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Wharf collapse at Baddeck, Nova Scotia

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Publisher’s version / Version de l'éditeur:
https://doi.org/10.4224/40003475
Laboratory Memorandum (National Research Council Canada. Hydraulics Laboratory); no. HY-65, 1966-05-16

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L.O.	NATIONAL RESEARCH COUNCIL	NO. HY-65
FILE CM2-17-14P-8	DIVISION OF MECHANICAL ENGINEERING OTTAWA, CANADA	PAGE 1 OF 19
PREPARED BY T.M.D.	LABORATORY MEMORANDUM	COPY NO. 1
CHECKED BY J.W.K.	SECTION HYDRAULICS	DATE 16 May 1966

SECURITY CLASSIFICATION LIMITED

SUBJECT WHARF COLLAPSE AT BADDECK, NOVA SCOTIA

PREPARED BY T. M. Dick

ISSUED TO Department of Public Works,
Ottawa, Canada.

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Main Report

Supplement to Report

COLLISION OF S.S. "BIRKNACK" WITH THE GOVERNMENT WHARF AT BADDECK, NOVA SCOTIA

1.0 Reference

On the 28 December, 1965, the Department of Public Works requested in a letter that a technical study of the wharf at Baddeck should be undertaken by the National Research Council.

2.0 Objective

The extension to the wharf at Baddeck was struck by the S.S. "Birknack" as it was departing after taking on cargo. After the collision the extension of the dock collapsed and this study is aimed at finding out possible reasons for the structural failure.

3.0 Wharf Plans and Construction

Details of the wharf are contained in the specifications and their accompanying plan (Ref. No. E-197). Additional information concerning the types of wood used to construct the wharf, the pile penetration and the design live load was supplied in a letter from Public Works dated 11 January 1966.

4.0 Type of Structure

The wharf extension was a typical piled wharf constructed of timber with a reinforced concrete deck.

Point bearing timber piles arranged in parallel bents supported the deck. Across the top of the piles 12" x 12" pile caps and corbels supported a laminated timber sub-deck upon which a reinforced concrete deck was laid.

The laminated sub-deck was composed of alternating 2" x 4" and 2" x 6" creosoted planks placed on edge and spiked together. Grooves and daps were made to provide shear connection between the sub-deck and reinforced concrete deck. Reinforcing of the slab was ½" diameter bars at 9" o.c. placed in each direction (viz. parallel and normal to slab span).

Referring to sketch Fig. 1, brace piles were driven to prevent swaying in the OY direction. No brace piles were driven in the OX direction.

5.0 Description of Accident

The events leading to the collision of the S.S. "Birknack" with the dock are not entirely clear. However it appears that the ship when going astern struck the dock on the east side at a point 12 feet distant from the south east corner. After the impact the ship did not come to rest until the stern reached a point about 20 feet past the eastern face of the wharf.

Estimates of the ship's speed at the moment of impact vary. The lowest estimate, by the pilot, is ¼ knot and the next nearest is 1 knot. It is worth noting that the ship's astern speed seems to have been high enough to make some of the spectators realize a collision was imminent and inevitable.

Descriptions of the damage to the dock and ship and other evidence indicates that the ship struck the dock almost directly stern on. The impact was severe enough to fracture the concrete slab and cause an indentation of 4-5" in the edge of the concrete deck.

6.0 Assumptions and Checking Procedure for Piles

Before the lateral stability under impact can be assessed the vertical loads on the piles must be determined.

Bearing in mind that in the elastic design of a structure the usual procedure is to so proportion a structure that under load no part of it is stressed beyond allowable limits and since all the piles are the same size it is sufficient to check the adequacy of the most heavily loaded pile.

By inspection it was decided to check a typical bent of 6 piles. In Dwg. E-197 or Fig. 1 bents 3, 4, 5, 6 and 7 composed of piles in rows C, D, E, F, G, H, are typical. Bent 4 was selected and the loads computed for each pile.

6.1 Computed Loads on Piles

Details of the load calculations are contained in Appendix A. Live Load or design superimposed loading was stated by the Department of Public Works to be 250 lb per sq. ft.

The results are tabulated below in Table 1.

Table 1. Pile Vertical Loading. Typical Bent No. 4

Load in lbs.

Row	C	D	E	F	G	H
Dead Load	5073	7129	7655	7655	7129	5073
Live Load	9000	18700	19400	19400	18700	9000
Combined	14073	25829	27055	27055	25829	14703

It is clear that the most heavily loaded piles are those in rows EF and since they are all the same should have been chosen to support a load of 27,055 lbs.

6.2 Strength of Vertical Piles

The strength of a vertical structural member loaded coaxially in compression depends inversely on its slenderness ratio defined as:

Slenderness Ratio $= \ell_s / r_{min}$ where: ℓ_s = length of equivalent simple column

r_{min} = minimum radius of gyration.

For rectangular sections (breadth "b", depth "d") the minimum radius of gyration is $d^2/12$ but for purposes of design the timber codes employ ℓ/d in place of ℓ/r min.

Three end conditions are generally recognized in structural design, namely,

i free - without constraint, lateral motion and rotation possible

ii pinned - fixed in position but can rotate

iii fixed - fixed in position and prevented from rotation.

For various combinations of end condition; the length of the simple column is related to the actual column length as follows:

End Conditions:		ℓ_s/ℓ
Pinned	Pinned	1
Fixed	Pinned	0.7
Free	Fixed	2
Fixed	Fixed	0.5

6.3 Assumptions for Checking Design

Referring to Fig. 1 two buckling planes (viz. OX, OY) will be considered.

For both directions the lower end of the pile is fixed in the soil. The exact position in the soil below which the pile is considered fixed is doubtful. However, upper and lower limits can be set and these are indicated on Fig. 2. An assumption for normal design is also indicated but this is a matter of individual choice.

At the upper end of the pile, the conditions of fixity are not the same for directions OX and OY.

OY Direction: The piles are fastened to the corbels or pile cap by a 1" diam. x 30 inch drift bolt. Brace piles are connected to the pile caps effectively holding the pile ends in place. Furthermore piles D, E, F and G are also fastened to the brace piles by 1 ¼" diam. U-bolts about 4 feet below the pile cap or corbel. Consequently, piles D, E, F, G are considered "fixed" and piles H and D are considered "pinned" in the OY direction.

Hence ℓ_s for piles H and D equals $.7\ell$ the actual length and $\ell/2$ for piles D, E, F, G.

OX Direction: Positive structural means of holding the tops of the piles in position is not provided for direction OX. No brace piles are provided. As mentioned above the piles are fastened to the corbels by a single 1" diam. drift bolt.

A simple joint like this serves to hold the pile cap in place but provides only very limited or zero resistance to an applied bending moment. The chief reason preventing the development of end moment is the lack of resistance of the pile cap or corbel to compression perpendicular to the grain. With reference to Fig. 4. the end moment M(AB) is given by

$$M_{(AB)} = \frac{2EI}{\ell}\left[2\phi_A + \phi_B - \frac{3\delta}{\ell}\right]$$

$$\text{For } \phi_B = 0, \quad = 2EI/\ell\left[2\phi_A - 3\delta/\ell\right]$$

If there is zero end moment, i.e. $M_{AB} = 0$ then $2\phi_A = 3\delta/\ell$ Assuming sway occurs the end of the pile will tend to press into the pile cap in an effort to develop a resistance moment at the top. The maximum indentations at the pile edge for zero moment are tabulated below for a 40' pile.

Sway	ϕ_A	Maximum indentation required for zero moment
1"	1/320 radians	.0187"
2"	2/320	.0374"
3"	3/320	.0748"

These small indentations are easily realizable and consequently the joint is essentially pinned.

Some other secondary effects are also present. These are, resistance of the brace piles in bending, connection of the deck to the existing wharf (although structural positive connection was not made), the action of the wale and fenders along the edge of the wharf and the group action. Nevertheless as shown normal design procedure would be to consider that the top of the pile is able to sway and consequently for the OX direction the pile connection is "free". For this condition $\ell_s = 2\ell$. It is quite clear that the condition for the OX direction governs the strength of the pile since it is the weaker configuration.

6.4 Allowable Pile Loads

Details of the calculations are given in Appendix B. NOTE that the slenderness ratio of the piles for the fixed-free end condition exceeds the maximum permissible of 50 by code standards. Consequently, the design is inadmissible for this condition. However, loads have been computed in the manner directed by the code for purposes of comparison. Below in Table 2 the various allowable pile loads have been tabulated. The critical Euler load has been included.

Table 2. Allowable Pile Loads

*Not permitted. Slenderness ratio exceeds 50

Fixed-Free				Fixed-Pinned			Remarks
Design Length ℓ ft	Equivalent Simple Col ℓ_s ft	Allowable Load Pa lb.	Euler Load Pe lb	Equivalent Simple Col ℓ_s ft.	Allowable Load Pa lb.	Euler Load Pe lb	
43	86	1790 *	5490	29.1	14600	44800	Conservative
40	80	2080 *	6340	28.0	17000	51600	Normal
35	70	3950 *	10950	24.5	32200	89400	Risky

Note that the failure load is three times the allowable load. Calculation of the failure load can be considerably in error, owing to bends in the piles, eccentric loading, variations in the material properties and conditions of end fixity. This failure load is computed for a "fixed-free" condition. If the design conditions were "fixed-pinned" the loads are increased by a factor of $\frac{4}{0.49}$ or 8 times.

It is obvious therefore that provision of brace piles to ensure the "pinned-fixed" condition would have increased the allowable loads considerably. As it is the structure was in an intermediate and indeterminate condition. Calculation of the exact allowable load is in consequence not feasible and the safe decision would be to use "fixed free" conditions.

7.0 Comparison of Allowable and Actual Loads

Allowable and actual loads are listed in Table 3 below.

Table 3. Loads for Typical Bent

*Not permitted by Code. Slenderness ratio exceeds 50

Applied Loads Kips

Row	C	D	E	F	G	H
Dead Load	5,073	7,129	7,655	7,655	7,129	5,073
Live Load	9,000	18,700	19,400	19,400	18,700	9,000
Combined	14,073	25,829	27,055	27,055	25,829	14,073

Allowable Loads Kips

	Fixed-Free		Fixed-Pinned	
Length	Code	Euler	Code	Euler
35'	3,950*	10,950	32,200	89,400
40'	2,080*	6,340	17,000	51,600
43'	1,790*	5,490	14,600	44,800

It is quite clear that for the typical bent, the combined design load exceeds that allowable loads based on the "fixed-free" end condition and for column D E G F still exceeds the allowable load for "fixed-pinned" conditions. Clearly, the piles are overloaded on the basis of normal design procedures.

With the "fixed-free" condition, the dead load alone is of the same order as the critical Euler load. Consequently if the "fixed-free" condition became a reality, the dock would be in an unstable equilibrium condition.

8.0 Docking forces

Dynamic forces induced by the ship when docking are usually absorbed by the fender system. The dock structure is designed to resist the dynamic reaction from the fenders. Naturally, the distribution of forces will depend upon the type of fender employed and its capacity to absorb impact forces.

Common design assumptions are the following:

a) The ship approaches at a small angle to the dock face. An angle of 10° or so is often assumed.

b) After impact, the ship rebounds; the bow moving away from contact with the dock.

c) The component of the ship's velocity normal to the dock face varies from about 0.5 to 1.0 feet per second. The higher figure is appropriate for smaller ships in exposed locations and the lower for larger ships docking with the assistance of tugs.

d) Only half of the kinetic energy calculated from the normal velocity is absorbed by the fendering system and dock structure. This rule only applies for small angles of approach. With a head on collision, the total kinetic energy of the ship must be absorbed and not just the normal component.

Referring to Fig. 3 the kinetic energy E is given by

$$E = \frac{1}{2}\frac{W}{g}V^2$$

W = displacement weight of ship
g = 32.2 ft/sec^2
V = velocity normal to dock

Forces in the fender and structure are found by equating

$$\frac{E}{2} = \text{Sum of strain energy in fender and structure.}$$

For restitution after the load is removed no part of the structure or fender should exceed the elastic limit.

8.1 Weight of S.S. "Birknack"

Lloyds register of shipping records the ship dimensions as

Overall length	321'-2"
Extreme breadth	48'-5"
Summer draft	19'-11½"

From reports, the ship has a sloped stern and as a rough estimate the following waterline dimensions were selected:

Length	L	311 ft
Breadth	B	48 ft
Block coefficient	k.	0.85

At the time of the accident, Department of Transport pilotage records show that the maximum draft was 19'. The average draft is not known.

Substituting in,

$W = \frac{kLBDw}{2{,}000}$ where W = displacement
w = sea water density
and D = draft feet

one obtains $D = 18'$ $w = 7300$ tons (short)
$D = 19'$ $W = 7700$ tons (short)

For further calculations it is proposed to consider the displacement tonnage to be <u>7500</u> tons.

8.2 Kinetic Energy for Normal and Collision Conditions

Assuming that the ship approached the dock at an angle of 10° at such a velocity that the normal component was 1 **foot** per second then the kinetic energy normal to the dock is:

$$E = \frac{1}{2}\frac{W}{g}V^2 = \frac{7500 \times 2000}{2.32.2} \times (1)^2 = \underline{233{,}000} \text{ ft lbs}$$

Of this the dock and fender is assumed to absorb

$$E/2 = 116{,}500 \text{ ft. lbs.}$$

With an astern on collision the normal component of velocity equals the ship velocity. Estimates of the ship speed vary but the lowest is ¼ knot and the next 1 knot. Below have been tabulated the total kinetic energies of the vessel approaching along a path normal to the dock.

Table 4. Total Kinetic Energy

Ship Velocity		Kinetic Energy
Knots	ft/sec.	ft lbs
¼	0.42	41,400
½	0.845	165,000
¾	1.278	373,000
1	1.69	665,000
0.42	0.708	116,000

As shownin the Table a head on collision at 0.42 knots or 0.708 ft/sec is equivalent to the design conditions noted above.

8.3 Fenders on Baddeck Dock

Examination of the fenders used at Baddeck reveals that they were ill adapted for absorbing energy and would do little but provide rubbing strips. Ships docking at an angle will easily strike the edge of the deck because of the flare of the bows. Furthermore the impact force can be applied directly opposite a pile bent or individual pile in the bent.

Consequently the timber fenders would absorb very little energy and the thrust during docking must be largely absorbed by the structure itself.

In the OY direction, the thrust would be opposed by the brace piles and since their deflection would be relatively low, docking forces would be high and damage to the fenders or ship would result. In the OX direction the piles would sway and lower forces could be expected.

In Appendix C the behaviour of the piles under axial load and thrust have been investigated. It is seen that if the axial load approaches the Euler collapse load the applied bending moment increases quickly to a very large value indicating that no lateral thrust can be maintained. The usual code rule for design confirms the result.

At impact, the energy of the ship would have been opposed by the brace piles in bending. The calculations in Appendix C indicate that the resistance of the brace piles in bending is trivial and they were required to provide 33 times their allowable load based on the lowest estimate of the ship's velocity at impact.

These results indicate the reason for the abrupt failure of the dock and the lack of damage to the ship.

9.0 Conclusions

1. The vertical loads on the piles making up the main bents exceed allowable loads based on usual engineering design procedures.

2. Owing to the excessive vertical load the dock was in a state of incipient failure. The force of the collision overcame the secondary factors giving stability and placed the

piles unequivocally in their most unfavourable condition for axial load. Once this condition was established, immediately after impact, the dock would probably collapse under its own weight. The continuing force of the ship aggravated the situation giving rise to immediate and complete destruction of the structure.

3. The only force available to resist the ship was that of the brace piles in bending. These were quite inadequate to resist the collision of the ship even if it were moving at the lowest estimated speed of ¼ knot.

SUPPLEMENT TO REPORT

WHARF COLLAPSE BADDECK, NOVA SCOTIA

1.0 Reference

Re: Letter from G. Millar 24 March 1966

An evaluation of the wharf has been requested if brace piles had been placed to prevent motion in the OX direction.

2.0 Assumed Design Conditions

1. The brace piles in the OX direction will be at the same angle to the vertical as used for the OY direction.
2. The concrete and timber deck is capable of distributing the impact forces.
3. Brace piles will be placed at bents 2 to 8 inclusive and in rows G to D.

3.0 Typical Brace Pile Arrangement

An outline sketch of the arrangement for a brace pile unit is shown in Figure 3. Two brace piles are employed. The horizontal thrust F is shown as acting from the right. Vertical loading of dead and live load is represented by W. Note in the structure brace piles in bents 2 and 8 are considered one unit.

4.0 Effect of Additional Brace Piles for OX Direction

4.1 Vertical Loads

Additional brace piles effectively prevent the deck from moving in the OX and OY direction.

Design conditions would be for the fixed-pinned configuration. Consequently the minimum vertical strength is as listed in Table 3 for the fixed-pinned situation. Although additional brace piles represent a substantial improvement the dead load alone still overloads many of the piles above their design value.

4.2 Lateral Loads

Sway or impact is resisted by the brace piles. The typical arrangement shown in Fig. 3 is structurally redundant, that is to say there is one more member than really necessary to ensure stability. Methods of solution for statically indeterminate structures depend upon exact knowledge of the elastic properties of the material and the joint behaviour. The joints used in timber construction may have greater or lessor looseness and the elastic properties of the wood vary with direction of stress, humidity and source. Consequently, the computation of forces by elastic methods is an exercise in futility.

The matter can be resolved by making the usual engineering assumptions.

It is assumed:

a) The vertical load W is carried wholly by the vertical pile qs.

b) Force F is resisted by the piles pq and qr.

c) Piles pq and qr contribute equally.

Assumption (c) is only true up to some limit. As force F increases either pile pq or pile rq will fail first. Pile pq may collapse by buckling or pile rq may pull out of the soil.

The limiting strength of a pile in tension is very indeterminate and can only be obtained with certainty by testing on the site. However an estimate can be made by using published values to ascertain reasonable limits of strength for the tension pile.

The brace piles for the wharf are embedded 26 feet through layers of soft silt, soft clay and clay of which we shall assume 24 feet has effective skin friction. Representative forces can be calculated from published values. Terzaghi and Peck[1] and Hough[2] suggest ultimate values of skin friction varying between 200 to 600 lbs/sq. ft of surface area. Table 5 below lists the ultimate tensile force for pile rq in tension.

TABLE 5 Ultimate Pull Out Loads for Pile in Tension

Skin Friction Stress	Pile Surface Area	Ultimate Load
200 lb/sq. ft	43.9 sq. ft	8,780 lb
400	43.9	17,560
600	43.9	26,340

---

1. Terzaghi and Peck. Soil Mechanics in Engineering Practice.
2. Hough. Basic Soils Engineering.

In Appendix D, the design compression load for pile pq is 25,000 lbs and the failure euler load is 38,300 lbs. Clearly if F is increased to the limit of failure the tension piles would fail first. In the working range the tension piles may or may not exceed the limiting pull out load depending upon the soil character.

4.3 Engineering Assumptions

The practical solution and one often adopted is to consider only the piles in compression, providing enough of them to resist the design horizontal force in safety. The tension piles are considered redundant but of course they serve to stiffen and strengthen the structure. For forces in the opposite direction, the piles exchange roles.

4.4 Impact Loading

Calculation of forces in a structure due to impact is a complex problem. With ships the usual procedure is to assume that the ship approaches at a certain speed and angle to the dock and the ½ of the kinetic energy of the velocity component normal to line of the wharf is absorbed by the fender and structure. The equation can be expressed as follows:

Kinetic energy of ship transferred to dock = Work done = Strain energy of structures and fenders.

$$k\left(\frac{\text{mass } V^2}{2}\right) = \int_0^X F\,dx = \tfrac{1}{2} FX = \text{Strain energy}$$

where V is the normal component of velocity
k is a transfer factor
f is the force on the dock
X is the displacement of the point of impact in direction normal to wharf line.

Fenders ensure that "X" is large which reduces F. A small displacement X causes a larger force F. Theoretically if X = 0 then F becomes infinite.

5.0 Computation of forces and displacements

Appendix D contains details of the computations of permissible deflections and the allowable and ultimate forces for a unit consisting of one batter pile and one vertical pile.

Using the same general arrangement of batter piles as for the OY direction, it is feasible to drive 24 sets of batter piles in the OX direction.

Consequently the available resistance force would be:

24 x 12,900 lb = 310,000 lb for design stresses
24 x 22,600 lb = 542,000 lb for Euler collapse

Below in Table 7, the value of X necessary to keep the forces within the limits above is tabulated for each collision speed.

Displacement Required to Absorb Collision Energy

Table 7.

Ship Collision Speed Knots	Kinetic Energy Ft lb	Necessary sway To design stress Inches	 to collapse Inches
¼	41,400	3.2	1.8
½	165,000	12.8	7.3
¾	373,000	28.9	16.5
1	665,000	51.2	29.4
Typical design	116,000	9	5.1

As shown in Appendix D, the permissible sway at design stresses is 0.58" and for Euler collapse 0.90". Consequently if the dock is not to be overloaded additional movement must be provided. The normal solution is to provide fenders which for a ship of the Birknack's size should provide at least 9" of movement to protect this particular dock.

6.0 Summary and Conclusion

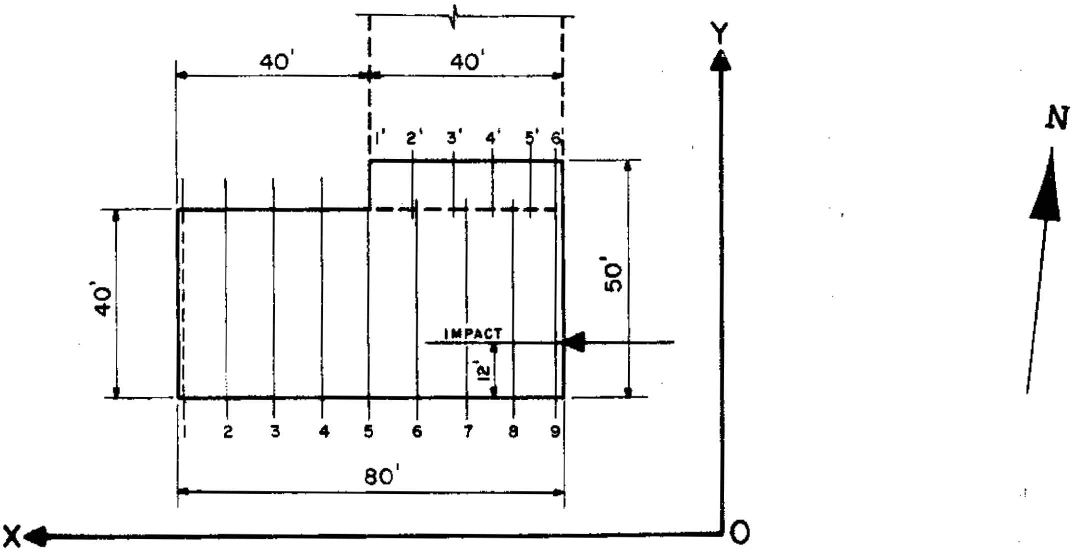
1. Addition of extra brace piles to resist forces in the OX direction would strengthen the structure and improve its capacity to carry vertical loads. However many of the piles would still have been overloaded.

2. Extra piles make the dock very stiff and without fenders extensive local damage to ship or dock would probably result from an injudiciously heavy impact during docking.

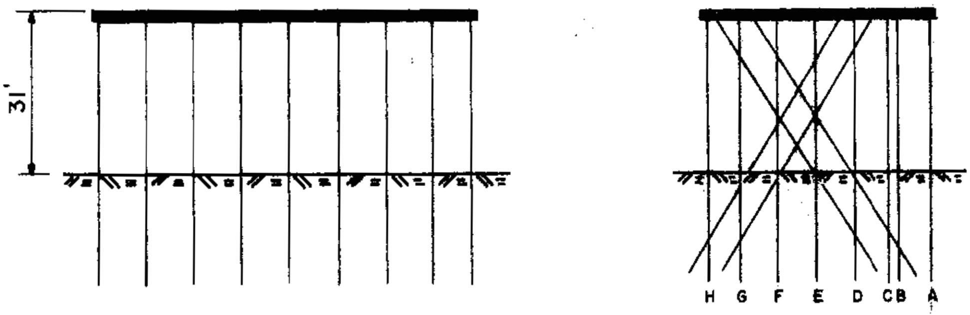
3. The dock would have probably survived a collision of ¼ knot but some local damage would have resulted to dock or ship. With higher collision speeds the local damage would progressively increase and general structural failure would be initiated.

4. A light timber piled dock should be protected with adequate fendering.

FIG. 1



PLAN



ELEVATION

END ELEVATION

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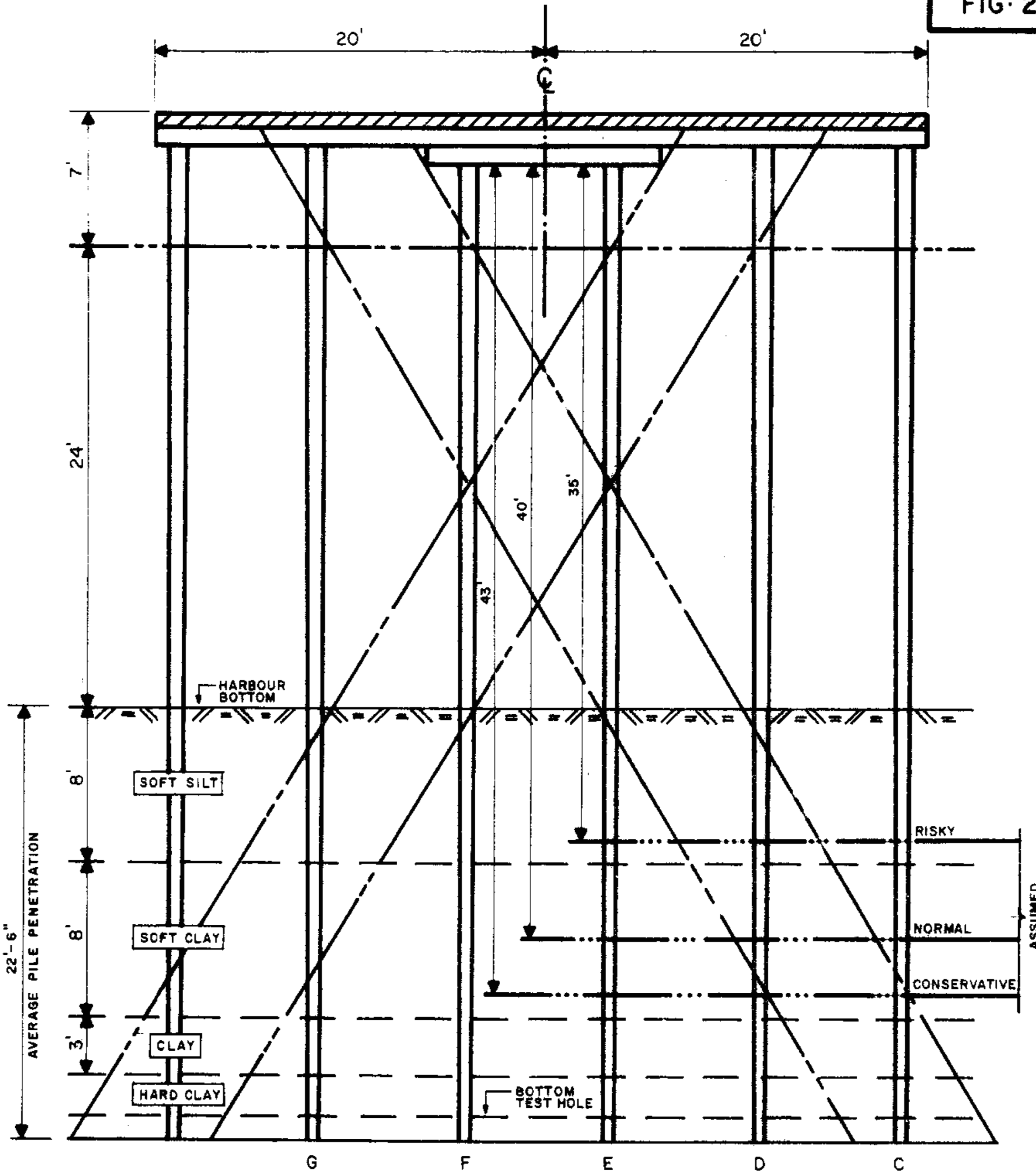
FIG· 2



TYPICAL BENT

1/8" = 1'

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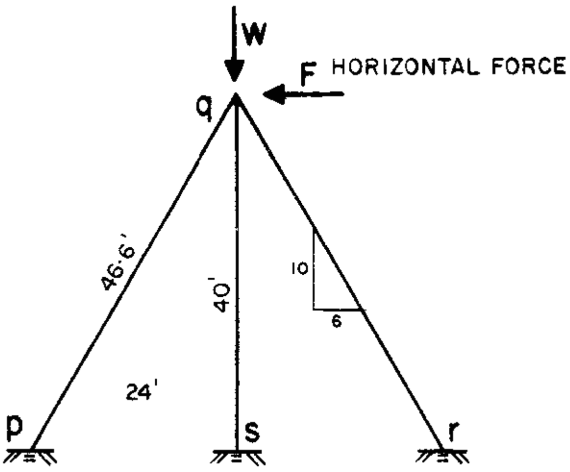
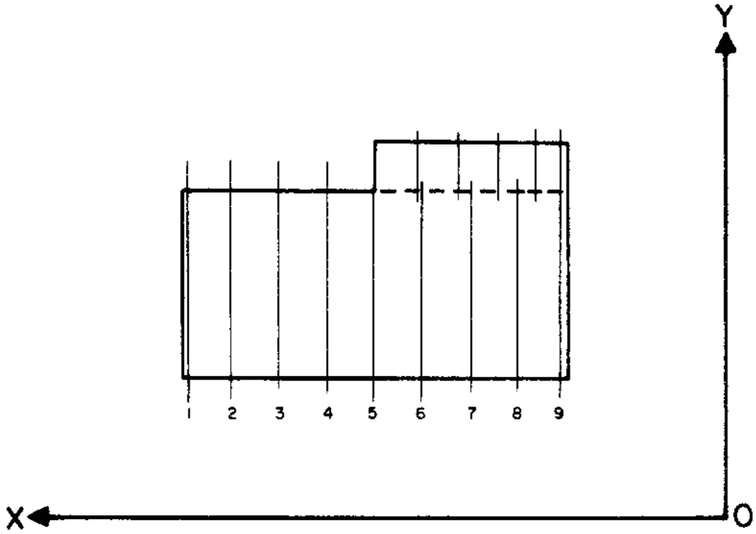
FIG· 3



TYPICAL UNIT BRACE PILES



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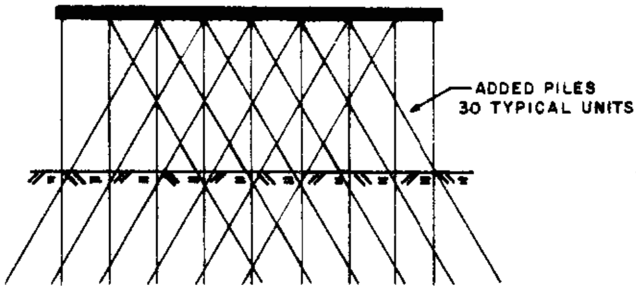

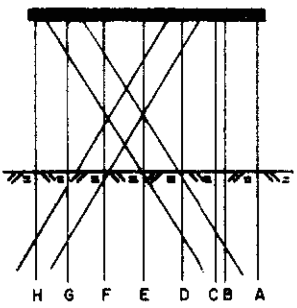

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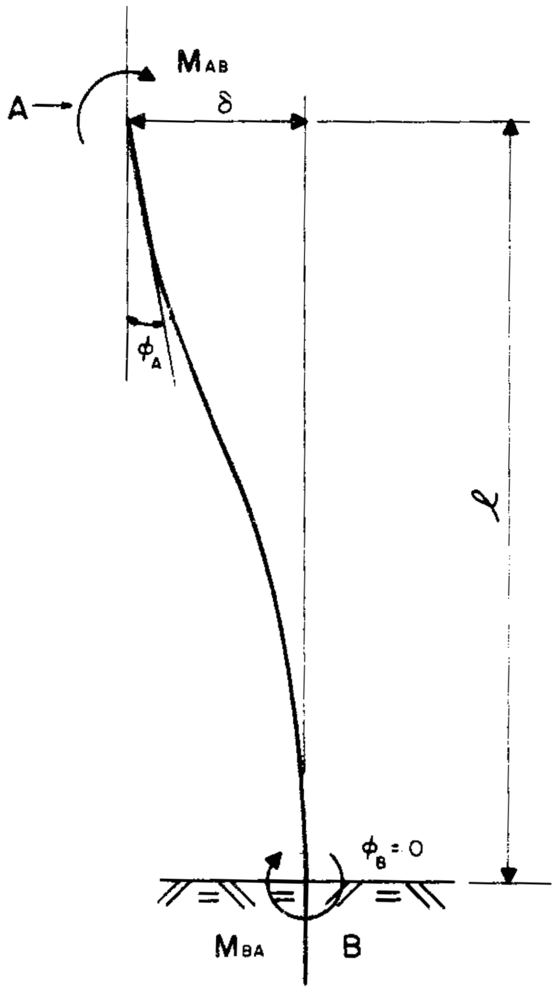
BRACE PILES ADDED

FIG. 4



$$M(AB) = \frac{2EI}{\ell}\left[2\phi_A + \phi_B - \frac{3\delta}{\ell}\right]$$

$$\phi_B = 0$$

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BADDECK WHARF

SWAY DIAGRAM

APPENDICES A, B and C

FOR LABORATORY MEMORANDUM HY-65

"Wharf Collapse at Baddeck, Nova Scotia"

by

T.M. Dick

Issued to: Department of Public Works,
Ottawa, Canada.

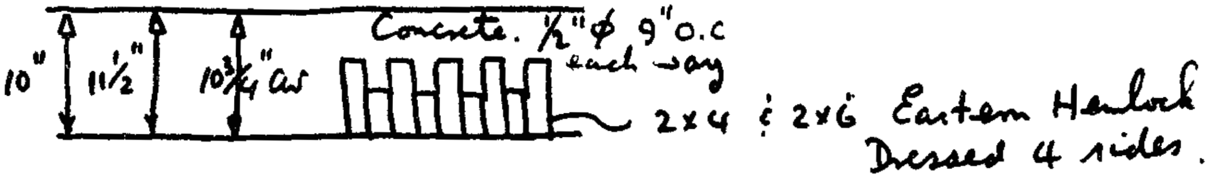
National Research Council
Hydraulics Section

16 May 1966

APPENDIX A

Appendix A Calculation of dead and live loads for typical bent.

Laminated Deck.



2×4 dressed = $1\frac{5}{8}" \times 3\frac{5}{8}"$
2×6 " = $1\frac{5}{8} \times 5\frac{1}{2}"$

Average thickness = $\dfrac{3\frac{5}{8}+5\frac{4}{8}}{2}$ = $4\frac{9}{16}"$

Vol. wood per ft² = $4\frac{9}{16} \times 12" \times 12"$ = 666 in³
neglecting grooves daps.

Total volume of deck = $12 \times 12 \times 10\frac{3}{4}$ = 1550 in³
Volume of concrete = 1550 − 666 = 884 in³.
Vol wood = 0.386 ft³/ft²
Vol concrete = 0.511 ft³/ft²

Wood density. Eastern Hemlock 30 lb/ft³
Creosote 10 "
40 "

Density of reinforced concrete with ½"φ at 9" o.c e.w. and unreinforced = 144.
Concrete density = 147 lb/ft²

Weight of wood = 0.386 × 40 = 15.4
conc = 0.511 × 147 = 75.2
90.6

Use Wt of deck = 91 lb/ft²

Pile Cap ē Corbel

B.C. fir = 34 lb/ft³
Creosote = 10
44.

12"×12" sawn timber.

Wt/ft = 44 ×1×1×1 = **44** lb/ft

Guard. Subguard and Chock.

Eastern Hemlock .. 30 lb/ft³ } 40
Creosote 10
Hardwood Chock 45 } 55.
Creosote 10

Weight for 10 lineal ft. of dock

Subguard. $\frac{10''\times 10''}{144} \cdot 10' \times .40$ = 278
Guard $\frac{6''\times 6''}{144} \cdot 10 \times 30$ = 75
Chock $\frac{2''\times 8''}{144} \cdot 3.5' \times 55$ = 21.4
374.4

Use. Wt per lineal foot = **37.5** lb

For fenders without subguard.
Weight = 75+21.4 = 96.4
Use wt/ft of dock = **10** lb.

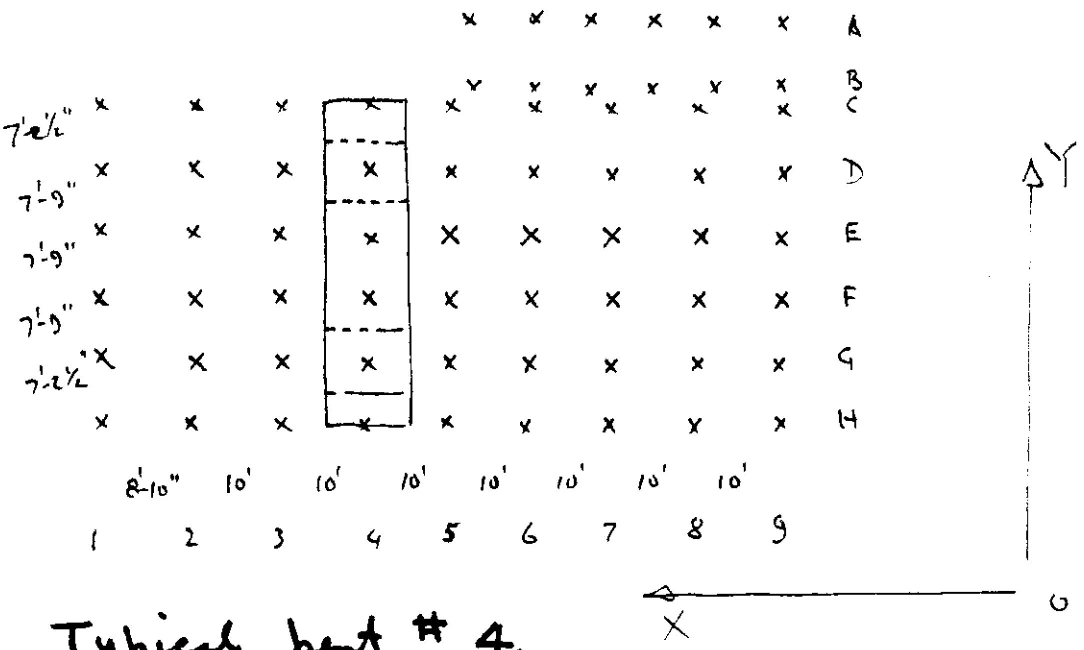
Wale 8"×10" Eastern Hemlock. . . 30 lb/ft³
Creosote 14
44.

wt per ft = $\frac{8''\times 10''}{144} \cdot 44$ = 24.4
Use .. **25** lb/ft

Fender Hardwood = 45
Creosote 12 } 57 lb/ft³

Weight of fender = $\frac{8''\times 8''}{144} \cdot 8' \cdot 57$ = **202** lb

Pile Loading.



Typical bent # 4.

Pile E. F.

Deck area = 10' × (7'-9" + 7'-9") = 10.15.5 = <u>155</u> ft^2

Wt of deck	= 155 × 91	=	14,100 lb
Wt of Pile Cap	15.5 × 44	=	682
Corbel	12 × 44	=	528
			15310

Dead Load / pile = $\frac{15310}{2}$ = <u>7655</u> lb

Live Load / pile = 250 . $\frac{155}{2}$ = <u>19,400 lb</u>

Pile G. & D.

Deck area = 10' × ($\frac{7.75 + 7.2}{2}$) = 74.8 ft^2.

Wt of deck =	74.8 . 91 lb/ft^2	=	6800 lb
" " pile cap =	7.48 × 44	=	329
			7,129

Dead load / pile = <u>7129</u> lb

Live load / pile 74.8 × 250 = <u>18,700</u> lb

Pile C.H.

Deck area = 3.6 × 10 = 36 ft^2

Wt of deck = 36 × 91 = 3280 lb

" " pile cap = 3.6 × 44 = 158

Wt of guard rubguard and chock = 10 × 37.5 = 375

Wt of Wale 10 × 25 = 250

Fenders 5 @ 202 1010

5073 lb

Dead load / pile = 5073 lb

Live load 36 × 250 = 9000 lb

APPENDIX B

allowable loads calculated for cols F or E

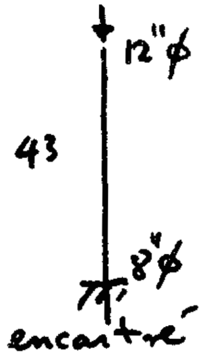
$K = 0.641\sqrt{\frac{E}{f_c}}$ $\qquad f_c = 0.9 \times 0.9 \times 1100$

Wet Cont. struct grade

$= 27.2$ $\qquad = \underline{\underline{890}}$ lb/in²

END CONDITIONS. Bottom fixed
Top free

① $l = 43'$ $\quad l_s = 2 . 43 = 86'$



$d_o = \text{Min } d + \frac{1}{3}(\text{Max } d - \text{Min } d)$
$= 8 + \frac{1}{3}(12 - 8) = 9.33$

$1\frac{1}{2} d_{min} = 12"$.

$l_s/d_o = \frac{2 . 43 . 12}{9.33} = 111 > K > 50$.

By code design not admissible since $l_s/d_o > 50$.

However. Calculating by same method.

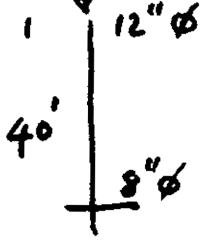
$P_a/A = \pi^2/36 \; E/(l_s/d)^2 = \frac{\pi^2 1.6 . 10^6}{36 . 111^2} = 35.6$ lb/in²

$P_a = 35.6 . \pi/4 \, d_{min}^2 = 35.6 . \frac{\pi}{4} . 8^2 = \underline{\underline{1790}}$ lb

Note: wt of pile & buoyant force neglected
Considered negligible.

$P_e = \frac{\pi^2 EI}{l_s^2} = \pi^2 \frac{1.6 . 10^6 . 0.049 \; 9.33^4}{(2 . 43 . 12)^2} = \underline{\underline{5490}}$ lb

2. $l = 40'$



d_o = as above $= 9.33$. $\quad d_{min} = 8"$

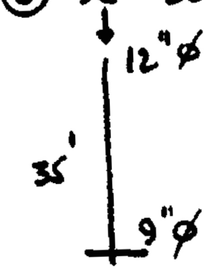
$l_s/d_o = \frac{2 . 40 . 12}{9.33} = 103 > K > 50$.

$\frac{P_a}{A} = \frac{\pi^2}{36} \, E/(l_s/d_o)^2 = \frac{\pi^2 . 1.6 . 10^6}{36 . (103)^2} = 41.4$ lb/in

$P_a = 41.4 . \pi/4 \, 8^2 = \underline{\underline{2080}}$ lb

$P_e = \frac{\pi^2 EI}{l_s^2} = \pi^2 \frac{1.6 . 10^6 . 049 \; 9.33^4}{(2 . 40 . 12)^2} = \underline{6340}$ lb

③ $\ell = 35.$



$d_0 = d_{min} + \frac{1}{3}(d_{max} - d_{min})$

$= 9 + \frac{1}{3}(12-9) = 10''$

$\frac{\ell_s}{d_0} = \frac{2 . 35 . 12}{10} = 84 > K > 50$

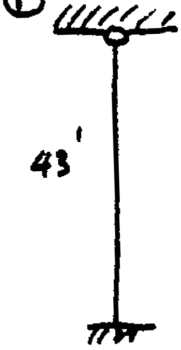
$P_a/A = \frac{\pi^2}{36} \cdot \frac{E}{(\ell_s/d_0)^2} = \frac{\pi^2 \cdot 1.6 \cdot 10^6}{36 \cdot 84^2} = 62.2 \text{ lb/in}^2$

$P_a = A_{min} \times 62.2 = \pi/4 \, (9)^2 \cdot 62.2 = \underline{\underline{3950}}$ lb

Euler. $P_e = \frac{\pi^2 EI}{(\ell_s)^2} = \frac{\pi^2 \cdot 1.6 \cdot 10^6 \cdot 0.049 \cdot 10^4}{(840)^2} = \underline{\underline{10,950}}$

END CONDITIONS. Bottom fixed
top pinned

①



$\ell = 43'$

$\ell_s = 0.7\ell = 0.7 . 43 . 12 = 361$

d_0 from before $= 9.33''$

$\ell_s/d_0 = 361/9.33 = 38.7 > K < 50$

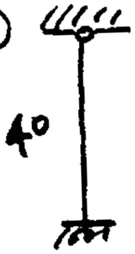
$P_a/A = \left(\frac{2}{0.7}\right)^2 \times$ Previous result $= 8.16 . 35.6$

$= 291$ lb/in

$P_a = 291 . \pi/4 \, 8^2 = 14,600$ lb

$P_e = (2/0.7)^2 . P_e$ before $= \underline{44,800}$ lb

②



$\ell = 40$

$\ell_s = 0.7\ell = 0.7 . 40 . 12 = 336$ in.

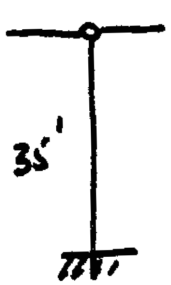
d_0 from before $= 9.33.$

$\ell_s/d_0 = \frac{336}{9.33} = 36 > K < 50.$

$P_a = \left(\frac{2}{0.7}\right)^2 . P_a$ before $= 8.16 . 2080 = 17000$ lb

$P_e = 8.16 . 6340 = 51,600$ lb

③



$l = 35'$

$l_s = 0.7 . l = 0.7 . 35 . 12 = 294$ in

$d_o = 10$ from before

$l_s / d_o = \frac{294}{10} = 29.4 > K < 50$

$P_a = 8.16 . P_a$ before $= 8.16 . 3950$

$= \underline{32,200}$ lb

$P_e = 8.16 . P_e$ before $= 8.16 . 10,950$

$= \underline{89,400}$ lb

APPENDIX C

Appendix C. Combined Bending and Axial Loading in Columns.

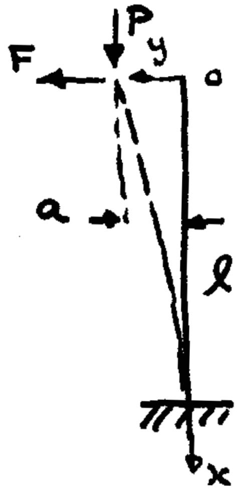
Code uses.

$$\frac{f_b}{F_b} + \frac{f_c}{F_c} \le 1$$

where: f_b = extreme fibre stress in bending
F_b = allowable bending stress
f_c = actual applied compressive stress by axial load
F_c = allowable axial compressive stress.

For this dock. the columns in the main bents are overloaded axially ie $f_c/F_c > 1$ ∴ Code requirement or rule cannot be applied

Ideal theoretical condition



Uniform column of length l, axial load P and horizontal force F at top. Origin of coords as shown +ve directions of x & y shown.

$$M = Fx + P(a-y) = EI\frac{d^2y}{dx^2} \quad \ldots 1$$

$$\frac{dM}{dx} = F - P\frac{dy}{dx}$$

$$\frac{d^2M}{dx^2} = -P\frac{d^2y}{dx^2}$$

$$\therefore \quad \frac{d^2M}{dx^2} + P\frac{d^2y}{dx^2} = 0 \quad \text{--}(2)$$

$$\frac{d^2M}{dx^2} + \frac{P}{EI}.\cancel{\frac{d^2y}{dx^2}}M = 0 \quad \ldots\ldots 3.$$

Let $\mu^2 = \frac{P}{EI}$ for convenience.

$\frac{d^2M}{dx^2} + \mu^2 M = 0$. Soln found by substituting $M = e^{\lambda x}$ and solving for λ

$\lambda = \pm i\mu$.

$\therefore \quad M = C_1 e^{i\mu x} + C_2 e^{-i\mu x}$ $C_1 C_2$ arbitrary.

This reverts to $M = A\cos\mu x + B\sin\mu x$

Let $x = 0$ $M = 0$. $0 = A\cos 0 + B\sin 0 \therefore A = 0$

$x = l$ $M = Fl + Pa$ $Fl + Pa = B\sin\mu l$

$$\therefore B = \frac{Fl + Pa}{\sin\mu l}.$$

Hence. $$M = (Fl + Pa)\frac{\sin\mu x}{\sin\mu l} \quad — 4$$

$M = M_{max}$ at $x = l$ obviously. $= Fl + Pa$.

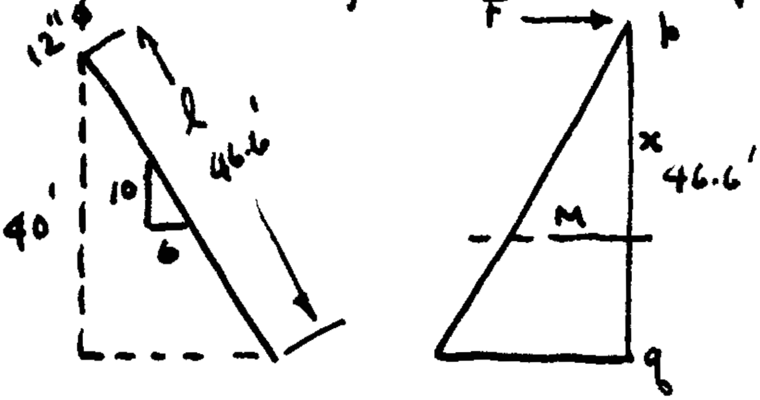
Since $P_e = \frac{\pi^2 EI}{l^2}$ $\therefore EI = \frac{P_e l^2}{\pi^2}$. and since $\mu = \sqrt{P/EI}$ it is possible to get.

$$M = (Fl + Pa) \cdot \frac{\sin (P/P_e)^{1/2}\pi \cdot x/l}{\sin (P/P_e)^{1/2}\pi} \quad \ldots ⑤$$

From 5 it is clear that as $P \to P_e$
$M \to \infty$

Resistance of brace piles after impact of ship.

Ship struck in dirn OX. Because vertical piles are overloaded and since there is no bracing in dirn OX then the Kinetic Energy of the ship must be resisted and absorbed by the brace piles in bending.



$M = Fx$

$M_{MAX} = \frac{f_b I_a}{y_a}$

$M_{MAX} = \frac{f_b I_a}{y_a} = \frac{f_b \cdot \pi/64 \; 7^4}{3.5''}$. $f_b = 1700 \cdot 0.90 \cdot 2$ wet impact

$= 3060 \text{ lb/in}^2$

$= \underline{10,300} \text{ lb ins}$

$F_{MAX} = \frac{M}{l} = \frac{10300}{46.6 \cdot 12} = 18.4 \text{ lb}$ — This is a trivial resistance. The actual resistance is possibly somewhat higher especially if the ground gives more support.

Strain Energy of bending $= \int_0^{x=l} \frac{M^2 dx}{2EI}$

$= \int_0^l \frac{F^2 x^2 dx}{2EI}$

Assuming I constant. F constant and E constant and averaging the value of d the diam so ~~that~~ $d_{av}^4 = \Sigma d^4$ then. as first approximation is

$$\bar{I}_{av} = \frac{\pi}{64} . d_{av}^4 = \frac{\pi}{64} . 10^4 = 490.$$

Also S.E. bending $= \frac{F^2}{2EI} \frac{l^3}{3}$ after integration

$$\therefore SE = \frac{F^2 . (46.6 . 12)^3}{6 . 1.6 . 10^6 . 490} = F^2 . 0.0373 \text{ in lb.}$$

Strain Energy which will equal the kinetic energy for lowest estimate of velocity

$\Sigma SE. = K.E. = 41400$ ft lb

$= 41400 \times 12$ in lb.

S.E per pile × number of piles = KE

$F^2 . 0.0373 \times 36 = 41400 . 12$ in lb

$F = 608.$ lbs.

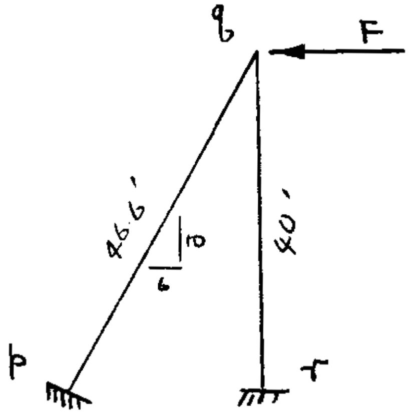
This force F per pile exceeds the force available at the design stress by 33 times.

Clearly, the lateral resistance of the dock to the ship impact is insufficient and negligible. It would still be inadequate for a design K.E of 116,500 ft lbs.

Under the impact, the ~~lateral~~ brace piles would fail and the vertical piles collapse.

APPENDIX D

SUPPLEMENT CALCULATIONS



FORCES pq $-1.94F$ (compression) limiting force.

rq $+1.67F$ (tension).

Allowable load for compression member pq.

End fixity Bot'm .. fixed

Top ... pinned

$l = 46.6'$ $l_s = 46.6 \times 0.7 = 32.6$

$d_o =$ Min d $+ \frac{1}{3}$ (Max d − Min d) $= 9.33$

d min $= 8''$ $\therefore d_o = 9.33''$

d max $= 12''$

$$l_s/d_o = \frac{32.6 \times 12}{9.33} = 42$$

$\therefore \frac{P_a}{A} = 250$ lb/in² For impact $\frac{P_a}{A} = 500$ lb/in².

$\therefore P_a = 500 \text{ lb/in}^2 \times \pi/4\ d^2_{\text{min}}$

$= 500 \times \frac{\pi}{4} \cdot 8^2 = \underline{\underline{25000 \text{ lb}}}$

Euler load

$$P_e = \frac{\pi^2 E I}{(l_s)^2} = \frac{\pi^2 \times 1.6 \times 10^6 \times 0.049 \cdot 9.33^4}{(32.6 \times 12)^2}$$

$$= \underline{\underline{38300 \text{ lb}}}$$

Maximum loading

Member	Design Stresses	Euler stress.
pq	−25,000 lb (1.94F)	38,300 lb
qr.	21,400 lb	33,000 lb
Force F	12,800 lb	19,750.

Change in length of structural members.

By Hooke's law.

$$\text{Stress} = E \times \text{Strain}.$$

$$P/A = E \times \frac{de}{dx}.$$

$$\therefore de = \frac{4P}{E\pi} \cdot \frac{dx}{d^2}$$

It will be assumed ~~that~~ the thrust ~~stress~~ diagram is rectangular throughout the length of the pile:

The diameter $d = \left(12 - \frac{0.1}{12} x\right)$ x in inches.

$$\therefore de = \frac{4P}{E\pi} \cdot \frac{dx}{\left(12 - \frac{0.1x}{12}\right)^2}$$

$$e = \frac{4P}{E\pi} \int_0^{\ell} \frac{dx}{\left(12 - \frac{0.1\,x}{12}\right)^2}$$

$$= \frac{4P}{E\pi} \left[\frac{12}{0.1\left[12 - \frac{0.1x}{12}\right]} \right]_0^{\ell}.$$

③D

change in length will be calculated for the whole length of the pile to the bearing stratum.

Pile pq. $l = 60' = 720''$

$$\varepsilon = \frac{4.25000}{1.6.10^6.3.14}\cdot\left[\frac{1440}{144-0.1\times720} - \frac{1440}{144}\right]$$

$$= \underline{0.198''}$$

By ratio $\underline{\text{Euler } \varepsilon = 0.305''}$

Pile rq. $l = 50' = 600''$

$$\varepsilon = \frac{4.21400}{1.6.10^6.3.14}\cdot\left[\frac{1440}{144-0.1(600)} - \frac{1440}{144}\right]$$

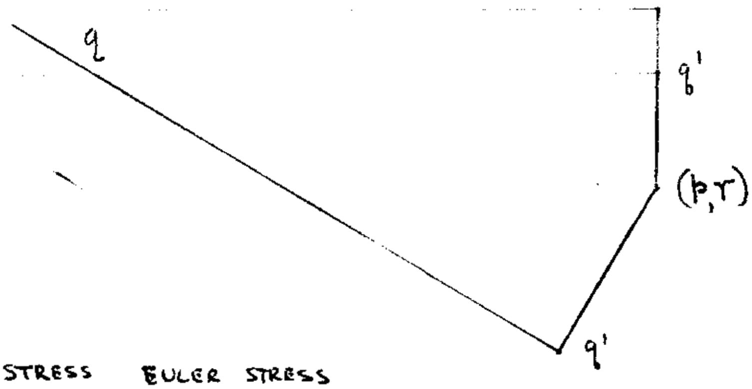
$$= \underline{\underline{0.122''}}$$

By ratio ε euler $= \underline{\underline{0.188}}$

These extension ~~will~~ possibly be somewhat greater than would occur in the structure since soil friction neglected ~~for~~ all other conditions being met. However, ~~give~~ yield in the soil, looseness of joints etc. will tend to increase the displacement. The displacements are thought to be representative.

Displacement Diagram.

JOINT. q. SCALE 0.1" DEFLECTION = 0.5"
OR 1" = 0.2" DEFLECTION



	DESIGN STRESS	EULER STRESS
HORIZ. DEFLECTION	0.58	0.90
VERT "	·12	·19

Structure can sway 0.58" without exceeding design compressive stress for dynamic loading and could reach 0.90" without failure.