Sam
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Chandler's Wharf

Pile Calculations
85-794

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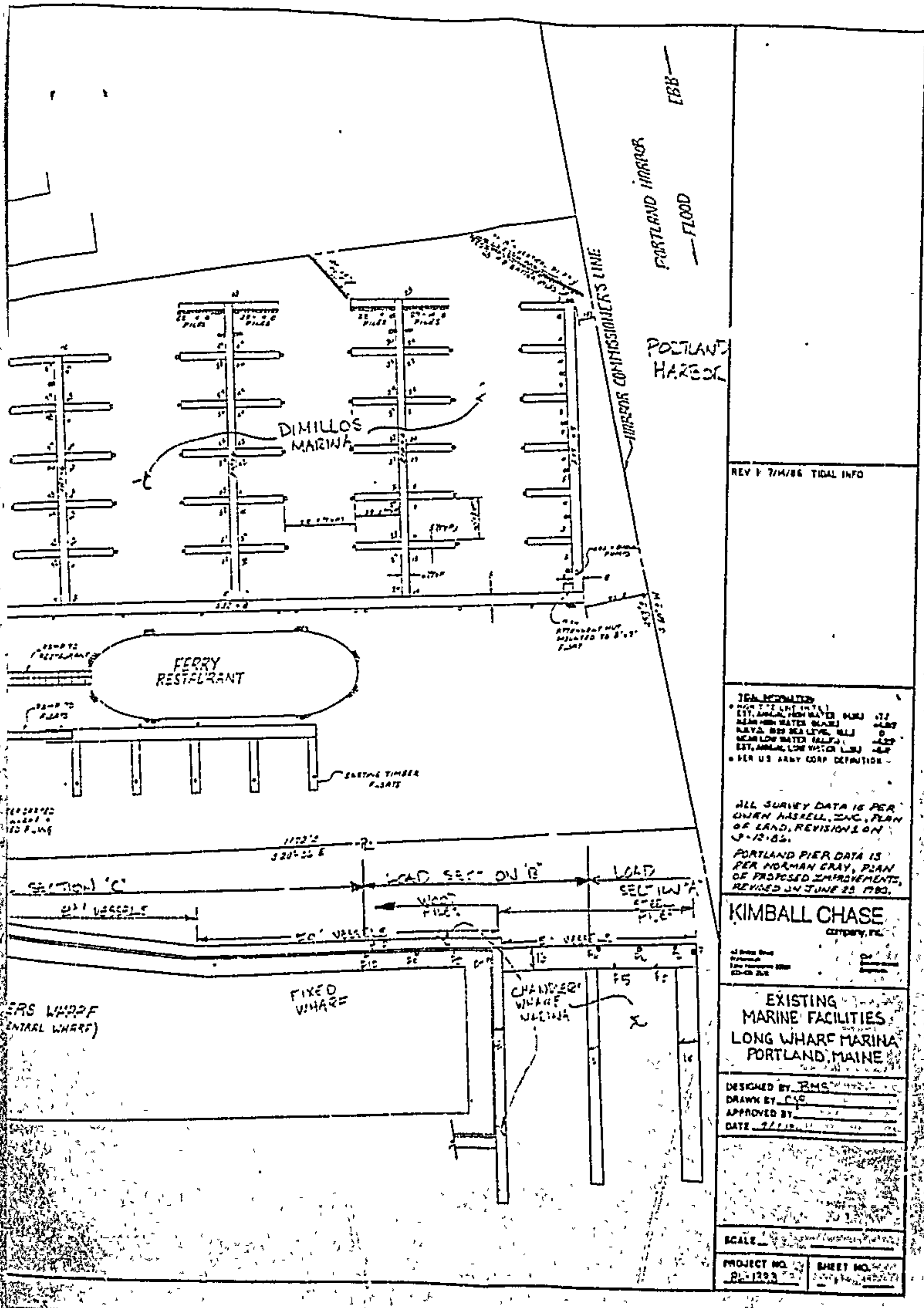
INTRODUCTION:

Chandler's Wharf and Marina is located on the north side of Portland Harbor just west of Dimillo's Marina and Ferry Restaurant in Portland, Maine (See Plan "1"). This site will experience several types of loadings, the magnitudes of which are based on the likelihood of the load occurring during a certain time period. The loadings which will be considered are: wave loading, wind loading, current loading and berthing loading. The time periods, in which a certain load is likely to occur, return periods, which will be considered will be 1,10,25 and 50 years. The 50 year return period will be the design condition. The site has been broken down into three separate load sections (See Plan "1"). The outermost section (Section A) is completely exposed to the north-east and as such, will receive 100% of the calculated wind and wave loads. (The longest open fetch of water and, usually, the strongest winds are out of the north-east quadrant.) The next section inboard, Section "B", receives protection from the north-east from Portland's waterfront including Dimillo's Marina's breakwaters and floating piers as such, wind and wave loads were reduced by 33%. In Section "C", the innermost section, the loads were reduced even further due to the protection provided by the Ferry Restaurant and the fixed wharf (Long Wharf). The wind load in Section "C" was taken to be 50% of the calculated load, and the maximum wave height as 1 foot. The boat sizes of the longest boats allowed are shown on Plan "1". In summary these are: 80' foot boats in Section "C" in the area marked on Plan "1" and 50' boats elsewhere. The following document is a summary of the calculations performed on the existing structure to determine suitability of the structure to facilitate vessels of this size. The document includes calculations, diagrams, references and recommendations.


EBB
PORTLAND HARBOR
FLOOD
HARBOR COMMISSIONERS LINE
PORTLAND HARBOR
DIMILLOS MARINA
REV 1 7/14/86 TIDAL INFO
FERRY RESTAURANT
ALL SURVEY DATA IS PER OWEN HASKELL, INC., PLAN OF LAND, REVISION 1 ON
PORTLAND PIER DATA IS PER NORMAN GRAY, PLAN OF PROPOSED IMPROVEMENTS, REVISED ON JUNE 25 1980.
SECTION 'C'
LOAD SECTION 'B'
LOAD SECTION 'A'
FIXED WHARF
CHANDLERS WHARF MARINA
KIMBALL CHASE
company, inc.
EXISTING MARINE FACILITIES
LONG WHARF MARINA
PORTLAND, MAINE
DESIGNED BY RMS
DRAWN BY
APPROVED BY
DATE
SCALE
PROJECT NO.
SHEET NO.

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CLIENT LIBERTY GROUP
PROJECT CHANDLER'S WHARF JOB NO 85-794
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WATER ELEVATIONS.

RETURN PERIOD (YEARS)	STILL WATER FLOOD ELEV. (SL)
1	7'
10	8'
25	8.5'
50	8.75'

DATUM: ELEVATIONS BASED ON 1929 NGVD [SEA LEVEL (SL)]
REFERENCE: EES INC. WAVE SURVEY, [SEE APPENDIX C]

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CURRENT LOAD:

EQUATION: $P_c = CV^2$

WHERE: P_c = PRESSURE DUE TO CURRENT

V = VELOCITY OF CURRENT

C = EMPIRICAL COEFFICIENT

REFERENCE: FLOATING PORTS, TSINKER EQ. (2-5) p. 92

MAXIMUM CURRENT VELOCITY = 1 KNOT

REFERENCE: BOATING ALMANAC, VOL. 1. p 257

CHOOSE C = .80 TO ALLOW FOR HULL SHAPES AND EXPOSED FLOATING PIER STRUCTURE.

REFERENCE: FLOATING PORTS, TSINKER p. 92

$$P_c = .80\ \frac{kN\ s^2}{m^4} \times \left(\frac{.3048\ m}{ft}\right)^4 \times \left(\frac{1}{4.448\ N}\right) \times \left(\frac{1000\ N}{kN}\right) \times (1\ \text{knot})^2 \times \left(\frac{1.7\ ft/sec}{\text{knot}}\right)^2$$

P_c = 4.5 lb/ft²

HOWEVER, MAXIMUM CURRENT OCCURS DURING EBB TIDE JUST BEFORE LOW TIDE WHILE MAXIMUM BENDING MOMENT ARM OCCURS DURING FLOOD TIDE. ALSO, THE CURRENT LOAD IS IN THE OPPOSITE DIRECTION TO THE OTHER DESIGN LOADS. THEREFORE, THE CURRENT LOAD WILL BE NEGLECTED.

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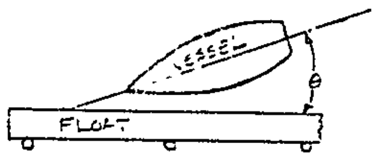
VESSEL BERTHING LOADS:



EQUATION:

$$E = \frac{[W_1 + W_2]\, V_n^2}{2g} K$$

WHERE: E = TOTAL IMPACT ENERGY

W_1 = DISPLACEMENT OF VESSEL

W_2 = HYDRODYNAMIC MASS

V_n = VESSEL VELOCITY NORMAL TO FLOAT

K = ECCENTRICITY FACTOR

g = GRAVITATIONAL ACCELERATION (32.2 FT/SEC²)

REFERENCE: FLOATING PORTS TSINKER ED (1986) p. 127

ASSUMPTIONS. V = 1 KNOT

ONLY VELOCITY WHICH IS NORMAL TO FLOAT WILL BE TAKEN INTO ACCOUNT, SINCE PARALLEL LOADS ARE NOT IN THE DIRECTION OF THE OTHER MAXIMUM DESIGN LOADS.

V_n = [SIN θ] × V = [SIN 10°] × 1.0 KNOT × 1.7 FT/SEC/KNOT = .295 FT/SEC

θ = 10°

W_1 FOR 50' VESSELS = 40,000 ± lbs } REFERENCE: BSCES, 1986
W_1 (FOR 80' VESSELS) = 120,000 ± lbs } WATERWAY SEMINAR (SEE APPENDIX A, CHART 1)

W_2 = C_h × W_1 REFERENCE: FLOATING PORTS, TSINKER, p. 121

C_h = 2.5 REFERENCE: FLOATING PORTS, TSINKER FIG. 2.18 (SEE APPENDIX A) CHART [illegible]

K = .50 (FOR 1/4 POINT BERTHING)
REFERENCE: FLOATING PORTS, TSINKER, FIG. 2.20 (SEE APPENDIX A) - CHART [illegible]

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VESSEL BERTHING LOADS (CONTINUED):

RESULTS:

VESSEL SIZE (FEET)	IMPACT ENERGY (FT-K)
50	120
80	400

CONCLUSIONS: IT IS UNLIKELY THAT THE BERTHING LOAD WILL OCCUR WITH ALL OTHER LOADS AT THEIR PEAKS. THEREFORE, BERTHING LOADS WILL NOT BE INCLUDED IN THE TOTAL DESIGN LOAD.

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Wind Load

WIND SPEEDS (MPH) SUSTAINED					
	PERCENT	RETURN PERIOD (YEARS)			
LOAD SECTION	REDUCTION	1	10	25	50
A	0%	40	60	68	76
B	33%	26.8	40.2	45.6	50.9
C	50%	20	30	34	38

REFERENCE: NOAA DATA RECORDED AT PORTLAND AIRPORT AT 30 FEET IN THE AIR. (SEE APPENDIX 'A' - CHART 2)

NOTES:
1) WIND SPEED AT GROUND LEVEL WILL BE DIMINISHED SO WIND SPEEDS ARE PROBABLY HIGH.
2) WIND IS ASSUMED TO BE FROM THE NORTH EAST QUADRANT.
3) REDUCTIONS ARE BASED ON THE SHELTERING EFFECTS OF DIMILLO'S MARINA, BRAKWATERS, FIVE WHARF AND FERRY RESTAURANT. LOAD SECTIONS 'B' + 'C' ARE BEHIND THE DIMILLO STRUCTURES IF THE WIND IS OUT OF THE NORTH-EAST QUADRANT AND AS SUCH THE WIND SPEED WILL BE REDUCED.

WIND PRESSURE:

LOAD SECTION "A":

RETURN PERIOD (YEARS)	WIND VELOCITY (MPH)	WIND PRESSURE (PSF)
1	40	7
10	60	15
25	68	19
50	76	23

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WIND LOAD (CONTINUED):

WIND PRESSURE:

LOAD SECTION 'B':

RETURN PERIOD (YEARS)	WIND VELOCITY (MPH)	WIND PRESSURE (PSF)
1	26.8	3.5
10	40.2	7.0
25	45.6	8.5
50	50.9	11.0

LOAD SECTION 'C':

RETURN PERIOD (YEARS)	WIND VELOCITY (MPH)	WIND PRESSURE (PSF)
1	20	2.0
10	30	4.0
25	34	5.5
50	38	6.5

REFERENCE: SMALL CRAFT HARBORS: DESIGN, CONSTRUCTION AND OPERATION, [illegible] AND [illegible], FOR U.S. ARMY CORPS OF ENGINEERING; FIG 20-B, p.15 (SEE APPENDIX "A" - CHART 5.)

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WIND LOAD (CONTINUED):

PROFILE HEIGHTS:

50' VESSEL — 7.0'
80' VESSEL — 8.8'

REFERENCES: LENGTH VS PROFILE HEIGHT (APPENDIX 'A' - CHART 4) BASED ON FIG 51 SMALL CRAFT HARBORS: DESIGN, CONSTRUCTION, AND OPERATION, DUNHAM AND FINN FOR US ARMY CORPS OF ENGINEERS (SEE APPENDIX 'A' CHART 6.

WIND LOADS: LOAD = PRESSURE × PROFILE HEIGHT

LOAD SECTION 'A':
MAXIMUM VESSEL SIZE - 50'
PROFILE HEIGHT - 7.0'

RETURN PERIOD (YEARS)	WIND VELOCITY (MPH)	WIND PRESSURE (PSF)	WIND LOAD (PLF)
1	40	7.0	49
10	60	15.0	105
25	68	19.0	133
50	76	23.0	161

LOAD SECTION 'B':
MAXIMUM VESSEL SIZE - 50'
PROFILE HEIGHT - 7.0'

RETURN PERIOD (YEARS)	WIND VELOCITY (MPH)	WIND PRESSURE (PSF)	WIND LOAD (PLF)
1	26.8	3.5	25
10	40.2	7.0	49
25	45.6	8.5	60
50	50.9	11.0	77

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WIND LOAD (CONTINUED):

LOAD SECTION "C":
VESSEL SIZES - 50' & 80'
(USE 80' PROFILE HEIGHT FOR CALCULATIONS BECAUSE IT IS THE WORST CASE.)
PROFILE HEIGHT - 8.8'

RETURN PERIOD (YEARS)	WIND VELOCITY (MPH)	WIND PRESSURE (PSF)	WIND LOAD (PLF)
1	20	2.0	18
10	30	4.0	55
25	34	5.5	48
50	38	6.5	57

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APP BY RMS

WAVE LOAD:

WAVE HEIGHT:

EQUATION:

$$H = R * \left[\frac{.283\,U^2}{g}\right] * \tanh\left[.530\left(\frac{gd}{U^2}\right)^{.75}\right] * \tanh\left\{\frac{.0125\left(\frac{gF}{U^2}\right)^{.42}}{\tanh\left[.530\left(\frac{gd}{U^2}\right)^{.75}\right]}\right\}$$

REFERENCE: LOW COST SHORE PROTECTION, U.S. ARMY CORPS OF ENGINEERS, 1981 p 49.

WHERE:

H = WAVE HEIGHT (FT)
U = WIND SPEED (FT/SEC)
F = FETCH LENGTH (FT)
d = AVERAGE WATER DEPTH OVER FETCH (FT)
g = 32.2 FT/SEC^2
R = HARBOR REDUCTION FACTOR

ASSUMPTIONS:

FETCH (F) = 21600 FT BASED ON NOAA CHART 13292 - PORTLAND HARBOR AND VICINITY

WATER DEPTH (d) = 30 FT. BASED ON NOAA CHART 13292 - 'PORTLAND HARBOR AND VICINITY'

HARBOR REDUCTION (R) = .75 BASED ON 'FSR WAVE SURVEY' (SEE APPENDIX C

RESULTS:

RETURN PERIOD (YEARS)	WIND SPEED (MPH)	PREDICTED WAVE HEIGHT (FT)
1	40	2.4
10	60	3.6
25	68	4.0
50	76	4.5

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WAVE LOAD (CONTINUED):

WAVE REDUCTIONS:

LOAD SECTION A: USE FULL PREDICTED WAVE HEIGHT

LOAD SECTION B: DUE TO PROTECTION FROM DIMILLO'S BREAKWATERS AND FLOATING PIERS MAXIMUM WAVE HEIGHT WHICH WILL BE CONSIDERED WILL BE .67 TIMES THE PREDICTED WAVE HEIGHT.

LOAD SECTION C: DUE TO THE PROTECTION FROM DIMILLO'S FERRY RESTAURANT AND FIXED WHARF, THE EFFECTIVE WIND FETCH IS PRACTICALLY ZERO SO THE MAXIMUM WAVE HEIGHT TO BE CONSIDERED WILL BE 1 FOOT.

WAVE LOADING:

EQUATION: FOR WAVE HEIGHT GREATER THAN 3 FEET
LOAD = 55 (WAVE HEIGHT − 1.62)
REFERENCE: "DETERMINATION OF MOORING LOAD AND TRANSMITTED WAVE HEIGHT FOR A FLOATING TIRE BREAKWATER", GILES AND ECKERT FOR US ARMY CORPS OF ENGINEERS.

FOR WAVE HEIGHT LESS THAN 3 FEET USE CHART - 7 APPENDIX "A" REFERENCE SAME AS ABOVE.

LOAD SECTION A:
NO WAVE HEIGHT REDUCTION

RETURN PERIOD (YEARS)	PREDICTED WAVE HEIGHT (FT)	PREDICTED WAVE LOAD (PLF)
1	2.4	53
10	3.6	109
25	4.0	131
50	4.5	155

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WAVE LOAD (CONTINUED):

WAVE LOADING:

LOAD SECTION B
PREDICTED WAVE HEIGHT = MAXIMUM WAVE HEIGHT × .6'

RETURN PERIOD (YEARS)	PREDICTED WAVE HEIGHT (FT)	PREDICTED WAVE LOAD (PLF)
1	1.6	20
10	2.4	53
25	2.7	66
50	3.0	77

LOAD SECTION C:
PREDICTED WAVE HEIGHT = 1 FOOT

RETURN PERIOD (YEARS)	PREDICTED WAVE HEIGHT (FT)	PREDICTED WAVE LOAD (PLF)
1	1	8
10	1	8
25	1	8
50	1	8

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LOADING SUMMARY:

LOAD SECTION "A":

RETURN PERIOD (YEARS)	WIND SPEED (MPH)	WIND LOAD (PLF)	WAVE HEIGHT (FT)	WAVE LOAD (PLF)	TOTAL LOAD (PLF)
1	40	49	2.4	53	102
10	60	105	3.4	109	214
25	66	133	4.0	131	264
50	76	161	4.5	158	319

LOAD SECTION "B":

RETURN PERIOD (YEARS)	WIND SPEED (MPH)	WIND LOAD (PLF)	WAVE HEIGHT (FT)	WAVE LOAD (PLF)	TOTAL LOAD (PLF)
1	26.8	25	1.6	20	45
10	40.2	49	2.4	53	102
25	45.3	60	2.7	66	126
50	50.9	77	3.0	77	154

LOAD SECTION "C":

RETURN PERIOD (YEARS)	WIND SPEED (MPH)	WIND LOAD (PLF)	WAVE HEIGHT (FT)	WAVE LOAD (PLF)	TOTAL LOAD (PLF)
1	20	18	1	5	26
10	30	35	1	8	43
25	34	48	1	8	56
50	35	57	1	8	65

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LOADING SUMMARY (CONTINUED):

NOTES:

1) NO BERTHING LOADS INCLUDED IN SUMMARY (SEE BERTHING LOAD CALCULATIONS)
2) NO CURRENT LOADS INCLUDED IN SUMMARY (SEE CURRENT LOAD CALCULATIONS)
3) REDUCTIONS IN WAVE HEIGHT AND WIND VELOCITIES TAKEN AS DESCRIBED IN RESPECTIVE CALCULATIONS
4) REFERENCES FOR SUMMARY TABLE INFORMATION GIVEN IN RESPECTIVE CALCULATION SECTIONS
5) "TOTAL LOAD" DOES NOT ACCOUNT FOR REDUCTION OF WIND LOADING DUE TO HEIGHT OF WAVE "SHELTERING" A PORTION OF THE PROFILE HEIGHT USED IN WIND LOAD CALCULATION.

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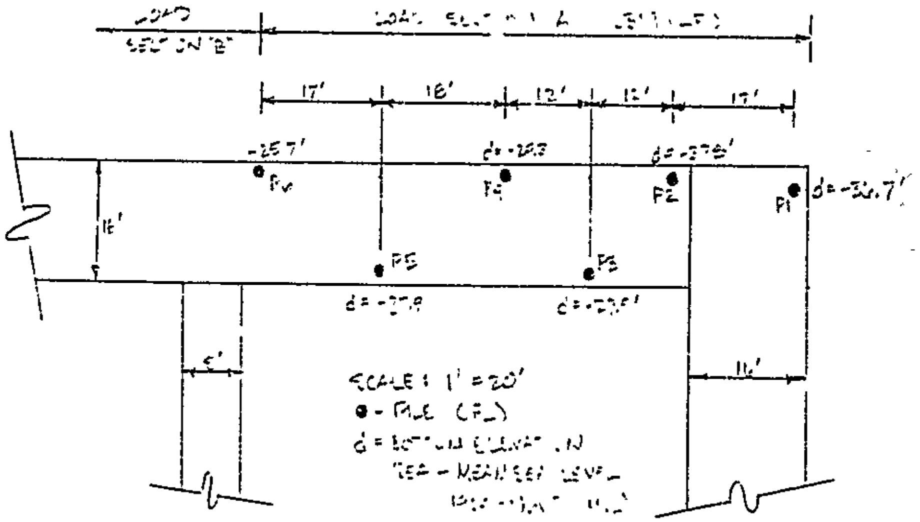
LOAD SECTION "A" ANALYSIS



ASSUMPTIONS:

DESIGN FOR 50 YEAR STORM

LOAD (W) = [illegible] = 319 PLF

*TAKES INTO ACCOUNT WIND AND WAVE EFFECTS ON SHIELDED VESSEL

[illegible] ELEVATION = +8.5 ' MSL

PILE FORCE (F) = C × WL

WHERE C IS THE APPLICABLE AISC COEFFICIENT FROM "MOMENTS SHEARS AND REACTIONS"

W = LOAD PER FOOT (351 PLF)

L = PILE SPACING (F)

F = FORCE ON PILE

POINT OF FIXITY OF PILE IS 7' BELOW THE BOTTOM ELEVATION

PILE WILL ACT LIKE A CANTILEVER BEAM

THE SOIL WILL BE ABLE TO TAKE THE LOAD (SEE ORIGINAL CALCULATIONS FOR CHANDLER'S WHARF MARINA, APPENDIX E)

ALL PILES ARE STEEL PIPE, 20" Ø, WALL THICKNESS ½", CONCRETE FILLED, SECTION MODULUS (S) = 145.7 in^3 (REFERENCE: TELEDYNE MATERIALS RESEARCH (SEE APPENDIX A - CHART B))

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LOAD SECTION "A" ANALYSIS (CONTINUED):

ASSUMPTIONS (CONTINUED):

ALLOWABLE BENDING STRESS (F_b) = .66 × YIELD STRENGTH (F_y)
F_y = 36,000 PSI FOR A-36 STEEL
HOWEVER FOR WIND ALLOWABLE STRESS CAN INCREASE BY 1/3
THEREFORE F_b = 31.7 KSI

CALCULATIONS:

PILE NUMBER	AISC LOAD COEFFICIENT (C)	EFFECTIVE LENGTH OF FLOAT (FT)	FORCE ON PILE (F) (LBS)	BOTTOM ELEVATION (MSL)	MOMENT ARM (7' FIXITY BELOW MSL & STORM TIDE)	MOMENT (M) (K-FT)
1	.50	6.75	3100	-36.7	52.5	163
2	1.143	15.00	6000	-37.6	53.6	322
3	1.143	12.25	4900	-27.9	43.7	214
4	1.143	15.00	6000	-29.3	45.1	271
5	1.143	18.00	7200	-27.9	43.7	315
6	.50	9.00	3200	-25.7	41.5	133

CHECK WORST CASE: PILE #2

$$f_b = M/S = \frac{322 \text{ K-FT} \times (12''/1')}{145.7 \text{ IN}^3} = 26.5 \text{ KSI}$$

ALLOWABLE F_b = 31.7 KSI ∴ STEEL PILES IN LOAD SECTION "A" ARE ADEQUATE IN CAPACITY TO WITHSTAND THE FORCES GENERATED BY A 50 YEAR STORM.

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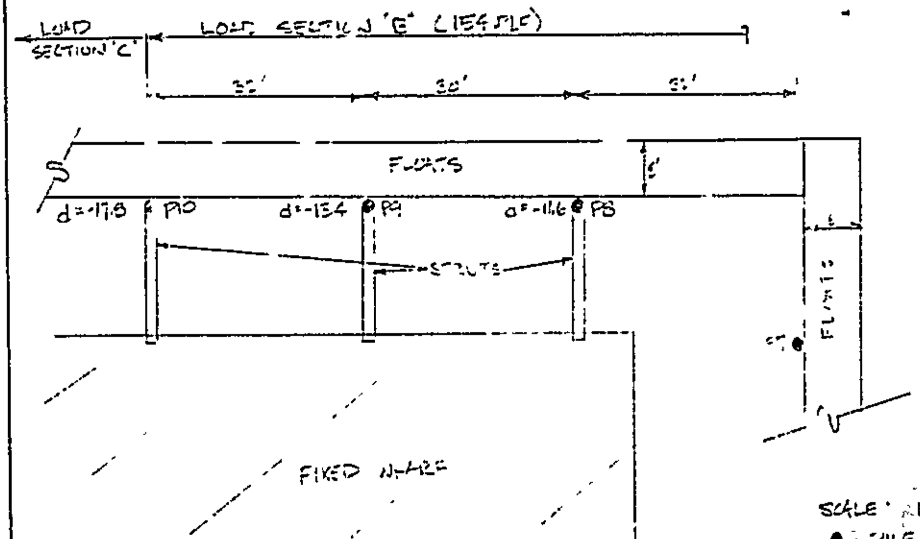
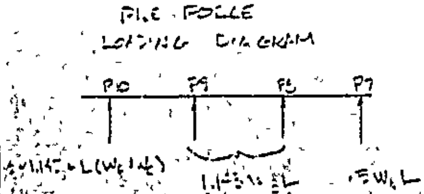
LOAD SECTION "E" ANALYSIS:

SINCE STEEL PILES WERE ADEQUATE FOR FORCES IN LOAD SECTION "A" THERE IS NO NEED TO CHECK THEIR STRENGTH IN LOAD SECTION "E" WHERE THE LOADS ARE SMALLER.

IN LOAD SECTION 'E' THE WOODEN PILES WILL BE FITTED WITH WOODEN STRUTS WHICH TIE THE PILES BACK TO THE FIXED WHARF.



SCALE: 1"=20'
● = PILE (PL)
d = BOTTOM ELEV. REF: MEAN SEA LEVEL



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LOAD SECTION 'B' ANALYSIS (CONTINUED):

ASSUMPTIONS:

DESIGN FOR 50 YEAR STORM
PILE FORCE AS SHOWN IN PILE FORCE LOADING DIAGRAM (PREVIOUS PAGE)
POINT OF FIXITY 5' BELOW BOTTOM ELEVATION
PILE WILL ACT AS A PROPPED CANTILEVER
THE SOIL WILL TAKE THE LOAD (SEE ORIGINAL CALCULATIONS FOR CHANDLER'S WHARF MARINA, APPENDIX E)
ALL PILES ARE OAK, 12" NOMINAL ϕ, SECTION MODULUS (S) OF 11" ϕ = 130.7 IN^3

ALLOWABLE BENDING STRESS (F_b) = 2450 PSI REFERENCE: "WOOD CONSTRUCTION - NATIONAL DESIGN SPECIFICATION" BY NATIONAL FOREST PRODUCTS ASSOCIATION, 1986 ED. p 28.
THE ALLOWABLE BENDING STRESS (F_b) CAN BE INCREASED BY 133% DUE TO THE FACT THAT THE LOAD IS FROM WIND. THEREFORE F_b = 3260 PSI
A WOODEN STRUT WILL BE FITTED TO PILES 8, 9, 10
PILE 7 IS SUFFICIENT BECAUSE LOADS WILL BE TRANSFERED TO STEEL PILES

PILE LOADS:

PILE NUMBER	FORCE ON PILE (F) (lbs)
7	3400
8	5500
9	5500
10	3800

LOAD SECTION 'E' ANALYSIS (CONTINUED):

EQUATIONS (CONTINUED):

$$R_1 = \frac{Pb^2}{2L^3}(a+2L)$$

$$M_2 = \frac{Pab}{2L^2}(a+L)$$

$$M_1 = R_1 a$$

$$b = L - a$$

WHERE: R_1 = HORIZONTAL REACTION AT STRUT
P = LOAD = 5500 lbs IN THIS CASE
L = LENGTH OF BEAM = 31.9' IN THIS CASE
a = DISTANCE BETWEEN STRUT AND LOAD
[a WILL RANGE FROM 3' TO 16.5']
M_1 = MOMENT AT LOAD POINT
M_2 = MOMENT AT POINT OF FIXITY

REFERENCE: AMERICAN INSTITUTE OF STEEL CONSTRUCTION, p 2-115

RESULTS:

a DISTANCE BETWEEN STRUT AND LOAD POINT (ft)	R_1 HORIZONTAL REACTION AT STRUT (lbs)	M_1 MOMENT AT LOAD POINT (ft-lbs)	M_2 MOMENT AT POINT OF FIXITY (ft-lbs)
3	4700	14,200	-5200
6	4000	23,800	-15,900
8	3500	27,800	-20,600
11	2800	30,800	-26,700
12	2500	30,600	-28,400
13	2300	30,300	-29,800
14	2100	29,700	-31,100
16	1700	27,500	-32,200
16.5	1600	26,800	-32,300

KIMBALL CHASE
company, inc.

CLIENT LIBERTY GROUP
PROJECT CHANDLER'S WHARF JOB NO 85-795
DETAIL
PAGE NO 21
CALCULATED BY KAF / JEM DATE 2/11/87
CHECKED BY DATE 2/12/87
APP BY GMS

LOAD SECTION 'B' ANALYSIS (CONTINUED):

RESULTS (CONTINUED):

ALLOWABLE PILE MOMENT = $S \times F_b$

$= (130.7\ \text{IN}^3) \times (3260\ \text{PSI}) \times [1/12\ \text{FT}/\text{IN}]$

$= 35,500$ FT-lbs

FROM LOADINGS, MAXIMUM MOMENT = 33,300 FT-lbs

THEREFORE WOODEN PILES WITH STRUTS IN LOAD SECTION 'B' ARE ADEQUATE TO WITHSTAND THE FORCES GENERATED BY A 50 YEAR STORM.

KIMBALL CHASE
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CLIENT [illegible] GROUP
PROJECT [illegible] WHARF — JOB NO 8[illegible]-79[illegible]
DETAIL
PAGE NO 22
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CHECKED BY — DATE
[illegible] FPS

WOOD PILE ALLOWABLE SOIL LOAD DETERMINATION

[illegible]

[illegible] = $\sqrt{EI/n}$

= $\sqrt{[illegible]}$

= 61.75 in

[illegible] 6[illegible]/T

SHEET 20 x 54 [illegible] = 4620 [illegible]

[illegible] ([illegible]/[illegible]) [illegible]

= 797.6 [illegible]

[illegible]

[illegible] = [illegible] x 500 x .92 = 4140 [illegible]

[illegible]

= 2070 [illegible]

FROM CONCLUSIONS FOR LATERALLY LOADED PILES
SHEET SL-2 DATED 1-6-86

MIN EMBED FOR [illegible] CURVE TO BE VALID = 2 x 2.4 x T
T = 61.75"

2 x 2.4 x 61.75/12 = 24.7'

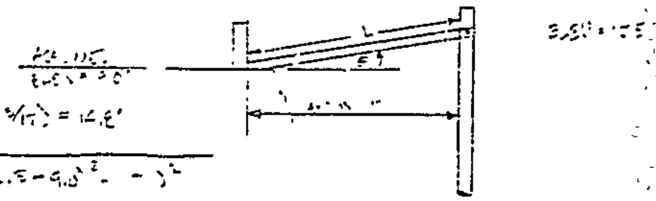
PILES HAVE 30 FT EMBEDMENT
∴ CURVE IS VALID

[illegible] x 2070 [illegible] >> ACTUAL LOAD (4[illegible]

KIMBALL CHASE
company, inc.

CLIENT ____ PROJECT ____ JOB NO 95-151
DETAIL ____ PAGE NO ____
CALCULATED BY ____ DATE ____
CHECKED BY ____ DATE ____

STRUT ANALYSIS:



$\theta = \tan^{-1}(5/17) = 16.8'$

$L = \sqrt{(17.5 - 9.0)^2 + (\ )^2}$
$= 17.6'$

FROM LOAD SECTION & ANALYSIS RESULTS, MAXIMUM HORIZONTAL REACTION FORCE ON STRUT WILL BE 4600 LBS

AXIAL LOAD = $\frac{4600 \text{ LBS}}{\cos 16.8°}$ = 4800 LBS

F_c = 675 PSI

E = 1400,000 PSI

$K = .671 \sqrt{E/F_c} = .671 \sqrt{\frac{1400000}{675}} = 45.5$

ASSUME d = 4"

L/d = ... = 52.8

SINCE $L/d > K$

$F_c' = \frac{.30 \times E}{(L/d)^2} = \frac{.30 \times 1400000 \text{ PSI}}{(52.8)^2} = 150 \text{ PSI}$

FOR 2 STRUTS EACH TAKES ½ OF LOAD, LOAD = $\frac{4800}{2}$ = 2400

$A = \text{LOAD}/F_c = \frac{2400 \text{ LBS}}{150 \text{ PSI}}$ = 16 IN² REQ'D

USE 2 4x4

REF: WOOD CONSTRUCTION - NATIONAL DESIGN SPECIFICATION, NATIONAL FOREST PRODUCTS ASSOCIATION

KiMBALL CHASE
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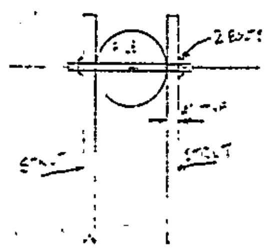
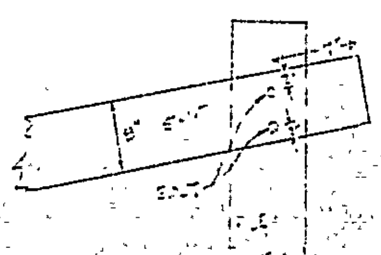
CLIENT: [illegible] GROUP
PROJECT: [illegible]
JOB NO: [illegible]
DETAIL:
PAGE NO. 24
CALCULATED BY: [illegible]
DATE: 3/11/87
CHECKED BY:
DATE: 3/12/87
[illegible] BY RISS

STRUT CONNECTION ANALYSIS:

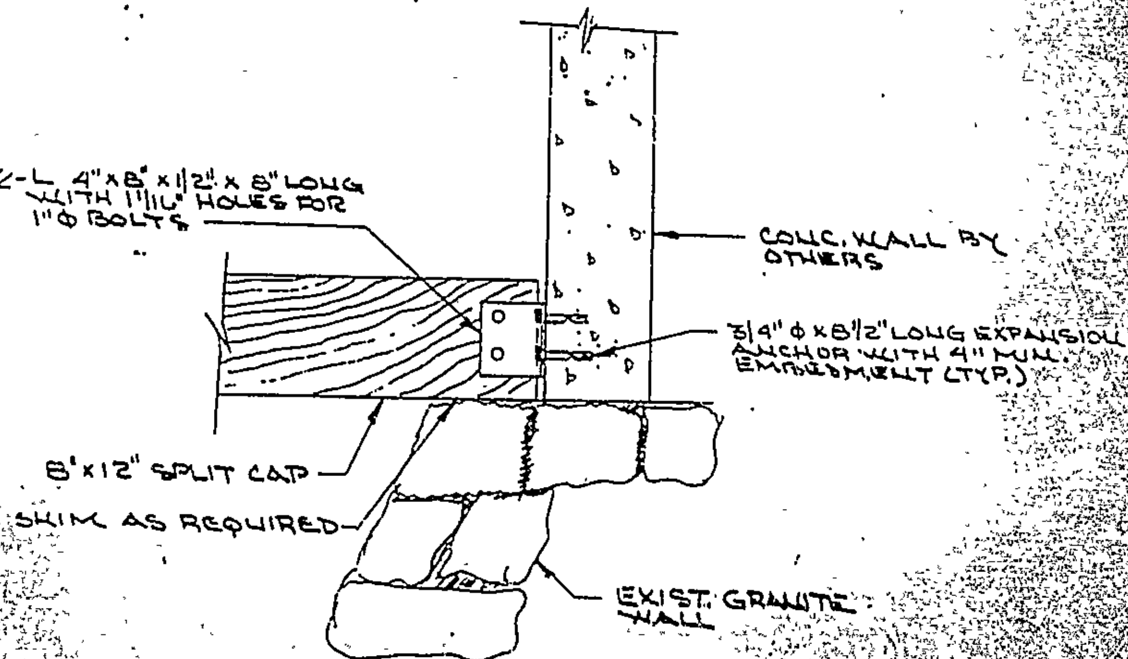
USE 2-1" Ø THROUGHBOLTS THROUGH PILE ℄
GIVES END DISTANCE OF 7D = 7"
AND A EDGE DISTANCE OF 2"

REFERENCE: "WOOD CONSTRUCTION - NATIONAL DESIGN SPECIFICATIONS", NATIONAL FOREST PRODUCTS ASSOCIATION.





DETAIL OF STRUT
CONNECTION TO CEMENT WALL



TYPICAL SPLIT-CAP END
CONNECTION
SCALE: 3/4" = 1'-0"

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CLIENT [illegible]
PROJECT [illegible] JOB NO [illegible]
DETAIL
PAGE NO 26
CALCULATED BY [illegible] DATE [illegible]
CHECKED BY DATE [illegible]

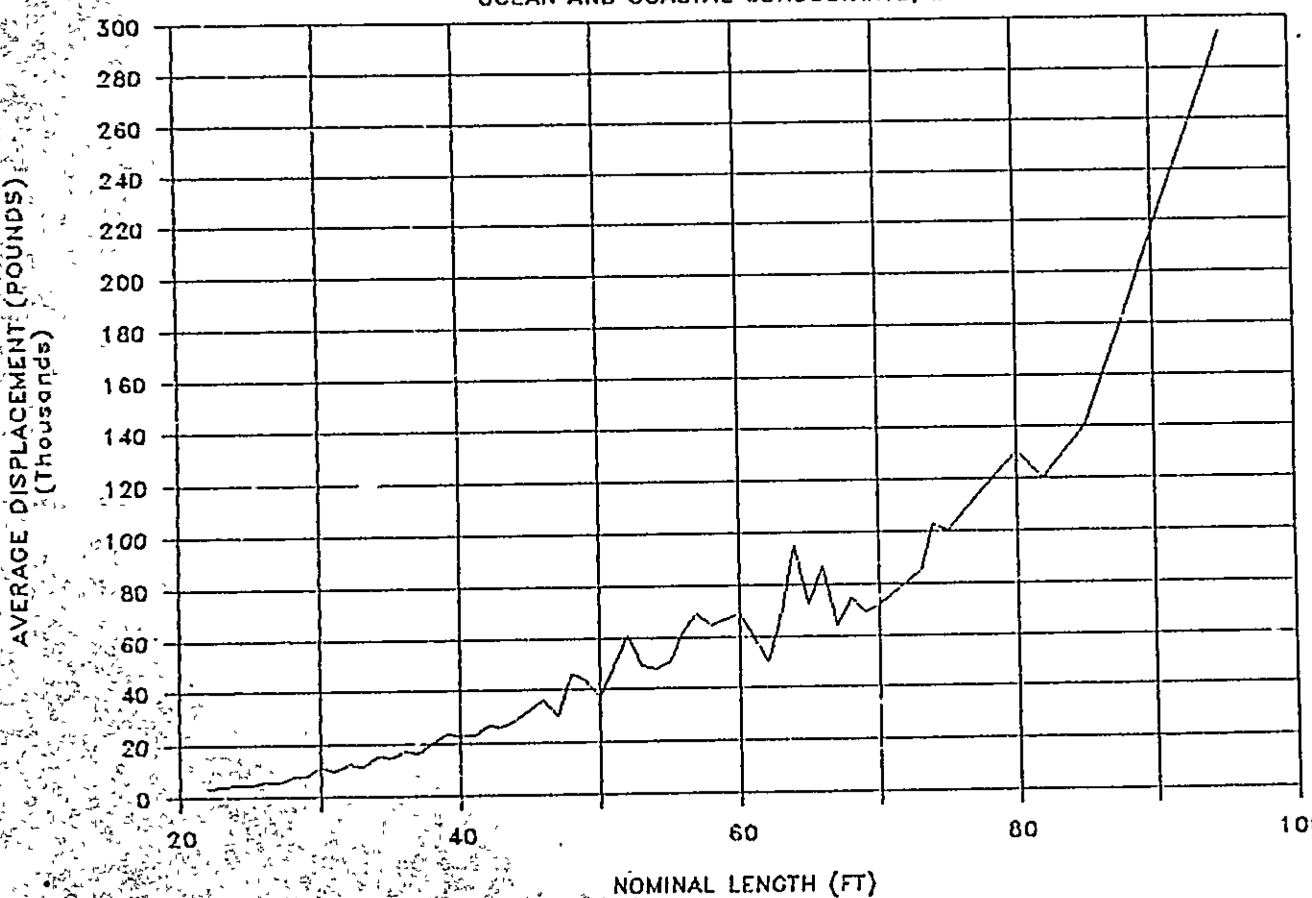
LOAD SECTION 'C' ANALYSIS

PILE FORCE = 1.1K? + 33' × 6E PLF = [illegible] lbs
WHICH WILL YIELD A MOMENT GREATER THAN THE PILE ALONE CAN SUPPORT, SO LOAD SECTION 'C' WILL ALSO HAVE TO BE FITTED WITH STRUTS. AS THE LOAD CRITERIA IN SECTION "C" IS LESS THAN THAT OF SECTION "B" NO FURTHER CALCULATIONS ARE REQUIRED

APPENDIX A

POWER BOAT DATA

OCEAN AND COASTAL CONSULTANTS, INC.



RETURN PERIOD (IN YEARS)	PREDICTED EXTREME WIND BASED ON OPTIMAL EXTREME VALUE TYPE 2. DISTRIBUTION (GAMMA = 7.00000)	PREDICTED EXTREME WIND BASED ON EXTREME VALUE TYPE 1 DISTRIBUTION	ESTIMATED STAN. DEV. SAMPL. ERROR CRAMER-RAO	ESTIMATED STAN. DEV. SAMPL. ERROR METH. OF MOM.
2.0	46.59	47.17	1.31	1.31
3.0	50.10	50.92	1.60	1.66
4.0	52.49	53.33	1.82	1.96
5.0	54.34	55.11	2.00	2.20
6.0	55.85	56.53	2.15	2.41
7.0	57.15	57.70	2.28	2.58
8.0	58.28	58.71	2.39	2.73
9.0	59.29	59.59	2.48	2.86
10.0	60.20	60.37	2.57	2.98
20.0	66.45	65.41	3.14	3.76
30.0	70.34	68.32	3.48	4.22
37.0	72.44	69.81	3.66	4.46
40.0	73.23	70.36	3.72	4.55
50.0	75.55	71.94	3.91	4.80
60.0	77.49	73.23	4.06	5.01
70.0	79.17	74.[illegible]2	4.19	5.19
80.0	80.66	75.26	4.31	5.34
90.0	81.99	76.09	4.41	5.48
100.0	83.20	76.84	4.49	5.60
200.0	91.63	81.71	5.08	6.39
300.0	96.96	84.56	5.43	6.66
400.0	100.93	86.58	5.67	7.19
500.0	104.12	88.14	5.86	7.44
600.0	106.80	89.42	6.02	7.65
700.0	109.12	90.50	6.15	7.83
800.0	111.18	91.44	6.26	7.99
900.0	113.02	92.27	6.36	8.12
1000.0	114.70	93.00	6.45	8.24
2000.0	126.38	97.86	7.04	9.04
3000.0	133.77	100.71	7.39	9.51
4000.0	139.27	102.72	7.64	9.85
5000.0	143.70	104.29	7.83	10.11
6000.0	147.43	105.56	7.99	10.32
7000.0	150.66	106.65	8.12	10.49
8000.0	153.51	107.58	8.23	10.65
9000.0	156.07	108.41	8.33	10.79
10000.0	158.40	109.15	8.43	10.91
50009.0	198.70	120.42	9.81	12.78
100000.0	219.11	125.28	10.41	13.58
500000.0	275.16	136.57	11.80	15.45
1000000.0	303.[illegible]4	141.43	12.40	16.26

CHART 2

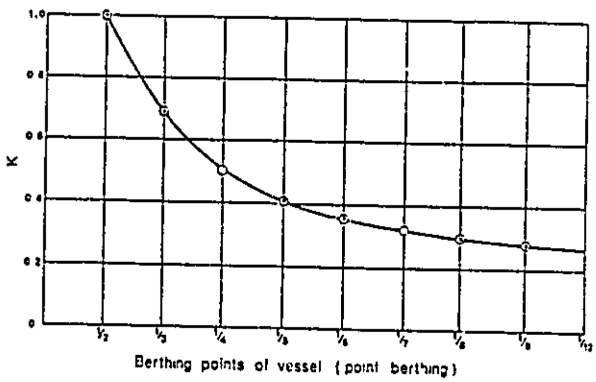


Figure 2-20. Eccentricity factor K vs. berthing point of the ship (From Seiby Polimer Chemical Co. Ltd.)

ergy, as the time it takes for the pier to react is much longer than the time over which the impact is applied. To calculate the impact force resisted by the pier, it is necessary to consult load/deflection curves of fender manufacturers. In recent years, used tires have been adopted for fendering, with varying degrees of success. Although the energy-absorption characteristics of used tires cannot be compared with commercial fenders, from consideration of cost they appear to absorb satisfactorily the energy of impact produced by small vessels. Load/deflection curves for used tires must be obtained experimentally, as the results differ depending on the particular type of tire and configuration used.

To arrive at these curves experimentally, first it is necessary to plot curve

$$F = f(d_f)$$

where F = force applied to the fender.
d_f = corresponding fender deflection (Figure 2-21).

The area between curve $F=f(d_f)$ and the axis d_f represents the work (energy) done by the fender. The curve $E=f(d_f)$, which is an integral curve in relationship to curve $F=f(d_f)$, could then be plotted.

ractices

...s ship approaches a solid-wall
...occurs, increasing with the de-

...e of added mass, some authori-
...ng factors [30]

...mensions.
...onger than the length of the ship.
...er than the length of the ship.

... in Reference 24.
...nsidered an open structure.
...of a ship moving in open water is
... increases considerably during
...he berthing process, because the
...volume of water involved in the
...he berth. This volume varies with

...uch as type of structure (open
...ng properties of fenders.
...nderwater shape and draft, and the
...facilities (e.g., sideways, parallel
... a certain angle).
...ater depth and possible impact of

...es of many slender structures in-
...mass of fluid displaced by their
...ction, some authorities determine
...eight of water of cylindrical shape
...ssel draft and a length equal to the

(2-50)

...d mass (w_2) ... fully loaded large
... became 0.5 to 0. ... times the ...

CHAP 4

In current design practice, however, other recommendations are found:[24,31,32] Some investigators suggested that the value of hydrodynamic (added) mass could be as much as 1.3 to 3.6 times the ship displacement tonnage.

Results of some laboratory experiments on the relationship between the added mass coefficient and the ship beam-to-water depth ratio can be found in Figure 2-18.

The International Commission for Improving the Design of Fender Systems, formed by the Permanent International Association of Navigation Congresses (PIANC), in its recent report[24] recommends, unless the designer has good reason to adopt other values, that the value of hydrodynamic mass (W_2) range between 1.5 W_1 (for very large under-keel clearance, say 0.5 × draught) and 1.8 W_1 (for very small under-keel clearance, say 0.1 × draught).

The author supports the recommendation that the added mass of the ship determined by Equation 2-50 be used for large vessels approaching the dock while being slowly pushed sideways toward and parallel to the berthing structure.

In the case of the ship's direct approach to the berthing structure, the value of added mass should be determined by good judgment, and by evaluating all berthing conditions previously discussed. A degree of uncertainty in the mechanism of ship/dock interaction does present prob-

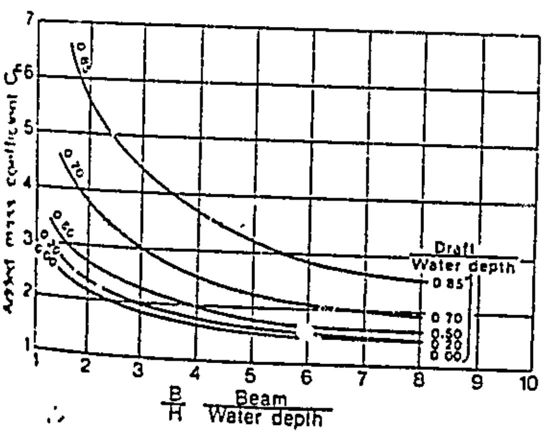


Figure 2-18. Added mass coefficient C_m.[33]

Figure 80-A. Isotachs showing fastest mile of wind (in miles per hour) 30 feet above ground, 50-year period of recurrence (from ASCE Paper 6038, Journal of Structural Division, July 1968).

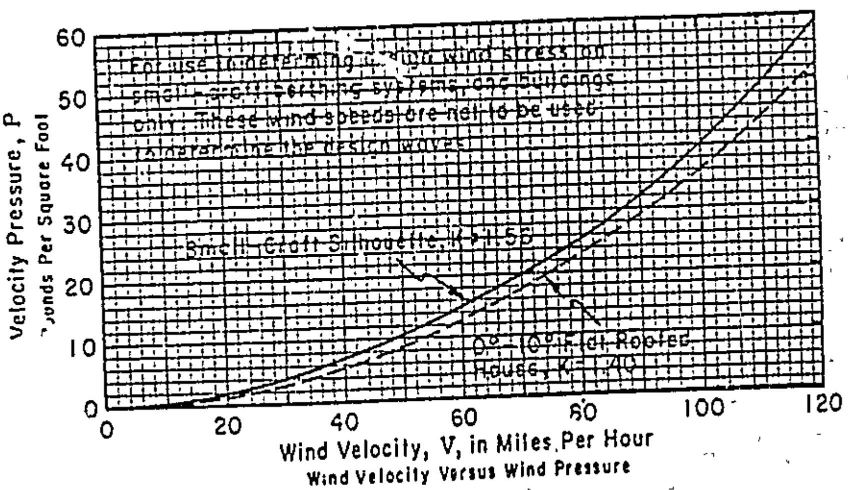
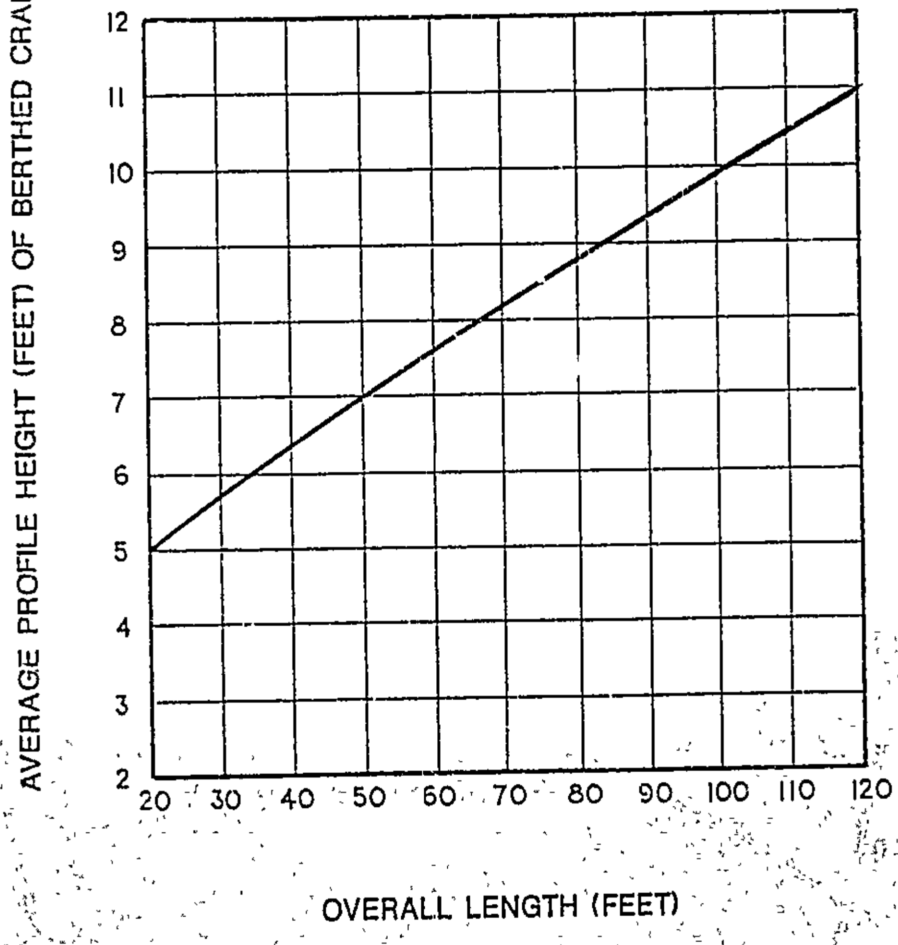


Figure 80-B. Windloading against a vertical face.

CHART 6



AVERAGE PROFILE HEIGHT (FEET) OF BERTHED CRAFT
12
11
10
9
8
7
6
5
4
3
2
20
30
40
50
60
70
80
90
100
110
120
OVERALL LENGTH (FEET)

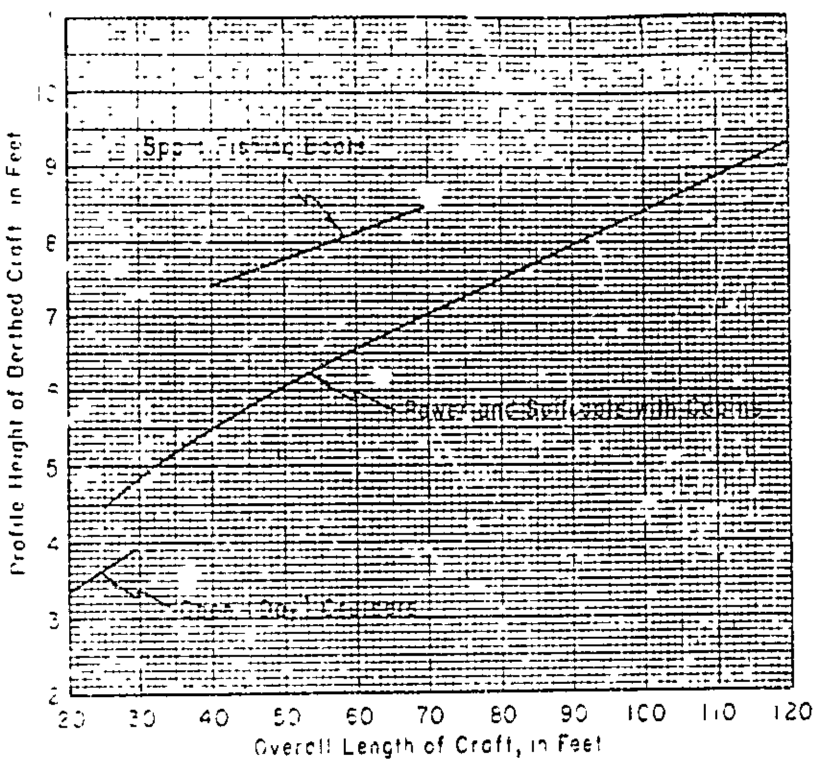


Figure 81 Average profile height versus length of craft

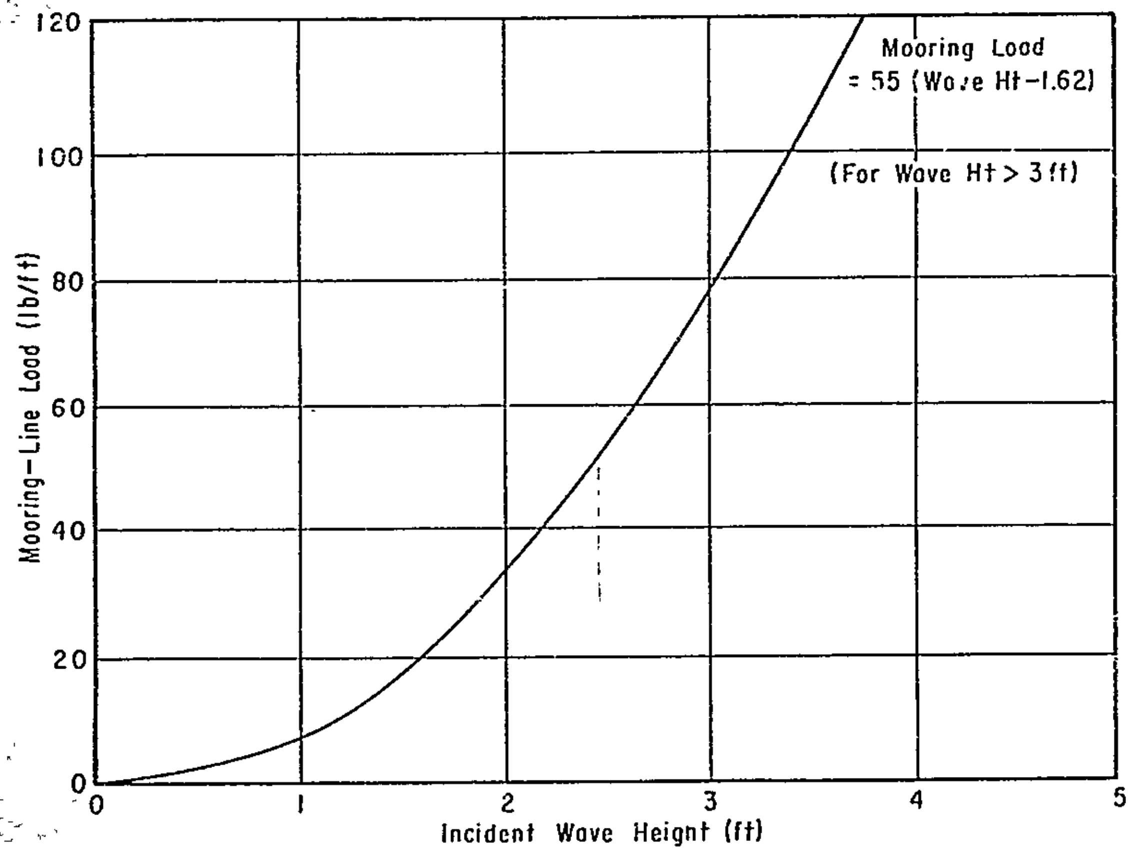


Figure 4. Design curve for predicting mooring loads per foot of breakwater length for a given incident wave height.

CHART 8

BY______ DATE______
CHKD. BY______ DATE______

SHEET NO.______ OF 108
FILE NO.______

PIPE PROPERTIES					
NOMINAL SIZE 20" DIA.		SCHEDULE NO. 30			
DIMENSIONS	OUTSIDE DIA. (O.D.)	20.000	IN.		FT.
	INSIDE DIA. (I.D.)	19.000	IN.		FT.
	WALL THICKNESS	0.500	IN.		FT.
	INSULATION THICKNESS		IN.		FT.
AREA	METAL CROSS-SECTION	30.63	$IN.^2$		$FT.^2$
	FLOW AREA	283.5	$IN.^2$		$FT.^2$
MECHANICAL PROPERTIES	MOMENT OF INERTIA	1457.0	$IN.^4$		$FT.^4$
	SECTION MODULUS	145.7	$IN.^3$		$FT.^3$
WEIGHTS	PIPE WEIGHT	8.750	LBS./IN.	105.0	LBS./FT.
	WATER WEIGHT	10.233	LBS./IN.	122.8	LBS./FT.
	INSULATION WEIGHT		LBS./IN.		LBS./FT.
	PIPE + WATER + INSULATION		LBS./IN.		LBS./FT.
	PIPE + WATER	18.983	LBS./IN.	227.8	LBS./FT.
	PIPE + INSULATION		LBS./IN.		LBS./FT.
MATERIAL	TYPE NO.	GRADE			

TELEDYNE MATERIALS RESEARCH
ENGINEERS WALTHAM, MASSACHUSETTS

APPENDIX B

KIMBALL CHASE
company, inc.

CLIENT [illegible]
PROJECT [illegible] JOB NO 85 [illegible]
DETAIL
PAGE NO 5E 1
CALCULATED BY KAH
DATE 1-6-86
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DATE

STEEL PILE 6.7 [illegible] 3C

SHEAR = 7 x (295 [illegible] = 2065 F USE 2100 F

M = 295(7)(49)(12) = 1,214,220 IN F

RELATIVE STIFFNESS OF PILE $T = \sqrt[5]{EI/F_h}$

$= \sqrt[5]{\dfrac{29000000 \text{ PSI} (362 \text{ IN}^4)}{1 \text{ F/IN}^2}}$

= 101 IN

$W = C_u Q_{hs}/T = 1(2100 \text{ F})/101 \text{ IN} (1\text{F}/12\text{IN})$

= 249.5 SAY 250 F/FT

ULTIMATE LATERAL BRG PRESSURE / UNIT LENGTH OF PILE @ GIVEN DEPTH

Qd = 9cB 9 x 500 F/FT x 1.06 =

Qd = 4781 F/FT

F.S. = 2 Qd/2 = 2390 F/FT

[illegible] 1112

KIMBALL CHASE
company, inc.

CLIENT Liberty Group
PROJECT [illegible]
JOB NO. 85 [illegible]
DETAIL
PAGE NO. [illegible]
CALCULATED BY [illegible]
DATE 1-6-86
CHECKED BY
DATE

CALCULATE AREA OF CURVE OF SOIL RESISTANCE

FIRST DEPTH COEFF = 2.4

T = 101 in = 8.4 ft

DEPTH = 2.4 (8.4) = 20.2

SOIL REACTION D 10 Cw Ohg / T [illegible]

SUM OF THE AREAS UNDER CURVES (ABOVE Z=2.4)

$= \frac{16.12}{20.2} \, Qhg = 0.8\,Qhg$

SUM OF THE AREAS UNDER CURVES (BELOW Z=2.4)

$1.16 / 20.2 \; Qhg = 0.06\,Qhg$

* CONCLUSION: FOR LATERALLY LOADED PILE

USING THE REACTION VS. DEPTH COEFFICIENT CURVE FROM PRAKASH (SEE FIGURE A)
WE CONCLUDE THAT MINIMUM EMBEDMENT FOR A PILE IN ORDER THAT THE CURVE IS VALID IS APPROX 2 x 2.4 x T
WHERE $T = \sqrt[5]{EI / n_h}$ ∴ BY FINDING THE AREA UNDER THE CURVE WE CAN DETERMINE THE SOIL REACTION
THE SUMMATION OF THE AREAS ~~[illegible]~~ WILL BE APPROX EQUAL TO Qhg

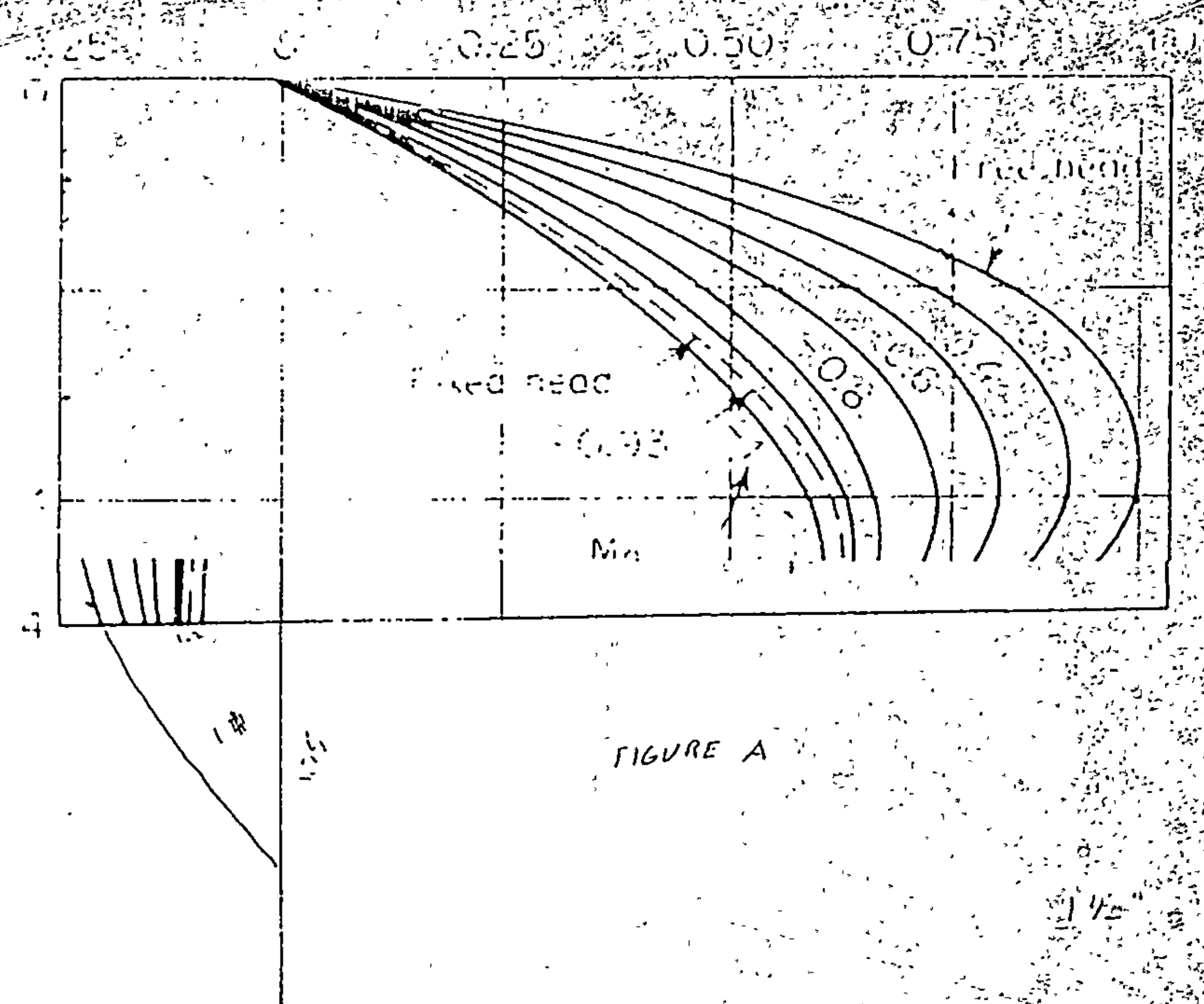

FIGURE A

KIMBALL CHASE
company, inc.

CLIENT LIBERTY GROUP
PROJECT CENTRAL WHARF JOB NO. 85-1031
DETAIL PAGE NO SE 3
CALCULATED BY KAH DATE
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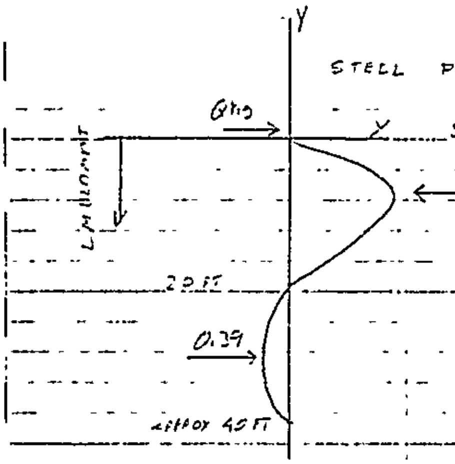
STEEL PILE EMBEDMENT CONT'D.



TOTAL LOAD ON SOIL Qhg = 2100 #

EQUILIBRIUM (1.47 - 0.39) Qhg = Qhg

ALLOWABLE LOAD ON SOIL FROM SE-1 = 2390#/FT

MAXIMUM SOIL REACTION (FROM FIGURE A) = Cu Qhg/T

Cu = 0.96 MAX T = 8.4

SOIL MAX = 241 #/FT FOR EMBEDMENT OF 40 FT.

WITH AN EMBEDMENT OF 40 FT THE MAXIMUM SOIL BRG PRESSURE IS ONLY APPROX 10% OF THAT ALLOWED. BY SHORTENING THE EMBEDMENT LENGTH THE ZERO POINT ON THE CURVE ABOVE, (WE ASSUME) WILL MOVE UP THE Y AXIS AND THE MAXIMUM SOIL BRG PRESSURE WILL INCREASE BY AN APPROXIMATELY EQUAL RATE. IE A REDUCTION IN EMBEDMENT LENGTH WILL NOT CAUSE AN INCREASE IN MAXIMUM SOIL BRG PRESSURE OF 10 FOLD.

γ = effective unit weight of soil, pcf
z = depth, ft
ϕ = angle of shearing resistance [illegible]

The working load should not exceed $Q_u/2$ beyond a depth of $4B$ under any circumstances. [illegible] $Q_u/2$ the soil reaction at any depth is given by [illegible]

$C_p Q_u$ 9432.7
9432.7

where [illegible]
C = soil reaction coefficient from Fig. 47
[illegible]
[illegible] $\sqrt{EI/n_h}$ in
[illegible]
EI = [illegible] stiffness [illegible] lb·in²
[illegible] of the total reaction [illegible]
Th[illegible] directly proportional [illegible]

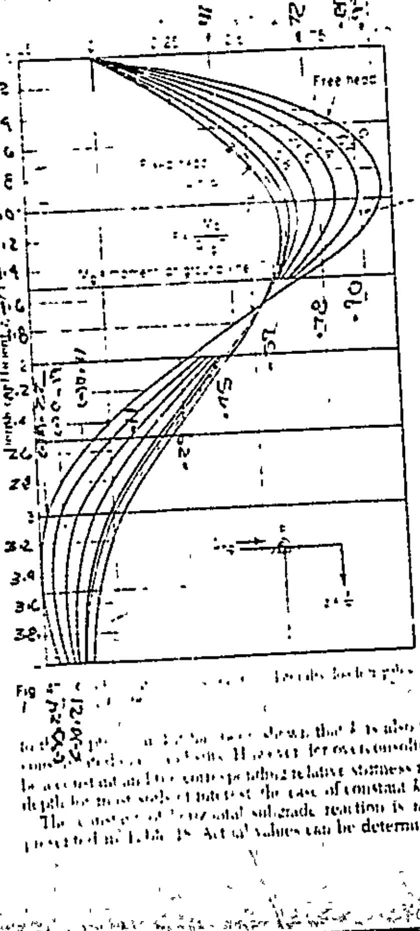


Fig. 47 [illegible]

[illegible] also proportional to depth for [illegible] overconsolidated clays k is usually assumed [illegible] relative stiffness is $\sqrt{EI/k}$. Since k is proportional [illegible] constant and [illegible] the case of constant k is not considered here. [illegible] depth for most soils of interest [illegible] subgrade reaction is $k_h = kz$. Typical values of [illegible]

The [illegible] subgrade reaction is [illegible] Actual values can be determined experimentally by driving two [illegible] presented in Table 18. [illegible]

γ = effective unit weight of soil, pcf
z = depth, ft
ϕ = angle of shearing resistance

*The working load should not exceed $Q_u/2$ beyond a depth of $4B$ under any circumstances.

For lateral loads smaller than $Q_u/2$ the soil reaction at any depth is given by

$$p = \frac{C_p Q_g}{T} \quad \text{lb}$$

where p = soil reaction, lb/ft
C_p = soil reaction coefficient from Fig. 47
Q_g = shear at ground surface, lb
T = relative stiffness of pile = $\sqrt[5]{EI/n_h}$, in
EI = flexural stiffness of pile, lb-in²
n_h = constant of horizontal subgrade reaction, lb/in.³

The modulus of subgrade reaction $k = p/y$ is the ratio of the total reaction per unit length of pile to the corresponding deflection. For granular soils k is directly proportional

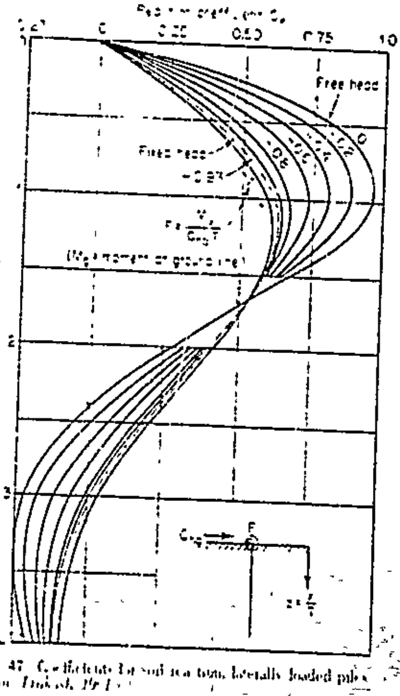


Fig. 47 Coefficients for soil reaction, laterally loaded piles.

to the depth z, and it has been shown that k is also proportional to depth for normally consolidated clays and silts. However, for overconsolidated clays k is usually assumed to be a constant and the corresponding relative stiffness is $\sqrt[4]{EI/k}$. Since k is proportional to depth for most soils of interest, the case of constant k is not considered here.

The constant of horizontal subgrade reaction is $n_h = k/z$. Typical values of n_h are presented in Table 18. Actual values can be determined experimentally by driving two

where Q_d = ultimate load on group, tons
B = width of pile group, out to out, ft
L = length of pile group, out to out, ft
l = pile length, ft
q_u = average unconfined compressive strength of clay within length l, tons/sq ft
q_u = average unconfined compressive strength within a distance B below the pile tips, tons/sq ft

The maximum probable loading on the group should not exceed $Q_d/3$. Although the base bearing capacity may contribute considerably to the capacity of the pile group, it should be noted that the greatest benefit of a friction pile foundation is obtained with the longest piles possible within the limits of economy. The longer the pile the smaller the settlement in most instances.

Piles driven into clays that increase materially in strength with depth may be analyzed as for friction piles. However, the point resistance, which may represent a sizable proportion of the pile capacity, can be determined only by means of loading tests. The safe load on a group may be taken as the safe load per pile as determined from load tests times the number of piles in the group. In some localities pile-driving formulas have been adjusted to indicate safe loads corresponding to the pile-driving resistance. Such formulas should never be used outside the geological region in which they were developed.

Piles driven through relatively soft materials to a stiff or hard clay act in point bearing. The load capacity of a group of such piles is equal to the product of the number of piles in the group and the safe load per pile without regard to their spacing. However, these conditions are ideal for the development of negative skin friction (Art. 73), which may be a sizable proportion of the pile load capacity. The magnitude of the negative skin friction can be determined from pile load tests (Art. 35). It may also be estimated as the average shearing resistance of the soft material multiplied by the surface area of the embedded piles.

76. Settlement of Pile Foundations Any pile foundation which has a compressible stratum located below the pile tips is likely to settle, and the magnitude of the settlement should be predicted. It is computed in the same manner as for footings on clay except that the change in pressure Δp is determined somewhat differently depending upon whether the piles act in point bearing or as friction piles.

For a point-bearing pile foundation, the load on the pile group is assumed to be applied to the subsoil at the level of the pile tips on an area equal to the plan area of the pile group. Below the tips, it is considered to be spread uniformly at an angle of 30° from the vertical.

The settlement of a group of friction piles is computed in a similar manner. However, the level of the application of the load to the subsoil is less certain, as load is transferred through much of the length of the piles. A commonly used approximate procedure is based on the assumption that the load is applied at the lower third point of the piles. The load is assumed to spread at an angle of 30° from the vertical, and any compressible material below the lower third point is assumed to contribute to the settlement of the group.

77. Laterally Loaded Piles Where a pile-supported structure is subjected to lateral loads, the vertical piles may provide more lateral resistance than is commonly realized. Prevailing rules of thumb commonly permit an arbitrary lateral load per pile—often 1000 lb—without any consideration as to the type of pile or the soil in which it is driven. Since a pile-supported structure does not transmit load directly to the soil beneath the pile cap, frictional resistance should not be assumed between the base of the structure and the underlying soil. Therefore, the piles must be adequate to resist all lateral loads.

The ultimate lateral bearing pressure per unit length of pile at a given depth in clay is

$$Q_d = 9cB = 4.5q_uB \tag{44}$$

and in sands

$$Q_d = 3B\gamma' z \frac{1 + \sin\phi}{1 - \sin\phi} \tag{45}$$

where Q_d = ultimate load per unit length of pile, lb/ft
c = cohesion, psf
q_u = unconfined compressive strength, psf $= 2c$
B = width of pile, ft

APPENDIX C

Department of Human Services
Division of Health Engineering
(207) 289-3826

PLUMBING APPLICATION

PROPERTY ADDRESS

Town Or Plantation	Portland, Maine
Street Subdivision Lot #	206 Chandler's Wharf

PROPERTY OWNERS NAME

Last: Denney First George

Applicant Name:	Scribner & Iverson, Inc.
Mailing Address of Owner/Applicant (If Different)	54 Warren Ave., P.O.Box 879 Portland, Maine 04104

PORTLAND 5367 TOWN COPY

Date Permit Issued: 4/10/95 $ 4 FEE ☐ Double Fee Charged

Local Plumbing Inspector Signature

L.P.I. # 0124

Owner/Applicant Statement

I certify that the information submitted is correct to the best of my knowledge and understand that any falsification is reason for the Local Plumbing Inspector to deny a Permit.

[signature] 2/23/95
Signature of Owner/Applicant Date

Caution: Inspection Required

I have inspected the installation authorized above and found it to be in compliance with the Maine Plumbing Rules.

W/O insp JM 10-95
Local Plumbing Inspector Signature Date Approved

PERMIT INFORMATION

This Application Is for
1. ☐ NEW PLUMBING
2. ☐ RELOCATED PLUMBING

Type Of Structure To Be Served:
1. ☐ SINGLE FAMILY DWELLING
2. ☐ MODULAR OR MOBILE HOME
3. ☐ MULTIPLE FAMILY DWELLING
4. ☐ OTHER - SPECIFY: ____

Plumbing To Be Installed By:
1. ☐ MASTER PLUMBER
2. ☐ OIL BURNERMAN
3. ☐ MFG'D HOUSING DEALER/MECHANIC
4. ☐ PUBLIC UTILITY EMPLOYEE
5. ☐ PROPERTY OWNER

LICENSE # 015512

Hook-Up & Piping Relocation Maximum of 1 Hook-Up	Number	Column 2 Type of Fixture	Number	Column 1 Type Of Fixture
☐ HOOK-UP: to public sewer in those cases where the connection is not regulated and inspected by the local Sanitary District.		Hosebibb / Sillcock		Bathtub (and Shower)
		Floor Drain		Shower (Separate)
OR		Urinal		Sink
☐ HOOK-UP: to an existing subsurface wastewater disposal system.		Drinking Fountain		Wash Basin
		Indirect Waste		Water Closet (Toilet)
		Water Treatment Softener, Filter, etc		Clothes Washer
☐ PIPING RELOCATION: of sanitary lines, drains, and piping without new fixtures.		Grease/Oil Separator		Dish Washer
		Dental Cuspidor		Garbage Disposal
		Bidet		Laundry Tub
Number of Hook-Ups & Relocations		Other: ____	01	Water Heater
$. Hook-Up & Relocation Fee		Fixtures (Subtotal) Column 2		Fixtures (Subtotal) Column 1
				Fixtures (Subtotal) Column 2
			01	Total Fixtures
			$.	Fixture Fee
			$.	Hook-Up & Relocation Fee
			$ 4.00	Permit Fee (Total)

SEE PERMIT FEE SCHEDULE FOR CALCULATING FEE

12.00

PLUMBING APPLICATION

2

Department of Human Services
Division of Health Engineering
(207) 289-3826

PROPERTY ADDRESS

Town Or Plantation	Portland
Street Subdivision Lot #	314 Chandlers Wharf

PROPERTY OWNERS NAME

Last: Cole First: Jerry & Susan

Applicant Name.	Scribner & Iverson, Inc.
Mailing Address of Owner/Applicant (If Different)	54 Warren Ave., P.O. Box 8379 Portland, Maine 04104

PORTLAND 5348 TOWN COPY

Date Permit Issued. 3/16/95 $ 4 FEE ☐ Double Fee Charged

Local Plumbing Inspector Signature LPI # 0124

Owner/Applicant Statement

I certify that the information submitted is correct to the best of my knowledge and understand that any falsification is reason for the Local Plumbing Inspector to deny a Permit.

Signature of Owner/Applicant 3/10/95 Date

Caution: Inspection Required

I have inspected the installation authorized above and found it to be in compliance with the Maine Plumbing Rules.

W/O mqa - TM Local Plumbing Inspector Signature 10-95 Date Approved

PERMIT INFORMATION

This Application Is for

1. ☐ NEW PLUMBING
2. ☐ RELOCATED PLUMBING

Type Of Structure To Be Served:

1 ☐ SINGLE FAMILY DWELLING
2. ☐ MODULAR OR MOBILE HOME
3 ☐ MULTIPLE FAMILY DWELLING
4 ☐ OTHER - SPECIFY: ____________

Plumbing To Be Installed By:

1 ☐ MASTER PLUMBER
2. ☐ OIL BURNERMAN
3 ☐ MFG'D HOUSING DEALER/MECHANIC
4 ☐ PUBLIC UTILITY EMPLOYEE
5 ☐ PROPERTY OWNER

LICENSE # 05512

	Hook-Up & Piping Relocation Maximum of 1 Hook Up	Number	Column 2 Type of Fixture	Number	Column 1 Type Of Fixture
	HOOK-UP: to public sewer in those cases where the connection is not regulated and inspected by the local Sanitary District.		Hosebibb / Sillcock		Bathtub (and Shower)
			Floor Drain		Shower (Separate)
	OR		Urinal		Sink
	HOOK-UP: to an existing subsurface wastewater disposal system.		Drinking Fountain		Wash Basin
			Indirect Waste		Water Closet (Toilet)
			Water Treatment Softener, Filter, etc		Clothes Washer
	PIPING RELOCATION: of sanitary lines, drains, and piping without new fixtures		Grease/Oil Separator		Dish Washer
			Dental Cuspidor		Garbage Disposal
			Bidet		Laundry Tub
	Number of Hook-Ups & Relocations		Other: ____________	01	Water Heater
$	Hook-Up & Relocation Fee		Fixtures (Subtotal) Column 2		Fixtures (Subtotal) Column 1
					Fixtures (Subtotal) Column 2
				01	Total Fixtures
				$	Fixture Fee
				$	Hook-Up & Relocation Fee
				$ 4.00	Permit Fee (Total)

SEE PERMIT FEE SCHEDULE FOR CALCULATING FEE

Department of Human Services
Division of Health Engineering
(207) 289-3826

2

PLUMBING APPLICATION

PROPERTY ADDRESS

Town Or Plantation: Portland

Street Subdivision Lot #: 702 Chandler Wharf

PROPERTY OWNERS NAME

Last: Sweeney First: Bill

Applicant Name: Andy MacMillan

Mailing Address of Owner/Applicant (If Different): 59 Marlborough Rd Portland ME

PORTLAND 5018 TOWN COPY

Date Permit Issued: 12 / 21 / 94 $ 20 FEE ☐ Double Fee Charged

Local Plumbing Inspector Signature L.P.I. # 0124

Chief Plumbing Inspector

Owner/Applicant Statement

I certify that the information submitted is correct to the best of my knowledge and understand that any falsification is reason for the Local Plumbing Inspector to deny a Permit

Andy MacMillan — Signature of Owner/Applicant — 3 2 94 Date

Caution: Inspection Required

I have inspected the installation authorized above and found it to be in compliance with the Maine Plumbing Rules.

W/O Vago FM — Local Plumbing Inspector Signature — 10-95 Date Approved

PERMIT INFORMATION

This Application Is for

1. ☐ NEW PLUMBING
2. ☒ RELOCATED PLUMBING

Type Of Structure To Be Served:

1. ☒ SINGLE FAMILY DWELLING
2. ☐ MODULAR OR MOBILE HOME
3. ☐ MULTIPLE FAMILY DWELLING
4. ☐ OTHER — SPECIFY ________

Plumbing To Be Installed By:

1. ☒ MASTER PLUMBER
2. ☐ OIL BURNERMAN
3. ☐ MFG'D. HOUSING DEALER / MECHANIC
4. ☐ PUBLIC UTILITY EMPLOYEE
5. ☐ PROPERTY OWNER

LICENSE # 07133

Hook-Up & Piping Relocation — Maximum of 1 Hook-Up

☐ HOOK-UP: to public sewer in those cases where the connection is not regulated and inspected by the local Sanitary District

OR

☐ HOOK-UP: to an existing subsurface wastewater disposal system

☐ PIPING RELOCATION of sanitary lines, drains, and piping without new fixtures.

___ Number of Hook Ups & Relocations

$ ___ Hook-Up & Relocation Fee

OR

☐ TRANSFER FEE [$6.00]

Number	Column 2 Type of Fixture	Number	Column 1 Type of Fixture
	Hosebibb / Sillcock		Bathtub (and Shower)
	Floor Drain		Shower (Separate)
	Urinal		Sink
	Drinking Fountain		Wash Basin
	Indirect Waste		Water Closet (Toilet)
	Water Treatment Softener, Filter, etc		Clothes Washer
	Grease / Oil Separator		Dish Washer
	Dental Cuspidor		Garbage Disposal
	Bidet		Laundry Tub
	Other ________		Water Heater
	Fixtures (Subtotal) Column 2		Fixtures (Subtotal) Column 1
			Fixtures (Subtotal) Column 2
			Total
		$ 4	Fixture Fee
		$	Transfer Fee
		$	Hook-Up & Relocation Fee
		$ 20	Permit Fee (Total)

SEE PERMIT FEE SCHEDULE FOR CALCULATING FEE