Global Journal of Researches in Engineering: E
Civil And Structural Engineering
Volume 15 Issue 1 Version 1.0 Year 2015
Type: Double Blind Peer Reviewed International Research Journal
Publisher: Global Journals Inc. (USA)
Online ISSN: 2249-4596 & Print ISSN: 0975-5861

Design of Semi – Flexible and Flexible Dolphins with Concrete Pile Caps

By Vitaly B. Feygin, P.E.

Introduction- In recent years, Port and marine industry design standards were shifting their focus towards performance based and elasto-plastic limit state design criteria. Whilst performance based criteria for wharves and piers were well explained and covered by POLA/POLB[1], performance based design of the dolphins was never reviewed. Current PIANC WG-33[2] only briefly discussed design of the flexible dolphins. Some of the WG-33 statements related to fender supporting structures are ambiguous, and not well understood. In author' opinion, PIANC provisions do not differentiate between rigid, semi-flexible and flexible dolphin systems making conflicting statements. The following study covers several aspects associated with design of semi-flexible and flexible dolphin systems, and addresses design issues which were insufficiently covered by PIANC and national marine codes. The list of covered issues includes:

- fender selection conflicts
- concept of impact dynamic amplification
- utilization of the ductility concept for performance based design criteria
- the concept of capacity protected elements and proper application of overload factors
- detailing mistakes in pile to pile cap connections.

This paper reviews design of the flexible dolphin systems with concrete pile caps, explaining common design misconceptions and filling the gaps in the current design practice.

GJRE-E Classification : FOR Code: 861001

DESIGNOFSEMIFLEXIBLEANDFLEXIBLEDOLPHINSWITHCONCRETEPILECAPS

Strictly as per the compliance and regulations of :

Design of Semi – Flexible and Flexible Dolphins with Concrete Pile Caps

Vitaly B. Feygin, P.E.

I. Introduction

In recent years, Port and marine industry design standards were shifting their focus towards performance based and elasto-plastic limit state design criteria. Whilst performance based criteria for wharves and piers were well explained and covered by POLA/POLB[1], performance based design of the dolphins was never reviewed. Current PIANC WG-33[2] only briefly discussed design of the flexible dolphins. Some of the WG-33 statements related to fender supporting structures are ambiguous, and not well understood.

In author' opinion, PIANC provisions do not differentiate between rigid, semi-flexible and flexible dolphin systems making conflicting statements.

The following study covers several aspects associated with design of semi-flexible and flexible dolphin systems, and addresses design issues which were insufficiently covered by PIANC and national marine codes. The list of covered issues includes:

- fender selection conflicts
- concept of impact dynamic amplification
- utilization of the ductility concept for performance based design criteria
- the concept of capacity protected elements and proper application of overload factors
- detailing mistakes in pile to pile cap connections.

This paper reviews design of the flexible dolphin systems with concrete pile caps, explaining common design misconceptions and filling the gaps in the current design practice.

II. Force of Abnormal Impact and Fender Selection

Clause 4.2.8.4(d) of PIANC WG-33 states: "It is considered advisable to check the supporting structure against failure for loads substantially greater, (of 2-3 times greater), than the reactions due to abnormal impact…" but does not explain the cause of magnification factor, and why force magnification is advisable?

This paper investigates two plausible sources of impact force magnification advisable by PIANC provision:

- Dynamic component of abnormal impact during the ship berthing operation, A_D
- Ductility factor, μ_D - requirement rooted in the performance based analysis.

The study was structured as a step by step approach:

Effect of the ductility factor component of the magnification coefficient, k_{cm}, was analyzed in section III, and an investigation of the dynamic component of abnormal impact was reviewed in Section IIa, following review of the fender selection (*Example 1*).

Example 1. Fender Selection for Rigid Dolphin

Fender selection and analysis of forces acting on a rigid dolphin during abnormal berthing impact are explained below. All denominations used in this analysis correspond to denominations of PIANC WG-33.

(D)isplacement = 1.49E5mT

Cm = 1.48E5

Cs = 1.00

Ce = 0.73

Cc = 1.00

*m = D*Cm*Cs*Ce = 1.61E8kg* – composite mass

V_0 *= 0.15m/sec* – Vessel speed at initial impact

$E_O = m*V^2/2 = 1{,}815kN\text{-}m$ – Kinetic Energy of impact

C_{AB} *= 1.25* – Factor for abnormal impact applied to berthing energy (PIANC WG-33, Table 4.2.5)

$E'_{AB} = E_O * C_{AB} = 2{,}268kN\text{-}m$ – Energy of Abnormal Impact

Manufacturing Factors, TCF for Energy	TCF
Vessel approaching angle relative to a berth, α=10 deg	1.0
Manufacturing tolerance	0.9
Velocity	1.0
Temperature 42C	0.917
Composite TCF	0.825
Required Abnormal Energy, $E_{AB} = E'_{AB} / TCF$	2,748kN-m
Required Normal Energy, $E_N = E_O / TCF$	2,199kN-m

Author: e-mail: vbfeygin@gmail.com

Select Fender Panel *SCN 2000E0.9* (Trelleborg Fender Catalogue)[3]

Manufacturer data:

E_R = 2700kN-m - rated energy of the selected fender panel

R_R = 2610kN - rated reaction of the selected fender

H = 2.0 m - height of the selected fender panel

Manufacturing Factors, TCF for Energy	TCF
Vessel approaching angle relative to a berth, α=10 deg	1.0
Manufacturing tolerance	1.1
Velocity	1.0
Temperature 42C	1.08
Composite TCF	1.188

Abnormal Berthing Reaction, $R_{AB} = R_R * TCF = 2610*1.188 = 3,101kN$ – per PIANC WG-33

- ❖ Factored Rated Fender Reaction per Australian Standard AS 4997-2005[4] (L.F. = 1.5),

SCN 2000E0.9

$$R_{AB} = R_R * TCF*1.5 = 2610*1.188*1.5 = 4,651kN$$

- ❖ Fender selection based on requirements of US UFC 4-152-01[4]:

E_o=1,815kN-m ===> $E_N = E_o / TCF = 1815 / 0.825 = 2,200kN\text{-}m$

Fender corresponding to that level of energy absorption: *SCN 1800 E1.0* (E=2185kN-m provided vs. 2200kN-m required, R_R = 2350kN)

Thus, Factored Rated Fender Reaction (L.F. = 1.6),

$$R_u = 2350*1.6 = 3,760kN$$

- ❖ Factored Fender Reaction proposed by author, *SCN 2000E0.9*

$R_{AB\ modif.} = R_{AB} * (L.F. /C_{AB}) = 3,101*(1.5 / 1.25) = 3,720kN$

The later reaction is compliant with provisions of PIANC W-33 and provisions of both national codes, AS 4997-2005 and UFC 4-152-01[5], given here as examples.

**Note:

Different National Marine Codes may have slightly varying load factors:

UFC 4-152-01, Table 3.6, for example, demands Load Factor =1.6 for Berthing loads, whilst

Australian Standard AS 4997-2005 requires berthing Load Factor = 1.5. Effect of such variations is negligent.

The more troublesome fact is that some National codes make conflicting statements.

Australian national code AS 4997-2005 in cl. 5.3.2.5, for example, states that:

"The ultimate strength design of the fender support structures should then consider the greater load of:

a) The rated fender reaction load, with appropriate Limit State load factors applied; and/ or
b) The abnormal berthing case reaction (maximum fender reaction), considered as an Ultimate Limit State load condition."

Statement (a) effectively overrides statement (b) since statement (a) is based on characteristics of the fender selected for abnormal berthing energy absorption, Ultimate Limit State condition, by definition. Application of the additional load factor on top of rated fender reaction leads to a condition where a second load factor is applied on top of already factored load. Review of several latest projects in Eastern and Western Australia revealed that designers incorrectly interpreted cl. 5.3.2.5 of AS 4997-2005.

However, If designer assumes that statement (a) is based on the factored reaction of the fender selected for a nominal or normal impact, statement (a) of AS 4997 becomes reminiscent of position taken by UFC 4-152-01.

US Unified Facility Criteria, UFC 4-152-01 "Design of Piers and Wharves" (Table 3.6) sets L.F. = 1.6 for berthing reaction, ignoring PIANC C_{AB} energy magnification factor for abnormal impact.

Two different methods suggested by two major national Codes yield two vastly different results based on two different fenders:

- *Method 1* based on C_{AB} factor for Abnormal Impact,

R_{AB} =3101kN *(SCN 2000 E0.9)*

And

- *Method 2* based on factored reaction of the fender selected for normal energy absorption,

R_U = 2,350*1.6=3,760kN *(SCN 1800 E1.0)*

Comparison of both methods indicates that:

- ❖ Fender selection based on method 1 will satisfy energy absorption criteria of PIANC WG-33, but will not comply with load factors set for Berthing load by designated national standard.

- Fender selection based on method 2 may not satisfy energy absorption criteria of PIANC WG-33

Comment 1:

Compliance with both methods is achieved by applying a correction multiplier K_R= L.F. / C_{AB} to Abnormal Berthing Reaction R_{AB}.

Correction factor K_R preserves fender size based on Abnormal Impact Energy requirements of PIANC WG-33 and complies with Limit State Load Factors set by national marine standards.

In the studied project, the Limit State Load factor was based on a wrong interpretation of AS 4997, cl. 5.3.2.5. As a result, selected ultimate limit state reaction corresponding to abnormal impact applied to the breasting dolphin was overestimated by 25%.

Another design issue frequently yielding conflicting results is related to a proper selection of the C_{AB} -factor (Factor for Abnormal Impact Applied to a Berthing Energy).

Frequently, the owner dictates the largest C_{AB}-factor from WG-33, Table 4.2.5 (CAB=2.0).

It shall be understood that selection of a stiffer fender penalizes the dolphin structure for no reason and defeats the purpose of the rubber fender, in a first place.

Such definitions as “largest” or “smallest” vessel (WG-33, Table 4.2.5) are frequently misinterpreted. Erroneously, the difference in C_{AB}-factors for largest and smallest vessels may be as high as 40%.

However, C_{AB} is a composite energy factor directly proportional to the vessel composite mass (m), and square power of the approaching berthing speed, V^2.

Thus, C_{AB} for the smallest vessel can be determined from the following formula:

$$C'_{AB\ smallest} = C_{AB\ largest\ vessel} * (m_{\ smallest\ vessel} / m_{\ largest\ vessel}) * (V_{\ smallest\ vessel} / V_{\ largest\ vessel})^{2} \qquad \textit{(Formula 1)}$$

but shall be restricted by the following boundaries:

$C'_{AB\ smallest} < C_{AB\ smallest}$ as given by PIANC WG-33, Table 4.2.5

(The above formulas shall be used for similar types of vessels only and should not be applied when the same dolphin is utilized for the berthing of dissimilar vessels like tankers and general cargo carriers, etc. Such an arrangements shall be avoided, anyway).

Since the approaching speed (vector of the approaching speed is normal to the berthing key line) of largest and smallest vessels may be identical, C_{AB} variation will depend on a mass ratio of both vessels.

PIANC WG-33, Cl. 4.2.8.5 clearly states that Table 4.2.5 shall be used as a general guidance only, and the “designers' judgment should be paramount in determining the appropriate factor”.

All of the above relates to the fender selection for rigid dolphin structures.

Fender selection process for semi-flexible and flexible dolphins is slightly different.

The following discussion requires a clear explanation of the differences between Semi-Flexible and Flexible Dolphin systems.

In accordance with PIANC, flexible dolphin consists of vertical or near vertical piles cantilevered from the waterbed, and such dolphin system absorbs berthing energy via horizontal deflection of the pile heads under the berthing impact.

The group of dolphins described above includes both semi-flexible and flexible dolphin systems.

Comment 2, below, explains the difference between two subgroups.

Comment 2:

Semi-flexible dolphin consist of a group of vertical or near vertical piles cantilevered from the waterbed and designed to absorb the energy of impact by horizontal deflection within the elastic boundaries where dolphin pile sections do not undergo elasto-plastic deformations.

Flexible dolphins having similar construction features are designed as ductile structures with elasto-plastic deformations within the pile sections. Piles in such dolphins undergo partial plastification and allow residual inelastic deformation of the dolphin.

The following example explains conceptual design of the Flexible Dolphin System.

Example 1A. Fender Selection for *Semi-Flexible and Flexible* Dolphins.

Step 1. Start with the assumption that between 15% and 20% of abnormal impact energy is absorbed by elastic or elasto-plastic deformations of the dolphin structure itself. Validity of that assumption will require verification.

Step 2. Ignore manufacturing composite factors for energy absorption, and select fender based on

$E'_{AB} = E_O * C_{AB} = 2{,}268 kN\text{-}m$; hence, fender size can be dropped from SCN2000E0.9 to SCN1800E1.2

Step 3. Select Fender Panel *SCN1800E 1.2* (Trelleborg Fender Catalogue).

Manufacturer data:

E_R = 2303kN-m - rated energy of the selected fender panel
R_R = 2476kN - rated reaction of the selected fender
H = 1.8m - height of the selected fender panel

Manufacturing Factors, TCF for Energy	TCF
Vessel approaching angle relative to a berth, α=10 deg	1.0
Manufacturing tolerance	1.1
Velocity	1.0
Temperature 42C	1.08
Composite TCF	1.188

Step 2. Determine Factored Fender Reaction.

$$R_{AB\ modif.} = R_{AB} *TCF* (L.F. /C_{AB}) =2,476*1.188* (1.5 / 1.25) = 3,530kN$$

Compare fender reactions of the Rigid and Flexible dolphin systems:

R_{AB} *rigid* = 3,720kN

R_{AB} *flexible* = 3,530kN

a) Abnormal Impact Dynamic Magnification

❖ *Example 2. Impact Dynamic Magnification*

Table 1 shows analysis of the impact impulse length (τ) for a rigid dolphin system in a tabular format.

Table 1 : Analysis of the impulse length (Rubber Fender only)

% of rat'd reaction		deflection		(E)nergy per defl. step	Total (E)nergy	Rem'g kin'c (E)nergy	Rem'g speed V_{rem}	fender compr. time rate, dt	Time to % of defl.	Impulse length, τ	spring kf
%	kN	%	m	kN-m	kN-m	kN-m	m/s	sec	sec	sec	kN/m
1	2	3	4	5	6	7	8	9	10	11	12
0	0	0%	0.00	0	0	2,303	0.169		0.00		
0.2	495	5%	0.09	22	22	2,281	0.168	0.13	0.13		5502
0.4	990	10%	0.18	67	89	2,214	0.166	0.13	0.27		5502
0.6	1486	15%	0.27	116	205	2,098	0.161	0.14	0.41		5430
0.75	1857	20%	0.36	144	349	1,954	0.156	0.14	0.55		5158
0.8	1981	22%	0.39	55	405	1,898	0.153	0.05	0.59		5095
0.9	2228	26%	0.47	167	571	1,732	0.147	0.13	0.73		4762
0.95	2352	29%	0.52	124	695	1,608	0.141	0.09	0.82		4506
1	2476	36%	0.64	295	991	1,312	0.128	0.23	1.05	1.05	3842
0.95	2352	42%	0.76	282	1273	1,030	0.113	0.24	1.29		3089
0.9	2228	47%	0.84	186	1458	845	0.102	0.19	1.48		2645
0.85	2105	49%	0.88	86	1544	759	0.097	0.10	1.58		2386
0.8	1981	53%	0.95	129	1673	630	0.088	0.17	1.75		2096
0.75	1857	56%	1.02	135	1808	495	0.078	0.21	1.96		1829
0.74	1832	61%	1.09	275	1948	355	0.066	0.47	2.21		1682
0.8	1981	67%	1.20	206	2153	150	0.043	0.49	2.71		1655
0.9	2228	69%	1.25	102	2256	47	0.024	0.36	3.07		1789
1	2476	72%	1.30	47	2303	0	0.000	1.04	4.11	1.90	1910
1.19	2946	75%	1.35	146	2449	(146)	0.000	0.00	4.11		2183

Line 2 of *Table 1* is given as an example of the behind the scene calculations:

$E_{i\ abs} = Ri*\Delta_{\Delta} = 0.5* (0+495) * (0.09-0) = 22kN\text{-}m$

$\Sigma E_{abs} = 0+ 22 = 22kN\text{-}m$

$E_{kinetic\ remaining} = 2303- 22 = 2281kN\text{-}m$

$V_{rem'g} = SQRT [2*2281*10^3 / (1.61*10^8)] = 0.168m/sec$

$t_{compr\ time\ rate} = 0.5*(0.09-0) / (0.168+0.169) = 0.133sec$

$t_{to\ \%\ of\ \Delta} = 0+0.133 = 0.133sec$

The length of impact impulse between 60% and 70% of fender deflection was calculated as τ = (4.11-2.21) =1.90 sec.

Where,

Length of impulse, τ was based on assumption of indefinitely rigid supporting structure.

The reaction provided by the fender at the beginning of the impulse (point at 65% of fender deflection) is equal to about 75% of the rated reaction, R_R, of the selected fender.

From Table 1, maximum rated reaction, R_R =2,476 kN occurs at 0.63m fender deflection or 1.8m–0.63m=1.17m standoff.

A similar analysis was done for a flexible dolphin system. Results of that analysis are summarized in *Table 2*.

The flexible dolphin of the studied case was constructed of four 1500 mm O.D. pipe piles with a 25 mm thick wall. Corrosion allowance for pipe piles 3 m below the mud line and above was taken as 6mm.

Effective pile O.D. or D = 1488 mm

Wall thickness, t = 19 mm

D/t = 68

I_{eff} = 9.4625*1010 mm4 full elastic moment of inertia of 4 piles

H = L_c = 32.0m effective height of the pile (between the points of max. flexure)

m = 595,650kg mass of the pile cap + mass of 1 / 4 of pile effective height

$k_d = 12EI_{eff} / (L_c)^3$ = 6,930kN/m dolphin spring value

k_f – variable spring values of the rubber fender are summarized in *Table 1*

k_{comp} – composite (dolphin + fender) serial spring is summarized in column 18 of *Table 2*

Table 2 : Analysis of the impulse length (Rubber Fender + Flexible Dolphin)

% of rat'd reaction		fend. defl	Δd	Δf	tot. defl	Dolphin Energy	Fender (E)nergy per step	Fender (E)nergy total	Fender (E)nergy TCF adust.	Tot System (E)nergy	Rem kin'c (E)nergy	Rem'g speed V_{rem}	Syst compr. time rate, dt	Time to En'gy abs.	Impulse length, τ	spring kd	Comp. k tot
%	kN	%	m	m	m	kN-m	kN-m	kN-m	kN-m	kN-m	kN-m	m/s	sec	sec	sec	kN/m	kN/m
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
0	0	0%	0.00	0.00	0.00	0	0	0	0	0	2,303	0.169	0.00	0.00		6931	0
0.2	495	5%	0.07	0.09	0.16	18	45	45	37	54	2,249	0.167	0.24	0.24		6931	3067
0.4	990	10%	0.14	0.18	0.32	71	89	134	110	181	2,122	0.162	0.25	0.49		6931	3067
0.6	1486	15%	0.21	0.27	0.49	159	139	273	225	384	1,919	0.154	0.26	0.75		6931	3045
0.75	1857	20%	0.27	0.36	0.63	249	160	433	358	606	1,697	0.145	0.23	0.98		6931	2957
0.8	1981	22%	0.29	0.39	0.67	283	57	490	405	688	1,615	0.142	0.08	1.06		6931	2936
0.9	2228	26%	0.32	0.47	0.79	358	176	667	550	909	1,394	0.131	0.21	1.27		6931	2822
0.95	2352	29%	0.34	0.52	0.86	399	127	794	655	1054	1,249	0.124	0.14	1.41		6931	2731
1	2476	36%	0.36	0.64	1.00	442	303	1097	905	1347	956	0.109	0.30	1.71	1.71	6931	2472
0.95	2352	42%	0.34	0.76	1.10	399	275	1372	1132	1531	772	0.098	0.24	1.95		6931	2137
0.9	2228	47%	0.32	0.84	1.16	358	181	1553	1281	1640	663	0.091	0.17	2.12		6931	1915
0.85	2105	49%	0.30	0.88	1.19	320	83	1636	1350	1670	633	0.089	0.06	2.18		6931	1775
0.8	1981	53%	0.29	0.95	1.23	283	125	1761	1453	1736	567	0.084	0.13	2.31		6931	1609
0.75	1857	56%	0.27	1.02	1.28	249	130	1891	1561	1809	494	0.078	0.16	2.47		6931	1447
0.74	1832	61%	0.26	1.09	1.35	242	264	2025	1671	1913	390	0.070	0.40	2.71		6931	1354
0.8	1981	67%	0.29	1.20	1.48	283	214	2238	1847	2130	173	0.046	0.56	3.27	0.56	6931	1336
0.9	2228	69%	0.32	1.25	1.57	358	108	2347	1937	2295	8	0.010	0.750	4.020		6931	1422
1	2476	72%	0.36	1.30	1.65	442	125	2472	2040	2482	(179)	0.000	NA	NA		6931	1498
1.19	2946	75%	0.43	1.35	1.78	626	159	2631	2171	2797	(494)	0.000	NA	NA		6931	1660

Where,

$\Delta di = Hi*(Lc)^3 / (12EI_{eff})$ - deflection calculated at every instance of the impact force.

k_{comp} represents an average composite spring constant within two elastic regions of the fender spring between 0% to 35% and 65% to 70% of the fender deflection.

τ/T represents the ratio of impulse length to First Natural Period of the structure, T

$$T = 2\pi * (m / k_{comp})^{1/2} = 2*3.14* (595{,}650/2{,}901*103)^{1/2} = 2.85sec \quad \text{(Formula 2a)}$$

$$T = 2\pi * (m / k_{comp})^{1/2} = 2*3.14* (595{,}650/1{,}325*103)^{1/2} = 4.21sec \quad \text{(Formula 2b)}$$

Table 2 shows energy absorption capacity of the system at every force increment.

At every instance, the force acting on the rubber fender is reacted by the dolphin, and both deflections shown in *Table 2* and *Figure 1* contribute to energy absorption of the semi-flexible or flexible dolphin system.

Figure 1 shows Buckling Fender Reaction-Deflection curves of the flexible dolphin structure, where deflection of the fender is shown with sign (+) and deflection of the dolphin is shown with sign (–) for the purpose of convenience only. The algebraic sign has no physical meaning in the presented graph. Energy absorbed by the rubber fender and dolphin structure can be estimated by integrating area under the curves.

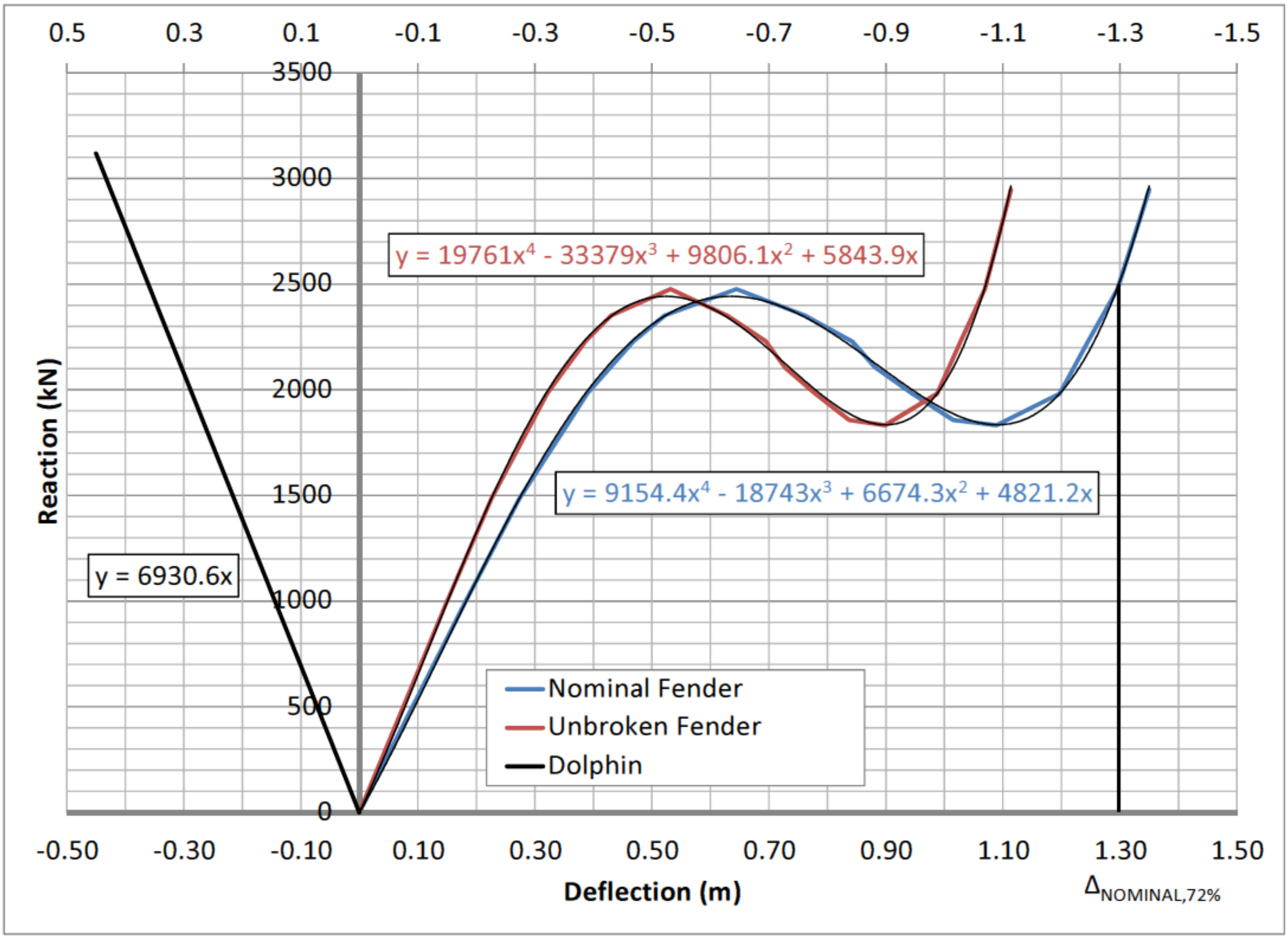


Figure 1 : Pile and Fender Reaction-Deflection Curve

Reaction-deflection curve of the fender can be closely fit by a polynomial curve generated by Excel. Using Excel's trend line option, designer can derive the formula for the curve and calculate fender energy absorption by integrating area under the curve within the deflection range. Investigation of the possible Dynamic Amplification based on the fender data presented in the Table 1 is presented below:

Dynamic Magnification,

$$A_D = (1+T/\pi\tau*\sin(\pi\tau/T)) \qquad \text{(Formula 3)}$$

is normally applied to the initial impact force.

P-y curve of the rubber fender (*Figure 1*) indicates that during the ship contact with the fender, system experiences impact impulse twice:

- Primary impact during initial contact, and
- Secondary impact at 67% of the fender standoff (after fender buckling, at about 40% of the initial fender standoff.)

However, results of the fender compression analysis consolidated in *Table 2* indicate that the system absorbs all Kinetic Energy of impact at about 67% of the original fender height or 0.67*1.80 = 1.20m standoff.

Summary of Impact Force magnification is shown in Figures 3A and 3B.

Table 3A : Impact Force Magnification
(Rubber Fender)- Rigid Dolphin

T	$T/\pi\tau$	$\pi\tau/T$	Dynamic Magnification	Amplified Reaction
sec			$A_D = 1+(T/\pi\tau)*\sin(\pi\tau/T)$	kN
1.84	0.56	1.79	1.55 $*R_R$	2,476
			but < R_R	
	0.31	3.23	0.73 $*R_R$	2,476
			$= 0.75*A_D*R_R$	

Table 3B : Impact Force Magnification
(Flexible Dolphin)

T	$T/\pi\tau$	$\pi\tau/T$	Dynamic Magnification	Amplified Reaction
sec			$A_D = 1+(T/\pi\tau)*\sin(\pi\tau/T)$	kN
2.85	0.53	1.89	1.50 $*R_R$	2,476
			but < R_R	
4.18	2.38	0.42	1.48 $*R_R$	2,476
			$= 0.75*A_D*R_R$	

When length of impulse (τ) approaches the length of the First Natural Period of the structure (T), dynamic amplification, A_D approaches 1.0, or purely static response.

Thus, when

$\tau/T ==> 0$	$A_D ==> 2.0$ – classical case of dynamic amplification.
$\tau/T ==> 1.0$	$A_D ==> 1.0$ – purely static response

The nature of the fender buckling negates any possibility of dynamic impact magnification unless proposed fender was improperly selected, or was very stiff. That statement is true for rigid dolphins.

Flexible dolphins have another line of defense against dynamic impact amplification: dolphin deflection itself.

Comment 3:

A summary of impact magnification analysis shown in *Table 3* indicates that flexible dolphin protected by a rubber fender does not experience dynamic impact amplification. The graph presented in *Figure 1* indicates that dynamic amplification becomes a strong possibility only in rigid dolphin systems when the fender was underrated and deflected beyond the point of specified maximum deflection, or was overrated and had not buckled.

In the Rigid Dolphin case, energy is absorbed entirely by the rubber fender deflection, requiring a larger-sized fender; whereas in the Flexible Dolphin case, about 20% of the kinetic energy is absorbed by the flexible dolphin structure itself. That allows selection of the smaller and softer fender.

An additional energy absorption mechanism based on plastic deformation of the flexible dolphin is further discussed in section IIIa.

Benefits of the Flexible Dolphin system become clear after comparison of torsional effects of the tangential force for both rigid and flexible systems (*Table 4*).

Table 4 : Reaction vs. standoff

System	Fender Reaction (kN)	Fender Stand off at max reaction, (m)	Distance from the fender panel at stand off to C.G. of the pile cap. (m)	Torsional Moment acting on the dolphin pile group, M_T (kN-m)
Rigid Dolphin	2,610	1.30	=4.5+1.3 5.80	15,138μ
Flexible Dolphin	**2,476**	**1.17**	**=4.5+1.17 5.67**	**14,038 μ**

Where,

μ =0.20 – fender panel friction coefficient

III. Structural Design of the Flexible Dolphin

Analysis of dolphin plastic deformations (performance based design criteria) requires design philosophy utilizing and defining special members known as "Capacity Protected Elements." The term "Capacity Protected Elements" was first introduced by CALTRAN[3], but design boundaries of such elements were never fully explained.

Comment 4:

An element shall be treated as Capacity Protected when elastic failure of the element changes the boundary condition of support or critical connection.

That concept was vaguely discussed by PIANC WG-33, clause 6.6.4:

"The following load factors for the limit state design method are advised…

depending on the pile capacity to resist overloads by plastic yielding.

- No yielding possible, $\gamma = 1.25$
- Yielding possible until a displacement of at least two times the maximum elastic displacement, $\gamma = 1.00$ "

Rewriting PIANC statement:

"The following load factors for the limit state design method are advised…

depending on the pile capacity to resist overloads by plastic yielding:

- Pile to pile cap connection detail yields prior to yielding of dolphin piles, $\gamma = 1.25$
- For Piles undergoing elasto-plastic deformations which are less than twice the elastic deflection based on gross moment of inertia of the affected piles, overload factor γ shall be interpolated utilizing Figure 7 (γ in this case is ranging from 1.0 to 1.25 at extreme)."

Possibility of overload of an essentially elastic Capacity Protected Element (CPE) is strong when pile material does not reach the yield point within the two times the max elastic deflection. Forces acting on the pile at the level of the pile cap soffit are than determined from the following equations:

$$M_o^{\ pile} = \gamma * M_p^{\ pile} \qquad \text{(Formula 4)}$$

$$V_o^{\ pile} = 2 * M_o^{\ pile} / L_c \qquad \text{(Formula 5)}$$

Where,

M_p – pile plastic moment capacity, at the location of the first plastic hinge.

If the shear plug was designed as a composite reinforced concrete section, it is expected that the first plastic hinge will develop at, or slightly below, the soffit of the pile cap.

L_c – the distance between maximum moments in the pile (distance between the pile cap soffit and point of pile virtual fixity)

Modified forces shall be used for the design of the Capacity Protected Elements within the pile cap. Such elements related to the pile–to–pile cap connection detail comprise:

- Pile shear plug within the pile cap shown in *Figure 2* and
- Top and bottom shear plug confinement reinforcement shown in *Figure 3*.

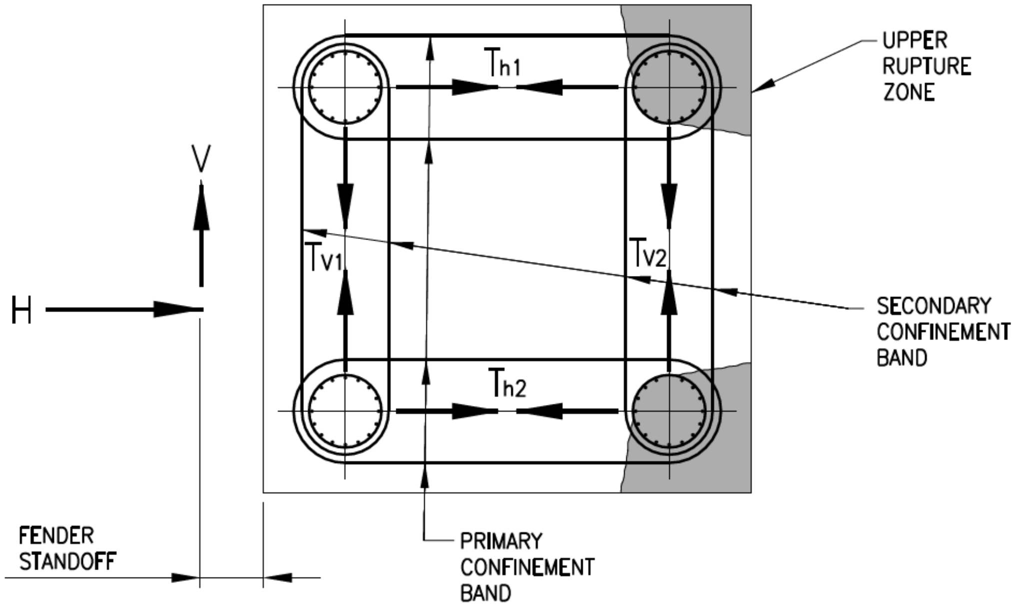


Figure 2A : Top confinement band reinforcement

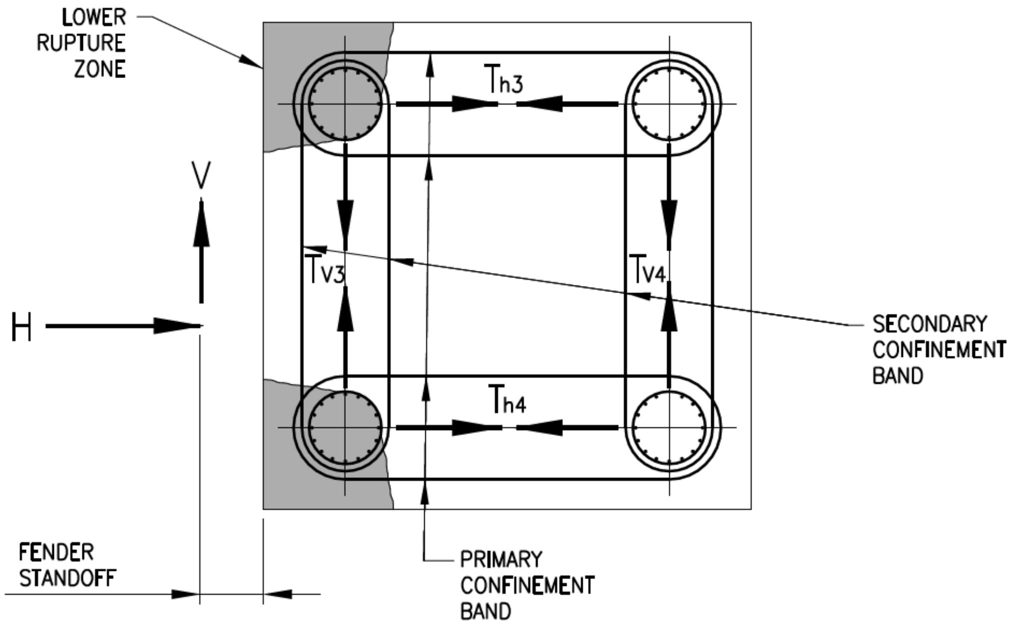


Figure 2B : Bottom confinement band reinforcement

Figure 2 : Pile cap plan confinement band reinforcement

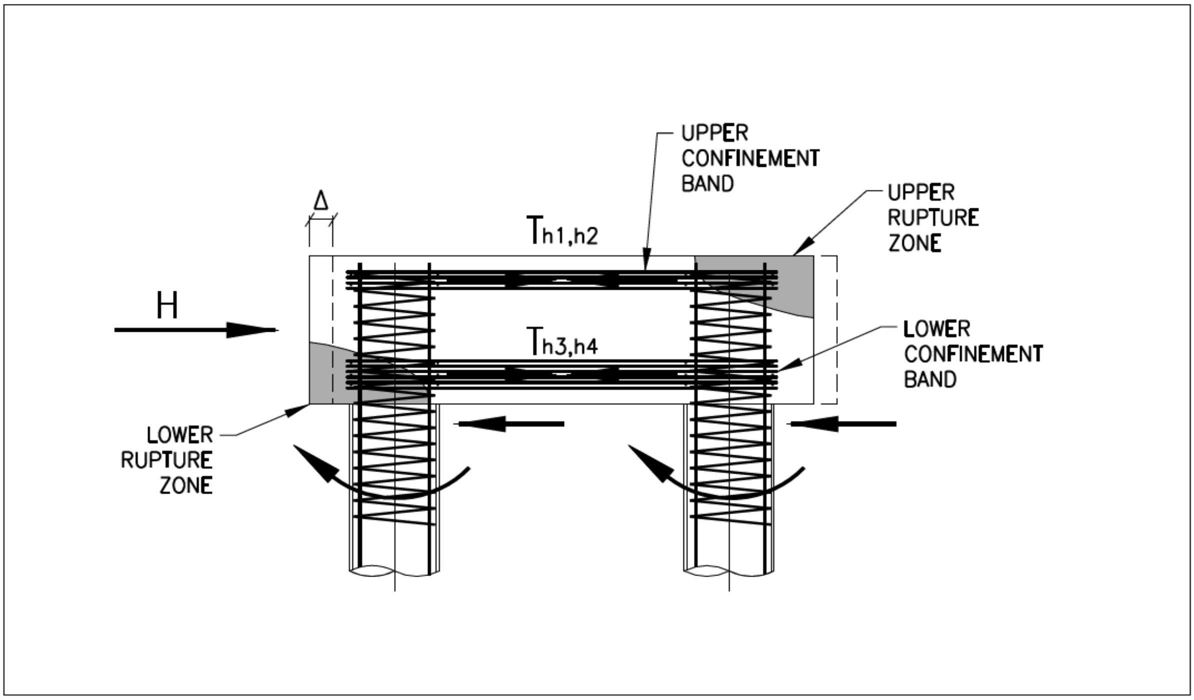


Figure 3

Comment 5

If dolphin undergoes elasto-plastic deformations (Flexible Dolphin), total deflection of the dolphin will be based on the moment of inertia of the remaining elastic part of the pipe section, I_{eff}

For calculating deflection within the elasto-plastic mode, the designer must calculate a new moment of inertia for partially plastisized pipe pile section. It shall be understood that I_{eff} is a variable number depending on the extent of the plasticized extremities of the section. The following outlines step by step analytical procedure for calculation of the Effective Moment of Inertia and Ultimate Flexural Capacity of the partially plastisized pipe section.

Step 1. Calculate Effective Moment of Inertia of the pipe section with

O.D = 2R and

I.D. = 2r.

t(hickness) = R-r

Step 2. Define the angle between the neutral axis and the edge of the slice, (α), as shown in Fig. 4

Step 3. Define chords confined by a small increment dα:

Exterior and interior archs of the pipe confined by dα can be approximated by a chord length,

$$R * d(\alpha) \qquad \text{(Formula 6)}$$

$$r * d(\alpha) \qquad \text{(Formula 7)}$$

Step 4. Calculate area of the pipe shell confined by d (α):

$$dA_i = 1/2 * (R+r) * t * d(\alpha) \qquad \text{(Formula 8)}$$

Step 5. Define the distance from the neutral axis to the elementary area,

$$y_i = y\alpha = 1/2 * (R+r) * \sin(\alpha) \qquad \text{(Formula 9)}$$

Step 6. Calculate moment of inertia of the pipe section confined by the central angle (α) in each of the 4 quadrants,

$$I_{eff} = 2\int y_i^2\, dA_i = 2((R+r)/2)^3 * t * \int_{-a}^{\alpha} \sin^2(\alpha) * d(\alpha)$$

$$I_{eff} = 1/4 * (R+r)^3 * t * [0.5 * \alpha - 0.25 * \sin 2(\alpha)] \Big| \text{ over integration limits} \qquad \text{(Formula 10)}$$

For checking formula, set integration limits between (π/2) and (–π/2) for fully elastic section:

$$I_\alpha = I_{a\,eff} = 1/4 * (R+r)^3 * t * [0.5 * \alpha - 0.25 * \sin 2(\alpha)] \Big| = 0.25 * (R+r)^3 * t * (1.57) \qquad \text{(Formula 11)}$$

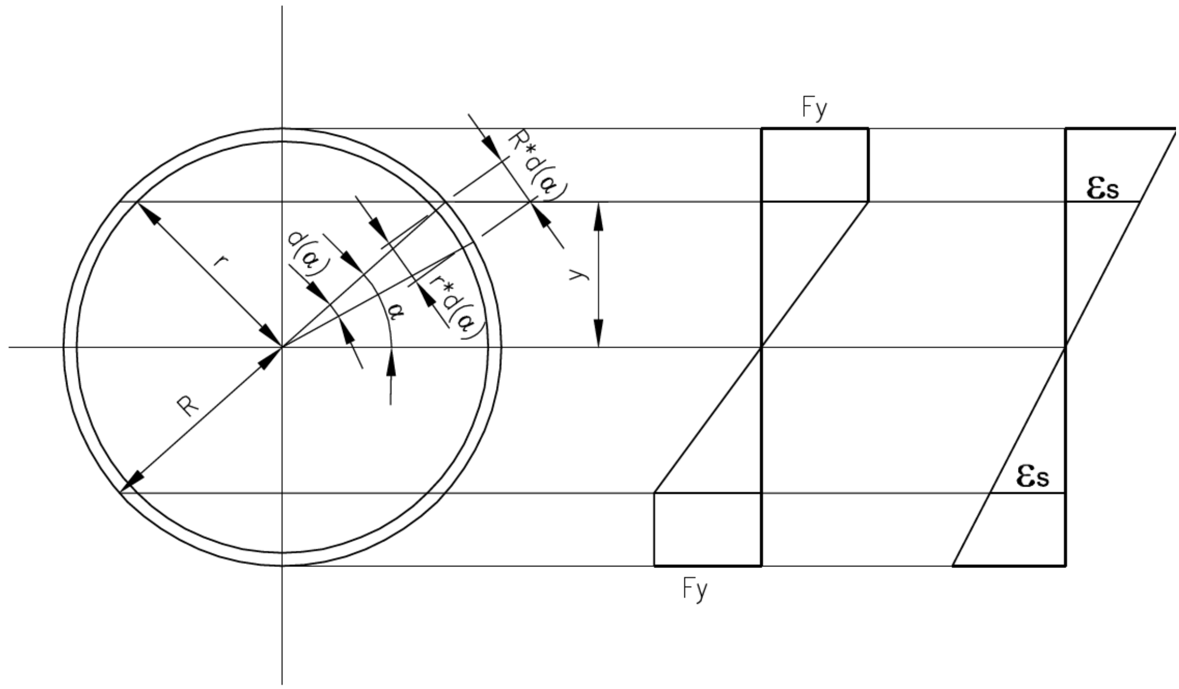


Figure 4 : Pipe section plastification

Step 7. Try central angle (α) satisfying flexural demand.

Step 8. Calculate Elastic Section Modulus. (Elastic Section Modulus varies with central angle α)

$$S_\alpha = I_{\alpha\ eff} / y_\alpha \qquad \text{(Formula 12)}$$

Where,

Iα and *yα* are effective moment of inertia ($I_{\alpha\ eff}$) and (y) corresponding to a central angle (α)

Step 9. Moment taken by elastic portion of the section

$$M_{el} = F_y * S\alpha \qquad \text{(Formula 13)}$$

Step 10. Calculate Plastic Section Modulus, Z= $\Sigma dA_i * y_i$

$$Z_\alpha = 4\int y_i * dA_i = 2*0.5*(R+r)^2 * t * \int_0^{\pi/2} \sin(\alpha)*d(\alpha)$$

$$Z_\alpha = -1.0*(R+r)^2 * t * \cos(\alpha) \Big| \text{ over integration limits} \qquad \text{(Formula 14)}$$

For checking formula, set integration limits between (π/2) and (0) for fully plastic section.

$$Z_\alpha = (R+r)^2 * t \quad \text{(fully plastic section)} \qquad \text{(Formula 15)}$$

Moment taken by a plastisized portion of the section

$$M_{pl} = F_y * Z\alpha \qquad \text{(Formula 16)}$$

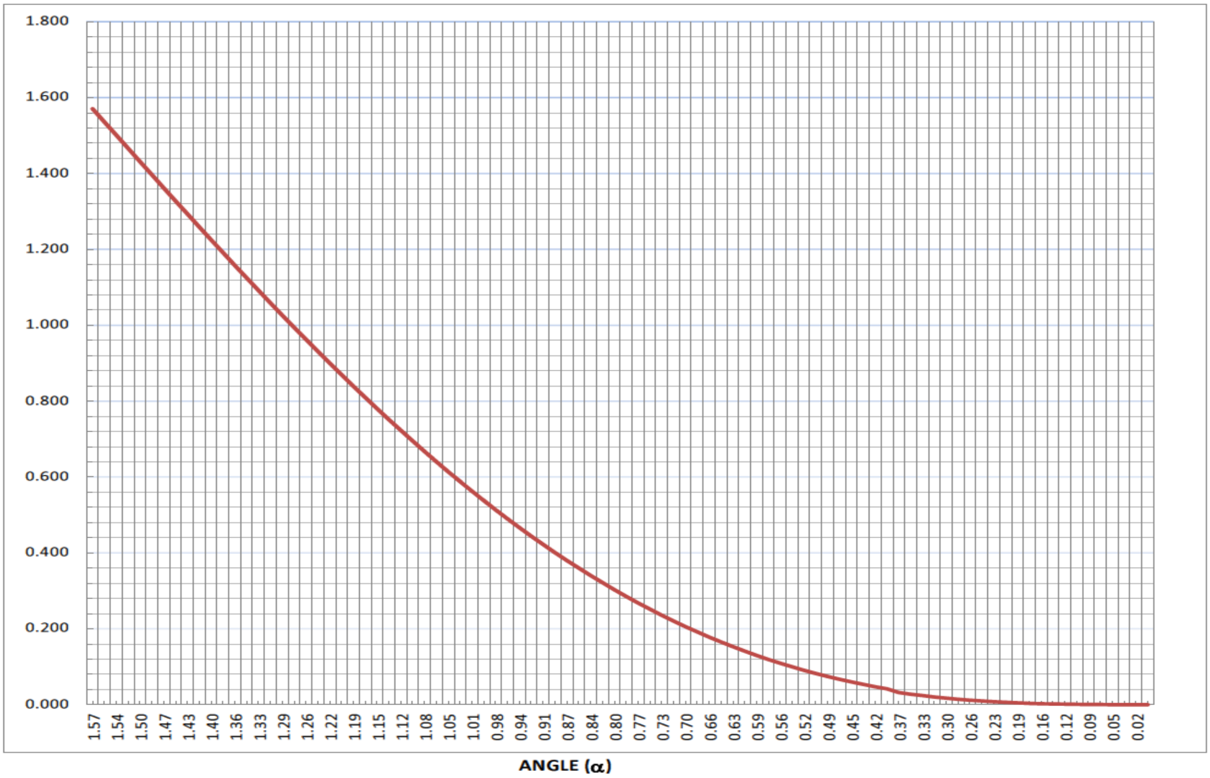
Step 11. Total moment capacity of the section is described by Formula 17

$$M_{el\text{-}pl} = F_y * (S\alpha + Z\alpha) \qquad \text{(Formula 17)}$$

Step 11 concludes analysis of partially plastisized pipe section.

Compliance with clause 6.6.4 of PIANC WG-33: “deflection equal to 2 times elastic deflection,” requires at least part of the pipe section to be in a plastic mode, thus reducing the effective moment of inertia of the pile section to the level where the elasto-plastic section deflects twice the magnitude of initial elastic displacement. The new moment of inertia for such section is defined by Formula 10.

Figure 5 provides a useful tool for a quick calculation of I_{eff}, utilizing a simple graph:



$$I_{eff} = 1/4 * (R+r)^3 * t * [0.5 * \alpha - 0.25 * \sin 2(\alpha)]$$

I eff = 0.25*t*(R+r)^3 * k

k= [0.5 * a – 0.25 * sin2(a)]

I eff vs. (α)

Figure 5

a) Dolphin Ductility

Elasto-plastic behavior of the pile section (Flexible Dolphin) opens concept of dolphin ductility.

Figure 6 shows the Force vs. Deflection Graph where maximum ultimate deflection (Δ_{du}) is limited by the ability of the dolphin to absorb plastic deformations without losing stability. The ratio of the max displacement (Δ_{du}) to the elastic displacement of the dolphin (Δ_{de}) is called dolphin system ductility factor (μ_D).

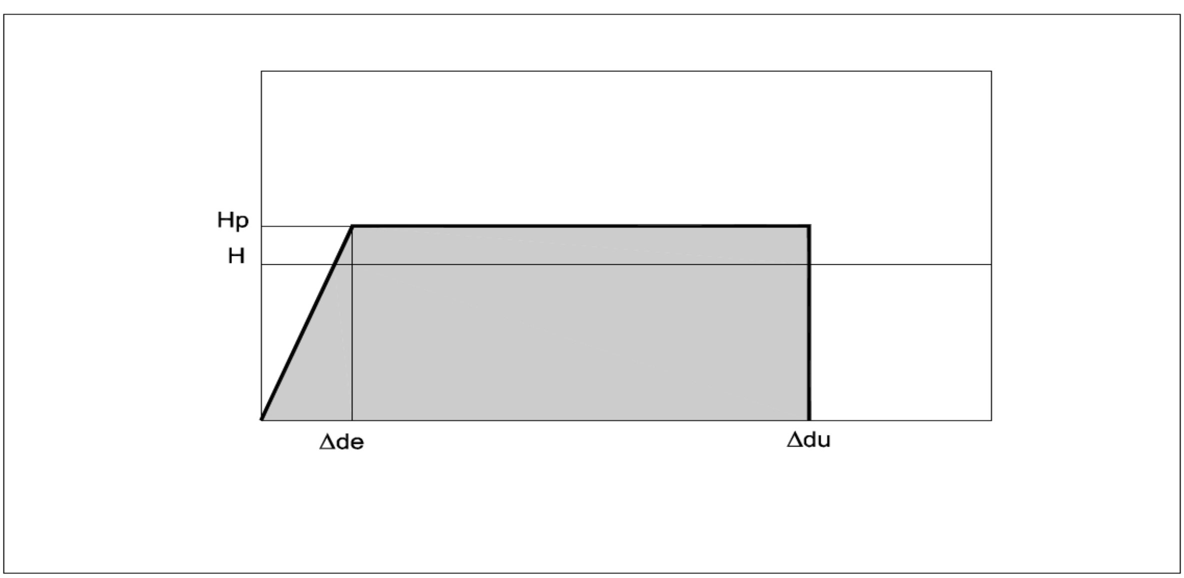


Figure 6 : Force vs. Deflection

$$\mu_D = \Delta_{du} / \Delta_{de} \quad \text{(Formula 18)}$$

Equating the work done by the hypothetical external force (H) to the energy absorbed by the dolphin:

$$H*\Delta_{du} = 0.5H_p*\Delta_{de} + H_p*(\Delta_{du} - \Delta_{de}) \quad \text{(Formula19)}$$

Where,

$H*\Delta_{du}$ – is work done by a hypothetical impact force (H)

$0.5H_p*\Delta_{de} + H_p*(\Delta_{du} - \Delta_{de})$ – Energy absorbed by a dolphin prior to being forced into instability.

Rewriting Formula 19 in terms of H_p /H:

$$H_p / H = 2\mu_D / (2\mu_D - 1) \quad \text{(Formula 20)}$$

Formula 20 establishes the relationship between Dolphin Capacity (H_p) and Demand Load (H),

Where H is the maximum anticipated load.

It should be understood that ductility factor applies only to flexible dolphins, but does not have any physical meaning for semi-flexible systems exhibiting fully elastic behavior.

Comment 6:

A ductility factor of $\mu_D < 3$ shall be used as a target for flexible dolphin design. Ductility factors in that range allow the structure to be in continual use while undergoing insignificant structural repairs.

A ductility factor of $4 < \mu_D < 7$ defines damage criteria associated with moderate damage to the dolphin structure.

However, certain design limitations shall apply:

- Design shall rely on plastic deformations of the pile material, but not on elasto-plastic deformations of the soil.
- Pipe pile shall not be subjected to ovalization and/or buckling.
- Residual plastic deflection should not be excessive.

Comment 7:

Pile ovalization shall be checked when pile D/t > 60. Corrosion allowance must be considered for that type of analysis.

b) *Dolphin Ductility Check. Design of the Pile-to-Concrete Pile Cap connection detail for a Flexible Dolphin System*

Flexible dolphins frequently show significant signs of distress at the pile-to-pile cap connections. When unsuspecting engineers design and detail the pile cap of the flexible dolphin similarly to the pile cap of the rigid dolphin, without proper investigation of the path of load resistance, pile-to-pile cap connection detail becomes a weak link in the pile cap design: pile prying action ruptures concrete of an improperly reinforced pile cap.

❖ *Example 3. Dolphin Ductility Check. Design of Pile–to–Pile Cap connection*

Example 3 provides a detailed review of the pile–to–pile cap design based on Elastic Foundation Analysis. In certain circumstances, outlined in Comments below, the Strut-and-Tie model or pin-pin support boundary may be used, as well.

Magnitude of the direct impact and magnitude of the fender standoff affect the torsional component of the force acting on the most critically loaded pile of the dolphin. Geometry of the pile cap is given below:

Modified factored impact force based on fender/dolphin interaction:

$$R=2476*L.F/CAB = 2476*1.5/1.25 = 2{,}971kN \quad \text{(Table 2)}$$

Standoff at rated reaction prior to fender buckling, $D_{stand\ off}$ = 1.80-0.63 =1.17m

Distance between piles in both directions, $d_x = d_y$ = 6m

Size of the pile cap = 9m x 9m

Polar moment of inertia of a 4 vertical pile dolphin system,

$$I_p = 4*(d_x/2)^2 + 4*(d_y/2)^2 = 2*4*9 = 72m^4$$

Torsion due to tangential force, M_T = 14,038*μ= 2,808kN-m - un-factored (see Table 4).

Torsional component of the force parallel to the force of direct impact,

$$Pt = M_T*d_x / I_p = 2{,}808*3/72 = 117kN \text{ - un-factored}$$

The factored force acting on the critical pile in that case is:

$$H_i = 2{,}971/4 + (2{,}971/2{,}476)*117 = 843kN$$

(where un-factored fender reaction at calculated fender deflection, R = 2,476kN)

The critical load combination acting on a single pile:

M_u = 11,497kN-m	- factored moment	(FEA output)
V_u = H= 803kN	- factored force demand	(Force Demand)

- *Analysis of the dolphin ductility*

Steel material, F_y = 344MPa

Plastic Section modulus of a single pile,

$$Z = (D^3 - d^3) / 6 = (1488^3 - 1450^3) / 6 = 41.00*10^6\ mm^3$$

Elastic Section Modulus of a full pipe section single pile,

$$S = 0.098175*(D^4 - d^4) / D = 0.098175*(1488^4 - 1450^4)/1488 = 31.79*10^6 mm^3$$

Pile material, Fy =344MPa

Pile length between the pile cap soffit and *point of pile virtual fixity*, Lc= 32m

The concept of a point of virtual fixity shall be further explained, since frequently this point is determined incorrectly.

Comment 8:

A point of virtual fixity is a fictitious point where artificial fixity introduced into the pile model creates deflection effect similar to a pile model with soil springs described by p-y curves supplied by a geotechnical investigation report.

Full plastic moment capacity of the pile, $M_p = 344*41.00 = 14,104 kN\text{-}m$

Elastic moment capacity of the pile, $M_e = 344*31.79 = 10,936 kN\text{-}m$

Plastic capacity of a single pile, $H_p = 2*M_p / L_c = 2*14,104/32 = 882kN$ (Plastic Capacity)

Elastic capacity of a single pile, $H_e = 2*M_e / L_c = 2*10,936/32 = 683kN$ (Elastic Capacity)

$H_p / H = 882/803 = 1.10$ ==>

Based on Formula 20 and Table 5 calculated ductility factor of the pile,

$H_p / H = 2\mu_D / (2\mu_D - 1) = 1.10$ ===> $\mu_D = 5.50$

The dolphin experiences elasto-plastic behavior, falling under category of flexible dolphins. (Comments 2 and 5).

Accordingly, the dolphin will experience, not a minor, but, a moderate distress which will require a longer time down for remediation repair.

In performance based design criteria, it is important to know the residual deflection of the system. That parameter allows engineer to determine projected useful life of the structure.

Residual plastic displacement of the system can be estimated from the following equation,

$$\Delta_{res} = \Delta_{I\ eff} - \Delta_{I\ gross},\qquad \text{(Formula 20)}$$

Where,

$\Delta_{I\ eff}$ - deflection based on effective moment of inertia of elasto-plastic section determined from Formula 10

$\Delta_{I\ gross}$ - deflection based on moment of inertia of fully elastic section.

Table 5, below, provides a comparison between ductility factors based on recommendations of PIANC WG-33, clause 4.2.8.4(d), and ductility factors recommended by CALTRAN[5] and other reputable performance based criteria guidelines.

Table 5 : Comparison of Ductility factors

H_p / H	μ_D	Remarks
3	–	**Rigid Dolphin.** Case is outside of performance based criteria. No inelastic displacement is anticipated. Pile to pile cap connection design requires application of 25% overload factor for design of Capacity Protected Elements.
2	1	**Semi-Flexible Dolphin**. Case is outside of performance based criteria. No residual inelastic displacement is anticipated. Close to 20% of impact energy is absorbed by dolphin elastic deflection. Pile to pile cap connection design requires application of 25% overload factor (γ=1.25) for design of Capacity Protected Elements.
1.2	3	**Flexible Dolphin** Case is within performance based criteria. Minor structural damage. Minor to moderate residual inelastic deflection should be anticipated. More than 25% of impact energy is absorbed by dolphin elasto-plastic deflection. Overload factor, γ for pile to pile cap connection design shall be determined from **Figure 4**. Overload factor application required for design of Capacity Protected Elements.
1.15	4	**Flexible Dolphin**. Case is within performance based criteria. Moderate structural damage. Moderate residual inelastic deflection should be anticipated. More than 25% of impact energy is absorbed by dolphin elasto-plastic deflection. No overload factor (γ =1.0) required for design of Capacity Protected Elements.

Iterating on angle (α) (*Formula 10*), designer can determine:

a) an elasto-plastic section satisfying the factored moment demand (M_u =11,497kN);

b) calculate effective moment of inertia (I_{eff});

c) estimate additional elastic displacement associated with I_{eff}, based on energy absorption requirements

Comment 9:

Clause 4.2.8.4(d) of PIANC WG-33 requires design of the fender supporting structure for a force of 2 (μ_D =1) to 3 times (μ_D =0.75) greater than the force of abnormal impact.

Review of such requirement indicates that it lays outside of performance based criteria promoting rigid to semi –flexible dolphins rather than flexible dolphins with residual plastic deformations.

Table 5 provides good correlations between ductility factor μ_D and ratio of Dolphin Capacity (H_p) and Demand Load (H)

The data presented in Table 5 explains Clause 4.2.8.4(d), but also indicates that a good design practice should target fully elastic semi-flexible dolphin system.

$$1.6 < H_p / H < 2.0 \text{ or } 1.0 < \mu_D < 1.33$$

However, forensic investigation of the designed dolphin requires iteration process.

Try central elastic angle α = 75deg = 1.32 rad. Utilizing Formula 9,

$$I_{eff} = 1/4*(R+r)^3*t*(0.5*\alpha - 0.25*\sin 2\alpha)\Big|_{(-1.32\,rad < \alpha < 1.32\,rad)} =$$

$$=0.25*(744+725)^3*19*[(0.5*1.32-0.25*0.48)-(-0.5*1.32+0.25*0.48)] = 16.26*10^9\ mm^4/\text{pile}$$

$$I_{eff} = 16.26*10^9\ mm^4/\text{pile, or } 65.05*10^9 mm^4 / \text{per 4 dolphin piles}$$

$$I_{gross} = 23.65*10^9 mm^4/\text{pile, or } 94.60*10^9 mm^4 / \text{per 4 dolphin piles}$$

Elastic Section Modulus

$$y_\alpha = 1/2*(R+r)*\sin\alpha = 0.5(744+725)*\sin 1.32 = 711.5mm$$

$$S = I_{eff} / y_\alpha = 16.26*10^9 / 711.5 = 22.85*10^6\ mm^3$$

$$M_{el} = F_y * S = 344*22.85 = 7{,}861\text{kN-m}$$

$$M_{pl} = F_y*Z = F_y*t*(R+r)^2*\cos\alpha\Big|_{1.32\,rad < \alpha < 1.57\,rad} = 344*19*(744+725)^2*0.248 = 3{,}498\text{kN-m}$$

$M_{el\text{-}pl}$ = 7,861+3,498 = 11,809kN-m > M_u =11,497kN-m (2.7% difference, deflection results will be acceptable)

Total elasto-plastic deflection experienced by dolphin,

$$PL^3/(12E*I_{eff}) = (803*1.25/1.5)*32^3/(12*2*10^8*16.26*10^{-3}) = 0.56m$$

Elastic deflection experienced by gross section of dolphin,

$$PL^3/(12E*I_{eff}) = (803*1.25/1.5)*32^3/(12*2*10^8*23.65*10^{-3}) = 0.38m$$

and

$$\Delta_{el\text{-}pl} / \Delta_{el} = [PL^3/(12E*I_{eff})] / [PL^3/(12E*I_{gross})] = 0.56/0.38 = 1.47 < 2.0$$ (PIANC WG-33, clause 6.6.4)

Residual plastic deformation of the dolphin,
$\Delta_{res} = \Delta_{I\,eff} - \Delta_{I\,gross} = 0.56 - 0.38 = 0.18m$

Calculated residual deflection is excessive.

Study of the case indicates that dolphin was designed as a Flexible system, and therefore will have a fairly short useful life considering magnitude of the residual deflection.

Utilizing the graph shown in *Figure 7,* engineer can find overload factor (γ =1.12) utilized for analysis of the Capacity Protected Elements within the pile cap.

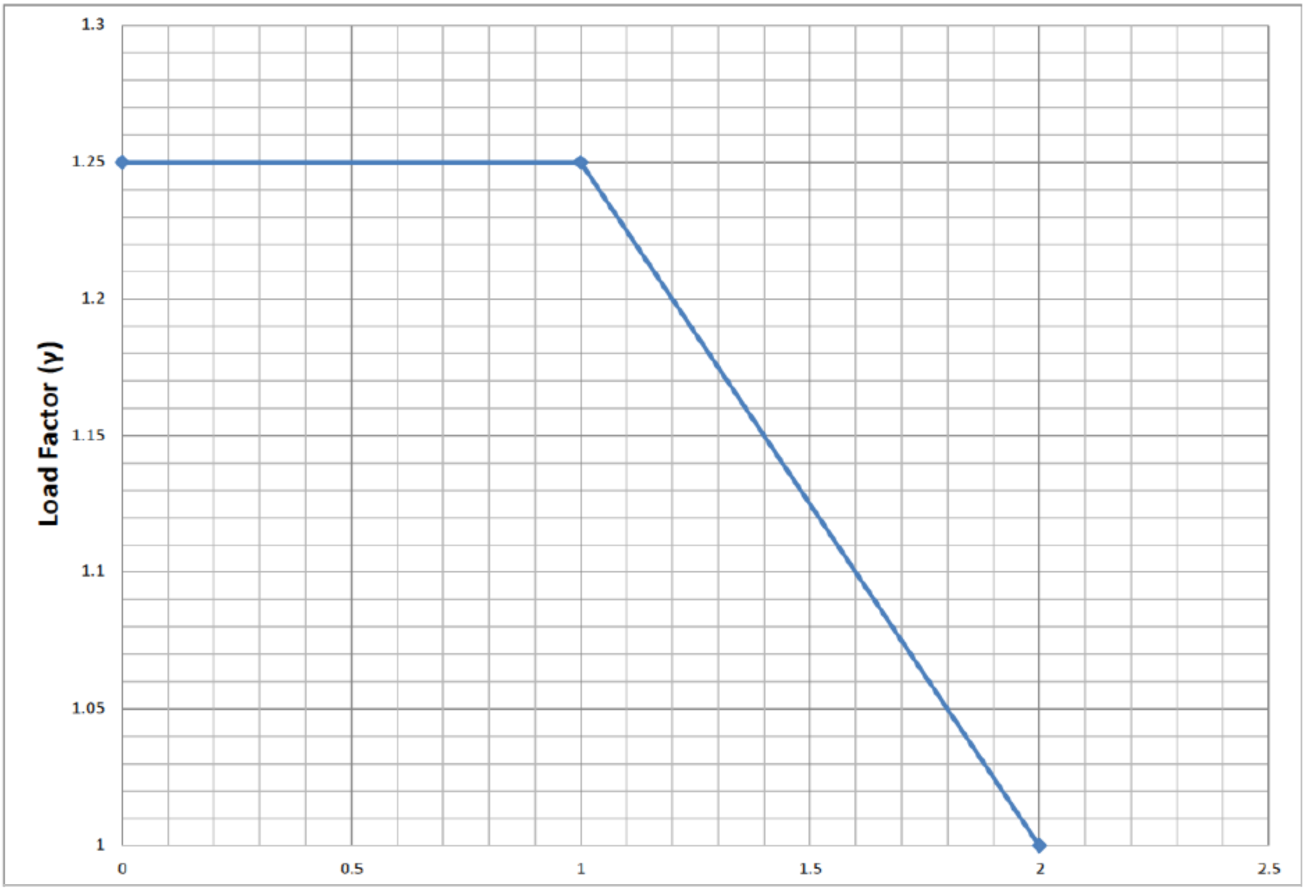


Figure 7 : Ratio of (Elasto-Plastic Deflection) / (Elastic Deflection)

Comment 10:

A stiffer dolphin structure will require more robust pile-to-pile cap connection detail and may shift the system into the rigid dolphin category, while softer system will push dolphin into the flexible design category. Both, rigid and flexible dolphins present extreme and hardly rational design cases. Rational design shall be based on semi-flexible dolphin system philosophy.

- *Pile–to-Concrete Pile Cap connection design*

Figures 2 and 3 show pile cap failure zones developing as a result of "shear plug" prying action caused by the pile rotation. Rotation of the shear plug and its rigidity impose heavy reaction forces against the confining bands (Figures 2A and 2B). Sensitivity of the pile-to-pile cap connection detail becomes obvious if it is viewed as an inverted pile embedded into a rigid medium.[7]

The detail should be modeled as a short beam on an elastic foundation utilizing a two-point p-y curve of soft rock as a substitution for a concrete p-y curve. Pile-to-pile cap connection detail (*Figures 2 and 3*) has two Capacity Protected Elements:

- Shear plug detail (pile extension into the pile cap)

and

- Shear plug confinement reinforcement for top and bottom rupture zones.

Therefore, the shear plug confinement band shall be designed for the restraining of shear plug rotation.

The forces

$$M_o^{\,pile} = \gamma * M^{\,pile} = 1.12M_{pile} = 11{,}497 * 1.12 = 12{,}071 kN\text{-}m \quad (\gamma = 1.12, \text{ See Figure 7})$$
$$V_o^{\,pile} = H = 2 * 1.12\, M_{pile} / L_c = 2.1\, M_{pile} / L_c = 803 * 1.12 = 843 kN$$

in the model were applied at the level of the pile cap soffit.

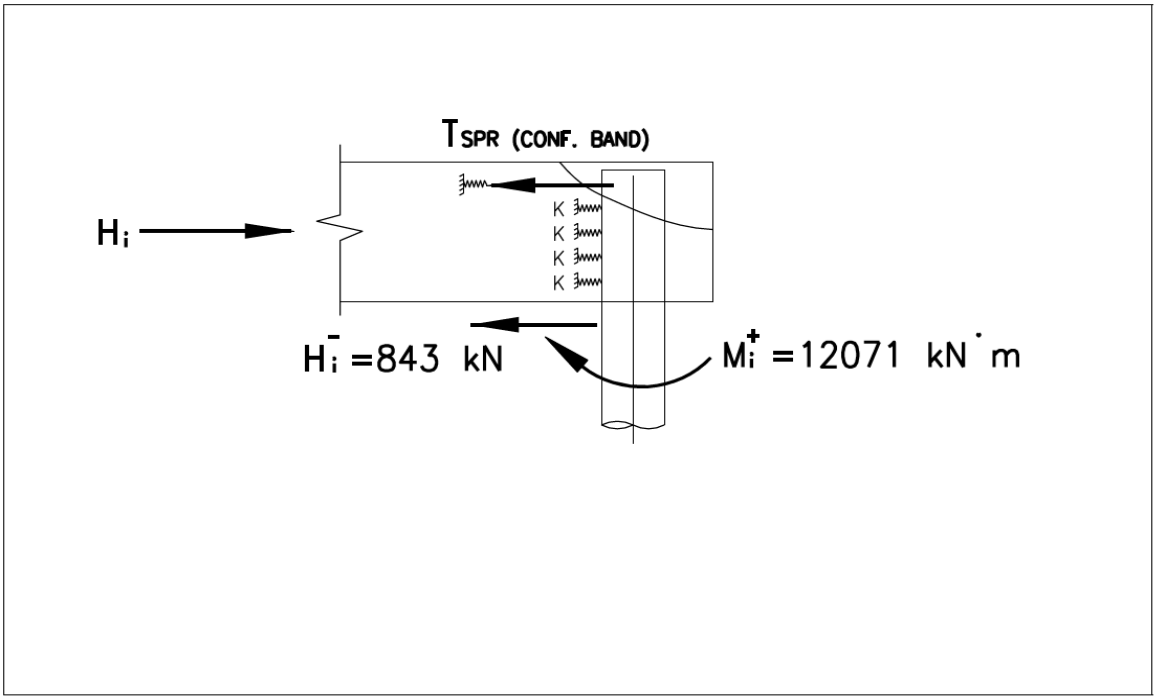


Figure 8 : Upper band confinement. free body diagram

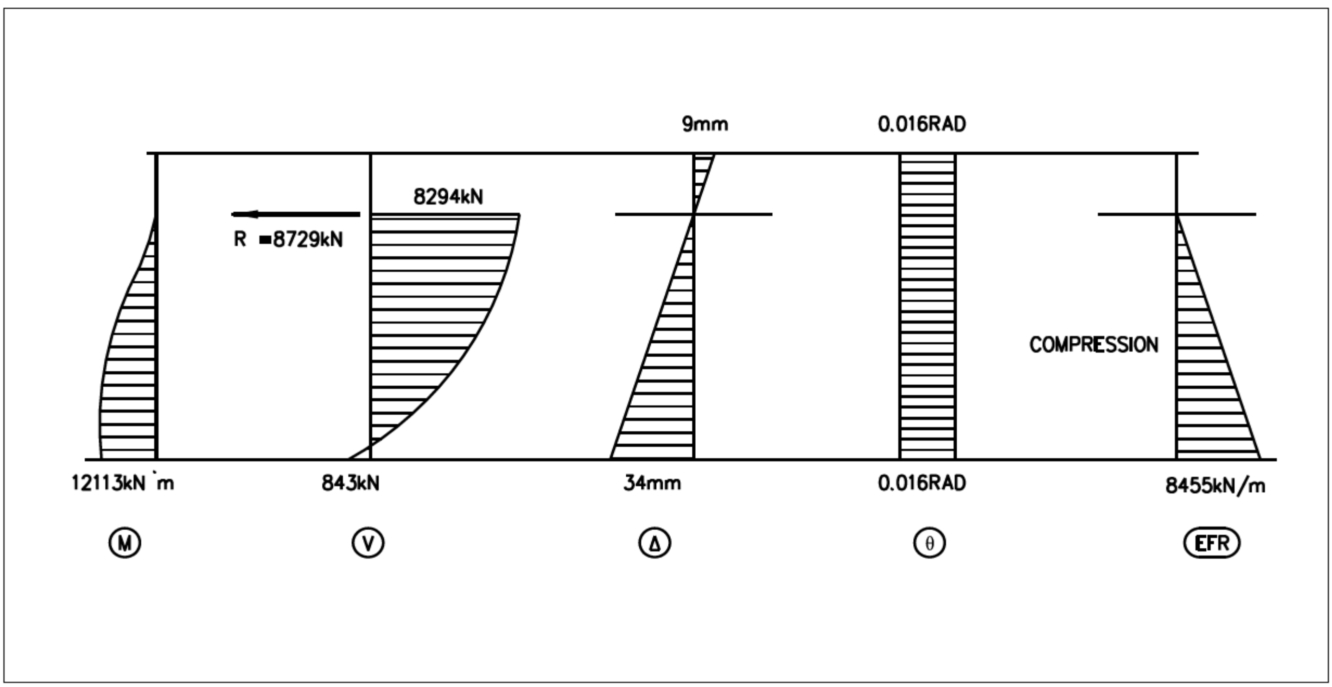


Figure 9 : Upper band boundary

Figure 8 shows the pile-to-pile cap connection free body diagram for the upper level confining reinforcement band; and *Figure 9* shows moment (M), shear (V), deflection (Δ), Slope (Θ), and Elastic Foundation Reaction (EFR) diagrams for the upper level of confinement extracted from VersaBeam 3.0 (ROMAK) analytical software.

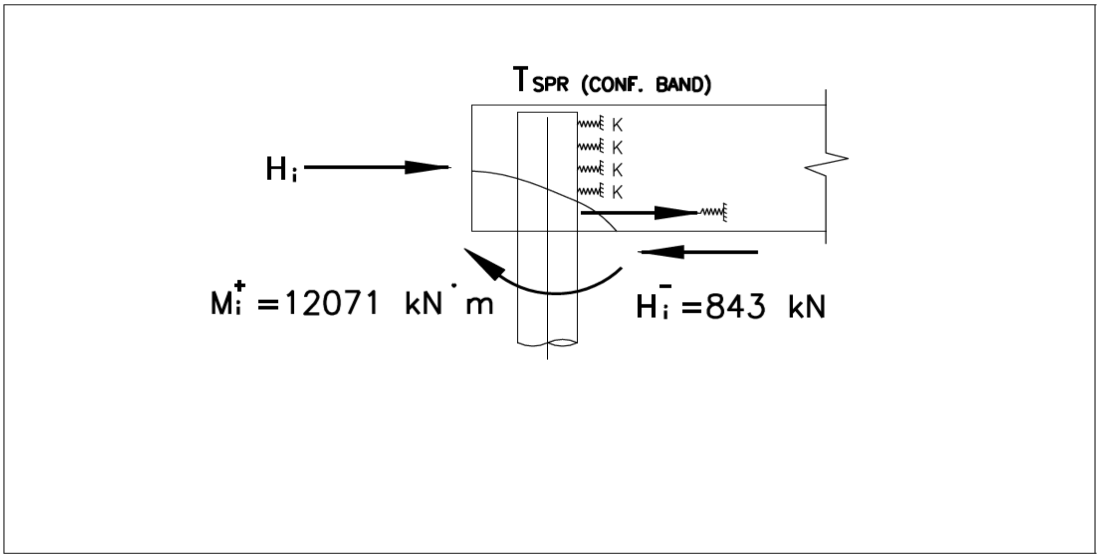


Figure 10 : Lower band confinement. free body diagram

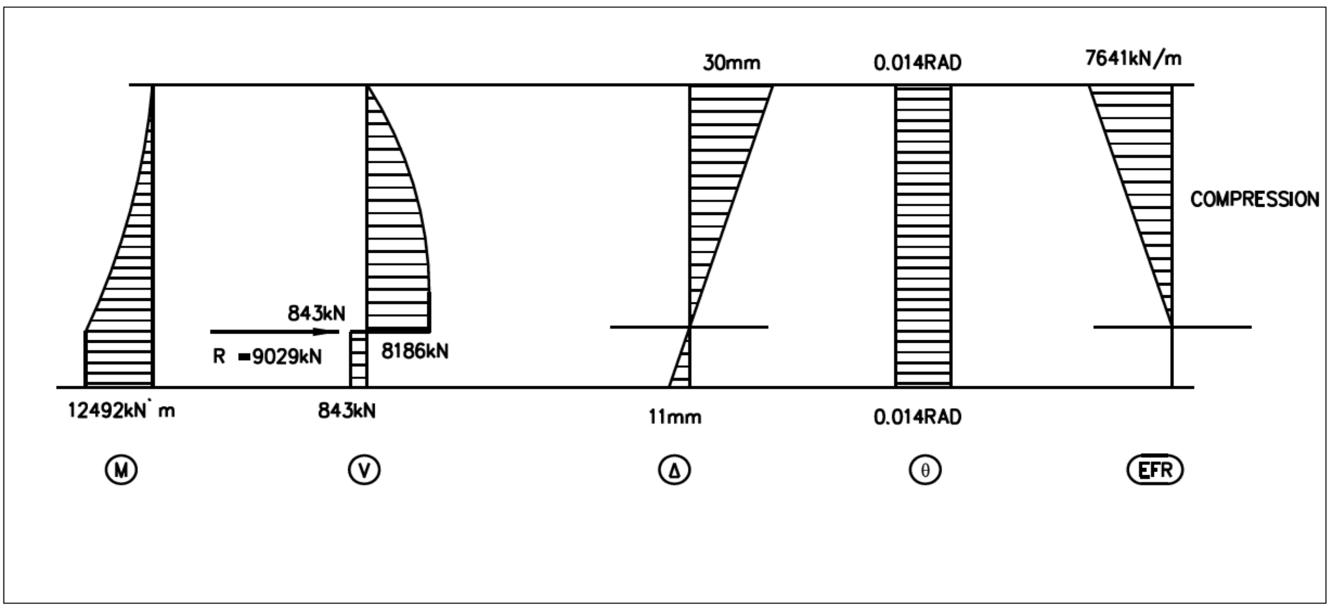


Figure 11 : Lower band boundary

Figures 10 and 11 : Provide similar diagrams for analysis of lower confinement

De-bonded length of the confining band was assumed to be L_{db}= 600mm.

During initial ship impact, the concrete around confining strap spalls, and the exterior layer of confining band de-bonds in between stirrups, or U-bars, anchoring band reinforcement in both lateral directions.

Results of the analytical runs for both cases are summarized below:

- Top confinement band model

R^{top}_{spr} = 8,294kN

M^{top} = 12,113kN-m

EFR = 8,455kN/m

F_y = 551MPa (ASTM A706 high strength mild steel)

Area of the primary lower band reinforcement was calculated as

$$A^{top}_s = R^{top}_{spr} / 0.9*F_y = 8,249*10^3 / (0.9*551) = 16,634mm^2$$

- Bottom confinement band model

R^{bot}_{spr} = 9,029kN

M^{bot} = 12,493kN-m

EFR = 7,641kN/m

F_y = 551MPa (ASTM A706 high strength mild steel)

Area of the primary lower band reinforcement was calculated as

$$A^{bot}_{s} = R^{bot}_{spr} / 0.9*F_y = 9{,}029*10^3 / (0.9*551) = 18{,}207mm^2$$

Effective width of the shear plug (1.2m) was determined from the shear plug geometry.

Therefore, bearing stress under the shear plug

$$f_{brg} = 8{,}455kN / (1.2*1.0) = 7.05MPa < 0.85*35MPa = 30MPa$$

Confinement band reinforcement shall be placed as compactly as possible, placing 4 leg bands in one layer when possible.

Comment 11:

Based on Design Memorandum of WSDoT (February 14, 2012), ASTM A706 Grade 80 steel (Fy =80 ksi or 551MPa) may be used for elements not experiencing inelastic deformations. Grade 80 reinforcement steel can be effectively utilized for design and detailing of capacity protected elements experiencing tensile forces only.

It shall be noted that Figures 8 and 10 show a constant slope and, as a result, exaggerated deflection of the pile shear plug. It is considered prudent and conservative to artificially increase stiffness of the pile shear plug until it starts behaving as a short, stiff beam on an elastic foundation. Such an approach yields slightly conservative results for the magnitude of the reaction force resisted by Capacity Protected Elements (confining straps). However, for investigation of the concrete crushing and plug deflection, designer shall use the real stiffness of the pile shear plug.

Size of the secondary confinement reinforcement, in the direction perpendicular to the primary confinement, can be determined from the ratio of secondary force (force parallel to the fender face panel) to primary force acting during abnormal berthing impact.

Analysis of the path of resistance not only requires proper identification of the analytical problem, but also selection of the proper modeling technique. In the studied case, the engineer utilized the Strut–and–Tie model for the purpose of Pile–to–Pile cap connection analysis. However, comparison of the analytical model and details on the design drawings showed incompatibility between analysis and detailing.

As a result, lever arm between forces of the resisting couple was grossly overestimated, leading to a 20% deficiency in the area of confining band reinforcement.

Several critical points, discussed below, outline conditions necessary for compatibility between analytical model and design details of the Pile–to–Pile cap connection.

Comment 12:

- Confining Band in tension should not be modeled as a pin support. Note: Confining band acts as a spring, when subjected to a Direct Tension Force (DTF). Length of the spring shall be taken as a distance between confining band lateral supports. Lateral supports must support band in both orthogonal directions.
- Depending on pile position in relation to the force direction, the spring band support should be applied at the top or bottom of the shear plug, but never at both locations. When band is reinforced similarly to a concrete column where ties confine longitudinal rebars and provide direct bearing support for a shear plug, detailing may allow modeling of the confining band in the compression zone as a pin support. Nevertheless, utilization of the Strut–and–Tie model requires additional reinforcement detail: tying top and bottom confining bands with evenly spaced vertical closed stirrups.
- Where design and detailing do not satisfy special provisions of bullet (2) of these Comments, tension spring support shall be coupled with compression E(lastic) (F)oundation (R)eaction of the concrete medium.

Today, in a team work environment, analytical models and detailing are frequently poorly coordinated. Model-design incompatibility happens more often than it could be anticipated.

The Strut–and–Tie model requires special detailing, and there are certain geometrical limits when special detailing becomes economically unviable (particularly when pile diameter exceeds 762 mm).

Another serious omission in the design of the pile–to–pile cap connection is frequently related to detailing of the pile embedment where the designer leaves dowels of the shear plug (within the depth of the pile cap) without spiral or tie confinement.

Such an omission leads to a deficient boundary condition of the shear plug itself, changing fixed connection to a partial fixity with significant rotational capability. Each dowel of the plug, in that case, acts as a single rebar, reducing rigidity of the shear plug to a sum of rigidities of individual dowels.

IV. Buckling and Ovalization

PIANC WG-33, cl. 6.6.4, purposely excludes possibility of plastic deformations in the soil due to the high unpredictability of such deformations and excludes two critical design parameters:

- local buckling and
- effect of ovalization on local buckling.

Buckling and ovalization must be checked at abnormal impact force which is interpreted as an Ultimate Limit State force.

However, ovalization and local buckling frequently occur prior to plastic yielding. Corrosion, defined as a corrosion allowance, may and will greatly affect pipe pile ovalization and local buckling.

APA -RP2A[8] sets overall and local pile buckling criteria for large diameter pipe piles.

Pile Overload Analysis based on Plastic design (APA- RP2A, section 3.3.1c) provides the following equation:

$$P_u / (A * F_{xc}) + 0.637(\arcsin(M_u / F_{xc} * Z) < 1.0 \qquad \text{(Formula 21)}$$

Where,

P_u and M_u are factored Axial force and Bending Moment.

Z – plastic section modulus

$F_{xc} < 1.2F_y$ - plastic local buckling depending on pile diameter to wall thickness ratio, D/t (API- RP2A, section 3.2.2.b)

Considering average weighted load factor for Ultimate Strength design to be close to 1.5, the ratio of $(1.2/1.5) * F_y$ yields stress design limit of $0.8*F_y$.

PIANC WG-33 does not establish any credible criteria for large diameter pile buckling or pile “egging,” while possibility of such failure prior to plastic buckling is high.

Section ovalization along the soil elastic foundation reduces pipe section moment of inertia, and simultaneously increases the chance of section buckling.

Comments 13:

Whilst circular section can be checked for plastic deformations, there are no established or credible analytical procedures for the buckling of an oval section.

Therefore, it is important to exclude possible ovalization of the pipe pile below the ground surface. The ovalization problem presents designer with two options:

- Option 1: Adjust pipe shell thickness and verify that Von Mises stresses in the pipe shell below the ground surface do not exceed $0.6F_y$. Forces derived from abnormal impact analysis shall be treated as service level loads for that check.
- Option 2: Fill pipe pile annular space with concrete.

Ovalization of the section shall not be allowed, and Von Mises stresses shall be limited to $0.6*F_y$. Such a requirement is only slightly conservative, but fairly safe approach.

V. Model for Checking Pile Ovalization

This paper does not review ovalization problem below the ground level. Investigation of ovalization is a fairly complex task requiring soil spring / pipe shell interaction. Ovalization check below the ground level is generally required when D/t ratio exceeds 60, and is rarely presents a problem. Ovalization issue at the shear plug, however, is frequently neglected. On several reviewed projects the length of the shear plug embedment was underestimated, and at least on one project ovalization of the pile at the pile cap soffit was clearly visible. Model for checking ovalization at the shear plug is shown in *Figures 12 and 13*. The main reason for that check is to determine the required length of the shear plug embedment into the pipe pile. The length of the shear plug embedment shall be sufficient for prevention of the stresses in the pipe pile from reaching steel yield point. It would be recommended to keep stresses in the pile below $0.9F_y$. at Ultimate Limit State. Stress in the pile shall be checked assuming corrosion allowance at the end of the useful life of the structure.

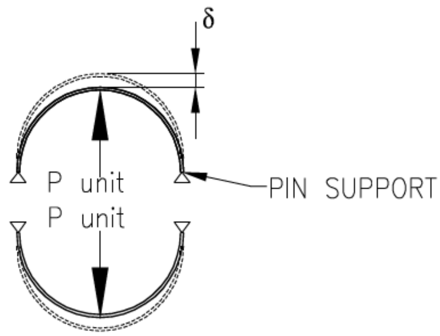


Figure 12 : Elastic Foundation Spring Analysis

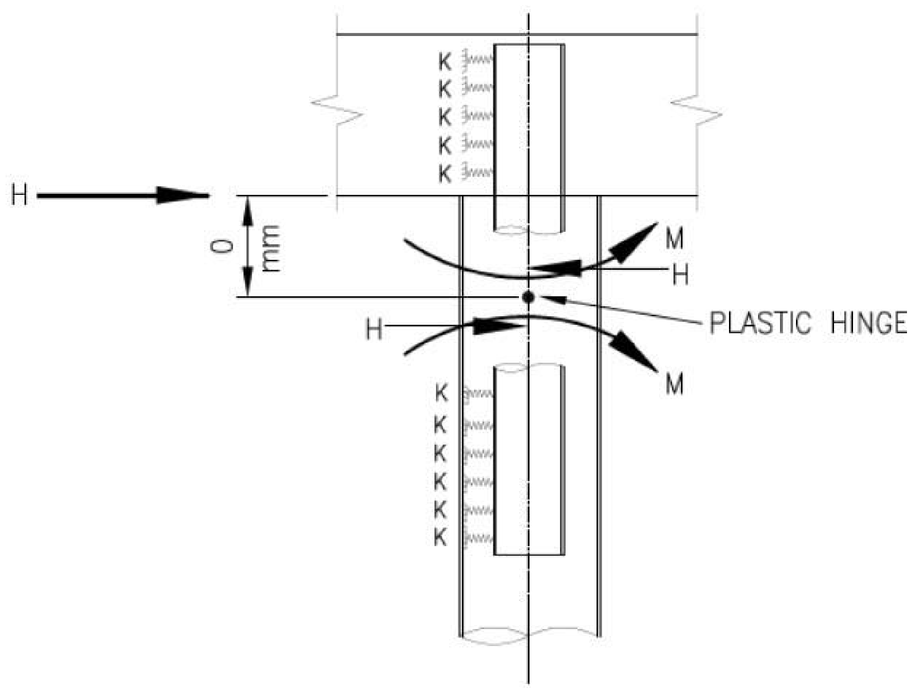


Figure 13 : Shear plug Elastic Foundation Model

VI. Summary of the Connection Detailing Requirements

- Pipe pile shall be extended into the pile cap to the full height of the cap, or alternatively, dowels of the shear plug embedded into the concrete pile cap shall be always confined by a spiral with a pitch not greater than 150mm.
- Shear plug dowels confinement is necessitated by stiffness requirements of the shear in a short pile failure mode. Spiral volumetric ratio and spiral pitch shall be determined from formulas provided by CALTRAN.
- Confinement reinforcement shall run in orthogonal directions shown in *Figure 3*.
- Confinement reinforcement shall be designed with stirrups or ties preventing excessive de-bonding during potential concrete spall. Such ties must be spaced not wider than 600 mm c/c
- Secondary confinement reinforcement does not need to be larger than 20% of the area of primary reinforcement for berthing dolphins. For mooring dolphins, area of primary and secondary confinement will depend on angular positions of the mooring lines.

VII. Summary of the Case Study

Review of the case indicates that while a flexible dolphin solution presents a viable alternative solution to a rigid dolphin system, the engineer should aim for the design of a semi-flexible system exhibiting both elastic behavior and the ability to absorb kinetic energy of impact into a sizable deflection in the dolphin structure.

References Références Referencias

1. Port of Long Beach. Wharf Design Criteria. POLB WDC Version 3.0, February 29, 2013.
2. PIANC WG-33. Guidelines for the Design of Fender Systems: 2002. International Navigation Association.
3. Trelleborg. High Performance Fenders. Section 1.
4. Australian Standard AS4997-2005. "Guidelines for the Design of Marine Structures."
5. Unified Facility Criteria, UFC 4-152-01, "Design of Piers and Wharves."
6. CALTRAN. Seismic Design Criteria. Version 1.6, November 2010. California Department of Transportation.
7. Seismic Design of Pile –to- Pile Cap Connections in Flexible Pier Structures. Vitaly B. Feygin. Structure, SEI. March 2012.
8. API RP-2A "Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms-Working Stress Design", American Petroleum Institute, 2007.