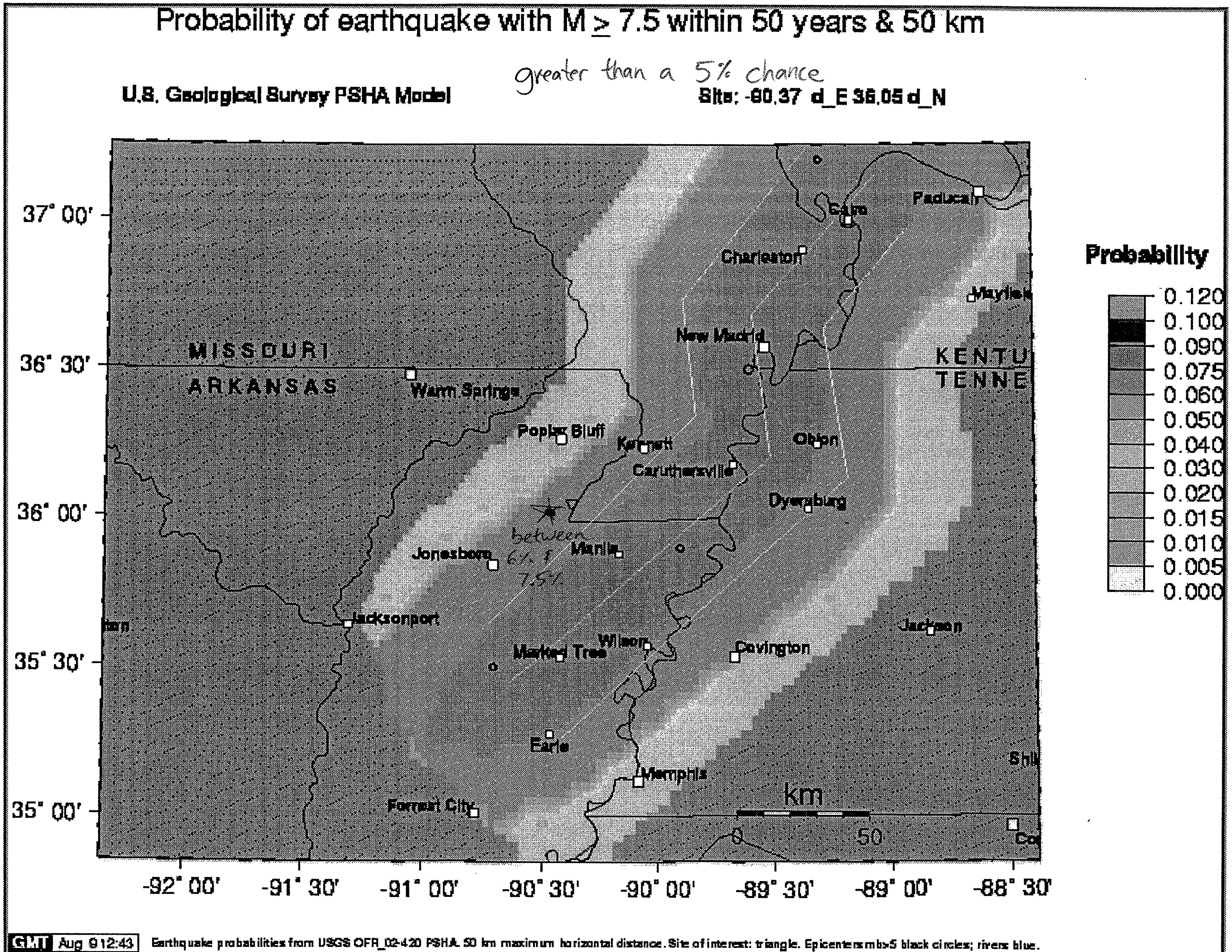
To obtain a design displacement, elastic analysis displacements are multiplied by a factor, R_d. A unique magnification factor is applied to each orthogonal direction. R is found in SDC D by dividing spectral force by the plastic capacity.

Table 4.3 provides seismic period times if given **soil** site class and **earthquake** strength.

$S_s := 1.157$ $\quad 0.4 \cdot S_s = 0.463$ Short period (0.2 second) mapped design spectral acceleration. Use 0.45 on Table 4.3

$Mw := 8$ From the USGS earthquake probability mapping website. There is a 6.75% chance of a 7.5 magnitude quake. Perhaps there is a 5% chance of 7.75 magnitude quake. Conservatively use highest column.

From Table 4.3, the characteristic ground motion period is: $T_c := 0.68$


Probability of earthquake with M ≥ 7.5 within 50 years & 50 km
greater than a 5% chance
U.S. Geological Survey PSHA Model
Site: -90.37 d_E 36.05 d_N
MISSOURI
ARKANSAS
KENTU
TENNE
Paducah
Cairo
Charleston
New Madrid
Warm Springs
Poplar Bluff
Kennett
Caruthersville
Obion
Dyersburg
between 6% & 7.5%
Jonesboro
Manila
Jacksonport
Marked Tree
Wilson
Covington
Jackson
Earle
Memphis
Forrest City
km
0
50
37° 00'
36° 30'
36° 00'
35° 30'
35° 00'
-92° 00'
-91° 30'
-91° 00'
-90° 30'
-90° 00'
-89° 30'
-89° 00'
-88° 30'
Probability
0.120
0.100
0.090
0.075
0.060
0.050
0.040
0.030
0.020
0.015
0.010
0.005
0.000
GMT Aug 9 12:43
Earthquake probabilities from USGS OFR_02-420 PSHA. 50 km maximum horizontal distance. Site of interest: triangle. Epicenters mb>5 black circles; rivers blue.

Frequency of the structure:

From WinSeisab, the mode 1 frequency for the structure, ignoring the abutment wall stiffness as it might not be there in a massive earthquake is:

$T := 0.308$

If $\frac{T_c}{T} > 1$ $\quad R_d = \left(1 - \frac{1}{R}\right)\cdot\frac{T_c}{T} + \frac{1}{R} > 1$ $\quad \frac{T_c}{T} = 2.208$

R is determined from spectral force divided by plastic capacity, so it can not be calculated until after the Seisab analysis has been completed.

$kip := 1000lb$ $\quad ksi := \frac{kip}{in^2}$

$M_p = F_y \cdot Z < 1.5M_y$

$M_p := 402kip \cdot ft$

Plastic moment for one pile, calculated on the next sheet. Perhaps this value should be multiplied by 9 piles?

$F_{spectral} := (568.2 + 129.8)kip$

The largest force from Seisab, from abutments.

$F_{spectral} := (335 + 97)kip \cdot ft$

The largest seismic moment induced in a column. Assume this is the "spectral force".

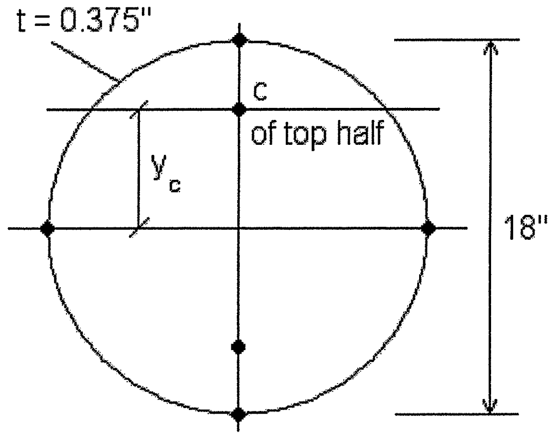
$R := \frac{F_{spectral}}{M_p}$ $\quad R = 1.075$

$R := 3$

The R factor seems low. In SDC B, R is given at 2. For SDC C, R is 3. I will use at least the value used for SDC C.

$R_d := \left(1 - \frac{1}{R}\right)\cdot\frac{T_c}{T} + \frac{1}{R}$ $\quad R_d = 1.805$

Plastic Section Properties of an 18" Pipe



Assume a 1/16" reduction in wall thickness for corrosion.

$d := 18in - 2\left(\frac{1}{16}in\right)$ $\quad d = 17.875\,in$

$t := 0.4375in - \frac{1}{16}in$ $\quad t = 0.375\,in$

$d_{in} := d - 2t$ $\quad d_{in} = 17.125\,in$

$r := \frac{d + d_{in}}{2 \cdot 2}$ $\quad r = 8.75\,in$

This radius is to the center of wall thickness.

$y_c := \frac{2r}{\pi}$ $\quad y_c = 5.57\,in$

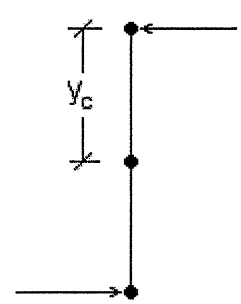
Formula for distance to the centroid of the top half (or bottom half) is typically found in a statics book.

$A_{half} := \pi \cdot r \cdot t$ $\quad A_{half} = 10.308\,in^2$

When the whole section is yielding, the plastic modulus is used. The force of the top half will be in tension and act at the centroid of the upper half. The bottom force will be in compression and act at its centroid. The moment arm between them is $2y_c$.

$Z := A_{half} \cdot 2y_c$ $\quad Z = 114.844\,in^3$

$kip := 1000lb$ $\quad ksi := \frac{kip}{in^2}$ $\quad F_y := 42ksi$



$M_p := Z \cdot F_y$ $\quad M_p = 401.953\,kip \cdot ft$

$$I := \frac{\pi \cdot \left(\frac{d}{2}\right)^4}{4} - \frac{\pi \cdot \left(\frac{d_{in}}{2}\right)^4}{4} \qquad I = 789.596\,in^4$$

$c := \frac{d}{2}$ $\quad c = 8.938\,in$ $\quad$ Surface farthest from the neutral axis, outside of shape.

$r_g := \left(\frac{I}{2 \cdot A_{half}}\right)^{0.5}$ $\quad r_g = 6.189\,in$ $\quad r^2 \cdot A = I$ $\quad$ Area times a distance^2 = Moment of Inertia
This distance is radius of gyration.

$S := \frac{I}{c}$ $\quad S = 88.346\,in^3$

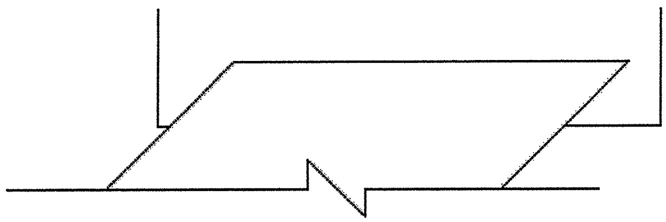
$M_y := S \cdot F_y$ $\quad M_y = 309.212\,kip \cdot ft$

$M_n := S \cdot 1.1 \cdot F_y$ $\quad M_n = 340.133\,kip \cdot ft$

Determine Seismic Displacement Demands (Figure 1.3E) - Step 6

<u>Define Bridge ERS (Section 5.1.1 and Section 3.3)</u>

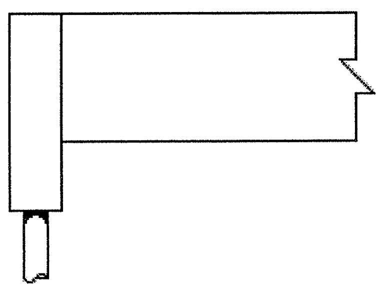
From Figure 3.3.1a the most likely ERS is: Transverse or Longitudinal Response



Abutment required to resist the design earthquake elastically.

Longitudinal passive pressure must be less than 0.70 of the value obtained using the procedure given in Article 5.2.3. Considered later in step.

Earthquake Resisting Components that Require Owner's Approval (Figure 3.3.2)



Plumb piles that are not capacity protected (e.g., integral abutment piles or pile-supported seat abutments that are not fused transversely.

Ensure limited ductility response in piles according to Article 4.7.1
Conventional Ductile Response: $4.0 < \mu_D < 8.0$ is checked in step 9.

<u>Select Analysis Procedure (Section 5.4.2)</u>

Choose Uniform Load Method - Analysis Procedure 1

<u>Mathematical Modeling Requirements (Section 5.5)</u>

Must be modeled in Seisab with nodes at span quarter points (3 int nodes) and at column third points (2 int nodes).

Abutment Modeling (Section 5.2) $kip := 1000lb$

Cracked section properties should be used for the abutment when conducting an Equivalent Static Analysis.

The cap applied moment does not exceed the cracking moment - so it has full moment of inertia (see cap sectional properties from old LRFD design).

The steel shell **pile innards** are concrete, but **were reduced by 0.4 during initial sectional property calculations**, so the model is good.

Temperature elongation/contraction (needed for soil properties)

Procedure B $T_{max} := 115$ $T_{min} := 15$

Procedure A $T_{max} := 120$ $T_{min} := 0$ USE procedure A

$\alpha := 0.0000066$ Coefficient of thermal expansion, see LRFD 5.4.2.2

$L := 90ft$ Length of bridge

$\Delta := \alpha \cdot L \cdot (T_{max} - T_{min})$ Elongation range due to uniform thermal expansion, LRFD 3.12.2

$\Delta = 0.855\,in$

$T_{set} := 70$ Assume the bridge is built at about this temperature.

$T_{\Delta 1} := T_{max} - T_{set}$ $T_{\Delta 1} = 50$ $T_{\Delta 2} := T_{min} - T_{set}$ $T_{\Delta 2} = -70$

$$T := \left| \begin{array}{l} x \leftarrow |T_{\Delta 2}| \ \text{ if } |T_{\Delta 1}| < |T_{\Delta 2}| \\ x \leftarrow |T_{\Delta 1}| \ \text{ otherwise} \\ \text{return } x \end{array} \right.$$

This temperature is the extreme temperature variation from when the bridge is set up.

$T = 70$

$\Delta_T := \alpha \cdot L \cdot T$ $\Delta_T = 0.499\,in$

The foundation of this bridge is steel sheel piles which are fairly flexible. Their movement is required for determining passive and active pressures. One abutment supposedly takes 2/3 of the soil pressure and the other takes 1/3. But since the bridge is short and tied together, both supports should move the same distance - half of the total movement.

$\Delta_{supports} := \frac{\Delta_T}{2}$ $\qquad$ $\Delta_{supports} = 0.249\,in$

During an earthquake, the bridge will move:

$\Delta_{EQ} := 0.093ft$ $\qquad$ $\Delta_{EQ} = 1.116\,in$ $\qquad$ Longitudinal Movement from Seisab. Calculated many steps later and brought back to this point.

$\Delta_{EQS} := \frac{\Delta_{EQ}}{2}$ $\qquad$ $\Delta_{EQS} = 0.558\,in$ $\qquad$ Distribute half of the movement to each support.

<u>To Find Soil Properties</u>

I need to know the density of the soil and its angle of friction.

$N := 21.8$ $\qquad$ The average standard penetration for the left side (significantly lower than the right side).

$N_c := 0.77 \cdot N \cdot \log\left(\frac{20}{0.0105 \cdot 95.6}\right)$ $\qquad$ $N_c = 21.811$ $\qquad$ Corrected N value. Since I don't know what the english units are for N besides blows per foot, assuming this is ok.

Mostly medium dense sand is wet or moist over the entire length of borehole.

From Table 2.4 in Principles of Foundation Engineering (1984, Das) the density and angle of soil friction can be approximately correlated to N.

$D_r := 0.48$ $\qquad$ $\phi := 39.2$

From an SPILE chart, wet medium dense sand weighs...

$\gamma := 110 \frac{lb}{ft^3}$

Determine amount of active and passive pressure.

The abutment wall is about 6' tall. Even though the bridge has integral abutments, the piles are surrounded with sand to allow them to deflect easily. Hopefully since the abutments are less stiff, a lower force will be transmitted into the substructure from the abutments. How much passive pressure is activated?

$H := 6\text{ft}$

$$\frac{\Delta_{supports}}{H} = 3.465 \times 10^{-3}$$

Normal movement due to temperature. From Figure 4.4 in the "Manual for the Design of Bridge Foundations" and using the compacted medium dense sand graph line, some active pressure and a good amount of passive pressure is created.

$K_a := 0.30 \qquad K_p := 2.8$

$$\frac{\Delta_{EQS} + \Delta_{supports}}{H} = 0.011$$

Earthquake. Fully activates passive pressure.

$K_a := 0.25 \qquad K_p := 4$

Any form of passive pressure offers far more resistance than active pressure. See figure 4.4 in the same book.

The natural soil around the abutments is a medium stiff clay. I am assuming in these calculations that the poor soil around the abutments will be taken out and backfilled with a medium dense sand (select fill). This is the only material for which there is a compacted soil pressures.

$k_p := 4$ The extreme case will have the highest passive pressure.

$\gamma_s := 110\frac{\text{lb}}{\text{ft}^3}$ Backfill unit weight

$H_w := 6\text{ft}$ Height of Backwall

$p_p := k_p \cdot \gamma_s \cdot H_w \qquad p_p = 2.64\frac{\text{kip}}{\text{ft}^2}$

$P_p := p_p \cdot H_w \qquad P_p = 15.84\frac{\text{kip}}{\text{ft}}$ See Article 5.2.3.3 in New Guidelines

$k_{eff1} := \frac{P_p}{0.02 \cdot H_w} \qquad k_{eff1} = 11\frac{\frac{\text{kip}}{\text{in}}}{\text{ft}}$ Equation 5.2

The new "Recommended LRFD Guidelines for the Seismic Design of Highway Bridges" in section 5.2.3.3 gives a procedure for estimating the passive pressure force. The passive pressure may also be assumed equal to:

$$P_p := 0.70 \cdot \frac{2 \cdot H}{3} \cdot \frac{kip}{ft^2} \qquad P_p = 2.8\frac{kip}{ft}$$

70% because integral abutment. See beginning of step.

$$K_{effl} := \frac{P_p}{0.02H} \qquad K_{effl} = 1.944\frac{\frac{kip}{in}}{ft}$$

Stiffness of Abutment, much lower than the value calculated by a better analysis. Do not use unless no other option.

Other Specifications in Section 5.2

Shear keys should be designed to resist a lateral force of 0.2 times the abutment dead load.

Either elastic resistance or fusing (need more information on how fusing works) should be used to accomodate transverse abutment loading.

The stiffness of the piles should be ignored if deflection is more than 4 inches. Generally, the pile resistance is very small compared to passive resistance of an abutment. This provision is to make calculations easier for the engineer - he does not need the pile stiffness.

Foundations (section 5.3)

For seismic category D, the foundation modeling method 2 must be used. From Table 5.1 on page 5-12, pile bents must be modeled using soil springs based on P-y curves **or** with an estimated depth to fixity.

Graphs for calculating spring constants are in Appendix C. An example on how to apply them was provided to me by a coworker. The **spring constants are calculated** for the abutments in another mathcad sheet appended to the end of the calculations. I also calculated the depth to fixity for the interior bents, which was not covered in Appendix C, in another mathcad sheet.

The bridge was modeled in Seisab with soil springs on the abutments (since you don't even enter the piles on Seisab with them), but **fixed interior bent piles** at their estimated depth of fixity.

Liquefaction for seismic zone D shall be considered, see Article 6.8. Many details are in Appendix D and E. But the empirical method is not given in the new guide. This analysis was accomplished in another mathcad sheet using older equations, and the soil found strong.

If liquefaction would have been a problem, the solution is generally adding reinforcement to the piles to make sure they do not buckle when the soil loses its bracing capability.

There are no liquefaction problems with this bridge (See liquefaction sheet).
The soil springs and depths to fixity are calculated.

Effective Section Properties (section 5.6)

When calculating the section properties which were input for the steel shell piles into Seisab, the concrete contribution was multiplied by 0.4 to account for cracking. I think that is a reasonable value, but for more accuracy, the axial load and percent steel on the piles can be plotted on Figure 5.4 of the new guidelines to recieve in most cases a higher number.

Flexural stiffness (I), shear stiffness (G), and torsional stiffness (J), all must be reduced to account for the concrete cracking. The deck stiffness should be between 0.5 and 0.75 the full stiffness, I will use 0.75. The Torsional Moment should be 0.2 times the normal stiffness. The reduced values should be used in the Seisab model. Pile Caps should be assumed rigid (with full stiffness).

My Seisab run, previously completed will be modified by these stiffness parameters.

The change in period from full section properties is about 1%.

Conduct a Demand Analysis (section 5.1.2)

The model may need to be run serveral times for different conditions. For Mud Slough Ditch, Seisab was run once with abutment soil, and once ignoring the soil stiffness behind the abutments.

Determine Displacement Demands along Member Local Axis

1.12" at the abutments longitudinally, and about the same transversely. The piles are modeled individually in Com624p with a displacement of about 3" at the top. But the piles in Com624p had a free head, where in Seisab they are fixed at the top.

After this step, show spring constant calcs and liquefaction.

Liquefaction of Soils at Big Slough Ditch - Recommended LRFD Guidelines Appendix D

D.2.1 Preliminary Screening for Liquefaction

An evaluation of liquefaction hazard potential is not required if:

Groundwater is deeper than 15m.
Bedrock is under the site.
Blowcounts are above 30 for all soil samples.
Soil is clayey (see D-4 for restrictions).

Mud Slouch Ditch does not meet any of these conditions and therefore a liqefaction analysis should be performed. FHWA SA-97-076 Chapter 8 pg 111, states that if N > 30 for a soil layer, liquefaction will not occur. This is obvious from the guidelines chart on D-9 - and so any layer with N > 30 will not be checked.

The "simplified procedure" is mentioned on guidelines page D-6, and the depth to which it gives reasonable estimates is 12m. According to FHWA October 1996 book, "Seismic Design of Bridges - Design Example No. 3" p. A7, the max depth below ground surface for which liquefaction can occur is 50'. However the recommended depth to consider is 25m. This analysis will consider depths to 100ft, but it should be noted that beyond 40' the results will just be guidelines.

Simplified Method, previously known as Empirical Method

The water level shown on the boring log sheet is at a depth of 27.8'. Another boring log in the area, when the surface elevation is matched up, has water at a depth of 26.7'. Since there is a slight change of soil strength at 25', I will add the water table in at 25'.

A normalized SPT blowcount value is the base parameter which can be correlated with a graph to find the cyclic resistance ratio. This graph is on page D-9 for 7.5 magnitude earthquakes and on page D-10 there is a graph which shows correction factors to move the earthquake to different magnitudes.

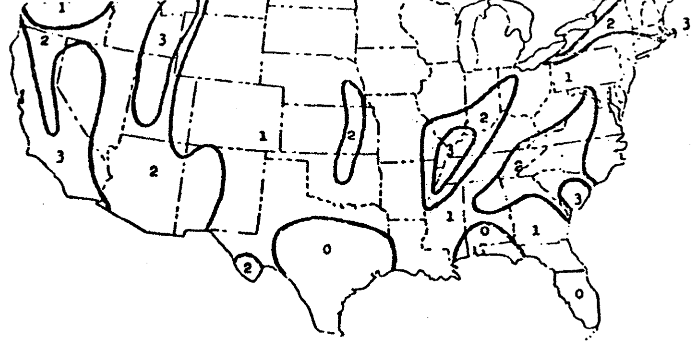
Assume the design earthquake magnitude is 8 (page 48). Figure D.2.4-2 shows a range of correction factors from about 0.6 to 0.9. Use a middle value of 0.75.

$A := 0.48$ Estimated acceleration at ground surface (SDC D) - The new guidelines give an SD1 of 0.53. Does this correlate to the A of the old code?

$M := 8$ EarthQuake Magnitude from 1975 AASHTO Interim Specifications Bridges p. 12. The M=8 maximum acceleration is 0.50, so A=0.48 puts us close. This chart is photocopied; see the next page.

$MSF := 0.75$ Magnitude Scaling Factor, to reduce stress resistance from the default M = 7.5 value to M = 8.

Bedrock Acceleration Contour Maps ... tion Contour Map in order that more exact values of peak rock acceleration can be used in design, making the specification a more practical and reasonable design criteria.



Zone 0—no damage
Zone 1—minor damage, MM. V and Vi
Zone 2—moderate damage, MM. VII
Zone 3—major damage, MM. VIII and larger

FIG. 1—Seismic risk map of the United States (Algermissen)

TABLE I—Representative Magnitude—Intensity Relations

Magnitude	Maximum Intensity (Modified Mercalli)
2	I-II
3	III
4	V
5	VI-VII
6	VII-VIII
7	IX-X
8	XI

TABLE II—Maximum Ground Accelerations and Durations of Strong Phase of Shaking

	Magnitude	Maximum acceleration (%g)	Duration (sec)
	5.0	9	2
	5.5	15	6
	6.0	22	12
	6.5	29	18
	7.0	37	24
	7.5	45	30
	$\boxed{8.0}$	$\boxed{50}$	34
	8.5	50	37
	[illegible]	[illegible]	
	[illegible]	12 [illegible]	
	6.75	33	
Use	6.50	30	

From Design Spectrum A ≈ 0.48

Not exact match but in ballpark

REFERENCES

1. Housner, G. W., "Strong Ground Motion," Chapter 4, "Earthquake Engineering," Robert L. Wiegel, Coordinating Editor, Prentice-Hall, Inc., Englewood Cliffs, New Jersey, 1970, page 79.
2. Podolny, Jr., Walter, "Toward an Understanding of Earthquakes," and James D. Cooper in U.S. F.H.W.A. publication "Highway Focus," Volume 6, No. 1, March 1974.
3. Algermissen, S. T., "Seismic Risk Studies in the United States," Proceedings Fourth World Conference on Earthquake Engineering, Volume 1, pages 14-27, Santiago, Chile, 1969.
4. Richter, C. F., "Elementary Seismology," W. H. Freeman and Company, San Francisco and London, 1958, page 353.

From 1975 AASHTO
Interim Specifications BRIDGES

Table II needs to be in the Guidelines

Layer 1 (0' to 4.5')

Moist, Medium Dense, Gray Sand and Gravel

$N_1 := 10$ $\gamma_1 := 110 \cdot \frac{lb}{ft^3}$ $\phi := 30$

$\sigma_1 := \gamma_1 \cdot 4.5ft$ $\sigma_1 = 495 \frac{lb}{ft^2}$

Layer 2 (4.5' to 15.5')

Moist, Medium Stiff to Stiff, Gray and Brown Clay with Sand Seams

$N_2 := 9$ $\gamma_2 := 120 \cdot \frac{lb}{ft^3}$

$\sigma_2 := \gamma_2 \cdot 11ft$ $\sigma_2 = 1.32 \times 10^3 \frac{lb}{ft^2}$

Layer 3 (15.5' to 25')

Wet, Medium Dense, Gray Sand

$N_3 := 16$ $\gamma_3 := 116 \frac{lb}{ft^3}$

$\sigma_3 := \gamma_3 \cdot 9.5ft$ $\sigma_3 = 1.102 \times 10^3 \frac{lb}{ft^2}$

Layer 4 (25' to 35') - Beginning of Water Table

Wet, Medium Dense, Gray Sand

$N_4 := 18$ $\gamma_4 := 60 \frac{lb}{ft^3}$ $t := 10ft$ thickness of layer

$\sigma_4 := \gamma_4 \cdot t$ $\sigma_4 = 600 \frac{lb}{ft^2}$ $z := 30ft$ depth to middle of layer

$\sigma_o := \sigma_1 + \sigma_2 + \sigma_3 + \frac{\sigma_4}{2}$ $\sigma_o = 1.571 \times 10^4 \frac{kg}{m^2}$ overburden weight above middle of layer

$r_d := 1 - \frac{0.015z}{3.281ft}$ $r_d = 0.863$ $\tau_{av} := 0.65 \cdot A \cdot \sigma_o \cdot r_d$

$\tau_{av} = 4.228 \times 10^3 \frac{kg}{m^2}$ Stress Ratio: $ratio_\sigma := \frac{\tau_{av}}{\sigma_o}$ $ratio_\sigma = 0.269$

Liquefaction Strength

$\sigma_o = 1.608 \frac{ton}{ft^2}$ Use this number on Figure 8 (Seismic Design of Highway Bridge Foundations, vol 2, p. 31) to find Cn. $C_n := 0.79$

Corrected Blowcount: $N_{4c} := C_n \cdot N_4$ $N_{4c} = 14.22$

Use the corrected blowcount to find the resisting stress ratio on Figure D.2.4-1

$ratio_r = \frac{\tau}{\sigma_v}$ $ratio_r := 0.155$ Resisting Stress - I have no data on the percent fines, will use the < 5% curve. Seems weakest.

$FS_4 := \frac{MSF \cdot ratio_r}{ratio_\sigma}$ $FS_4 = 0.432$ Factor of Safety

$$SAFE := \begin{vmatrix} s \leftarrow \text{"YES"} \quad \text{if } FS_4 > 1 \\ s \leftarrow \text{"NO"} \quad \text{otherwise} \\ \text{return } s \end{vmatrix}$$

$SAFE = \text{"NO"}$

Layer 5 (35' to 55')

Wet, Medium Dense, Gray Sand

$N_5 := 27$ $\gamma_5 := 67 \frac{lb}{ft^3}$ $t := 20ft$ thickness of layer

$\sigma_5 := \gamma_5 \cdot t$ $\sigma_5 = 1.34 \times 10^3 \frac{lb}{ft^2}$ $z := 45ft$ depth to middle of layer

$\sigma_o := \sigma_1 + \sigma_2 + \sigma_3 + \sigma_4 + \frac{\sigma_5}{2}$ $\sigma_o = 2.044 \times 10^4 \frac{kg}{m^2}$ overburden weight above middle of layer

$r_d := 1 - \frac{0.015z}{3.281ft}$ $r_d = 0.794$

$\tau_{av} := 0.65 \cdot A \cdot \sigma_o \cdot r_d$ $\tau_{av} = 5.066 \times 10^3 \frac{kg}{m^2}$

Stress Ratio: $ratio_\sigma := \frac{\tau_{av}}{\sigma_o}$ $ratio_\sigma = 0.248$

Liquefaction Strength

$\sigma_o = 2.093\,\frac{ton}{ft^2}$ $\quad C_n := 0.68$ $\quad N_{5c} := C_n \cdot N_5$ $\quad N_{5c} = 18.36$

$ratio_r = \frac{\tau}{\sigma_v}$ $\quad ratio_r := 0.20$ Resisting Stress

$FS_5 := \frac{MSF \cdot ratio_r}{ratio_\sigma}$ $\quad FS_5 = 0.605$ Factor of Safety

$$SAFE := \left| \begin{array}{l} s \leftarrow \text{"YES"} \quad \text{if } FS_5 > 1 \\ s \leftarrow \text{"NO"} \quad \text{otherwise} \\ \text{return } s \end{array} \right.$$

Layer 6 (55' to 90')

$SAFE = \text{"NO"}$

Wet, Dense, Gray Sand

$N_6 := 44$ $\quad \gamma_6 := 74\,\frac{lb}{ft^3}$ $\quad t := 35ft$ thickness of layer

$\sigma_6 := \gamma_6 \cdot t$ $\quad \sigma_6 = 2.59 \times 10^3\,\frac{lb}{ft^2}$ $\quad z := 72.5ft$ depth to middle of layer

$\sigma_o := \sigma_1 + \sigma_2 + \sigma_3 + \sigma_4 + \sigma_5 + \frac{\sigma_6}{2}$ $\quad \sigma_o = 3.004 \times 10^4\,\frac{kg}{m^2}$ overburden weight above middle of layer

$r_d := 1 - \frac{0.015z}{3.281ft}$ $\quad r_d = 0.669$

$\tau_{av} := 0.65 \cdot A \cdot \sigma_o \cdot r_d$ $\quad \tau_{av} = 6.265 \times 10^3\,\frac{kg}{m^2}$

Stress Ratio: $\quad ratio_\sigma := \frac{\tau_{av}}{\sigma_o}$ $\quad ratio_\sigma = 0.209$

Liquefaction Strength

$\sigma_o = 3.076\,\frac{ton}{ft^2}$ $\quad C_n := 0.53$ $\quad N_{6c} := C_n \cdot N_6$ $\quad N_{6c} = 23.32$

$ratio_r = \frac{\tau}{\sigma_v}$ $\quad ratio_r := 0.265$ Resisting Stress

$FS_6 := \frac{MSF \cdot ratio_r}{ratio_\sigma}$ $\quad FS_6 = 0.953$ Factor of Safety

$$SAFE := \left| \begin{array}{l} s \leftarrow \text{"YES"} \quad \text{if } FS_6 > 1 \\ s \leftarrow \text{"NO"} \quad \text{otherwise} \\ \text{return } s \end{array} \right.$$

$SAFE = \text{"NO"}$

Layer 7 (90' to 101.5')

Moist, Medium Stiff, Gray Clay with Organic Matter

$N_7 := 6$ $\gamma_7 := 58\frac{lb}{ft^3}$ $t := 11.5ft$ thickness of layer

$\sigma_7 := \gamma_7 \cdot 11.5ft$ $\sigma_7 = 667\frac{lb}{ft^2}$ $z := 95.75ft$ depth to middle of layer

$\sigma_o := \sigma_1 + \sigma_2 + \sigma_3 + \sigma_4 + \sigma_5 + \sigma_6 + \frac{\sigma_7}{2}$ $\sigma_o = 3.799 \times 10^4 \frac{kg}{m^2}$ overburden weight above middle of layer

$r_d := 1 - \frac{0.015z}{3.281ft}$ $r_d = 0.562$

$\tau_{av} := 0.65 \cdot A \cdot \sigma_o \cdot r_d$ $\tau_{av} = 6.664 \times 10^3 \frac{kg}{m^2}$

Stress Ratio: $ratio_\sigma := \frac{\tau_{av}}{\sigma_o}$ $ratio_\sigma = 0.175$

Liquefaction Strength

$\sigma_o = 3.89\frac{ton}{ft^2}$ $C_n := 0.45$ $N_{7c} := C_n \cdot N_7 + 7.5$ $N_{7c} = 10.2$

This layer is a clay and will generally be fine. Using Silt equation for N_{7c}.

$ratio_r = \frac{\tau}{\sigma_v}$ $ratio_r := 0.11$ Resisting Stress

$FS_7 := \frac{MSF \cdot ratio_r}{ratio_\sigma}$ $FS_7 = 0.47$ Factor of Safety

$$SAFE := \left| \begin{array}{l} s \leftarrow \text{"YES"} \quad \text{if } FS_7 > 1 \\ s \leftarrow \text{"NO"} \quad \text{otherwise} \\ \text{return } s \end{array} \right.$$

$SAFE = \text{"NO"}$

There are serious liquefaction problems here. Under current provisions no layer liquifies - see calculations starting on page 142.

Combine Orthogonal Displacements - Step 7

Combination of Load Cases 1 and 2 (Section 4.4)

This combination is done automatically by WinSeisab. But here done manually for show. Using "soil down.ssb" since it produces the largest displacements.

Load Case	Long.	Trans.
1	0.093	0
2	0	0.087
3	0.093	0.026
4	0.028	0.087

Load Case 3 (LC 1 + 30% of LC 2)

$trans := 0.087 \cdot 0.3 \qquad trans = 0.026$

Load Case 4 (LC 2 + 30% of LC 1)

$long := 0.093 \cdot 0.3 \qquad long = 0.028$

SDC D, Determine Δ - Pushover - Step 8 & 9

$kip := 1000lb$

$ksi := \frac{kip}{in^2}$

Structure Displacement Capacity for SDC B, C, D. (Section 4.8)

WinSeisab does not have a pushover analysis, now required for SDC D. Therefore com624p was used to analyze the interior bent piles with an estimated depth to fixity of 15'. From AASHTO 6.12.2.3.2, unless a steel shell pile is encased in concrete, the maximum moment is the plastic moment of the steel section alone (even with the tube filled with concrete). To be sure the pile can attain this moment, the following check shall be performed.

Since the column is not encased in concrete at max moment location use AASHTO 6.12.2.3.2.

$D := 17.875in$ diameter of the pipe

$t := 0.375in$ thickness of pipe

$E := 29000ksi$

$F_y := 42ksi$

If $\frac{D}{t} < 2.0 \cdot \sqrt{\frac{E}{F_y}}$ Then $M_n = M_p$ $\frac{D}{t} = 47.667$

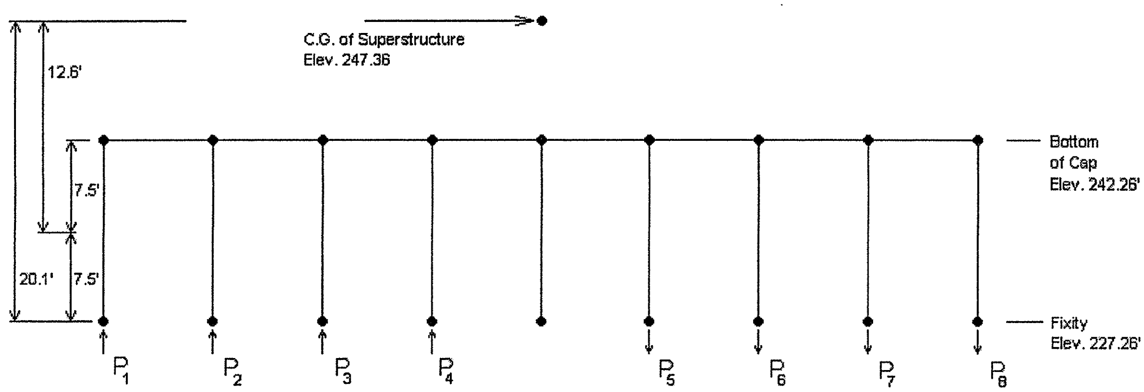
Otherwise $M_n = M_y$ $2\sqrt{\frac{E}{F_y}} = 52.554$

$M_p := 401.953 \cdot kip \cdot ft$ $M_y := 309.212 kip \cdot ft$ See pages 51 and 52.

$$M_n := \begin{vmatrix} M \leftarrow M_p \text{ if } \frac{D}{t} < 2 \cdot \sqrt{\frac{E}{F_y}} \\ M \leftarrow M_y \text{ otherwise} \\ \text{return } M \end{vmatrix}$$

$M_n = 401.953\, kip \cdot ft$

Pushover Analysis Sketch



$EL_{CGSS} := 247.36ft$ Elevation of the superstructure center of gravity.

$EL_{BOT} := 227.26ft$ Elevation of the bottom of the pile

$EL_{TOP} := 242.26ft$ Elevation of the bottom of cap (top of pile).

$H := EL_{CGSS} - EL_{BOT} - \frac{15ft}{2}$ $H = 12.6\,ft$ Distance from column contraflexure to superstructure c.g.

$H_{col} := 15ft$

$V := 302.1kip$ Max Bent Pseudoforce from Seisab

$s_c := 9ft$ Column spacing

The bent force will be divided to the individual piles. To do so, balance the moment in the bent. In each balance equation, the sum of the distance to each pile from the c.g. squared, will be needed.

$s_{sum} := 2\cdot\left[s_c^{\,2} + (2s_c)^2 + (3s_c)^2 + (4s_c)^2\right]$ $s_{sum} = 4.86 \times 10^3\,ft^2$

$P_1 := V\cdot H\cdot\left(\frac{4\cdot s_c}{s_{sum}}\right)$ $P_1 = 28.196\,kip$ $P_8 := P_1$

$P_2 := V\cdot H\cdot\left(\frac{3\cdot s_c}{s_{sum}}\right)$ $P_2 = 21.147\,kip$ $P_7 := P_2$

$$P_3 := V \cdot H \cdot \left(\frac{2 \cdot s_c}{s_{sum}}\right) \qquad P_3 = 15.321\,\text{kip} \qquad P_6 := P_3$$

$$P_4 := V \cdot H \cdot \left(\frac{s_c}{s_{sum}}\right) \qquad P_4 = 7.66\,\text{kip} \qquad P_5 := P_4$$

$P_c := 38.63\text{kip}$ $\quad P_w := 12.5\text{kip}$ $\quad$ Dead Load, concrete and wearing surface from MDX

$$P_{dl} := 0.9 \cdot P_c + 0.65 \cdot P_w \qquad P_{dl} = 42.892\,\text{kip}$$

Combine the earthquake load with dead load. Then using Com624p, with 15' to fixity in rock and no soil, find the moment and rotation that goes with each load.

Plastic Capacity of Steel Tube w/ Axial Load (Reserve Moment Capacity w/ given Axial load)

$P_1 := P_{dl} - P_1$	$P_1 = 12.25\,\text{kip}$	$M_{p1} := 374.29\text{kip}\cdot\text{ft}$	$\phi_{y1} := 0.001$	$\phi_{p1} := 0.00988$
$P_2 := P_{dl} - P_2$	$P_2 = 19.911\,\text{kip}$	$M_{p2} := 382.47\text{kip}\cdot\text{ft}$	$\phi_{y2} := 0.001103$	$\phi_{p2} := 0.00988$
$P_3 := P_{dl} - P_3$	$P_3 = 27.571\,\text{kip}$	$M_{p3} := 373.97\text{kip}\cdot\text{ft}$	$\phi_{y3} := 0.001103$	$\phi_{p3} := 0.00988$
$P_4 := P_{dl} - P_4$	$P_4 = 35.232\,\text{kip}$	$M_{p4} := 373.97\text{kip}\cdot\text{ft}$	$\phi_{y4} := 0.001103$	$\phi_{p4} := 0.00988$
$P_5 := P_{dl} + P_5$	$P_5 = 50.552\,\text{kip}$	$M_{p5} := 403.01\text{kip}\cdot\text{ft}$	$\phi_{y5} := 0.001073$	$\phi_{p5} := 0.00988$
$P_6 := P_{dl} + P_6$	$P_6 = 58.213\,\text{kip}$	$M_{p6} := 401.71\text{kip}\cdot\text{ft}$	$\phi_{y6} := 0.001043$	$\phi_{p6} := 0.00988$
$P_7 := P_{dl} + P_7$	$P_7 = 65.873\,\text{kip}$	$M_{p7} := 401.71\text{kip}\cdot\text{ft}$	$\phi_{y7} := 0.001043$	$\phi_{p7} := 0.00988$
$P_8 := P_{dl} + P_8$	$P_8 = 73.534\,\text{kip}$	$M_{p8} := 401.71\text{kip}\cdot\text{ft}$	$\phi_{y8} := 0.001043$	$\phi_{p8} := 0.00988$

Are the above values from Com624p reasonable? Lets run the numbers by hand for P_8. In the AISC Steel LRFD 3rd Ed., Table 4-7 on page 4-66 for a unfilled 18" steel pipe with t=0.375" that is 15' long (assume k=1, fixed-fixed):

$$P_u := 658\text{kip} \qquad \frac{P_8}{P_u} = 0.112$$

From AASHTO LRFD 6.9.2.2 for P/Pu < 0.2

$$\frac{P}{2P_u} + \frac{M}{M_n} < 1 \qquad M := \left(1 - \frac{P_8}{2P_u}\right) \cdot M_p \qquad M = 380.491\,\text{kip}\cdot\text{ft}$$

The moment given by AASHTO LRFD 6.9.2.2 is fairly close to the 401.71k' given by Com624p and so it could also be assumed that the accompanying rotation is correct.

Check to see if the columns can handle the shear from this maximum moment.

$V_{p1} := \dfrac{M_{p1}}{0.5H_{col}}$ $\quad V_{p1} = 49.905\,\text{kip}$

$V_{p2} := \dfrac{M_{p2}}{0.5H_{col}}$ $\quad V_{p2} = 50.996\,\text{kip}$

$V_{p3} := \dfrac{M_{p3}}{0.5H_{col}}$ $\quad V_{p3} = 49.863\,\text{kip}$

$V_{p4} := \dfrac{M_{p4}}{0.5H_{col}}$ $\quad V_{p4} = 49.863\,\text{kip}$

$V_{p5} := \dfrac{M_{p5}}{0.5H_{col}}$ $\quad V_{p5} = 53.735\,\text{kip}$

$V_{p6} := \dfrac{M_{p6}}{0.5H_{col}}$ $\quad V_{p6} = 53.561\,\text{kip}$

$V_{p7} := \dfrac{M_{p7}}{0.5H_{col}}$ $\quad V_{p7} = 53.561\,\text{kip}$

$V_{p8} := \dfrac{M_{p8}}{0.5H_{col}}$ $\quad V_{p8} = 53.561\,\text{kip}$

$V_p := V_{p1} + V_{p2} + V_{p3} + V_{p4} + V_{p5} + V_{p6} + V_{p7} + V_{p8}$ $\quad V_p = 415.045\,\text{kip}$

The shear from moment is far more than the shear directly from the earthquake. The earthquake shear should be adjusted upwards. And the process iterated.

$V := 410\text{kip}$ Try 410 kips

$P_1 := V \cdot H \cdot \left(\dfrac{4 \cdot s_c}{s_{sum}}\right)$ $\quad P_1 = 41.586\,\text{kip}$ $\quad P_8 := P_1$

$P_2 := V \cdot H \cdot \left(\dfrac{3 \cdot s_c}{s_{sum}}\right)$ $\quad P_2 = 31.19\,\text{kip}$ $\quad P_7 := P_2$

$P_3 := V \cdot H \cdot \left(\dfrac{2 \cdot s_c}{s_{sum}}\right)$ $\quad P_3 = 20.793\,\text{kip}$ $\quad P_6 := P_3$

$P_4 := V \cdot H \cdot \left(\dfrac{s_c}{s_{sum}}\right)$ $\quad P_4 = 10.397\,\text{kip}$ $\quad P_5 := P_4$

Combine the earthquake load with dead load. Then using Com624p, with 15' to fixity in rock and no soil, find the moment and rotation that goes with each load.

$P_1 := P_{dl} - P_1$ $\quad P_1 = 1.306\,\text{kip}$ $\quad M_{p1} := 374.29\text{kip}\cdot\text{ft}$

$P_2 := P_{dl} - P_2$ $\quad P_2 = 11.702\,\text{kip}$ $\quad M_{p2} := 374.29\text{kip}\cdot\text{ft}$

$P_3 := P_{dl} - P_3$ $\quad P_3 = 22.099\,\text{kip}$ $\quad M_{p3} := 373.97\text{kip}\cdot\text{ft}$

$P_4 := P_{dl} - P_4$ $\quad P_4 = 32.495\,\text{kip}$ $\quad M_{p4} := 373.97\text{kip}\cdot\text{ft}$

$P_5 := P_{dl} + P_5$ $\quad P_5 = 53.289\,\text{kip}$ $\quad M_{p5} := 403.01\text{kip}\cdot\text{ft}$

$P_6 := P_{dl} + P_6$ $\quad P_6 = 63.685\,\text{kip}$ $\quad M_{p6} := 401.71\text{kip}\cdot\text{ft}$

$P_7 := P_{dl} + P_7$ $\quad P_7 = 74.082\,\text{kip}$ $\quad M_{p7} := 401.59\text{kip}\cdot\text{ft}$

$P_8 := P_{dl} + P_8$ $\quad P_8 = 84.478\,\text{kip}$ $\quad M_{p8} := 370.05\text{kip}\cdot\text{ft}$

$$V_{p1} := \frac{M_{p1}}{0.5H_{col}} \quad V_{p1} = 49.905\,\text{kip} \qquad V_{p5} := \frac{M_{p5}}{0.5H_{col}} \quad V_{p5} = 53.735\,\text{kip}$$

$$V_{p2} := \frac{M_{p2}}{0.5H_{col}} \quad V_{p2} = 49.905\,\text{kip} \qquad V_{p6} := \frac{M_{p6}}{0.5H_{col}} \quad V_{p6} = 53.561\,\text{kip}$$

$$V_{p3} := \frac{M_{p3}}{0.5H_{col}} \quad V_{p3} = 49.863\,\text{kip} \qquad V_{p7} := \frac{M_{p7}}{0.5H_{col}} \quad V_{p7} = 53.545\,\text{kip}$$

$$V_{p4} := \frac{M_{p4}}{0.5H_{col}} \quad V_{p4} = 49.863\,\text{kip} \qquad V_{p8} := \frac{M_{p8}}{0.5H_{col}} \quad V_{p8} = 49.34\,\text{kip}$$

$$V_p := V_{p1} + V_{p2} + V_{p3} + V_{p4} + V_{p5} + V_{p6} + V_{p7} + V_{p8} \qquad V_p = 409.717\,\text{kip}$$

Close. Iteration complete. Com624 also gives the stiffness of the pile.

$EI_1 := 27529.2\text{kip}\cdot\text{ft}^2$ $\quad EI_4 := 28254.2\text{kip}\cdot\text{ft}^2$ $\quad EI_7 := 32085.4\text{kip}\cdot\text{ft}^2$ $\quad \phi_{y8} := \dfrac{0.001013}{\text{in}}$

$EI_2 := 27529.2\text{kip}\cdot\text{ft}^2$ $\quad EI_5 := 31299.3\text{kip}\cdot\text{ft}^2$ $\quad EI_8 := 30441.7\text{kip}\cdot\text{ft}^2$ $\quad \phi_{p1} := \dfrac{0.00970}{\text{in}}$

$EI_3 := 28254.2\text{kip}\cdot\text{ft}^2$ $\quad EI_6 := 32095.8\text{kip}\cdot\text{ft}^2$ $\quad \phi_{p8} := \dfrac{0.00988}{\text{in}}$

$$k_1 := \frac{3 \cdot EI_1}{(0.5H_{col})^3} \qquad k_1 = 195.763 \frac{kip}{ft} \qquad \Delta y_1 := \frac{V_{p1}}{k_1} \qquad \Delta y_1 = 3.059\,in$$

$$k_2 := \frac{3 \cdot EI_2}{(0.5H_{col})^3} \qquad k_2 = 195.763 \frac{kip}{ft} \qquad \Delta y_2 := \frac{V_{p2}}{k_2} \qquad \Delta y_2 = 3.059\,in$$

$$k_3 := \frac{3 \cdot EI_3}{(0.5H_{col})^3} \qquad k_3 = 200.919 \frac{kip}{ft} \qquad \Delta y_3 := \frac{V_{p3}}{k_3} \qquad \Delta y_3 = 2.978\,in$$

$$k_4 := \frac{3 \cdot EI_4}{(0.5H_{col})^3} \qquad k_4 = 200.919 \frac{kip}{ft} \qquad \Delta y_4 := \frac{V_{p4}}{k_4} \qquad \Delta y_4 = 2.978\,in$$

$$k_5 := \frac{3 \cdot EI_5}{(0.5H_{col})^3} \qquad k_5 = 222.573 \frac{kip}{ft} \qquad \Delta y_5 := \frac{V_{p5}}{k_5} \qquad \Delta y_5 = 2.897\,in$$

$$k_6 := \frac{3 \cdot EI_6}{(0.5H_{col})^3} \qquad k_6 = 228.237 \frac{kip}{ft} \qquad \Delta y_6 := \frac{V_{p6}}{k_6} \qquad \Delta y_6 = 2.816\,in$$

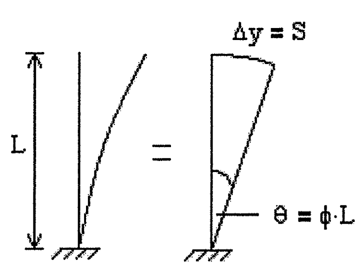
$$k_7 := \frac{3 \cdot EI_7}{(0.5H_{col})^3} \qquad k_7 = 228.163 \frac{kip}{ft} \qquad \Delta y_7 := \frac{V_{p7}}{k_7} \qquad \Delta y_7 = 2.816\,in$$

$$k_8 := \frac{3 \cdot EI_8}{(0.5H_{col})^3} \qquad k_8 = 216.474 \frac{kip}{ft} \qquad \Delta y_8 := \frac{V_{p8}}{k_8} \qquad \Delta y_8 = 2.735\,in$$

Calculate Displacement Capacities - choose **case 8** since it has highest load.

Longitudinal Direction

$$\Delta y := \phi_{y8} \cdot \frac{H_{col}{}^2}{3} \qquad \Delta y = 10.94\,in$$



This formula is from the Holmes St. Tennessee example. I can not derive this equation and do not know where it comes from. On a circle, a secant length can be found by multiplying the interior angle by the radius. Δy is analogous to the secant, and ϕy times a length must be analagous to the radius. So for a cantilevered pile, $\Delta y = \phi L^2$. For a pinned-pinned pile, Δy would be half since the max deflection would occur in the middle. For a fixed-fixed pile with more stiffness on the bottom, the deflection would be smaller yet and towards the bottom, perhaps typified by the formula used here.

Length of plastic yielding is the maximum of the following (New Guidelines 4.11.8):

$$L_p := \begin{array}{|l} L \leftarrow \dfrac{H_{col}}{8} \ \text{if} \ \dfrac{H_{col}}{8} > 18in \\ L \leftarrow 18in \ \text{otherwise} \\ \text{return } L \end{array}$$

$L_p = 22.5\,in$

$$\Delta p := (\phi_{p8} - \phi_{y8}) \cdot L_p \cdot \left(H_{col} - \frac{L_p}{2}\right)$$

$\Delta p = 33.667\,in$

Where did Tennessee get this equation?

$$\Delta u := \Delta p + \Delta y$$

$\Delta u = 44.607\,in$

At this displacement earthquake resisting elements reach their inelastic deformation capacity.

$R_d := 1.805$

Deflection multiplier. Note: To calculate this value, R=3 from SDC C was used since it seemed more conservative than the SDC D R=0.9.

$$\Delta_{long} := R_d \cdot 0.087ft$$

$\Delta_{long} = 1.884\,in$

Magnified deflection, see section 4.3, and the Seisab results from Step 7.

$$\Delta_{pd} := \begin{array}{|l} \Delta \leftarrow \Delta_{long} - \Delta y \ \text{if} \ \Delta_{long} - \Delta y > 0in \\ \Delta \leftarrow 0in \ \text{otherwise} \\ \text{return } \Delta \end{array}$$

$\Delta_{pd} = 0\,in$

Member Ductility Requirement (Section 4.9)

$$\mu_D := 1 + \frac{\Delta_{pd}}{\Delta y}$$

$\mu_D = 1$

For multiple column bents, μ_D should be less than 8.

$$RESULT := \begin{array}{|l} R \leftarrow \text{"PASS"} \ \text{if} \ \mu_D < 8 \\ R \leftarrow \text{"FAIL"} \ \text{otherwise} \\ \text{return } R \end{array}$$

RESULT = "PASS"

Calculate Displacement Capacities - choose **case 8** since it has highest load.

<u>Transverse Direction</u>

$$\Delta y := \phi_{y8} \cdot \frac{\left(\frac{H_{col}}{2}\right)^2}{3} \qquad \Delta y = 2.735\,in$$

Since the transverse direction is stiffer than the longitudinal, we can use half the height.

Length of plastic yielding is the maximum of the following (New Guidelines 4.11.8):

$$L_p := \left| \begin{array}{l} L \leftarrow \frac{H_{col}}{8} \ \text{ if } \ \frac{H_{col}}{8} > 18in \\ L \leftarrow 18in \ \text{ otherwise} \\ \text{return } L \end{array} \right. \qquad \boxed{L_p = 22.5\,in}$$

$$\Delta p := (\phi_{p8} - \phi_{y8}) \cdot L_p \cdot \left(\frac{H_{col}}{2} - \frac{L_p}{2}\right) \qquad \Delta p = 15.711\,in$$

Where did Tennessee get equation?

$$\Delta u := \Delta p + \Delta y \qquad \Delta u = 18.446\,in$$

At this displacement earthquake resisting elements reach their inelastic deformation capacity.

$R_d := 1.805$ Deflection multiplier. Note: To calculate this value, R=3 from SDC C was used since it seemed more conservative than the SDC D R=0.9.

$$\Delta_{long} := R_d \cdot 0.093ft \qquad \Delta_{long} = 2.014\,in$$

Magnified deflection, see section 4.3, and the Seisab results from Step 7.

$$\Delta_{pd} := \left| \begin{array}{l} \Delta \leftarrow \Delta_{long} - \Delta y \ \text{ if } \ \Delta_{long} - \Delta y > 0in \\ \Delta \leftarrow 0in \ \text{ otherwise} \\ \text{return } \Delta \end{array} \right. \qquad \boxed{\Delta_{pd} = 0\,in}$$

Member Ductility Requirement (Section 4.9)

$$\mu_D := 1 + \frac{\Delta_{pd}}{\Delta y} \qquad \mu_D = 1$$

For multiple column bents, μ_D should be less than 8.

$$\text{RESULT} := \left| \begin{array}{l} R \leftarrow \text{"PASS"} \ \text{ if } \ \mu_D < 8 \\ R \leftarrow \text{"FAIL"} \ \text{ otherwise} \\ \text{return } R \end{array} \right. \qquad \boxed{\text{RESULT} = \text{"PASS"}}$$

SDC C, Pushover would not be required - Step 8

Structure Displacement Capacity for SDC B, C (Section 4.8)

WinSeisab does not have a pushover analysis, now required for SDC D. The following is a simple analysis that bypasses the need for a pushover analysis if the bridge is in SDC C or lower.

$H_o := 15ft$ — Column height from bottom of footing (or end of pile) to top of column. Equivalently, the shortest distance between the point of maximum moment and the contra-flexure point. So the depth to fixity.

$\Lambda := 2$ — Fixity Factor. Equal to 1 for a fixed base, free head. Equal to 2 for a fixed-fixed condition. The value can be interpolated for head conditions between free and fixed. Use 2 for transverse, and 1 for longitudinal.

$B_o := 1.5ft$ — Column width or diameter

$$x := \frac{\Lambda \cdot B_o}{H_o} \qquad x = 0.2$$

The displacement capacity is equal to the larger of the two equations below.

$$\Delta_1 := \frac{H_o}{100ft}\cdot(-2.32\cdot\ln(x) - 1.22)\cdot ft \qquad \Delta_1 = 4.525\,in$$

$$\Delta_2 := \frac{H_o}{100ft} ft \qquad \Delta_2 = 1.8\,in$$

$$\Delta_c := \left| \begin{array}{l} \Delta \leftarrow \Delta_1 \text{ if } \Delta_1 > \Delta_2 \\ \Delta \leftarrow \Delta_2 \text{ otherwise} \\ \text{return } \Delta \end{array} \right.$$

At this displacement earthquake resisting elements acheive their inelastic deformation capacity.

$\Delta_c = 4.525\,in$

This value is 7.42 inches in the longitudinal direction.

SDC C, Global Structure Displacement Requirement - Step 9

Determination of Seismic Lateral Displacements Demands (Section 4.3)

If the bridge is in SDC C, the deflection equation comparison is very simple.

Because the bridge is moving as a unit, and there is no skew, displacement is the same at the abutments and interior bents.

The lateral deflection from Seisab is:

$\Delta_{lat} := 0.093\text{ft}$

The deflection is multiplied by R_d and then compared with the pushover deflection. If less, the deflection check is passed.

$R_d := 1.805$ $\quad\quad$ $\Delta_{mag} := \Delta_{lat} \cdot R_d$ $\quad\quad$ $\Delta_{mag} = 2.014\,\text{in}$

Inelastic Deflection for seismic elements. $\quad\quad$ $\Delta_c := 4.53\text{in}$

$$\Delta_c := \left| \begin{array}{l} \Delta \leftarrow \text{"OK"} \quad \text{if } \Delta_c > \Delta_{mag} \\ \Delta \leftarrow \text{"NO GOOD"} \quad \text{otherwise} \\ \text{return } \Delta \end{array} \right.$$

Is the bridge deflection within acceptable range?

$\Delta_c = \text{"OK"}$

FOR PUSHOVER ANALYSIS

18" Steel Shell Shaft with Axial and Shear on top 07041

****
ULTIMATE BENDING RESISTANCE AND FLEXURIAL RIGIDITY
****

DIAMETER = 17.88 IN

STEEL SHELL THICKNESS = .38 IN

STEEL SHELL YIELD STRENGTH = 42.000000 KIP/IN**2

MODULUS OF ELASTICITY OF STEEL = 29000.000000 KIP/IN**2
SQUASH LOAD CAPACITY = 877.45 KIP

OUTPUT RESULTS FOR AN AXIAL LOAD = 73.53 KIP / 65.87 / 58.21 /
****

MOMENT IN-KIP	EI = $\frac{M}{\text{rotation}}$ KIP-IN**2	rotation PHI 1/IN	MAX STR IN/IN	N AXIS IN
23.081	.23081E+08	.000001	.00013	130.691
115.407	.23081E+08	.000005	.00016	33.290
207.732	.23081E+08	.000009	.00020	22.468
300.058	.23081E+08	.000013	.00023	18.305
385.503	.22677E+08	.000017	.00027	16.092
479.810	.22848E+08	.000021	.00030	14.695
552.204	.22088E+08	.000025	.00033	13.708
669.484	.23086E+08	.000029	.00037	13.112
719.203	.21794E+08	.000033	.00040	12.516
854.118	.23084E+08	.000037	.00044	12.218
943.858	.23021E+08	.000041	.00047	11.899
970.779	.21573E+08	.000045	.00051	11.622
1053.496	.21500E+08	.000049	.00054	11.324
1223.396	.23083E+08	.000053	.00056	11.026
1899.399	.22884E+08	.000083	.00083	10.371
2608.236	.23082E+08	.000113	.00107	9.834
3251.436	.22737E+08	.000143	.00134	9.729
3804.720	.21993E+08	.000173	.00161	9.680
4205.126	.20715E+08	.000203	.00192	9.834
4379.593	.18797E+08	.000233	.00220	9.834
4488.631	.17067E+08	.000263	.00249	9.834
4561.499	.15568E+08	.000293	.00277	9.834
4613.036	.14282E+08	.000323	.00305	9.834
4651.795	.13178E+08	.000353	.00334	9.834
4681.373	.12223E+08	.000383	.00362	9.834
4704.241	.11390E+08	.000413	.00390	9.834
4722.417	.10660E+08	.000443	.00419	9.834
4737.519	.10016E+08	.000473	.00447	9.834
4749.829	.94430E+07	.000503	.00476	9.834
4759.709	.89300E+07	.000533	.00504	9.834
4768.614	.84700E+07	.000563	.00532	9.834
4775.615	.80533E+07	.000593	.00561	9.834
4781.829	.76755E+07	.000623	.00589	9.834
4787.697	.73318E+07	.000653	.00617	9.834
4791.838	.70159E+07	.000683	.00646	9.834
4795.979	.67265E+07	.000713	.00674	9.834
4800.119	.64605E+07	.000743	.00702	9.834
4802.959	.62134E+07	.000773	.00731	9.834
4805.557	.59845E+07	.000803	.00759	9.834

4808.155	.57721E+07	.000833	.00788	9.834
4810.753	.55745E+07	.000863	.00816	9.834
4813.009	.53897E+07	.000893	.00844	9.834
4814.511	.52162E+07	.000923	.00873	9.834
4816.012	.50535E+07	.000953	.00901	9.834
4817.514	.49008E+07	.000983	.00929	9.834
4819.016	.47572E+07	.001013	.00958	9.834
4820.518	.46218E+07	.001043	.00986	9.834

18" Steel Shell Shaft with Axial and Shear on top 07041

UNITS--ENGL

****

PILE DEFLECTION, BENDING MOMENT, SHEAR & SOIL RESISTANCE

****

I N P U T   I N F O R M A T I O N

****

THE LOADING IS STATIC

PILE GEOMETRY AND PROPERTIES

PILE LENGTH = 300.00 IN
MODULUS OF ELASTICITY OF PILE = .290E+05 KIP/IN**2
1 SECTION(S)

X	DIAMETER	MOMENT OF INERTIA	AREA
IN	IN	IN**4	IN**2
.00			
	17.875	.789E+03	.206E+02
300.00			

SOILS INFORMATION

X-COORDINATE AT THE GROUND SURFACE = 60.00 IN

SLOPE ANGLE AT THE GROUND SURFACE = .00 DEG.

1 LAYER(S) OF SOIL

LAYER 1
THE LAYER IS A ROCK
X AT THE TOP OF THE LAYER = 170.00 IN
X AT THE BOTTOM OF THE LAYER = 1000.00 IN
VARIATION OF SOIL MODULUS, k = .200E+04 LBS/IN**3

DISTRIBUTION OF EFFECTIVE UNIT WEIGHT WITH DEPTH
2 POINTS
X,IN WEIGHT,LBS/IN**3

170.00	.75E-01
1000.00	.75E-01

DISTRIBUTION OF STRENGTH PARAMETERS WITH DEPTH
2 POINTS

X,IN	C,LBS/IN**2	PHI,DEGREES	E50
170.00	.284E+04	.000	.100E-02
1000.00	.284E+04	.000	.100E-02

FINITE DIFFERENCE PARAMETERS

NUMBER OF PILE INCREMENTS	=	60
TOLERANCE ON DETERMINATION OF DEFLECTIONS	=	.100E-03 IN
MAXIMUM NUMBER OF ITERATIONS ALLOWED FOR PILE ANALYSIS	=	100
MAXIMUM ALLOWABLE DEFLECTION	=	.36E+03 IN

INPUT CODES

OUTPT = 1
KCYCL = 1
KBC = 1
KPYOP = 1
INC = 2

18" Steel Shell Shaft with Axial and Shear on top 07041

UNITS--ENGL

O U T P U T   I N F O R M A T I O N
****

GENERATED P-Y CURVES

THE NUMBER OF CURVE IS = 4

THE NUMBER OF POINTS ON EACH CURVE = 17

DEPTH BELOW GS IN	DIAM IN	C KIP/IN**2
-60.00	17.875	.3E+04

Y IN	P KIP/IN
.000	.000
.003	16239.081
.006	32478.163
.009	41003.676
.011	41815.630
.014	42627.584
.017	43439.538
.020	44251.492
.023	45063.447
.026	45875.401
.029	46687.355
.031	47499.309

.034	48311.263
.037	49123.217
.040	49935.171
.043	50747.125
.046	.000

DEPTH BELOW GS IN	DIAM IN	C KIP/IN**2	
-24.00	17.875	.3E+04	
		Y IN	P KIP/IN
		.000	.000
		.003	16239.081
		.006	32478.163
		.009	41003.676
		.011	41815.630
		.014	42627.584
		.017	43439.538
		.020	44251.492
		.023	45063.447
		.026	45875.401
		.029	46687.355
		.031	47499.309
		.034	48311.263
		.037	49123.217
		.040	49935.171
		.043	50747.125
		.046	.000

DEPTH BELOW GS IN	DIAM IN	C KIP/IN**2	
30.00	17.875	.3E+04	
		Y IN	P KIP/IN
		.000	.000
		.003	16239.081
		.006	32478.163
		.009	41003.676
		.011	41815.630
		.014	42627.584
		.017	43439.538
		.020	44251.492
		.023	45063.447
		.026	45875.401
		.029	46687.355
		.031	47499.309
		.034	48311.263
		.037	49123.217
		.040	49935.171
		.043	50747.125
		.046	.000

DEPTH BELOW GS IN	DIAM IN	C KIP/IN**2	
120.00	17.875	.3E+04	
		Y IN	P KIP/IN
		.000	.000
		.003	16239.081
		.006	32478.163
		.009	41003.676
		.011	41815.630

.017	43439.538
.020	44251.492
.023	45063.447
.026	45875.401
.029	46687.355
.031	47499.309
.034	48311.263
.037	49123.217
.040	49935.171
.043	50747.125
.046	.000

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An applied moment of (33.9 k)(12') = 508.5 k' should be expected.

PILE LOADING CONDITION

LATERAL LOAD AT PILE HEAD = .339E+02 KIP (see pg 115)
APPLIED MOMENT AT PILE HEAD = .000E+00 IN-KIP
AXIAL LOAD AT PILE HEAD = .845E+02 KIP (see pg 70)

X	MAX DEFLECTION FROM APPLIED LOAD	APPLIED MOMENT	TOTAL STRESS	SHEAR	SOIL RESIST	FLEXURAL RIGIDITY
IN	IN	IN-KIP	LBS/IN**2	KIP	LBS/IN	KIP-IN**2
*****	**********	**********	**********	**********	**********	**********
.00	.308E+01	.000E+00	.410E+04	.361E+02	.000E+00	.229E+08
10.00	.282E+01	.361E+03	.818E+04	.339E+02	.000E+00	.229E+08
20.00	.257E+01	.721E+03	.123E+05	.339E+02	.000E+00	.229E+08
30.00	.232E+01	.108E+04	.163E+05	.339E+02	.000E+00	.229E+08
40.00	.207E+01	.144E+04	.204E+05	.339E+02	.000E+00	.229E+08
50.00	.183E+01	.180E+04	.245E+05	.339E+02	.000E+00	.229E+08
60.00	.160E+01	.216E+04	.285E+05	.339E+02	.000E+00	.229E+08
70.00	.138E+01	.252E+04	.326E+05	.339E+02	.000E+00	.229E+08
80.00	.117E+01	.287E+04	.366E+05	.339E+02	.000E+00	.229E+08
90.00	.968E+00	.323E+04	.407E+05	.339E+02	.000E+00	.229E+08
100.00	.784E+00	.358E+04	.447E+05	.339E+02	.000E+00	.229E+08
110.00	.615E+00	.394E+04	.487E+05	.339E+02	.000E+00	.229E+08
120.00	.463E+00	.429E+04	.526E+05	.339E+02	.000E+00	.229E+08
130.00	.330E+00	.464E+04	.566E+05	.339E+02	.000E+00	.229E+08
140.00	.218E+00	.499E+04	.606E+05	.339E+02	.000E+00	.229E+08
150.00	.127E+00	.533E+04	.645E+05	.339E+02	.000E+00	.229E+08
160.00	.595E-01	.568E+04	.684E+05	.339E+02	.000E+00	.229E+08
170.00	.168E-01	.602E+04	.723E+05	-.744E+02	.433E+05	.229E+08
180.00	-.807E-03	.346E+04	.432E+05	-.319E+03	-.457E+04	.229E+08
190.00	-.319E-02	.812E+03	.133E+05	-.176E+03	-.181E+05	.229E+08
200.00	-.153E-02	-.146E+03	.575E+04	-.392E+02	-.871E+04	.229E+08
210.00	-.275E-03	-.210E+03	.648E+04	.851E+01	-.156E+04	.229E+08
220.00	.109E-03	-.857E+02	.507E+04	.109E+02	.621E+03	.229E+08
230.00	.102E-03	-.101E+02	.421E+04	.430E+01	.582E+03	.229E+08
240.00	.355E-04	.885E+01	.420E+04	.445E+00	.202E+03	.229E+08
250.00	.163E-05	.629E+01	.417E+04	-.476E+00	.930E+01	.229E+08
260.00	-.500E-05	.182E+01	.412E+04	-.322E+00	-.284E+02	.229E+08
270.00	-.290E-05	-.812E-01	.410E+04	-.888E-01	-.165E+02	.229E+08
280.00	-.703E-06	-.326E+00	.410E+04	.858E-02	-.399E+01	.229E+08
290.00	.179E-06	-.122E+00	.410E+04	.198E-01	.102E+01	.229E+08
300.00	.500E-06	.000E+00	.410E+04	.000E+00	.284E+01	.229E+08

501.7 k'

Verification of Com624 Pushover Analysis and Results / Final Thoughts

$L := 15ft$ $\quad$ $V_{pile} := 33.9kip$

$I := 789in^4$ $\quad$ Elastic Property

$E := 29000ksi$ $\quad$ Steel Modulus

$$\Delta_V := \frac{V_{pile} \cdot L^3}{3E \cdot I} \qquad \Delta_V = 2.88\,in$$

Com624 shows 3.08" of deflection for the same load. Com624 run verified.

The maximim stress in the steel tube is 68.7ksi, above the elastic range of 42ksi. The maximum equivalent stress for the concrete filled tube is 75.2ksi (page 89). So ok for stress.

The steel tube elastic moment capacity is 309 k*ft (page 52).
The steel tube plastic moment capacity is 402 k*ft (page 51).
The moment capacity of steel tube and concrete is 469.0 k*ft (page 89).

The shear due to earthquake loads is 33.9 kips (page 115). Multiply by the pile height (15 feet), and the total applied moment is **508.5 k*ft** (page 80) - not good!

However to get the 33.9 kips, **Seisab assumes** the **top** of the column **fixed**. Another Seisab run with pinned top columns has a lower shear of 24.4 kips (which makes sense since a pinned top is less stiff and would attract less load). This value times 15 is 366 kip*ft, which passes the column moment capacity check.

This realization does not affect the pushover analysis results, because the shear value to find stiffnesses is determined independantly and iteratively.

Under LRFD old provisions, the earthquake shear load is 15.8 (page 133) which would have caused an applied moment of 237 k*ft - within even elastic moment capacity.

While investigating the cause of the 508.5 k' moment, it was discovered that Seisab required two (2) more height parameters to correctly model the bridge. The additional parameters, C.G. of Cap, and Bearing Elevation, alter the max moment and other results by less than 5%. Therefore the analysis will continue with the current runs.

The steel shell piles are ok.

OUTPUT RESULTS FOR AN AXIAL LOAD = 50.55 KIP
****

MOMENT IN-KIP	EI KIP-IN**2	PHI 1/IN	MAX STR IN/IN	N AXIS IN
23.081	.23081E+08	.000001	.00009	92.640
115.407	.23081E+08	.000005	.00013	25.680
207.732	.23081E+08	.000009	.00016	18.240
298.283	.22945E+08	.000013	.00019	15.354
375.499	.22088E+08	.000017	.00023	13.708
459.209	.21867E+08	.000021	.00026	12.814
577.107	.23084E+08	.000025	.00030	12.218
659.020	.22725E+08	.000029	.00033	11.761
709.497	.21500E+08	.000033	.00036	11.324
854.069	.23083E+08	.000037	.00039	11.026
935.485	.22817E+08	.000041	.00043	10.933
1038.730	.23083E+08	.000045	.00047	10.728
1042.656	.21279E+08	.000049	.00049	10.430
1127.771	.21279E+08	.000053	.00053	10.430
1915.784	.23082E+08	.000083	.00078	9.834
2608.240	.23082E+08	.000113	.00103	9.536
3279.087	.22931E+08	.000143	.00130	9.498
3868.949	.22364E+08	.000173	.00157	9.464
4230.617	.20840E+08	.000203	.00186	9.536
4400.061	.18884E+08	.000233	.00213	9.536
4507.030	.17137E+08	.000263	.00241	9.536
4578.784	.15627E+08	.000293	.00268	9.536
4629.623	.14333E+08	.000323	.00296	9.536
4667.896	.13224E+08	.000353	.00323	9.536
4697.124	.12264E+08	.000383	.00351	9.536
4719.731	.11428E+08	.000413	.00378	9.536
4737.704	.10695E+08	.000443	.00406	9.536
4752.641	.10048E+08	.000473	.00433	9.536
4764.817	.94728E+07	.000503	.00461	9.536
4774.590	.89580E+07	.000533	.00488	9.536
4783.399	.84963E+07	.000563	.00515	9.536
4790.323	.80781E+07	.000593	.00543	9.536
4796.469	.76990E+07	.000623	.00570	9.536
4802.273	.73542E+07	.000653	.00598	9.536
4806.366	.70371E+07	.000683	.00625	9.536
4810.460	.67468E+07	.000713	.00653	9.536
4814.553	.64799E+07	.000743	.00680	9.536
4817.358	.62320E+07	.000773	.00708	9.536
4819.923	.60024E+07	.000803	.00735	9.536
4822.489	.57893E+07	.000833	.00763	9.536
4825.054	.55910E+07	.000863	.00790	9.536
4827.281	.54057E+07	.000893	.00818	9.536
4828.760	.52316E+07	.000923	.00845	9.536
4830.240	.50685E+07	.000953	.00873	9.536
4831.720	.49153E+07	.000983	.00900	9.536
4833.200	.47712E+07	.001013	.00928	9.536
4834.679	.46354E+07	.001043	.00955	9.536
4836.159	.45071E+07	.001073	.00982	9.536

OUTPUT RESULTS FOR AN AXIAL LOAD = 35.23 KIP /27.57
****

MOMENT IN-KIP	EI KIP-IN**2	PHI 1/IN	MAX STR IN/IN	N AXIS IN
23.081	.23081E+08	.000001	.00007	67.274
115.407	.23081E+08	.000005	.00010	20.607
206.932	.22992E+08	.000009	.00014	15.405
286.195	.22015E+08	.000013	.00017	13.410
385.186	.22658E+08	.000017	.00020	12.295
453.030	.21573E+08	.000021	.00024	11.622
570.337	.22813E+08	.000025	.00027	11.225
669.404	.23083E+08	.000029	.00030	10.728
756.270	.22917E+08	.000033	.00034	10.678
787.312	.21279E+08	.000037	.00037	10.430
929.909	.22681E+08	.000041	.00041	10.290
954.265	.21206E+08	.000045	.00044	10.132
1039.088	.21206E+08	.000049	.00048	10.132
1195.787	.22562E+08	.000053	.00051	9.947
1915.787	.23082E+08	.000083	.00076	9.536
2371.260	.20985E+08	.000113	.00100	9.238
3000.798	.20985E+08	.000143	.00127	9.238
3578.051	.20682E+08	.000173	.00153	9.238
3887.911	.19152E+08	.000203	.00180	9.238
4054.579	.17402E+08	.000233	.00206	9.238
4160.349	.15819E+08	.000263	.00233	9.238
4231.445	.14442E+08	.000293	.00260	9.238
4281.868	.13257E+08	.000323	.00286	9.238
4319.847	.12238E+08	.000353	.00313	9.238
4348.860	.11355E+08	.000383	.00339	9.238
4371.305	.10584E+08	.000413	.00366	9.238
4389.151	.99078E+07	.000443	.00392	9.238
4403.981	.93107E+07	.000473	.00419	9.238
4416.072	.87795E+07	.000503	.00446	9.238
4425.773	.83035E+07	.000533	.00472	9.238
4434.517	.78766E+07	.000563	.00499	9.238
4441.388	.74897E+07	.000593	.00525	9.238
4447.486	.71388E+07	.000623	.00552	9.238
4453.243	.68197E+07	.000653	.00578	9.238
4457.301	.65261E+07	.000683	.00605	9.238
4461.358	.62572E+07	.000713	.00632	9.238
4465.416	.60100E+07	.000743	.00658	9.238
4468.191	.57803E+07	.000773	.00685	9.238
4470.730	.55675E+07	.000803	.00711	9.238
4473.268	.53701E+07	.000833	.00738	9.238
4475.807	.51863E+07	.000863	.00764	9.238
4478.008	.50146E+07	.000893	.00791	9.238
4479.466	.48532E+07	.000923	.00818	9.238
4480.925	.47019E+07	.000953	.00844	9.238
4482.384	.45599E+07	.000983	.00871	9.238
4483.843	.44263E+07	.001013	.00897	9.238
4485.301	.43004E+07	.001043	.00924	9.238
4486.760	.41815E+07	.001073	.00950	9.238
4487.667	.40686E+07	.001103	.00977	9.238

OUTPUT RESULTS FOR AN AXIAL LOAD = 19.91 KIP
****

MOMENT IN-KIP	EI KIP-IN**2	PHI 1/IN	MAX STR IN/IN	N AXIS IN
23.081	.23081E+08	.000001	.00004	41.907
115.452	.23090E+08	.000005	.00008	15.496
196.146	.21794E+08	.000009	.00011	12.516
279.499	.21500E+08	.000013	.00014	11.324
392.409	.23083E+08	.000017	.00018	10.728
446.853	.21279E+08	.000021	.00021	10.430
530.147	.21206E+08	.000025	.00024	10.132
658.410	.22704E+08	.000029	.00028	10.008
761.697	.23082E+08	.000033	.00031	9.834
852.753	.23047E+08	.000037	.00035	9.826
923.960	.22536E+08	.000041	.00038	9.645
1038.679	.23082E+08	.000045	.00041	9.536
1131.007	.23082E+08	.000049	.00045	9.536
1223.334	.23082E+08	.000053	.00049	9.536
1741.722	.20985E+08	.000083	.00074	9.238
2595.435	.22968E+08	.000113	.00100	9.212
3226.650	.22564E+08	.000143	.00124	9.077
3853.890	.22277E+08	.000173	.00150	9.056
4171.562	.20550E+08	.000203	.00177	9.091
4345.982	.18652E+08	.000233	.00203	9.114
4459.167	.16955E+08	.000263	.00230	9.133
4536.082	.15482E+08	.000293	.00257	9.147
4589.586	.14209E+08	.000323	.00283	9.153
4589.586	.13002E+08	.000353	.00313	9.238
4589.586	.11983E+08	.000383	.00339	9.238
4589.586	.11113E+08	.000413	.00366	9.238
4589.586	.10360E+08	.000443	.00392	9.238
4589.586	.97031E+07	.000473	.00419	9.238
4589.586	.91244E+07	.000503	.00446	9.238
4589.586	.86109E+07	.000533	.00472	9.238
4589.586	.81520E+07	.000563	.00499	9.238
4589.586	.77396E+07	.000593	.00525	9.238
4589.586	.73669E+07	.000623	.00552	9.238
4589.586	.70285E+07	.000653	.00578	9.238
4589.586	.67197E+07	.000683	.00605	9.238
4589.586	.64370E+07	.000713	.00632	9.238
4589.586	.61771E+07	.000743	.00658	9.238
4589.586	.59374E+07	.000773	.00685	9.238
4589.586	.57155E+07	.000803	.00711	9.238
4589.586	.55097E+07	.000833	.00738	9.238
4589.586	.53182E+07	.000863	.00764	9.238
4589.586	.51395E+07	.000893	.00791	9.238
4589.586	.49725E+07	.000923	.00818	9.238
4589.586	.48159E+07	.000953	.00844	9.238
4589.586	.46690E+07	.000983	.00871	9.238
4589.586	.45307E+07	.001013	.00897	9.238
4589.586	.44004E+07	.001043	.00924	9.238
4589.586	.42773E+07	.001073	.00950	9.238
4589.586	.41610E+07	.001103	.00977	9.238

OUTPUT RESULTS FOR AN AXIAL LOAD = 12.25 KIP

****

MOMENT IN-KIP	EI KIP-IN**2	PHI 1/IN	MAX STR IN/IN	N AXIS IN
23.081	.23081E+08	.000001	.00003	29.223
109.335	.21867E+08	.000005	.00006	12.814
207.746	.23083E+08	.000009	.00010	11.026
276.623	.21279E+08	.000013	.00013	10.430
360.500	.21206E+08	.000017	.00017	10.132
484.716	.23082E+08	.000021	.00020	9.834
564.079	.22563E+08	.000025	.00023	9.657
669.371	.23082E+08	.000029	.00027	9.536
761.698	.23082E+08	.000033	.00030	9.536
839.394	.22686E+08	.000037	.00033	9.416
860.369	.20985E+08	.000041	.00036	9.238
944.307	.20985E+08	.000045	.00040	9.238
1028.246	.20985E+08	.000049	.00043	9.238
1112.184	.20985E+08	.000053	.00047	9.238
1878.270	.22630E+08	.000083	.00072	9.102
2363.046	.20912E+08	.000113	.00097	8.940
2990.403	.20912E+08	.000143	.00122	8.940
3589.197	.20747E+08	.000173	.00148	8.940
3892.981	.19177E+08	.000203	.00174	8.940
4058.762	.17420E+08	.000233	.00199	8.940
4164.143	.15833E+08	.000263	.00225	8.940
4235.028	.14454E+08	.000293	.00251	8.940
4285.318	.13267E+08	.000323	.00276	8.940
4323.205	.12247E+08	.000353	.00302	8.940
4352.151	.11363E+08	.000383	.00328	8.940
4374.547	.10592E+08	.000413	.00354	8.940
4392.355	.99150E+07	.000443	.00379	8.940
4407.154	.93175E+07	.000473	.00405	8.940
4419.221	.87857E+07	.000503	.00431	8.940
4428.902	.83094E+07	.000533	.00456	8.940
4437.630	.78821E+07	.000563	.00482	8.940
4444.487	.74949E+07	.000593	.00508	8.940
4450.574	.71438E+07	.000623	.00533	8.940
4456.319	.68244E+07	.000653	.00559	8.940
4460.370	.65306E+07	.000683	.00585	8.940
4464.420	.62615E+07	.000713	.00610	8.940
4468.470	.60141E+07	.000743	.00636	8.940
4471.240	.57843E+07	.000773	.00662	8.940
4473.774	.55713E+07	.000803	.00687	8.940
4476.308	.53737E+07	.000833	.00713	8.940
4478.842	.51899E+07	.000863	.00739	8.940
4481.038	.50180E+07	.000893	.00764	8.940
4482.494	.48564E+07	.000923	.00790	8.940
4483.951	.47051E+07	.000953	.00816	8.940
4485.407	.45630E+07	.000983	.00841	8.940
4486.863	.44293E+07	.001013	.00867	8.940
4488.319	.43033E+07	.001043	.00893	8.940
4489.775	.41843E+07	.001073	.00919	8.940
4490.680	.40713E+07	.001103	.00944	8.940
4491.419	.39642E+07	.001133	.00970	8.940

P-Δ Capacity Requirement - Step 10

P-Δ Capacity Requirement for SDC C & D (Section 4.3)

A column bent by an earthquake has reduced strength. Gravity loads on such a column should be considered. The magnitude of displacement can only be accurately determined by a non-linear time history analysis. Instead of this, the following equation can be used:

$P_{dl} \cdot \Delta_r < 0.25M_p$ for concrete members

$kip := 1000lb$

$ksi := \frac{kip}{in^2}$

$M_p := 401.95kip \cdot ft$ plastic moment

$\Delta_r = \Delta_D - \Delta_S$

$\Delta_D := 2.014in$ The deflection from seisab magnified by R_d.

$\Delta_S := 0in$ Pile shaft displacement from maximum moment. I do not know what this value is, but using zero will be conservative.

$\Delta_r := \Delta_D - \Delta_S$ $\Delta_r = 2.014\,in$

This check probably rarely controls. Even if the deflection is made equal to the unelastic deflection, the P-Δ capacity is ok.

$P_c := 38.63kip$

$P_w := 12.5kip$

$P_{dl} := 0.9P_c + 0.65P_w$ Dead load combination for Extreme Event I.

$P_{dl} \cdot \Delta_r = 7.199\,kip \cdot ft$ < $0.25M_p = 100.487\,kip \cdot ft$

$$x := \left| \begin{array}{l} x \leftarrow \text{"OK"} \quad \text{if } P_{dl} \cdot \Delta_r < 0.25 \cdot M_p \\ x \leftarrow \text{"NO GOOD"} \quad \text{otherwise} \\ \text{return } x \end{array} \right.$$

Is P-Δ capacity acceptable?

$x = \text{"OK"}$

Factored Reactions - Strength I - k

Includes ductility, redundancy, and operational factors.

Location	DC1	DC2+DW	Live+Imp Down	Live+Imp Up	Max Total	Min Total
0.00	15.72	4.5	111.73	-13.59	131.98	6.66
Steel	1.31					
Conc	14.42					
28.00	48.29	17.4	172.36	-15.37	238.12	50.39
Steel	1.34					
Conc	46.94					
62.00	48.29	17.4	172.36	-15.37	238.12	50.39
Steel	1.34					
Conc	46.94					
90.00	15.72	4.5	111.73	-13.59	131.98	6.65
Steel	1.31					
Conc	14.42					

Unfactored Reactions - k

Includes ductility, redundancy, and operational factors.

Location	DC1	DC2+DW	Live+Imp Down	Live+Imp Up	Max Total	Min Total
0.00	12.58	3.1	63.85	-7.76	79.60	7.99
Steel	1.04					
Conc	11.53					
28.00	38.63	12.5	98.49	-8.78	149.65	42.38
Steel	1.07					
Conc	37.55	51.13 kip				
62.00	38.63	12.5	98.49	-8.79	149.65	42.38
Steel	1.07					
Conc	37.55					
90.00	12.58	3.1	63.85	-7.77	79.60	7.99
Steel	1.04					
Conc	11.53					

Member/Component Performance Requirement - Step 11

This step consists of following many "substeps" on two flowcharts, Figure 1.3F and Figure 1.3G. This bridge is a Type1** bridge, consisting of a concrete filled steel pipe substructure.

$kip := 1000lb$

Combined Axial Compression and Flexure (Section 7.6.1)

$ksi := \frac{kip}{in^2}$

$$\frac{P_u}{P_r} + \frac{B \cdot M_u}{M_{rc}} < 1.0 \quad \text{and} \quad \frac{M_u}{M_{rc}} < 1.0$$

$K := 2$ — Effective length factor in LRFD 4.6.2.5. K=1 transversly and K=2 longitudinally. Column will flex about weak axis, so use K=2.

$l := 15ft$ — Length from top of column to fixity.

$r_s = \sqrt{\frac{I}{A}}$ $\quad r_s := 6.189in$ — Radius of Gyration, see Step 5: 18" Pipe Plastic Section Properties.

$C_1 := 1$ — Constant from LRFD Table 6.9.5.1.1

$C_2 := 0.85$

$F_y := 42ksi$ — Yield strength of steel shell.

$F_{yr} := 60ksi$ — Yield strength of reinforcement steel.

$A_r := 0in^2$ — Area of longitudinal reinforcement

$A_s := 20.616in^2$ — Area of steel shell

$f_c := 3.5ksi$ — 28 day concrete strength

$A_c := (17.124in)^2 \cdot \frac{\pi}{4}$ $\quad A_c = 230.303\,in^2$ — Inside concrete area

$$F_e := F_y + C_1 \cdot F_{yr} \cdot \left(\frac{A_r}{A_s}\right) + C_2 \cdot f_c \cdot \left(\frac{A_c}{A_s}\right) \qquad F_e = 75.234\,ksi$$

$C_3 := 0.4$ Constant from LRFD Table 6.9.5.1.1

$E := 29000\text{ksi}$ $E_c := 33\cdot(145)^{1.5}\cdot 3500^{0.5}\dfrac{\text{lb}}{\text{in}^2}$ $E_c = 3.409\times 10^6\dfrac{\text{lb}}{\text{in}^2}$

$n := \dfrac{E}{E_c}$ $n = 8.507$

$E_e := E\cdot\left[1+\left(\dfrac{C_3}{n}\right)\cdot\left(\dfrac{A_c}{A_s}\right)\right]$ $E_e = 4.423\times 10^4\,\text{ksi}$

$\lambda := \left(\dfrac{K\cdot l}{r_s\cdot\pi}\right)^2\cdot\dfrac{F_e}{E_e}$ $\lambda = 0.583$

$$P_n := \begin{array}{|l} p \leftarrow 0.66^{\lambda}\cdot F_e\cdot A_s \ \text{ if } \lambda < 2.25 \\ p \leftarrow \dfrac{0.88\cdot F_e\cdot A_s}{\lambda} \ \text{ otherwise} \\ \text{return } p \end{array}$$

Compressive resistance of a composite column.

$P_n = 1.217\times 10^3\,\text{kip}$

$\phi_c := 0.8$ $P_r := \phi_c\cdot P_n$ $P_r = 973.834\,\text{kip}$

Below is the factored moment resistance calculations for a concrete filled steel pipe. Using Method 2 - Approximate Geometry (Recommended LRFD Guidelines - 7.6.2).

$D := 18\text{in} - 2\cdot\dfrac{1}{16}\text{in}$ $t := 0.4375\text{in} - \dfrac{1}{16}\text{in}$ $t = 0.375\,\text{in}$

$h_n := \dfrac{A_c\cdot f_c}{2D\cdot f_c + 4t\cdot(2F_y - f_c)}$ $h_n = 3.278\,\text{in}$

$\phi_f := 1.0$ For flexure $Z := 114.84\text{in}^3$ Plastic Modulus

$$M_{rc} := \phi_f\cdot\left[\left(Z - 2\cdot t\cdot h_n^{\,2}\right)\cdot F_y + \left[\frac{2}{3}\cdot(0.5\cdot D - t)^3 - (0.5\cdot D - t)\cdot h_n^{\,2}\right]\cdot f_c\right]$$

$M_{rc} = 468.954\,\text{kip}\cdot\text{ft}$

Calculate Pn again based on λ=0.

$\lambda := 0$

$$P_n := \left| \begin{array}{l} p \leftarrow 0.66^{\lambda} \cdot F_e \cdot A_s \ \text{ if } \lambda < 2.25 \\ p \leftarrow \dfrac{0.88 \cdot F_e \cdot A_s}{\lambda} \ \text{ otherwise} \\ \text{return } p \end{array} \right.$$

Compressive resistance of a composite column.

$P_n = 1.551 \times 10^3 \text{ kip}$

$\phi_c := 0.8$ $\qquad P_{ro} := \phi_c \cdot P_n$ $\qquad P_{ro} = 1.241 \times 10^3 \text{ kip}$

$P_{rc} := \phi_c \cdot A_c \cdot f_c$ $\qquad P_{rc} = 644.849 \text{ kip}$

$B := \dfrac{P_{ro} - P_{rc}}{P_{rc}}$ $\qquad B = 0.924$ $\qquad \dfrac{P_u}{P_r} + \dfrac{BM_u}{M_{rc}} < 1.0$

$P_u := 238.12\text{kip}$

Factored loads from MDX

$M_u := \left(367^2 + 109^2\right)^{\frac{1}{2}} \text{kip} \cdot \text{ft}$

Moments due to EarthQuake from Seisab run. This is transverse plus longitudinal moments.

$\dfrac{P_u}{P_r} + \dfrac{B \cdot M_u}{M_{rc}} = 0.999$

Combined load equation.

$$\text{Result} := \left| \begin{array}{l} r \leftarrow 1 \\ r \leftarrow 0 \ \text{ if } \dfrac{P_u}{P_r} + \dfrac{B \cdot M_u}{M_{rc}} > 1 \\ r \leftarrow 0 \ \text{ if } \dfrac{M_u}{M_{rc}} > 1 \\ r \leftarrow \text{"FAIL"} \ \text{ if } r < 1 \\ r \leftarrow \text{"PASS"} \ \text{ otherwise} \\ \text{return } r \end{array} \right.$$

Combined Axial Compression and Flexure.

Result = "PASS"

7.6.3 Beams and Connections

Capacity protected members must be designed to resist the forces from hinging in the concrete filled pipes.

4.12 Satisfy Support Requirements Seat Width

$$N = \left(4 + \Delta_{ot} + 1.65\Delta_{eq}\right)\cdot\left(1 + \frac{S_k^{\ 2}}{4000}\right) > 12in$$

$\Delta_{ot} := 1$ — Movement from thermal expansion/contraction, shrinkage, or creep. 1 inch minimum for a 100' bridge. Our bridge is 90' long with 0.454" of deflection. Should probably use the 1" minimum.

$\Delta_{eq} := 0.093 \times 12$ — Maximum longitudinal displacement (in).

$S_k := 0$ — Skew angle in degrees measured from a line normal to the span. Ignoring the skew term since it dramatically lowers the seat width.

$N := \left(4 + \Delta_{ot} + 1.65\Delta_{eq}\right)in$ $\qquad N = 6.841\,in$

$$Seat_{width} := \left|\begin{array}{l} p \leftarrow N \\ p \leftarrow 12in \ \text{ if } N < 12in \\ \text{return } p \end{array}\right.$$

Compressive resistance of a composite column.

$Seat_{width} = 12\,in$

4.14 Superstructure Shear Keys

Shear keys are typically designed to fuse at the "Life Safety Design Event" and stay elastic at a lower more frequent earthquake event.

$V_{ok} = 2V_{nk}$ — The overstrength shear key capacity should be used in assessing adjacent members.

Transverse Shear Design $\quad kip := 1000lb \quad R_{dabut} := 0.8$

$EQ := 302.1kip$ Max transverse seismic force at Int. Bents from Seisab (no abutment soil condition)

$V_{nk} := \dfrac{EQ}{R_{dabut}}$ $\quad V_{nk} = 377.625\,kip$ See above, also from LRFD 3.10.7

$V_{ok} := 2 \cdot V_{nk}$ Is the overstrength shear key capacity used to design shear blocks, or only shear keys and adjacent members? Not using overstrength for this design.

It is office policy that shear blocks must be used in a Seismic 3 or 4 region. If four shear blocks are assumed (75' wide bridge), the total required shear capacity can be divided by 4.

$V_u := \dfrac{V_{nk}}{4}$ $\quad V_u = 94.406\,kip$

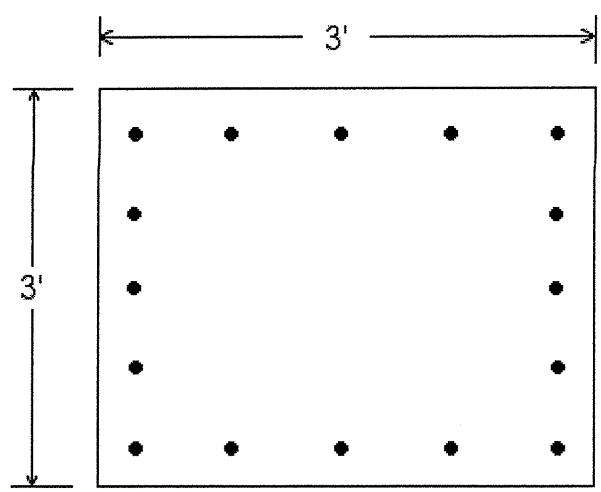
Four 3'-0 wide by 3'-0" long shear blocks are used, USE SHEAR FRICTION design (LRFD 5.8.4).

Use **#4 rebars** for the shear blocks, 16 total.

$f_y := 60\dfrac{kip}{in^2}$ $\quad A_{vf} := 16 \cdot \pi \left(\dfrac{4in}{16}\right)^2$

$\mu := 1$ $\quad A_{vf} = 3.142\,in^2$

$V_n := A_{vf} \cdot f_y \cdot \mu$ $\quad V_n = 188.496\,kip$



Plan View

But must be less than:

$f_c := 3.5\dfrac{kip}{in^2}$ $\quad A_{cv} := (3ft)^2$

$Check1 := 0.2f_c \cdot A_{cv}$ $\quad Check1 = 907.2\,kip$

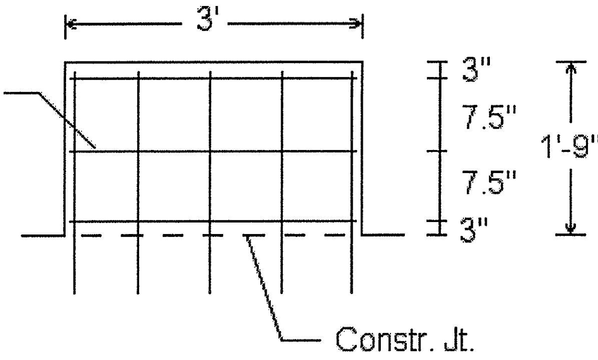
$Check2 := 0.8 \cdot \dfrac{kip}{in^2} \cdot A_{cv}$ $\quad Check2 = 1.037 \times 10^3\,kip$

$$SHEAR\_BLOCK := \begin{array}{|l} s \leftarrow \text{"PASS"} \quad \text{if } V_n < Check1 < Check2 \\ s \leftarrow \text{"PASS"} \quad \text{if } V_n < Check2 < Check1 \\ s \leftarrow \text{"FAIL"} \quad \text{otherwise} \\ \text{return } s \end{array}$$

SHEAR_BLOCK = "PASS"

$V_\phi := 0.85 \cdot V_n$

$V_\phi = 160.221\,kip$



$$SHEAR\_BLOCK := \begin{array}{|l} s \leftarrow \text{"PASS"} \quad \text{if } V_u < V_\phi \\ s \leftarrow \text{"FAIL"} \quad \text{otherwise} \\ \text{return } s \end{array}$$

SHEAR_BLOCK = "PASS"

Use 4 shear blocks

6.2 Foundation Investigation

Two borings of the site, near the interior bents were drilled. Piles were chosen to support the bridge based on economics. Groundwater level, and blow counts are detailed on the boring logs. Liquefaction has been considered and found to be inconsequential.

6.4 Pile Cap Foundation Design

This design does NOT use a pile cap foundation, just piles.

6.5 Drilled Shafts

Drilled shafts shall be designed like columns in SDC B, C, or D. If only one drilled shaft is used per bent, its lateral force must be multiplied by 1.5 - this guarantees stability.

6.7 Abutment Design

For monolithic abutments where the abutment forms an integral part of the bridge superstructure, the abutment shall be designed using one of two ways depending on the contribution level accounted for in the analytical model:

1. At a minimum the abutment shall be designed to resist the lesser of the **active pressure** applied by the abutment backfill, or the maximum estimated longitudinal earthquake force transferred to the abutment.

A. Checked against a recently modified AHTD backwall sheet to protect against failure from backfill pressure. The backwalls are good.

2. If the abutment is part of the ERS and required to mobilize the full **passive pressure**, a reduction factor greater than or equal to **0.5** shall be applied to the design forces (provided a brittle failure does not exist in the load path transmitted to the superstructure.

Vertical acceleration may be omitted.

For SDC D (5.2.4.2) transverse loading, the stiffness contribution of piles 16" or less shall be ignored if the abutment displacement is greater than 4 inches.

For SDC D bridges, the abutment skew should be minimized. **Approach slabs providing structural support between approach fills and and abutments shall be provided.** Slabs shall be adequately linked to abutments using flexible ties. Liquefaction may govern.

Excepting approach slabs, abutments are good.

Design is COMPLETE according to the new guideline sheets. Going ahead and checking bearing stress as done in the old design.

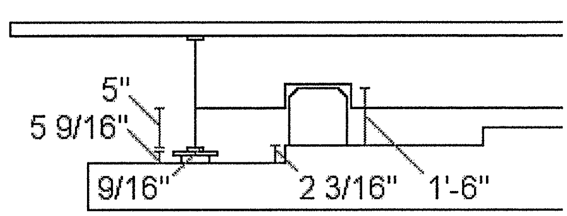
Bearing Stress (5.7.5)

The factored bearing stress shall be taken as: $P_r = \phi \cdot P_n$ $\quad \phi := 0.85$

This bearing check is on the transverse load coming in to bear on the shear blocks. The force is transmitted by a concrete longitudinal restrainer at the interior bents which is one (1) foot wide. The height of the bearing area is calculated as shown.

$$h_b := \left(18 + 2 + \frac{3}{16} - \frac{9}{16} - 5 - \frac{9}{16} - 5\right) in$$

$$h_b = 9.063\, in$$



Subtract the chamfered corner of the shear block from the bearing area.

$h_b := h_b - \frac{3}{4}in$ $\quad h_b = 8.313\,in$ $\quad m := 1$ Modification factor

$A_1 := h_b \cdot 12in$ $\quad A_1 = 99.75\,in^2$ The bearing area of the concrete transverse restrainer on a shear block.

$P_n := 0.85 \cdot f_c \cdot A_1 \cdot m$ $\quad P_r := \phi \cdot P_n$ $\quad P_r = 252.243\,kip$

$$BEARING := \left| \begin{array}{l} b \leftarrow \text{"PASS"} \quad \text{if } V_u < P_r \\ b \leftarrow \text{"FAIL"} \quad \text{otherwise} \\ \text{return } b \end{array} \right.$$

$BEARING = \text{"PASS"}$

Shear Ties for the **Shear Block** (LRFD 5.8.3.3)

$V_n := \frac{V_u}{0.85}$ $\quad V_n = 111.066\,kip$ $\quad b_v := 36in - 2 \cdot 3in$ $\quad d_v := 21in - 2 \cdot 3in$

$V_c := 2 \cdot 3500^{0.5} \cdot \left(\frac{lb}{in^2}\right) \cdot b_v \cdot d_v$ $\quad V_c = 53.245\,kip$

$V_s := V_n - V_c$ $\quad V_s = 57.821\,kip$

The concrete needs reinforcement to handle the shear stress.

Five (5) rows of bars with five (5) bars in each row. So 25 bars total.

$A_4 := \pi \cdot \left(\frac{4in}{16}\right)^2$ $\quad A_4 = 0.196\,in^2$ $\quad A_r := 25 \cdot A_4$ $\quad A_r = 4.909\,in^2$

$V_s := 60ksi \cdot A_r$

$$Shear_{steel} := \left| \begin{array}{l} v \leftarrow \text{"PASS"} \quad \text{if } V_s > V_n - V_c \\ v \leftarrow \text{"FAIL"} \quad \text{otherwise} \\ \text{return } v \end{array} \right.$$

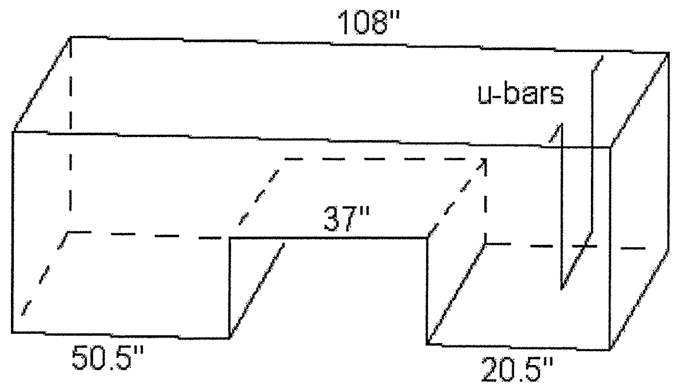
Provide #4 bars at 7.5" spacing as shown previously.

$Shear_{steel} = \text{"PASS"}$

Concrete Diaphragm Design

There is **shear friction** between the slab and diaphragm (LRFD 5.8.4).

Try 9 #5 U-Bars at 12"



$n := 9 \qquad A_4 := \pi\cdot\left(\frac{5in}{16}\right)^2 \qquad m := 1$

$V_n := n\cdot A_4\cdot 60\frac{kip}{in^2}\cdot m \qquad V_n = 165.67\,kip$

$b_v := 12in \qquad d_v := 108in \qquad A_{cv} := b_v\cdot d_v$

$Check1 := 0.2f_c\cdot A_{cv} \qquad Check1 = 907.2\,kip$

$Check2 := 0.8\cdot\frac{kip}{in^2}\cdot A_{cv} \qquad Check2 = 1.037\times 10^3\,kip$

$$V_N := \left|\begin{array}{l} min \leftarrow Check1 \;\; \text{if } Check1 < Check2 \\ min \leftarrow Check2 \;\; \text{otherwise} \\ v \leftarrow V_n \;\; \text{if } V_n < min \\ v \leftarrow min \;\; \text{if } V_n > min \\ \text{return } v \end{array}\right.$$

$V_N = 165.67\,kip$

$$CONCDIA_V := \left|\begin{array}{l} v \leftarrow \text{"PASS"} \;\; \text{if } V_N > V_u \\ v \leftarrow \text{"FAIL"} \;\; \text{otherwise} \\ \text{return } v \end{array}\right.$$

$CONCDIA_V = \text{"PASS"}$

Longitudinal Steel

Use #6 bars longitudinal, see Job 110390, bridge 06983

Longitudinal Restrainers Design

$EQ := 281.2kip$ Maximum Longitudinal Force in Seisab at Interior Bents.

$beams := 9$ There are nine beams that will take longitudinal force.

$V_u := \frac{EQ}{R_{dabut} \cdot beams}$ $V_u = 39.056\,kip$ Each restrainer will carry 39 kip.

Use 1/16" weld: $throat := \left(\frac{1in}{4}\right) \cdot \sin(45)$ $throat = 0.213\,in$

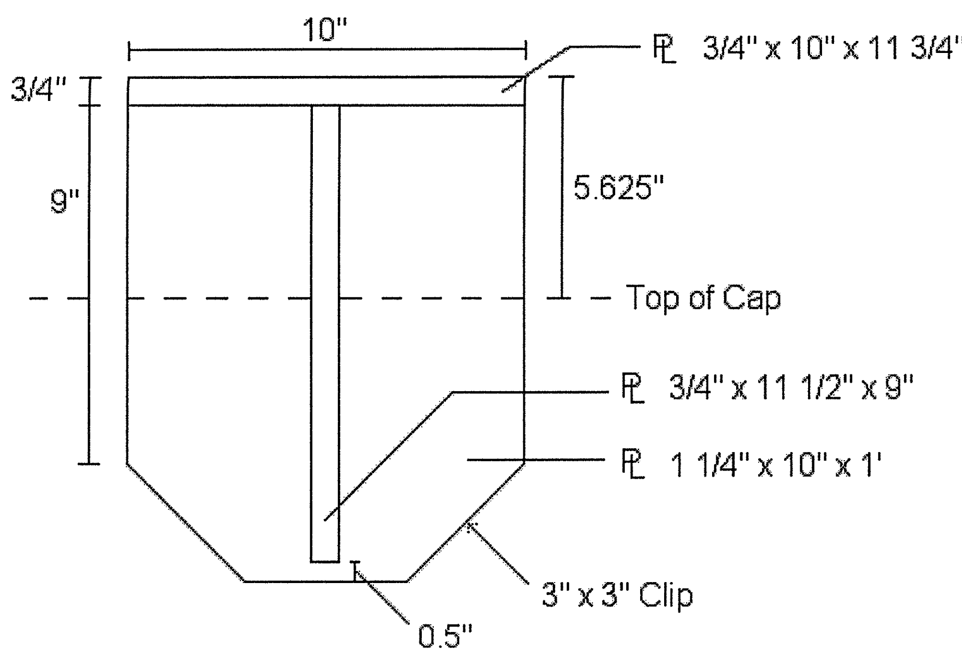
$F_u := 58 \cdot \frac{kip}{in^2}$ Grade 36 weld

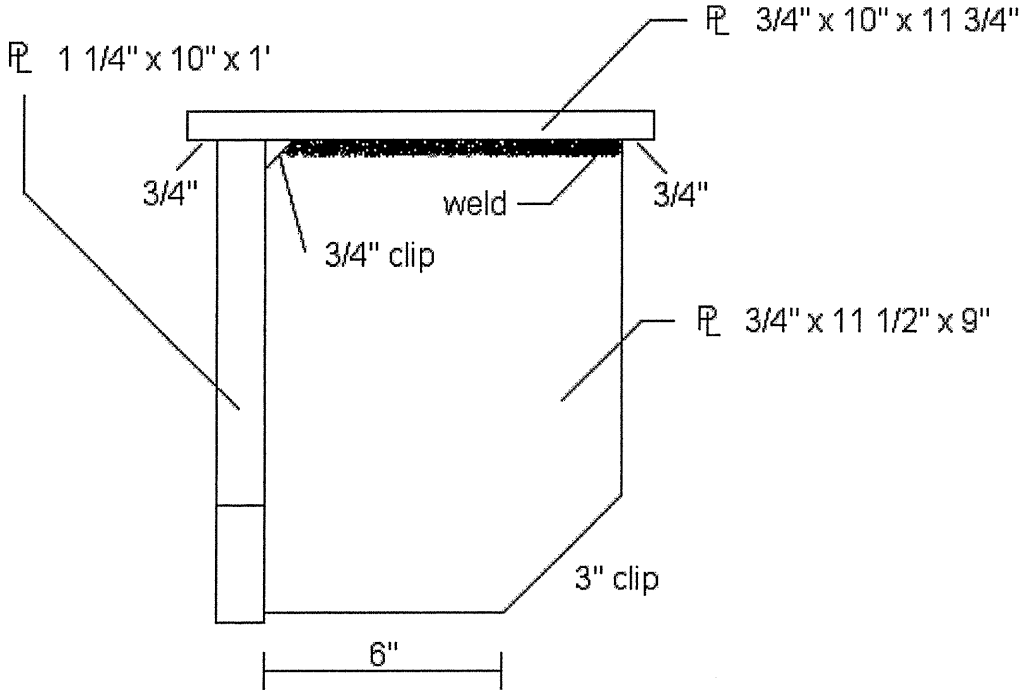
$F := 0.45 \cdot F_u$ $F = 26.1\,\frac{kip}{in^2}$ Weld Strength

Required length of weld: $L := \frac{V_u}{F \cdot throat}$ $L = 7.034\,in$

Allowable bearing stress on concrete is : $f_{ca} := \phi \cdot 0.85 \cdot f_c$ $f_{ca} = 2.083\,\frac{kip}{in^2}$

minimum concrete area $Area_{min} := \frac{V_u}{f_{ca}}$ $Area_{min} = 18.754\,in^2$





$A_{bearing} := 10in \cdot 3.375in + 7in \cdot 3in$

$BEARING_{AREA} :=$ | a ← "PASS" if $A_{bearing} > Area_{min}$
| a ← "FAIL" otherwise
| return a

$BEARING_{AREA} = \text{"PASS"}$

$A_{bearing} = 54.75\,in^2$

$L_{weld} := 2\left(9in - \frac{3}{4}in\right)$

$LENGTH_{WELD} :=$ | l ← "PASS" if $L_{weld} > L$
| l ← "FAIL" otherwise
| return l

$LENGTH_{WELD} = \text{"PASS"}$

$L_{weld} = 16.5\,in$

See Job 100381 for determination of plate thicknesses.

Pile Connection to Cap (Axial - Check Standard Strap Design)

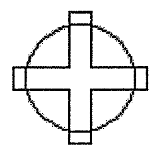
Maximum Seismic Axial Force from Column at Bent 2 in Seisab: $P_u := \dfrac{59.1kip}{R_{dabut}}$ $\quad P_u = 73.875\,kip$

Max strength of fillet weld = 0.45Fu and Grade 36 Straps, Fu = 58ksi.

$$F = 26.1\,\frac{kip}{in^2}$$

Use 1/4" WELD: $throat := \dfrac{1in}{4} \cdot \sin(45)$ $\quad throat = 0.213\,in$

Use four (4) straps: $P_u := \dfrac{P_u}{4}$



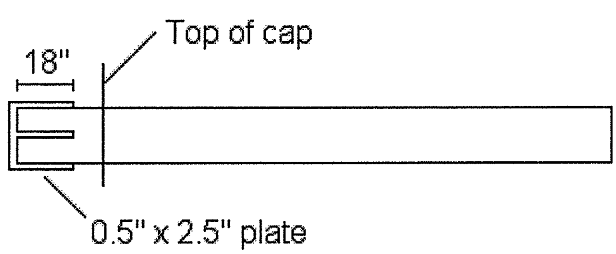
$$L := \frac{P_u}{F \cdot throat} \qquad L = 3.326\,in$$

Use a Plate 0.5" x 2.5"

$$\sigma_{max} := \frac{P_u}{0.5in \cdot 2.5in} \qquad \sigma_{max} = 14.775\,\frac{kip}{in^2}$$

$$PLATE_{CHECK} := \begin{array}{|l} p \leftarrow \text{"PASS"} \quad \text{if } \sigma_{max} < 36 \cdot \dfrac{kip}{in^2} \\ p \leftarrow \text{"FAIL"} \quad \text{otherwise} \\ \text{return } p \end{array}$$

$PLATE_{CHECK} = \text{"PASS"}$



Pile Check

Pile checks, and liquefaction checks for all 4 bents, have already been completed by Kyle. See his job folder.

07041 new code

```
                                   W     W  III  N   N
                                   W     W   I   NN  N
                                   W  W  W   I   N N N
                                    W W W    I   N  NN
                                     W W    III  N   N

  SSSSSSS   EEEEEEEEE  IIIIIIIII   SSSSSSS    AAAAAAA   BBBBBBBB
 SSSSSSSSS  EEEEEEEEE  IIIIIIIII  SSSSSSSSS  AAAAAAAAA  BBBBBBBBB
 SS     SS  EE            III     SS     SS  AA     AA  BB     BB
 SSS        EE            III     SSS        AA     AA  BB    BBB
  SSSSSS    EEEEE         III      SSSSSS    AAAAAAAAA  BBBBBBBB
   SSSSSS   EEEEE         III       SSSSSS   AAAAAAAAA  BBBBBBBB
       SSS  EE            III           SSS  AA     AA  BB    BBB
 SS     SS  EE            III     SS     SS  AA     AA  BB     BB
 SSSSSSSSS  EEEEEEEEE  IIIIIIIII  SSSSSSSSS  AA     AA  BBBBBBBBB
  SSSSSSS   EEEEEEEEE  IIIIIIIII   SSSSSSS   AA     AA  BBBBBBBB


*****************************************************************
*                                                               *
*                           WinSeisab                           *
*                                                               *
*                  Seismic Analysis of Bridges                  *
*                                                               *
*       Version 5.0.7                     Release 10/2003       *
*                                                               *
*                                                               *
*                                                               *
*                    Imbsen Software Systems                    *
*                                                               *
*                         www.Imbsen.com                        *
*                                                               *
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*            Windows (GUI) By:  CV-McBridge Software            *
*                                                               *
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*                                                               *
*------------------------- Licensed To: ------------------------*
*                                                               *
*        Arkansas State Highway & Transportation Dept.          *
*                                                               *
*------------------------- Mar 03, 2004 ------------------------*
*                                                               *
*                                                               *
*                                                               *
*                  Written By:  Roy Imbsen                      *
*                               Jon Lea                         *
*                               Clark Verkler                   *
*                               James Gates                     *
*                                                               *
*****************************************************************

Date:  13-SEP-06                                  Time:  14:40:34
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- - - - WinSEISAB - - - - (Version 5.0.7) 13-SEP-06
Imbsen and Associates, Inc.

Mud Slough Ditch

UNIFORM LOAD METHOD RESULTS

VIBRATION CHARACTERISTICS

MODE	PERIOD	CS
1	0.230	1.20
2	0.294	1.20

- - - - WinSEISAB - - - - (Version 5.0.7) 13-SEP-06
Imbsen and Associates, Inc.

Mud Slough Ditch

UNIFORM LOAD METHOD RESULTS

ABUTMENT PSEUDO DISPLACEMENTS

ITEM	LC	....LEFT FACE.... LNGTUDNL	TRNSVRSE	....RGHT FACE.... LNGTUDNL	TRNSVRSE	...OPNNG/CLSNG... LNGTUDNL	TRNSVRSE
ABU 1	1	0.052	0.000	0.052	0.000	0.000	0.000
	2	0.000	0.084	0.000	0.084	0.000	0.000
	3	0.052	0.025	0.052	0.025	0.000	0.000
	4	0.016	0.084	0.016	0.084	0.000	0.000
ABU 4	1	0.052	0.000	0.052	0.000	0.000	0.000
	2	0.000	0.084	0.000	0.084	0.000	0.000
	3	0.052	0.025	0.052	0.025	0.000	0.000
	4	0.016	0.084	0.016	0.084	0.000	0.000

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

- - - - WinSEISAB - - - - (Version 5.0.7) 13-SEP-06
Imbsen and Associates, Inc.

Mud Slough Ditch

UNIFORM LOAD METHOD RESULTS

COLUMN PSEUDO FORCES

CL	LOC	LC	....LNGITUDNL.... SHEAR	MOMENT	....TRANSVRSE.... SHEAR	MOMENT	AXIAL	TORSION
BNT 2								
1	BOT	1	17.5	181.	0.0	0.	0.1	0.0
		2	0.1	1.	31.6	316.	57.2	0.1
		3	17.5	181.	9.5	95.	17.2	0.0
		4	5.3	55.	31.6	316.	$\boxed{57.2}$	0.1
1	TOP	1	17.5	163.	0.0	0.	0.1	0.0
		2	0.1	1.	31.6	306.	57.2	0.1
		3	17.5	164.	9.5	92.	17.2	0.0
		4	5.3	50.	31.6	306.	57.2	0.1
2	BOT	1	17.5	181.	0.0	0.	0.4	0.0
		2	0.1	1.	32.8	324.	12.5	0.1
		3	17.5	181.	9.8	97.	4.2	0.0
		4	5.3	55.	32.8	324.	12.7	0.1
2	TOP	1	17.5	163.	0.0	0.	0.4	0.0
		2	0.1	1.	32.8	321.	12.5	0.1
		3	17.5	164.	9.8	96.	4.2	0.0
		4	5.3	50.	32.8	321.	12.7	0.1
3	BOT	1	17.5	181.	0.0	0.	0.8	0.0
		2	0.0	0.	32.9	325.	4.1	0.1
		3	17.5	181.	9.9	97.	2.1	0.0
		4	5.3	55.	32.9	325.	4.3	0.1
3	TOP	1	17.5	163.	0.0	0.	0.8	0.0
		2	0.0	0.	32.9	322.	4.1	0.1
		3	17.5	164.	9.9	97.	2.1	0.0
		4	5.3	49.	32.9	322.	4.3	0.1
4	BOT	1	17.5	181.	0.1	1.	6.8	0.0
		2	0.0	0.	32.8	324.	0.2	0.1
		3	17.5	181.	10.0	98.	6.9	0.0
		4	5.3	54.	32.8	324.	2.2	0.1

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

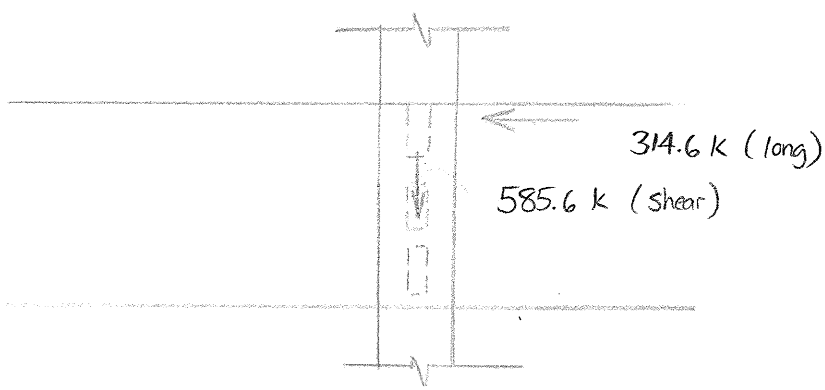
- - - - WinSEISAB - - - - (Version 5.0.7) 13-SEP-06
Imbsen and Associates, Inc.

Mud Slough Ditch

UNIFORM LOAD METHOD RESULTS

ABUTMENT PSEUDO FORCES

ITEM	LC	VERT SHEAR	W/R TO BRIDGE C.L. LONGITUDNL	W/R TO BRIDGE C.L. TRANSVERSE	W/R TO ITEM C.L. NORMAL	W/R TO ITEM C.L. PARALLEL
ABU 1	1	23.8	448.4	0.0	448.4	0.0
	2	0.0	0.0	312.9	0.0	312.9
	3	23.8	$\boxed{448.4}$	93.9	448.4	93.9
	4	7.1	134.5	$\boxed{312.9}$	134.5	312.9
ABU 4	1	23.8	448.4	0.0	448.4	0.0
	2	0.0	0.0	312.9	0.0	312.9
	3	23.8	448.4	93.9	448.4	93.9
	4	7.1	134.5	312.9	134.5	312.9



INTEGRAL BRIDGE → USE BENT FORCES

See next page

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

- - - - WinSEISAB - - - -   (Version 5.0.7)   13-SEP-06
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Mud Slough Ditch

UNIFORM LOAD METHOD RESULTS

BENT PSEUDO FORCES

ITEM	LC	VERT SHEAR	W/R TO BRIDGE C.L. LONGITUDNL	W/R TO BRIDGE C.L. TRANSVERSE	W/R TO ITEM C.L. NORMAL	W/R TO ITEM C.L. PARALLEL
BNT 2	1	37.6	157.3	0.0	157.3	0.0
	2	0.0	0.0	292.8	0.0	292.8
	3	37.6	$\boxed{157.3}$	87.8	157.3	87.8
	4	11.3	47.2	$\boxed{292.8}$	47.2	292.8
BNT 3	1	37.6	157.3	0.0	157.3	0.0
	2	0.0	0.0	292.8	0.0	292.8
	3	37.6	157.3	87.8	157.3	87.8
	4	11.3	47.2	292.8	47.2	292.8

LONG PESTR

2(157.3) = 314.6

SHEAR

2(292.8) = 585.6

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

07041 NEW UP. SSB

```
                                  W     W  III  N   N
                                  W     W   I   NN  N
                                  W  W  W   I   N N N
                                   W W W    I   N  NN
                                    W W    III  N   N

  SSSSSSS   EEEEEEEEE  IIIIIIIII   SSSSSSS    AAAAAAA   BBBBBBBB
 SSSSSSSSS  EEEEEEEEE  IIIIIIIII  SSSSSSSSS  AAAAAAAAA  BBBBBBBBB
 SS     SS  EE            III     SS     SS  AA     AA  BB     BB
 SSS        EE            III     SSS        AA     AA  BB    BBB
  SSSSSS    EEEEE         III      SSSSSS    AAAAAAAAA  BBBBBBBB
   SSSSSS   EEEEE         III       SSSSSS   AAAAAAAAA  BBBBBBBB
       SSS  EE            III           SSS  AA     AA  BB    BBB
 SS     SS  EE            III     SS     SS  AA     AA  BB     BB
 SSSSSSSSS  EEEEEEEEE  IIIIIIIII  SSSSSSSSS  AA     AA  BBBBBBBBB
  SSSSSSS   EEEEEEEEE  IIIIIIIII   SSSSSSS   AA     AA  BBBBBBBB
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*                                                              *
*                          WinSeisab                           *
*                                                              *
*                 Seismic Analysis of Bridges                  *
*                                                              *
*       Version 5.0.7                     Release 10/2003      *
*                                                              *
*                                                              *
*                                                              *
*                   Imbsen Software Systems                    *
*                                                              *
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*                                                              *
*        Arkansas State Highway & Transportation Dept.         *
*                                                              *
*------------------------ Mar 03, 2004 ------------------------*
*                                                              *
*                                                              *
*                                                              *
*                 Written By:  Roy Imbsen                      *
*                              Jon Lea                         *
*                              Clark Verkler                   *
*                              James Gates                     *
*                                                              *
****************************************************************

Date:  28-AUG-06                                 Time:  15:44:09
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- - - - WinSEISAB - - - - (Version 5.0.7) 28-AUG-06
Imbsen and Associates, Inc.

Mud Slough Ditch

UNIFORM LOAD METHOD RESULTS

VIBRATION CHARACTERISTICS

MODE	PERIOD	CS
1	0.086	1.17
2	0.226	1.20

- - - - WinSEISAB - - - - (Version 5.0.7) 28-AUG-06
Imbsen and Associates, Inc.

Mud Slough Ditch

UNIFORM LOAD METHOD RESULTS

ABUTMENT PSEUDO DISPLACEMENTS

ITEM	LC	....LEFT FACE.... LNGTUDNL	TRNSVRSE	....RGHT FACE.... LNGTUDNL	TRNSVRSE	...OPNNG/CLSNG... LNGTUDNL	TRNSVRSE
ABU 1	1	0.007	0.000	0.007	0.000	0.000	0.000
	2	0.000	0.049	0.000	0.049	0.000	0.000
	3	0.007	0.015	0.007	0.015	0.000	0.000
	4	0.002	0.049	0.002	0.049	0.000	0.000
ABU 4	1	0.007	0.000	0.007	0.000	0.000	0.000
	2	0.000	0.049	0.000	0.049	0.000	0.000
	3	0.007	0.015	0.007	0.015	0.000	0.000
	4	0.002	0.049	0.002	0.049	0.000	0.000

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

- - - - WinSEISAB - - - - (Version 5.0.7) 28-AUG-06
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Mud Slough Ditch

UNIFORM LOAD METHOD RESULTS

COLUMN PSEUDO FORCES

			....LNGITUDNL....		....TRANSVRSE....			
CL	LOC	LC	SHEAR	MOMENT	SHEAR	MOMENT	AXIAL	TORSION
BNT 2								
1	BOT	1	2.3	24.	0.0	0.	0.0	0.0
		2	0.2	2.	18.7	187.	33.8	0.1 STRAP
		3	2.4	25.	5.6	56.	10.2	0.0
		4	0.9	9.	18.7	187.	33.8	0.1
1	TOP	1	2.3	22.	0.0	0.	0.0	0.0
		2	0.2	2.	18.7	181.	33.8	0.1
		3	2.4	22.	5.6	54.	10.2	0.0
		4	0.9	8.	18.7	181.	33.8	0.1
2	BOT	1	2.3	24.	0.0	0.	0.1	0.0
		2	0.1	1.	19.4	192.	7.4	0.1
		3	2.4	25.	5.8	57.	2.3	0.0
		4	0.8	8.	19.4	192.	7.4	0.1
2	TOP	1	2.3	22.	0.0	0.	0.1	0.0
		2	0.1	1.	19.4	190.	7.4	0.1
		3	2.4	22.	5.8	57.	2.3	0.0
		4	0.8	8.	19.4	190.	7.4	0.1
3	BOT	1	2.3	24.	0.0	0.	0.1	0.0
		2	0.1	1.	19.4	192.	2.4	0.1
		3	2.4	24.	5.8	58.	0.8	0.0
		4	0.8	8.	19.4	192.	2.4	0.1
3	TOP	1	2.3	22.	0.0	0.	0.1	0.0
		2	0.1	1.	19.4	191.	2.4	0.1
		3	2.4	22.	5.8	57.	0.8	0.0
		4	0.8	7.	19.4	191.	2.4	0.1
4	BOT	1	2.3	24.	0.0	0.	0.9	0.0
		2	0.0	0.	19.4	192.	0.1	0.1
		3	2.4	24.	5.8	58.	0.9	0.0
		4	0.7	8.	19.4	192.	0.4	0.1

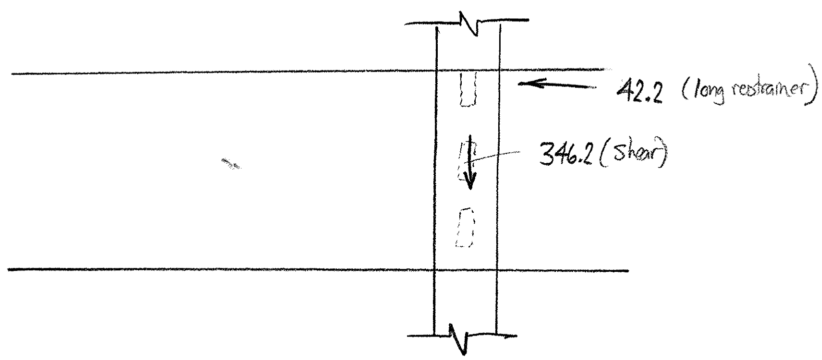
*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

Mud Slough Ditch

UNIFORM LOAD METHOD RESULTS

ABUTMENT PSEUDO FORCES

ITEM	LC	VERT SHEAR	W/R TO BRIDGE C.L. LONGITUDNL	W/R TO BRIDGE C.L. TRANSVERSE	W/R TO ITEM C.L. NORMAL	W/R TO ITEM C.L. PARALLEL
ABU 1	1	3.2	568.2	0.0	568.2	0.0
	2	0.0	0.0	432.6	0.0	432.6
	3	3.2	568.2	129.8	568.2	129.8
	4	1.0	170.5	432.6	170.5	432.6
ABU 4	1	3.2	568.2	0.0	568.2	0.0
	2	0.0	0.0	432.6	0.0	432.6
	3	3.2	568.2	129.8	568.2	129.8
	4	1.0	170.5	432.6	170.5	432.6



USE BENT FORCES FOR RESTRAINERS
(INTEGRAL ABUTMENTS)

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

- - - - WinSEISAB - - - - (Version 5.0.7) 28-AUG-06
Imbsen and Associates, Inc.

Mud Slough Ditch

UNIFORM LOAD METHOD RESULTS

BENT PSEUDO FORCES

ITEM	LC	VERT SHEAR	W/R TO BRIDGE C.L. LONGITUDNL	W/R TO BRIDGE C.L. TRANSVERSE	W/R TO ITEM C.L. NORMAL	W/R TO ITEM C.L. PARALLEL
BNT 2	1	5.0	21.1	0.0	21.1	0.0
	2	0.0	0.0	173.1	0.0	173.1
	3	5.0	21.1	51.9	21.1	51.9
	4	1.5	6.3	173.1	6.3	173.1
BNT 3	1	5.0	21.1	0.0	21.1	0.0
	2	0.0	0.0	173.1	0.0	173.1
	3	5.0	21.1	51.9	21.1	51.9
	4	1.5	6.3	173.1	6.3	173.1

LONG RESTR.
2(21.1) = 42.2

SHEAR RESTR.
2(173.1) = 346.2

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

07041 NEW DOWN.SSB

```
                  W     W  III  N   N
                  W     W   I   NN  N
                  W  W  W   I   N N N
                   W W W    I   N  NN
                    W W    III  N   N

 SSSSSSS   EEEEEEEEE  IIIIIIIII   SSSSSSS    AAAAAAA   BBBBBBBB
SSSSSSSSS  EEEEEEEEE  IIIIIIIII  SSSSSSSSS  AAAAAAAAA  BBBBBBBBB
SS     SS  EE            III     SS     SS  AA     AA  BB     BB
SSS        EE            III     SSS        AA     AA  BB    BBB
 SSSSSS    EEEEE         III      SSSSSS    AAAAAAAAA  BBBBBBBB
  SSSSSS   EEEEE         III       SSSSSS   AAAAAAAAA  BBBBBBBB
      SSS  EE            III           SSS  AA     AA  BB    BBB
SS     SS  EE            III     SS     SS  AA     AA  BB     BB
SSSSSSSSS  EEEEEEEEE  IIIIIIIII  SSSSSSSSS  AA     AA  BBBBBBBBB
 SSSSSSS   EEEEEEEEE  IIIIIIIII   SSSSSSS   AA     AA  BBBBBBBB

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*                          WinSeisab                            *
*                                                               *
*                 Seismic Analysis of Bridges                   *
*                                                               *
*       Version 5.0.7                      Release 10/2003      *
*                                                               *
*                                                               *
*                                                               *
*                   Imbsen Software Systems                     *
*                                                               *
*                        www.Imbsen.com                         *
*                                                               *
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*                                                               *
*           Windows (GUI) By:  CV-McBridge Software             *
*                                                               *
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*                                                               *
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*------------------------ Mar 03, 2004 ------------------------*
*                                                               *
*                                                               *
*                                                               *
*                 Written By:  Roy Imbsen                       *
*                              Jon Lea                          *
*                              Clark Verkler                    *
*                              James Gates                      *
*                                                               *
*****************************************************************

Date:   28-AUG-06                                Time:   15:55:38
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- - - - WinSEISAB - - - -   (Version 5.0.7)   28-AUG-06
Imbsen and Associates, Inc.

Mud Slough Ditch

UNIFORM LOAD METHOD RESULTS

VIBRATION CHARACTERISTICS

MODE	PERIOD	CS
1	0.308	1.20
2	0.298	1.20

Mud Slough Ditch

UNIFORM LOAD METHOD RESULTS

ABUTMENT PSEUDO DISPLACEMENTS (Same as Bent)

ITEM	LC	....LEFT FACE.... LNGTUDNL	TRNSVRSE	....RGHT FACE.... LNGTUDNL	TRNSVRSE	...OPNNG/CLSNG... LNGTUDNL	TRNSVRSE
ABU 1	1	0.093	0.000	0.093	0.000	0.000	0.000
	2	0.000	0.087	0.000	0.087	0.000	0.000
	3	0.093	0.026	0.093	0.026	0.000	0.000
	4	0.028	0.087	0.028	0.087	0.000	0.000
ABU 4	1	0.093	0.000	0.093	0.000	0.000	0.000
	2	0.000	0.087	0.000	0.087	0.000	0.000
	3	0.093	0.026	0.093	0.026	0.000	0.000
	4	0.028	0.087	0.028	0.087	0.000	0.000

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

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Mud Slough Ditch

UNIFORM LOAD METHOD RESULTS

COLUMN PSEUDO FORCES

CL	LOC	LC	....LNGITUDNL.... SHEAR	MOMENT	....TRANSVRSE.... SHEAR	MOMENT	AXIAL	TORSION
BNT 2								
1	BOT	1	31.2	323.	0.0	0.	0.1	0.0
		2	0.1	1.	32.6	326.	59.1	0.1
		3	31.3	323.	9.8	98.	17.8	0.0
		4	9.5	98.	32.6	326.	59.1	0.1
1	TOP	1	31.2	292.	0.0	0.	0.1	0.0
		2	0.1	1.	32.6	316.	59.1	0.1
		3	31.3	293.	9.8	95.	17.8	0.0
		4	9.5	89.	32.6	316.	59.1	0.1
2	BOT	1	31.2	323.	0.0	0.	0.8	0.0
		2	0.1	1.	33.8	334.	12.9	0.1
		3	31.3	323.	10.2	100.	4.7	0.0
		4	9.4	98.	33.8	334.	13.2	0.1
2	TOP	1	31.2	292.	0.0	0.	0.8	0.0
		2	0.1	1.	33.8	331.	12.9	0.1
		3	31.3	292.	10.2	100.	4.7	0.0
		4	9.4	88.	33.8	331.	13.2	0.1
3	BOT	1	31.2	323.	0.0	0.	1.5	0.0
		2	0.0	0.	33.9	335.	4.2	0.1
		3	31.3	323.	10.2	101.	2.8	0.0
		4	9.4	97.	33.9	335.	4.7	0.1
3	TOP	1	31.2	292.	0.0	0.	1.5	0.0
		2	0.0	0.	33.9	333.	4.2	0.1
		3	31.3	292.	10.2	100.	2.8	0.0
		4	9.4	88.	33.9	333.	4.7	0.1
4	BOT	1	31.2	323.	0.2	1.	12.2	0.0
		2	0.0	0.	33.8	335.	0.2	0.1
		3	31.2	323.	10.4	102.	12.3	0.0
		4	9.4	97.	33.9	335.	3.8	0.1

STRAP

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

Mud Slough Ditch

UNIFORM LOAD METHOD RESULTS

ABUTMENT PSEUDO FORCES

ITEM	LC	VERT SHEAR	W/R TO BRIDGE C.L. LONGITUDNL	W/R TO BRIDGE C.L. TRANSVERSE	W/R TO ITEM C.L. NORMAL	W/R TO ITEM C.L. PARALLEL
ABU 1	1	42.5	324.5	0.0	324.5	0.0
	2	0.0	0.0	303.6	0.0	303.6
	3	42.5	324.5	91.1	324.5	91.1
	4	12.8	97.4	303.6	97.4	303.6
ABU 4	1	42.5	324.5	0.0	324.5	0.0
	2	0.0	0.0	303.6	0.0	303.6
	3	42.5	324.5	91.1	324.5	91.1
	4	12.8	97.4	303.6	97.4	303.6

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

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Mud Slough Ditch

UNIFORM LOAD METHOD RESULTS

BENT PSEUDO FORCES

ITEM	LC	VERT SHEAR	W/R TO BRIDGE C.L. LONGITUDNL	W/R TO BRIDGE C.L. TRANSVERSE	W/R TO ITEM C.L. NORMAL	W/R TO ITEM C.L. PARALLEL
BNT 2	1	67.2	281.2	0.0	281.2	0.0
	2	0.0	0.0	302.1	0.0	302.1
	3	67.2	281.2	90.6	281.2	90.6
	4	20.2	84.4	302.1	84.4	302.1
BNT 3	1	67.2	281.2	0.0	281.2	0.0
	2	0.0	0.0	302.1	0.0	302.1
	3	67.2	281.2	90.6	281.2	90.6
	4	20.2	84.4	302.1	84.4	302.1

LONG. RESTR.

2 (281.2) = 562.4 kip

SHEAR RESTR.

2(302.1) = 604.2 kip

CONTROLS

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

Seismic Evaluation of 100566 - Bridge 07041 (Mud Slough Ditch)

90' Three Span Continuous Bridge (28' - 34' - 28')
No Skew
Composite W-Beam
Trestle Pile Bents
Integral Pile Abutment
Steel Shell Piles
Jointless w/ Elasto Bearings @ Int. Bents.
Exisitng Bridge located at Lat. = 36.0503 Long. = -90.3689

$A := 0.226$ From 1988 USGS Maps (10% in 50 yrs)

Bridge Classification is "Other" (LRFD 3.10.3)

$$\text{Zone}_{\text{seismic}} := \left| \begin{array}{l} z \leftarrow 1 \ \text{ if } \ A < 0.09 \\ z \leftarrow 2 \ \text{ if } \ 0.09 < A < 0.19 \\ z \leftarrow 3 \ \text{ if } \ 0.19 < A < 0.29 \\ z \leftarrow 4 \ \text{ otherwise} \\ \text{return } z \end{array} \right.$$

$\text{Zone}_{\text{seismic}} = 3$

$S := 1.2$ Soil Profile Type II (Stiff Clay and Medium Dense Sand)

$T_m := 0.305$ From WinSeisab with interior bent piles fixed at estimated fixity and only the piles providing stiffness at the abutments. This is for mode 1.

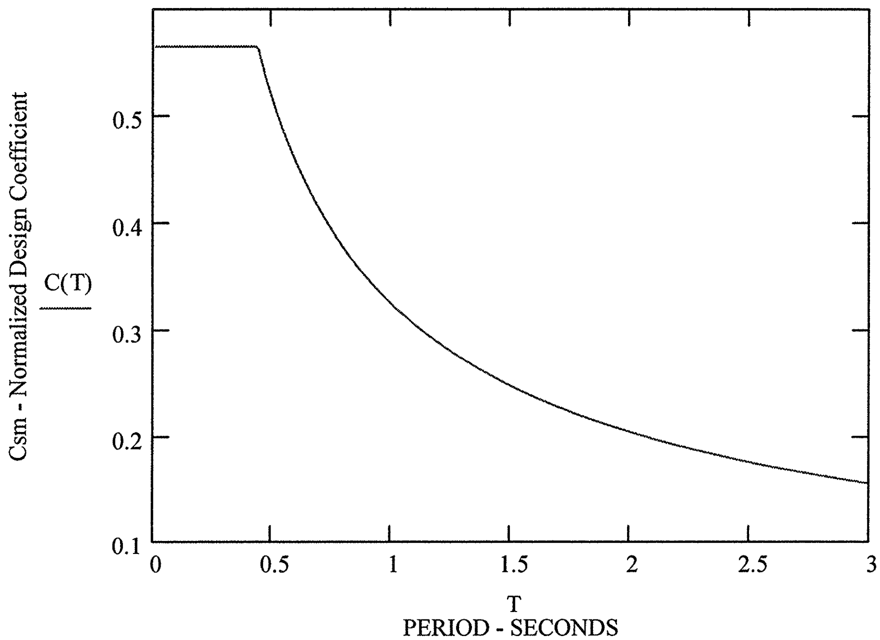
$Tm := 0.086$ With the abutment soil adding stiffness.

$T := 0.01, 0.02 .. 3$ Defining a range of seconds from 0 to 3 seconds.

$$C_{sm}(T) := \frac{1.2A \cdot S}{T^{\frac{2}{3}}}$$

$$C(T) := \left| \begin{array}{l} c \leftarrow C_{sm}(T) \ \text{ if } \ C_{sm}(T) < 2.5 \cdot A \\ c \leftarrow 2.5 \cdot A \ \text{ otherwise} \\ \text{return } c \end{array} \right.$$

The "elastic seismic response coefficient" function is defined above and graphed below for a period range between 0 and 3 seconds.



3.10.7 Response Modification Factors

To be able to use the response modification factors, the transverse reinforcement in regions of expected plastic hinges must be **hooked** - with a 135 degree bend plus an extension of the larger of 6d or 3 inches. (LRFD 5.10.2.2)

The area of longitudinal column steel must be between 0.01 and 0.06 the concrete gross area. There are many other requirements, see LRFD 5.10.11 for more information.

Concrete filled pipe piles must be anchored to the cap by steel dowels with a minimum steel ratio of 0.01. The upper end of every pile shall be reinforced and confined as a potential plastic hinge location. See LRFD 5.13.4.6 for more information.

$R_{dsub} := 5.0$ Response modification factor for composite steel piles or multicolumn bents.

$R_{dabut} := 0.8$ Response modification factor for superstructure loads to abutment.

All other cases the response modification factor is 1.0.

3.10.7.2 Application

Seismic loads can act in any direction, and the R-factor shall be used for both orthogonal axes of the substructure.

3.10.8 Combination of Seismic Force Effects

There are only two (2) load cases.

100% of the absolute value of the force in one perpendicular direction, combined with 30% of the absolute value in the second perpendicular direction.

100% of the absolute value of the force in the second perpendicular direction, combined with 30% of the absolute value in the first perpendicular direction.

3.10.9.2 Seismic Zone 1

Horizontal design force shall be at least 0.2 times the vertical reaction due to the permanent dead load and live loads assumed during an earthquake. Unless the bridge is very far off the ground (and maybe even then) this earthquake requirement will control over wind loads.

For longitudinal forces, each fixed bent gets the total dead load divided equally by the number of fixed bents.

For transverse forces, each bent gets its own tributary dead load.

This force is used to design the elastomeric bearings with its sole and masonry plates.

No dynamic analysis is required for Seismic Zone 1.

3.10.9.3 Seismic Zone 2

A dynamic analysis is required starting at seismic zone 2. Table 4.7.4.3.1-1 gives the minimum analysis requirements Except for foundations, the forces found in this dynamic analysis shall be divided by the R-factor. With foundations, other than pile bents, the dynamic analysis forces are divided by half the R-factor.

3.10.9.4.1 Seismic Zone 3 and 4

A dynamic analysis, like in seismic zone 2, is required. However, the foundation forces are not reduced by an R-factor. Therefore damage to the structure should occur in the columns by inelastic hinging.

By taking into account inelastic hinging, you can reduce the forces in the foundation. Find the maximum axial and flexural load that the columns/piers can carry, and then reduce the foundation design to match the column maximum capacity.

Transverse Shear Design $\qquad$ $kip := 1000lb$

$EQ := 141.2kip$ $\qquad$ Max transverse seismic force at Int. Bents from Seisab (no abutment soil condition)

$V_u := \frac{EQ}{R_{dabut}}$ $\qquad$ $V_u = 176.5\,kip$ $\qquad$ See above, also from LRFD 3.10.7

It is office policy that shear blocks must be used in a Seismic 3 or 4 region. If four shear blocks are assumed (75' wide bridge), the total required shear capacity can be divided by 4.

$V_u := \frac{V_u}{4}$ $\qquad$ $V_u = 44.125\,kip$

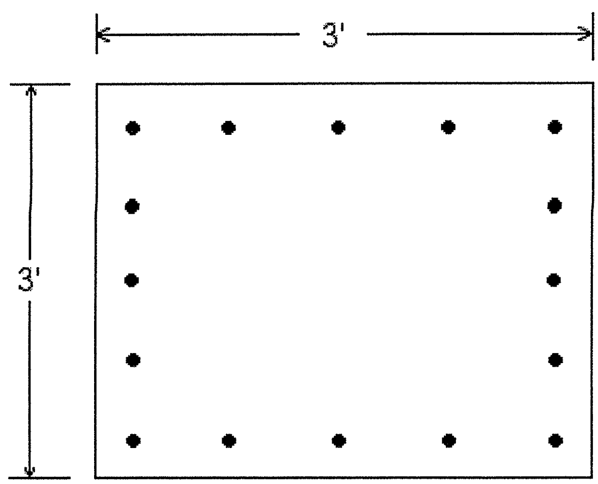
Four 3'-0 wide by 3'-0" long shear blocks are used, USE SHEAR FRICTION design (LRFD 5.8.4).

Use **#4 rebars** for the shear blocks, 16 total.

$f_y := 60\frac{kip}{in^2}$ $\qquad$ $A_{vf} := 16\cdot\pi\left(\frac{4in}{16}\right)^2$

$\mu := 1$ $\qquad$ $A_{vf} = 3.142\,in^2$

$V_n := A_{vf}\cdot f_y\cdot\mu$ $\qquad$ $V_n = 188.496\,kip$



Plan View

But must be less than:

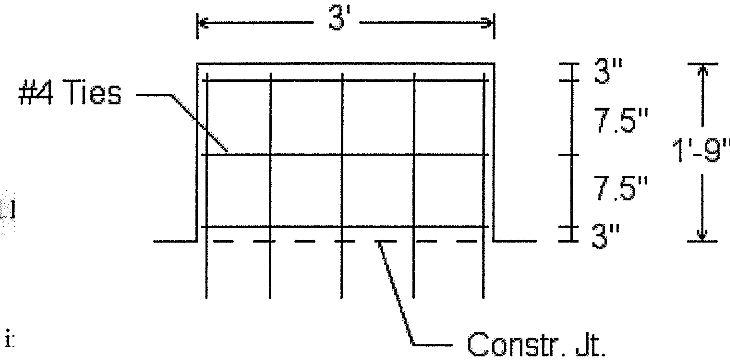
$f_c := 3.5\frac{kip}{in^2}$ $\qquad$ $A_{cv} := (3ft)^2$

$Check1 := 0.2f_c\cdot A_{cv}$ $\qquad$ $Check1 = 907.2\,kip$

$Check2 := 0.8\cdot\frac{kip}{in^2}\cdot A_{cv}$ $\qquad$ $Check2 = 1.037\times10^3\,kip$

SHEAR_BLOCK := | s ← "PASS" if V_n < Check1 < Check2
| s ← "PASS" if V_n < Check2 < Check1
| s ← "FAIL" otherwise
| return s

R_BLOCK = "PASS"



$V_\phi := 0.85 \cdot V_n$ $\qquad V_\phi = 160.2211$

SHEAR_BLOCK := | s ← "PASS" i:
| s ← "FAIL" otherwise
| return s

SHEAR_BLOCK = "PASS"

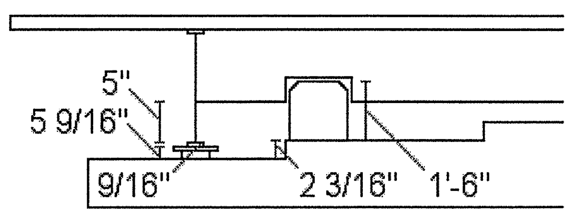
Instead of using 4 shear blocks, **2 shear blocks would be adequate.**

Bearing Stress (5.7.5)

The factored bearing stress shall be taken as: $\qquad P_r = \phi \cdot P_n \qquad \phi := 0.7$

This bearing check is on the transverse load coming in horizontally to bear on the shear blocks. The force is transmitted by a concrete longitudinal restrainer at the interior bents which is one (1) foot wide. The height of the bearing area is calculated as shown.

$$h_b := \left(18 + 2 + \frac{3}{16} - \frac{9}{16} - 5 - \frac{9}{16} - 5\right) in$$



$h_b = 9.063\,in$

Subtract the chamfered corner of the shear block from the bearing area.

$h_b := h_b - \frac{3}{4} in \qquad h_b = 8.313\,in \qquad m := 1$ Modification factor

$A_1 := h_b \cdot 12in$ $\qquad A_1 = 99.75\,in^2$

The bearing area of the concrete transverse restrainer on a shear block.

$P_n := 0.85 \cdot f_c \cdot A_1 \cdot m \qquad P_r := \phi \cdot P_n \qquad P_r = 207.729\,kip$

$$\text{BEARING} := \begin{array}{|l} b \leftarrow \text{"PASS"} \;\; \text{if } V_u < P_r \\ b \leftarrow \text{"FAIL"} \;\; \text{otherwise} \\ \text{return } b \end{array}$$

$\text{BEARING} = \text{"PASS"}$

Shear Ties for the **Shear Block** (LRFD 5.8.3.3)

$V_n := \dfrac{V_u}{0.85} \qquad V_n = 51.912\,kip \qquad b_v := 36in - 2 \cdot 3in \qquad d_v := 21in - 2 \cdot 3in$

$V_c := 2 \cdot 3500^{0.5} \cdot \left(\dfrac{lb}{in^2}\right) \cdot b_v \cdot d_v \qquad V_c = 53.245\,kip$

$V_s := V_n - V_c \qquad V_s = -1.333\,kip$

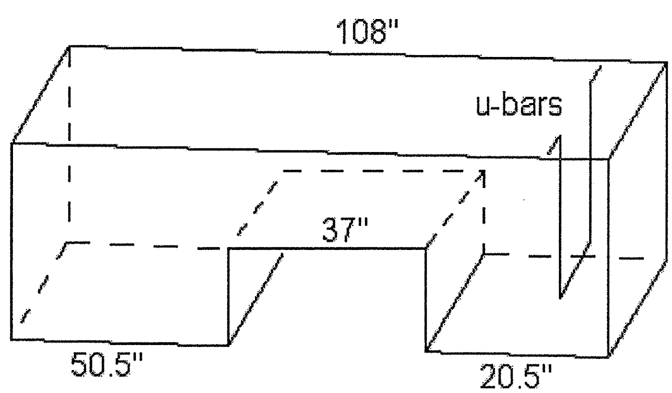
The concrete alone can handle the shear stress, but there will be required minimum reinforcement. The minimum reinforcement requirement can be waived if:

$$\frac{V_n}{A_{cv}} < 0.100ksi$$

$\dfrac{V_n}{A_{cv}} = 0.04\,\dfrac{kip}{in^2}$

No reinforcement is required at all. I will provide #4 bars at 7.5" spacing as shown previously.

Concrete Diaphragm Design



There is **shear friction** between the slab and diaphragm (LRFD 5.8.4).

Try 9 #4 U-Bars at 12"

$n := 9 \qquad A_4 := \pi \cdot \left(\dfrac{4in}{16}\right)^2 \qquad m := 1$

$V_n := n \cdot A_4 \cdot 60 \frac{kip}{in^2} \cdot m \qquad V_n = 106.029\,kip$

$b_v := 12in \qquad d_v := 108in \qquad A_{cv} := b_v \cdot d_v$

$Check1 := 0.2 f_c \cdot A_{cv} \qquad Check1 = 907.2\,kip$

$Check2 := 0.8 \cdot \frac{kip}{in^2} \cdot A_{cv} \qquad Check2 = 1.037 \times 10^3\,kip$

$$V_N := \left| \begin{array}{l} min \leftarrow Check1 \;\; \text{if } Check1 < Check2 \\ min \leftarrow Check2 \;\; \text{otherwise} \\ v \leftarrow V_n \;\; \text{if } V_n < min \\ v \leftarrow min \;\; \text{if } V_n > min \\ \text{return } v \end{array} \right.$$

$V_N = 106.029\,kip$

$$CONCDIA_V := \left| \begin{array}{l} v \leftarrow \text{"PASS"} \;\; \text{if } V_N > V_u \\ v \leftarrow \text{"FAIL"} \;\; \text{otherwise} \\ \text{return } v \end{array} \right.$$

$CONCDIA_V = \text{"PASS"}$

Longitudinal Steel

Use #6 bars longitudinal, see Job 110390, bridge 06983

Longitudinal Restrainers Design

$EQ := 134kip$ Maximum Longitudinal Force in Seisab at Interior Bents.

$beams := 9$ There are nine beams that will take longitudinal force.

$V_u := \frac{EQ}{R_{dabut} \cdot beams} \qquad V_u = 18.611\,kip$ Each restrainer will carry 18.6 kip.

Use 1/16" weld: $throat := \left(\frac{1\,in}{4}\right)\cdot \sin(45)$ $throat = 0.213\,in$

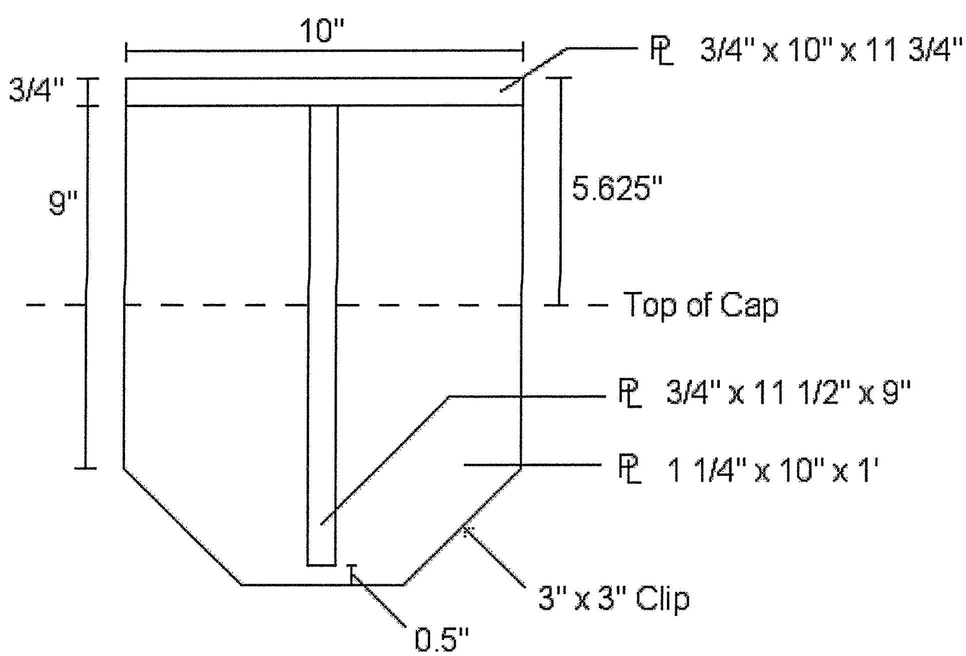
$F_u := 58\cdot\frac{kip}{in^2}$ Grade 36 weld

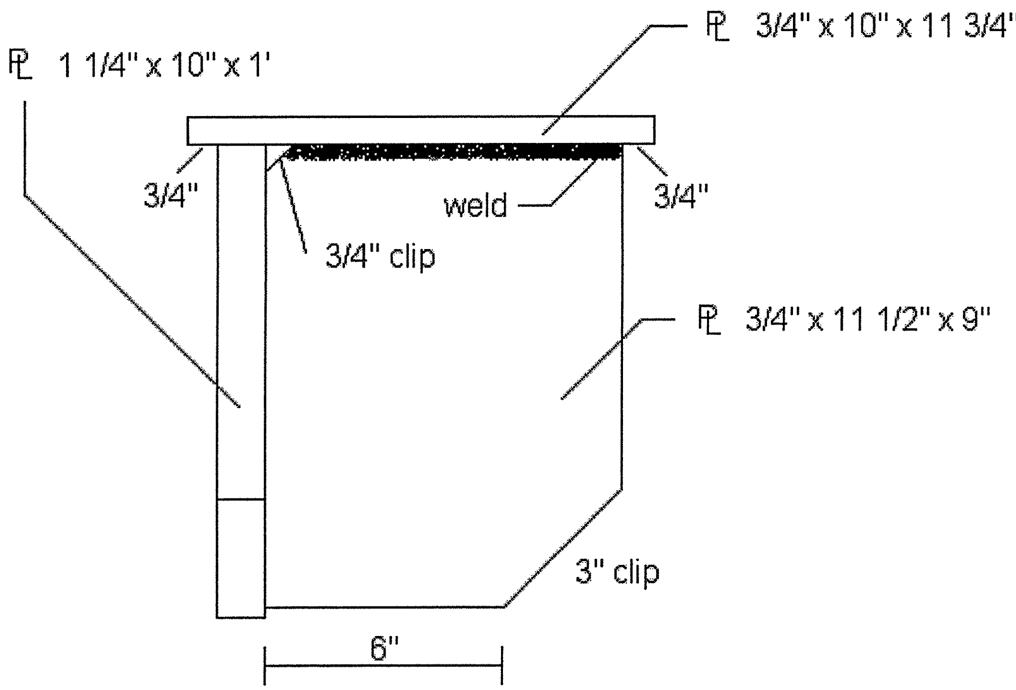
$F := 0.45\cdot F_u$ $F = 26.1\,\frac{kip}{in^2}$ Weld Strength

Required length of weld: $L := \frac{V_u}{F\cdot throat}$ $L = 3.352\,in$

Allowable bearing stress on concrete is : $f_{ca} := \phi\cdot 0.85\cdot f_c$ $f_{ca} = 2.083\,\frac{kip}{in^2}$

minimum concrete area $Area_{min} := \frac{V_u}{f_{ca}}$ $Area_{min} = 8.937\,in^2$





$A_{bearing} := 10in \cdot 3.375in + 7in \cdot 3in$

$BEARING_{AREA} = \text{"PASS"}$

$A_{bearing} = 54.75\,in^2$

$$BEARING_{AREA} := \left| \begin{array}{l} a \leftarrow \text{"PASS"} \quad \text{if } A_{bearing} > Area_{min} \\ a \leftarrow \text{"FAIL"} \quad \text{otherwise} \\ \text{return } a \end{array} \right.$$

$L_{weld} := 2\left(9in - \frac{3}{4}in\right)$

$LENGTH_{WELD} = \text{"PASS"}$

$L_{weld} = 16.5\,in$

$$LENGTH_{WELD} := \left| \begin{array}{l} l \leftarrow \text{"PASS"} \quad \text{if } L_{weld} > L \\ l \leftarrow \text{"FAIL"} \quad \text{otherwise} \\ \text{return } l \end{array} \right.$$

See Job 100381 for determination of plate thicknesses.

Pile Connection to Cap (Axial - Check Standard Strap Design)

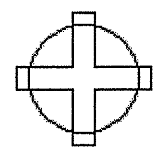
Maximum Seismic Axial Force from Column at Bent 2 in Seisab: $P_u := \dfrac{26.7\text{kip}}{R_{dabut}}$ $\quad P_u = 33.375\,\text{kip}$

Max strength of fillet weld = 0.45Fu and Grade 36 Straps, Fu = 58ksi.

$$F = 26.1\,\frac{\text{kip}}{\text{in}^2}$$

Use 1/4" WELD: $\text{throat} := \dfrac{1\text{in}}{4}\cdot\sin(45)$ $\quad \text{throat} = 0.213\,\text{in}$

Use four (4) straps: $P_u := \dfrac{P_u}{4}$



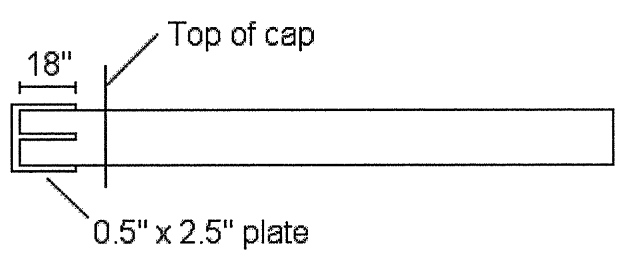
$$L := \frac{P_u}{F\cdot \text{throat}} \qquad L = 1.503\,\text{in}$$

Use a Plate 0.5" x 2.5"

$$\sigma_{max} := \frac{P_u}{0.5\text{in}\cdot 2.5\text{in}} \qquad \sigma_{max} = 6.675\,\frac{\text{kip}}{\text{in}^2}$$

$PLATE_{CHECK}$ = "PASS"

$$PLATE_{CHECK} := \left| \begin{array}{l} p \leftarrow \text{"PASS"} \quad \text{if } \sigma_{max} < 36\cdot\dfrac{\text{kip}}{\text{in}^2} \\ p \leftarrow \text{"FAIL"} \quad \text{otherwise} \\ \text{return } p \end{array} \right.$$



Pile Check

Pile checks, and liquefaction checks for all 4 bents, have already been completed by Kyle. See his job folder.

without Soil - max displacement

```
                         W     W  III  N   N
                         W     W   I   NN  N
                         W  W  W   I   N N N
                          W W W    I   N  NN
                           W W    III  N   N

 SSSSSSS   EEEEEEEEE  IIIIIIIII   SSSSSSS    AAAAAAA   BBBBBBBB
SSSSSSSSS  EEEEEEEEE  IIIIIIIII  SSSSSSSSS  AAAAAAAAA  BBBBBBBBB
SS     SS  EE            III     SS     SS  AA     AA  BB     BB
SSS        EE            III     SSS        AA     AA  BB    BBB
 SSSSSS    EEEEE         III      SSSSSS    AAAAAAAAA  BBBBBBBB
  SSSSSS   EEEEE         III       SSSSSS   AAAAAAAAA  BBBBBBBB
      SSS  EE            III           SSS  AA     AA  BB    BBB
SS     SS  EE            III     SS     SS  AA     AA  BB     BB
SSSSSSSSS  EEEEEEEEE  IIIIIIIII  SSSSSSSSS  AA     AA  BBBBBBBBB
 SSSSSSS   EEEEEEEEE  IIIIIIIII   SSSSSSS   AA     AA  BBBBBBBB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*                         WinSeisab                             *
*                                                               *
*                 Seismic Analysis of Bridges                   *
*                                                               *
*       Version 5.0.7                     Release 10/2003       *
*                                                               *
*                                                               *
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*                  Written By:  Roy Imbsen                      *
*                               Jon Lea                         *
*                               Clark Verkler                   *
*                               James Gates                     *
*                                                               *
*****************************************************************

Date:  18-AUG-06                                 Time:  12:24:15
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Mud Slough Ditch

UNIFORM LOAD METHOD RESULTS

VIBRATION CHARACTERISTICS

MODE	PERIOD	CS
1	0.305	0.56
2	0.298	0.56

Mud Slough Ditch

UNIFORM LOAD METHOD RESULTS

ABUTMENT PSEUDO DISPLACEMENTS

ITEM	LC	....LEFT FACE.... LNGTUDNL	TRNSVRSE	....RGHT FACE.... LNGTUDNL	TRNSVRSE	...OPNNG/CLSNG... LNGTUDNL	TRNSVRSE
ABU 1	1	0.043	0.000	0.043	0.000	0.000	0.000
	2	0.000	0.041	0.000	0.041	0.000	0.000
	3	0.043	0.012	0.043	0.012	0.000	0.000
	4	0.013	0.041	0.013	0.041	0.000	0.000
ABU 4	1	0.043	0.000	0.043	0.000	0.000	0.000
	2	0.000	0.041	0.000	0.041	0.000	0.000
	3	0.043	0.012	0.043	0.012	0.000	0.000
	4	0.013	0.041	0.013	0.041	0.000	0.000

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

- - - - WinSEISAB - - - -   (Version 5.0.7)   18-AUG-06
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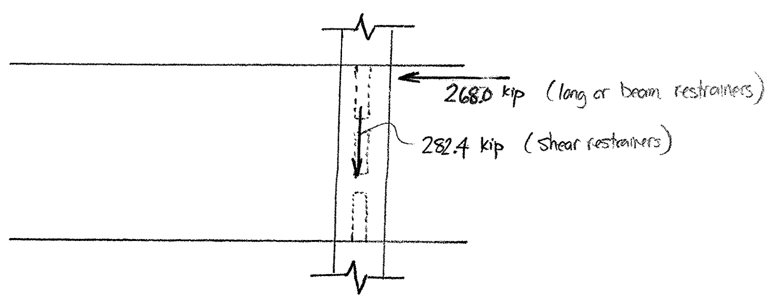
Mud Slough Ditch

UNIFORM LOAD METHOD RESULTS

ABUTMENT PSEUDO FORCES

ITEM	LC	VERT SHEAR	W/R TO BRIDGE C.L. LONGITUDNL	W/R TO BRIDGE C.L. TRANSVERSE	W/R TO ITEM C.L. NORMAL	W/R TO ITEM C.L. PARALLEL
ABU 1	1	20.6	149.2	0.0	149.2	0.0
	2	0.0	0.0	142.0	0.0	142.0
	3	20.6	149.2	42.6	149.2	42.6
	4	6.2	44.8	142.0	44.8	142.0
ABU 4	1	20.6	149.2	0.0	149.2	0.0
	2	0.0	0.0	142.0	0.0	142.0
	3	20.6	149.2	42.6	149.2	42.6
	4	6.2	44.8	142.0	44.8	142.0



INTEGRAL ABUTMENT - USE BENT FORCES FOR RESTRAINERS

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

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Mud Slough Ditch

UNIFORM LOAD METHOD RESULTS

ABUTMENT FOUNDATION SPRING PSEUDO FORCES

ABUT	LC	KF1F1	KF2F2	KF3F3	KM1M1	KM2M2	KM3M3
1	1	149.2	20.6	0.0	0.0	0.0	1.6
	2	0.0	0.0	142.0	18.5	7.7	0.0
	3	149.2	20.6	42.6	5.5	2.3	1.6
	4	44.8	6.2	142.0	18.5	7.7	0.5
4	1	149.2	20.6	0.0	0.0	0.0	1.6
	2	0.0	0.0	142.0	18.5	7.7	0.0
	3	149.2	20.6	42.6	5.5	2.3	1.6
	4	44.8	6.2	142.0	18.5	7.7	0.5

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

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Mud Slough Ditch

UNIFORM LOAD METHOD RESULTS

COLUMN PSEUDO FORCES

CL	LOC	LC	....LNGITUDNL....		....TRANSVRSE....				
			SHEAR	MOMENT	SHEAR	MOMENT	AXIAL	TORSION	
BNT 2									
1	BOT	1	14.9	152.	0.0	0.	0.0	0.0	
		2	0.0	0.	15.2	152.	27.6	0.0	
		3	14.9	152.	4.6	46.	8.3	0.0	
		4	4.5	46.	15.2	152.	27.6	0.0	
1	TOP	1	14.9	141.	0.0	0.	0.0	0.0	
		2	0.0	0.	15.2	148.	27.6	0.0	STRAP DESIGN
		3	14.9	141.	4.6	44.	8.3	0.0	
		4	4.5	43.	15.2	148.	27.6	0.0	
2	BOT	1	14.9	152.	0.0	0.	0.4	0.0	
		2	0.0	0.	15.8	156.	6.0	0.0	
		3	14.9	152.	4.7	47.	2.2	0.0	
		4	4.5	46.	15.8	156.	6.1	0.0	
2	TOP	1	14.9	141.	0.0	0.	0.4	0.0	
		2	0.0	0.	15.8	155.	6.0	0.0	
		3	14.9	141.	4.7	47.	2.2	0.0	
		4	4.5	43.	15.8	155.	6.1	0.0	
3	BOT	1	14.9	152.	0.0	0.	0.7	0.0	
		2	0.0	0.	15.8	157.	2.0	0.0	
		3	14.9	152.	4.8	47.	1.3	0.0	
		4	4.5	46.	15.8	157.	2.2	0.0	
3	TOP	1	14.9	141.	0.0	0.	0.7	0.0	
		2	0.0	0.	15.8	155.	2.0	0.0	
		3	14.9	141.	4.8	47.	1.3	0.0	
		4	4.5	42.	15.8	155.	2.2	0.0	
4	BOT	1	14.9	152.	0.1	1.	5.8	0.0	
		2	0.0	0.	15.8	156.	0.8	0.0	
		3	14.9	152.	4.8	48.	6.1	0.0	
		4	4.5	46.	15.8	157.	2.6	0.0	

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

Mud Slough Ditch

UNIFORM LOAD METHOD RESULTS

BENT PSEUDO FORCES

ITEM	LC	VERT SHEAR	W/R TO BRIDGE C.L. LONGITUDNL	W/R TO BRIDGE C.L. TRANSVERSE	W/R TO ITEM C.L. NORMAL	W/R TO ITEM C.L. PARALLEL
BNT 2	1	31.9	134.0	0.0	134.0	0.0
	2	0.0	0.0	141.2	0.0	141.2
	3	31.9	134.0	42.4	134.0	42.4
	4	9.6	40.2	141.2	40.2	141.2
BNT 3	1	31.9	134.0	0.0	134.0	0.0
	2	0.0	0.0	141.2	0.0	141.2
	3	31.9	134.0	42.4	134.0	42.4
	4	9.6	40.2	141.2	40.2	141.2

SHEAR RESTRAINER DESIGN CONTROL (282.4 kip)

LONGITUDINAL RESTRAINER DESIGN CONTROL (268.0 kip)

USE THESE FORCES DOUBLED

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

With Soil - max force

```
                         W     W  III  N    N
                         W     W   I   NN   N
                         W  W  W   I   N N  N
                          W W W    I   N   NN
                           W W    III  N    N

 SSSSSSS   EEEEEEEEE  IIIIIIIII   SSSSSSS    AAAAAAA   BBBBBBBB
SSSSSSSSS  EEEEEEEEE  IIIIIIIII  SSSSSSSSS  AAAAAAAAA  BBBBBBBBB
SS     SS  EE            III     SS     SS  AA     AA  BB     BB
SSS        EE            III     SSS        AA     AA  BB    BBB
 SSSSSS    EEEEE         III      SSSSSS    AAAAAAAAA  BBBBBBBB
  SSSSSS   EEEEE         III       SSSSSS   AAAAAAAAA  BBBBBBBB
      SSS  EE            III          SSS   AA     AA  BB    BBB
SS     SS  EE            III     SS     SS  AA     AA  BB     BB
SSSSSSSSS  EEEEEEEEE  IIIIIIIII  SSSSSSSSS  AA     AA  BBBBBBBBB
 SSSSSSS   EEEEEEEEE  IIIIIIIII   SSSSSSS   AA     AA  BBBBBBBB


*****************************************************************
*                                                               *
*                          WinSeisab                            *
*                                                               *
*                 Seismic Analysis of Bridges                   *
*                                                               *
*      Version 5.0.7                      Release 10/2003       *
*                                                               *
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*                                                               *
*                                                               *
*                                                               *
*                  Written By:  Roy Imbsen                      *
*                               Jon Lea                         *
*                               Clark Verkler                   *
*                               James Gates                     *
*                                                               *
*****************************************************************

Date:  18-AUG-06                                 Time:  12:03:23
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Mud Slough Ditch

UNIFORM LOAD METHOD RESULTS

VIBRATION CHARACTERISTICS

MODE	PERIOD	CS
1	0.086	0.56
2	0.226	0.56

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Mud Slough Ditch

UNIFORM LOAD METHOD RESULTS

ABUTMENT PSEUDO DISPLACEMENTS

ITEM	LC	....LEFT FACE.... LNGTUDNL	TRNSVRSE	....RGHT FACE.... LNGTUDNL	TRNSVRSE	...OPNNG/CLSNG... LNGTUDNL	TRNSVRSE
ABU 1	1	0.003	0.000	0.003	0.000	0.000	0.000
	2	0.000	0.023	0.000	0.023	0.000	0.000
	3	0.003	0.007	0.003	0.007	0.000	0.000
	4	0.001	0.023	0.001	0.023	0.000	0.000
ABU 4	1	0.003	0.000	0.003	0.000	0.000	0.000
	2	0.000	0.023	0.000	0.023	0.000	0.000
	3	0.003	0.007	0.003	0.007	0.000	0.000
	4	0.001	0.023	0.001	0.023	0.000	0.000

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

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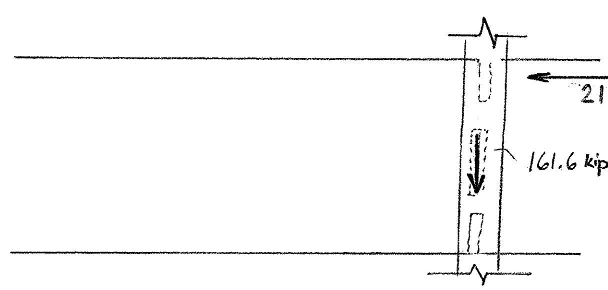
Mud Slough Ditch

UNIFORM LOAD METHOD RESULTS

ABUTMENT PSEUDO FORCES

			W/R TO BRIDGE C.L.		W/R TO ITEM C.L.	
ITEM	LC	VERT SHEAR	LONGITUDNL	TRANSVERSE	NORMAL	PARALLEL
ABU 1	1	1.6	272.7	0.0	272.7	0.0
	2	0.0	0.0	202.4	0.0	202.4
	3	1.6	272.7	60.7	272.7	60.7
	4	0.5	81.8	202.4	81.8	202.4
ABU 4	1	1.6	272.7	0.0	272.7	0.0
	2	0.0	0.0	202.4	0.0	202.4
	3	1.6	272.7	60.7	272.7	60.7
	4	0.5	81.8	202.4	81.8	202.4

The same since no skew.



USE BENT FORCES
INTEGRAL ABUTMENTS

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

Mud Slough Ditch

UNIFORM LOAD METHOD RESULTS

ABUTMENT FOUNDATION SPRING PSEUDO FORCES

ABUT	LC	KF1F1	KF2F2	KF3F3	KM1M1	KM2M2	KM3M3
1	1	272.7	1.6	0.0	0.0	0.0	0.1
	2	0.0	0.0	202.4	10.6	13.6	0.0
	3	272.7	1.6	60.7	3.2	4.1	0.1
	4	81.8	0.5	202.4	10.6	13.6	0.0
4	1	272.7	1.6	0.0	0.0	0.0	0.1
	2	0.0	0.0	202.4	10.6	13.6	0.0
	3	272.7	1.6	60.7	3.2	4.1	0.1
	4	81.8	0.5	202.4	10.6	13.6	0.0

same as previous page

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

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Mud Slough Ditch

UNIFORM LOAD METHOD RESULTS

COLUMN PSEUDO FORCES

CL	LOC	LC	LNGITUDNL SHEAR	LNGITUDNL MOMENT	TRANSVRSE SHEAR	TRANSVRSE MOMENT	AXIAL	TORSION
BNT 2								
1	BOT	1	1.2	12.	0.0	0.	0.0	0.0
		2	0.1	1.	8.7	87.	15.8	0.0
		3	1.2	12.	2.6	26.	4.7	0.0
		4	0.4	4.	8.7	87.	15.8	0.0
1	TOP	1	1.2	11.	0.0	0.	0.0	0.0
		2	0.1	1.	8.7	85.	15.8	0.0
		3	1.2	11.	2.6	25.	4.7	0.0
		4	0.4	4.	8.7	85.	15.8	0.0
2	BOT	1	1.2	12.	0.0	0.	0.0	0.0
		2	0.0	0.	9.0	89.	3.4	0.0
		3	1.2	12.	2.7	27.	1.1	0.0
		4	0.4	4.	9.0	89.	3.4	0.0
2	TOP	1	1.2	11.	0.0	0.	0.0	0.0
		2	0.0	0.	9.0	89.	3.4	0.0
		3	1.2	11.	2.7	27.	1.1	0.0
		4	0.4	4.	9.0	89.	3.4	0.0
3	BOT	1	1.2	12.	0.0	0.	0.1	0.0
		2	0.0	0.	9.1	90.	1.2	0.0
		3	1.2	12.	2.7	27.	0.4	0.0
		4	0.4	4.	9.1	90.	1.2	0.0
3	TOP	1	1.2	11.	0.0	0.	0.1	0.0
		2	0.0	0.	9.1	89.	1.2	0.0
		3	1.2	11.	2.7	27.	0.4	0.0
		4	0.4	4.	9.1	89.	1.2	0.0
4	BOT	1	1.2	12.	0.0	0.	0.5	0.0
		2	0.0	0.	9.0	89.	0.5	0.0
		3	1.2	12.	2.7	27.	0.6	0.0
		4	0.4	4.	9.1	89.	0.6	0.0

STRAP DESIGN

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

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Mud Slough Ditch

UNIFORM LOAD METHOD RESULTS

BENT PSEUDO FORCES

ITEM	LC	VERT SHEAR	W/R TO BRIDGE C.L. LONGITUDNL	W/R TO BRIDGE C.L. TRANSVERSE	W/R TO ITEM C.L. NORMAL	W/R TO ITEM C.L. PARALLEL
BNT 2	1	2.5	10.5	0.0	10.5	0.0
	2	0.0	0.0	80.8	0.0	80.8
	3	2.5	10.5	24.2	10.5	24.2
	4	0.7	3.1	80.8	3.1	80.8
BNT 3	1	2.5	10.5	0.0	10.5	0.0
	2	0.0	0.0	80.8	0.0	80.8
	3	2.5	10.5	24.2	10.5	24.2
	4	0.7	3.1	80.8	3.1	80.8

SHEAR RESTRAINER DESIGN CONTROL

LONGITUDINAL RESTRAINER DESIGN CONTROL

USE THESE FORCES
FOR RESTRAINERS

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

Liquefaction of Soils at Big Slough Ditch under current Provisions

The water level shown on the boring log sheet is at a depth of 27.8'. Another boring log in the area, when the surface elevation is matched up, has water at a depth of 26.7'. Since there is a slight change of soil strength at 25', I will add the water table in at 25'.

According to FHWA October 1996 book, "Seismic Design of Bridges - Design Example No. 3" p. A7, the max depth below ground surface for which liquefaction can occur is 50'. From FHWA SA-97-076 Chapter 8 pg 111, if N>30, liquefaction will not occur. But for this bridge, all the layers will be checked.

$A := 0.225$ Acceleration at ground surface (in seismic zone 3)

$M := 6.75$ EarthQuake Magnitude from 1975 AASHTO Interim Specifications Bridges p. 12
The M=6 maximum acceleration is 0.22, so A=0.225 puts us into M=6.5.
The chart for correcting N values has 6 and 6.75. Use 6.75.

Layer 1 (0' to 4.5')

Moist, Medium Dense, Gray Sand and Gravel

$N_1 := 10$ $\gamma_1 := 110 \cdot \frac{lb}{ft^3}$ $\phi := 30$

$\sigma_1 := \gamma_1 \cdot 4.5ft$ $\sigma_1 = 495 \frac{lb}{ft^2}$

Above water table. Therefore not in danger of liquefaction.

Layer 2 (4.5' to 15.5')

Moist, Medium Stiff to Stiff, Gray and Brown Clay with Sand Seams

$N_2 := 9$ $\gamma_2 := 120 \cdot \frac{lb}{ft^3}$

$\sigma_2 := \gamma_2 \cdot 11ft$ $\sigma_2 = 1.32 \times 10^3 \frac{lb}{ft^2}$

Above water table. Therefore not in danger of liquefaction.

Layer 3 (15.5' to 25')

Wet, Medium Dense, Gray Sand

$N_3 := 16 \qquad \gamma_3 := 116\dfrac{lb}{ft^3}$

$\sigma_3 := \gamma_3 \cdot 9.5ft \qquad \sigma_3 = 1.102 \times 10^3 \dfrac{lb}{ft^2}$

Above water table as reported in soil borings. The water level is conservatively raised up 2.8 feet for this analysis, but it is possible that seasonal variations would vary the water level more than this. However, assume this layer is <u>not in danger</u> of liquefaction.

Layer 4 (25' to 35') - Beginning of Water Table

Wet, Medium Dense, Gray Sand

$N_4 := 18 \qquad \gamma_4 := 60\dfrac{lb}{ft^3} \qquad t := 10ft$ thickness of layer

$\sigma_4 := \gamma_4 \cdot t \qquad \sigma_4 = 600\dfrac{lb}{ft^2} \qquad z := 30ft$ depth to middle of layer

$\sigma_o := \sigma_1 + \sigma_2 + \sigma_3 + \dfrac{\sigma_4}{2} \qquad \sigma_o = 3.217 \times 10^3 \dfrac{lb}{ft^2}$ overburden weight above middle of layer

$r_d := 1 - \dfrac{0.015z}{3.281ft} \qquad r_d = 0.863$

$\tau_{av} := 0.65 \cdot A \cdot \sigma_o \cdot r_d \qquad \tau_{av} = 405.958\dfrac{lb}{ft^2}$

Stress Ratio: $ratio_\sigma := \dfrac{\tau_{av}}{\sigma_o} \qquad ratio_\sigma = 0.126$

Liquefaction Strength

$\sigma_o = 1.609 \frac{ton}{ft^2}$ Use this number on Figure 8 (Seismic Design of Highway Bridge Foundations, vol 2, p. 31) to find Cn. $C_n := 0.79$

Corrected Blowcount: $N_{4c} := C_n \cdot N_4$ $N_{4c} = 14.22$

Use the corrected blowcount to find the resisting stress ratio on Figure 7 (Seismic Design of Highway Bridge Foundations, vol 2, p. 29).

$ratio_r = \frac{\tau}{\sigma_v}$ $ratio_r := 0.175$ Resisting Stress

$FS_4 := \frac{ratio_r}{ratio_\sigma}$ $FS_4 = 1.387$ Factor of Safety

$$SAFE := \left| \begin{array}{l} s \leftarrow \text{"YES"} \text{ if } FS_4 > 1 \\ s \leftarrow \text{"NO"} \text{ otherwise} \\ \text{return } s \end{array} \right.$$

$SAFE = \text{"YES"}$

Layer 5 (35' to 55')

Wet, Medium Dense, Gray Sand

$N_5 := 27$ $\gamma_5 := 67 \frac{lb}{ft^3}$ $t := 20ft$ thickness of layer

$\sigma_5 := \gamma_5 \cdot t$ $\sigma_5 = 1.34 \times 10^3 \frac{lb}{ft^2}$ $z := 45ft$ depth to middle of layer

$\sigma_o := \sigma_1 + \sigma_2 + \sigma_3 + \sigma_4 + \frac{\sigma_5}{2}$ $\sigma_o = 4.187 \times 10^3 \frac{lb}{ft^2}$ overburden weight above middle of layer

$r_d := 1 - \frac{0.015z}{3.281ft}$ $r_d = 0.794$

$\tau_{av} := 0.65 \cdot A \cdot \sigma_o \cdot r_d$ $\tau_{av} = 486.37 \frac{lb}{ft^2}$

Stress Ratio: $ratio_\sigma := \frac{\tau_{av}}{\sigma_o}$ $ratio_\sigma = 0.116$

Liquefaction Strength

$\sigma_o = 2.094 \dfrac{ton}{ft^2}$ Use this number on Figure 8 (Seismic Design of Highway Bridge Foundations, vol 2, p. 31) to find Cn. $C_n := 0.68$

Corrected Blowcount: $N_{5c} := C_n \cdot N_5$ $N_{5c} = 18.36$

Use the corrected blowcount to find the resisting stress ratio on Figure 7 (Seismic Design of Highway Bridge Foundations, vol 2, p. 29).

$ratio_r = \dfrac{\tau}{\sigma_v}$ $ratio_r := 0.22$ Resisting Stress

$FS_5 := \dfrac{ratio_r}{ratio_\sigma}$ $FS_5 = 1.894$ Factor of Safety

$$SAFE := \left| \begin{array}{l} s \leftarrow \text{"YES"} \quad \text{if } FS_4 > 1 \\ s \leftarrow \text{"NO"} \quad \text{otherwise} \\ \text{return } s \end{array} \right.$$

$SAFE = \text{"YES"}$

<u>Layer 6 (55' to 90')</u>

Wet, Dense, Gray Sand

$N_6 := 44$ $\gamma_6 := 74 \dfrac{lb}{ft^3}$ $t := 35ft$ thickness of layer

$\sigma_6 := \gamma_6 \cdot t$ $\sigma_6 = 2.59 \times 10^3 \dfrac{lb}{ft^2}$ $z := 72.5ft$ depth to middle of layer

$\sigma_o := \sigma_1 + \sigma_2 + \sigma_3 + \sigma_4 + \sigma_5 + \dfrac{\sigma_6}{2}$ $\sigma_o = 6.152 \times 10^3 \dfrac{lb}{ft^2}$ overburden weight above middle of layer

$r_d := 1 - \dfrac{0.015z}{3.281ft}$ $r_d = 0.669$

$\tau_{av} := 0.65 \cdot A \cdot \sigma_o \cdot r_d$ $\qquad \tau_{av} = 601.511 \dfrac{lb}{ft^2}$

Stress Ratio: $\quad ratio_\sigma := \dfrac{\tau_{av}}{\sigma_o} \qquad ratio_\sigma = 0.098$

Liquefaction Strength

$\sigma_o = 3.076 \dfrac{ton}{ft^2}$ Use this number on Figure 8 (Seismic Design of Highway Bridge Foundations, vol 2, p. 31) to find Cn. $\quad C_n := 0.53$

Corrected Blowcount: $\quad N_{6c} := C_n \cdot N_6 \qquad N_{6c} = 23.32$

Use the corrected blowcount to find the resisting stress ratio on Figure 7 (Seismic Design of Highway Bridge Foundations, vol 2, p. 29).

$ratio_r = \dfrac{\tau}{\sigma_v} \qquad ratio_r := 0.28$ Resisting Stress

$FS_6 := \dfrac{ratio_r}{ratio_\sigma} \qquad FS_6 = 2.864$ Factor of Safety

$$SAFE := \left| \begin{array}{l} s \leftarrow \text{"YES"} \;\; \text{if } FS_4 > 1 \\ s \leftarrow \text{"NO"} \;\; \text{otherwise} \\ \text{return } s \end{array} \right. \qquad SAFE = \text{"YES"}$$

<u>Layer 7 (90' to 101.5')</u>

Moist, Medium Stiff, Gray Clay with Organic Matter

$N_7 := 6 \qquad \gamma_7 := 58 \dfrac{lb}{ft^3} \qquad t := 11.5ft$ thickness of layer

$\sigma_7 := \gamma_7 \cdot 11.5ft \qquad \sigma_7 = 667 \dfrac{lb}{ft^2} \qquad z := 95.75ft$ depth to middle of layer

$\sigma_o := \sigma_1 + \sigma_2 + \sigma_3 + \sigma_4 + \sigma_5 + \sigma_6 + \dfrac{\sigma_7}{2} \qquad \sigma_o = 7.78 \times 10^3 \dfrac{lb}{ft^2}$ overburden weight above middle of layer

$$r_d := 1 - \frac{0.015z}{3.281ft} \qquad r_d = 0.562$$

$$\tau_{av} := 0.65 \cdot A \cdot \sigma_o \cdot r_d \qquad \tau_{av} = 639.786 \frac{lb}{ft^2}$$

Stress Ratio: $ratio_\sigma := \frac{\tau_{av}}{\sigma_o}$ $ratio_\sigma = 0.082$

Liquefaction Strength

$\sigma_o = 3.89 \frac{ton}{ft^2}$ Use this number on Figure 8 (Seismic Design of Highway Bridge Foundations, vol 2, p. 31) to find Cn. $C_n := 0.45$

Corrected Blowcount: $N_{7c} := C_n \cdot N_7 + 7.5$ $N_{7c} = 10.2$

Use the corrected blowcount to find the resisting stress ratio on Figure 7 (Seismic Design of Highway Bridge Foundations, vol 2, p. 29).

$ratio_r = \frac{\tau}{\sigma_v}$ $ratio_r := 0.122$ Resisting Stress

$FS_7 := \frac{ratio_r}{ratio_\sigma}$ $FS_7 = 1.484$ Factor of Safety

$$SAFE := \left| \begin{array}{l} s \leftarrow \text{"YES"} \quad \text{if } FS_4 > 1 \\ s \leftarrow \text{"NO"} \quad \text{otherwise} \\ \text{return } s \end{array} \right.$$

$SAFE = \text{"YES"}$

None of the soil layers beneath this bridge will liquify under an extreme earthquake. Soil foundation is good.