Progress Report

March 31, 2007

PROPOSAL TO THE FEDERAL HIGHWAY ADMINISTRATION

TASK ORDER DTFH61-06-T-70006

FOR THE DEVELOPMENT OF
GUIDE SPECIFICATIONS FOR BRIDGES VULNERABLE TO COASTAL STORMS
AND
HANDBOOK OF RETROFIT OPTIONS FOR BRIDGES VULNERABLE TO COASTAL STORMS

by

Modjeski and Masters, Inc.

with

Moffatt and Nichol, Inc.
Ocean Engineering Associates, Inc.
D'Appolonia, Inc.
Dr. Dennis R. Mertz

INTRODUCTION

We received Notice to Proceed on this Work Order on August 14, 2006.

This report covers work done in March, 2007. During this month the team held two conference calls (March 19 and March 30) and Dr. Kulicki had several phone conversation with Mr. Shelden and Dr. Sheppard to iron out the wording of the draft specifications. The minutes of the March 19th and March 30th conference calls are attached as Attachments A and B.

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TASK 1 – MEETINGS

No new meetings with the BWTF took place in this reporting period. A meeting between some members of the research team and the members of the BWTF who have ocean/hydraulics interests is scheduled for April 19th in Baltimore. A meeting with the full Task Force is scheduled for June 12 and 13th in Raleigh, North Carolina.

TASK 2 – REVIEW, SUMMARIZE, AND AUGMENT LITERATURE

We received some comments on the literature search submitted with our February progress report. We reviewed the comments and prepared a response in the form of a letter to the Chairman of the BWTF. A copy of the letter is attached as Attachment C. Some of the comments also requested detailed samples of force calculations. Attachment D of this report includes samples of the calculations used to determine the forces on and capacities of different structural components.

If our response is satisfactory, **we are asking for permission to submit an invoice for Task 2.**

TASK 3 – REVIEW AND SUPPLEMENT ONGOING FORCE STUDIES

Dr. Shepard continued his work on testing the bridge model in the lab with and without overhang. Results of this test will be presented during the April 19th meeting.

Our team received some comments on the submissions related to Task 3 in our February progress report. The following requests were included in these comments:

- The reviewers requested that a description of different wave force calculation methods be provided, including the equations used by each method. This information is being assembled and will be ready for the April meeting.
- The reviewers also requested that the results of the lab test be compared to different calculation methods. These comparisons are forthcoming. Some of the comparison results have been conducted and others are underway. These results will be ready for presentation in the April 19th meeting.
- We have reviewed preliminary comparisons between lab measurements and

calculations and are resolving questions and verifying that data used consistent assumptions and structure and wave parameters.

TASK 4 – COMPILE AND CATALOG RETROFIT OPTIONS

We did not receive any comments on the submission we made with our February progress report. We are assuming that no comments will be received and **we are asking for permission to submit an invoice for Task 4.**

TASK 5 – PERFORM ANALYTICAL STUDY OF RETROFIT OPTIONS

No progress to-date.

TASK 6 – DEVELOP A GUIDE SPECIFICATION AND A RETROFIT HANDBOOK FOR ADOPTION BY AASHTO

TASK 6A - GUIDE SPECIFICATION

We continue to expand and refine the draft of the specifications. Most areas have been sufficiently developed for the 50% submission with the exception of the articles containing the wave force calculation equations for superstructures. The latter articles will be developed once a method of analysis is selected. We are working toward a formal submission on or before April 30th as scheduled. However, we may make an earlier preliminary submission to the members of the BWTF who will attend the April 19th meeting in Baltimore. The purpose of the preliminary submission will not be to solicit comments from the BWTF. Rather, the purpose will be to show the general direction of this task.

TASK 6B - RETROFIT HANDBOOK

Work on updating the outline that was submitted earlier continued. Work on the development of the body of the manual started in late March.

TASK 7 – DEVELOP FINAL REPORT AND RECOMMENDATIONS FOR FURTHER STUDIES

No progress

TASK 8 – PREPARE EXECUTIVE SUMMARY AND PRESENTATION MATERIALS

No progress

FUTURE WORK – NEXT MONTH

1. Formulate a recommendation on wave force calculation process.

2. Meet with a group of members of the BWTF in Baltimore on April 19th to review the work on selecting a calculation method.

3. Continue working on the strawman for design specifications

4. Continue to research the reliability and recurrence issues.

SCHEDULE

The schedule previously agreed to is shown below as “Proposed Completion Dates”.

Task 2 – We are asking that our submission for Task 2 supplemented by our response to the BWTF comments be considered as fulfilling the deliverables for Task 2. Further refinements will be done as part of our final deliverables for the project.

Task -3 – The comparative studies of four wave force prediction methods was completed and appended to earlier progress reports. Comparisons to lab test results are forthcoming. The decision as to which method to recommend and development of any possible design aids will likely be made during the April 19th meeting subject to the full BWTF’s approval. We expect that to be done by late April.

At the moment no other dates are in jeopardy. See attached schedule.

EFFORT EXPENDED

Tables showing hours expended per task are shown below. The tables indicate that most team members have already exceeded the hours allotted to the early tasks of the project.

SCHEDULE

TASK	Date shown in Work Plan	PROPOSED COMPLETION DATES
Notice to Proceed	September 1, 2006	
Kickoff Meeting	December 4,5,6, 2006	
Task 2	December 15, 2006	January 15, 2007
Task 3	December 15, 2006	February 28, 2007
Task 4	January 26, 2007	March 31, 2007
Task 5	March 2, 2007	April 30, 2007
Task 6 50% Draft Specification and Manual 90% Draft Specification and Manual 100% Draft Specification and Manual Interim Report Tasks 2 to 6	 February 15, 2007 May 31, 2007 August 15, 2007 July 15, 2007	 May 15, 2007 * July 31, 2007 October 15, 2007 September 15, 2007
Task 7 Draft Final	 June 30, 2007 September 15, 2007	 August 31, 2007 November 15, 2007
Task 8 – Executive Summary Draft 4 to 6 page summary Final 4 to 6 page summary	 June 30, 2007 August 31, 2007	 August 31, 2007 October 31, 2007
Task 8 – 13 hour slides Draft Final	 November 30, 2007 January 31, 2008	 January 31, 2008 March 31, 2008

* Early draft will be provided before the April meeting to the BWTF members attending that meeting.

Total hours spent per task to-date

To-Date Work Hours by Task

Modjeski and Masters, Inc.

FHWA Project on
Development of Guide Specifications for Bridges Vulnerable to Coastal Storms
and
Handbook for Retrofit Options for Bridges vulnerable to Coastal Storms

Labor Costs

Category	Task 2	Task 3	Task 4	Task 5	Task 6	Task 7	task 8
John M. Kulicki	81	14.5	18		42		
Wagdy G. Wassef	21.5	7.5	47		9.5		
Tim Stuffle	132	16	233				
Tom Rogers	50.5						
Don Miller				1.5			
Jeff Forest				14			
Don Price	25						
Sherood Herb			96				
Subtotal	310	38	388	15.5	51.5	0	0

Moffatt & Nichol

FHWA Project on
Development of Guide Specifications for Bridges Vulnerable to Coastal Storms
and
Handbook for Retrofit Options for Bridges vulnerable to Coastal Storms

Labor Costs

Category	Task 2	Task 3	Task 4	Task 5	Task 6	Task 7	task 8
Mike Knott / John Headland	8						
Jeff Shelden	72.5	127			51.5		
Paul Tschirky	19.5	13					
Graphics / CAD / Admin							
Subtotal	100	140	0	0	51.5	0	0

OEA, Inc.

FHWA Project on
Development of Guide Specifications for Bridges Vulnerable to Coastal Storms
and
Handbook for Retrofit Options for Bridges vulnerable to Coastal Storms

Labor Costs

Category	Task 2	Task 3	Task 4	Task 5	Task 6	Task 7	task 8
D. M. Sheppard	8	169			15		
P. Dompe	44	211			2		
Subtotal	52	380	0	0	17	0	0

D’Appolonia, Inc.

FHWA Project on
Development of Guide Specifications for Bridges Vulnerable to Coastal Storms
and
Handbook for Retrofit Options for Bridges vulnerable to Coastal Storms

Labor Costs

Category	Task 2	Task 3	Task 4	Task 5	Task 6	Task 7	task 8
Jim Withiam			5				
Ed Voytko			4.5				
Colleen Campbell			20.75				
Subtotal	0	0	0	0	0	0	0

Attachment A

Conference Call Log

March 19th, 2007 Conference Call

Harrisburg, Pennsylvania
March 20, 2007

MEMORANDUM

TO: Modjeski and Masters, Inc.

RE: MINUTES FOR THE MARCH 19, 2007, TEAM TELEPHONE CONFERENCE CALL – DTFH61-06-T-70006

PN2560

A conference call was held on March 19, 2007. The following were in attendance:

Dr. Max Sheppard (OEA)
Mr. Philip Dompe (OEA)
Dr. Jim Withiam (D'Appolonia)
Mr. Jeff Shelden (M&N)
Dr. Dennis Mertz (University of Delaware)
Dr. John Kulicki (M&M)
Dr. Wagdy Wassef (M&M)

Following are the items of discussions and the decisions made.

DISCUSSIONS

- **Meeting with the Task Force**

  Dr. Kulicki informed the team that the task force is still looking into our request that our team meet with the full Task Force or, at least, the Task Force members with interests in waves and hydraulics. The team's consensus is that the face-to-face meeting is the best possible way to sort things out. However, if our request is rejected, we will try to have a conference call with the Task Force members with waves/hydraulics interests.

- **Laboratory Testing and Analysis Methods**
  - Dr. Sheppard is conducting additional testing on the specimen without railings or overhangs. The results will be used to verify the modified Kaplan equations. Afterwards, he will test the specimen with railings and overhangs.

- The team will compare the test results with Wallingford and with the Modified Kaplan methods. MN and OEA will produce the Wallingford and modified Kaplan comparisons, respectively.
- Development of the non-dimensional plots may start once the test on the specimen without railings or overhangs is completed and the accuracy of the equations has been verified. The development of the plots will concentrate on the case of bridges with railing and overhangs. This sequence would probably allow the development of the plots before the specimen with these components is lab-tested. The possibility of producing "proof of concept" sample plots to share with the Task force based on the testing without railings or overhangs was generally endorsed..

- **Analysis Levels** Due to concerns regarding the availability of data based on the joint probability of different parameters, Drs. Sheppard and Mertz suggested basing Level I on the 100 year value for each parameter then adjust the load factor to reflect any refinements applied to the analysis based on opinions of a group of experts. For Level I, only the parameters that can be easily recognized and refined by non-ocean engineers will be included. For Level II, other parameters may be refined. In all cases, the refinement will result in a reduction factor to be applied to the load factor. The reduction factor will keep getting smaller as more refinements are applied.
  For Level III, numerical simulations will be required and the load factor for Level III will reflect that refined analyses are used.

  This approach will be discussed again after all parties digest the information.

- **Specifications Strawman** Mr. Shelden revised the Specifications Strawman and Dr. Sheppard suggested further revisions. Some articles were discussed and Dr. Kulicki suggested that some of the paragraph that do not include "specifications" be moved to the commentary column. Dr. Kulicki will make revisions to the current version of the specifications (the version that include MN and OEA's revisions) and redistribute to the team.

WAGDY G. WASSEF

Attachment B

Conference Call Log

March 30th, 2007 Conference Call

Harrisburg, Pennsylvania
April 2, 2007

MEMORANDUM

TO: Modjeski and Masters, Inc.

RE: MINUTES FOR THE MARCH 30, 2007, TEAM TELEPHONE CONFERENCE CALL – DTFH61-06-T-70006

PN2560

A conference call was held on March 19, 2007. The following were in attendance:

Dr. Max Sheppard (OEA)
Mr. Philip Dompe (OEA)
Dr. Jim Withiam (D'Appolonia)
Mr. Jeff Shelden (M&N)
Dr. John Kulicki (M&M)
Dr. Wagdy Wassef (M&M)

Following are the items of discussions and the decisions made.

DISCUSSIONS

- **Meeting with the Task Force**

  Dr. Kulicki informed the team that a group of the task force members will meet with our team in Baltimore on April 19th.

- **Response to Task Force Comments on our Team's February Progress Report**

  The Task Force comments were discussed. Dr. Kulicki will prepare a response for review by the team and, possibly, for enclosure with March Progress Report.

- **Laboratory Testing and Analysis Methods**
  - Dr. Sheppard is continuing with the lab testing of the specimen without railings and overhangs. When wave frequency is close to the natural

frequency of the specimen unanticipated effects show up in the results. Filtering these effects is being attempted.

- The ratio between the bridge width and the wave length appears to be a factor in determining the accuracy of the calculation methods. This ratio is changed in the lab test by changing the wave length in the wave tank.

- Based on observations made during testing, OEA developed a better estimate of Air entrapment.

- MN compared lab test results to Wallingford based on certain assumption regarding air entrapment. The comparisons will be updated to take advantage of the new estimate of air entrapment.

- OEA is comparing test results to Modified Kaplan. It appears that Modified Kaplan over-predicts quasi static forces and under-predicts the slamming force. It appears that the discrepancy is caused by using the drag and inertia coefficients that were based on earlier limited measurements. These coefficients need to be revisited.

- The lab test and the comparisons to the calculation methods should be complete on April 11 or 12. A recommendation on the method to be included in the specifications can be made by mid April and may be presented during the April 19th meeting.

- Some non-dimensional plots were developed as a proof-of-concept. More plots will be developed once a method of calculations is selected.

- **<u>Specifications Strawman</u>** Revisions to the Specifications Strawman were discussed. With the exception of the articles dealing with the force calculation method, most major articles have been developed and are being refined.

WAGDY G. WASSEF

Attachment C

Response to Task Force Comments
on
the February, 2007, Progress Report

MODJESKI and MASTERS

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ESHENAUR
WASSEF
STRAIN
NEWMAN
CLANCY
BORZOK
DILLMAN
JOHNS
MURPHY
EGENRIEDER

April 4, 2007

Mr. Greg R. Perfetti
State Bridge and Design Engineer
North Carolina Department of Transportation
1581 Mail Service Center
Raleigh, NC 27699-1581

RE: TASK ORDER DTFH61-06-T-70006 PN2560
Development of Guide Specification for Bridges
Vulnerability to Coastal Storms and Handbook of
Retrofit Options for Bridges Vulnerable to Coastal Storms
Response to Review Comments on February 2007 Progress Report

Dear Greg:

We are responding to comments received on our February Progress Report.

Many of the review comments related to the literature survey attached to the Progress Report. According to the approved Work Plan, "some of the literature reviews by others are soon to be completed or have been completed quite recently. It should not be necessary to update these data. We will review selected relevant published papers and reports". Therefore, when reviewing reports such as the report by Douglass, et al., which cited other studies, we did not feel it was necessary to list those studies as they were included in the Douglass report.

Our current plan is to excerpt part of the reviews in the literature survey in developing the chapter in our report dealing with the state of literature. We expect to organize this material under more functional titles such as literature dealing with:

- The general nature of coastal storms.
- The specifics of hurricanes Katrina, Rita and Ivan.
- The response of structures and bridge structures in particular.
- Past laboratory studies.
- Relevant methods of calculating wave forces.
- The reliability and probabilistic aspects needed to develop load factors.
- Etc.

Two of the reviewers asked if we had any conclusions or a sense of what we had learned from the literature survey. As a concise summary:

- We learned or had reinforced, that many other reviewers have looked at the general subject, have compiled extensive damage reports, have dealt with the uncertainties associated with wind, wave, surge, etc. in other applications, and have done experimental studies with measured results on structures but mostly on structures other than bridges.
- It reinforced the need for a modern well-thought laboratory experiment such as that being conducted at the University of Florida in support of work underway at the Florida Department of Transportation and in support of this project.
- We were left with the conclusion that several of the past studies on force had either less than direct correspondence to bridge-type geometry, or had very simplifying assumptions and that the Wallingford, Douglass and Kaplan methods were the most fruitful places to start.
- We found that some of the previous studies dealing with uncertainty had a reasonable correspondence to the work of this project and, as indicated on the memorandum on COV, attached to the Progress Report, have tried to expand and adapt that information.

We propose to defer to the final report writing for further refinement of the particular citations, and the development of the consistent level of coverage on the work that most heavily influences the present studies. For example, we have certainly utilized the information in some of the papers by Allsop and the Wallingford group mentioned by Dr. Kreibel. However, we found several of these reports to be highly repetitive, some contained errors, and some were probably largely superseded by the reports dealing with what we refer to the Wallingford Linear Method.

With respect to the suggestion that we drop references that are not archived and cannot be independently located by others, we would suggest that the PowerPoint presentations and similar material that fell under this category be retained for the time being under a separate heading as some of these will probably never be published. We would probably call this “unreviewed material”. Most of these PowerPoint presentations came from the Wave Workshop, put on by the FHWA, and represented a reasonable presentation of the State of the Art at that time.

With regard to the “Resistance Calculations of Failed Components”, some of the requested sample calculations have been appended to our March 2007 Progress Report.

Moffatt & Nichol (MN) and Ocean Engineering Associates, Inc. (OEA) have been preparing a concise review of the equations used for the Douglass Wallingford and Kaplan methods along with some sample calculations. These will be provided in the project report and an interim submission will be made as soon as these are available.

As to breaking the test cases into smaller subgroups, this will be done in the final report as well. Our purpose of submitting this time was to show that the work promised in the

December 4th and 5th Task Force Meeting was in fact being done and that the additional studies suggested at that time were essentially complete.

Dr. Kreibel states in his comments that "Of course the ultimate decision of the three methods requires comparison to data and it would be useful to start showing comparisons to lab data..." This work has been underway for the last several weeks and the team has reviewed some of the initial output. OEA and the students at UF continue to improve their understanding of how their data collection system works, and some of the laboratory work is being revised at this writing. We expect that on the April 19th meeting, we will have the information necessary to make a recommendation, present our reasons with comparisons to the lab data, and receive input from the hydraulic members of the task force.

With regard to the personal note on COVs in Dr. Kreibel's comments, we were very glad to receive his report as data from it further verified the relatively large COVs which the team has assembled and as discussed in the memorandum by MN attached to our Progress Report.

We trust that this is an adequate response. If you have additional questions or comments, please advise.

Very truly yours,

JOHN M. KULICKI, Ph.D., P.E.,
President/CEO and Chief Engineer

JMK:dml

Attachment D

Sample Calculations

MODJESKI AND MASTERS, INC.
ENGINEERS

Projects Coastal - waves
Subject Sample calculations

Made by IOS
Date
Checked by
Date
Sheet No. 1 of

1 BLOCK = 5MM

STEP 1 Calculate the Areas of beams, decks, rails, diaphragms, etc. (see sheets 1-2)

STEP 2 Calculate the total areas of unsubmerged concrete, submerged concrete, and air for cross sections with and without diaphragms. (see sheet 5)

STEP 3 Use known Areas, lengths, and unit weights to calculate volumes and loads in a spreadsheet. (see page 6)

Pages 7, 8, 9 show the derivation of Eq 1 and Eq 2. Eq 2 is used on page 5. Eq 1 was not used herein.

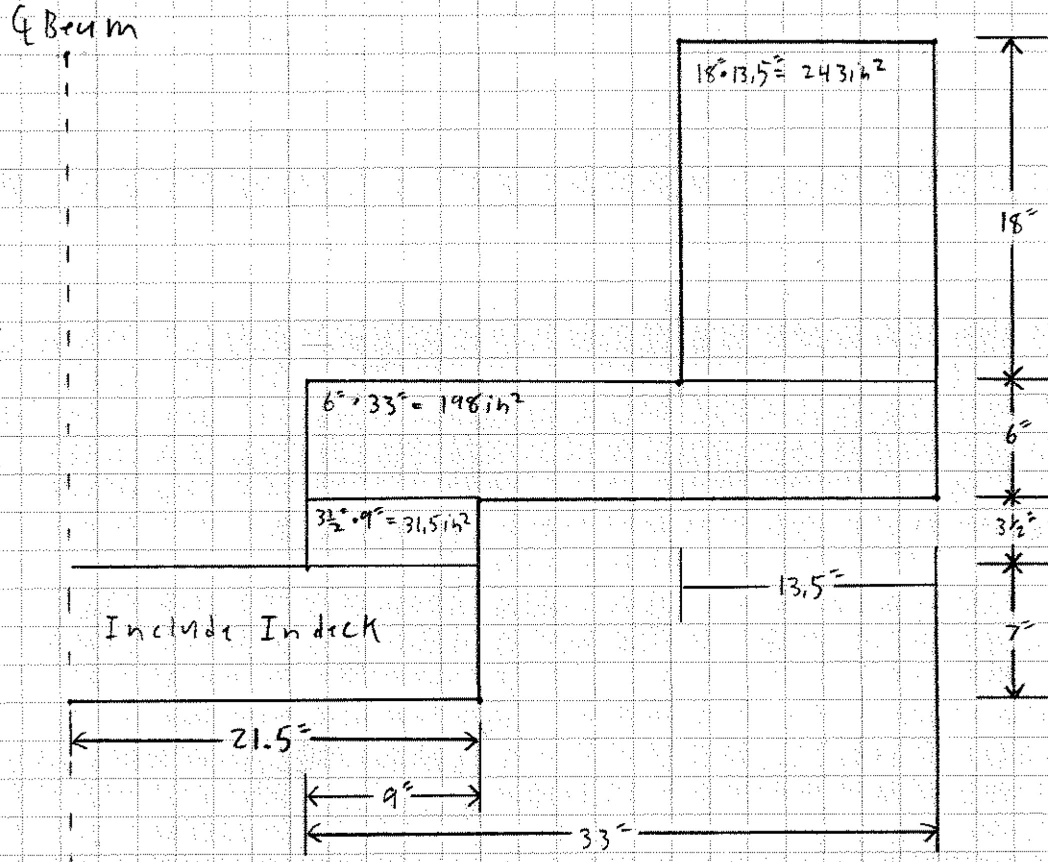
In the following calculations water was assumed to rise slowly to the level of the top of the Deck.

These calculations are based on static water, no wave or dynamic forces are included.

Cross Section Areas

• Deck thickness variable, assume 7" everywhere for boyancy calculations

railings

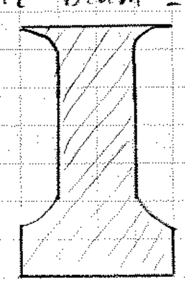


Area of one railing = 243 in² + 198 in² + 31.5 in² = 472.5 in²

Area of one railing = 472.5 in²

Beams

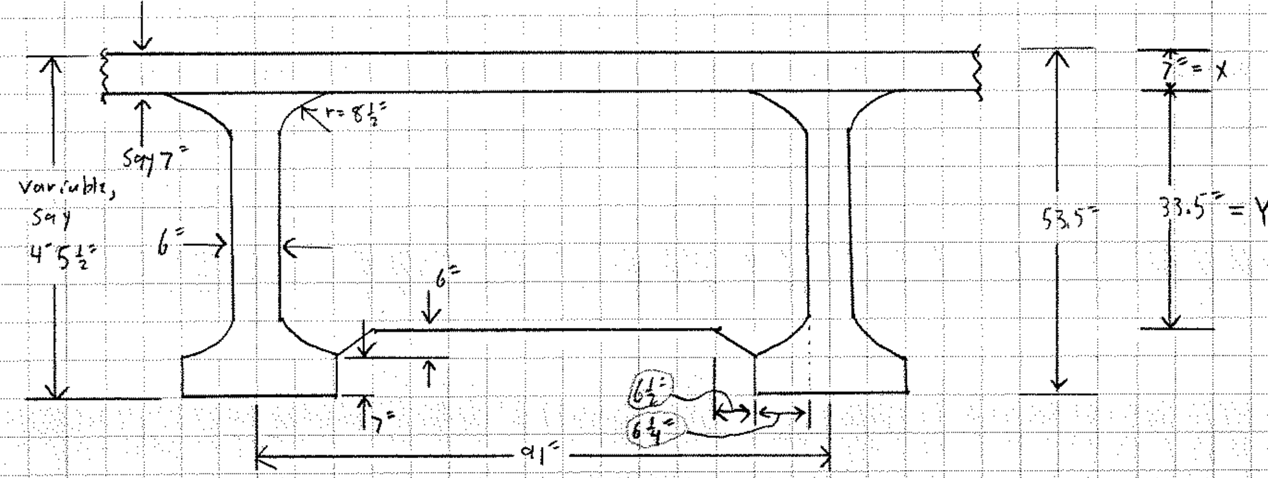
Area of one Beam = 465 in² (see structural calcs)



Area of beam does not include deck

Area of beam = 465 in²

Diaphragms / uncompressed air



• Area of uncompressed air

$= \left[(91" - 6") \cdot (53.5" - 7" - 7" - 6") \right] - 2 \cdot \left[(8\tfrac{1}{2}" \cdot 8\tfrac{1}{2}") - (\tfrac{1}{4} \cdot \pi \cdot 8.5"^2) \right]$

$= (85" \cdot 33.5") - 2\,(15.50\,in^2) = 2816.5\,in^2$

$\boxed{2816\,in^2 = \text{Area uncompressed air per diaphragm}}$

• Area of diaphragm

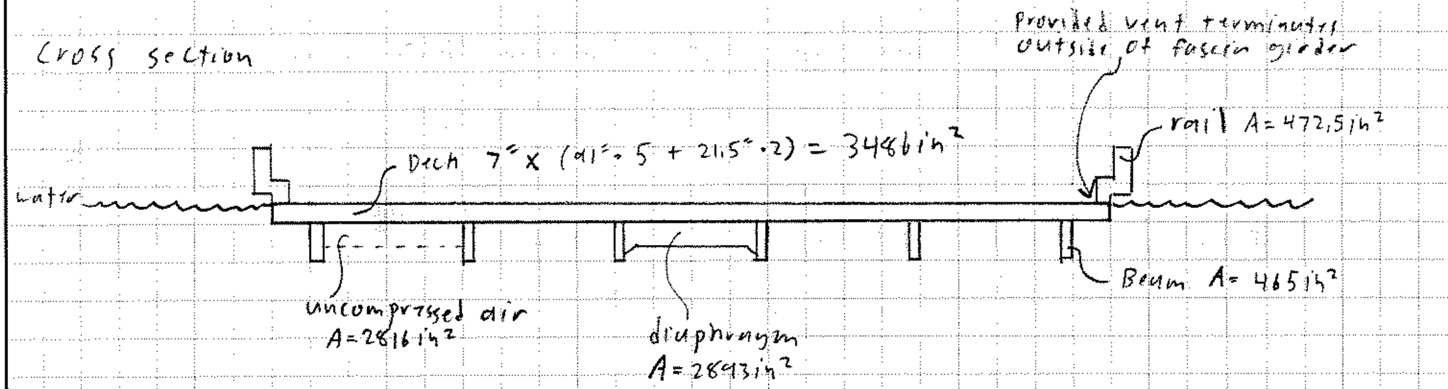
= Area uncompressed air + 2 • (6", 6 1/4", 6 1/2")

Approximate as a triangle
$A = \frac{1}{2} \cdot 6" \cdot 12.75" = 38.25\,in^2$

$= 2816.5\,in^2 + 2 \cdot 38.25\,in^2 = 2893\,in^2$

$\boxed{2893\,in^2 = \text{Area of typical diaphragm}}$

Cross section



• Cross section without diaphragm

unsubmerged concrete

$A = 2 \cdot 472.5 in^2 = 945 in^2 = \boxed{6.56 ft^2}$

submerged concrete

$A = 6 \cdot 465 in^2 + 3486 in^2 = 6276 in^2 = \boxed{43.58 ft^2}$

uncompressed air

$A = 5 \cdot 2816 in^2 = 14{,}080 in^2 = \boxed{97.78 ft^2}$

compressed air

$x = 7"$ $\quad Y = 33.5"$

$$\frac{v_f}{v_i} = \frac{-2117 - 64 \cdot \frac{7}{12} + \sqrt{\left(2117 + 64 \cdot \frac{7}{12}\right)^2 + \left(541{,}952 \cdot \frac{33.5}{12}\right)}}{128 \cdot \frac{33.5}{12}} \qquad (Eq\ 2)$$

$\frac{v_f}{v_i} = 0.91347$

$A = 0.91347 \cdot 97.78 ft^2 = \boxed{89.32 ft^2}$

• Cross section with diaphragm

unsubmerged concrete $= \boxed{6.56 ft^2}$

submerged concrete

$A = 6 \cdot 465 in^2 + 5 \cdot 2893 in^2 + 3486 in^2 = 20{,}741 in^2 = \boxed{144.03 ft^2}$

• thickness of diaphragms

2 end diaphragms @ $9\frac{1}{2}"$ thick + 2 intermediate @ 9" thick = 37" = 3.08' of diaphragms

65' − 3.08' = 61.92' ⇒ 61.92' without diaphragm

Made: TJS Date: 1/2207
Checked:______ Date:______

6

I-10 Lake Ponchartrain Buoyancy Calculations, Static Water Up to Top of Deck

		Unsubmerged			Uncompressed Air			Compressed Air Iterated h		
		Area (ft^2)	Volume (Ft^3)	Weight (kip)	Area (ft^2)	Volume (Ft^3)	Weight (kip)	Area (ft^2)	Volume (Ft^3)	Weight (kip)
Cross Section Without Diaphragm (61.92 ft)	Unsubmerged Concrete	50.14	3104.67	465.7	6.56	406.20	60.9	6.56	406.20	60.9
	Submerged Concrete	0	0.00	0.0	43.58	2698.47	232.1	43.58	2698.47	232.1
	Air	0	0.00	0.0	97.78	6054.54	-387.5	89.32	5530.69	-354.0
Cross Section With Diaphragm (3.08 ft)	Unsubmerged Concrete	150.59	463.82	69.6	6.56	20.20	3.0	6.56	20.20	3.0
	Submerged Concrete	0	0.00	0.0	144.03	443.61	38.2	144.03	443.61	38.2
			Total Weight	**535.3**		Total Weight	**-53.3**		Total Weight	**-19.8**

Buoyancy Load = 535.3 - (-19.8) = 555.1 kips
Buoyancy Load = 104% of selfweight

Ideal Gas Law

$PV = nrt$

n, r, and t remain constant

$P_i V_i = P_f V_f$

P_i = standard atmospheric pressure (neglect drop in pressure due to hurricane)
P_f = (standard atmospheric pressure) + (hydrostatic pressure)

$P_i = 14.7 \frac{lb}{in^2}$ (standard atmospheric pressure = 14.695948...... psi)
$= 14.7 \frac{lb}{in^2} \cdot \frac{144 in^2}{ft^2} = 2116.8\ lb/ft^2$
$P_i \approx 2117\ lb/ft^2$ ← use for calculation

$P_f = P_i + \gamma_{seawater} \cdot h_{seawater}$

$\gamma_{freshwater} \approx 62.4\ lb/ft^3$
density of seawater (at the surface) is about 2.7% to 3.5%
heavier than fresh water according to various online sources

$62.4 \cdot 1.027 = 64.08$
$62.4 \cdot 1.035 = 64.58$

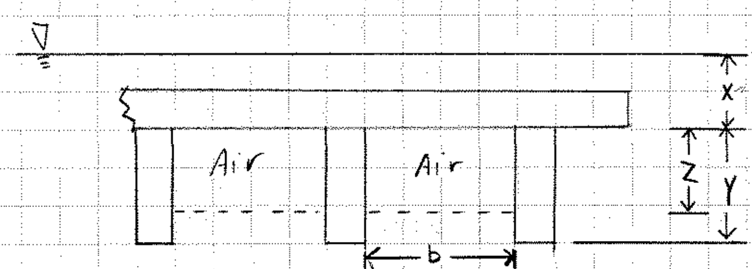
bay water likeley brakish, (less salinity than seawater), say $64\ lb/ft^3$

$P_i\ V_i = P_f\ V_f$

$V_f = V_i \cdot \frac{P_i}{P_f}$

$$\boxed{V_f = V_i \cdot \frac{2117\ lb/ft^2}{2117\ lb/ft^2 + (64\ lb/ft^3 \cdot h)}}$$ where h = final pressure head of seawater in feet Eq 1

$$\frac{V_f}{V_i} = \frac{2117\ lb/ft^2}{2117\ lb/ft^2 + (64\ lb/ft^3 \cdot h)}$$



Initial height of Airspace = Y
Initial pressure head of seawater = X + Y
Final height of Airspace = Z
Final pressure head of seawater = X + Z
Width of air void = b

from ideal gas Law

$$Vf = Vi \cdot \frac{2117\ ^{lb}/_{ft^2}}{2117\ ^{lb}/_{ft^2} + 64 \cdot h}$$ where h = final pressure head in feet (Eq 1)

• ASSUME air void is rectangular, so $Vf = b \cdot Z \cdot L$
$Vi = b \cdot Y \cdot L$
(L = Length of air void)

equation becomes

$$b \cdot Z \cdot L = b \cdot Y \cdot L \cdot \frac{2117}{2117 + 64 \cdot (X+Z)}$$

$$Z = \frac{2117 Y}{2117 + 64 \cdot (X+Z)}$$

$$2117 Y = 2117 Z + 64 XZ + 64 Z^2$$

$$\underbrace{64}_{a} Z^2 + \underbrace{(2117 + 64X)}_{b} Z \underbrace{- 2117 Y}_{c} = 0$$ Quadratic

$$Z = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a} = \frac{-(2117 + 64X) \pm \sqrt{(2117 + 64X)^2 - 4 \cdot 64 \cdot (-2117Y)}}{2 \cdot 64}$$

$$Z = \frac{-2117 - 64X \pm \sqrt{(2117 + 64X)^2 + 541{,}952 Y}}{128}$$

Z is always positive, so

$$Z = \frac{-2117 - 64X + \sqrt{(2117 + 64X)^2 + 541{,}952 Y}}{128}$$ assuming rectangular void

since rectangular void, assummed

$$\frac{V_f}{V_i} = \frac{Z}{Y}$$

so

$$\boxed{\frac{V_f}{V_i} = \frac{-2117 - 64 \cdot X + \sqrt{(2117 + 64 \cdot X)^2 + 541{,}952 \cdot Y}}{128 \cdot Y}}$$

Eq 2

Notes: Assumes rectangular void
All dimensions (X and Y) are in feet
Assumes $\gamma_{water} = 64$ lb/ft^3
Assumes Patm = 14.7 psi
Assumes Ideal Gas Law valid

Notes:

- Determine material strengths from bridge plans and old ASTM standards (Summary on page 11)
- Check load path from beam to bent to find weakest link (pages 12-15)
- Assume direct tension or direct shear. Simultaneous tensile and shear loading is difficult to address
- The extent to which fasteners were simultaneously engaged is unknown.
- Bridge plans may not match as-built conditions. Specific information on as-built conditions is not available
- The "correct" coefficient of friction to use is not readily apparent. A range is likely the best choice. Examine frictional resistance (pages 16-18)
- Examine behavior of entire span. (pages 19-21)

I-10 Lake Ponchartrain Bearings

Pictures of damage indicate that an end welded stud type connection (not a strap type connection) was used. No information could be obtained on this. The following reflects the details shown in the contract drawings.

Information on bearings for concrete spans:

- Steel and bronze bearings used
- Only fixed end exterior girders had vertical restraint (two connections per span)
- Uplift load path from beam to pier is as follows
    - Two straps transfer force from the beam into the bronze plate
        - The straps each have two legs, which are hooked and embedded into the beam
        - The straps are attached to the bronze plate – a weld symbol seems to appear on one of the drawings, but the quality/clarity of the drawing is poor
    - Two bolts transfer the force from the bronze plate into to the pier
        - The anchor bolts are embedded in the pier cap
- Area of steel for bolts and straps are about the same
    - Two 1" diameter bolts = 1.57 square inches
    - Four legs of 1" x 3/8" bar = 1.50 square inches
- Steel grades as follows
    - Steel straps and base plates
        - plans specify A7-58T (See File 3 sheet 7, General notes, Bullet #14)
        - See 1958 ASTM Standard pages 448 to 450
            - fu = 60 ksi minimum to 72 ksi maximum
            - fy = 33 ksi minimum
    - Steel Anchor bolts
        - 1955 LA Bridge Spec calls for ASTM A 307 Grade A bolts where high strength bolts are not required (see page 320 of 1955 LA spec)
        - See 1958 ASTM Standard pages 748 to 751
            - 1" diameter Grade A bolt
            - Minimum tensile strength = 33,500 lb
            - Based on 55 ksi strength
            - Stress area = 0.606 in^2 based on a given formula

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Projects
Subject I-10 Lake P bearings
Made by ____ Date
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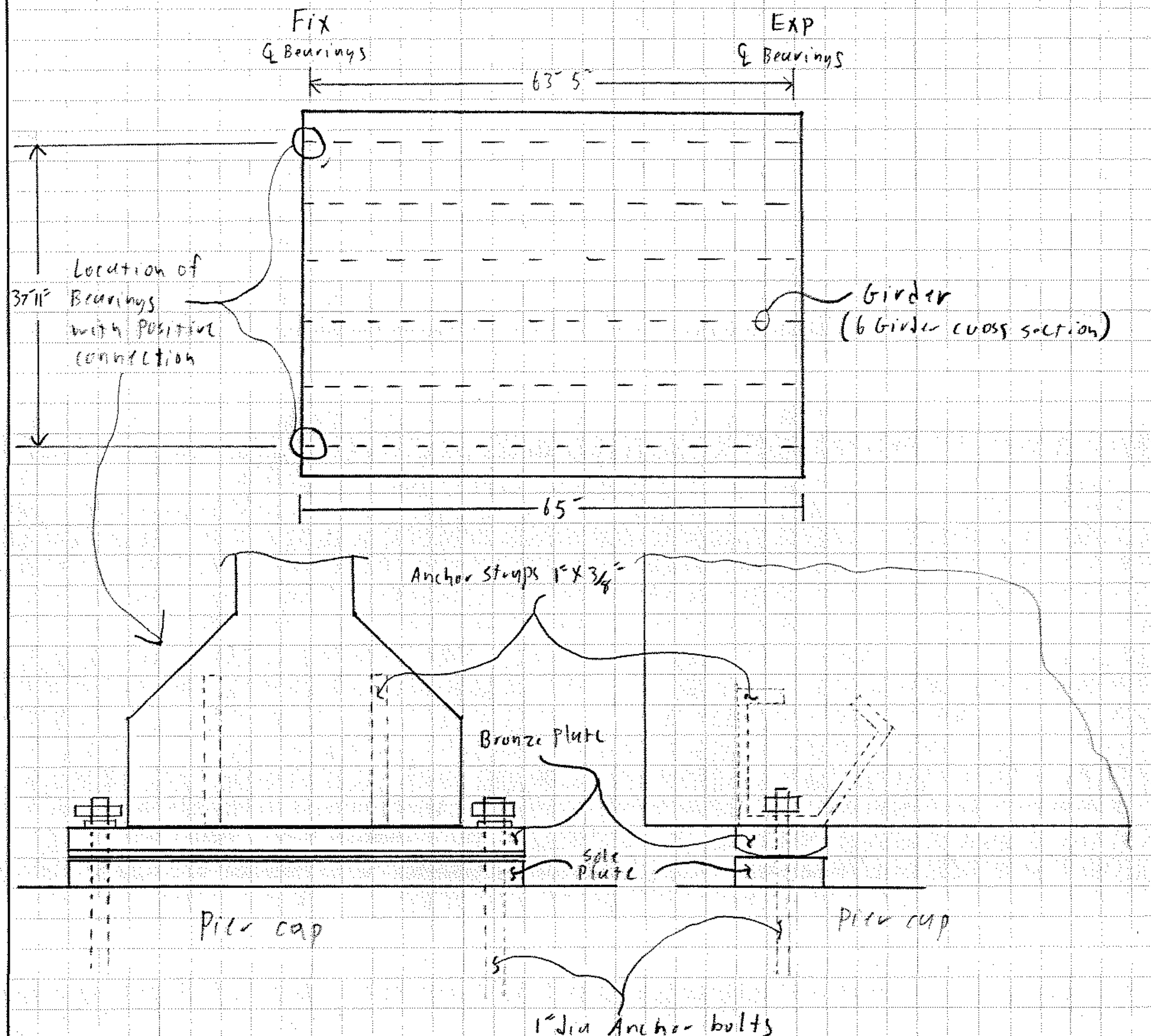
Sheet No. 12 of ____



From photos
- In some cases Anchor straps broke (studs?)
- In some cases Anchor bolts broke
- Anchor straps to bronze plate connection failure not observed in photos
- Shear or tension failure not known

- Tension capacity of bolt

  - from ASTM $T_n = 33{,}500$ lb/bolt

  - from AASHTO LRFD 6.13.2.10

$$T_n = 0.76\, A_b\, F_{ub}$$
$$= 0.76 \cdot (\pi \cdot 0.5^2) \cdot (55\,ksi) = 32.8\,K\ /\text{bolt}$$

∴ 1958 ASTM and 1998 AASHTO LRFD close, difference due to stress area of bolt, ASTM more accurately calculates this

say $T_n = 33{,}500$ lb/bolt or 67,000 lb/bearing

- Tension capacity of straps
  - AASHTO 6.13.5 - connection elements
  - consider Yield on gross section, and Fracture on net section

$$\left.\begin{aligned} P_{ny} &= F_y A_g \\ P_{nu} &= F_u A_n U \end{aligned}\right] \text{AASHTO 6.8.2}$$

$F_y = 33\,ksi$ minimum

$F_u = 60\,ksi$ minimum, 72 ksi maximum

- $P_{ny} = 33\,ksi \cdot (3/8'' \cdot 1'') = 12{,}375$ lb / strap Leg

- $P_{nu} = F_u A_n U$ if weld used $A_n = A_g$, $U = 1.0$ since all members connected

$P_{nu} = 60\,ksi \cdot (3/8'' \cdot 1'') \cdot 1.0 = 22{,}000$ lb/strap leg

$P_{ny} = 12{,}375$ lb/strap leg or 49,500 lb/bearing

but straps do not break until

$P_{nu} = 22{,}000$ lb/strap leg or 88,000 lb/bearing

- Shear capacity of bolts

Threads excluded from shear plane $R_n = 0.48\ A_b\ F_{ub}\ N_s$
Threads included in shear plane $R_n = 0.38\ A_b\ F_{ub}\ N_s$
] AASHTO 6.13.2.7

N_s = # shear planes per bolt = 1

Detail of anchor bolt included in the plans, but it is unclear if the shear plane includes the threads

excluded $R_n = 0.48 \cdot (\pi \cdot 0.5^2) \cdot 55\text{ ksi} \cdot 1 = 20.7$ K/bolt
included $R_n = 0.38 \cdot (\pi \cdot 0.5^2) \cdot 55\text{ ksi} \cdot 1 = 16.4$ K/bolt

assuming both bolts simultaneously engaged (optomistic)

Shear capacity of bolts

threads excluded R_n = 20.7 K/bolt = 41.4 K/bearing
threads included R_n = 16.4 K/bolt = 32.8 K/bearing

- Shear capacity of Anchor straps (Assuming direct shear through legs with no transfer of load to concrete through bearing pressure along welded part of strap)

AASHTO 6.13.5 - Connection elements

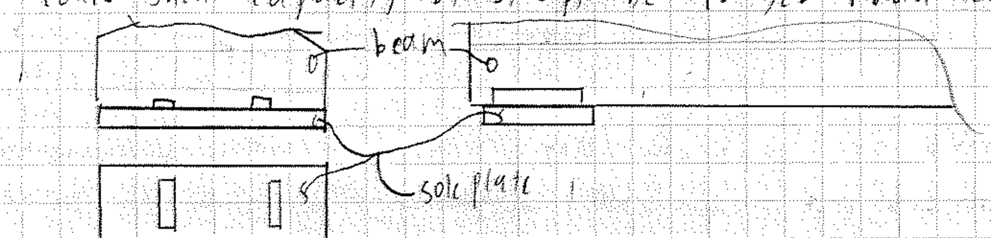
$R_n = 0.58\ A_g\ F_y$ 6.5.13.3

$R_n = 0.58 \cdot (3/8" \cdot 1") \cdot 33\text{ ksi} = 7.18$ kip/strap leg

Shear capacity of straps (direct shear on straps only)

R_n = 7.18 K/strap leg = 28.7 K/bearing

- could shear capacity of straps be larger than calculated above?

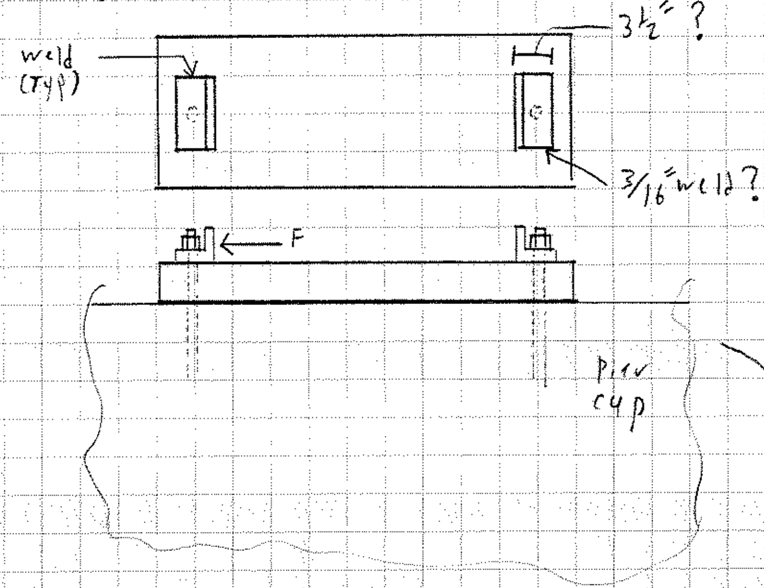


- suppose straps did not extend into beam, just tabs welded to sole plate, what is the capacity of this in shear??

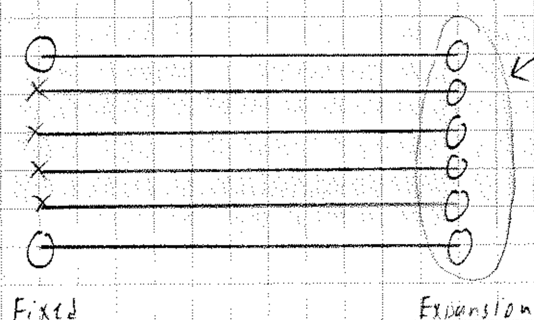
(3/8" high) · (about 5" long) · (2 tabs) · (0.85 · 5 ksi) ≈ 16 kips / bearing

Possible shear capacity of straps = 28.7 K/bearing + 16 K/bearing = 44.7 K/bearing

• If free end is not lifted above angles, shear resistance is provided at both ends



• If Angles are welded to base plate, the force F will/may be shared by both bolts and the shear plane will be between the base plate and the pier cap, thus threads are likely excluded from the shear plane. (If threads excluded $R_n = 20.7$ K/bolt)



• For predicting failure load angle capacity will not be a factor, even if the angle deforms substantially it will still stop the beam from sliding.

• check weld between angle and plate – Provided info inadequate, make assumptions
say 7" of 3/16" weld weld material = ??

strength of weld between 1 angle and plate (Fillet weld in shear)

$$= \underbrace{(0.60 \cdot f_{weld})}_{\text{material stress}} \underbrace{0.7071 \cdot 3/16}_{\text{effective throat}} \cdot \underbrace{7"}_{\text{length}} = 0.56\, f_{weld} = 18.5 \text{ K} = \text{weld strength}$$

say 33 ksi base metal (Grade A7)

• Say shear strength of connection governed by single bolt because weld attaching angle to plate is weak shear capacity bearing = 20.7 K

Marks' Standard Handbook for Mechanical Engineers

Eighth Edition

Theodore Baumeister, Editor-in-Chief
Eugene A. Avallone, Associate Editor
Theodore Baumeister III, Associate Editor

Table 1. Coefficients of Static and Sliding Friction

(Reference letters indicate the lubricant used; numbers in parentheses give the sources. See footnote)

Materials	Static		Sliding	
	Dry	Greasy	Dry	Greasy
Hard steel on hard steel	0.78 (1)	0.11 (1, *a*)	0.42 (2)	0.029 (5, *h*)
		0.23 (1, *b*)		0.081 (5, *c*)
		0.15 (1, *c*)		0.080 (5, *i*)
		0.11 (1, *d*)		0.058 (5, *j*)
		0.0075 (18, *p*)		0.084 (5, *d*)
		0.0052 (18, *h*)		0.105 (5, *k*)
				0.096 (5, *l*)
				0.108 (5, *m*)
				0.12 (5, *a*)
Mild steel on mild steel	0.74 (19)		0.57 (3)	0.09 (3, *a*)
				0.19 (3, *u*)
Hard steel on graphite	0.21 (1)	0.09 (1, *a*)		
Hard steel on babbitt (ASTM No. 1)	0.70 (11)	0.23 (1, *b*)	0.33 (6)	0.16 (1, *b*)
		0.15 (1, *c*)		0.06 (1, *c*)
		0.08 (1, *d*)		0.11 (1, *d*)
		0.085 (1, *e*)		
Hard steel on babbitt (ASTM No. 8)	0.42 (11)	0.17 (1, *b*)	0.35 (11)	0.14 (1, *b*)
		0.11 (1, *c*)		0.065 (1, *c*)
		0.09 (1, *d*)		0.07 (1, *d*)
		0.08 (1, *e*)		0.08 (11, *h*)
Hard steel on babbitt (ASTM No. 10)		0.25 (1, *b*)		0.13 (1, *b*)
		0.12 (1, *c*)		0.06 (1, *c*)
		0.10 (1, *d*)		0.055 (1, *d*)
		0.11 (1, *e*)		
Mild steel on cadmium silver				0.097 (2, *f*)
Mild steel on phosphor bronze			0.34 (3)	0.173 (2, *f*)
Mild steel on copper lead				0.145 (2, *f*)
Mild steel on cast iron		0.183 (15, *c*)	0.23 (6)	0.133 (2, *f*)
Mild steel on lead	0.95 (11)	0.5 (1, *f*)	0.95 (11)	0.3 (11, *f*)
Nickel on mild steel			0.64 (3)	0.178 (3, *x*)
Aluminum on mild steel	0.61 (8)		0.47 (3)	
Magnesium on mild steel			0.42 (3)	
Magnesium on magnesium	0.6 (22)	0.08 (22, *y*)		
Teflon on Teflon	0.04 (22)			0.04 (22, *f*)
Teflon on steel	0.04 (22)			0.04 (22, *f*)
Tungsten carbide on tungsten carbide	0.2 (22)	0.12 (22, *a*)		
Tungsten carbide on steel	0.5 (22)	0.08 (22, *a*)		
Tungsten carbide on copper	0.35 (23)			
Tungsten carbide on iron	0.8 (23)			
Bonded carbide on copper	0.35 (23)			
Bonded carbide on iron	0.8 (23)			
Cadmium on mild steel			0.46 (3)	
Copper on mild steel	0.53 (8)		0.36 (3)	0.18 (17, *a*)
Nickel on nickel	1.10 (16)		0.53 (3)	0.12 (3, *w*)
Brass on mild steel	0.51 (8)		0.44 (6)	
Brass on cast iron			0.30 (6)	
Zinc on cast iron	0.85 (16)		0.21 (7)	
Magnesium on cast iron			0.25 (7)	
Copper on cast iron	1.05 (16)		0.29 (7)	
Tin on cast iron			0.32 (7)	
Lead on cast iron			0.43 (7)	
Aluminum on aluminum	1.05 (16)		1.4 (3)	
Glass on glass	0.94 (8)	0.01 (10, *p*)	0.40 (3)	0.09 (3, *a*)
		0.005 (10, *q*)		0.116 (3, *v*)
Carbon on glass			0.18 (3)	
Garnet on mild steel			0.39 (3)	
Glass on nickel	0.78 (8)		0.56 (3)	
Copper on glass	0.68 (8)		0.53 (3)	
Cast iron on cast iron	1.10 (16)		0.15 (9)	0.070 (9, *d*)
				0.064 (9, *n*)
Bronze on cast iron			0.22 (9)	0.077 (9, *n*)
Oak on oak (parallel to grain)	0.62 (9)		0.48 (9)	0.164 (9, *r*)
				0.067 (9, *s*)
Oak on oak (perpendicular)	0.54 (9)		0.32 (9)	0.072 (9, *s*)
Leather on oak (parallel)	0.61 (9)		0.52 (9)	
Cast iron on oak			0.49 (9)	0.075 (9, *n*)
Leather on cast iron			0.56 (9)	0.36 (9, *t*)
				0.13 (9, *n*)
Laminated plastic on steel			0.35 (12)	0.05 (12, *t*)
Fluted rubber bearing on steel				0.05 (13, *t*)

(1) Campbell, *Trans. ASME*, 1939; (2) Clarke, Lincoln, and Sterrett, *Proc. API*, 1935; (3) Beare and Bowden, *Phil. Trans. Roy. Soc.*, 1935; (4) Dokos, *Trans. ASME*, 1946; (5) Boyd and Robertson, *Trans. ASME*, 1945; (6) Sachs, *Zeit. f. angew. Math. und Mech.*, 1924; (7) Honda and Yamada, *Jour. I of M*, 1925; (8) Tomlinson, *Phil. Mag.*, 1929; (9) Morin, *Acad. Roy. des Sciences*, 1838; (10) Claypoole, *Trans. ASME*, 1943; (11) Tabor, *Jour. Applied Phys.*, 1945; (12) Eyssen, General Discussion on Lubrication, *ASME*, 1937; (13) Brazier and Holland-Bowyer, General Discussion on Lubrication, *ASME*, 1937; (14) Burwell, *Jour. SAE*, 1942; (15) Stanton, "Friction," Longmans; (16) Ernst and Merchant, Conference on Friction and Surface Finish, M.I.T., 1940; (17) Gongwer, Conference on Friction and Surface Finish, M.I.T., 1940; (18) Hardy and Bircumshaw, *Proc. Roy. Soc.*, 1925; (19) Hardy and Hardy, *Phil. Mag.*, 1919; (20) Bowden and Young, *Proc. Roy. Soc.*, 1951; (21) Hardy and Doubleday, *Proc. Roy. Soc.*, 1923; (22) Bowden and Tabor, "The Friction and Lubrication of Solids," Oxford; (23) Shooter, *Research*, 4, 1951.

(*a*) Oleic acid; (*b*) Atlantic spindle oil (light mineral); (*c*) castor oil; (*d*) lard oil; (*e*) Atlantic spindle oil plus 2 percent oleic acid; (*f*) medium mineral oil; (*g*) medium mineral oil plus ½ percent oleic acid; (*h*) stearic acid; (*i*) grease (zinc oxide base); (*j*) graphite; (*k*) turbine oil plus 1 percent graphite; (*l*) turbine oil plus 1 percent stearic acid; (*m*) turbine oil (medium mineral); (*n*) olive oil; (*p*) palmitic acid; (*q*) ricinoleic acid; (*r*) dry soap; (*s*) lard; (*t*) water; (*u*) rape oil; (*v*) 3-in-1 oil; (*w*) octyl alcohol; (*x*) triolein; (*y*) 1 percent lauric acid in paraffin oil.

Summary of bolt / strap / bearing capacities

- The following assume: no over strength or corrosion
  no shear / tension interaction
  perfect simultaneous engagement

• Tension capacity

- Fixed end exterior, governed by bolts 33.5 K/bolt 67 K/bearing
- All other bearing locations 0 Kip/bearing

• Shear capacity

- Fixed end exterior - governed by bolts (assume excluded thread) 20.7 K/bolt 41.4 K/bearing
- Fixed end interior 0 K/bearing
- Expansion end → weld not effective, governed by single bolt
  20.7 K/bolt and 20.7 K/bearing
  → However, once span lifts above clip angles there is no shear capacity → 0 Kip/bearing
- Friction - optimistically assume friction acts in combination with all other fasteners

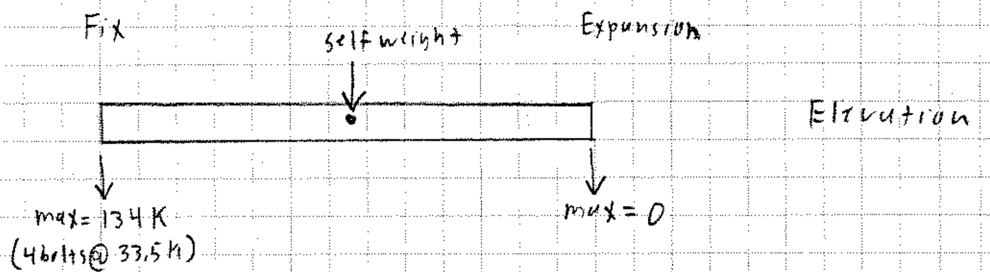
assume $\mu_{static} \approx 0.34$*
↳ mild steel and bronze plate
assume dry as opposed to greasy
see Marks standard handbook for Mechanical Engineers

* AASHTO Suggests values ranging from 0.10 to 0.40

Friction $= 0.34 \cdot (\text{selfweight} - \text{uplift}) \geq 0$

• Selfweight of span
unsubmerged span weighs about 535 K (see buoyancy calcs)

Uplift Resistance of Entire Span



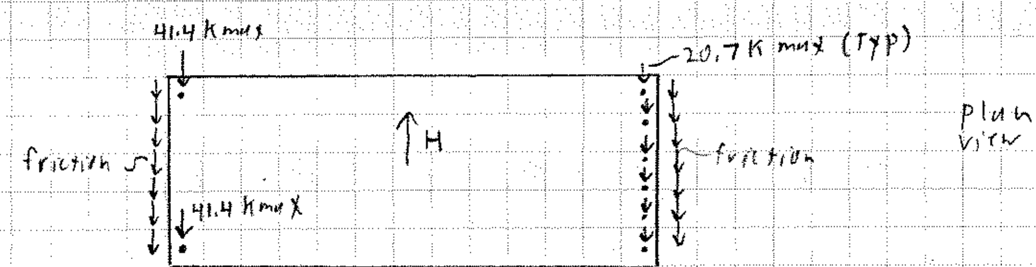
Minimum resistance to uplift = s.w. = 535 K
Maximum resistance to uplift = SW + Bolts
= 535 K + 134 K
= 669 K

| Equilibrium not maintained. span moving

say uplift resistance between 535 K and 670 K

Lateral Resistance of entire Span

- Lateral resistance is dependent on verticle load
  - → as verticle load is applied, friction is reduced
  - → once selfweight is over come expansion bearings will no longer provide lateral restraint (beams lift above angles)

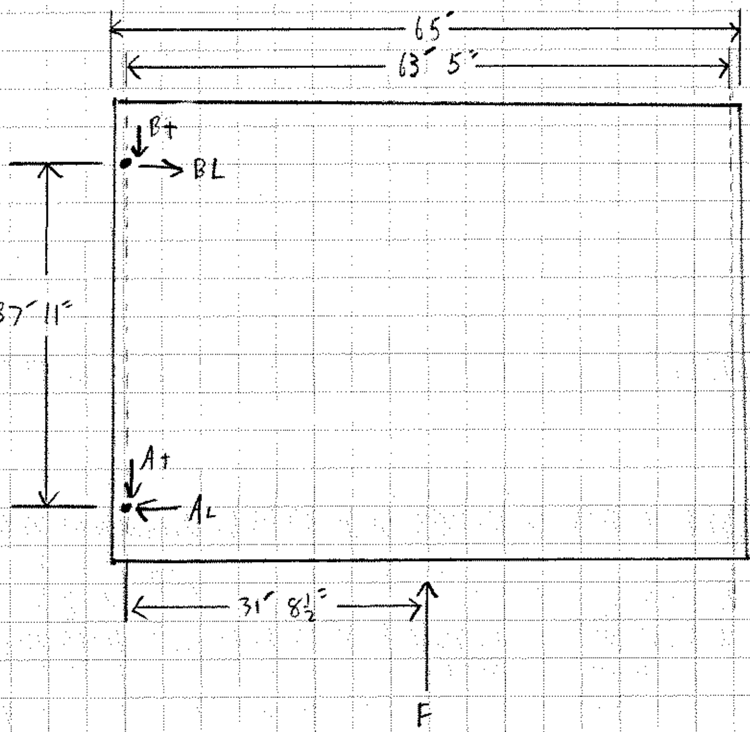


- assume equal reaction at ends of spans
  $2 \cdot 41.4\,K = 82.8\,K < 6 \cdot 20.7\,K = 124.2\,K$
  say fixed end controls, fasteners provide 82.8 K resistance at each end

- when uplift = 0 $H_{max} = 2 \cdot 82.8 + 535\,K \cdot 0.34 = 347.5\,K$

- when uplift ≈ dead load, but angles still engaged
  $H_{max} = 2 \cdot 82.8 + 0\,K \cdot 0.34 = 165.6\,K$

• If free end of span lifted above angles shear resistance comes only from fixed end exterior girders

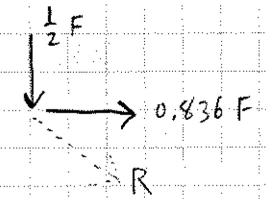


Assume $B_t = A_t = \frac{F}{2}$

$\sum M_A = 0 = BL \cdot 37'11" - F \cdot 31'8\tfrac{1}{2}"$

$BL = F \cdot \frac{31'8\frac{1}{2}"}{37'11"} = 0.836\,F$

$AL = BL = 0.836\,F$



$R = \sqrt{\tfrac{1}{2}^2 + 0.836^2} = 0.974\,F$

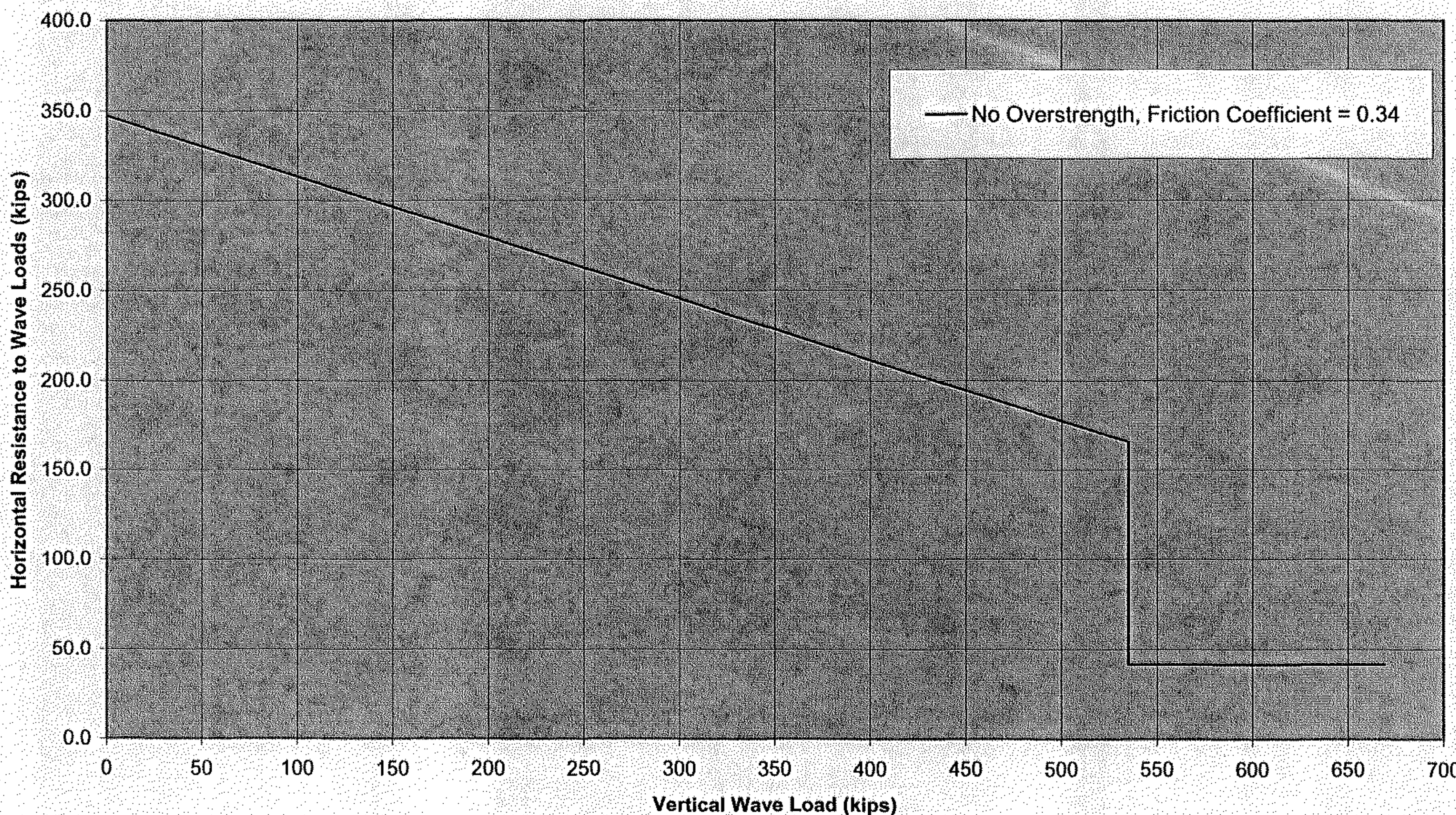
∴ Each bearing is subject to a resultant shear of magnitude about equal to F

↳ Each bolt subjected to a shear of $\frac{F}{2}$

∴ $F_{max} \approx 2\,R_n \approx$ shear capacity 1 fixed end exterior bearing

$\boxed{F_{max} \approx 41.4\,K}$

I-10 Lake P 65' Concrete Spans
Horizontal Resistance of Span Vs Vertical Load
S.W. = 535k with Coefficient of Friction = 0.34
Bearings at Free End Ineffective After Vertical Load Exceeds S.W.



STEP 1 Determine Section properties, prestress forces, Center of gravity of strands, Beam self weight, etc. (Pages 24-28)

STEP 2 Determine cracking moments using stress calcs. In the example contained herein only 1 cracking moment was determinde because the structure was monolithic. For a conventional cast in place deck 2 cracking moments would be calculated. One for the deck, then one for the top of the beam. In this case additional section properties would need to be calculated. (Uncracked beam, with deck steel, without deck concrete). (Pages 29-30)

STEP 3 Determine ultimate capacity of beam in negative bending using strain compatability. (Pages 31-32)

STEP 4 Estimate loads that may cause negative bending failure (Pages 33 and 35)

STEP 5 Examine results and assumptions used to calculate capacities. Determine if negative bending could be a concern (Page 34)

Summary of properties for I-10 Lake Ponchartrain Bridge
As Built Typical 65' Prestressed Girder – Interior Beam of cross section

Precast Prestressed Beam/Deck Segments

- Cast monolithically (all in one piece)
- Span = 63' 4" center to center of bearings
- 5 ksi concrete (not verified)

Individual Typical Interior Beam (including deck)

- Effective width of deck = 91" (controlled by beam spacing)
- Deck thickness = 6.5" at midspan (no deduction made for integral wearing surface)
- Gross area = 1056 in^2
- Self-weight = 1.100 k/ft
- 28 prestressing strands per beam
  - 7/16" diameter, stress relieved, seven wire strand
  - Area = 0.108 or 0.109 in^2 (0.108 in^2 used in calculations)
  - Initial tension = 18,900 lb per strand (after elastic shortening, before losses)
  - Total effective prestress = 423 kips (assuming 35 ksi loss from Pi to Pe)
  - cgs = 4.71"
- Mild steel
  - Plans unclear, appears to be about 9 #4 bars in the deck for each interior beam

Uncracked Gross Section Properties (not transformed)

Y Bar =	36.47
Inertia =	351,941
Ybot =	36.47
Sbot =	9,650
Ytop =	16.78
Stop =	20,975

Uncracked Transformed Section Properties

Y Bar =	36.08
Inertia =	373,211
Ybot =	36.08
Sbot =	10,343
Ytop =	17.17
Stop =	21,740

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Projects
Subject I-10 Lake Ponchartrain As built
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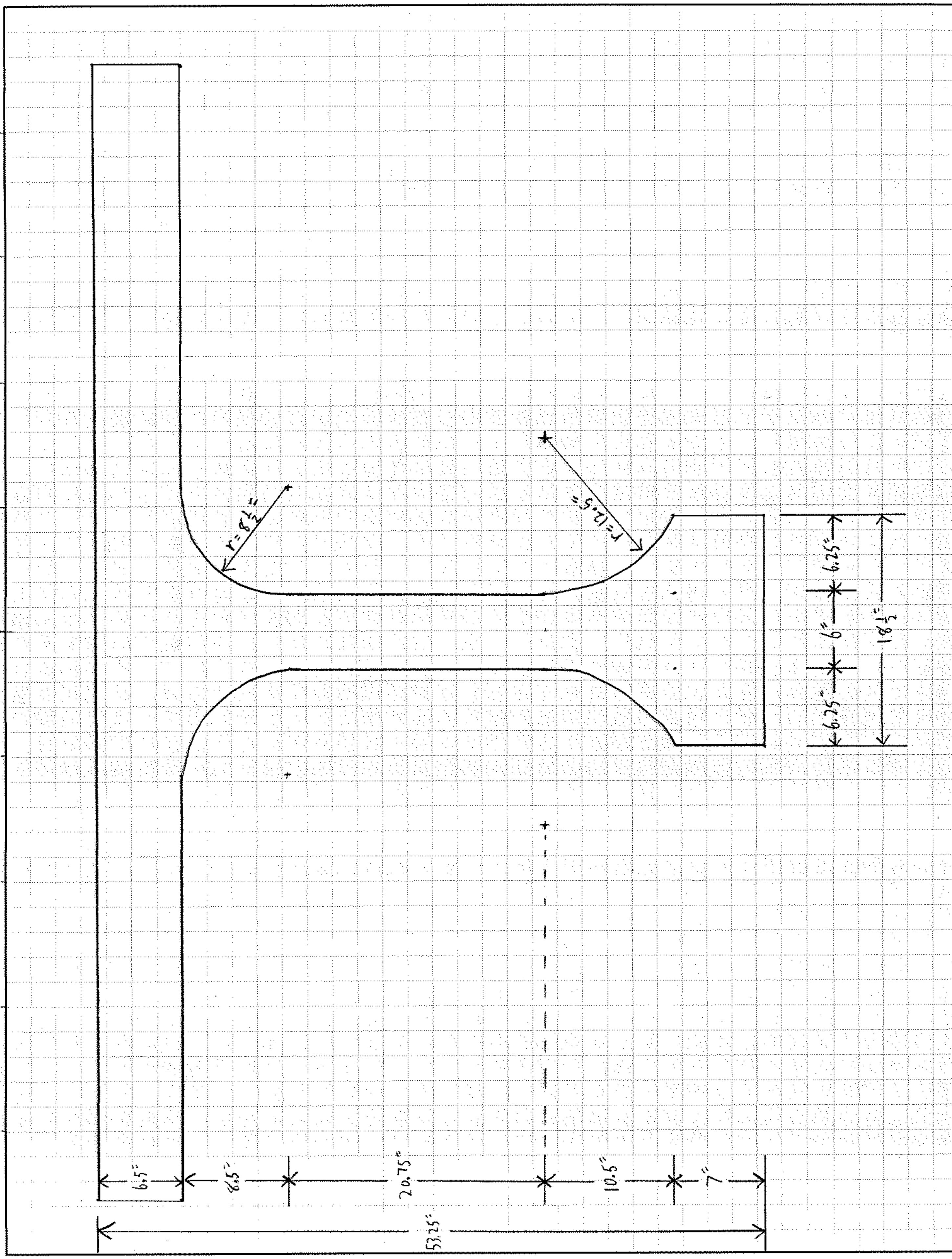
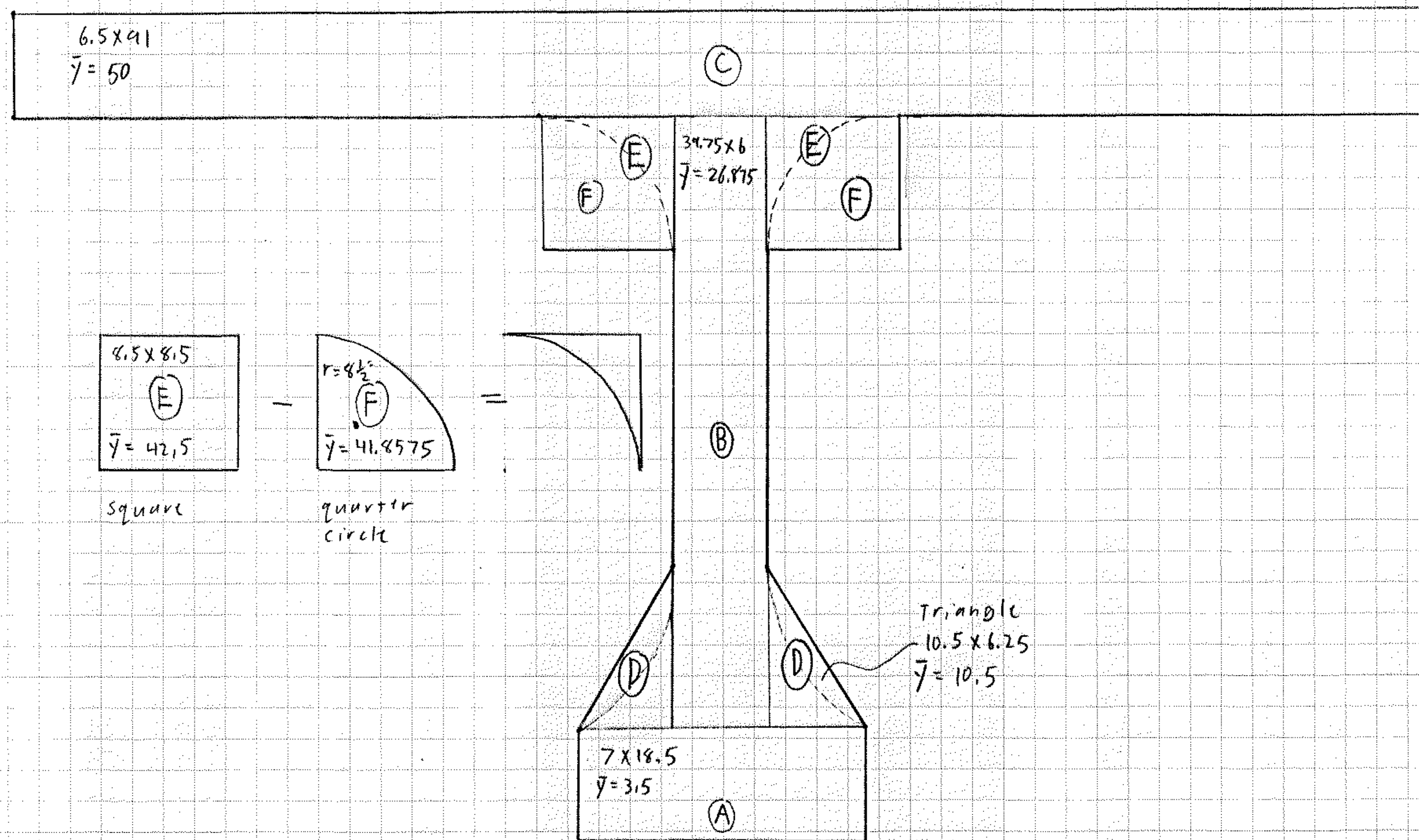
Date
Checked by
Date
Sheet No. 24 of



1 BLOCK = 5MM



D - use a triangle to approximate this Area - true shape involves odd fraction of a circle
E - Positive Area
F - Negative Area subtracted out to get correct shape
G through K prestressing strands E = 28,000 ksi | for simplicity assume as squares with same area as strand / bar
L mild steel in deck E = 29,000 ksi

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Projects
Subject I-10 Lake Ponchartrain As built

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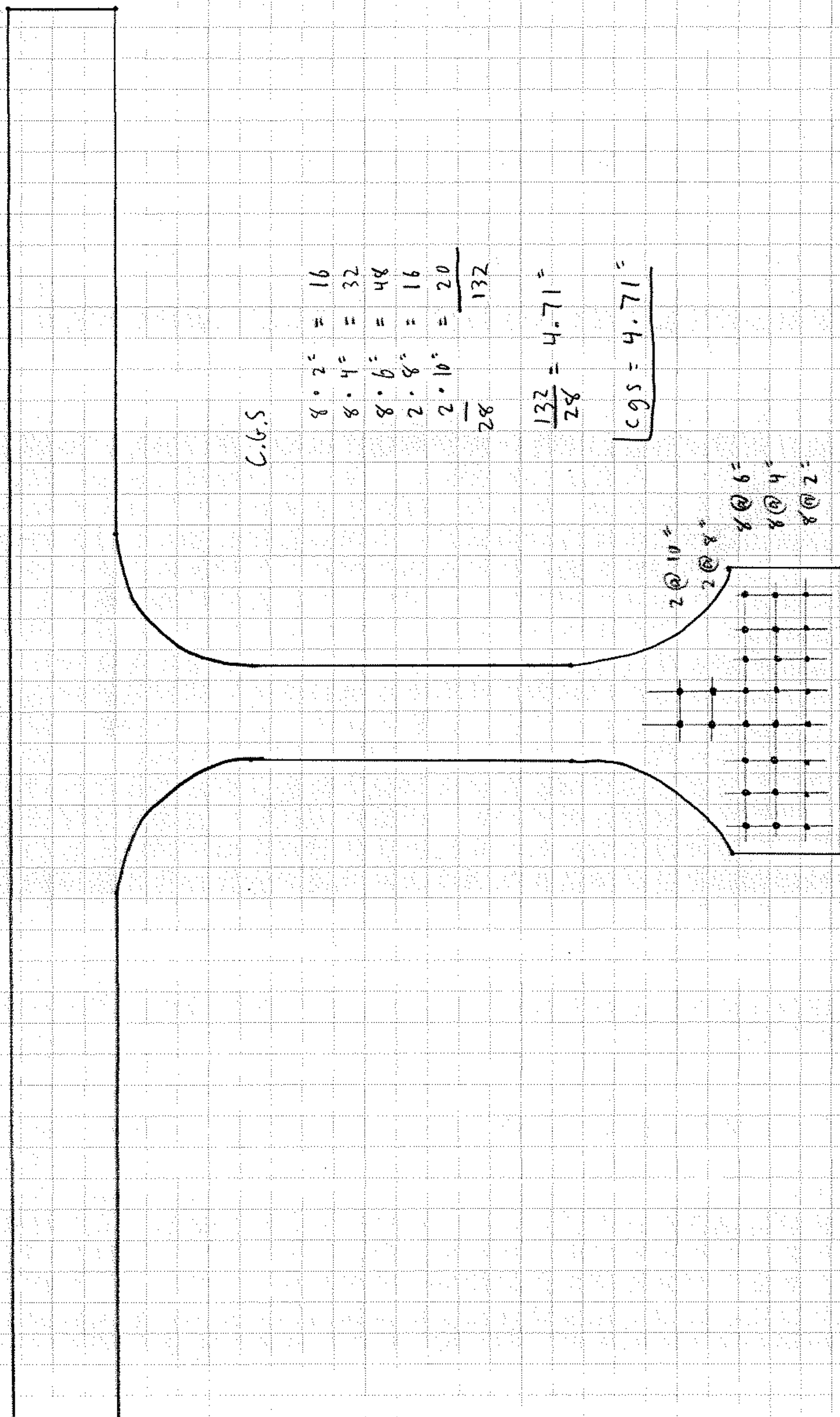


C.G.S

$8 \cdot 2" = 16$
$8 \cdot 4" = 32$
$8 \cdot 6" = 48$
$2 \cdot 8" = 16$
$2 \cdot 10" = 20$

$\overline{28}$ $\quad$ $\overline{132}$

$\frac{132}{28} = 4.71"$

$\boxed{cgs = 4.71"}$

1 BLOCK = 5MM

Made: TJS Date: 2006
Checked:______ Date:_______

I-10 Lake Ponchartrain Bridge
As Built Typical Prestressed Beam - 65 ft Length - 63' 4" Span
Uncracked Section, Gross Section Properties (Not Transformed)

Section	Number	Height or Radius (in)	Width (in)	Shape	y bar (in)	Area (in^2)	y bar * A (in^3)	Inertia, I (in^4)	d (in)	A*d^2 (in^4)	I + Ad^2 (in^4)
A	1	7	18.5	R	3.5	129.5	453	528.8	-32.97	140,774	141,303
B	1	39.75	6	R	26.875	238.5	6410	31403.7	-9.60	21,960	53,364
C	1	6.5	91	R	50	591.5	29575	2082.6	13.53	108,271	110,354
D	2	10.5	6.25	T	10.5	65.6	689	402.0	-25.97	44,262	44,664
E	2	8.5	8.5	R	42.5	144.5	6141	870.0	6.03	5,253	6,123
F	2	8.5	NA	C	41.8575	-113.5	-4750	-573.0	5.39	-3,293	-3,866
					(Sum)	1,056.1	38,518			(Sum)	351,941

Y Bar = 36.47
Inertia = 351,941
Ybot = 36.47
Sbot = 9,650
Ytop = 16.78
Stop = 20,975

27

Made: TJS   Date: 2006
Checked:______ Date:_______

I-10 Lake Ponchartrain Bridge
As Built Typical Prestressed Beam - 65 ft Length - 63' 4" Span
Uncracked Section, Transformed Section Properties

Section	Number	Height or Radius (in)	Width (in)	Shape	y bar (in)	N or (N-1)	Transformed Area (in^2)	y bar * A (in^3)	Inertia, I (in^4)	d (in)	A*d^2 (in^4)	I + Ad^2 (in^4)
A	1	7	18.5	R	3.5	1	129.5	453	528.8	-32.58	137,486	138,014
B	1	39.75	6	R	26.875	1	238.5	6410	31403.7	-9.21	20,223	51,626
C	1	6.5	91	R	50	1	591.5	29575	2082.6	13.92	114,560	116,643
D	2	10.5	6.25	T	10.5	1	65.6	689	402.0	-25.58	42,952	43,354
E	2	8.5	8.5	R	42.5	1	144.5	6141	870.0	6.42	5,950	6,820
F	2	8.5	NA	C	41.8575	1	-113.5	-4750	573.0	5.77	-3,784	-3,211
G	8	0.3286	0.3286	R	2	5.95	5.1	10	0.0	-34.08	5,972	5,972
H	8	0.3286	0.3286	R	4	5.95	5.1	21	0.0	-32.08	5,292	5,292
I	8	0.3286	0.3286	R	6	5.95	5.1	31	0.0	-30.08	4,652	4,652
J	2	0.3286	0.3286	R	8	5.95	1.3	10	0.0	-28.08	1,014	1,014
K	2	0.3286	0.3286	R	10	5.95	1.3	13	0.0	-26.08	874	874
L	9	0.4472	0.4472	R	50	6.20	11.2	558	0.2	13.92	2,161	2,162
					(Sum)		1,085.3	39,161			(Sum)	373,211

Y Bar = 36.08
Inertia = 373,211
Ybot = 36.08
Sbot = 10,343
Ytop = 17.17
Stop = 21,740

Beam Properties (use Transformed when Applicable)

$A = 1056\ in^2$
$sw = 1.10\ K/ft$
$span = 63' 4''$
effective prestress $= P_e = 423\ K$
$cgs = 4.71''$
$\bar{y} = 36.08''$ (transformed)
$e = \bar{y} - cgs = 31.37''$
$S_{bot} = 10,343\ in^3$ (transformed)
$S_{top} = 21,740\ in^3$ (transformed)
$P_e = 423\ K$

$$M_{sw} = \frac{wL^2}{8} = \frac{1.10\ K/ft \cdot 63.33'^2}{8} = 551.5\ kft = 6,618\ kin$$

Stress state in beam, long term, no external loads

- $$\sigma_{top} = \frac{P}{A} - \frac{P \cdot e}{S_{top}} + \frac{M_{sw}}{S_{top}}$$

$$= \frac{423\ K}{1056\ in^2} - \frac{423\ K \cdot 31.37''}{21,740\ in^3} + \frac{6,618\ K\text{-}in}{21,740\ in^3}$$

$$= 0.400 - 0.610 + 0.304$$

Note: if s.w. removed, top of beam/arch is in tension

$\boxed{\sigma_{top} = 0.095\ ksi\ compression}$

- $$\sigma_{bottom} = \frac{P}{A} + \frac{P \cdot e}{S_{bot}} - \frac{M_{sw}}{S_{top}}$$

$$= \frac{423\ K}{1056\ in^2} + \frac{423\ K \cdot 31.37''}{10,343\ in^3} - \frac{6,618\ K\text{-}in}{10,343\ in^3}$$

$$= 0.400 + 1.283 - 0.640$$

$\boxed{\sigma_{bot} = 1.044\ ksi\ compression}$

- What moment would it take to crack the top of the beam/deck

  - assume deck/beam will crack at $7.5\sqrt{f'c}$

    $7.5\sqrt{5{,}000} = 530$ psi tension

  $\Delta\sigma$ to cause cracking = 0.095 ksi comp + 0.530 ksi ten = 0.625 ksi

  if $\Delta\sigma = 0.625$ ksi tension deck cracks

  $\sigma = \frac{M}{S}$ $\quad\quad \sigma \cdot S = M$

  Mcrack = −0.625 ksi • 21,740 in³ = −13,587 k-in = −1132 k•ft

  Mcrack = −13,587 kin = −1132 kft

- At Mcrack what is the stress state at the bottom of the beam?

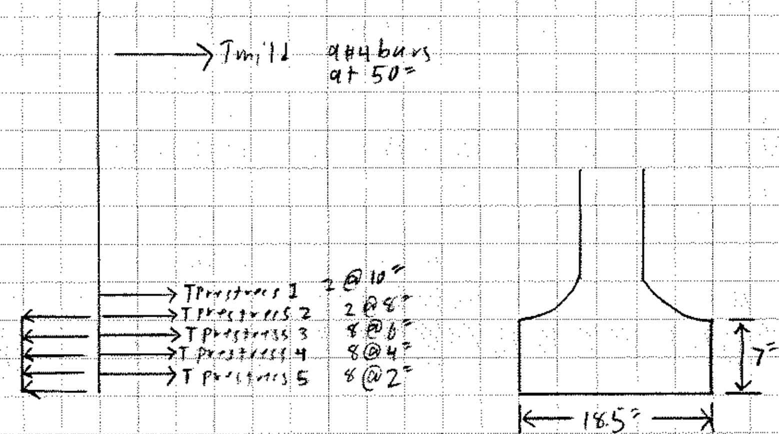
  $\sigma_{bot} = 1.044 \text{ ksi comp} + \frac{13{,}587 \text{ k}\cdot\text{in}}{10{,}343 \text{ in}^3} =$

  $= 1.044 + 1.314 = 2.358 \text{ ksi} = 0.47 f'c$

  at Mcr $\sigma_{bot} < \frac{1}{2} f'c$ ∴ top of beam controls

Ultimate moment capacity in Negative Bending

a = depth stress block
b = width stress block
c = depth N.A.
$c = \frac{a}{\beta_1}$
$\beta_1 = 0.80$ for 5 ksi



Assuming a = 5.66" (Iterations not shown)

T = 445 k (from strain compatability spread sheet)

C = (0.85 · 5 ksi) · (18.5" · 5.66") = 445 k

T = C ∴ Equilibrium achieved

Moment caused by steel about bottom of beam = Ms = 599.15 k·ft (from spreadsheet)

Moment caused by concrete about bottom of beam = Mc

$Mc = C \cdot \frac{a}{2}$ (rectangular stress block)

$= 455\ k \cdot \frac{(5.66/12)'}{2} = 107.30\ k \cdot ft$

Mn = Ms − Mc = 599.15 k·ft − 107.30 k·ft = 491.85 kft

∴ say Mn = 492 kft

To break beam upwards self weight must be overcome

Applied Mu = 551 kft + 492 kft = 1,043 k·ft

Applied Moment to cause ultimate type failure in negative bending = 1,043 k·ft

32

Made: TJS Date:2/12/2007
Checked:_______ Date:______

I-10 Lake Ponchartrain -- As Built -- Moment Capacity of Beams
Strain Compatability Spreadsheet
Input Values are Shaded

Layer Description		Mild Tension Layer 1	Mild Tension Layer 2	Mild Compression Layer 3	Mild Compression Layer 4	Prestress only Layer 5	Prestress only Layer 6	Prestress only Layer 7	Prestress only Layer 8	Prestress only Layer 9
Mild Steel	fy (ksi)	60	0	0	0					
	E (ksi)	29000	1	1	1	28500	28500	28500	28500	28500
Prestressing Steel	fpi (ksi)	0	0	0	0	175	175	175	175	175
	fse (ksi)	0	0	0	0	140	140	140	140	140
d		50.0	0	0	0	2	4	6	8	10
#bars/strands/wires		9	0	0	0	8	8	8	2	2
Area of bar/strand/wire (in^2)		0.2	0	0	0	0.108	0.108	0.108	0.108	0.108
Area of Steel As (in^2)		1.8	0	0	0	0.864	0.864	0.864	0.216	0.216
Strain		0.01820	-0.00300	-0.00300	-0.00300	0.00276	0.00361	0.00446	0.00530	0.00615
Calculated stress (ksi)		527.8	0.0	0.0	0.0	78.7	102.8	127.0	151.2	175.3
Maximum Stress (ksi)		60	60	0	0	250	250	250	250	250
fsi (ksi)		60.0	0.0	0.0	0.0	78.7	102.8	127.0	151.2	175.3
(Asi*fsi) (kips)		108.0	0.0	0.0	0.0	68.0	88.9	109.7	32.7	37.9
Sum (Asi*fsi) (kips)		445.09								
M about bottom of beam (k*ft)		450.0	0.0	0.0	0.0	11.3	29.6	54.9	21.8	31.6
Sum of M about bottom (k*ft)		599.15								

For empty layers set equal to 1
fpi is not used in calcs
for empty layers set equal to 0
As = (Area of bar) * (# of bars)
strain = 0.003(d/c-1) +(fse/E)
Calc. Stress = E*strain (mild) or PCI 1999 pg 11-22 for 250 ksi
will be equal to ultimate or yield, negative for compression
fsi = smaller of (max stress, calculated stress)
tension is positive

Beta 1 =	0.80
a (in)	5.66
c (in)	7.075

= a/Beta 1

** PCI Handbook 1999 page 11-22 equation for 250 ksi low lax seven wire strand is used.
The I-10 Lake Ponchartrain Bridge spans used 250 ksi stress relieved strands.
For strain < or = 0.0076 fps=28500eps and for strain > 0.0076 fps = 250 - 0.04/(eps-0.0064)
This equation assumes E=28500 ksi, say ok even though E=28000 ksi was assumed earlier

• What moments would be caused at midspan due to Loss of self weight and trapped air if the section were submerged in static water to the level of the deck?

Submersion of concrete (single interior beam)

$1056 in^2 \cdot \frac{1 ft^2}{144 in^2} \cdot 64 \, lb/ft^3 = 469 \, lb/ft$

Trapped air

$A = 2816 \, in^2$ (uncompressed, per air cavity see buoyancy calcs)

$\frac{V_f}{V_i} = 0.913$ for water at deck level (see buoyancy calcs)

$2816 in^2 \cdot 0.913 \cdot \frac{1 ft^2}{144 in^2} \cdot 64 \, lb/ft^3 = 1143 \, lb/ft$

Total buoyancy $= 469 \, lb/ft + 1143 \, lb/ft = 1612 \, lb/ft$

midspan moment $= \frac{-1.612 \, k/ft \cdot 63.33'^2}{8} = -808 \, kft$

$\boxed{\text{negative moment due to buoyancy} = -808 \, k \cdot ft}$

Summary of negative bending

- Moment to crack top of section = -1132 K·ft
- At M_{cr} $\sigma_{bottom} \approx \frac{1}{2} f'c$
- Ultimate negative moment capacity $M_n \approx -492$ K ft
- To break beam upwards selfeight + M_n must be applied, so an applied negative moment of about 1043 K·ft cause an ultimate moment type failure
- Because $M_{cr} = -1132$ kft $>$ $(M_n + M_{sw}) \approx -1000$ kft, once the beam cracks it is failed.
- It could be argued that $7.5\sqrt{f'c}$ is too low to truely represent rupture. This is true, $12\sqrt{f'c}$ may be obtained in tests.

  It could also be argued that 5 ksi does not represent the actual concrete strength.

  The above two arguments could be used to increase M_{cr}. However, it could also be argued that the deck could have transverse cracks from service.

- Say beam will be damaged / destroyed by a negative moment on the order of 1,043 to 1132 K·ft.
- negative moment due to buoyancy ≈ -808 kft
- when wave forces are added to buoyancy, beams anchored to the pier could very well be destroyed or damaged in negative bending.

what loads will cause all beams in section to fail in negative bending

- Assume exterior beams have roughly the same capacity as interior beams
- Assume loads are distributed evenly between all beams
- For distributed loads (applied center to center of bearings)

$M = \frac{wL^2}{8}$ for a single beam $N = \#$ of beams

$$P = wL = \frac{8 \cdot N \cdot M}{L}$$

- use L = C to C of bearings

For $M_{cr} = 1{,}132$ K·ft

$$P = \frac{8 \cdot 6 \cdot 1{,}132 \text{ K·ft}}{63.33'} = 858 \text{ K} \text{ for distributed Load}$$

For $M_{applied} = 1{,}043$ K ft (for ultimate failure after cracking

$$P = \frac{8 \cdot 6 \cdot 1043 \text{ K ft}}{63.33'} = 790 \text{ K} \text{ for distributed load}$$

Above loads only valid if load uniformly distributed longitudinally and transversely

STEP 1 - Calculate moment capacity of pile using strain compatability (sheets 37-41)

STEP 2 - Analyze single pile using com 624 (see sheet 42)

Lateral Analysis 1 Fixed head pile, applied shear, allow plastic moment to form (sheets 43-49)

Lateral Analysis 2 Replace rigid location where plastic moment formed with a hinge and applied Mp, apply shear until second plastic hinge forms (sheets 50-56)

Longitudinal analysis 1 Free head pile, applied shear, allow plastic moment to form (sheets 57-63)

STEP 3 - Relate behavior of single pile to bent behavior.

In this case, multiply Vmax by 3. This assumes bent cap capacity will not govern.

Material Information

- Class P concrete, say 5 ksi
- Effective stress in prestressing, say 120 ksi

See bridge plans
See ENR Articles
See Raymond pile literature
See ASTM Data

Notes:

Moment capacity is insensitive to fpe, it was varied from 100 to 135 ksi with a change in moment capacity of about 0.5%

Moment capacity is not completely insensitive to concrete strength. Using f'c = 8 ksi increased moment capacity by about 7.5%

Both of the abovementioned factors are insignificant when compared to the effect of "L" on "Vmax".

I-10 Lake Ponchartrain As-built Bents -- Moment Capacity of Piles

For hollow circular sections in bending.
Case A is valid when the depth of the stress block is less than or equal to the wall thickness.
Case B is valid when the depth of the stress block is greater than or equal to the wall thickness.
Formulas not explained are from Naaman text, second edition, pages 791 to 792.
Fill is shaded cells only.

OD (in) =	54	
ID (in) =	44	
T (in) =	5	= (OD - ID) /2
a (in) =	8.741	
f'c (ksi)=	5	

Input the:
inside diameter (ID)
outside diameter (OD)
depth of the stress block (a)
concrete strength (f'c)

a Too Big				
Case A Not Valid		Case B Valid		
Case A		Case B		
Phi (rad)	NA	Phi o (rad)	1.65625	
		Phi I (rad)	1.18354	
Ae (in^2) =	NA	Ae (in^2) =	178.19	
Xbar (in) =	NA	Xbar (in) =	22.52	
C (kips) =	NA	C (kips) =	757.33	= Ae * 0.85 * f'c
Dtop (k*ft) =	NA	Dtop (k*ft) =	4.48	= OD/2 - Xbar
Mtop (k*ft) =	NA	Mtop (k*ft) =	282.53	= C * Dtop / 12

Finds the:
area of the stress block (Ae) (assumes walls are solid)
distance from the center of the circle to the centroid of the stress block (Xbar)
concrete compressive force (C)
distance from the centroid of the stress block to the top of the section (Dtop)
moment that the compressive force causes about the top of the section (Mtop)

T =	757.34	Values from steel sheet
M steel=	2108.33	

To use spreadsheet input values in shaded cells (This sheet and next)

guess at a (depth of stress block) until $T = C$ (equilibrium achieved)

Then $M_n = M_{steel} - M_{concrete}$

I-10 Lake Ponchartrain As-built Bents -- Moment Capacity of Piles
Strain Compatability Spreadsheet
Input Values are Shaded

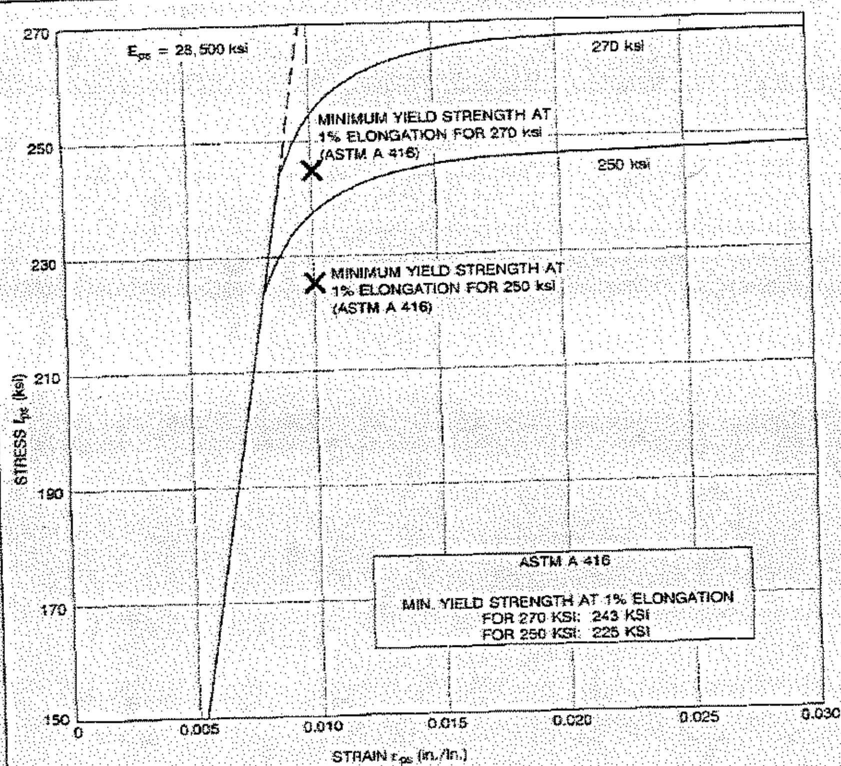
Layer Description		Mild Tension		Mild Compression		Prestress only						Notes
		Layer 1	Layer 2	Layer 3	Layer 4	Layer 5	Layer 6	Layer 7	Layer 8	Layer 9	Layer 10	
Mild Steel	fy (ksi)	0	0	0	0							
	E (ksi)	1	1	1	1	29000	29000	29000	29000	29000	29000	For empty layers set equal to 1
Prestressing Steel	fpi (ksi)	0	0	0	0	150	150	150	150	150	150	fpi is not used in calcs
	fse (ksi)	0	0	0	0	120	120	120	120	120	120	
d		0	0	0	0	3.46	9.76	20.69	33.31	44.24	50.54	for empty layers set equal to 0
#bars/strands/wires		0	0	0	0	24	24	24	24	24	24	
Area of bar/strand/wire (in^2)		0	0	0	0	0.0289	0.0289	0.0289	0.0289	0.0289	0.0289	
Area of Steel As (in^2)		0	0	0	0	0.6936	0.6936	0.6936	0.6936	0.6936	0.6936	As = (Area of bar) * (# of bars)
Strain		-0.003	-0.003	-0.003	-0.003	0.002087937	0.003817716	0.006818746	0.010283795	0.013284825	0.015014604	strain = 0.003(d/c-1) +(fse/E)
Calculated stress (ksi)		-0.003	-0.003	-0.003	-0.003	59.50619751	108.8049047	194.3342492	239.7007962	244.190121	245.3567222	Calc. Stress = E*strain (mild) = PCI 1999 pg 11-22 for 250 ksi (prestress)
Maximum Stress (ksi)		0	0	0	0	250	250	250	250	250	250	will be equal to ultimate or yield, negative for compression
fsi (ksi)		-0.003	-0.003	0	-0.003	59.50619751	108.8049047	194.3342492	239.7007962	244.190121	245.3567222	fsi = smaller of (max stress, calculated stress)
(Asi*fsi) (kips)		0	0	0	0	41.27349859	75.46708193	134.7902353	166.2564722	169.3702679	170.1794225	tension is positive
Sum (Asi*fsi) (kips)		757.34										
M about top of beam (k*ft)		0	0	0	0	11.90052543	61.37989331	232.4008306	461.5002575	624.4117211	716.7390013	
Sum of M about top (k*ft)		2108.33										

Beta 1 =	0.80	
a (in)	8.741	(from concrete sheet)
c (in)	10.92625	= a/Beta 1

** PCI Handbook 1999 page 11-22 equation for 250 ksi low lax seven wire strand is used. The lake P 1-10 bridge piles used 250 ksi stress relieved wires.
For strain < or = 0.0076 fps=28500eps and for strain > 0.0076 fps = 250 - 0.04/(eps-0.0064)

MATERIAL PROPERTIES
PRESTRESSING STEEL

Design Aid 11.2.5 Typical stress-strain curve, 7-wire low-relaxation prestressing strand



These curves can be approximated by the following equations:

250 ksi

$\varepsilon_{ps} \leq 0.0076 : f_{ps} = 28{,}500\,\varepsilon_{ps}$ (ksi)

$\varepsilon_{ps} > 0.0076 : f_{ps} = 250 - \dfrac{0.04}{\varepsilon_{ps} - 0.0064}$ (ksi)

270 ksi

$\varepsilon_{ps} \leq 0.0086 : f_{ps} = 28{,}500\,\varepsilon_{ps}$ (ksi)

$\varepsilon_{ps} > 0.0086 : f_{ps} = 270 - \dfrac{0.04}{\varepsilon_{ps} - 0.007}$ (ksi)

Figure 13.11 illustrates the two possible cases for the compression block of a hollow-cored section, depending on the location of the neutral axis at ultimate. The following results can be easily derived [Refs. 13.8, 13.52]:

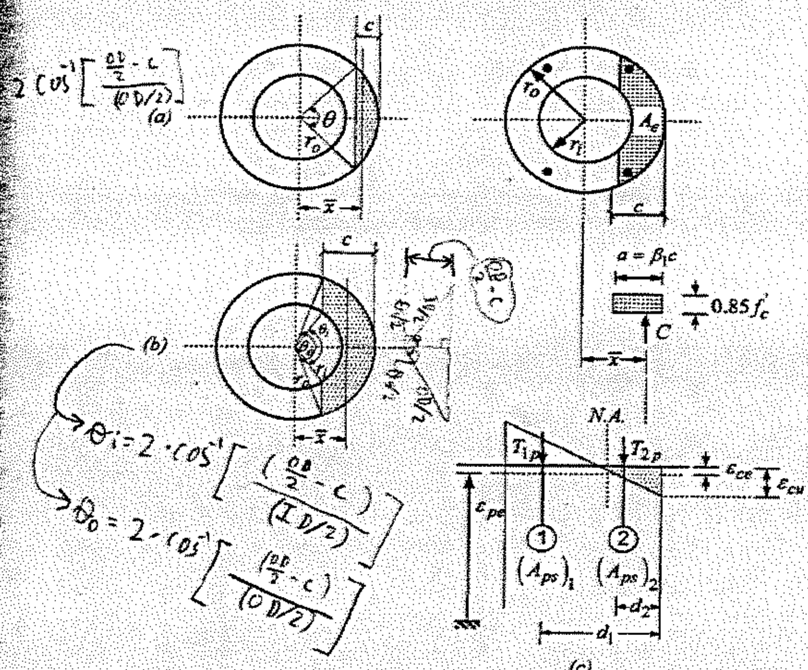


Figure 13.11 Cases of effective compression zone at ultimate for circular hollow-cored sections.

For case (a):

$$A_e = \frac{1}{2}(\theta - \sin\theta) r_o^2 \tag{13.21}$$

$$\bar{x} = \frac{\sqrt{2}}{3}\frac{(1-\cos\theta)^{1.5}}{\theta - \sin\theta} r_o \tag{13.22}$$

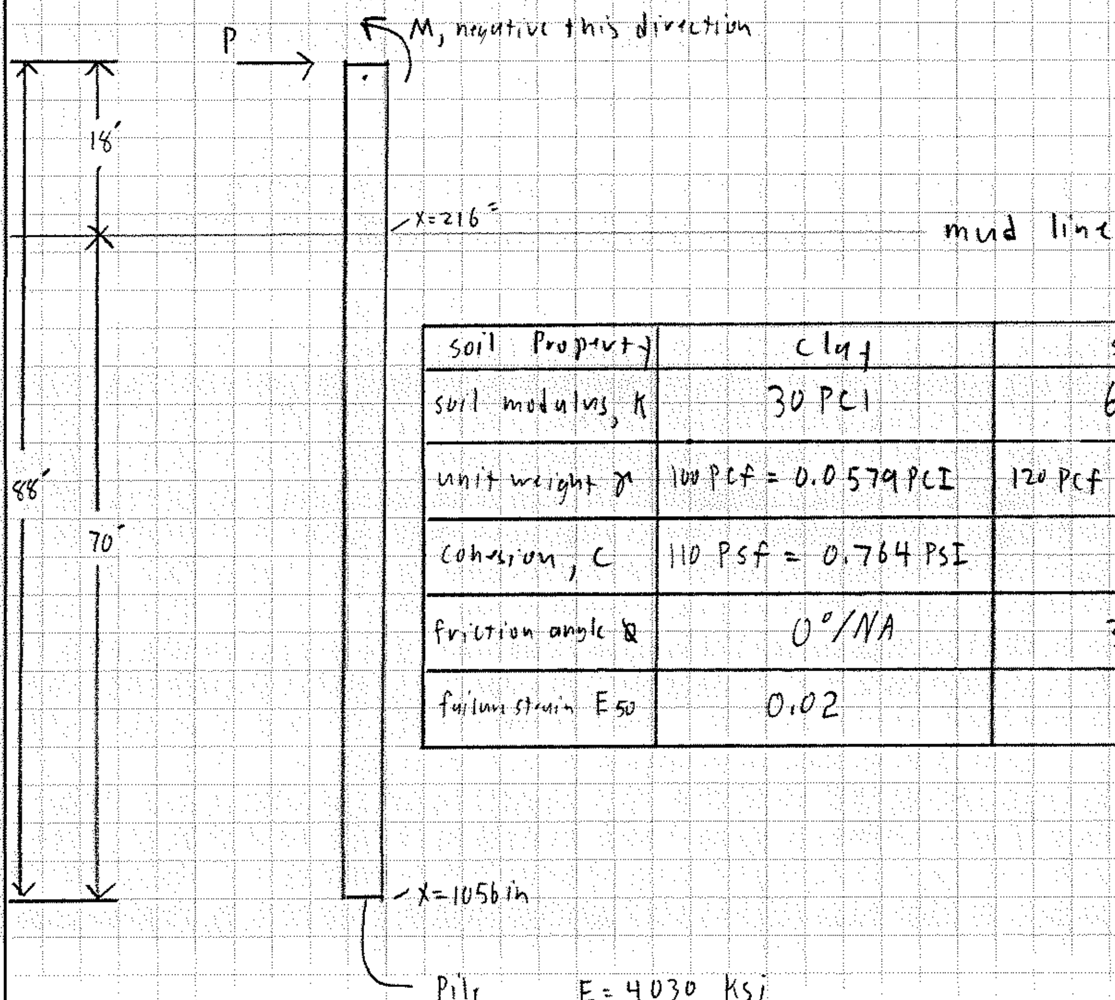
For case (b):

$$A_e = \frac{1}{2}(\theta_o - \sin\theta_o) r_o^2 - \frac{1}{2}(\theta_i - \sin\theta_i) r_i^2 \tag{13.23}$$

$$\bar{x} = \frac{\sqrt{2}}{3}\frac{(1-\cos\theta_o)^{1.5}\times r_o^3 - (1-\cos\theta_i)^{1.5}\times r_i^3}{(\theta_o - \sin\theta_o) r_o^2 - (\theta_i - \sin\theta_i) r_i^2} \tag{13.24}$$

where A_e is the effective compression area of the stress block, $\bar{x}$ is the distance from the section centroid to the centroid of the compression area, r_o is the outside radius of the section, r_i is the inside radius of the section, and the angles θs are in radians and as defined in Figs. 13.11*a* and 13.11*b*.

Assumed pile and soil layout for com 624



soil Property	clay	sand
soil modulus, K	30 PCI	60 PCI
unit weight γ	100 pcf = 0.0579 PCI	120 pcf = 0.0694 PCI
cohesion, C	110 psf = 0.764 PSI	0/NA
friction angle ϕ	0°/NA	32°
failure strain E_{50}	0.02	0/NA

Pile
$E = 4030$ ksi
Dia = 54"
$I = 233{,}000\ in^4$
$A = 770\ in^2$
$M_P = 1825$ Kft = 21,900 k·in = 0.219×10^5 Kin

Lake Ponchartrain – Lateral
Restrained Pile head in clay
MP forms at Pile head at 60.8K

54 inch diameter cylinder piles

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 = 1056.00 IN
MODULUS OF ELASTICITY OF PILE = .403E+04 KIP/IN**2
1 SECTION(S)

X	DIAMETER	MOMENT OF INERTIA	AREA
IN	IN	IN**4	IN**2
.00			
	54.000	.233E+06	.770E+03
1056.00			

SOILS INFORMATION

X-COORDINATE AT THE GROUND SURFACE = 216.00 IN

SLOPE ANGLE AT THE GROUND SURFACE = .00 DEG.

1 LAYER(S) OF SOIL

LAYER 1
THE LAYER IS A SOFT CLAY
X AT THE TOP OF THE LAYER = 216.00 IN
X AT THE BOTTOM OF THE LAYER = 1056.00 IN
VARIATION OF SOIL MODULUS, k = .300E+02 LBS/IN**3

DISTRIBUTION OF EFFECTIVE UNIT WEIGHT WITH DEPTH
2 POINTS

X,IN	WEIGHT,LBS/IN**3
216.00	.58E-01
1056.00	.58E-01

DISTRIBUTION OF STRENGTH PARAMETERS WITH DEPTH
2 POINTS

X,IN	C,LBS/IN**2	PHI,DEGREES	E50

216.00	.764E+00	.000	.200E-01
1056.00	.764E+00	.000	.200E-01

FINITE DIFFERENCE PARAMETERS
NUMBER OF PILE INCREMENTS = 88
TOLERANCE ON DETERMINATION OF DEFLECTIONS = .100E-03 IN
MAXIMUM NUMBER OF ITERATIONS ALLOWED FOR PILE ANALYSIS = 100
MAXIMUM ALLOWABLE DEFLECTION = .10E+03 IN

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

54 inch diameter cylinder piles

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 = 7

THE NUMBER OF POINTS ON EACH CURVE = 17

DEPTH BELOW GS IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
.00	54.000	.8E+00	.6E-01	.200E-01

Y,IN	P,LBS/IN
.000	.000
.022	12.377
.675	38.984
1.350	49.117
2.025	56.225
2.700	61.884
3.375	66.663
4.050	70.839
4.725	74.575
5.400	77.969
6.075	81.091
6.750	83.990
7.425	86.701
8.100	89.252
21.600	123.768
40.500	123.768

54.000	123.768

DEPTH BELOW GS IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
120.00	54.000	.8E+00	.6E-01	.200E-01

Y,IN	P,LBS/IN
.000	.000
.022	37.130
.675	116.953
1.350	147.352
2.025	168.676
2.700	185.652
3.375	199.988
4.050	212.518
4.725	223.724
5.400	233.907
6.075	243.273
6.750	251.969
7.425	260.102
8.100	267.757
21.600	371.304
40.500	371.304
54.000	371.304

DEPTH BELOW GS IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
240.00	54.000	.8E+00	.6E-01	.200E-01

Y,IN	P,LBS/IN
.000	.000
.022	37.130
.675	116.953
1.350	147.352
2.025	168.676
2.700	185.652
3.375	199.988
4.050	212.518
4.725	223.724
5.400	233.907
6.075	243.273
6.750	251.969
7.425	260.102
8.100	267.757
21.600	371.304
40.500	371.304
54.000	371.304

DEPTH BELOW GS IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
480.00	54.000	.8E+00	.6E-01	.200E-01

Y,IN	P,LBS/IN
.000	.000
.022	37.130
.675	116.953
1.350	147.352
2.025	168.676
2.700	185.652
3.375	199.988

4.050	212.518
4.725	223.724
5.400	233.907
6.075	243.273
6.750	251.969
7.425	260.102
8.100	267.757
21.600	371.304
40.500	371.304
54.000	371.304

DEPTH BELOW GS IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
600.00	54.000	.8E+00	.6E-01	.200E-01

Y,IN	P,LBS/IN
.000	.000
.022	37.130
.675	116.953
1.350	147.352
2.025	168.676
2.700	185.652
3.375	199.988
4.050	212.518
4.725	223.724
5.400	233.907
6.075	243.273
6.750	251.969
7.425	260.102
8.100	267.757
21.600	371.304
40.500	371.304
54.000	371.304

DEPTH BELOW GS IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
720.00	54.000	.8E+00	.6E-01	.200E-01

Y,IN	P,LBS/IN
.000	.000
.022	37.130
.675	116.953
1.350	147.352
2.025	168.676
2.700	185.652
3.375	199.988
4.050	212.518
4.725	223.724
5.400	233.907
6.075	243.273
6.750	251.969
7.425	260.102
8.100	267.757
21.600	371.304
40.500	371.304
54.000	371.304

DEPTH BELOW GS IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
840.00	54.000	.8E+00	.6E-01	.200E-01

54CR.OUT

Y,IN	P,LBS/IN
.000	.000
.022	37.130
.675	116.953
1.350	147.352
2.025	168.676
2.700	185.652
3.375	199.988
4.050	212.518
4.725	223.724
5.400	233.907
6.075	243.273
6.750	251.969
7.425	260.102
8.100	267.757
21.600	371.304
40.500	371.304
54.000	371.304

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PILE LOADING CONDITION

LATERAL LOAD AT PILE HEAD = .608E+02 KIP = 60.8K
SLOPE AT PILE HEAD = .000E+00 IN/IN
AXIAL LOAD AT PILE HEAD = .000E+00 KIP

X	DEFLECTION	MOMENT	TOTAL STRESS	SHEAR	SOIL RESIST	FLEXURAL RIGIDITY
IN	IN	IN-KIP	LBS/IN**2	KIP	LBS/IN	KIP-IN**2
*****	**********	**********	**********	**********	**********	**********
.00	.280E+01	-.219E+05	.254E+04	.608E+02	.000E+00	.939E+09
12.00	.280E+01	-.212E+05	.245E+04	.608E+02	.000E+00	.939E+09
24.00	.280E+01	-.204E+05	.237E+04	.608E+02	.000E+00	.939E+09
36.00	.279E+01	-.197E+05	.228E+04	.608E+02	.000E+00	.939E+09
48.00	.278E+01	-.190E+05	.220E+04	.608E+02	.000E+00	.939E+09
60.00	.277E+01	-.182E+05	.211E+04	.608E+02	.000E+00	.939E+09
72.00	.275E+01	-.175E+05	.203E+04	.608E+02	.000E+00	.939E+09
84.00	.273E+01	-.168E+05	.194E+04	.608E+02	.000E+00	.939E+09
96.00	.271E+01	-.160E+05	.186E+04	.608E+02	.000E+00	.939E+09
108.00	.268E+01	-.153E+05	.178E+04	.608E+02	.000E+00	.939E+09
120.00	.266E+01	-.146E+05	.169E+04	.608E+02	.000E+00	.939E+09
132.00	.263E+01	-.139E+05	.161E+04	.608E+02	.000E+00	.939E+09
144.00	.260E+01	-.131E+05	.152E+04	.608E+02	.000E+00	.939E+09
156.00	.256E+01	-.124E+05	.144E+04	.608E+02	.000E+00	.939E+09
168.00	.253E+01	-.117E+05	.135E+04	.608E+02	.000E+00	.939E+09
180.00	.249E+01	-.109E+05	.127E+04	.608E+02	.000E+00	.939E+09
192.00	.245E+01	-.102E+05	.118E+04	.608E+02	.000E+00	.939E+09
204.00	.241E+01	-.948E+04	.110E+04	.608E+02	.000E+00	.939E+09
216.00	.237E+01	-.875E+04	.101E+04	.604E+02	.592E+02	.939E+09
228.00	.233E+01	-.803E+04	.931E+03	.596E+02	.789E+02	.939E+09
240.00	.228E+01	-.732E+04	.848E+03	.586E+02	.983E+02	.939E+09

252.00	.224E+01	-.663E+04	.768E+03	.573E+02	.117E+03	.939E+09
264.00	.219E+01	-.595E+04	.689E+03	.557E+02	.136E+03	.939E+09
276.00	.214E+01	-.529E+04	.613E+03	.540E+02	.155E+03	.939E+09
288.00	.210E+01	-.465E+04	.539E+03	.520E+02	.171E+03	.939E+09
300.00	.205E+01	-.404E+04	.468E+03	.500E+02	.169E+03	.939E+09
312.00	.200E+01	-.345E+04	.400E+03	.480E+02	.168E+03	.939E+09
324.00	.195E+01	-.289E+04	.335E+03	.460E+02	.166E+03	.939E+09
336.00	.190E+01	-.235E+04	.272E+03	.440E+02	.165E+03	.939E+09
348.00	.185E+01	-.183E+04	.212E+03	.420E+02	.164E+03	.939E+09
360.00	.180E+01	-.134E+04	.155E+03	.401E+02	.162E+03	.939E+09
372.00	.174E+01	-.871E+03	.101E+03	.381E+02	.160E+03	.939E+09
384.00	.169E+01	-.425E+03	.493E+02	.362E+02	.159E+03	.939E+09
396.00	.164E+01	-.237E+01	.274E+00	.343E+02	.157E+03	.939E+09
408.00	.159E+01	.398E+03	.461E+02	.324E+02	.156E+03	.939E+09
420.00	.154E+01	.776E+03	.899E+02	.306E+02	.154E+03	.939E+09
432.00	.149E+01	.113E+04	.131E+03	.287E+02	.152E+03	.939E+09
444.00	.144E+01	.147E+04	.170E+03	.269E+02	.150E+03	.939E+09
456.00	.139E+01	.178E+04	.206E+03	.251E+02	.149E+03	.939E+09
468.00	.134E+01	.207E+04	.240E+03	.233E+02	.147E+03	.939E+09
480.00	.129E+01	.234E+04	.271E+03	.216E+02	.145E+03	.939E+09
492.00	.124E+01	.259E+04	.300E+03	.199E+02	.143E+03	.939E+09
504.00	.119E+01	.281E+04	.326E+03	.182E+02	.141E+03	.939E+09
516.00	.114E+01	.302E+04	.350E+03	.165E+02	.139E+03	.939E+09
528.00	.109E+01	.321E+04	.372E+03	.148E+02	.137E+03	.939E+09
540.00	.104E+01	.338E+04	.391E+03	.132E+02	.135E+03	.939E+09
552.00	.992E+00	.353E+04	.409E+03	.116E+02	.133E+03	.939E+09
564.00	.945E+00	.366E+04	.424E+03	.100E+02	.131E+03	.939E+09
576.00	.899E+00	.377E+04	.437E+03	.844E+01	.129E+03	.939E+09
588.00	.853E+00	.386E+04	.447E+03	.691E+01	.126E+03	.939E+09
600.00	.808E+00	.393E+04	.456E+03	.541E+01	.124E+03	.939E+09
612.00	.763E+00	.399E+04	.462E+03	.393E+01	.122E+03	.939E+09
624.00	.719E+00	.403E+04	.467E+03	.249E+01	.119E+03	.939E+09
636.00	.676E+00	.405E+04	.469E+03	.107E+01	.117E+03	.939E+09
648.00	.633E+00	.405E+04	.470E+03	-.322E+00	.114E+03	.939E+09
660.00	.590E+00	.404E+04	.468E+03	-.168E+01	.112E+03	.939E+09
672.00	.549E+00	.401E+04	.465E+03	-.301E+01	.109E+03	.939E+09
684.00	.508E+00	.397E+04	.460E+03	-.430E+01	.106E+03	.939E+09
696.00	.468E+00	.391E+04	.453E+03	-.556E+01	.103E+03	.939E+09
708.00	.428E+00	.384E+04	.444E+03	-.678E+01	.100E+03	.939E+09
720.00	.389E+00	.375E+04	.434E+03	-.797E+01	.973E+02	.939E+09
732.00	.350E+00	.364E+04	.422E+03	-.912E+01	.939E+02	.939E+09
744.00	.312E+00	.353E+04	.409E+03	-.102E+02	.904E+02	.939E+09
756.00	.274E+00	.340E+04	.394E+03	-.113E+02	.866E+02	.939E+09
768.00	.237E+00	.326E+04	.377E+03	-.123E+02	.825E+02	.939E+09
780.00	.201E+00	.310E+04	.360E+03	-.133E+02	.781E+02	.939E+09
792.00	.165E+00	.294E+04	.341E+03	-.142E+02	.731E+02	.939E+09
804.00	.129E+00	.276E+04	.320E+03	-.150E+02	.674E+02	.939E+09
816.00	.941E-01	.258E+04	.299E+03	-.158E+02	.606E+02	.939E+09
828.00	.594E-01	.238E+04	.276E+03	-.165E+02	.520E+02	.939E+09
840.00	.250E-01	.218E+04	.253E+03	-.170E+02	.390E+02	.939E+09
852.00	-.905E-02	.198E+04	.229E+03	-.171E+02	-.278E+02	.939E+09
864.00	-.428E-01	.177E+04	.206E+03	-.166E+02	-.467E+02	.939E+09
876.00	-.763E-01	.158E+04	.183E+03	-.160E+02	-.566E+02	.939E+09
888.00	-.109E+00	.139E+04	.161E+03	-.153E+02	-.638E+02	.939E+09
900.00	-.143E+00	.121E+04	.140E+03	-.145E+02	-.696E+02	.939E+09
912.00	-.175E+00	.104E+04	.121E+03	-.136E+02	-.746E+02	.939E+09
924.00	-.208E+00	.883E+03	.102E+03	-.127E+02	-.790E+02	.939E+09
936.00	-.241E+00	.737E+03	.854E+02	-.117E+02	-.829E+02	.939E+09
948.00	-.273E+00	.602E+03	.697E+02	-.107E+02	-.865E+02	.939E+09
960.00	-.305E+00	.479E+03	.556E+02	-.966E+01	-.898E+02	.939E+09
972.00	-.338E+00	.370E+03	.429E+02	-.856E+01	-.928E+02	.939E+09
984.00	-.370E+00	.274E+03	.318E+02	-.743E+01	-.957E+02	.939E+09
996.00	-.402E+00	.192E+03	.222E+02	-.627E+01	-.984E+02	.939E+09

1008.00	-.434E+00	.124E+03	.143E+02	-.507E+01	-.101E+03	.939E+09
1020.00	-.466E+00	.701E+02	.812E+01	-.384E+01	-.103E+03	.939E+09
1032.00	-.498E+00	.314E+02	.364E+01	-.259E+01	-.106E+03	.939E+09
1044.00	-.530E+00	.000E+00	.000E+00	.000E+00	-.108E+03	.939E+09
1056.00	-.562E+00	.000E+00	.000E+00	.000E+00	-.110E+03	.939E+09

COMPUTED LATERAL FORCE AT PILE HEAD = .60800E+02 KIP
COMPUTED SLOPE AT PILE HEAD = .18504E-16 IN/IN

THE OVERALL MOMENT IMBALANCE = -.462E-06 IN-KIP
THE OVERALL LATERAL FORCE IMBALANCE = .124E-05 LBS

OUTPUT SUMMARY

PILE HEAD DEFLECTION = .280E+01 IN
MAXIMUM BENDING MOMENT = -.219E+05 IN-KIP
MAXIMUM TOTAL STRESS = .254E+04 LBS/IN**2

NO. OF ITERATIONS = 23
MAXIMUM DEFLECTION ERROR = .782E-04 IN

S U M M A R Y   T A B L E
****

LATERAL LOAD (KIP)	BOUNDARY CONDITION BC2	AXIAL LOAD (KIP)	YT (IN)	ST (IN/IN)	MAX. MOMENT (IN-KIP)	MAX. STRESS (LBS/IN**2)
.608E+02	.000E+00	.000E+00	.280E+01	.185E-16	-.219E+05	.254E+04

Lake Pontchartrain - Lateral
Unrestrained Pile head in Clay
MP forms 27' below mud line at 112.7 K

50

54 inch diameter cylinder piles

UNITS--ENGL

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*****************************************************************
PILE DEFLECTION, BENDING MOMENT, SHEAR &  SOIL RESISTANCE
*****************************************************************
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I N P U T   I N F O R M A T I O N

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**********************************
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THE LOADING IS STATIC

PILE GEOMETRY AND PROPERTIES

```
PILE LENGTH                          =    1056.00 IN
MODULUS OF ELASTICITY OF PILE        =   .403E+04 KIP/IN**2
               1 SECTION(S)

            X           DIAMETER    MOMENT OF       AREA
                                     INERTIA
            IN             IN        IN**4          IN**2
           .00
                         54.000     .233E+06       .770E+03
        1056.00
```

SOILS INFORMATION

```
X-COORDINATE AT THE GROUND SURFACE     =     216.00 IN

SLOPE ANGLE AT THE GROUND SURFACE      =        .00 DEG.

1 LAYER(S) OF SOIL

LAYER  1
THE LAYER IS A SOFT CLAY
X AT THE TOP OF THE LAYER        =     216.00 IN
X AT THE BOTTOM OF THE LAYER     =    1056.00 IN
VARIATION OF SOIL MODULUS, k     =   .300E+02 LBS/IN**3


DISTRIBUTION OF EFFECTIVE UNIT WEIGHT WITH DEPTH
                  2 POINTS
            X,IN     WEIGHT,LBS/IN**3
          216.00         .58E-01
         1056.00         .58E-01

DISTRIBUTION OF STRENGTH PARAMETERS WITH DEPTH
                  2 POINTS
            X,IN        C,LBS/IN**2      PHI,DEGREES      E50
```

216.00	.764E+00	.000	.200E-01
1056.00	.764E+00	.000	.200E-01

```
FINITE DIFFERENCE PARAMETERS
    NUMBER OF PILE INCREMENTS                                  =        88
    TOLERANCE ON DETERMINATION OF DEFLECTIONS                  =   .100E-03 IN
    MAXIMUM NUMBER OF ITERATIONS ALLOWED FOR PILE ANALYSIS     =       100
    MAXIMUM ALLOWABLE DEFLECTION                               =    .10E+03 IN

INPUT CODES
    OUTPT  =  1
    KCYCL  =  1
    KBC    =  1
    KPYOP  =  1
    INC    =  1
```

54 inch diameter cylinder piles

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 = 7

THE NUMBER OF POINTS ON EACH CURVE = 17

DEPTH BELOW GS IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
.00	54.000	.8E+00	.6E-01	.200E-01

Y,IN	P,LBS/IN
.000	.000
.022	12.377
.675	38.984
1.350	49.117
2.025	56.225
2.700	61.884
3.375	66.663
4.050	70.839
4.725	74.575
5.400	77.969
6.075	81.091
6.750	83.990
7.425	86.701
8.100	89.252
21.600	123.768
40.500	123.768

54.000	123.768

DEPTH BELOW GS IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
120.00	54.000	.8E+00	.6E-01	.200E-01

Y,IN	P,LBS/IN
.000	.000
.022	37.130
.675	116.953
1.350	147.352
2.025	168.676
2.700	185.652
3.375	199.988
4.050	212.518
4.725	223.724
5.400	233.907
6.075	243.273
6.750	251.969
7.425	260.102
8.100	267.757
21.600	371.304
40.500	371.304
54.000	371.304

DEPTH BELOW GS IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
240.00	54.000	.8E+00	.6E-01	.200E-01

Y,IN	P,LBS/IN
.000	.000
.022	37.130
.675	116.953
1.350	147.352
2.025	168.676
2.700	185.652
3.375	199.988
4.050	212.518
4.725	223.724
5.400	233.907
6.075	243.273
6.750	251.969
7.425	260.102
8.100	267.757
21.600	371.304
40.500	371.304
54.000	371.304

DEPTH BELOW GS IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
480.00	54.000	.8E+00	.6E-01	.200E-01

Y,IN	P,LBS/IN
.000	.000
.022	37.130
.675	116.953
1.350	147.352
2.025	168.676
2.700	185.652
3.375	199.988

4.050	212.518
4.725	223.724
5.400	233.907
6.075	243.273
6.750	251.969
7.425	260.102
8.100	267.757
21.600	371.304
40.500	371.304
54.000	371.304

DEPTH BELOW GS IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
600.00	54.000	.8E+00	.6E-01	.200E-01

Y,IN	P,LBS/IN
.000	.000
.022	37.130
.675	116.953
1.350	147.352
2.025	168.676
2.700	185.652
3.375	199.988
4.050	212.518
4.725	223.724
5.400	233.907
6.075	243.273
6.750	251.969
7.425	260.102
8.100	267.757
21.600	371.304
40.500	371.304
54.000	371.304

DEPTH BELOW GS IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
720.00	54.000	.8E+00	.6E-01	.200E-01

Y,IN	P,LBS/IN
.000	.000
.022	37.130
.675	116.953
1.350	147.352
2.025	168.676
2.700	185.652
3.375	199.988
4.050	212.518
4.725	223.724
5.400	233.907
6.075	243.273
6.750	251.969
7.425	260.102
8.100	267.757
21.600	371.304
40.500	371.304
54.000	371.304

DEPTH BELOW GS IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
840.00	54.000	.8E+00	.6E-01	.200E-01

54CU.OUT

Y,IN	P,LBS/IN
.000	.000
.022	37.130
.675	116.953
1.350	147.352
2.025	168.676
2.700	185.652
3.375	199.988
4.050	212.518
4.725	223.724
5.400	233.907
6.075	243.273
6.750	251.969
7.425	260.102
8.100	267.757
21.600	371.304
40.500	371.304
54.000	371.304

**** WARNING ****
THE SOLUTION DID NOT CONVERGE
MAXIMUM DEFLECTION ERROR = .141E-02

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PILE LOADING CONDITION

LATERAL LOAD AT PILE HEAD = .113E+03 KIP — 112.7 K
APPLIED MOMENT AT PILE HEAD = -.219E+05 IN-KIP
AXIAL LOAD AT PILE HEAD = .000E+00 KIP

X IN	DEFLECTION IN	MOMENT IN-KIP	TOTAL STRESS LBS/IN**2	SHEAR KIP	SOIL RESIST LBS/IN	FLEXURAL RIGIDITY KIP-IN**2
*****	**********	**********	**********	**********	**********	**********
.00	.807E+02	-.219E+05	.254E+04	.113E+03	.000E+00	.939E+09
12.00	.795E+02	-.205E+05	.238E+04	.113E+03	.000E+00	.939E+09
24.00	.782E+02	-.192E+05	.222E+04	.113E+03	.000E+00	.939E+09
36.00	.770E+02	-.178E+05	.207E+04	.113E+03	.000E+00	.939E+09
48.00	.758E+02	-.165E+05	.191E+04	.113E+03	.000E+00	.939E+09
60.00	.745E+02	-.151E+05	.175E+04	.113E+03	.000E+00	.939E+09
72.00	.733E+02	-.138E+05	.160E+04	.113E+03	.000E+00	.939E+09
84.00	.720E+02	-.124E+05	.144E+04	.113E+03	.000E+00	.939E+09
96.00	.708E+02	-.111E+05	.128E+04	.113E+03	.000E+00	.939E+09
108.00	.695E+02	-.973E+04	.113E+04	.113E+03	.000E+00	.939E+09
120.00	.683E+02	-.838E+04	.971E+03	.113E+03	.000E+00	.939E+09
132.00	.670E+02	-.702E+04	.814E+03	.113E+03	.000E+00	.939E+09
144.00	.658E+02	-.567E+04	.657E+03	.113E+03	.000E+00	.939E+09
156.00	.645E+02	-.432E+04	.500E+03	.113E+03	.000E+00	.939E+09
168.00	.633E+02	-.297E+04	.344E+03	.113E+03	.000E+00	.939E+09

180.00	.620E+02	-.161E+04	.187E+03	.113E+03	.000E+00	.939E+09
192.00	.607E+02	-.262E+03	.303E+02	.113E+03	.000E+00	.939E+09
204.00	.595E+02	.109E+04	.126E+03	.113E+03	.000E+00	.939E+09
216.00	.582E+02	.244E+04	.283E+03	.112E+03	.124E+03	.939E+09
228.00	.570E+02	.378E+04	.438E+03	.110E+03	.166E+03	.939E+09
240.00	.557E+02	.509E+04	.590E+03	.108E+03	.208E+03	.939E+09
252.00	.545E+02	.637E+04	.738E+03	.105E+03	.250E+03	.939E+09
264.00	.532E+02	.761E+04	.882E+03	.102E+03	.292E+03	.939E+09
276.00	.519E+02	.882E+04	.102E+04	.982E+02	.334E+03	.939E+09
288.00	.507E+02	.997E+04	.116E+04	.940E+02	.371E+03	.939E+09
300.00	.494E+02	.111E+05	.128E+04	.895E+02	.371E+03	.939E+09
312.00	.482E+02	.121E+05	.140E+04	.851E+02	.371E+03	.939E+09
324.00	.470E+02	.131E+05	.152E+04	.806E+02	.371E+03	.939E+09
336.00	.457E+02	.141E+05	.163E+04	.762E+02	.371E+03	.939E+09
348.00	.445E+02	.149E+05	.173E+04	.717E+02	.371E+03	.939E+09
360.00	.432E+02	.158E+05	.183E+04	.672E+02	.371E+03	.939E+09
372.00	.420E+02	.166E+05	.192E+04	.628E+02	.371E+03	.939E+09
384.00	.408E+02	.173E+05	.200E+04	.583E+02	.371E+03	.939E+09
396.00	.395E+02	.180E+05	.208E+04	.539E+02	.371E+03	.939E+09
408.00	.383E+02	.186E+05	.215E+04	.494E+02	.371E+03	.939E+09
420.00	.371E+02	.191E+05	.222E+04	.450E+02	.371E+03	.939E+09
432.00	.358E+02	.197E+05	.228E+04	.405E+02	.371E+03	.939E+09
444.00	.346E+02	.201E+05	.233E+04	.361E+02	.371E+03	.939E+09
456.00	.334E+02	.205E+05	.238E+04	.316E+02	.371E+03	.939E+09
468.00	.322E+02	.209E+05	.242E+04	.271E+02	.371E+03	.939E+09
480.00	.310E+02	.212E+05	.245E+04	.227E+02	.371E+03	.939E+09
492.00	.298E+02	.214E+05	.248E+04	.182E+02	.371E+03	.939E+09
504.00	.286E+02	.216E+05	.250E+04	.138E+02	.371E+03	.939E+09
516.00	.274E+02	.217E+05	.252E+04	.932E+01	.371E+03	.939E+09
528.00	.262E+02	.218E+05	.253E+04	.487E+01	.371E+03	.939E+09
.7' below mudline $\boxed{540.00}$	.250E+02	$\boxed{.219E+05}$	.253E+04	.412E+00	.371E+03	.939E+09
552.00	.238E+02	.218E+05	.253E+04	-.404E+01	.371E+03	.939E+09
564.00	.226E+02	.218E+05	.252E+04	-.850E+01	.371E+03	.939E+09
576.00	.214E+02	.216E+05	.251E+04	-.129E+02	.370E+03	.939E+09
588.00	.202E+02	.215E+05	.249E+04	-.173E+02	.363E+03	.939E+09
600.00	.190E+02	.212E+05	.246E+04	-.217E+02	.356E+03	.939E+09
612.00	.179E+02	.209E+05	.243E+04	-.259E+02	.349E+03	.939E+09
624.00	.167E+02	.206E+05	.239E+04	-.300E+02	.341E+03	.939E+09
636.00	.155E+02	.202E+05	.234E+04	-.341E+02	.333E+03	.939E+09
648.00	.144E+02	.198E+05	.229E+04	-.380E+02	.324E+03	.939E+09
660.00	.132E+02	.193E+05	.224E+04	-.418E+02	.315E+03	.939E+09
672.00	.120E+02	.188E+05	.218E+04	-.456E+02	.306E+03	.939E+09
684.00	.109E+02	.182E+05	.211E+04	-.492E+02	.295E+03	.939E+09
696.00	.972E+01	.176E+05	.204E+04	-.526E+02	.285E+03	.939E+09
708.00	.857E+01	.169E+05	.196E+04	-.560E+02	.273E+03	.939E+09
720.00	.742E+01	.163E+05	.188E+04	-.592E+02	.260E+03	.939E+09
732.00	.628E+01	.155E+05	.180E+04	-.622E+02	.246E+03	.939E+09
744.00	.513E+01	.148E+05	.171E+04	-.651E+02	.230E+03	.939E+09
756.00	.399E+01	.140E+05	.162E+04	-.677E+02	.211E+03	.939E+09
768.00	.285E+01	.131E+05	.152E+04	-.701E+02	.189E+03	.939E+09
780.00	.171E+01	.123E+05	.142E+04	-.722E+02	.160E+03	.939E+09
792.00	.578E+00	.114E+05	.132E+04	-.739E+02	.111E+03	.939E+09
804.00	-.556E+00	.105E+05	.122E+04	-.739E+02	-.110E+03	.939E+09
816.00	-.169E+01	.963E+04	.112E+04	-.722E+02	-.159E+03	.939E+09
828.00	-.282E+01	.877E+04	.102E+04	-.702E+02	-.188E+03	.939E+09
840.00	-.395E+01	.795E+04	.921E+03	-.678E+02	-.211E+03	.939E+09
852.00	-.508E+01	.715E+04	.828E+03	-.651E+02	-.229E+03	.939E+09
864.00	-.620E+01	.638E+04	.740E+03	-.623E+02	-.245E+03	.939E+09
876.00	-.733E+01	.565E+04	.655E+03	-.593E+02	-.259E+03	.939E+09
888.00	-.845E+01	.496E+04	.575E+03	-.561E+02	-.272E+03	.939E+09
900.00	-.958E+01	.431E+04	.499E+03	-.528E+02	-.283E+03	.939E+09
912.00	-.107E+02	.369E+04	.428E+03	-.493E+02	-.294E+03	.939E+09
924.00	-.118E+02	.312E+04	.362E+03	-.457E+02	-.304E+03	.939E+09

936.00	-.129E+02	.260E+04	.301E+03	-.420E+02	-.313E+03	.939E+09
948.00	-.141E+02	.212E+04	.245E+03	-.382E+02	-.322E+03	.939E+09
960.00	-.152E+02	.168E+04	.195E+03	-.343E+02	-.330E+03	.939E+09
972.00	-.163E+02	.129E+04	.150E+03	-.303E+02	-.338E+03	.939E+09
984.00	-.174E+02	.955E+03	.111E+03	-.262E+02	-.346E+03	.939E+09
996.00	-.186E+02	.665E+03	.771E+02	-.220E+02	-.353E+03	.939E+09
1008.00	-.197E+02	.427E+03	.495E+02	-.177E+02	-.360E+03	.939E+09
1020.00	-.208E+02	.241E+03	.279E+02	-.133E+02	-.367E+03	.939E+09
1032.00	-.219E+02	.107E+03	.124E+02	-.891E+01	-.371E+03	.939E+09
1044.00	-.230E+02	.000E+00	.000E+00	.000E+00	-.371E+03	.939E+09
1056.00	-.242E+02	.000E+00	.000E+00	.000E+00	-.371E+03	.939E+09

COMPUTED LATERAL FORCE AT PILE HEAD = .11270E+03 KIP
COMPUTED MOMENT AT PILE HEAD = -.21900E+05 IN-KIP
COMPUTED SLOPE AT PILE HEAD = -.10249E+00

THE OVERALL MOMENT IMBALANCE = -.884E+00 IN-KIP
THE OVERALL LATERAL FORCE IMBALANCE = .254E+01 LBS

OUTPUT SUMMARY

PILE HEAD DEFLECTION = .807E+02 IN
MAXIMUM BENDING MOMENT = -.219E+05 IN-KIP
MAXIMUM TOTAL STRESS = .254E+04 LBS/IN**2

NO. OF ITERATIONS = 100
MAXIMUM DEFLECTION ERROR = .141E-02 IN

S U M M A R Y   T A B L E

****

LATERAL LOAD (KIP)	BOUNDARY CONDITION BC2	AXIAL LOAD (KIP)	YT (IN)	ST (IN/IN)	MAX. MOMENT (IN-KIP)	MAX. STRESS (LBS/IN**2)
.113E+03	-.219E+05	.000E+00	.807E+02	-.102E+00	-.219E+05	.254E+04

Lake Y longitudinal
Unrestrained pile head
clay 57
My forms at 650K
19 - below mudline

54 inch diameter cylinder piles

UNITS--ENGL

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*************************************************************
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                        =     1056.00 IN
    MODULUS OF ELASTICITY OF PILE      =    .403E+04 KIP/IN**2
                   1 SECTION(S)
```

X IN	DIAMETER IN	MOMENT OF INERTIA IN**4	AREA IN**2
.00			
	54.000	.233E+06	.770E+03
1056.00			

SOILS INFORMATION

```
    X-COORDINATE AT THE GROUND SURFACE     =      216.00 IN

    SLOPE ANGLE AT THE GROUND SURFACE      =         .00 DEG.

    1 LAYER(S) OF SOIL

    LAYER  1
    THE LAYER IS A SOFT CLAY
    X AT THE TOP OF THE LAYER          =     216.00 IN
    X AT THE BOTTOM OF THE LAYER       =    1056.00 IN
    VARIATION OF SOIL MODULUS, k       =   .300E+02 LBS/IN**3
```

DISTRIBUTION OF EFFECTIVE UNIT WEIGHT WITH DEPTH
2 POINTS

X,IN	WEIGHT,LBS/IN**3
216.00	.58E-01
1056.00	.58E-01

DISTRIBUTION OF STRENGTH PARAMETERS WITH DEPTH
2 POINTS

X,IN	C,LBS/IN**2	PHI,DEGREES	E50

216.00	.764E+00	.000	.200E-01
1056.00	.764E+00	.000	.200E-01

```
FINITE DIFFERENCE PARAMETERS
    NUMBER OF PILE INCREMENTS                              =       88
    TOLERANCE ON DETERMINATION OF DEFLECTIONS              =  .100E-03 IN
    MAXIMUM NUMBER OF ITERATIONS ALLOWED FOR PILE ANALYSIS =      100
    MAXIMUM ALLOWABLE DEFLECTION                           =   .10E+03 IN

INPUT CODES
    OUTPT  =  1
    KCYCL  =  1
    KBC    =  1
    KPYOP  =  1
    INC    =  1
```

54 inch diameter cylinder piles

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 = 7

THE NUMBER OF POINTS ON EACH CURVE = 17

DEPTH BELOW GS IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
.00	54.000	.8E+00	.6E-01	.200E-01

Y,IN	P,LBS/IN
.000	.000
.022	12.377
.675	38.984
1.350	49.117
2.025	56.225
2.700	61.884
3.375	66.663
4.050	70.839
4.725	74.575
5.400	77.969
6.075	81.091
6.750	83.990
7.425	86.701
8.100	89.252
21.600	123.768
40.500	123.768

54.000	123.768

DEPTH BELOW GS IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
120.00	54.000	.8E+00	.6E-01	.200E-01

Y,IN	P,LBS/IN
.000	.000
.022	37.130
.675	116.953
1.350	147.352
2.025	168.676
2.700	185.652
3.375	199.988
4.050	212.518
4.725	223.724
5.400	233.907
6.075	243.273
6.750	251.969
7.425	260.102
8.100	267.757
21.600	371.304
40.500	371.304
54.000	371.304

DEPTH BELOW GS IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
240.00	54.000	.8E+00	.6E-01	.200E-01

Y,IN	P,LBS/IN
.000	.000
.022	37.130
.675	116.953
1.350	147.352
2.025	168.676
2.700	185.652
3.375	199.988
4.050	212.518
4.725	223.724
5.400	233.907
6.075	243.273
6.750	251.969
7.425	260.102
8.100	267.757
21.600	371.304
40.500	371.304
54.000	371.304

DEPTH BELOW GS IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
480.00	54.000	.8E+00	.6E-01	.200E-01

Y,IN	P,LBS/IN
.000	.000
.022	37.130
.675	116.953
1.350	147.352
2.025	168.676
2.700	185.652
3.375	199.988

4.050	212.518
4.725	223.724
5.400	233.907
6.075	243.273
6.750	251.969
7.425	260.102
8.100	267.757
21.600	371.304
40.500	371.304
54.000	371.304

DEPTH BELOW GS IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
600.00	54.000	.8E+00	.6E-01	.200E-01

Y,IN	P,LBS/IN
.000	.000
.022	37.130
.675	116.953
1.350	147.352
2.025	168.676
2.700	185.652
3.375	199.988
4.050	212.518
4.725	223.724
5.400	233.907
6.075	243.273
6.750	251.969
7.425	260.102
8.100	267.757
21.600	371.304
40.500	371.304
54.000	371.304

DEPTH BELOW GS IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
720.00	54.000	.8E+00	.6E-01	.200E-01

Y,IN	P,LBS/IN
.000	.000
.022	37.130
.675	116.953
1.350	147.352
2.025	168.676
2.700	185.652
3.375	199.988
4.050	212.518
4.725	223.724
5.400	233.907
6.075	243.273
6.750	251.969
7.425	260.102
8.100	267.757
21.600	371.304
40.500	371.304
54.000	371.304

DEPTH BELOW GS IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
840.00	54.000	.8E+00	.6E-01	.200E-01

Y,IN	P,LBS/IN
.000	.000
.022	37.130
.675	116.953
1.350	147.352
2.025	168.676
2.700	185.652
3.375	199.988
4.050	212.518
4.725	223.724
5.400	233.907
6.075	243.273
6.750	251.969
7.425	260.102
8.100	267.757
21.600	371.304
40.500	371.304
54.000	371.304

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PILE LOADING CONDITION

LATERAL LOAD AT PILE HEAD = .650E+02 KIP = 65.0k
APPLIED MOMENT AT PILE HEAD = .000E+00 IN-KIP
AXIAL LOAD AT PILE HEAD = .000E+00 KIP

X	DEFLECTION	MOMENT	TOTAL STRESS	SHEAR	SOIL RESIST	FLEXURAL RIGIDITY
IN	IN	IN-KIP	LBS/IN**2	KIP	LBS/IN	KIP-IN**2
*****	**********	**********	**********	**********	**********	**********
.00	.261E+02	.000E+00	.000E+00	.650E+02	.000E+00	.939E+09
12.00	.257E+02	.780E+03	.904E+02	.650E+02	.000E+00	.939E+09
24.00	.252E+02	.156E+04	.181E+03	.650E+02	.000E+00	.939E+09
36.00	.247E+02	.234E+04	.271E+03	.650E+02	.000E+00	.939E+09
48.00	.242E+02	.312E+04	.362E+03	.650E+02	.000E+00	.939E+09
60.00	.237E+02	.390E+04	.452E+03	.650E+02	.000E+00	.939E+09
72.00	.233E+02	.468E+04	.542E+03	.650E+02	.000E+00	.939E+09
84.00	.228E+02	.546E+04	.633E+03	.650E+02	.000E+00	.939E+09
96.00	.223E+02	.624E+04	.723E+03	.650E+02	.000E+00	.939E+09
108.00	.218E+02	.702E+04	.813E+03	.650E+02	.000E+00	.939E+09
120.00	.213E+02	.780E+04	.904E+03	.650E+02	.000E+00	.939E+09
132.00	.209E+02	.858E+04	.994E+03	.650E+02	.000E+00	.939E+09
144.00	.204E+02	.936E+04	.108E+04	.650E+02	.000E+00	.939E+09
156.00	.199E+02	.101E+05	.118E+04	.650E+02	.000E+00	.939E+09
168.00	.195E+02	.109E+05	.127E+04	.650E+02	.000E+00	.939E+09
180.00	.190E+02	.117E+05	.136E+04	.650E+02	.000E+00	.939E+09
192.00	.185E+02	.125E+05	.145E+04	.650E+02	.000E+00	.939E+09
204.00	.181E+02	.133E+05	.154E+04	.650E+02	.000E+00	.939E+09
216.00	.176E+02	.140E+05	.163E+04	.643E+02	.116E+03	.939E+09
228.00	.171E+02	.148E+05	.172E+04	.627E+02	.154E+03	.939E+09
240.00	.167E+02	.155E+05	.180E+04	.606E+02	.191E+03	.939E+09

'below mudline

252.00	.162E+02	.163E+05	.188E+04	.581E+02	.227E+03	.939E+09
264.00	.158E+02	.169E+05	.196E+04	.552E+02	.263E+03	.939E+09
276.00	.153E+02	.176E+05	.204E+04	.518E+02	.298E+03	.939E+09
288.00	.149E+02	.182E+05	.211E+04	.481E+02	.328E+03	.939E+09
300.00	.144E+02	.187E+05	.217E+04	.441E+02	.324E+03	.939E+09
312.00	.140E+02	.192E+05	.223E+04	.403E+02	.321E+03	.939E+09
324.00	.135E+02	.197E+05	.228E+04	.364E+02	.318E+03	.939E+09
336.00	.131E+02	.201E+05	.233E+04	.326E+02	.314E+03	.939E+09
348.00	.127E+02	.205E+05	.237E+04	.289E+02	.311E+03	.939E+09
360.00	.122E+02	.208E+05	.241E+04	.252E+02	.307E+03	.939E+09
372.00	.118E+02	.211E+05	.244E+04	.215E+02	.304E+03	.939E+09
384.00	.114E+02	.213E+05	.247E+04	.179E+02	.300E+03	.939E+09
396.00	.110E+02	.215E+05	.249E+04	.143E+02	.296E+03	.939E+09
408.00	.105E+02	.217E+05	.251E+04	.108E+02	.292E+03	.939E+09
420.00	.101E+02	.218E+05	.252E+04	.731E+01	.288E+03	.939E+09
432.00	.971E+01	.218E+05	.253E+04	.387E+01	.284E+03	.939E+09
444.00	.930E+01	.219E+05	.253E+04	.485E+00	.280E+03	.939E+09
456.00	.890E+01	.219E+05	.253E+04	-.286E+01	.276E+03	.939E+09
468.00	.850E+01	.218E+05	.253E+04	-.615E+01	.272E+03	.939E+09
480.00	.810E+01	.217E+05	.252E+04	-.938E+01	.268E+03	.939E+09
492.00	.771E+01	.216E+05	.250E+04	-.126E+02	.263E+03	.939E+09
504.00	.731E+01	.214E+05	.248E+04	-.157E+02	.259E+03	.939E+09
516.00	.693E+01	.212E+05	.246E+04	-.188E+02	.254E+03	.939E+09
528.00	.654E+01	.210E+05	.243E+04	-.218E+02	.249E+03	.939E+09
540.00	.616E+01	.207E+05	.240E+04	-.248E+02	.244E+03	.939E+09
552.00	.578E+01	.204E+05	.236E+04	-.277E+02	.239E+03	.939E+09
564.00	.541E+01	.200E+05	.232E+04	-.305E+02	.234E+03	.939E+09
576.00	.503E+01	.196E+05	.227E+04	-.333E+02	.229E+03	.939E+09
588.00	.467E+01	.192E+05	.223E+04	-.360E+02	.223E+03	.939E+09
600.00	.430E+01	.188E+05	.217E+04	-.386E+02	.217E+03	.939E+09
612.00	.394E+01	.183E+05	.212E+04	-.412E+02	.211E+03	.939E+09
624.00	.358E+01	.178E+05	.206E+04	-.437E+02	.204E+03	.939E+09
636.00	.322E+01	.172E+05	.200E+04	-.461E+02	.197E+03	.939E+09
648.00	.286E+01	.167E+05	.193E+04	-.484E+02	.189E+03	.939E+09
660.00	.251E+01	.161E+05	.186E+04	-.506E+02	.181E+03	.939E+09
672.00	.216E+01	.155E+05	.179E+04	-.527E+02	.172E+03	.939E+09
684.00	.181E+01	.148E+05	.172E+04	-.548E+02	.163E+03	.939E+09
696.00	.147E+01	.141E+05	.164E+04	-.566E+02	.151E+03	.939E+09
708.00	.112E+01	.135E+05	.156E+04	-.584E+02	.139E+03	.939E+09
720.00	.783E+00	.127E+05	.148E+04	-.599E+02	.123E+03	.939E+09
732.00	.443E+00	.120E+05	.139E+04	-.613E+02	.102E+03	.939E+09
744.00	.106E+00	.113E+05	.131E+04	-.623E+02	.631E+02	.939E+09
756.00	-.230E+00	.105E+05	.122E+04	-.622E+02	-.816E+02	.939E+09
768.00	-.564E+00	.978E+04	.113E+04	-.610E+02	-.110E+03	.939E+09
780.00	-.896E+00	.906E+04	.105E+04	-.596E+02	-.129E+03	.939E+09
792.00	-.123E+01	.835E+04	.968E+03	-.580E+02	-.143E+03	.939E+09
804.00	-.156E+01	.766E+04	.888E+03	-.562E+02	-.155E+03	.939E+09
816.00	-.189E+01	.700E+04	.811E+03	-.543E+02	-.165E+03	.939E+09
828.00	-.221E+01	.636E+04	.737E+03	-.522E+02	-.174E+03	.939E+09
840.00	-.254E+01	.575E+04	.666E+03	-.501E+02	-.182E+03	.939E+09
852.00	-.287E+01	.516E+04	.598E+03	-.479E+02	-.189E+03	.939E+09
864.00	-.319E+01	.460E+04	.533E+03	-.456E+02	-.196E+03	.939E+09
876.00	-.352E+01	.407E+04	.471E+03	-.432E+02	-.203E+03	.939E+09
888.00	-.384E+01	.356E+04	.413E+03	-.407E+02	-.209E+03	.939E+09
900.00	-.416E+01	.309E+04	.358E+03	-.382E+02	-.214E+03	.939E+09
912.00	-.448E+01	.265E+04	.307E+03	-.355E+02	-.220E+03	.939E+09
924.00	-.481E+01	.224E+04	.259E+03	-.329E+02	-.225E+03	.939E+09
936.00	-.513E+01	.186E+04	.215E+03	-.301E+02	-.230E+03	.939E+09
948.00	-.545E+01	.151E+04	.175E+03	-.274E+02	-.235E+03	.939E+09
960.00	-.577E+01	.120E+04	.139E+03	-.245E+02	-.239E+03	.939E+09
972.00	-.609E+01	.925E+03	.107E+03	-.216E+02	-.244E+03	.939E+09
984.00	-.641E+01	.683E+03	.791E+02	-.187E+02	-.248E+03	.939E+09
996.00	-.673E+01	.476E+03	.552E+02	-.157E+02	-.252E+03	.939E+09

1008.00	-.706E+01	.306E+03	.355E+02	-.126E+02	-.256E+03	.939E+09
1020.00	-.738E+01	.173E+03	.201E+02	-.954E+01	-.260E+03	.939E+09
1032.00	-.770E+01	.774E+02	.896E+01	-.640E+01	-.263E+03	.939E+09
1044.00	-.802E+01	.000E+00	.000E+00	.000E+00	-.267E+03	.939E+09
1056.00	-.834E+01	.000E+00	.000E+00	.000E+00	-.270E+03	.939E+09

COMPUTED LATERAL FORCE AT PILE HEAD = .65000E+02 KIP
COMPUTED MOMENT AT PILE HEAD = .00000E+00 IN-KIP
COMPUTED SLOPE AT PILE HEAD = -.40137E-01

THE OVERALL MOMENT IMBALANCE = .207E-05 IN-KIP
THE OVERALL LATERAL FORCE IMBALANCE = -.737E-05 LBS

OUTPUT SUMMARY

PILE HEAD DEFLECTION = .261E+02 IN
MAXIMUM BENDING MOMENT = .219E+05 IN-KIP
MAXIMUM TOTAL STRESS = .253E+04 LBS/IN**2

NO. OF ITERATIONS = 35
MAXIMUM DEFLECTION ERROR = .741E-04 IN

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LATERAL LOAD (KIP)	BOUNDARY CONDITION BC2	AXIAL LOAD (KIP)	YT (IN)	ST (IN/IN)	MAX. MOMENT (IN-KIP)	MAX. STRESS (LBS/IN**2)
.650E+02	.000E+00	.000E+00	.261E+02	-.401E-01	.219E+05	.253E+04