Working Paper

Research Project GC 8719, Task 16
Ferry Landing Design

WINGWALL FIELD TESTING

by

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DISCLAIMER

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RESULTS OF PULL TESTS ON SOUTH WINGWALL AT EDMONDS TERMINAL

INTRODUCTION

As part of a project to develop criteria for ferry landings, the researchers conducted a pull test on the south wingwall at the Edmonds Ferry Terminal (Figures 1 and 2). The goal of the pull test was to find the force versus deflection relationship $(g(s))$ for this wingwall. The relationship will help in estimating the berthing energy that results from the ferry's berthing maneuvers.

Wingwalls act to transform or store a given amount of kinetic energy, which is a function of a given deflection. The deflections that result from the application of known forces may be measured to define the force versus deflection relationship.

$$F = g(s) \qquad \text{(Equation 1)}$$

where F = Force

$g(\,)$ = force vs. deflection relationship

s = deflection

Integration of this relationship provides a measure of the energy absorbed by the wall. That is, if $g(s)$ is the force versus deflection relationship, then the energy that is absorbed for a given wingwall deflection is as follows:

$$DE = h(s) \int_0^{s_{max}} g(s)ds \qquad \text{(Equation 2)}$$

where $h(s)$ = energy versus deflection relationship

If s is known, then $h(s)$ can be used to estimate the berthing energy for future landings.

This report describes the methodology and results of the test. Although the wall could not be calibrated for the full deflection that occurs during many berthing maneuvers, $g(s)$ was defined for deflections of up to 4 inches. The problem is that the relationship varies with the point of load application: the wingwall is stiffer in the middle

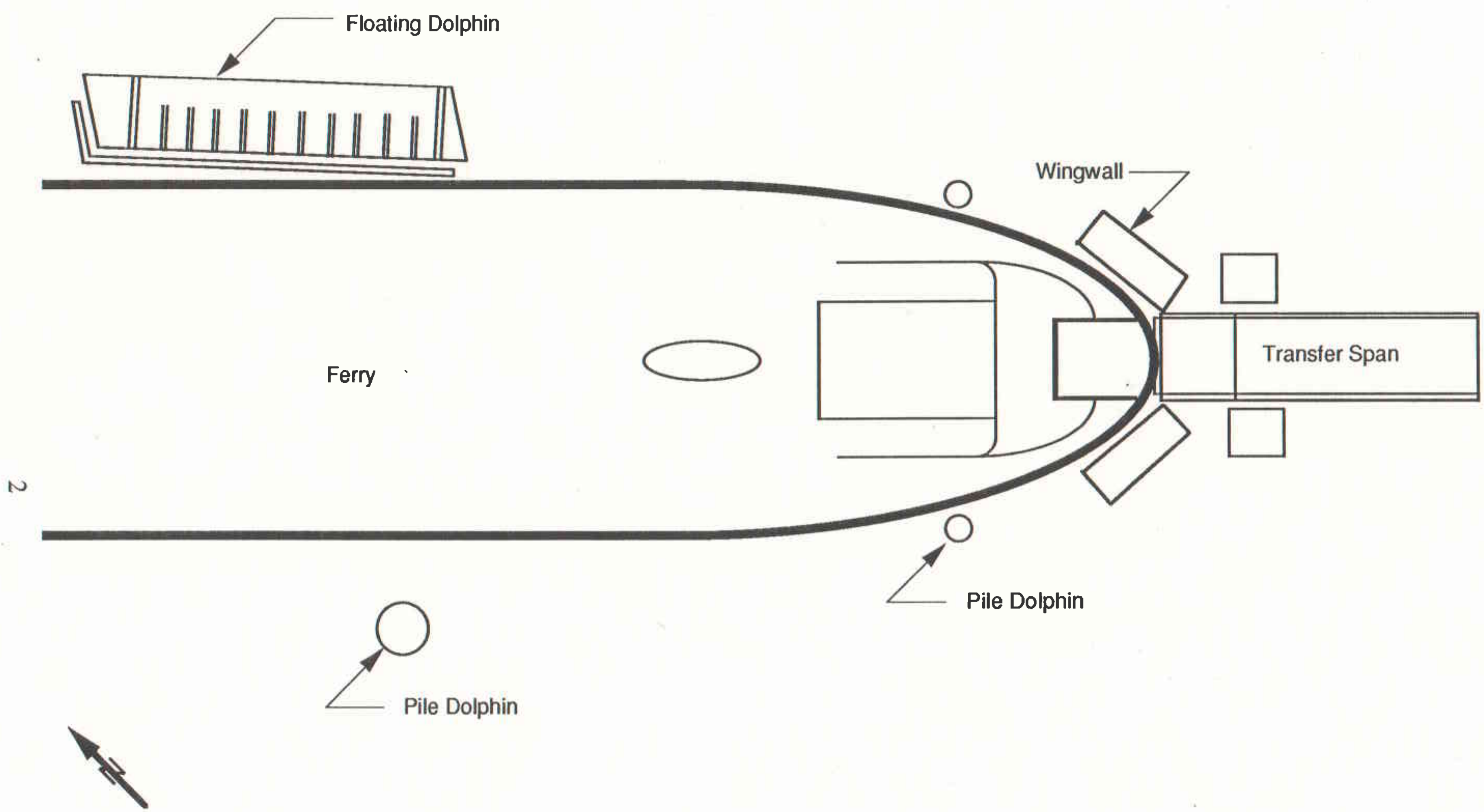
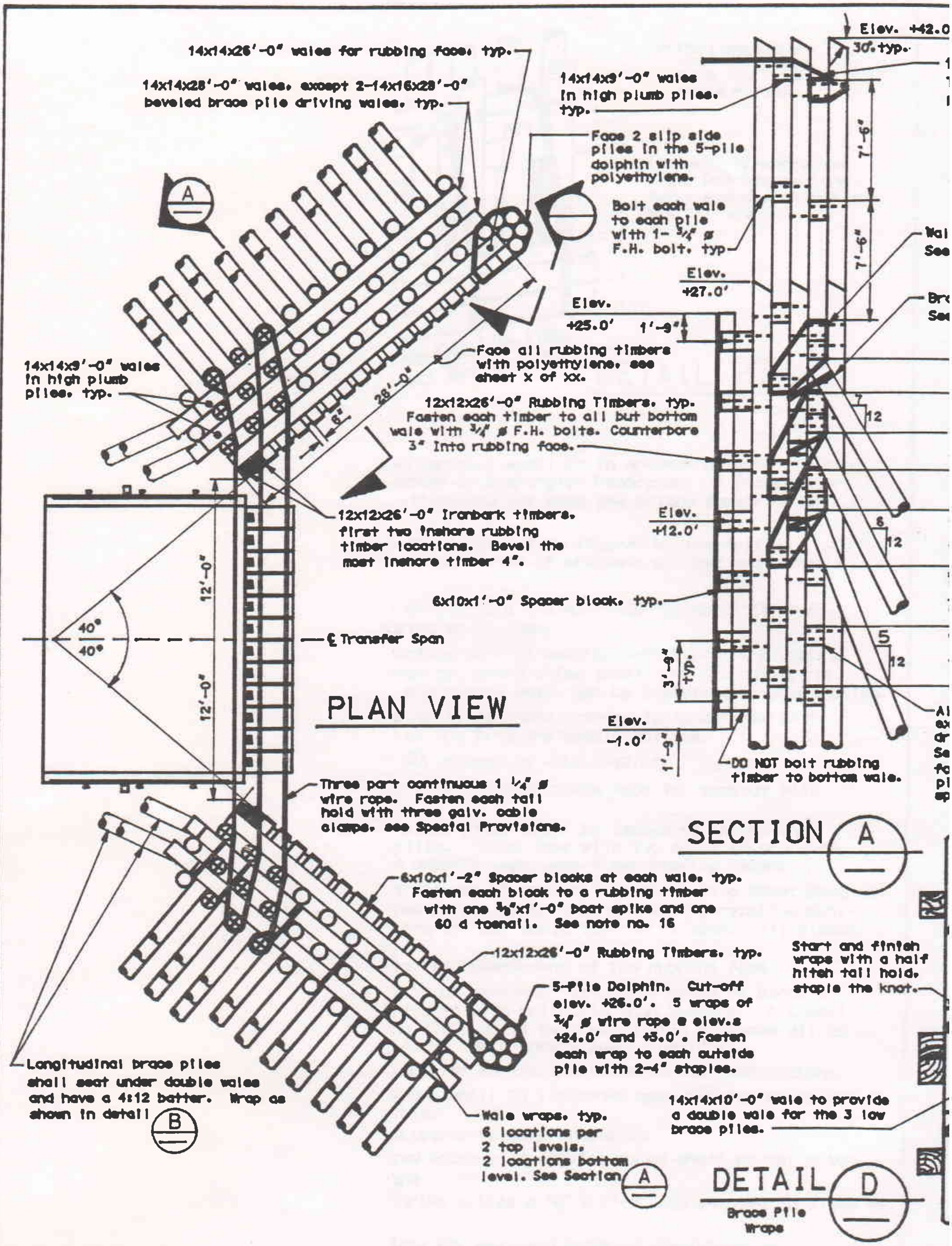


Figure 1. Edmonds Terminal

Figure 2. Wingwall (see following page)

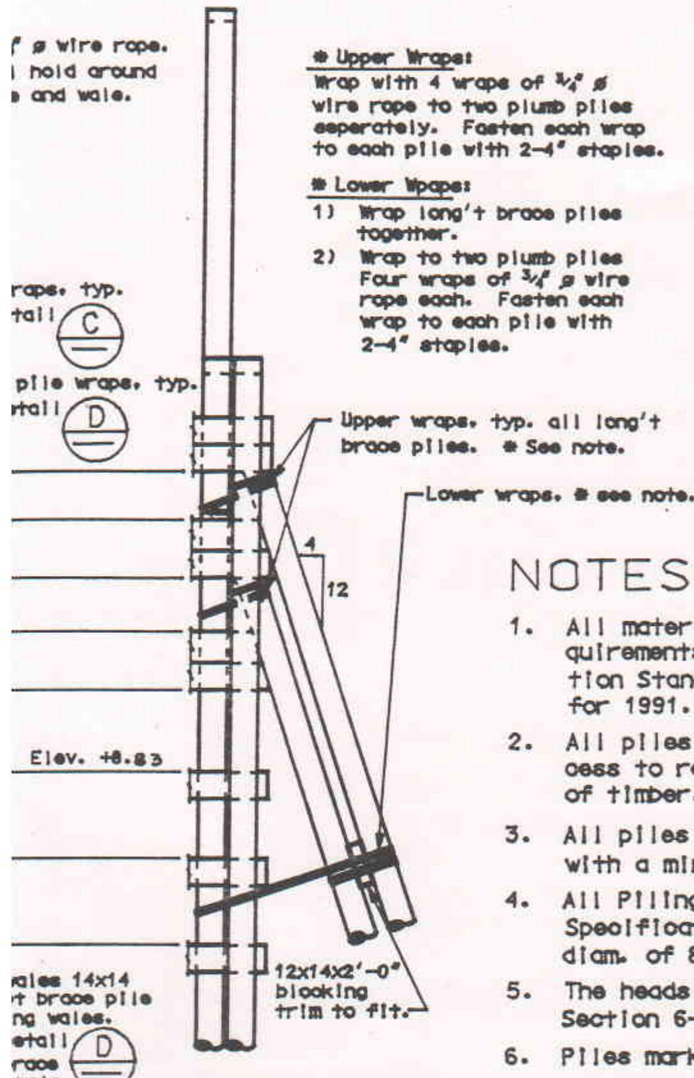


DESIGNED BY
ENTERED BY
CHECKED BY
PROJ. ENGR.
DIST. ADM.

DATE | DATE | REVISION | BY

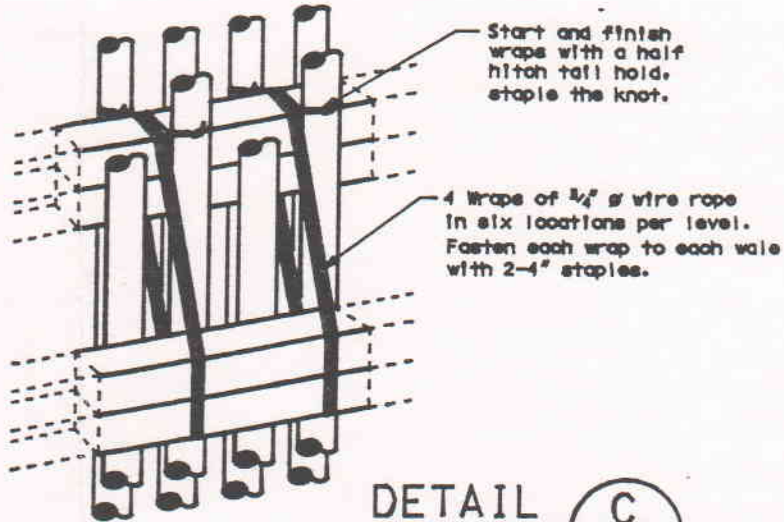
STATE | FED. AID PROJ. NO.

10 | WASH.



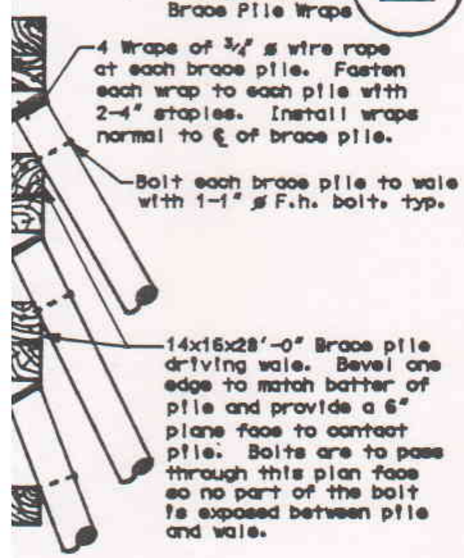
DETAIL B

Longitudinal Brace Pile Wraps



DETAIL C

Wale Wraps



NOTES:

1. All material and workmanship shall be in accordance with the requirements of the State of Washington Department of Transportation Standard Specifications for Road and bridge Construction for 1991.
2. All piles shall be new Douglas fir, treated by the full cell process to retain a minimum of 14# of creosote oil per cubic foot of timber.
3. All piles shall be driven to a minimum penetration of 15 feet with a minimum bearing of 30 tons.
4. All Piling shall conform to with Section 9-10.1 of the Standard Specifications except all brace piles shall have a minimum tip diam. of 8 inches, and straps shall not be installed on brace piles.
5. The heads of all plumb piles shall treated in accordance with Section 6-05.3(2)D of the Standard Specifications.
6. Piles marked thus ⊕ extend to elev. +42.0'.
7. Trim brace piles to give a 6 inch plane face for contact with plumb piles.
8. Irregular diameter plumb piles shall be dapped to insure that wales contact all piles. Treat daps with two coats of creosote oil and one coat of asphalt asbestose fiber roofing cement.
9. All timber shall be "Select Structural" grade Pacific coast Douglas fir, creosote treated by the empty cell process to retain a minimum of 10# of creosote oil per cubic foot of timber. All timber shall be rough cut.
10. Wales shall be installed independent of the rubbing face.
11. All wire rope shall be galvanized. All bolts shall be black . All bolts shall be fitted with malleable iron washers. All bolt holes drilled in the field shall be treated with creosote oil as per Section 6-04.3(4) of the Standard Specifications.
12. See Special provisions for springline installation instructions.
13. All wale and pile wraps shall be tightened mechanically so as to be as taut as possible.
14. All details are applicable to both wingwalls.
15. The bevel in the two brace pile driving wales shall be cut prior to creosote treatment.
16. Bottonhead twisted drive spikes @ ⅓" x 1'-0" may be used in place of boat spikes.
17. See Special Provisions for salvaged material Specification.

MARINE DIVISION

WASHINGTON STATE
DEPARTMENT OF TRANSPORTATION

and less stiff at the ends. The wall deflects easily for the first 1 to 2 inches, then becomes stiffer. Within the range of the test, $g(s)$ was a linear relationship for the larger deflections. The relationship $h(s)$ was then developed from the results of $g(s)$.

TEST SET-UP

Preliminary observations revealed that the majority of the berthing impacts occurred on the north wingwall; thus, the north wingwall was preferred for calibration. However, calibration of the north wall was impractical because of a marine park adjacent to the terminal on that side. The test equipment would have posed a safety threat for divers using the park, and the barge anchor could have damaged submerged structures or become tangled in the anchor lines of floats in the park area. For this reason, the south wall was selected for calibration, instead.

Previously observed maximum deflections around Pile 1 were 6 to 12 inches, so the researchers desired to define $g(s)$ and $h(s)$ to that range.

Figure 3 is a plan view of the berth. It shows the layout of the test equipment used to load the wingwall. Figure 4 is a detailed plan view and section of the wingwall and shows the proposed load points. The load was to be applied by a diesel powered winch mounted on an anchored barge. Previous observations indicated that roughly 11 feet of the ferry's belt line came in contact with the wingwall. Therefore, a spreader beam was designed to distribute the load from the winch cable to an 11-foot length of wall to simulate the rubbing edge of the ferry's belt rail. The beam was also needed to distribute the test load sufficiently to avoid damage to the plumb piles that were in contact with the beam and that normally do not receive a direct load. Cables attached to the ends of the beam were threaded through openings in the wall back to a block on the winch barge. The block was rigged with eight parts of line. One end of the line was attached to the barge through the dynamometer, and the other end led to the winch. The dynamometer was monitored to estimate the load in the line. A transit and two video cameras were

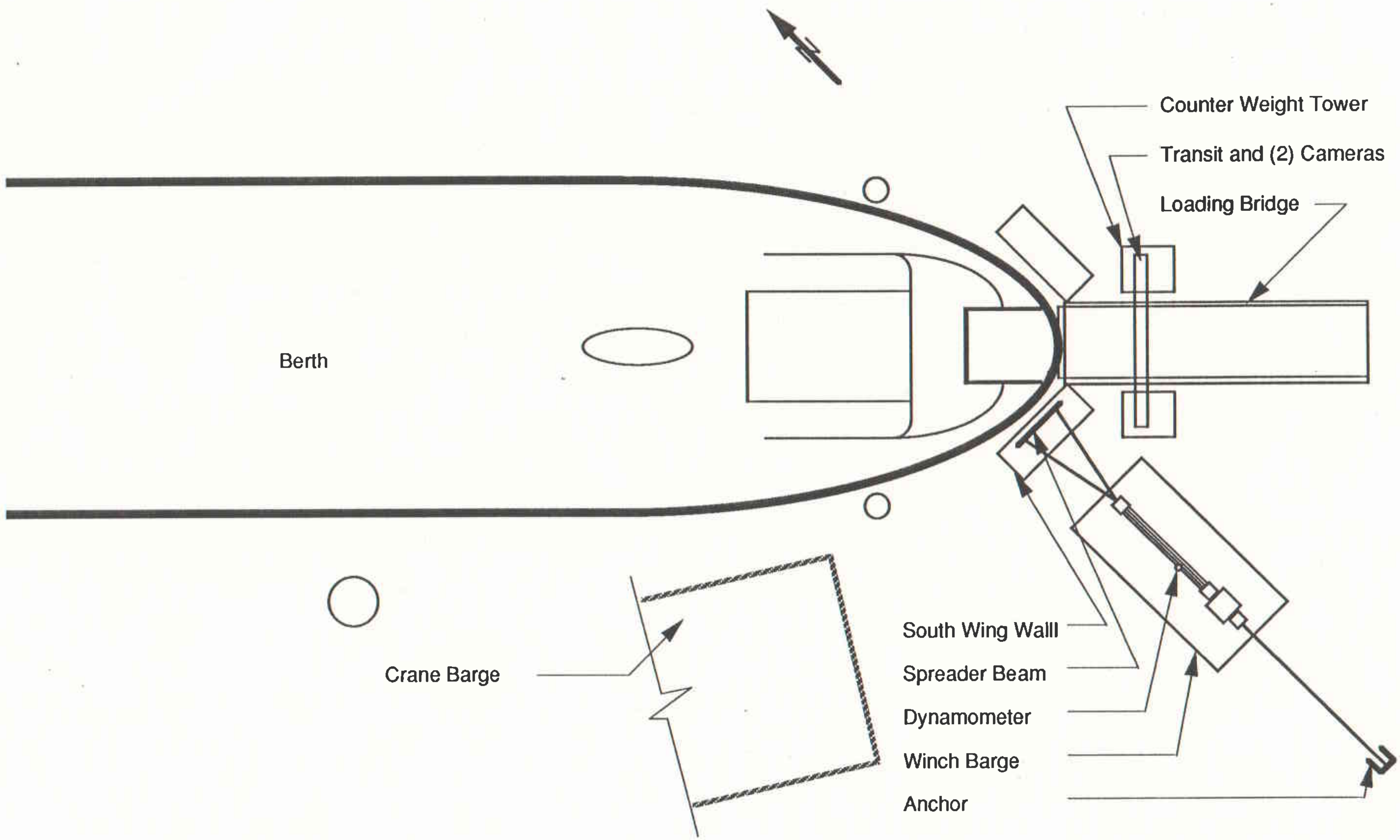


Figure 3. Test Layout

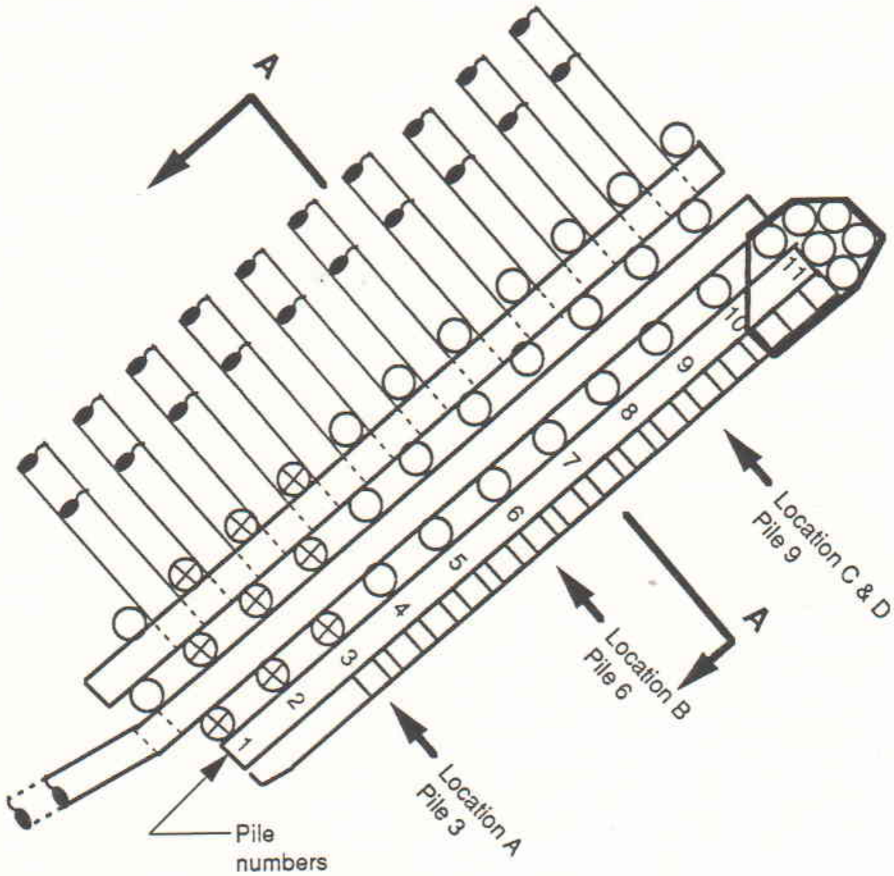
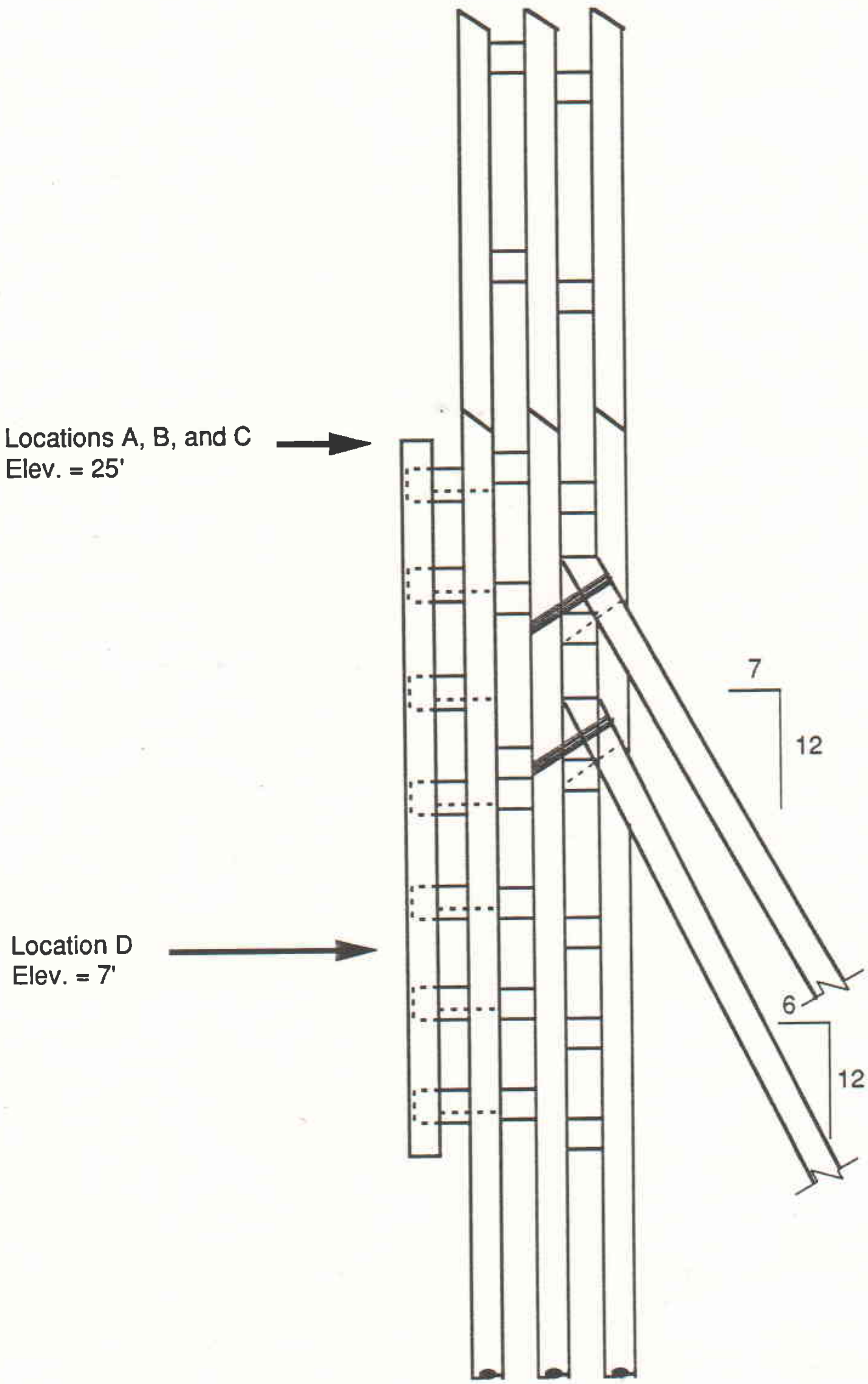


Figure 4. Wingwall Plan Showing Locations Where Force Was Applied



Section A-A

Figure 4b. Wingwall Elevation

mounted on the counterweight tower above and shoreward from the wingwall to allow the researchers to measure deflections by reading scales mounted on top of the wall.

The WSDOT selected Hurlen Construction, a local marine contractor, to provide the winch barge, 15-ton anchor, spreader beam, dynamometer, rigging and crane barge, and to conduct the test. The dynamometer was calibrated in the structures lab at the University of Washington (Table 1). The spreader beam design was provided by the research team (Appendix A and B). The transit and video cameras were supplied by the University of Washington.

TEST PROCEDURE

On Monday, May 21, 1990, equipment was placed at the site to conduct the test. Pull tests were desired from several elevations to investigate the change of $g(s)$ with elevation. For the first pull test, the spreader beam was placed in location D (Figure 2), 18 ft. below the top of the rubbing timbers. Placing the beam at this location was time consuming because the beam had to be pulled into place with a rope, and the rigging had to be threaded through the piling. Subsequent tests were performed with the spreader beam in locations A, B, and C (Figure 1, piles 3, 6, and 9), on top of the wingwall. Because the beam could be placed directly in these locations with a crane, set-up time was minimized. A comparison between the tests conducted at locations C and D (pile 9, upper and lower) showed little difference in $g(s)$. Because $g(s)$ was similar for points C and D, and because of the difficulty of placing the beam at lower levels, the research team decided to locate the beam at the top of the wall for all remaining tests.

Testing commenced on the following day, May 22nd. Four tests were conducted, one at each location: A, B, C, and D. The scales were observed with one transit and two video cameras. The maximum measured force was approximately 69 kips, which produced a deflection at the top of the wall of approximately 2.5 inches (Appendix C, Tests 1, 2, 3, and 4, May 22nd). At this maximum force the anchor slipped, and no more

Table 1. Ferry Project

Load Cell Calibration — 5-18-90

Final Test

Test Machine Reading (lb.)	Load Cell Reading (lb.)
2,000	2,000
4,000	4,000
6,000	5,950
8,000	8,000
10,000	10,000
12,000	12,000
14,000	14,000
16,000	15,950
18,000	18,000
20,000	19,950
22,000	22,100
24,000	24,000
26,000	26,100
28,000	28,100
30,000	30,150
32,000	32,100
34,000	34,200
36,000	36,000

Test With Stop Needle (Stop needles give maximum reading)

Test Machine Reading (lb.)	Load Cell Reading (lb.)
8,000	7,800
12,000	11,800
16,000	15,800
20,000	19,800
25,000	24,800
30,000	29,800

Testing Machine: 300 kip at University of Washington Structural Engineering Lab pulling force could be developed. Because previously observed deflections during berthing maneuvers had been in the range of 6 to 8 inches, the researchers decided to provide a second anchor to increase the reaction.

On May 29th, the contractor placed the second anchor (10-ton — the previously deployed anchor was 15-tons) and several more tests were conducted (Table 2). For these tests, a second transit was added so that both ends of the wall could be monitored simultaneously with a transit. A maximum force of approximately 110 kips was reached before the anchors yielded. The maximum deflection was approximately 4 inches. All tests were conducted with the beam at the top of the wall.

TEST RESULTS

The force versus deflection relationships *(g(s))* are shown in Figures 5, 6, and 7. The tests are summarized and the beam location for each test is noted in Table 1. The field data are presented in Appendix C. For all the test positions, the relationships were

Table 2. Summary of Pull Tests

Date	Test	Location of load (Pile number)	Calculated Load (k)	Deflection (in.)	Location of Reading (Pile number)
5/22/90	1	9*	57.2	2.44	11
	2	6	65.1	1.8	6
	3	9	68.7	2.63	11
	4	3	65.7	2.22	1
5/29/90	1	3	66.8	2.95	1
	2	3	93.9	3.8	1
	3	6	78.3	1.5	6
	4	9	83.5	3.25	11
	5	9	85.1	2.9	11
	6	3	92.0	3.6	1
	7	3	110.4	3.3	1

*Pull at elevation 7. All other pulls were at elevation 25.

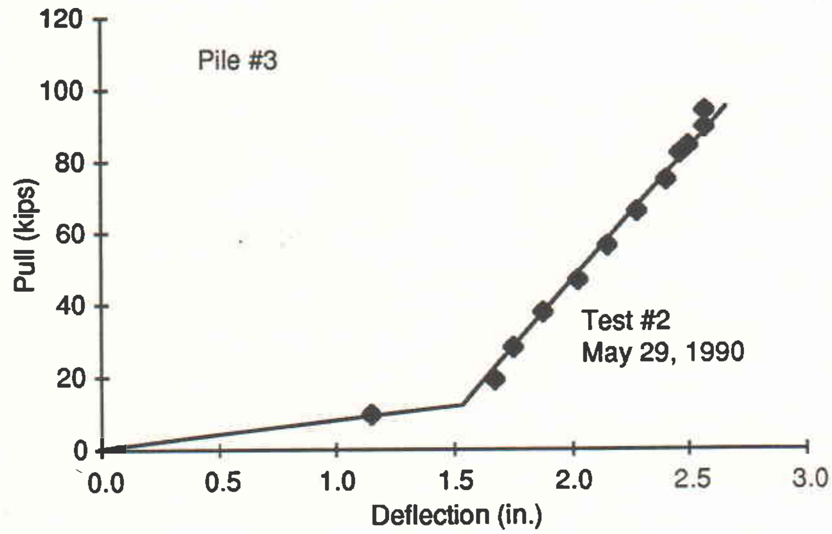


Figure 5. Deflection in Inches at Centerline of Spreader Beam
(Centerline of spreader beam at piling #3)

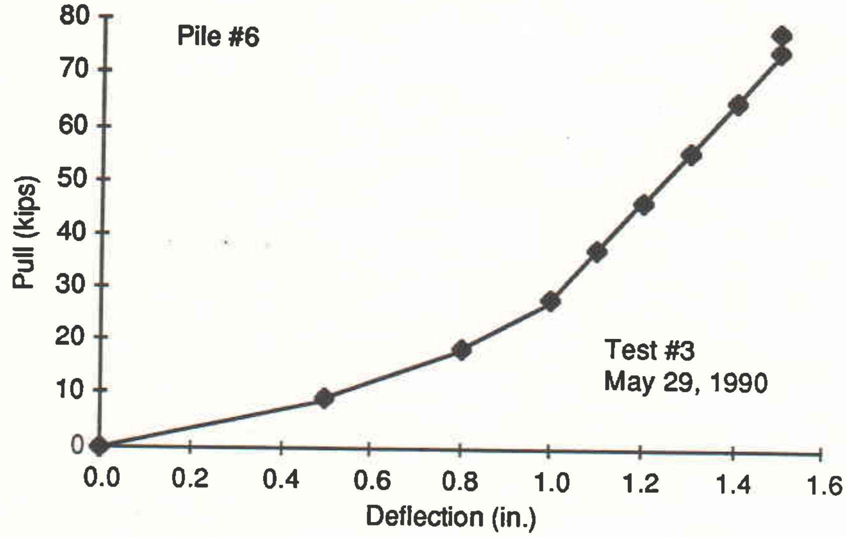


Figure 6. Deflection in Inches at Centerline of Spreader BM (Centerline at piling #6)

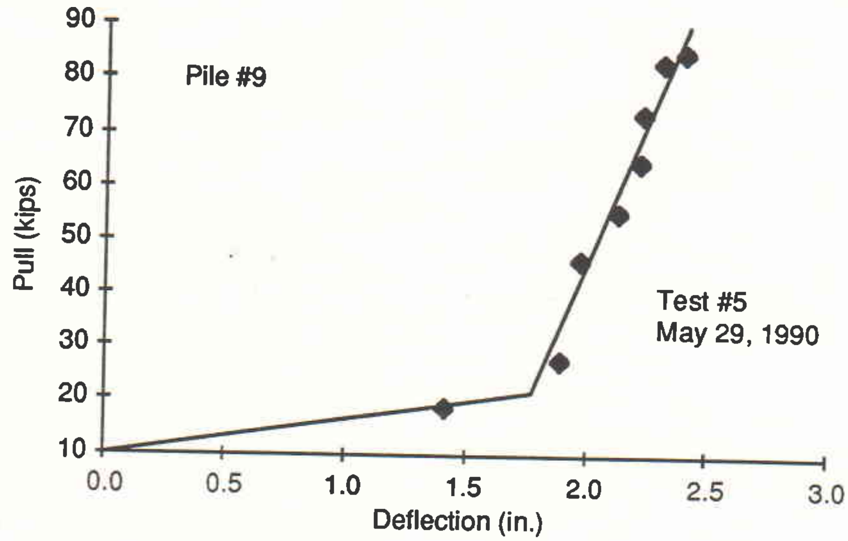


Figure 7. Deflection in Inches at Centerline of Spreader Beam
(Centerline of spreader beam at piling #9)

characterized by a sharp change in slope after the first 1 to 2 inches of deflection. The initial deflections were large for rather small forces. This is explained by an initial tightening of the wall when the slack in the connections was taken up. Resistance in this mode was most likely frictional because of the rubbing of timbers and cables.

After the wingwall had tightened up, the slope of the curve increased sharply and appeared to be linear over the range of test data. The maximum observed deflection was approximately 2.5 inches for loading at the ends of the wall and 1.5 inches for loading at the center of the wall. These were less than half the maximum deflection observed during an actual landing. However, it may be possible to extrapolate the results by assuming that the wall's stiffness is constant for greater values.

A comparison of the curves for the near end, middle, and far end of the wall revealed that the center of the wall was stiffer than either of the ends and that the far end was slightly stiffer than the near end. The center location may have been stiffer because the resistance of the entire wall was mobilized when a force was applied there. By comparison, a force applied at either end caused the wall to rotate about a point near the opposite end, thereby mobilizing only a portion of the wall's total resistance. The softness of the near end in comparison to the far end can be explained by two factors. First, most landing impacts occurred at the near end. Most likely, the joints become looser as the number of impacts increases. Second, the far end is reinforced by a group of five plumb piles which protect the wall from an end collision. These piles probably add some lateral stiffness.

Although the exact shape of the lower leg of the curve was not completely documented, the researchers assumed it to be linear for convenience of calculation. Any resulting error between the straight line and, say, a parabolic curve in this region would have been minimal because of the relatively small amount of energy represented in relation to the steeper portion of the curve. This assumption was born out by the $h(s)$

curves: the transition from the first region to the second was barely discernable (Figure 6).

In developing the $g(s)$ relationships, several adjustments were made to the raw data. These were as follows.

1. Adjustment for pulley friction (Appendix D). Because the anchors continually slipped during the test, the pulleys in the block rotated continuously, thus mobilizing a frictional force. A frictional loss of 5 percent per sheave was assumed. (Rossnagel, 1988, pg. 245) According to the contractor conducting the test, the sheave used had bronze bushings; the 5 percent value is typical for sheaves with bronze bushings. The resulting multiplier for eight parts of line and seven sheaves was 9.55, as opposed to the usual 8.00 for a static pull.
2. Adjustment for slope of line (Appendix E). The tests were conducted at varying levels of tide. The winch barge elevation naturally rose and fell with the tide, while the spreader beam elevation remained constant. This angle varied from 10 to 16 degrees from horizontal. The measured tension force was multiplied by the cosine of this angle to obtain the horizontal force exerted on the wall.
3. Location of applied load (Appendix F). The deflection measurements were taken at specific locations at the top of the wall. For tests where the spreader beam was located at the ends of the wall, the point of measured deflection and the point of applied load did not coincide. For these cases, the deflection at the point of load was calculated from the measured deflections and the geometry of the wall. Note that when berthing data are compared with pull test data, a similar adjustment must be made to the deflections tabulated for the event in question.

Table 3. Description of *g(s)* for Piles 1, 2, and 3

	K_{n_1} (k/in.)	Point of Inflection	K_{n_2} (k/in)
Pile 3	9.2	1.53	68.8
Pile 6	21.1	0.90	93.9
Pile 9	12.6	1.88	112.7

The relationship *h(s)* was developed from the records of Tests 2, 3, and 5 of May 29th. These tests produced more data points and were more consistent than other tests at the same locations. From the test data, piecewise linear relationships were developed (Table 3). One curve for *h(s)* was developed for load applications at each of piles 3, 6, and 9 (Figure 8).

CONCLUSIONS

From the results of the test the relationships, *g(s)* (force versus deflection) and *h(s)* (energy versus deflection) were developed for deflections of 2 to 4 inches. The relationship *g(s)* is piecewise linear: the wall is loose until the slack in connections is taken up, then stiffer until the maximum deflection is reached. The relationship varies, depending on where the load is applied. The wall is stiffer in the middle and more flexible at the ends. The wall tends to rotate as loads are applied at the end.

The maximum load applied by the test set-up was 110 kips, which produced a deflection of 3.3 in. at Pile 3. Observed deflections at Pile 3 during berthing events are 6 to 10 inches; thus *g(s)* must be extrapolated to estimate the berthing force at these deflections. Extension of the test to include this range of deflections would be desirable. However, for Pile 3, *g(s)* remains linear for deflections of up to 10 inches. The required force would be approximately 600 kips. Therefore, a significant change in the test set-up would be required to produce such a load.

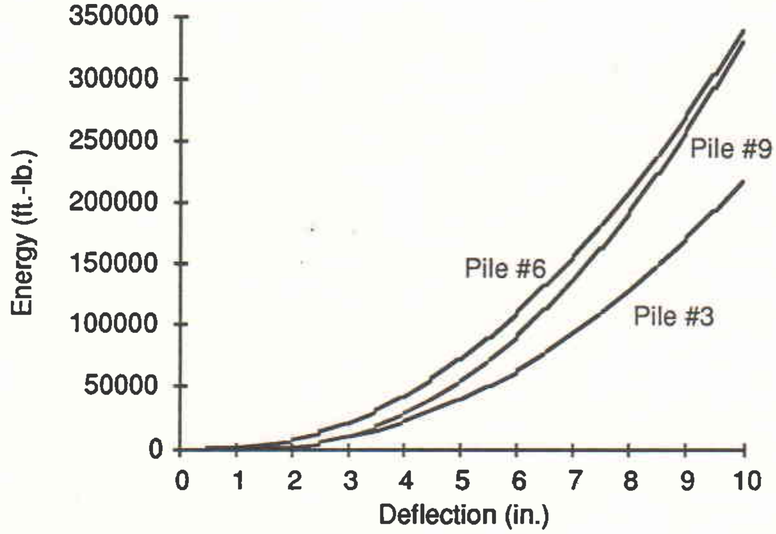


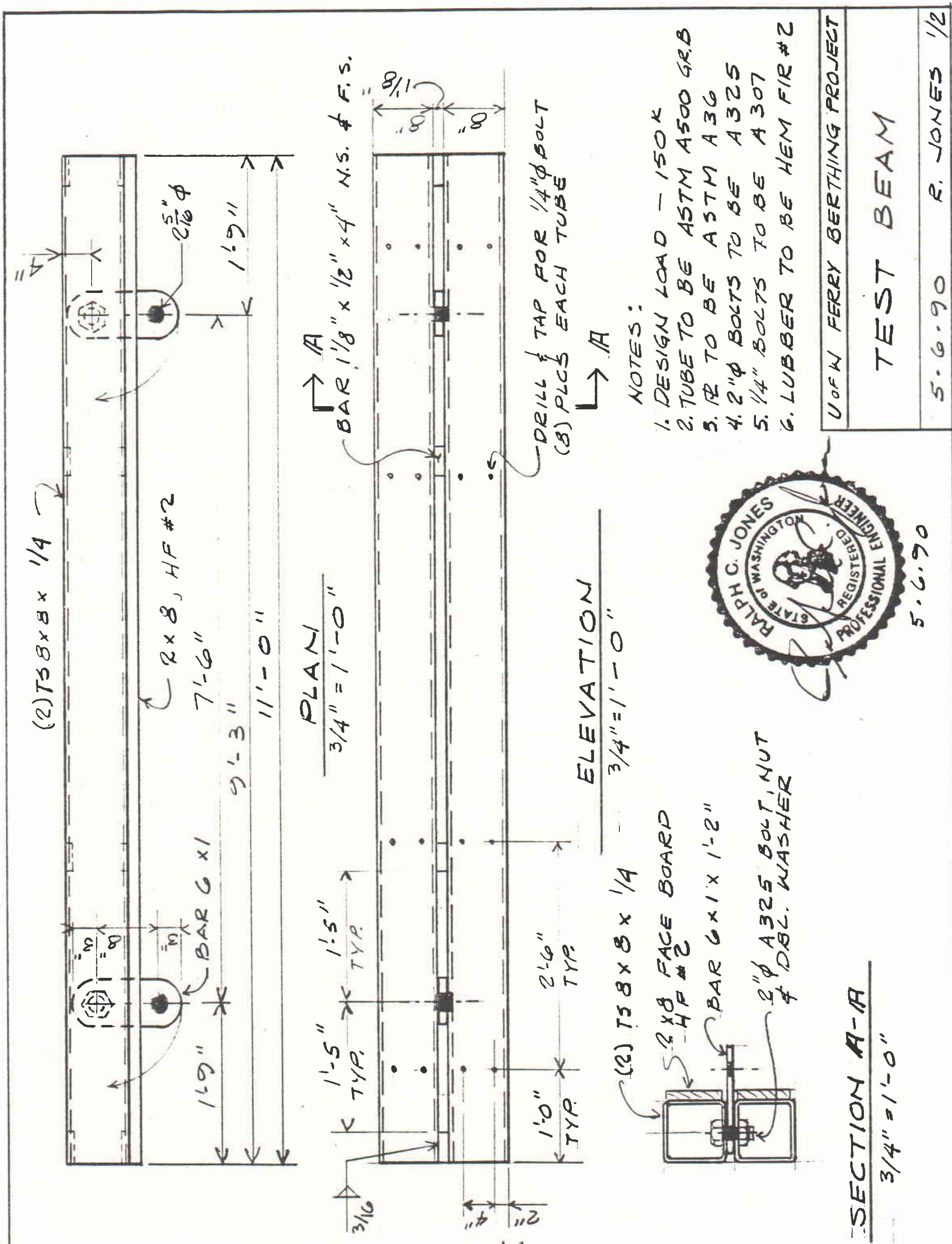
Figure 8. Energy and Deflection at Piles (Developed from Pull Test Data)

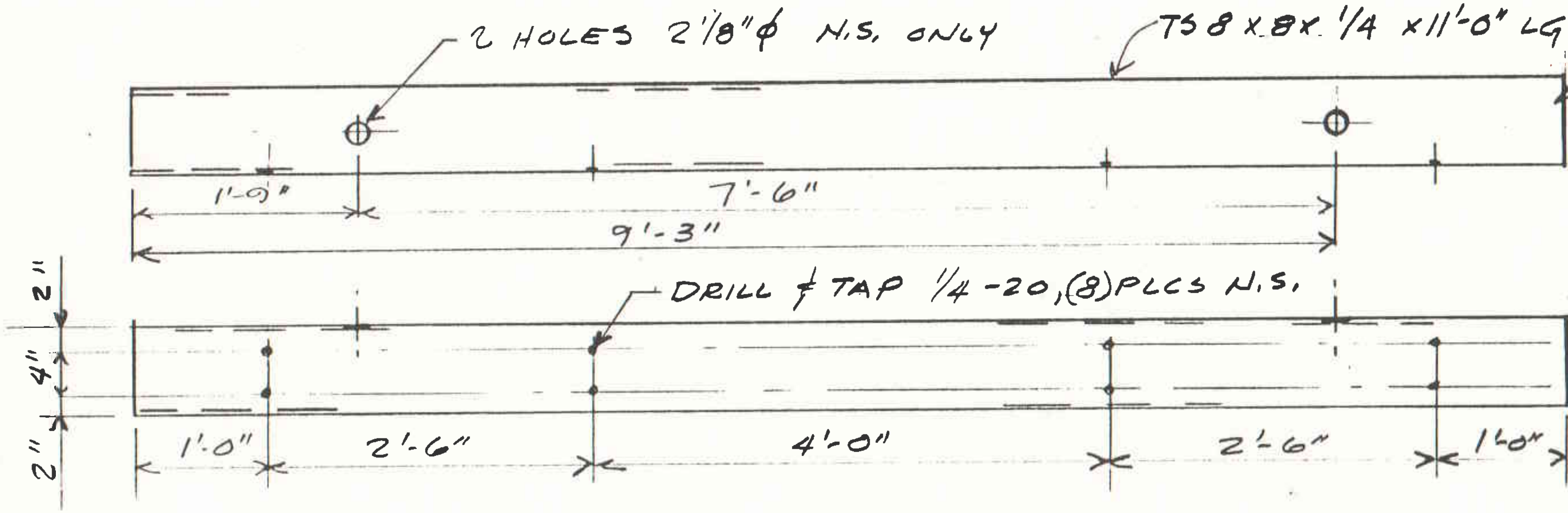
REFERENCES

1. Rossnagel, W.E., Higgins, L.R., and MacDonald, J.A., Handbook of Rigging for Construction Operations, fourth ed., McGraw-Hill, New York, 1988.

APPENDIX A

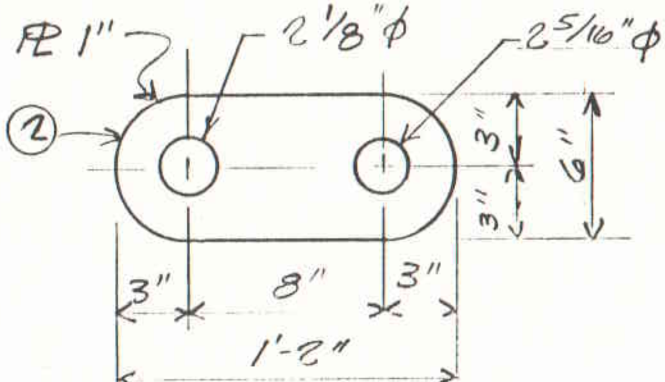
TEST BEAM DESIGN


U of W FERRY BERTHING PROJECT
TEST BEAM
5.6.90
R. JONES
1/2
NOTES:
1. DESIGN LOAD - 150K
2. TUBE TO BE ASTM A500 GR.B
3. PL TO BE ASTM A36
4. 2"φ BOLTS TO BE A325
5. 1/4" BOLTS TO BE A307
6. LUBBER TO BE HEM FIR #2
RALPH C. JONES
STATE OF WASHINGTON
REGISTERED PROFESSIONAL ENGINEER
5.6.90
PLAN
3/4" = 1'-0"
ELEVATION
3/4" = 1'-0"
SECTION A-A
3/4" = 1'-0"
(2) TS 8 x 8 x 1/4
2 x 8, HF #2
BAR 6 x 1
2 5/16" φ
7'-6"
9'-3"
11'-0"
1'-9"
BAR 1 1/8" x 1/2" x 4" N.S. & F.S.
DRILL & TAP FOR 1/4" φ BOLT
(8) PLCS EACH TUBE
1'-5" TYP.
2'-6" TYP.
1'-0" TYP.
3/16
(2) TS 8 x 8 x 1/4
2 x 8 FACE BOARD HF #2
BAR 6 x 1' x 1'-2"
2" φ A325 BOLT, NUT & DBL. WASHER

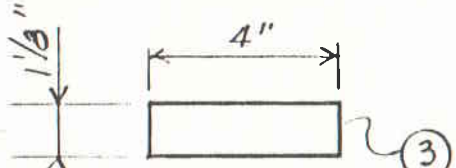


ONE REQ'D THUS

ONE REQ'D OPPOSITE HAND



2 REQ'D



8 REQ'D

BILL OF MATERIALS

#	QTY	DESCRIPTION
1	2	TS 8 x 8 x 1/4 x 11'-0"
2	2	BAR 6 x 1 x 1'-2"
3	8	BAR 1 1/8 x 1/2 x 0'-4"
4	2	2 x 8 x 11'-0" H.F #2
5	2	2"φ A325 BOLT x 4" W/NUT
6	16	1/4"φ x 2" F.H. ELEVATOR BOLT
7	4	2"φ WASHER

U of W FERRY BERTHING PROJECT

TEST BEAM DETAILS

5.6.90 R. JONES 2/2

APPENDIX B

CALCULATIONS FOR TEST BEAM DESIGN

SCOPE: SIZE BEAM AND ATTACHMENTS FOR CALABRATION OF WING WALL.

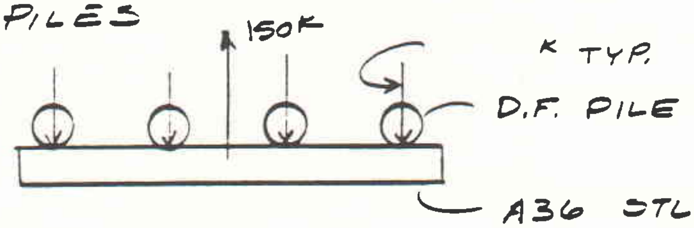
DESIGN CRITERIA:

1. CODES – AISC, MANUAL OF STEEL CONSTRUCTION
   – NATIONAL DESIGN SPECIFICATION (WOOD)
   – UNIFORM BUILDING CODE
   – STANDARD SPECIFICATION FOR HWY. BRDGS.
2. WINCH CAPACITY – 150^k MAX (TENSION)
3. STEEL SHAPES & PL – ASTM A36
4. PILES – DOUGLAS FIR, CREOSOTE TREATED

RALPH C. JONES
STATE of WASHINGTON
REGISTERED
PROFESSIONAL ENGINEER
5.6.90

ANAYSIS:

ASSUME LOAD BEAM CONTACTS 4 PLUMB PILES



PILE PROPERTIES FROM UBC TBL 25-E

$F_c = 1250$ PSI $F_b = 2450$ PSI $F_V = 115$ PSI

$F_{c\perp} = 230$ PSI $E = 1{,}500{,}000$ PSI

NOTE: PER 13.2.5 OF THE BRIDGE CODE A 65% INCREASE IS ALLOWED FOR LOADS OF 5 MIN. DURATION (RAILING LOADS) WHERE APPROPRIATE THIS WILL BE USED.

BEARING AREA –

$$A = \frac{37.5^k}{.230\text{KSI}\,(1.65)} = 78.8 \text{ IN}^2$$

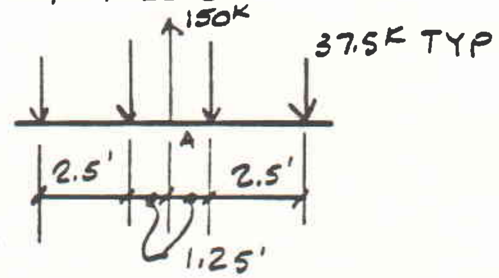
FOR $w = 3\frac{1}{4}"$ $h = 30.4$ IN

IF 5 PILES IN CONTACT, $T' = 30^k$, $A = 79.1 \text{ IN}^2$

$w = 3\frac{1}{4}$ $h = 24"$

USE 10' BM w/ 24" FACE PL CONTACT 5 PLUMB PILES

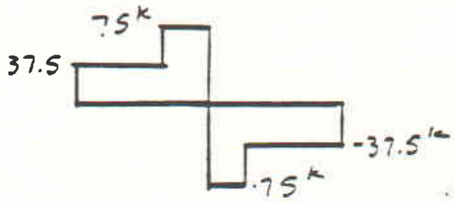
STEEL BM -
4 - PILES



$M_A = 1.25' \cdot 37.5^K + 3.75' \cdot 37.5^K$

$M_A = 187.5$ K-FT

$$S = \frac{187.5\,(12)}{24\,(1.33)} = 70.5\ IN^3$$



$V = 75^K$

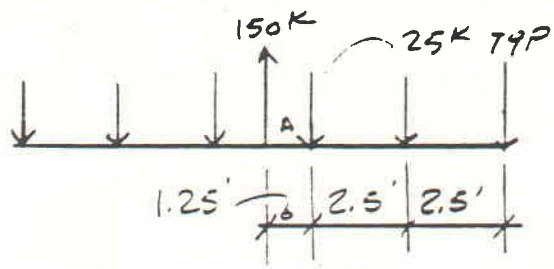
ASSUME $2\frac{1}{2}''\ \phi$ HOLE
& 7/16" WEB

$A_n = (12.25 - 2.5)\,.43 = 4.19\ IN^2$

$F_v A_n = 0.4(36)(1.33)(4.19)$
$= 80.3^K$ OK

USE W12 x 72
$S = 97.4$ $d = 12\frac{1}{4}$ $t_w = \frac{7}{16}$
$b_f = 12\frac{1}{8}$

6 - PILES

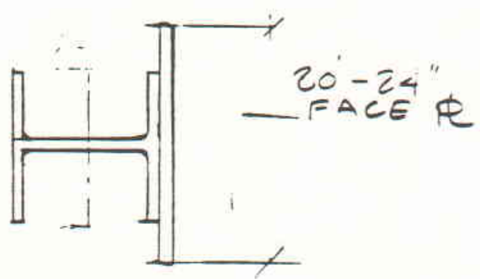


$M_A = 25^K(1.25' + 3.75' + 6.25')$

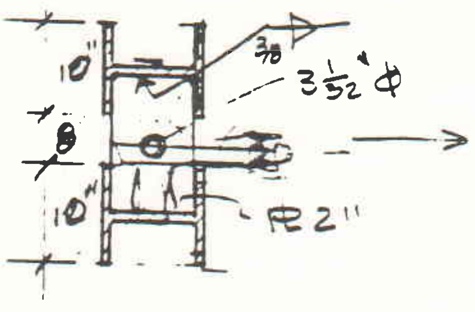
$M_A = 281.3$ K-FT

$$S = \frac{281.3\,(12)}{24\,(1.33)} = 105.7\ IN^3$$

USE W12 x 79
$S = 103$ $d = 12\frac{3}{8}$ $t_w = \frac{1}{2}$
$b_f = 12\frac{1}{8}$



AN ALTERNATE DESIGN WOULD BE TWO W12 BMS



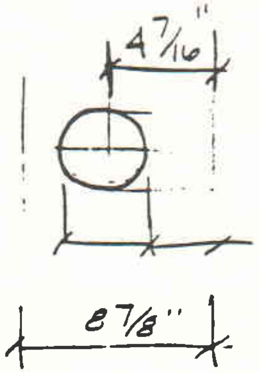
FOR 20" OF FACE

USE	2	W 12 x 53	S = 70.6
OR	2	W 10 x 49	S = 2 x 54.6
OR	2	W 10 x 54	S = 2 x 60.0

2 W10 x 54 WOULD ALLOW MORE SLOP INSIDE THE WALL & IS BETTER

GUSSET DESIGN

FROM TABLE OF MILLER SWIVELS, ALLOW 8" BETWEEN BEAMS. EYE TO BE $3\frac{1}{32}$" ϕ

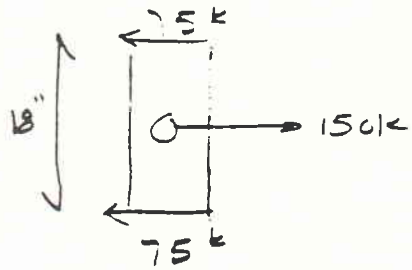


$$\tau = \frac{150^k}{2(4.438)(.4 \times 36 \times 1.33)} = .882''$$

$$t = 7/8''$$

OR

$$t = \frac{150^k}{1.5(58)\,3''} = .58''$$



$$M = 75^k(9'') = 675 \text{ k-in}$$

$$S = \frac{675}{22(1.33)} = 23.1 \text{ in}^3$$

$$S' = \frac{2\,(8.875^3 - 3.113^3)}{6\,(8.875)}$$

$$S' = 25\ \ 723$$

USE 2" GUSSET PL

AT ENDS USE PL 1/4" GUSSETS TO HOLD BMS //.

WELD

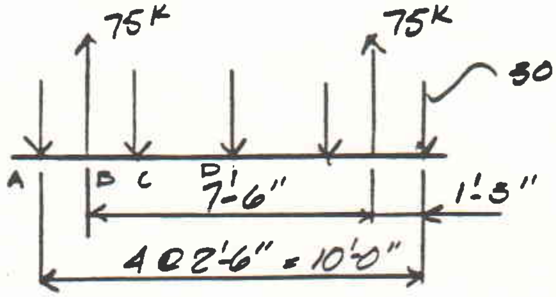
$L = 8.875''$ $\qquad .925$ K/IN PER 16^{TH}

$$SIZE = \frac{75^k}{8.875(1.33)(2)(.925)} = 3.4 \rightarrow 4/16 = 1/4$$

MIN SIZE IS 5/16" BOTH SIDES

THIS IS WORKABLE, BUT A BETTER SOLUTION WOULD BE TO ATTACH @ 2 POINTS TO REDUCE SIZE OF HARDWARE

BM SIZE FOR 2 POINT CONNECTION



$M_B = 1.25' \times 30^k = 37.5 \text{ K-FT}$

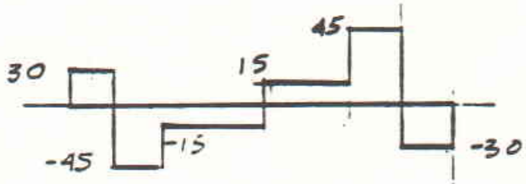
$M_D = 75^k(3.75') - 30^k(2.5') - 30^k(5.0')$

$M_D = 56.25 \text{ K-FT}$

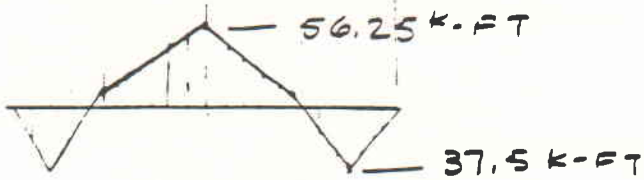
$M_C = 75(1.25) - 30(2.5)$

$M_C = 18.75 \text{ K-FT}$

$S = \dfrac{56.25(12)}{2(22)(1.33)} = 11.5 \text{ IN}^3$

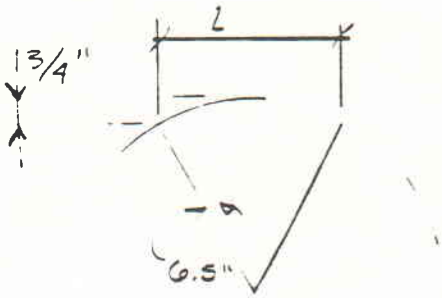


SHEAR



MOMENT

FLANGE WIDTH



$\alpha = \cos^{-1}\left(\dfrac{6.5 - .75}{6.5}\right) = 27.8°$

$l = 2 \sin 27.8 (6.5) = 6.1"$

$\text{BRG AREA} = \dfrac{30^k}{.230(1.65)} = 98 \text{ IN}^2$

$h = \dfrac{98}{6.1} = 16"$

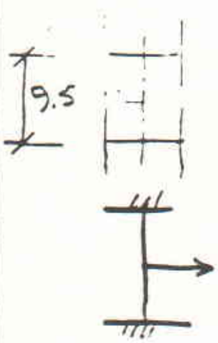
USE 2 W 8 X 28

OR 2 TS 8 x 8 x 3/16

$S = 24.3 \text{ IN}^3$ $b_f = 8"$

$t_w = .285"$ $V = 33.3^k$

$L_u = 17.5'$



PIN SIZE W/ WF BM

$M = \dfrac{PL}{4} = \dfrac{75^k(9.5)}{4} = 178 \text{ K-IN}$

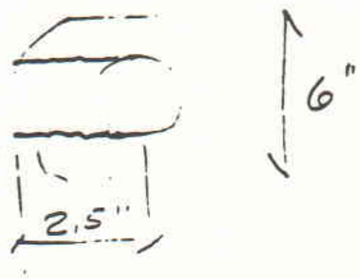
$S = \dfrac{178}{.75(42)(1.33)} = 4.24 \text{ IN}^2$

$d = \left[4.24(32)/\pi\right]^{\frac{1}{3}} = 3.5" \phi$

$A = 9.62 \text{ IN}^2$

$f_v = \dfrac{75^k}{2(9.62)} = 3.9 \text{ KSI}$ OK

BAR SIZE



TRY 1" PL

$$W = \frac{75^k}{.6(36)(1.33)} = 2.61''$$

SAY 2 5/16 EACH SIDE OF HOLE

$$f_v = \frac{75^k}{2(1'')(2.5)} = 15\,ksi < 14.4(1.33)$$ OK

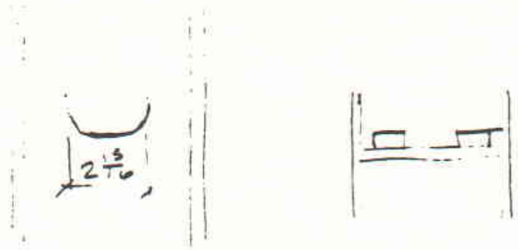
$$f_{brg} = \frac{75}{3.5(1)} = 21.4\,ksi < 1.5(58)$$ OK

USE BAR $6\frac{1}{8} \times 1''$

BEAM WEB

$$f_v = \frac{45^k}{(8.06 - 2.938)(.285)} = 30.8\,ksi$$

$$> 14.4(1.33) = 19.15$$



CHECK WEB OF TS 8x8x3/16
FROM AISC, 2"φ BOLT A325 DBL SHR
IS OK.

$$f_v = \frac{45^k}{2(.188)(8-2.125)} = 20.4\,ksi$$

$$f_v < .4(42)(1.33) = 22.3\,ksi$$ ok

$$f_{brg} = \frac{75^k}{2(.375)} = 100\,psi$$

$$f_{brg} < 1.5(58)(1.33) = 116\,psi$$ ok

BOLT TEAR OUT
$f = 75^k / 2 \times 4 \times .375 = 25\,ksi > 22.3\,ksi$
USE 1/4" WALL $f = 18.75\,ksi$

USE TS 8x8x1/4

APPENDIX C

FIELD MEASUREMENTS FROM PULL TESTS

APPENDIX C

FIELD MEASUREMENTS FROM PULL TEST

FIELD READINGS 5/22/90

Date: 5-22-90

Test #1 Beam at Pile 9, not corrected for tide level (Appendix E)

Time: 11:00 a.m.

Dynomometer Reading (kips)	Calculated Load (kips)	Deflection (in.)		
		(in.)	(in.)	(in.)
0	0			0
2	19.1			2.03
4	38.2			2.28
5	47.6			2.28
6	57.2			2.44

Date: 5-22-90

Test #2 Beam at Pile 6, not corrected for tide level (Appendix E)

Time: 1:10 p.m.

Dynomometer Reading (kips)	Calculated Load (kips)	Deflection (in.)		
		(in.)	(in.)	(in.)
0	0		0	
2	18.6		0.5	
3	27.9		0.8	
4	37.2		1.2	
5	46.5		1.4	
6	55.9		1.6	
7	65.1		1.8	

FIELD READINGS 5/22/90 (CONTINUED)

Date: 5-22-90

Test #3 Beam at Pile 9, not corrected for tide level (Appendix E)

Time: 1:50 p.m.

Dynomometer Reading (kips)	Calculated Load (kips)	Deflection (in.)	(in.)	(in.)
0	0			0
1	9.4			1.06
3	28.1			1.91
5	46.8			2.23
6	56.2			2.42
7	65.5			2.63
7.2	68.7			2.63

Date: 5-22-90

Test #4 Beam at Pile 3, not corrected for tide level (Appendix E)

Time: 2:38 p.m.

Dynomometer Reading (kips)	Calculated Load (kips)	Deflection (in.)	(in.)	(in.)
0	0	0		
1	9.4	0.54		
2.5	23.5	1.47		
4	37.6	2.02		
6	56.4	2.33		
7	65.7	2.22		

FIELD READINGS 5/29/90

Date: 5-29-90

Test #1 Beam near throat, c = .0.983, correction for tide level (Appendix E)

Time: ?

Dynomometer Reading (kips)	Calculated Load (kips)	Deflection (in.)	Deflection (in.)	Deflection (in.)
1	9.4	1.5	0.0	
2	18.8	1.6	0.1	
3	28.1	1.65	0.1	
4	37.6	1.9	0.1	
5	46.9	2.2	0.1	
6	56.32	2.4	0.2	
7	65.7	2.7	0.2	
7	65.7	2.95	0.1	
0	0.0	13.6	0.0	

Yakima Lands:

Initial impact	(2.6)
Rebound	(2.2)
In Slip, tied up	(-0.2)

Date: 5-29-90

Test #2 Beam at Pile 3, c = .983, correction for tide level (Appendix E)

Time: 10:15-10:38

Dynomometer Reading (kips)		Calculated Load (kips)	Deflection (in.)	Deflection (in.)	Deflection (in.)
1.0	10:15	9.39	1.6	0.4	
2.0		18.78	2.4	0.5	
3.0		28.2	2.5	0.5	
4.0		37.6	2.7	0.5	
5.0		46.9	2.95	0.5	
6.0		56.3	3.2	0.4	
7.0		65.7	3.4	0.4	
8.0		75.1	3.6	0.4	
8.8		82.6	3.7	0.4	
9		84.5	3.7	0.5	
9.6		90.1	3.8	0.5	
10	10:35	93.9	3.8	0.5	
0		0.0	0.0	0.0	

Positive deflections are away from the berth.
Negative deflections are toward the berth.

FIELD READINGS 5/29/90 (CONTINUED)

Date: 5-29-90

Test #3 Beam at Pile 6 (Top), c = .976, correction for tide level (Appendix E)

Time: 11:58-12:02

Dynomometer Reading (kips)		Calculated Load (kips)	Deflection (in.)	(in.)	(in.)
0	11:58		0	0	
1		9.3	0.7	0.5	
2		18.6	1.15	0.8	
3		28.0	1.4	1.0	
4	12:00	37.3	1.5	1.1	
5		46.6	1.65	1.2	
6		55.9	1.8	1.3	
7		65.2	1.9	1.4	
8		74.6	2.05	1.5	
8.4	12:02	78.3	2.1	1.5	

Positive deflections are away from the berth.
Negative deflections are toward the berth.

Date: 5-29-90

Test #4 Beam at Pile 9 (Top), c = .971, correction for tide level (Appendix E)

Time: 12:38-12:42

Dynomometer Reading (kips)		Calculated Load (kips)	Deflection (in.)	(in.)	(in.)*
0	12:38		0	0	0
1.4		13.0	0.0	0	0.5
2		18.3	-0.1	0	
3		27.8	-0.3	0.2	
4	12:39	37.1	-0.5	0.3	0.75
5		46.4	-0.55	0.4	1.75
6		55.6	-0.58	0.5	2.5
7		64.9			
8		74.2			3.0
9	12:42	83.5	-0.60	0.6	3.25

*Estimated from video recording
Positive deflections are away from the berth.
Negative deflections are toward the berth.

FIELD READINGS 5/29/90 (CONTINUED)

Date: 5-29-90

Test #5 Beam at Pile 9, c = .969, correction for tide level (Appendix E)

Time: 1:07-1:10

Dynomometer Reading (kips)		Calculated Load (kips)	Deflection Pile #1	Deflection Pile #6	Deflection Pile #11
2	1:07	18.5	0.0		1.7
3		27.8	-0.15		2.3
4		37.0	-0.15		
5		46.3	-0.15		2.4
6		55.5	-0.15		2.6
7		64.8	-0.15		2.7
8		74.0	-0.2		2.7
9		83.3	-0.2		2.8
9.2	1:10	85.1	-0.2		2.9

Positive deflections are away from the berth.
Negative deflections are toward the berth.

Date: 5-29-90

Test #6 Beam at Pile 3, c = .963,
Vessel in berth (Yakima), correction for tide level (Appendix E)

Time: 2:23-2:24

Dynomometer Reading (kips)		Calculated Load (kips)	Deflection Pile #1	Deflection Pile #6	Deflection Pile #11
0			2		0.0
1		9.2			
2		18.4	2.3		-0.1
3		27.6	2.5		-0.2
4	2:23	36.8	2.6		
5		46.0	2.8		-0.3
6		55.2			
7		64.4	3.1		-0.5
8		73.6	3.25		
9		82.8	3.5		-0.6
10	2:24	92.0	3.6		-0.7

Positive deflections are away from the berth.
Negative deflections are toward the berth.

FIELD READINGS 5/29/90 (CONTINUED)

Date: 5-29-90

Test #7 Beam at Pile 3, c = .963, correction for tide level (Appendix E)

Time: 2:39

Dynomometer Reading (kips)	Calculated Load (kips)	Deflection		
		Pile #1	Pile #6	Pile #11
0	0			
1	9.2	.8		-0.2
2	18.4	1.1		-0.4
3 2:39	27.6	1.3		-0.5
4	36.8			-0.6
5	46.0	1.8		
6	55.2	2.1		-0.8
7	64.4	2.3		-0.9
8	73.6	2.5		-1.0
9	82.8	2.7		-1.1
10	92.0	2.9		-1.2
11	101.2	3.1		-1.3
12	110.4	3.3		-1.4

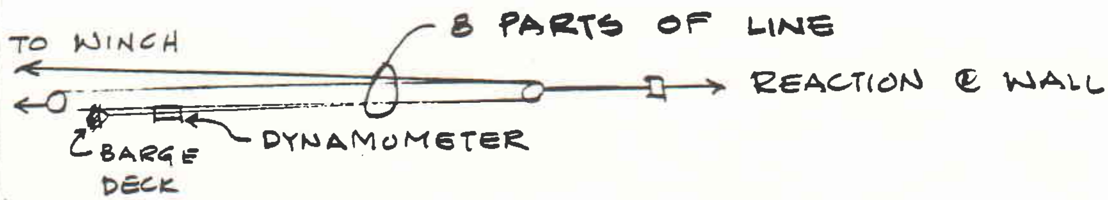
Positive deflections are away from the berth.
Negative deflections are toward the berth.

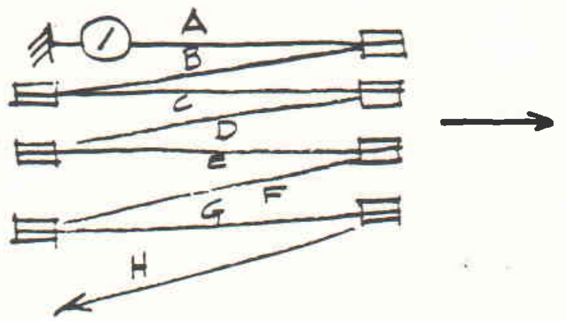
APPENDIX D

ADJUSTMENT FOR PULLEY FRICTION

(This calculation develops an adjustment for losses due to pulley friction in the blocks.)

COEFFICIENT FOR FRICTION IN BLOCK





ASSUME 5% SHEAVE EFFICIENCY

NOTE: THIS ANALYSIS ASSUMES THAT THE PULL IS NOT STATIC BECAUSE THE ANCHOR CONTINUALLY SLIPPED ALLOWING THE SHEAVES TO ROTATE. A STATIC PULL WOULD SIMPLY HAVE A MULTIPLIER OF 8.

$A = A(1) = A(1.000)$
$B = A(1.05) = A(1.050)$
$C = B(1.05) = A(1.103)$
$D = C(1.05) = A(1.158)$
$E = D(1.05) = A(1.216)$
$F = E(1.05) = A(1.276)$
$G = F(1.05) = A(1.340)$
$H = G(1.05) = A(1.407)$

$T = A + B + C + D + E + F + G + H$

$T = A(1.000 + 1.050 + 1.103 + 1.158 + 1.216 + 1.276 + 1.340 + 1.407)$

$T = A(9.55)$

DYNAMOMETER READING	FORCE IN LINE	HORIZONTAL FORCE $\phi = 12°$
0 KIPS	0 KIPS	0 KIPS
1	9.6	9.4
2	19.1	18.7
3	28.6	28.0
4	38.2	37.4
5	47.7	46.7
6	57.3	56.0
7	66.8	65.3
8	76.4	74.7
9	85.9	84.0
10	95.5	93.4
11	105.0	102.7
12	114.6	112.0

APPENDIX E

ADJUSTMENT FOR SLOPE OF LINE

(This calculation develops adjustments to measured force in the dynamometer to compensate for changes in tide level.)

(DAYLIGHT SAVINGS TIME)

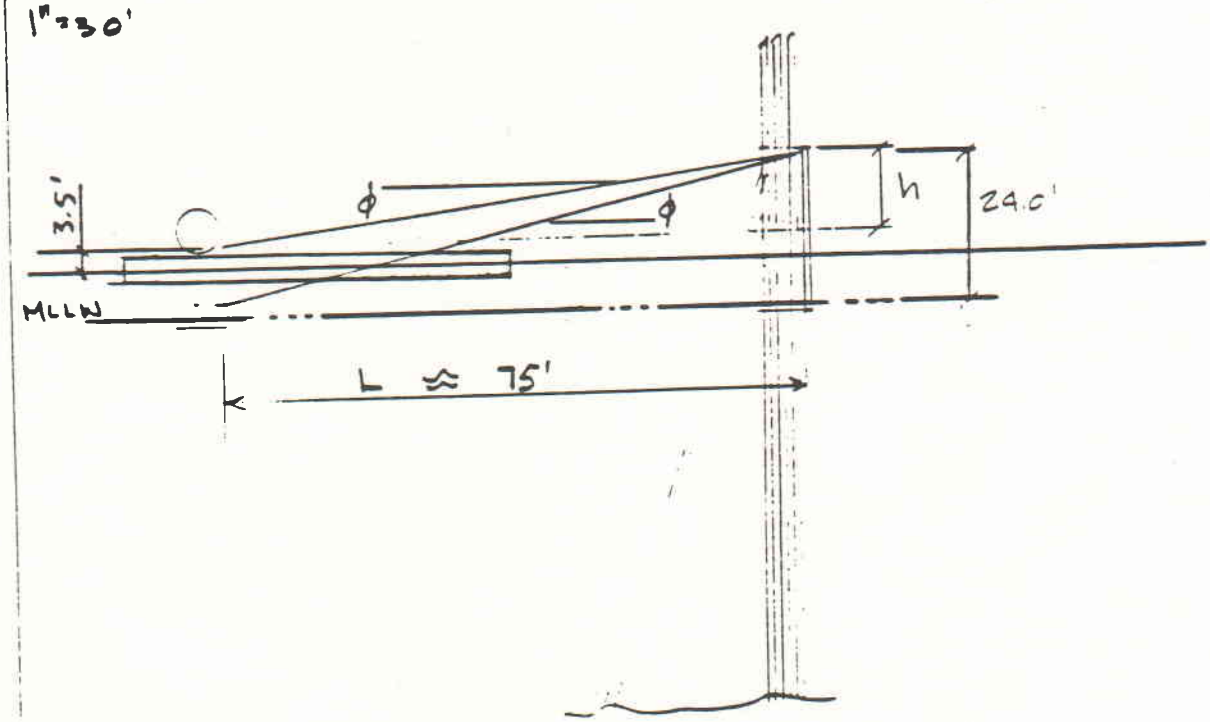
MAY 22

TEST TIMES	TIDE	h	φ	c
1. 11:00 AM	-1.25'	24-18+1.25-3.5=3.75	2.86°	.999
2. 1:10 PM	+3.30'	24-3.3-3.5 = 17.2'	13°	.975
3. 1:50 PM	+5.75'	24-5.75-3.5 = 14.75'	11°	.981
4. 2:38 PM	+7.00'	24-7.0-3.5 = 13.5'	10°	.984

MAY 29

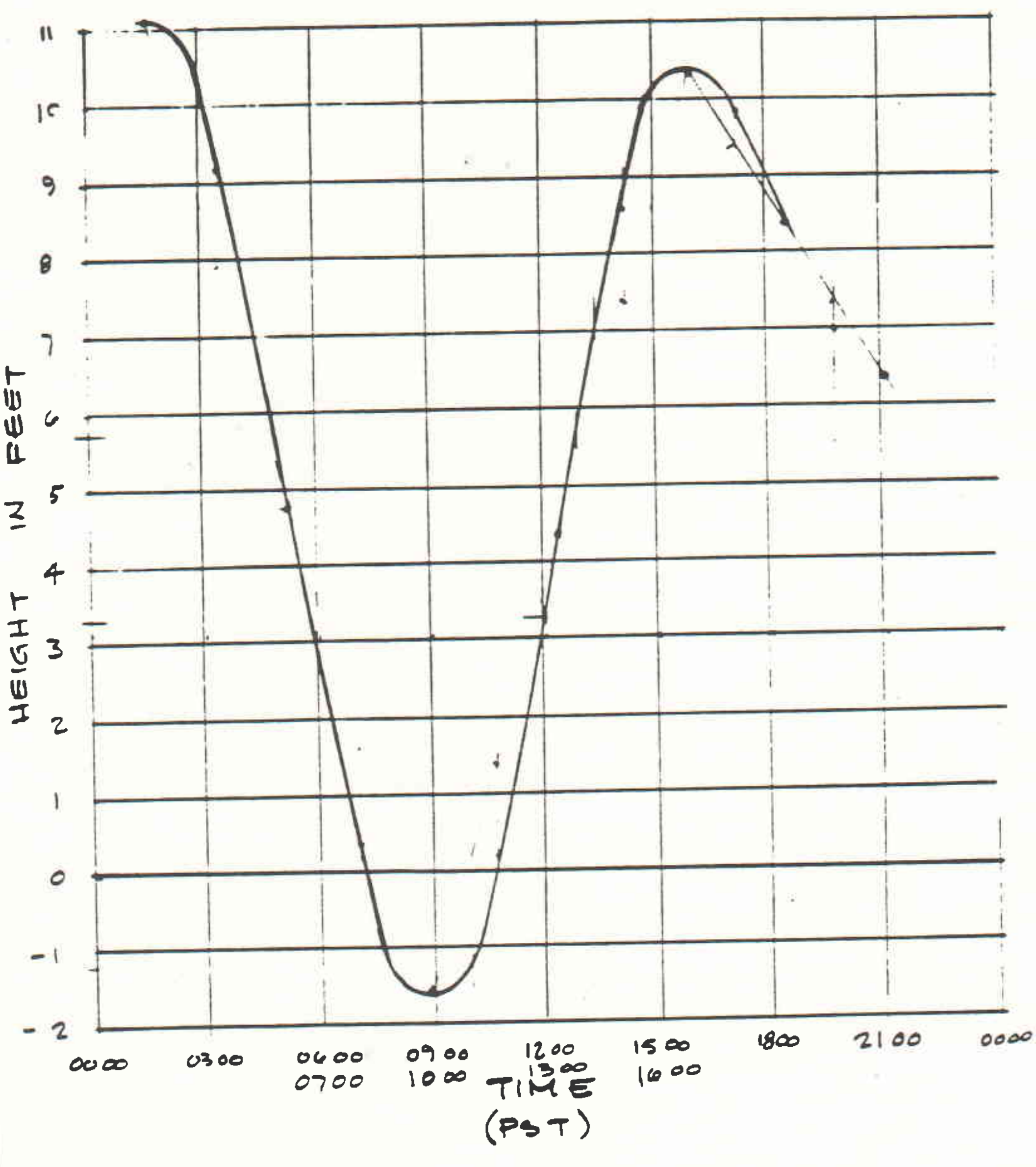
TEST TIMES	TIDE	h	φ	c
1.				
2. 10:25	+6.5'	24-6.5-3.5 = 14.0'	11°	.983
3. 12:05	+3.8'	24-3.8-3.5 = 16.7'	13°	.976
4. 12:43	+2.0'	24-2-3.5 = 18.5'	14°	.971
5. 13:10	+1.3'	24-1.3-3.5 = 19.2'	14°	.967
6. 14:16	-0.4'	24+.4-3.5 = 20.9'	16°	.963
7. 14:33	-0.6'	24+.6-3.5 = 21.1	16°	.963

1"=30'



TIDES 5-22-91
TABLE 5 SEATTLE / CORRECTIONS TABLE 6

TIME	LEVEL	Δ TIME	Δ LEVEL	TIME	LEVEL
01 39	11.6'	0 00	-0.5	0139	11.1
08 53	-1.6	+0 06	+0	08 59	-1.6
16 01	10.8	+0 00	-0.5	16 01	10.3
21 03	6.3	+0 06	+0	21 03	6.3



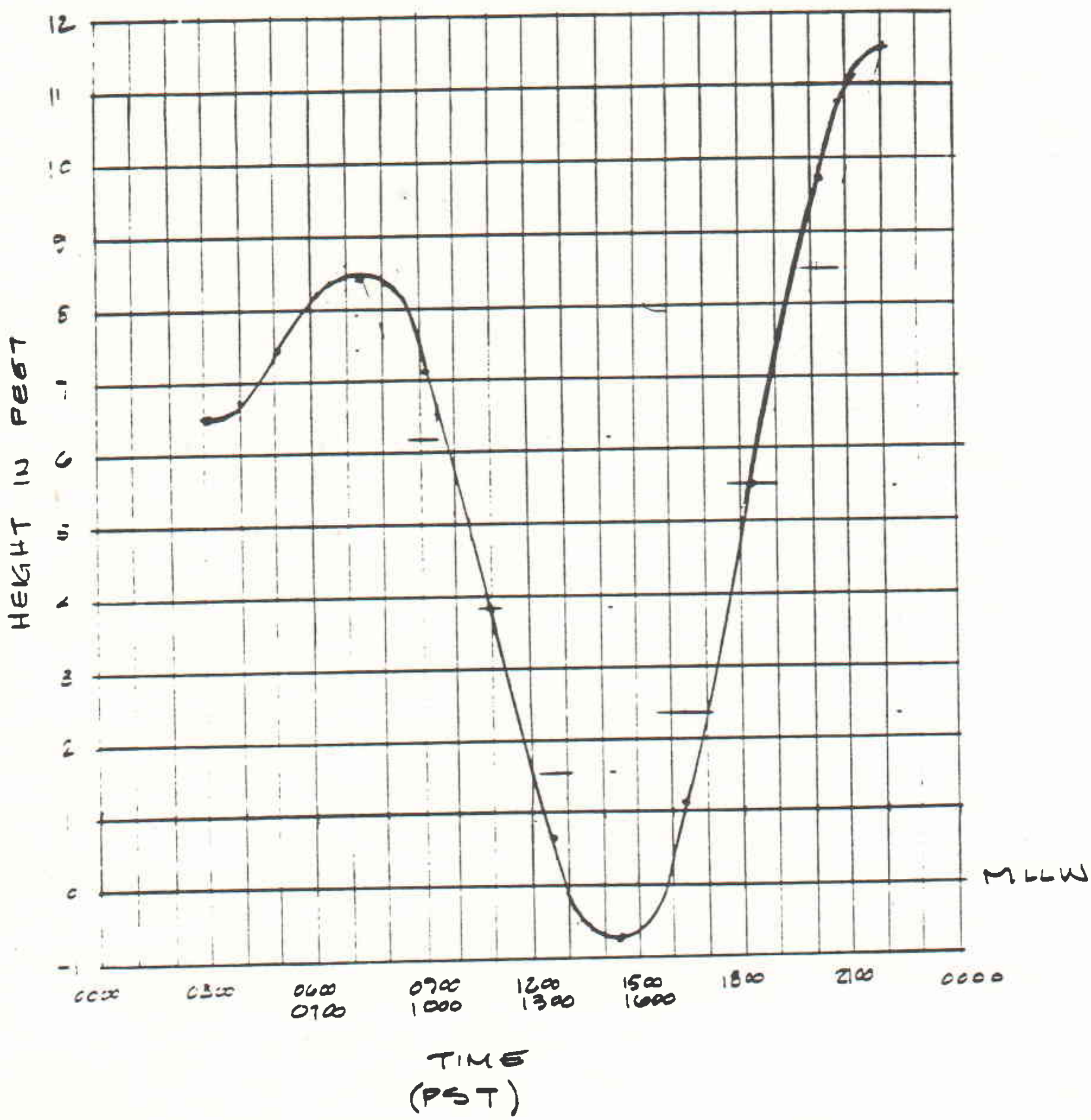
HEIGHTS REFER TO MEAN LOWER LOW WATER

FIGURE B-4

TIDES 5-29-71

TABL 5 SEATTLE / CORRECTIONS TABLE 6

TIME	LEVEL	Δ TIME	Δ LEVEL	TIME	LEVEL
0256	6.5'	+0 06	0.0	0302	6.5'
0724	8.9'	0 00	-0.5	0724	8.4'
1424	-0.7'	+0 06	0.0	1430	-0.7'
2202	12.1'	000	-0.5	2202	11.6'



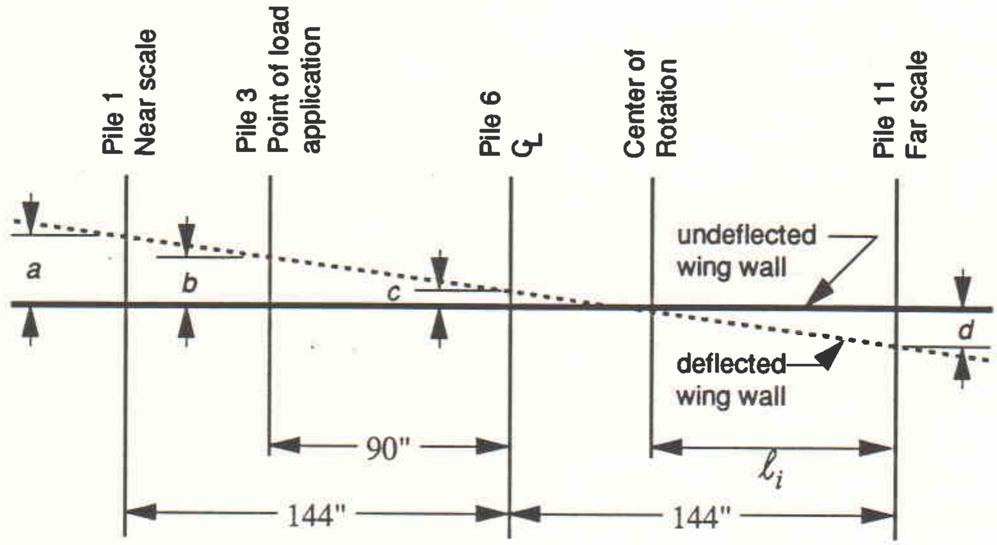
HEIGHTS REFER TO MEAN LOWER LOW WATER

FIGURE B-5

APPENDIX F

CALCULATIONS FOR DEFLECTIONS AT THE POINT OF LOAD

(For energy calculations it is necessary to use the force vs. deflection relationship for the point of load application. However, during the field test, the location of the deflection readings did not match the point of load application. The following tables show calculations for deflections at the point of load application.)



Calculated Load	Deflections (in.)				
	Pile #1 Near Scale *(a)*	Pile #3 Point of Load *(b)*	Pile #6 Middle Scale (c)	Pile #1 Far Scale (d)	Center of Rotation (in.)l_i
9.39	1.5	1.15	0.4	0.8	62.6
18.78	2.4	1.67	0.5	1.4	75.4
28.2	2.5	1.75	0.5	1.5	77.1
37.6	2.7	1.88	0.5	1.7	81.4
46.9	2.95	2.03	0.5	1.95	86.5
56.3	3.2	2.15	0.4	2.4	100.5
65.7	3.4	2.28	0.4	2.6	102.6
75.1	3.6	2.4	0.4	2.8	105.0
82.6	3.7	2.46	0.4	2.9	106.1
84.5	3.7	2.50	0.5	2.7	97.2
90.1	3.8	2.56	0.5	2.8	98.4
93.9	3.8	2.56	0.5	2.8	98.4

Notes:

Dimension a and c from field (see Appendix C, Test #2, 5-29-90)

$$b = [\frac{90}{144} \text{x} (a - c)] + c$$

$$d = a - 2(a - c)$$

$$l_i = \frac{90}{b} \text{x} d$$

CORRECTION FOR DEFLECTION AT POINT OF LOAD

Date: 5-29-90

Test #7

Calculated Load (kips)	Reflection Pile #3 *(a)* (in.)	*(b)* (in.)	*(c)* (in.)
0			
9.2	.8	0.69	-0.2
18.4	1.1	0.86	-0.4
27.6	1.3	0.98	-0.5
36.8			-0.6
46.0	1.8		
55.2	2.1	1.55	-0.8
64.4	2.3	1.69	-0.9
73.6	2.5	1.84	-1.0
82.8	2.7	1.98	-1.1
92.0	2.9	2.12	-1.2
101.2	3.1	2.26	-1.3
110.4	3.3	2.41	-1.4

Positive deflections are away from the berth.
Negative deflections are toward the berth.

Note: *a* and *d* from field measurements

$$b = \frac{234}{288}(a - d) + d$$

CORRECTION FOR DEFLECTION AT POINT OF LOAD

Date: 5-29-90

Test #5

Calculated Load (kips)	Deflection *(a)* (in.)	*(b)* (in.)	*(c)* (in.)
18.5	0.0	1.38	1.7
27.8	-0.15	1.84	2.3
37.0	-0.15		
46.3	-0.15	2.07	2.4
55.5	-0.15	2.08	2.6
64.8	-0.15	2.17	2.7
74.0	-0.2	2.16	2.7
83.3	-0.2	2.04	2.8
85.1	-0.2	2.52	2.9

Positive deflections are away from the berth.
Negative deflections are toward the berth.

Note: *a* and *d* are field measurements

$$b = \frac{234}{288}(d - a) + a$$