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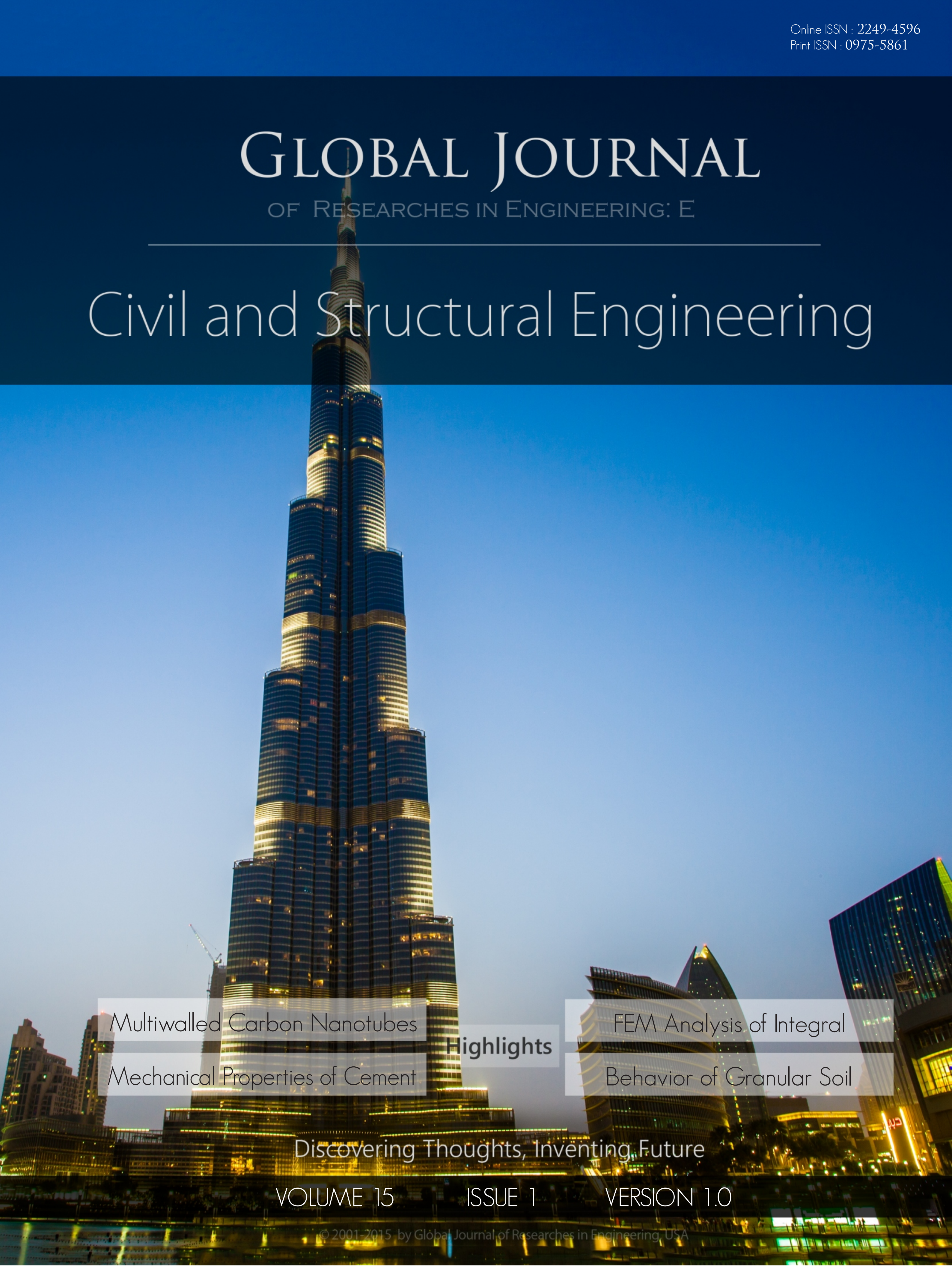

Highlights

- Multiwalled Carbon Nanotubes
- Mechanical Properties of Cement
- FEM Analysis of Integral
- Behavior of Granular Soil

Discovering Thoughts, Inventing Future

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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.

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DESIGNOFSEMIFLEXIBLEANDFLEXIBLEDOLPHINSWITHCONCRETEPILECAPS

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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.

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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
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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

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

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

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

$E'_{AB} = E_O * C_{AB} = 2{,}268 kN\text{-}m$ – 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 ===> $E_N = E_o / TCF = 1815 / 0.825 = 2{,}200$kN-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.

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- 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:

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

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:

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

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

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

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

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

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

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	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

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

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

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.

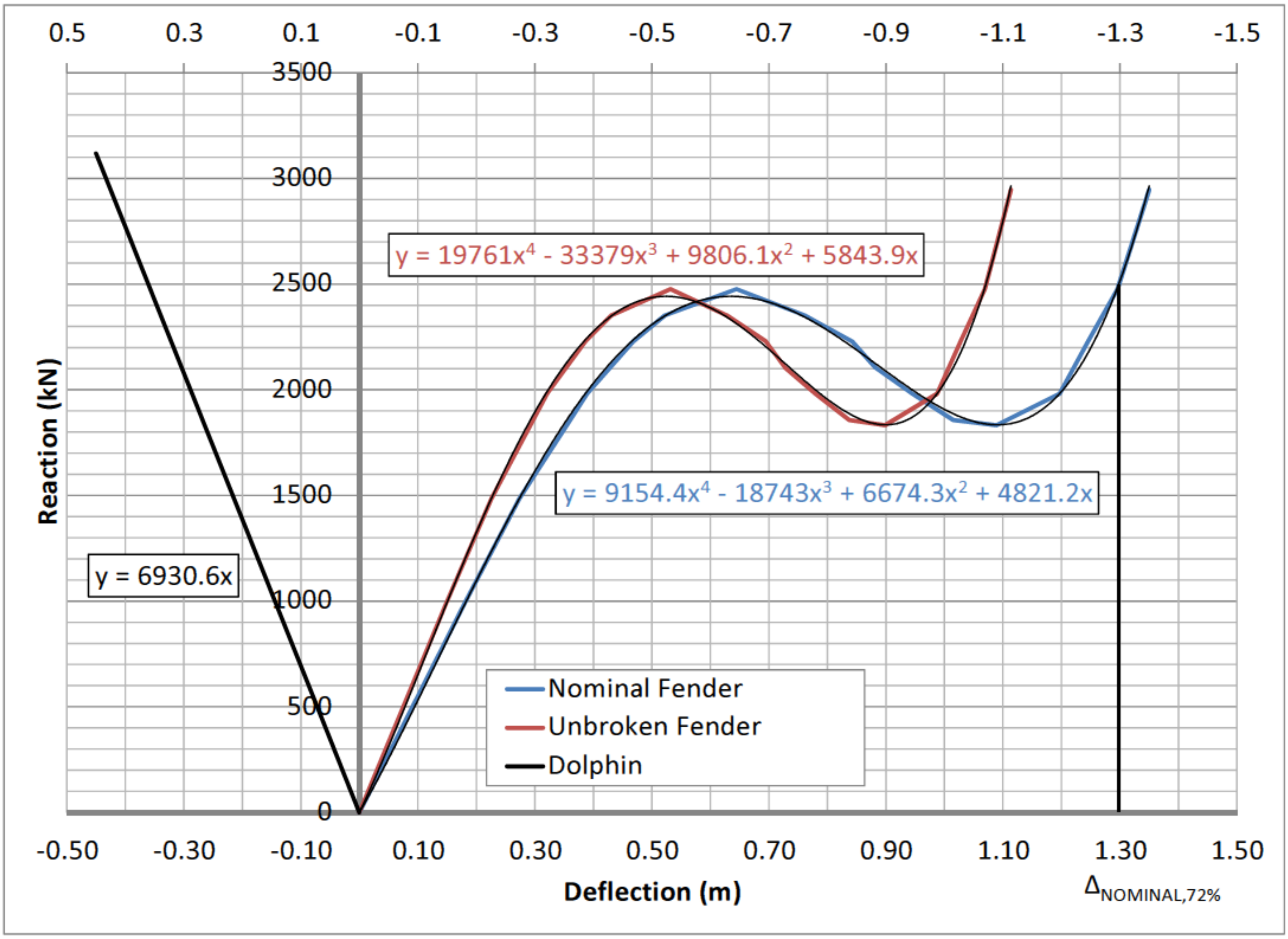

Figure 1 : Pile and Fender Reaction-Deflection Curve

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*.

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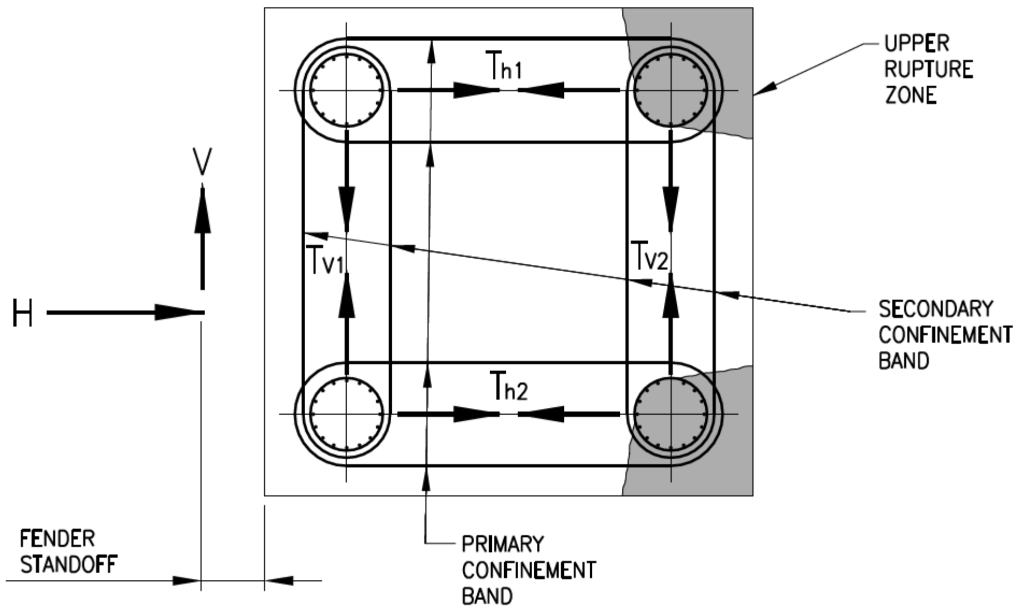


Figure 2A : Top confinement band reinforcement

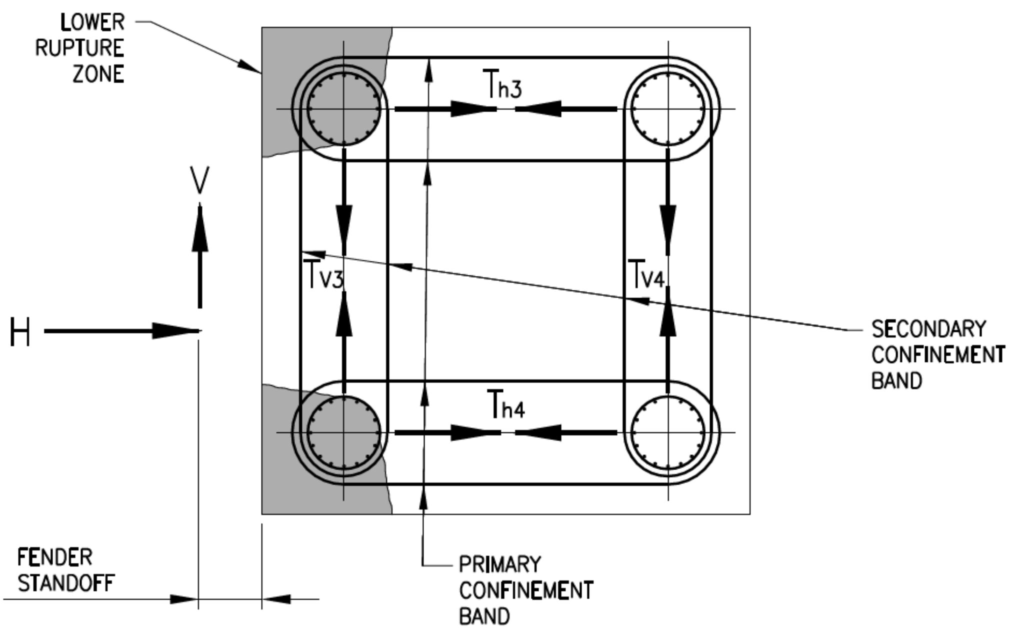


Figure 2B : Bottom confinement band reinforcement

Figure 2 : Pile cap plan confinement band reinforcement

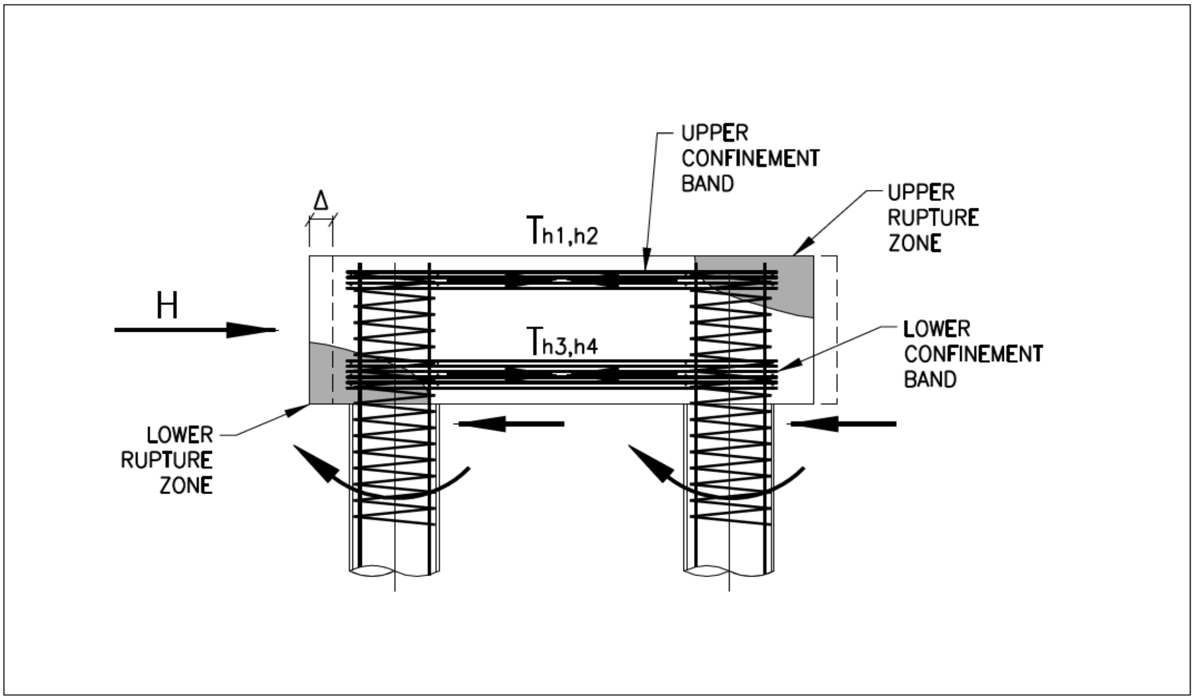


Figure 3

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:

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

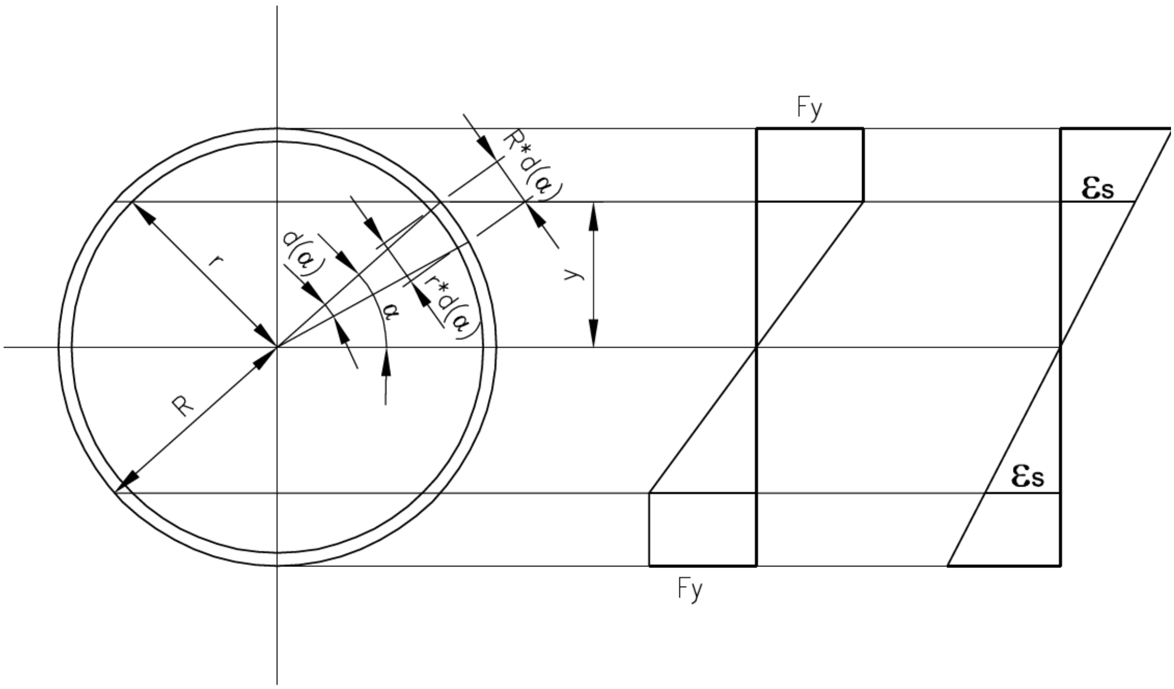


Figure 4 : Pipe section plastification

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

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

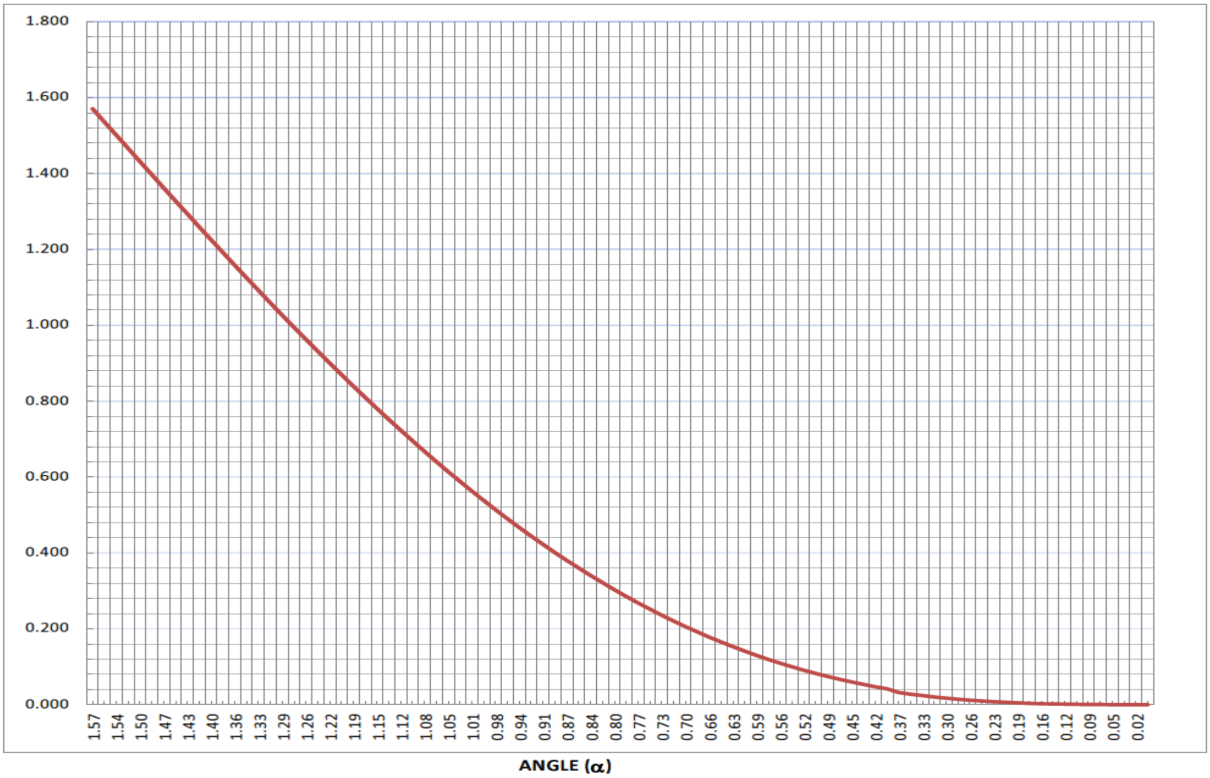
Step 11. Total moment capacity of the section is described by Formula 17

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

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:



$I_{eff} = 1/4 * (R+r)^3 * t * [0.5 * \alpha - 0.25 * \sin2(\alpha)]$

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).

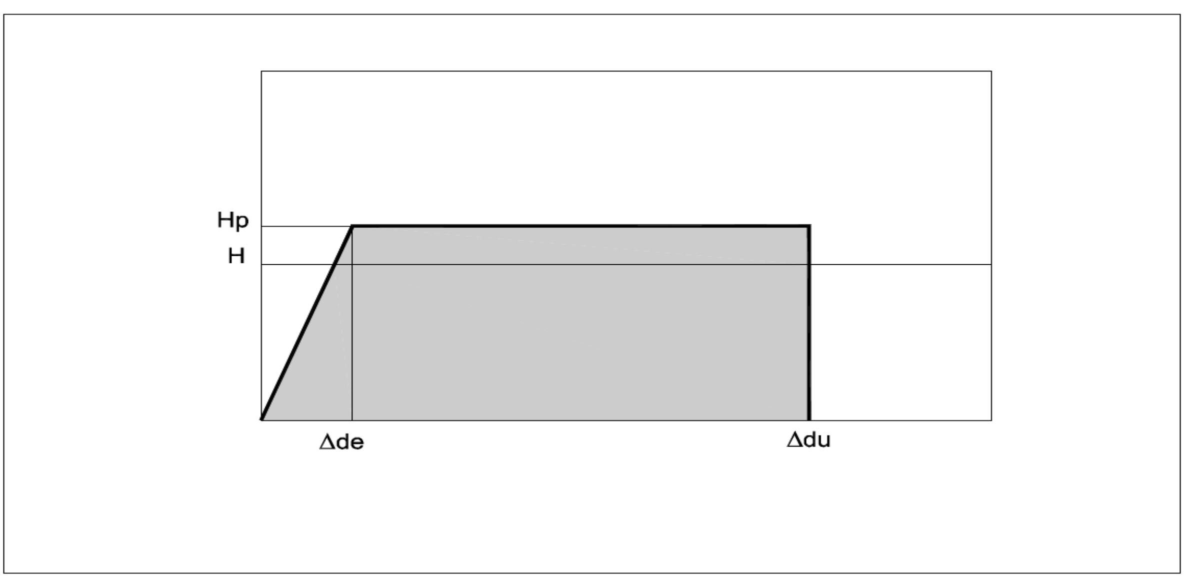


Figure 6 : Force vs. Deflection

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

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

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

$H_p / H = 2\mu_D / (2\mu_D - 1)$ (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* (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,

$Pt = M_T*d_x / I_p = 2{,}808*3/72 = 117kN$ - 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,* L_C= 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)

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

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

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

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,

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

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

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

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

Elastic Section Modulus

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

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

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

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

$M_{el\text{-}pl}$ = 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,

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

Elastic deflection experienced by gross section of dolphin,

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

and

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

Residual plastic deformation of the dolphin,

$$\boxed{\Delta_{res} = \Delta_{I\,eff} - \Delta_{I\,gross} = 0.56 - 0.38 = 0.18\text{m}}$$

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.

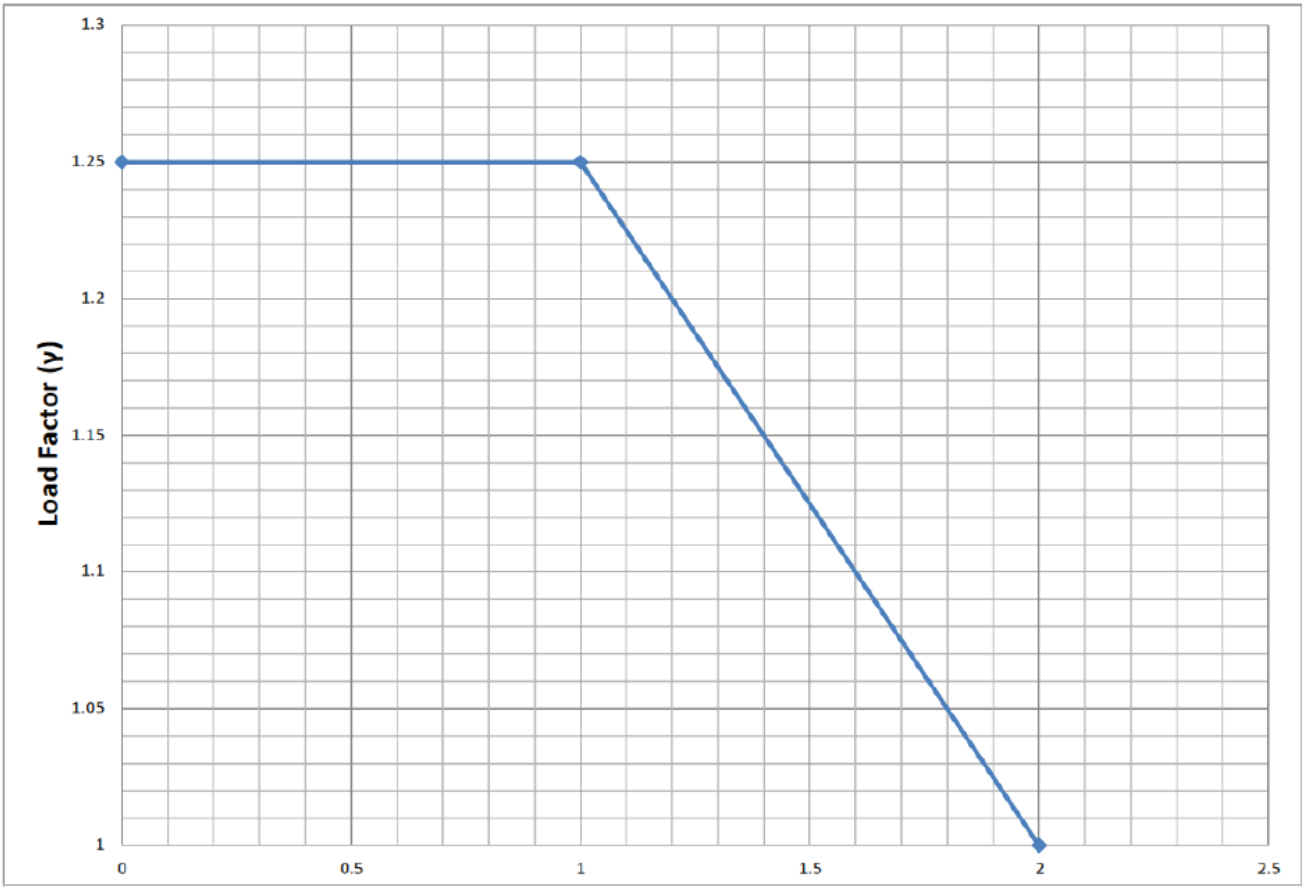


Figure 7 : Ratio of (Elasto-Plastic Deflection) / (Elastic Deflection)

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

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

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

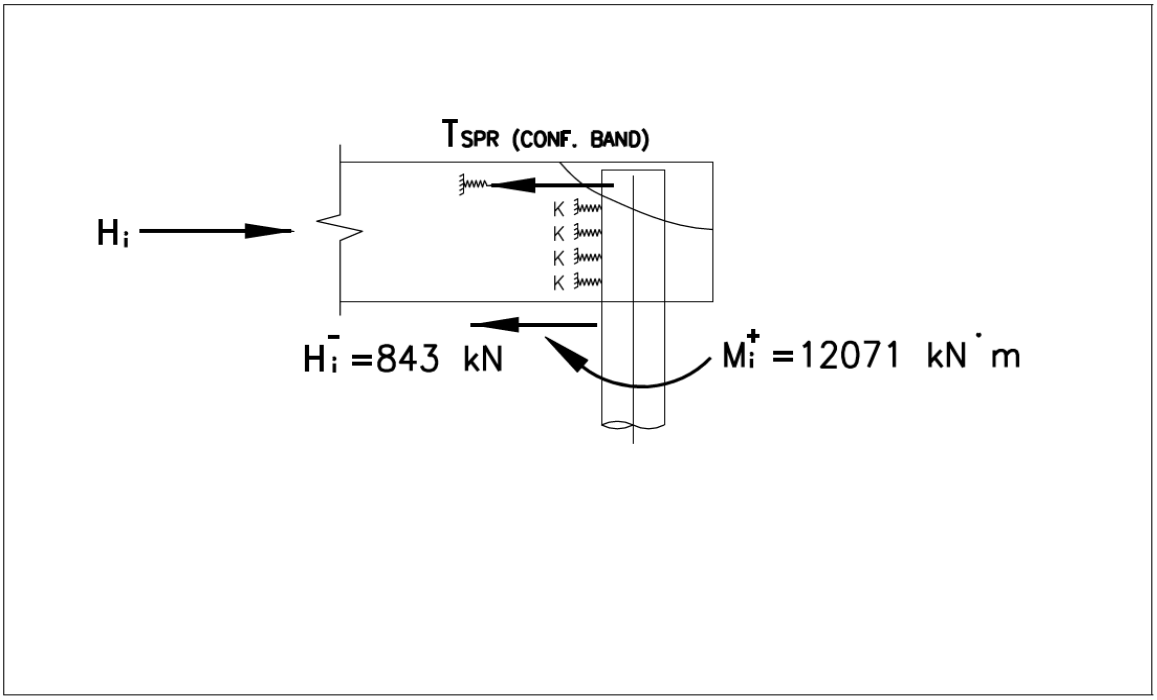


Figure 8 : Upper band confinement. free body diagram

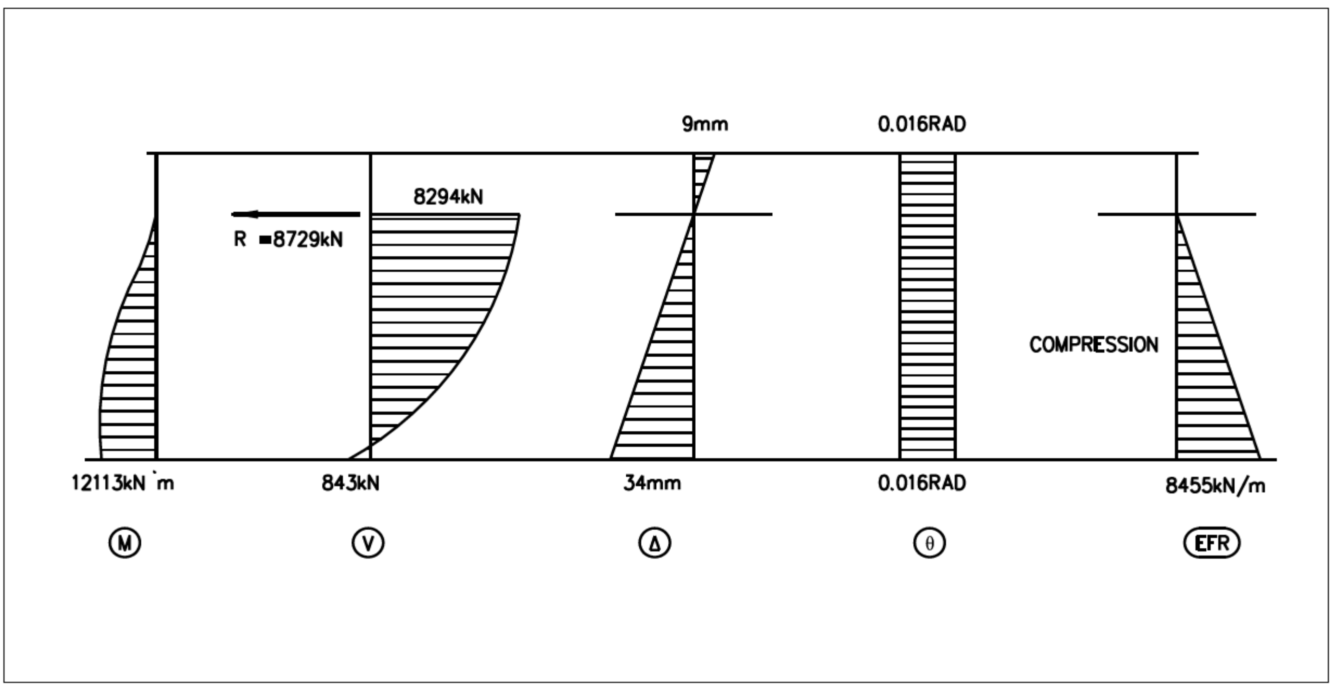


Figure 9 : Upper band boundary

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.

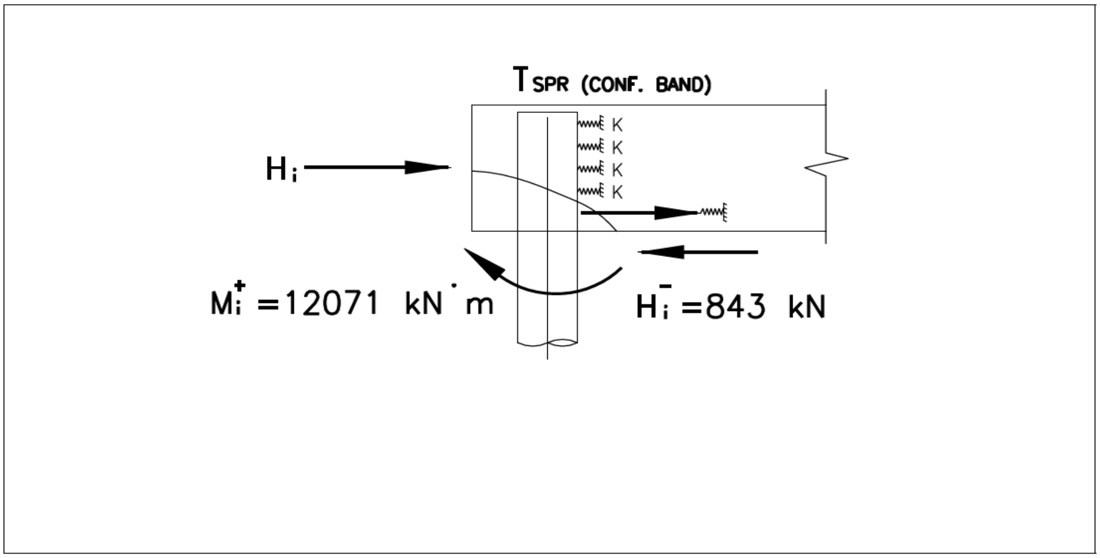


Figure 10 : Lower band confinement. free body diagram

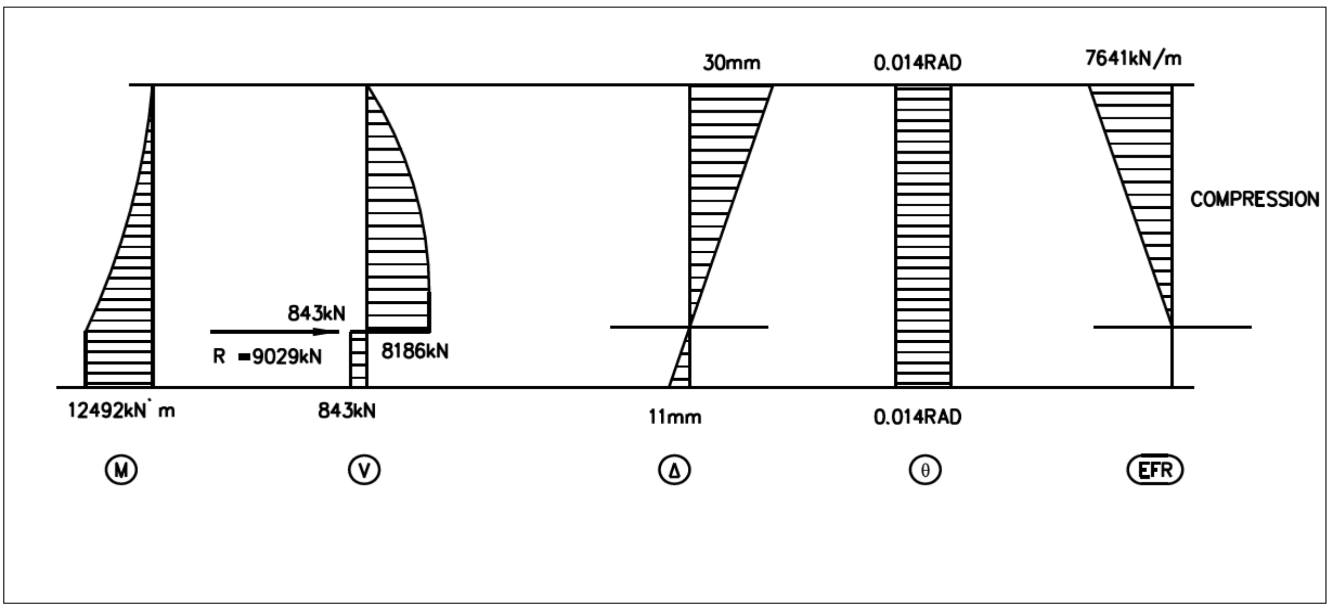


Figure 11 : Lower band boundary

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:

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

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.

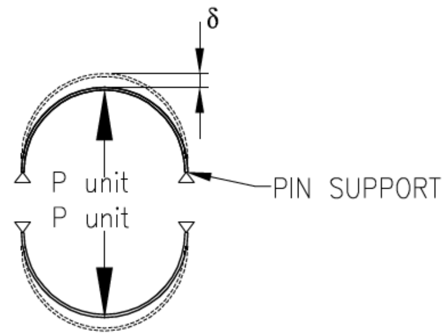


Figure 12 : Elastic Foundation Spring Analysis

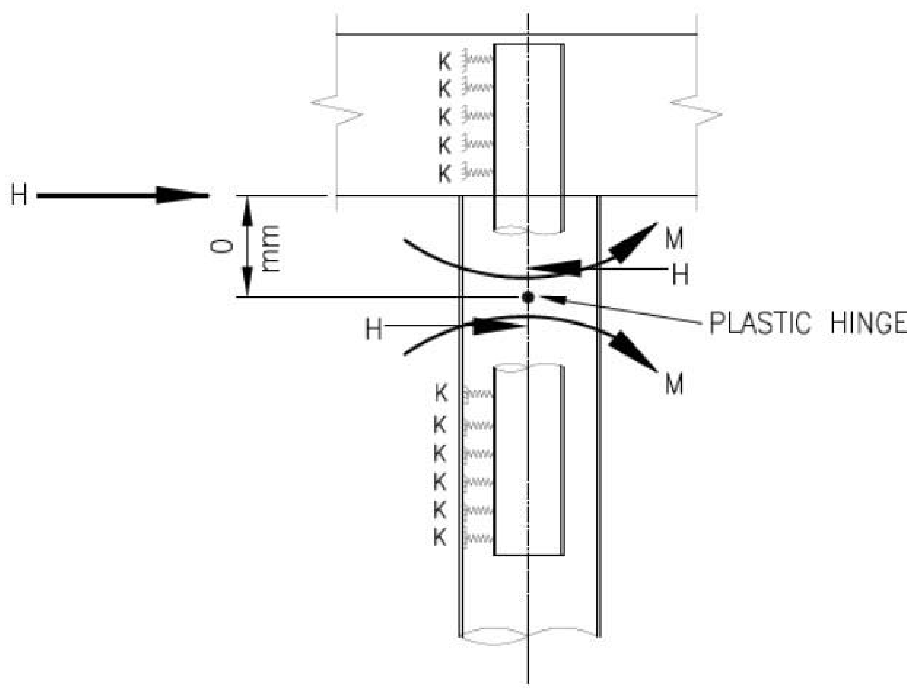


Figure 13 : Shear plug Elastic Foundation Model

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.

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Fascinating Improvement in Mechanical Properties of Cement Mortar using Multiwalled Carbon Nanotubes and Ferrite Nanoparticles

By M. A. Ahmed, Y. A. Hassanean, K. A. Assaf & M. A. Shawkey
Cairo University, Egypt

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Keywords: MWCNTs, $MnFe_2O_4$ nanoparticles, HRTEM, compressive strength, flexural strength.

GJRE-E Classification : FOR Code: 861099

FASCINATINGIMPROVEMENTINMECHANICALPROPERTIESOFCEMENTMORTARUSING MULTIWALLEDCARBONNANOTUBESANDFERRITENANOPARTICLES

Strictly as per the compliance and regulations of :

Fascinating Improvement in Mechanical Properties of Cement Mortar using Multiwalled Carbon Nanotubes and Ferrite Nanoparticles

M. A. Ahmed [α], Y. A. Hassanean [σ], K. A. Assaf [ρ] & M. A. Shawkey [ω]

Abstract- The Mn-Ferrite nanoparticles were prepared using citrate nitrate auto combustion method. The Multiwalled carbon nanotubes (MWCNTs) and $MnFe_2O_4$ nanoparticles were characterized by BET to measure the surface area. XRD data of $MnFe_2O_4$ nanoparticles clarified that the sample was formed in single phase spinel structure without any extra peaks indicating any secondary phase. The High-resolution transmission electron microscopy (HRTEM) micrograph of $MnFe_2O_4$ nanoparticles indicated that the particles are in an agglomerated state due to the absence of surfactant and high magnetic properties of Mn-Ferrite nanoparticles. Also, HRTEM micrograph showed that the walls of MWCNTs are straight having high crystallinity without any kinks. The mechanical properties were measured at different ratios of MWCNTs and nano-ferrite to cement. The obtained values indicated that the addition of MWCNTs and nano-ferrite increase the compressive and flexural strength of cement mortar and decrease the total intrusion volume.

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I. Introduction

Concrete is one of the most prevalent materials on the ground and holds promises to be a cornerstone for our expansion in construction industry. More than 10 billion tons of it are produced every year for everything from major infrastructure projects like bridges, tunnels, dams, to homes, stadiums, and skyscrapers. However, cementitious materials in general, are very brittle and characterized by a very low tensile strength and strain capacity [1, 2]. The mechanical property of concrete arises from a phenomenon that occurs at the micro and nano scale i.e. interlinking of dendrites of calcium silicate hydrates during the hardening process. Nanoscale binders can modify the structure of concrete material and enhance its properties including bulk density, mechanical performance, volume stability, durability and sustainability of concrete [3].

Within the last few years, an increasing interest is in the application of nanoparticles in concrete, because nanoparticles due to its high specific surface area and high activity offers the opportunity to improve the mechanical properties of concrete and enhance the understanding of concrete behavior [4]. CNTs and ferrite nanoparticles are quickly becoming one of the most promising nanomaterials because of their unique mechanical properties.

The superior mechanical properties of the CNTs and ferrite nanoparticles alone don't ensure the improvement of mechanical properties of cement. The properties of the concrete composite are strongly influenced by two major factors. The first is the dispersion of these nanomaterials within the cementitious matrix. The other is the bond strength and energy between the matrix and surface of the CNTs or ferrite nanoparticles [1].

Several researches have been done on the partial replacement of cement with supplementary nanomaterials to improve their mechanical properties. The most of these researches are focusing on using SiO_2 [5] nanoparticles and CNTs [1]. There are a few studies on incorporating of different nanoparticles such as Fe_2O_3 [6], Al_2O_3 [7], $CaCO_3$ [8], TiO_2 [9], ZnO_2 [10], ZrO_2 [11] and CuO [12].

Sulapha Peethamparan et al. [5] discussed the effects of nano-silica (NS) on setting time and early strengths of high volume slag mortar and concrete. He used a constant water-to-cementitious materials ratio (w/cm) 0.45 for all mixtures. He found that compressive strength of the slag mortars increased with the increase in NS dosages from 0.5% to 2.0% by mass of cementitious materials at various ages up to 91 days.

M. Razzaghi et al. [13] added Nano-ZrO_2 (NZ), Nano-Fe_3O_4 (NF), Nano TiO_2 (NT) and Nano-Al_2O_3 (NA) to concrete mixtures to investigate its mechanical properties and durability. Results of this study showed that nanoparticles can be very effective in improvement of both mechanical properties and durability of concrete. The results indicated that the Nano-Al_2O_3 is most effective nanoparticle of examined nanomaterials in improvement of mechanical properties of high performance concrete.

Zachary Grasley et al. [1] used carbon nanotubes and carbon nanofibers for enhancing the mechanical properties of cementitious materials. He added untreated CNTs and CNFs to cement matrix composites in concentrations of 0.1% and 0.2% by weight of cement. The flexural test was performed to

Author α ω: Materials Science lab. (1), physics Department, Faculty of science, Cairo University, Giza, Egypt. e-mail: moala47@hotmail.com
Author σ ρ: Department of civil engineering, Assiut University, Egypt.

record its mechanical properties at 7, 14, and 28 days. SEM images verified poor dispersion within the cement paste matrix, the bridging effects, which transferred the load across the nano and microcracks, and the fibers pull out because of their weak bond. For all cases, the addition of CNFs and CNTs improved flexural strength of the cement paste compared to the control sample.

The aim of this study is to find the optimized percentage of adding MWCNTs and $MnFe_2O_4$ nanoparticles to achieve the highest values of compressive and flexural strength of cement mortar.

II. Materials and Methods

a) Materials and Mixtures

i. *Cement*

Ordinary Portland Cement (OPC) grade (CEM I 52.5N) obtained from AL-Areash Cement Manufacturing Company of Egypt conforming to the British standard BS 12/1996 [14] was used as received. The chemical properties of the cement are obtained from Pnalytical Axios Advanced X-ray fluorescence (XRF) and the results are reported in Table (1).

Table 1 : Chemical and physical properties of Portland cement (wt %)

Al_2O_3	SiO_2	CaO	TiO_2	Na_2O	MgO	SO_3	K_2O	Fe_2O_3	L.O.I
4.46	15.15	66.89	0.37	0.58	0.58	4.02	0.22	4.49	3.24

ii. *MWCNTs and $MnFe_2O_4$ nanoparticles*

MWCNTs was used as received from Yurui (Shanghai) chemical Co., Ltd. with diameter 20-55 nm and average surface area was 98.31 m^2/g. The properties of MWCNTs are shown in Table (2). High-resolution transmission electron microscopy (HRTEM) are shown in Fig. (1).

$MnFe_2O_4$ nanoparticles with average diameter of 49 nm and average surface area of 27.28 m2/g was prepared by citrate nitrate auto combustion method at materials science lab. (1) [15, 16]. The properties of $MnFe_2O_4$ nanoparticles are also shown in Table (2). High-resolution transmission electron microscopy (HRTEM) and powder X-ray diffraction (XRD) diagrams of $MnFe_2O_4$ nanoparticles are shown in Figs. (2) and (3), respectively.

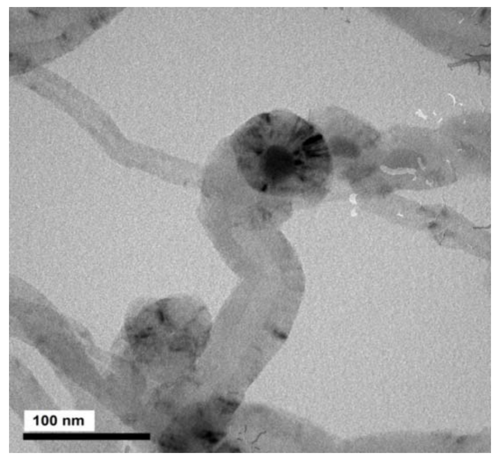


Fig. 1 : HRTEM micrograph of MWCNTs

Table 2 : Properties of MWCNTs and $MnFe_2O_4$ nanoparticles

	Average diameter/nm	Average surface area /(m^2/g)	Average volume /(cc/g)	Purity/%
MWCNTs	20-55	98.31	0.0494	97
$MnFe_2O_4$	49	27.28	0.0134	98-99

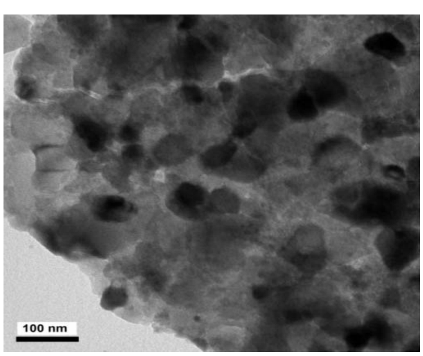


Fig. 2 : HRTEM micrograph of $MnFe_2O_4$ nanoparticles

iii. *Aggregates*

Coarse sand (0.5-2 mm) was used to produce cement mortar.

iv. *Superplasticizer*

Sika ViscoCrete® -5930 is an aqueous solution of modified polycarboxylate was used. Table (3) reports some of the physical and chemical properties of polycarboxylate admixture used in this study.

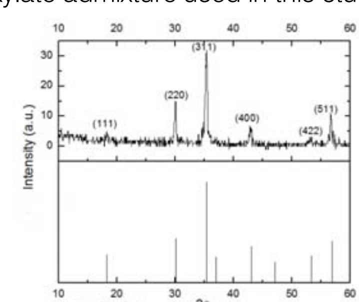


Fig. 3 : XRD analysis of $MnFe_2O_4$ nanoparticles

Table 3 : Physical and chemical characteristics of the superplacticizer admixture

Appearance	Colour	Specific gravity /(kg/L)	Na+Ppm	Ca+Ppm
Turbid liquid	Yellow-brown	1.08±0.005	18380	4.72

v. *Mixture Proportioning*

Nine Mixtures of cement mortar were prepared in the laboratory trials. These Mixtures included a reference sample of plain cement mortar, three mixutures of cement mortar with MWCNTs at 0.3 wt%, 0.5 wt% and 0.7wt% by weight of dry cement, three mixutures of cement mortar with $MnFe_2O_4$ nanoparticles at 0.3 wt%, 0.5 wt% and 0.7wt% and two mixutures of cement mortar with both MWCNTs and $MnFe_2O_4$ nanoparticles at 0.15 wt%, 0.3 wt% for each of them. Table (4) summarizes the composition of the nine mixtures.

The superplasticizer was dissolved in water, and then MWCNTs and $MnFe_2O_4$ nanoparticles were added and good stirred at a high speed for 2 min. The binder content of all mixtures was 635 kg/m^3. The total mixing time including homogenizing was 5 minutes.

b) Strength Evaluation Tests

Cubic Specimens with 50 mm edge length were used for compressive tests and prism specimens with dimensions 40 x 40 x 160 mm were used for flexural tests. The moulds were covered with polyethylene sheets and moistened for 24h. Then, the specimens were demoulded and cured in water at room temperature prior to test days [6] .The strength tests of the samples were determined after 2 and 14 days of curing. The tests were carried out triplicately and average strength values were obtained.

Table 4 : Mix Proportion of samples

Sample name wt%	MWCNTs wt%	$MnFe_2O_4$ nanoparticles	Quantities/(kg/m^3)					
			Water	SP	Sand	cement	MWCNTs	$Mnfe_2o_4$ nanoparticles
C0	0	0	238	11.7	1586	635.0	0	0
N 1-1	0.30	0	238	11.7	1586	633.1	1.90	0
N 1-2	0.50	0	238	11.7	1586	631.8	3.20	0
N 1-3	0.70	0	238	11.7	1586	630.5	4.50	0
N 2-1	0	0.30	238	11.7	1586	633.1	0	1.90
N 2-2	0	0.50	238	11.7	1586	631.8	0	3.20
N 2-3	0	0.70	238	11.7	1586	630.5	0	4.50
N 3-1	0.15	0.15	238	11.7	1586	633.1	0.95	0.95
N 3-2	0.30	0.30	238	11.7	1586	631.2	1.90	1.90

c) Mercury Intrusion Porosimetry (MIP)

MIP Poresizer 9320 V2.08 was used to characterize the pore structure in porous material as a result of its simplicity, quickness and wide measuring range of pore diameter [17, 18]. MIP gives us details about the dimensions of pores [17]. To prepare the samples for MIP measurement, the concrete specimens after 14 days of curing were first broken into smaller pieces, and then the cement paste fragments selected from the center of prisms were used to measure pore structure. The samples were immersed in acetone to stop hydration as fast as possible. Before mercury intrusion test, the samples were dried in an oven at about 110° C until constant weight is obtained by removing moisture in the pores. MIP is based on the assumption that the non wetting liquid mercury (the contact angle between mercury and solid is greater than 90°) will only intrude in the pores of porous material under pressure [17, 18]. Each pore size is quantitatively determined from the relationship between the volume of intruded mercury and the applied pressure [18]. The test apparatus used for pore structure measurement is Auto Pore III mercury porosimeter. The surface tension of mercury is taken as 485*10-5 N/cm (485 dyne/cm), and the contact angle selected is 130 deg. The maximum head pressure applied is (4.68 psi).

d) Field emission scanning electron microscope (FE-SEM)

After the samples had been tested, the fracture surface was cut into an approximately 1 × 1 × 0.5 mm. Then, a field emission scanning electron microscope (FE-SEM) (JSM-7500F, JEOL, Tokyo) was used to observe the fracture surface of the samples.

III. Results and Discussion

Figs. (4-7) show compressive and flexural strength of cement mortar specimens after 2 and 14 days of curing, respectively. The results show that the compressive and flexural strength increases by addition of MWCNTs content till 0.7 wt % replacements to cement mortar. This was due to the interfacial interactions between MWCNTs and cement hydrates to bridge nanocracks and pores to achieve good bonding with the cement hydration products.

On the other hand, by the addition of $MnFe_2O_4$ nanoparticles with 0.5 wt%, the compressive and flexural strength increase after which it decreases. The reasons that allow $MnFe_2O_4$ nanoparticles to increase the strength of concrete can be explaind as follows. The addition of $MnFe_2O_4$ nanoparticles reduces the quantity and size of $Ca(OH)_2$ crystals and fills the voids of Calcium Silicate Hydrate (C-S-H) gel structure. This make the structure of hydrated products denser and compact [12]. Increasing $MnFe_2O_4$ nanoparticles more than 0.5 wt%, the compressive strength reduces. This matter is because nanoparticles due to their high surface energy have the tendency towards agglomeration. When $MnFe_2O_4$ nanoparticles are over added to the concrete, it is not uniformly distributed in cement paste and due to agglomeration, weak zone appears in the concrete specimen. The highest values of compressive and flexural strength achieved by the addition of both MWCNTs and $MnFe_2O_4$ nanoparticles by 0.3 wt% for each of them with enhancement by 19 % for compressive strength and by 21% for flexural strength compared to the control specimen. This is due to the ability of $MnFe_2O_4$ nanoparticles to fill the voids at the nanoscale and MWCNTs to act as bridges across voids and cracks that ensure more compact and durable cement mixture.

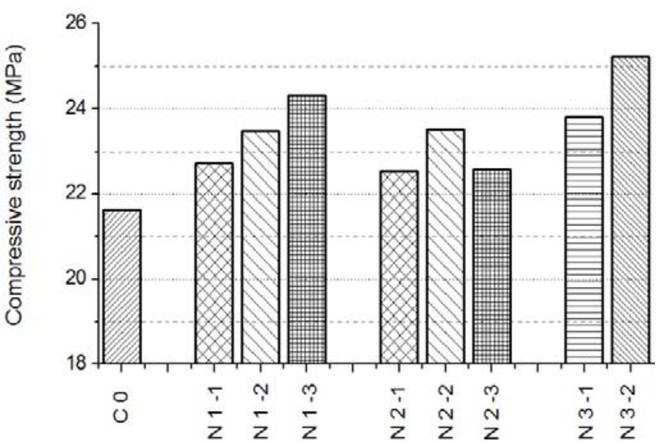


Fig. 4 : Compressive strength of specimens after 2 days of curing

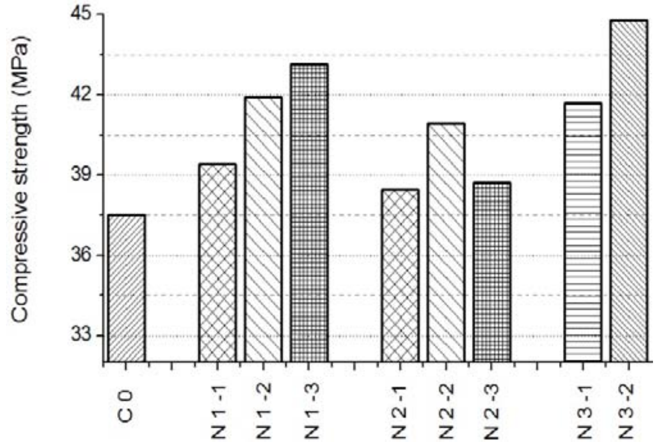


Fig. 5 : Compressive strength of specimens after 14 days of curing

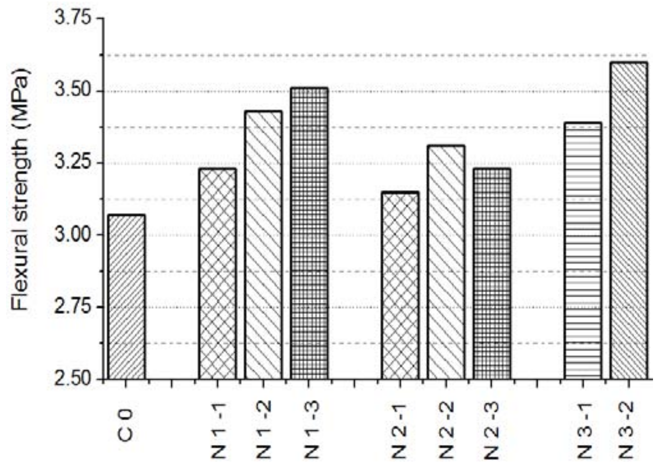


Fig. 6 : Flexural strength of specimens after 2 days of curing

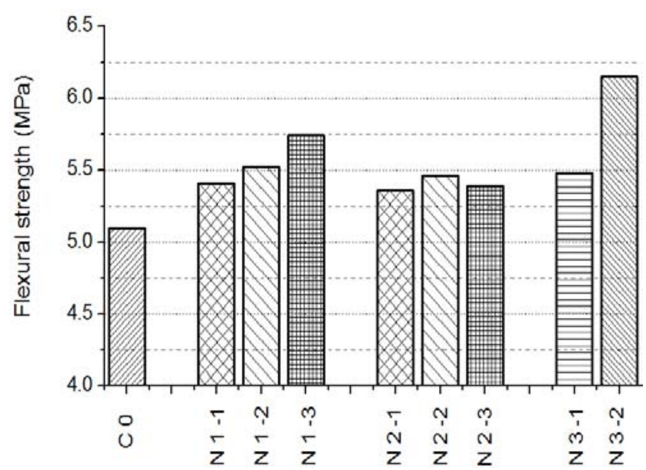


Fig. 7 : Flexural strength of specimens after 14 days of curing

The mercury intrusion results of the C0 specimen and N3-2 specimen are shown in Figs. (8, 9). Fig. (8) represents the variation of incremental intrusion, reflecting pore volume against pore diameter, which indicates that most pore diameter of the specimen are distributed between 0.1 micrometer to 1 micrometer. Fig. (9) represents the cumulative intrusion, reflecting the total connected pore volume of pore sizes. Table (5) shows that by the addition of both MWCNTs and $MnFe_2O_4$ nanoparticles by 0.3 wt% for each of them, total intrusion volume of specimens are decreased. This leads to decreasing total pore area and median pore diameter of cement mortar (area), but median pore diameter (volume) of these specimens is increased. On the other hand, Table (6) shows that the addition of MWCNTs and $MnFe_2O_4$ nanoparticles leads to decreasing the porosity, increase the average pore diameter and decreasing the bulk density and the apparent (skeletal) density of these specimens of cement mortar. This means that the regularity of porosity is similar to that of total intrusion volume and the regularity of average pore diameter is similar to median diameter (volume). The increase of average pore diameter and median diameter (volume) are due to the ability of MWCNTs and $MnFe_2O_4$ nanoparticles to fill the small pores. The decrease of density is due to the replacement of cement by MWCNTs and $MnFe_2O_4$ nanoparticles which have a lower density leading to a decrease in the density of the composite.

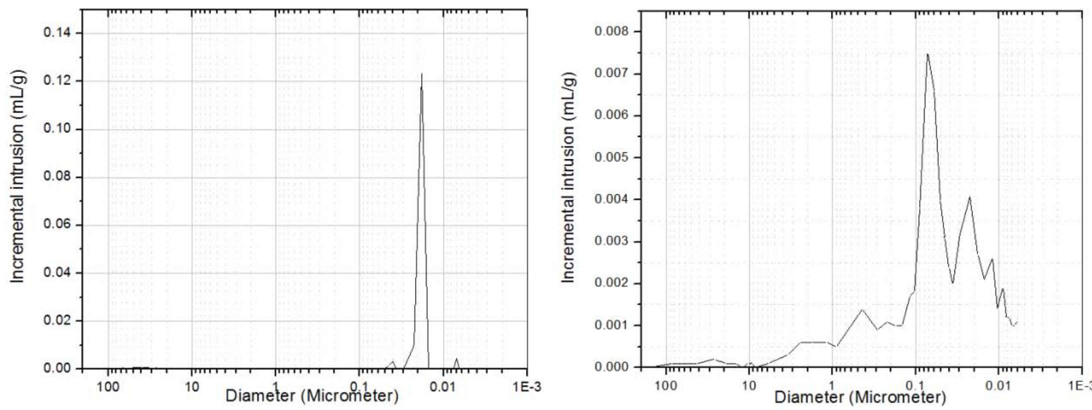


Fig. 8 : Incremental intrusion versus diameter for specimens of concrete (left: C0, right: N3-2)

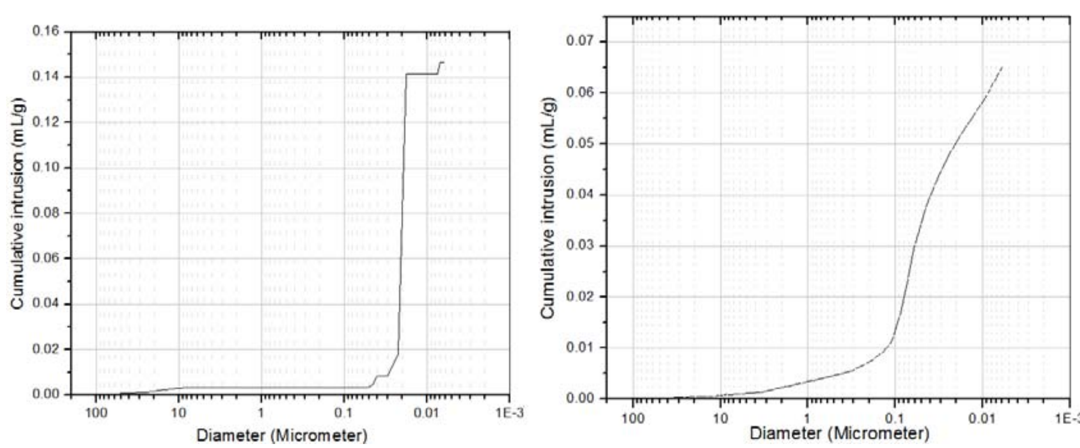


Fig. 9 : Cumulative intrusion versus diameter for specimens of concrete (left: C0, right: N3-2)

Table 5 : Total intrusion volume, total pore area, median pore diameter (volume) and median pore diameter (area) of C0 and N 3-2 specimens

Sample name	Total intrusion volume /(mL/g)	Total pore area /(m^2/g)	Median pore diameter (volume)/nm	Median pore diameter (area)/nm
C0	0.146	28.91	22.6	22.6
N 3-2	0.065	9.15	55.3	13.2

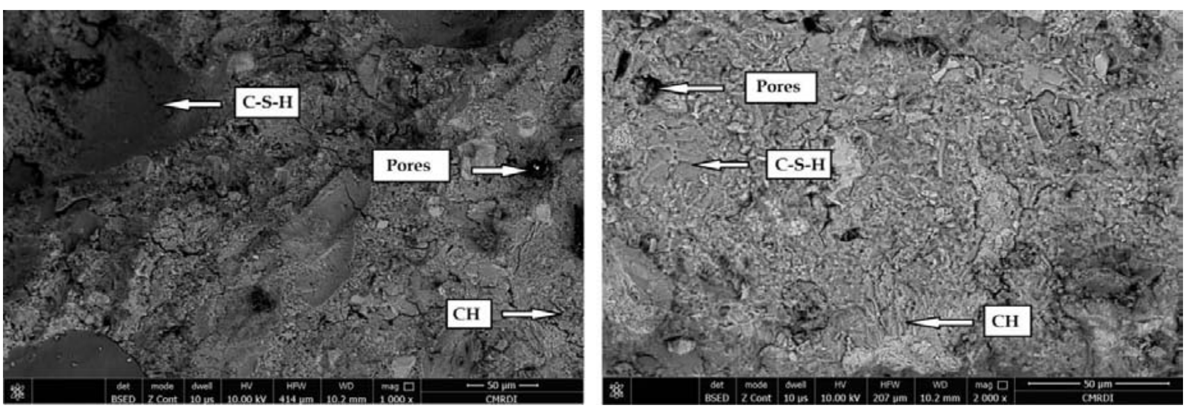
Table 6 : Average pore diameter, bulk density, apparent (skeletal) density and Porosity of C0 and N1 3-2 specimens

Sample name	Average pore diameter/nm	Bulk density /(g/mL)	Apparent (skeletal) density/(g/mL)	porosity/%
C0	20.3	2.48	3.89	36.32
N 3-2	28.4	2.17	2.53	16.00

Figs. (10, 11) present FE-SEM photographs of the cement mortar of C0 specimen and N3-2 specimen after 14 days of curing. The results confirmed an improved microstructure in the cement mortar with MWCNTs and $MnFe_2O_4$ nanoparticles addition. In the control specimen showed in Fig. (10a, 11a), the microstructures were non-compact, with extensive presence of large crystals of calcium hydroxide. However the voids among cement particles have been occupied by the hydration products, many connected capillary pores were observed.

The cement mortar specimen with MWCNTs and $MnFe_2O_4$ nanoparticles addition showed denser formations of hydration products than the control specimen as showed in Fig. (10b, 11b). It is obvious that, regardless of the presence of many pores, the density is significantly improved and the volume of pores reduced due to the ability of $MnFe_2O_4$ nanoparticles to fill the pores. This leads to improving impermeability thus the durability and the microstructure of the hardened cement-based materials [19]. The calcium hydroxide was appeared as ill-crystals [20] as shown in Fig. (10, 11). The pozzolanic reaction between $MnFe_2O_4$ nanoparticles and calcium hydroxide liberated during hydration produced additional C-S-H gel resulting in significant improvement in mechanical properties of blended mortar.

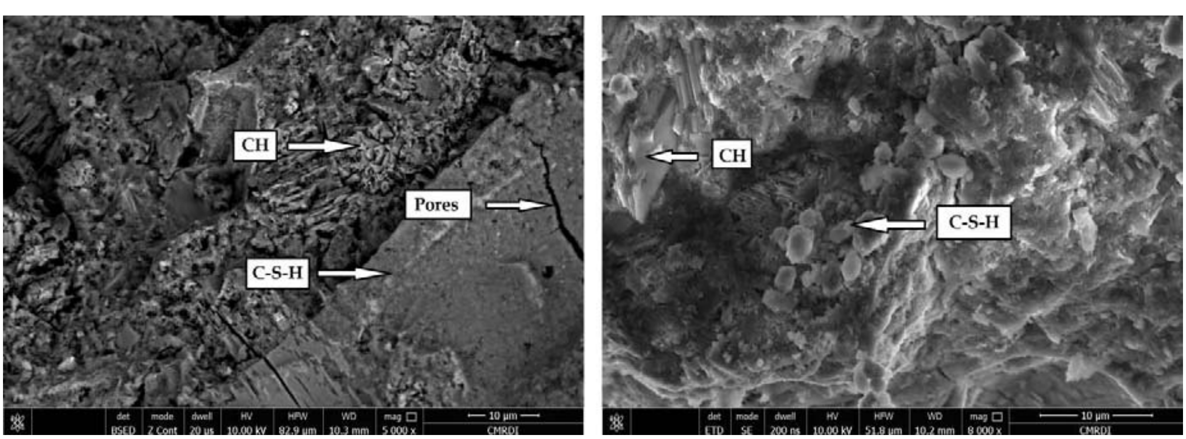
In addition, the microscopic observation also reveals that the MWCNTs were covered by C-S-H. The MWCNTs were found embedded as individual fibers in the paste and acting as bridges between hydrates and across cracks [20].



a) 50 μm

b) 50 μm

Fig. 10 : SEM micrograph of specimens of concrete after 14 days of curing (left: C0, right: N3-2)



a) 10 μm

b) 10 μm

Fig. 11 : SEM micrograph of specimens of concrete after 14 days of curing (left: C0, right: N3-2)

IV. Conclusions

The obtained results can be summarized as follows.

- The results showed that cement specimen reinforced with both MWCNTs and $MnFe_2O_4$ nanoparticles after 7 and 28 days of curing have higher compressive and flexural strength compared to the control specimen. MWCNTs and $MnFe_2O_4$ nanoparticles accelerate consumption of crystalline $Ca(OH)_2$ which quickly are formed into C-S-H during hydration of cement specially at early ages due to the high reactivity of these nanoparticles.
- The pore structure of cement mortar containing both MWCNTs and $MnFe_2O_4$ nanoparticles with 0.3 wt. % was improved and the volume of all mesopores and macropores was decreased.
- FE-SEM images showed that specimen reinforced with both MWCNTs and $MnFe_2O_4$ nanoparticles with 0.3 wt. % is more compact and less porous in the paste with admixture than the control one.

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Effect of pH on Shear Strength Behavior of Granular Soil

By Md. Motiur Rahman & Tahmina Tasnim Nahar
Pabna University of Science & Technology (PUST), Bangladesh

Abstract- In this research, the performances of pH value on shear strength behavior of granular soil have been studied. The shear strength of soil is an important term in most of the foundation engineering problems such as the bearing capacity of shallow foundation, slope stability of dam/embankment and lateral earth pressure on retaining walls. A series of direct shear test were conducted on two types of dry granular soils (taken from Rangpur and Rajshahi areas of Bangladesh) with different pH value (pH=0, pH=3.0, pH=5.0, pH=7.0 and pH=9.0). Hydrochloric acid (HCl) and ammonia (NH4) solution were used to monitor the pH of the solution for about thirty days. In all, 15 specimens of each type of soils were considered for direct shear test with dry condition at a constant density. The specimens were prepared by static compaction with different pH values solution (0, 3, 5, 7and 9) at same void ratio. Experiment result shows that the shear strength increase with increase of pH values of soil.

Keywords: pH values, shear strength, granular soil, hydrochloric acid, ammonia, void ratio.

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EFFECTOFPHONSHEARSTRENGTHBEHAVIOROFGRANULARSOIL

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Effect of pH on Shear Strength Behavior of Granular Soil

Md. Motiur Rahman [α] & Tahmina Tasnim Nahar [σ]

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Keywords: pH values, shear strength, granular soil, hydrochloric acid, ammonia, void ratio.

I. Introduction

All of the civil engineering structures involve some structural element with direct contact to the soil. The stability of this structures are mainly depends on the stability/strength of contact soil. Granular soil is one of the commonest materials that are widely used in the construction of civil engineering infrastructures, such as earth dams/embankments, roads and so on. The shear strength behavior of granular soil is complexive when they loaded [1]. The variations of the behavior mainly depend on the discrete nature of the particles like shape, size, surface texture, particles distribution and also depend on pH value which was studied in this paper. pH value of soil is decreasing day by day by acid rain, industrial residue, fertilizer, insecticides etc. However there is limited available information in existing literature on shear strength behavior of granular soil for different pH values soil. Considerable researches have been carried out on this purpose (Olukorede M. Osuolale, Olumide D. Falola and Mojeed A. Ayoola (2012) on Effect of pH on Geotechnical Properties of Laterite Soil Used in Highway Pavement Construction) [9], (Rahanuma Tajnin, Tabassum Abdullah, Md. Rokonuzzaman (2014) on Study on the salinity and pH and its effect on geotechnical properties of soil in south-west region of Bangladesh) [10] here shear strength is not considered as important properties of soil but in this paper only shear strength properties was investigated as an important factor on which pH affect.

II. Materials and Equipment

The two types of sandy soil used in this study which address as S-1 (S-1 soil sample has been collected from Tista river, Rangpur of Bangladesh which is locally called domar sand) and S-2 (S-2 soil sample has been collected from Padma river, Rajshahi of Bangladesh which is locally called local sand). Hydrochloric acid (HCl) and ammonia (NH4) solution were used to monitor the pH. The basic properties of two samples are presented in Table 1.

The basic equipments which are used in this study are: (i) Direct shear test device, (ii) Load and deformation dial gauge and (iii) Balance, (iv) pH meter etc.

Table 1 : Basic properties of soil sample S-1 and S-2 (Before contamination)

Basic Properties	Obtained Value	
	S-1	S-2
Grain Size Distribution:	0.33	0.22
Effective size, D_{10} (mm)	0.40	0.26
Diameter corresponding to 30% finer, D_{30} (mm)	0.60	0.35
Diameter corresponding to 60% finer, D_{60} (mm)	1.82	1.59
Uniformity co-efficient, $_{Cu}$	3.10	2.50
Fineness Modulus, FM		
Specific Gravity	2.64	2.61
Compaction:	1.63	2.59
Maximum dry density, $\rho_{d(max)}$ (gm/cm^3)	15.10	15.19
Optimum moisture content, OMC (%)	0.64	0.65
Void ratio, e		

Author α σ: Lecturer, Department of Civil Engineering, Pabna University of Science & Technology (PUST), Pabna, Bangladesh. e-mail: motiur@engineer.com

III. LABORATORY TESTING

a) Direct Shear Test Program and Procedure

A series of 30 direct shear tests carried out on 2 soil samples referred to as S-1 S-2 (15 tests for each sample). The basic properties of sample specimens were presented in Table 1. Each soil sample (S-1 and S-2) was divided into five portions. Each portion of the soil sample was stored in the big perforated plastic containers labeled A, B, C, D and E. The containers were perforated at the bottom so that the water can drain slowly in order to simulate the actual field condition. The Hydrochloric acid (HCl) and ammonia (NH4) were used to prepare solution that has pH of 3, 5, 7 and 9. The container that was labeled A is uncontaminated while the solutions with pH of 3, 5, 7 and 9 were poured into containers labeled B, C, D and E respectively. The five containers with its contents were then stored for about 30 days in the laboratory. After 30 days the samples were air dried and direct shear tests (3 samples from each container to determining average value) were carried out on them at same density and void ratio. To carry out these tests, a sample of soil is placed into the shear box. The size of the box is used 60 mm diameter and the sample is 33 mm thick. The soil is placed into the box by trimming 3 equal layers which gives void ratio 0.64 for S-1 and 0.65 for S-2. After the specimen is placed in the box, and all the other necessary adjustments are made, a known normal stress σ is to be applied (1.42 psi). Then a shearing force is applied. The normal load is kept constant throughout the test but the shearing force is applied at a constant rate of strain. The shearing displacement is recorded by a dial gauge. The procedure is repeated five times at different normal stresses (2.84, 7.11, 14.23 and 21.34psi) for each time. These results are plotted on a shearing diagram where σ (normal stress) is the abscissa and τ (shearing stress) the ordinate. The slope of the line gives the angle of internal friction (ϕ°) and the intercept on the ordinate gives the apparent cohesion (c psi). The shear strength is determined by using $\tau = c + \sigma \tan(\varphi)$.

IV. RESULTS AND DISCUSSIONS

a) Presentation of Test Result and Discussion

All the specimens were tested under dry condition. The results of the shear strength are presented in Table 2 below for each sample. The shear strength increases with increase in pH value. Figure 1 and 2 represents the shear strength versus pH values relationship and from this figure it is investigated that the shear strength increases with increase in pH value.

Table 2 : Shear strength variation chart of both sample

Sample		Shear strength (psi)				
		Con. A (pH=0)	Con. B (pH=3)	Con. C (pH=5)	Con. D (pH=7)	Con. E (pH=9)
S-1	1	2.95	3.09	3.15	3.48	3.60
	2	2.97	3.04	3.13	3.60	3.65
	3	3.01	3.08	3.17	3.56	3.59
	Average	2.98	3.07	3.15	3.55	3.61
S-2	1	2.79	2.84	2.99	2.99	3.25
	2	2.75	2.77	2.94	3.08	3.52
	3	2.74	2.82	2.86	2.92	3.45
	Average	2.76	2.81	2.93	3.00	3.41

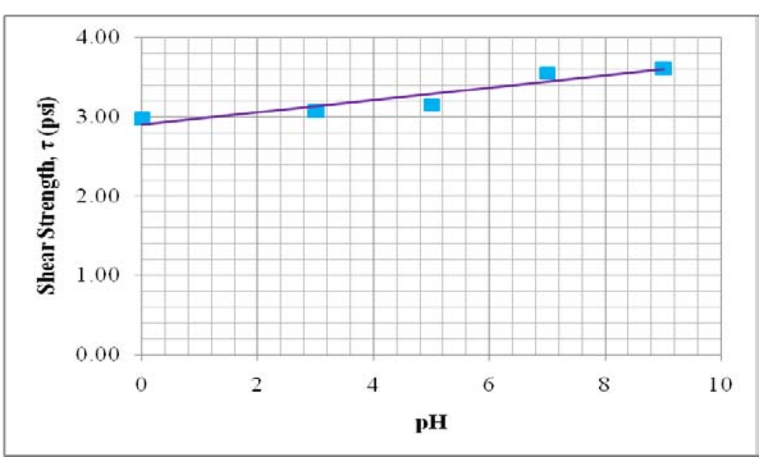

Figure 1 : Shear strength Vs. pH values of S -1

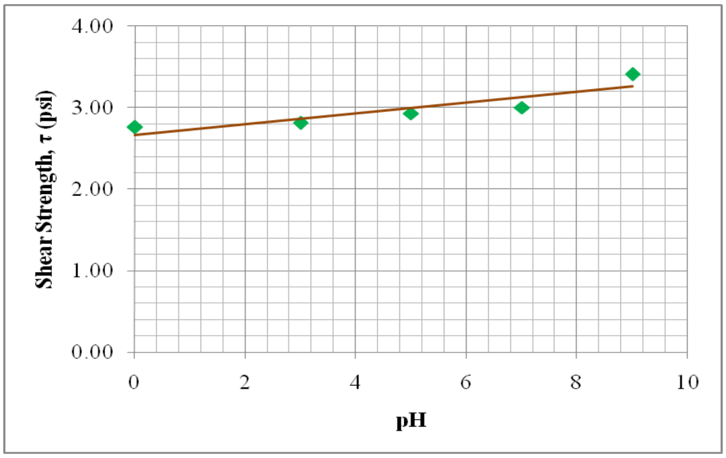

Figure 2 : Shear strength Vs. pH values of S -2

V. Conclusions and Recommendation

On the basis of literature test carried out following concluding remarks are made:

a) Shear strength of granular soil is increase with increase of pH value. So if we want to increase shear strength of acidic soil we have to increase pH value. The most common amendment to increase soil pH is lime ($CaCO_3$ or $MgCO_3$).
b) Farther investigation is required for others type of soil.

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FEM Analysis of Integral Abutment Bridges with Fixed and Pinned Pile Head Connections

By Yamuna Bhagwat, R.V. Raikar & Nikhil Jambhale
MBE's College of Engineering, India

Abstract- The comparative study on the effect of pile head connection with abutment on integral abutment bridges is presented in this paper. The influence on the design parameters such as bending moment, shear force and longitudinal stresses in deck slab has been considered. The study demonstrates that the design parameters are affected by the pile head to abutment connection. In addition, the results of DL (Dead Load) + temperature and DL + LL (Live Load) + temperature combination with varying span numbers have been compared with single span and with DL. Similarly the effect on interior and exterior girder has also been studied. In case of only DL, the negative maximum end Bending Moment (BM) reduced by 10.5% in the case of single span, 28.5% in two spans integral abutment bridge, while no change is observed in three spans of the integral abutment bridge. The positive BM, however, showed an increasing trend. An interesting outcome of the study is an inversely proportional relation between the number of spans and the design parameters. The increase in temperature tends to enhances negative BM and decreases positive BM. Furthermore, the SF in deck slab increased by 5.9% in two spans integral abutment bridge having pile head with pinned connection, however no change is observed in SF in single and three span configurations.

Keywords: *integral abutment bridge, pile head abutment connection, finite element method.*

GJRE-E Classification : FOR Code: 090599

FEMANALYSISOFINTEGRALABUTMENTBRIDGESWITHFIXEDANDPINNEDPILEHEADCONNECTIONS

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FEM Analysis of Integral Abutment Bridges with Fixed and Pinned Pile Head Connections

Yamuna Bhagwat [α], R.V. Raikar [σ] & Nikhil Jambhale [ρ]

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Keywords: integral abutment bridge, pile head abutment connection, finite element method.

I. Introduction

The Integral Abutment Bridges are bridges generally built with their superstructures integral with the abutments in the absence of expansion or contraction joints over the entire length of the superstructure. These are designed as single span or multi span and typically have stub-type abutments supported on piles and a continuous bridge deck from one embankment to the other. Although the small and flexible foundations facilitate horizontal movement or rocking of the support, the bridge structures react to the temperature changes and deform when subjected to the internally developed thermal stresses. The thermal effect is therefore an essential feature in the design of integral bridges and constitutes the biggest challenge in the analysis and design of the abutment.

*Author α: Asst. Professor, Department of Civil Engineering, MBE's College of Engineering, Ambajogai, Maharashtra, India.
e-mail: yamuna.bhagwat1@gmail.com*
Author σ: Professor, Department of Civil Engineering, KLEMSSCET Belgaum, Karnataka, India. e-mail: rvraikar@gmail.com
*Author ρ: Associate Professor, Department of Civil Engineering, KLEMSSCET Belgaum, Karnataka, India.
e-mail: nikhiljambhale@yahoo.co.in*

The rigid connection facilitates the integral bridges to act as a single unit in resisting thermal and brake loads. The integral design and construction eliminates joints in the bridge resulting in avoiding common issues related to bridges such as corrosion of reinforcements due to leakage of water and the use of de-icing chemicals through joints. Failure to proper response to unanticipated movements results in overstress and subsequent structural damage to the bridge elements via split and rupture of abutment bearings, abutment-rotation and abutment overturning. With an edge over many issues related to conventional bridge design and operation, the integral bridges are trending towards a definite change in the design of highway bridges. (Arsoy*et al.*, 1999; Manjunath and Bastwadkar, 2012; and Khodair and Hassiotis, 2013). David et al. (2010, 2014) found an increase in the performance of the integral bridge with short H-Piles and they reported that sufficient design is required in order to accommodate the effects due to thermal loading.

Khodair and Hassiotis (2013) studied the effect of temperature on integral bridges in conjunction with skew effect. According to them, the effect of temperature changes on daily and seasonal scales as well as the varying coefficient of thermal expansion between the various components of bridge superstructure in the horizontal and vertical directions results in cyclic expansion and contractions. Shreedhar et al. (2012) studied the behavior of integral bridge with and without soil interaction using STAAD Beava. Dunker and Liu (2007) extensively studied the behavior of integral bridges under various conditions such as the connections at abutments (fixed and pinned pile head), foundations and others. They used commercially available finite element software packages. The present study describes the effect of pile head connection with abutment on the various design parameters of deck slab of integral abutment bridge and behavior under temperature load. The commercially available finite element software SAP 2000 has been employed for the purpose. The bridge models are prepared for pile head with fixed connection and pinned connection and analyzed for load combinations like Dead Load (DL), Live Load (LL) and temperature. The effect of pile head connections on deck slab is studied by observing variations in Bending Moment (BM), shear force (SF), axial force and longitudinal stresses.

II. Model Description

Three cases of bridge models were developed by varying the length and number of spans. Single span with length of 60 m, two spans of 30 m each and three spans of 20 m each were considered with pile head having fixed connection and pinned connection. The 12 m width of the bridge was adopted with thickness of the deck slab as 0.25 m. The main girders are of 0.35 m ×1.5 m placed at a distance of 2.4 m c/c. The height of the integral abutment from the bottom of the abutment to bottom of girder is 3 m. Cast-in-situ piles of 1.1 m diameter and pier of 1.2 m diameter were considered in the present study. The models of integral abutment bridge developed using SAP 2000 are presented in Figure 1, Figure 2 and Figure 3 of single, two and three spans bridges, respectively.

The bridge models were developed using rigid links between deck slab and girder. The deck slab was modeled by quadrilateral shell element, which couples bending with membrane action and the longitudinal girders as well as diaphragm and piles were modeled as frame elements. The deck and girders were placed at their vertical locations of the centroid respectively. The composite action between the deck and girders were affected by the rigid links.

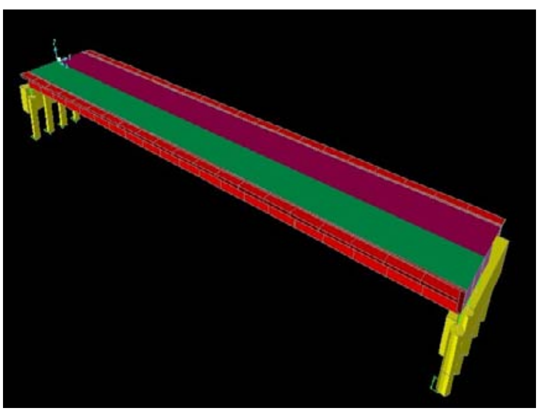

Fig.1 : 3-D view of the single span integral abutment bridge

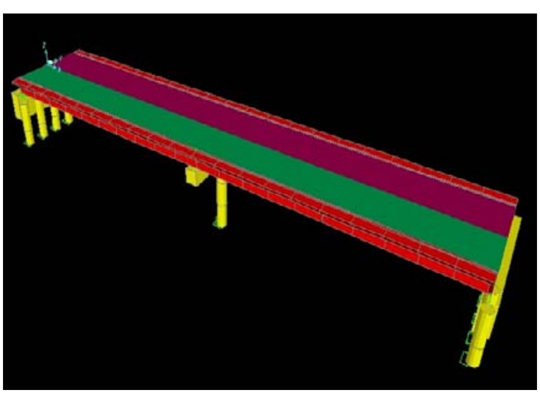

Fig. 2 : 3-D view of the two spans integral abutment bridge

The analysis was carried out by applying LL as per IRC: 6-2000 and by considering a change in temperature of +10°C. The standard characteristics of M30 concrete and Fe-415 steel were adopted as prescribed in IRC: 21 -2000. The single span bridge model post-analysis under LL and temperature stresses is shown in the Figure 4 and 5.

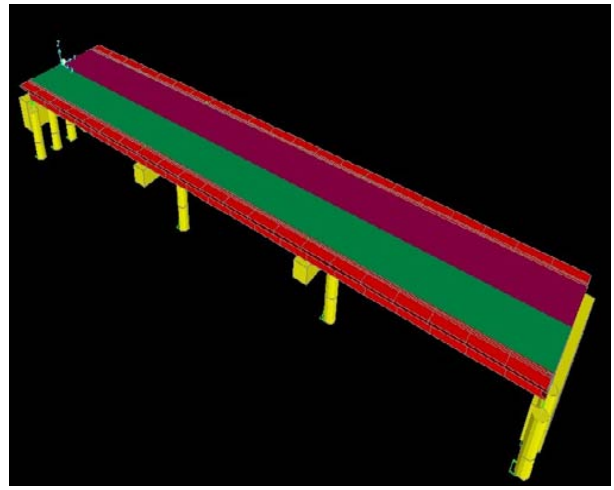

Fig. 3 : 3-D view of the three spans integral abutment bridge

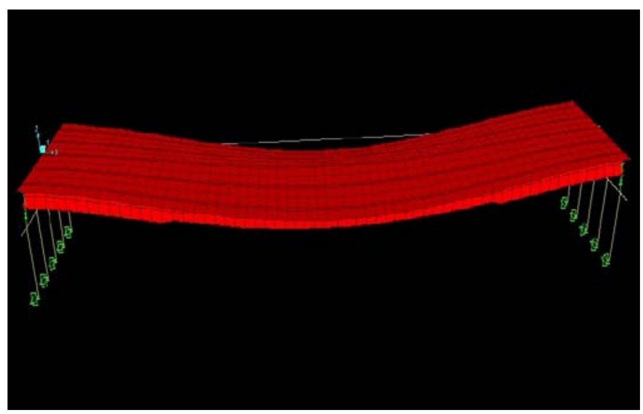

Fig. 4 : Deformed shape of the single span integral abutment bridge model for live load

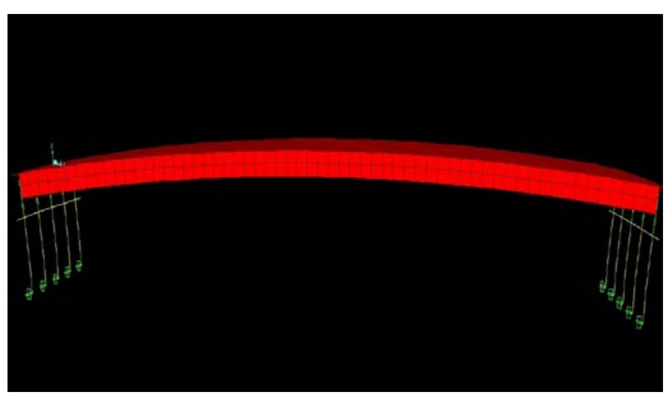

Fig. 5 : Deformed shape of the single span integral abutment bridge model for temperature load

III. Results and Discussion

The results of finite element analysis were compared for the bending moments (BM), shear forces (SF), longitudinal stresses in the deck slab, bending moments (BM) and shear forces (SF) in exterior and interior girders of integral abutment bridge having pile head with fixed and pinned connections have been discussed as follows.

Figure 6 and Figure 7 shows the comparison of BM in deck slab of integral abutment bridge, central girder and exterior girder of single span integral abutment bridge having pile head with fixed and pinned connection under only DL. It may be observed that the positive maximum BM increased by 17.69%, while negative maximum BM reduced by 10.5% in deck slab of pinned pile head connection as compared to fixed pile head connection. Similarly in the central girder, the positive maximum BM is increased by 17.79% and negative maximum BM is reduced by 10.62%. An increase of 17.62% in positive maximum BM and decrease of 10.31% of negative maximum BM was observed in the exterior girder.

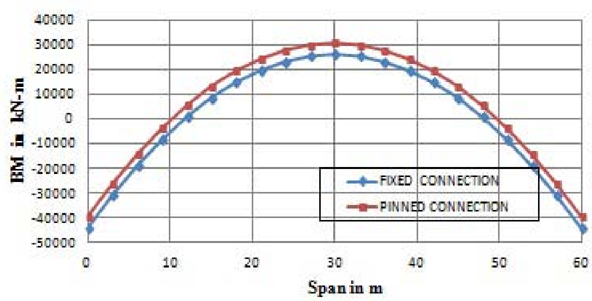

Fig. 6 : Comparison of BM variation in deck slab due to DL for pile head with fixed and pinned connection

No change in SF values were observed in the deck slab of single span integral abutment bridge having pile head fixed as compared with pile head pinned connection as shown in Figure 8. The central girder was subjected to lesser SF by 0.19% and increased in exterior girder by 0.74%in case of pinned pile head connection than that in fixed pile head connection. Figure 9 shows the variation in BM, SF and axial force.

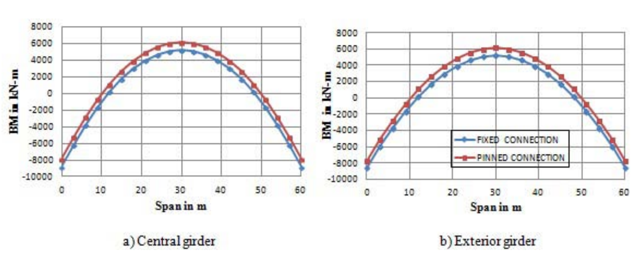

Fig. 7 : Comparison of BM variation in (a) central girder and (b) exterior girder due to DL for single span pile head with fixed and pinned connection

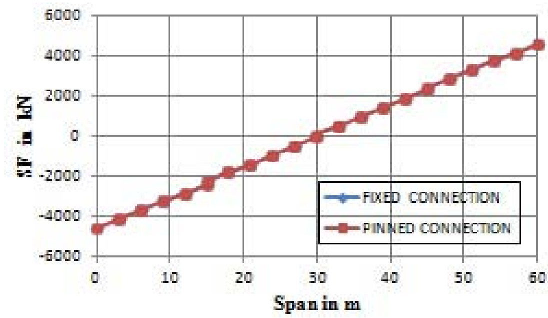

Fig. 8 : Comparison of SF variation in deck slab of single span integral abutment bridge due to DL for pile head with fixed and pinned connection

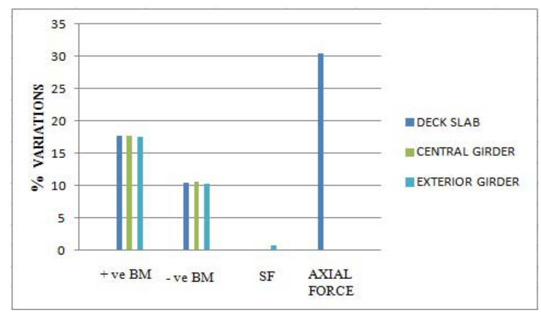

Fig. 9 : Comparison of percentage changes in all the parameters considered in deck slab, central girder and exterior girder in single span integral abutment bridge having pile head with pinned connection under DL with respect to single span bridge with fixed pile head

In the case of bridge deck slab with two spans, it was found that the positive and negative maximum BM increased by 10.93% and 11.4%, respectively in pinned pile head connection as compared with the fixed pile head connection as shown in the Figure 10. The negative BM at the end of deck slab reduced by 28.5% in case of pinned pile head connection as compared to the fixed pile head connection. It was also observed that in the central girder, the positive maximum BM increased by 10.5% and negative maximum BM increased by 10.9%, an increase of 11.4% in positive maximum BM and 11.5% in negative BM in case of exterior girder with pinned pile head connection as compared with the fixed pile head connection.

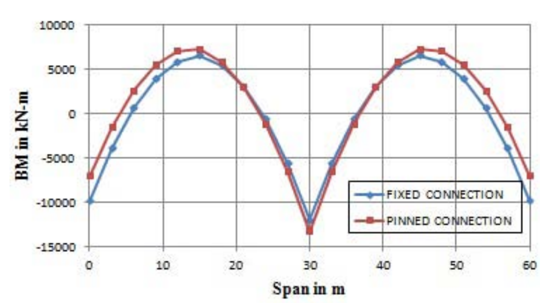

Fig. 10 : Comparison of BM variation in deck slab of two spans integral abutment bridge due to DL for pile head with fixed and pinned connection

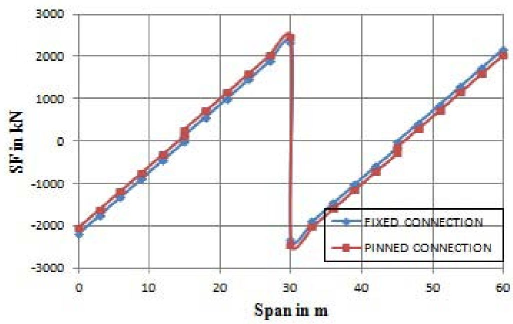

Fig. 11 : Comparison of SF variation in super structure of two spans integral abutment bridge due to DL for pile head fixed and pile head pinned connection

The Figure 11 presents the comparison of SF variations, which shows 5.9% increase of SF values in pinned pile head connection as compared with the fixed pile head connection. In the central girder, the SF reduced by 5.8% and in exterior girder SF increased by 6% with pile head having pinned connection than in fixed connection.

The percentage change in variation of all the parameters considered for two span integral abutment bridge having pile head with pinned connection in comparison with pile head fixed connection is shown in Figure 12.

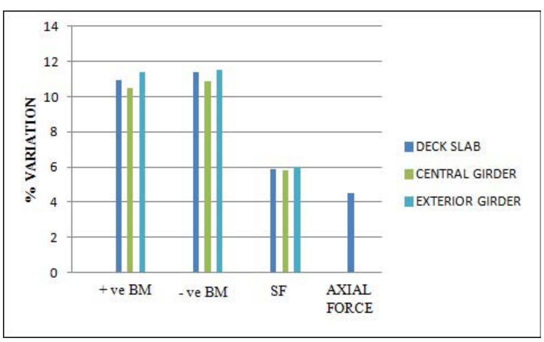

Fig. 12 : Comparison of percentage changes in all the parameters considered in deck slab, central girder and exterior girder in two spans integral abutment bridge having pile head with pinned connection under DL with respect to single span bridge with fixed pile head

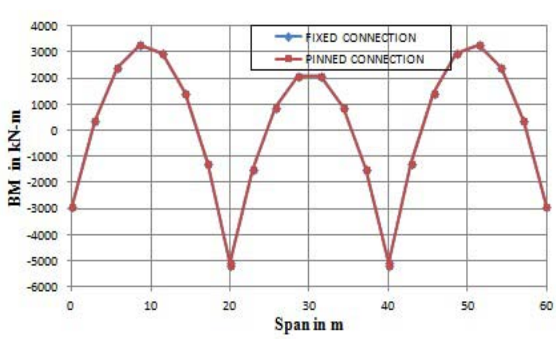

Fig. 13 : Comparison of BM variation in deck slab of three spans integral abutment bridge due to DL for pile head with fixed and pinned connection

An interesting outcome of the study was the negligible variation of BM and SF in deck slab of three spans integral abutment bridge having shaving pile head with fixed connection when compared with pinned connection as shown in Figures 13 and 14.

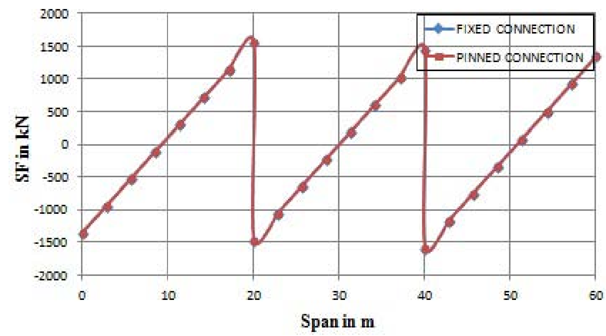

Fig. 14 : Comparison of SF variation in deck slab of three spans integral abutment bridge due to DL for pile head with fixed and pinned connection

Similarly, no change in the variation of BM and SF was observed in central and exterior girder of three spans integral abutment bridge having pile head with fixed connection as compared with pile head pinned connection bridge.

The variations of BM and SF in the deck slab, central girder and exterior girder under DL+ Temperature (10°C positive increase in temperature), DL + LL+ Temperature loading condition were similar as under only DL case when compared with pile head having fixed connection with pinned connection.

The changes in percentage of BM, SF and longitudinal stress in deck slab under only DL case with pile head having fixed and pinned connection for two spans and three spans with respect to single span is shown in Figures 15 and 16. In case of DL, the BM observed was maximum for single span (60 m). However, for two spans (30 m each) integral abutment bridge BM reduced upto 75% and for three spans (20 m each), it has reduced to 88% as compared with single span. This reduction in BM may be attributed to the increase in number of spans and the decrease in span length. SF also has maximum value for single span (60 m) integral abutment bridge. For two spanned bridge, the SF reduces to 50% and for three spans (20 m each) SF further reduced to 66% as compared with single span. Axial force was maximum for single span integral abutment bridge (60 m), while in case of two spans (30 m each) and three spans (20 m each) integral abutment bridge, the axial force reduced respectively by 77.5% and 90% as compared with single span bridge.

Longitudinal extreme top and bottom fibre stresses were maximum for single span (60 m), and they reduced to 75% for two spans (30 m each) integral abutment bridge and 89% for three spans (20 m each) integral abutment bridge as compared to single span.

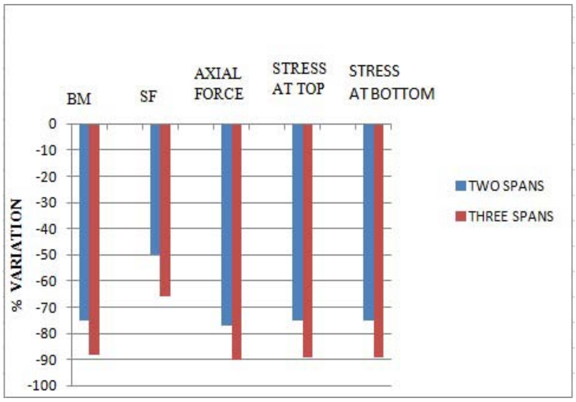

Fig. 15 : Comparison of percentage changes in all the parameters considered in deck slab of integral abutment bridge having pile head with fixed connection for two and three spans with respect to single span due to dead load

The variation of BM, SF, axial force and longitudinal stresses in the deck slab, central girder and exterior girder due to DL, DL + temperature (10°C change in temperature) and DL + LL (IRC 70R wheeled vehicle) + temperature load (10°C change in temperature) were compared for different spans with fixed pile head connection. Following charts from Figure 17 to 22 shows percentage changes in variation of all the parameters of integral abutment bridges having pile head with fixed connection for single, two and three spans bridges due to DL+ temperature and DL + LL + temperature cases in reference to only dead load for deck slab, central girder and exterior girders.

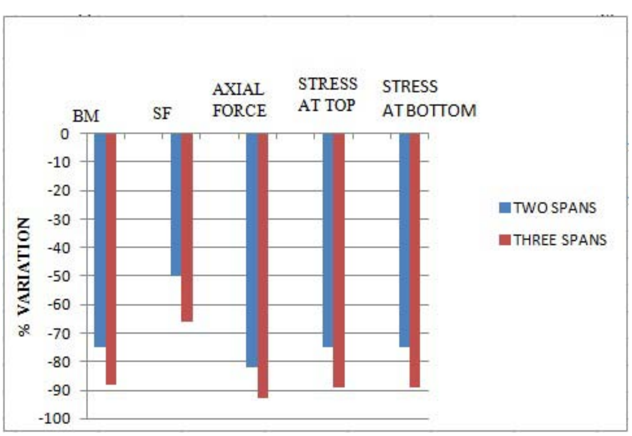

Fig. 16 : Comparison of percentage changes in all the parameters considered in deck slab of integral abutment bridge having pile head with pinned connection for two and three spans with respect to single span due to dead load

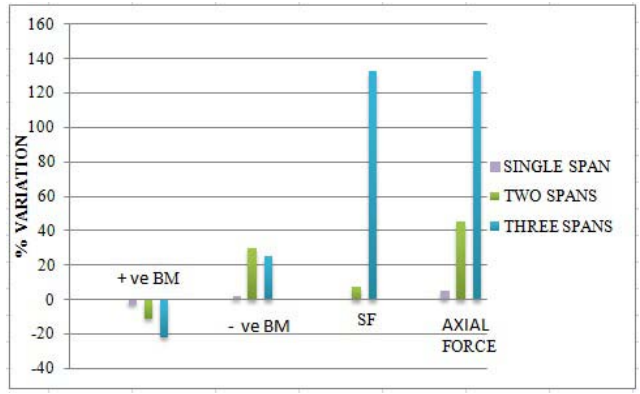

Fig. 17 : Comparison of percentage changes in all the parameters considered in deck slab of single, two and three spans integral abutment bridge due to DL+ temperature with respect to DL

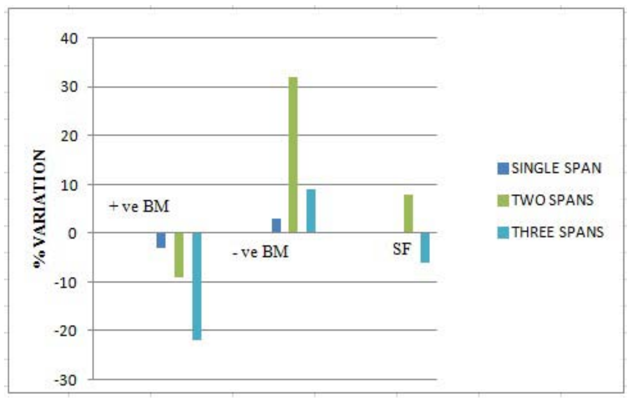

Fig. 18 : Comparison of percentage changes in all the parameters considered in central girder of single, two and three spans integral abutment bridge due to DL+ temperature with respect to DL

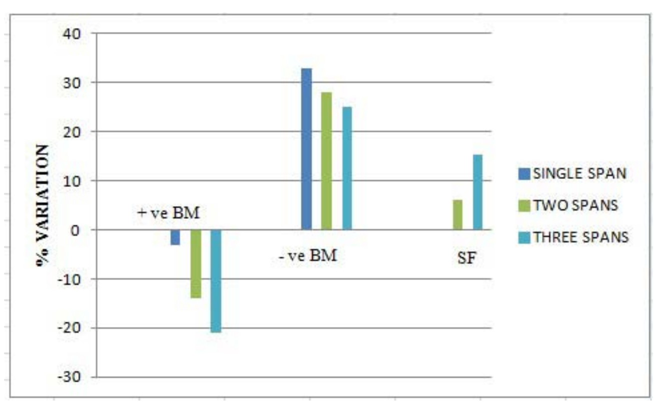

Fig. 19 : Comparison of percentage changes in all the parameters considered in exterior girder of single, two and three spans integral abutment bridge due to DL+ temperature with respect to DL

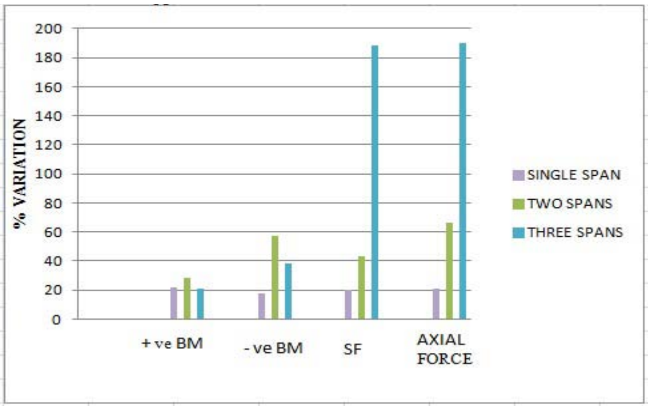

Fig. 20 : Comparison of percentage changes in all the parameters considered in deck slab of single, two and three spans integral abutment bridge due to DL+ LL + temperature with respect to DL

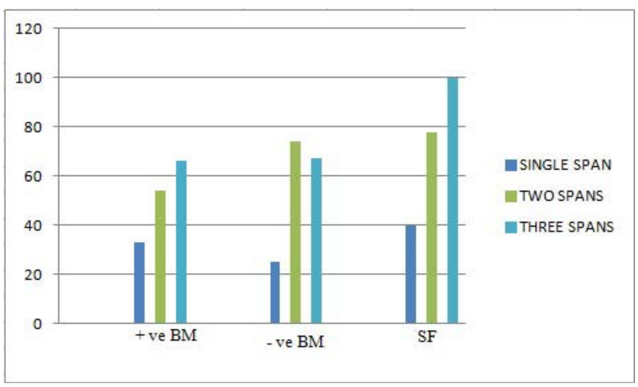

Fig. 21 : Comparison of percentage changes in all the parameters considered in exterior girder of single, two and three spans integral abutment bridge due to DL+ LL + temperature with respect to DL

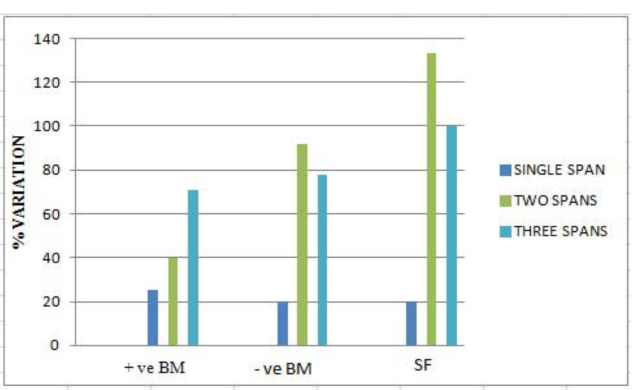

Fig. 22 : Comparison of percentage changes in all the parameters considered in central girder of single, two and three spans integral abutment bridge due to DL+ LL + temperature with respect to DL

IV. Conclusions

The following conclusions are drawn from the present analysis:

- The design parameters are affected by the pile head to abutment connection in integral abutment bridges.
- The negative BM at the end of deck slab and girders tend to reduce by 10.5% in single span and 28.5% in two spans, while there is no change in three spanned integral abutment bridge. Correspondingly, reduction of stresses at the end of deck slab is observed for the bridges having pinned pile head connection as compared with fixed pile head connection.
- An increase in SF at the deck slab was observed with a magnitude of 5.9% in two spans integral abutment bridge having pile head with pinned connection, whereas there was no change in SF in single and three spans. In the central girder, a decrease in SF and in external girder increase in SF is however observed in single and two spans bridge and there is no change in three spanned bridge girders.
- Abutment and deck connection can be designed for less BM in integral abutment bridge having pile head pinned connection as compared with fixed connection.
- The relation between the number of spans and all the design parameters was found to be inversely proportional. As the number of span increased, the design parameters such as BM and SF drastically decreased. The percentage reduction was observed to be the same for integral abutment bridges having pile head with fixed and pinned connection.
- 6. An inversely proportional relation was also observed between the number of spans and the top and bottom fibre stresses in deck slab. The stresses tend to decrease with increase in number of spans.
- The increase in temperature increases the negative moment when compared only with DL because of its hogging effect decreases in the positive BM. This trend is opposite to that of only DL which shows increase in positive BM and decrease in negative BM.
- With DL + temperature combination, the positive BM is increased by a magnitude of 17.69% and negative BM reduced by 10.5% in deck slab and girders with single span. In two span integral abutment bridge, the both positive and negative BM increased by nearly 10.93% and 11.4% respectively. However, there is no change in three spans bridge with pinned pile head as compared with fixed condition. Similar trend is also observed with DL + LL + temperature case.
- No change in shear force was observed in deck slab of one and three spanned bridges, but in case of two spans, there is 5.9% increase for the bridge

with pinned pile head connection as compared with fixed connection. Further, SF decreased in central girder and increased in exterior girder for one and two span bridges and there is no change in three spans bridge. Similar change in percentage is found in DL + LL + temperature case.

- The positive maximum BM in deck slab of integral abutment for different spans reduced in case of DL and temperature combination as compared only with DL. On the other hand negative maximum BM shows increasing trend in case of both DL and temperature and DL, LL, and temperature cases. Similar trend is also observed in interior and exterior girders.
- The SF in deck slab of integral abutment bridge for different spans increased both in case of DL + temperature combination and DL + LL + temperature combination as compared with DL, but it is zero for single span bridge with DL+ temperature combination.

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Effect of Different Bed Configuration on Flow Resistance under Different Flow Regimes in an Open Channel

By Muhammad Yaseen, Muhammad Afzal & Khalida Khan
University of the Punjab, Pakistan

Abstract- This study was conducted to evaluate the effect of different bed configuration/bed-forms on flow resistance for different flow conditions in an open channel. The study was limited to investigate whether the flow resistance increases or decreases. The inter-relationship of flow discharge on the friction factor (f) and their quantitative relationship was also determined. A physical model was constructed in the Model Tray Hall of Centre of Excellence in Water Resource Engineering (CEWRE), University of Engineering & Technology Lahore, Pakistan. The sediment commonly available in rivers of Pakistan was used in the channel as bed load under different scenarios. The sediments as bed load were used having the size ranging from 0.5 to 1.2 mm. The bed-forms were predicted using the Athaullah, Simons, Richardson and Van Rijn's Approach. Darcy–Weisbach equation was used to compute the friction factor (f). The results showed that the friction factor (f) in clear water decreased with increase of discharge upto 18 liter per second and a plane bed type was formed. For flow of 18 to 25 liter per second, a ripple bed type was formed due to increase in friction factor. For flow rate of 25 to 40 liter per second the friction factor decreased and dune bed type was formed.

Keywords: bed-forms; channel capacity; friction factor; open channel; suspended sediment.

GJRE-E Classification : FOR Code: 090599

EFFECTOFDIFFERENTBEDCONFIGURATIONONFLOWRESISTANCEUNDERDIFFERENTFLOWREGIMESINANOPENCHANNEL

Strictly as per the compliance and regulations of :

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Muhammad Yaseen [α], Muhammad Afzal [σ] & Khalida Khan [ρ]

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I. Introduction

Knowledge of flow resistance for different flow conditions helps in better understanding of flood routing, backwater curve computation and scouring. Flow resistance may be caused by roughness of the grain surface and form resistance. The resistance in open channel depends on the dimensions of the streams and roughness of its sides as well as on the shape of the channel, the degree of saturation of the stream with suspended sediments and in case of alluvial channels, dunes formed as a result of interaction between the stream flow and channel under erosion. The sediments can be transported either as bed load or suspended load or both. The bed load is the material which rolls, slides or bounces by saltation along the bed almost without leaving the bed whereas the suspended load consists of the particles which remain in suspension in the flow. In steady uniform flow in rigid boundary as well as in alluvial streams, there is a relationship between the mean velocity of flow U, the water surface slope S, the hydraulic radius R, and the characteristics of the channel boundary. Such a relationship is commonly known as flow resistance equation. A resistance equation is essential in the design of irrigation channels, river enhancement works, sediment transport studies, etc. However, the problem of predicting the resistance to flow and velocity distribution in alluvial streams are elaborated by two factors. Firstly, the configuration of the bed changes with changes in flow conditions. This changing bed condition makes it very complicated to describe the resistance due to these bed forms by a constant resistance coefficient. Secondly, under certain conditions, a part of the sediment load is transported in suspension. The material that goes into suspension changes the flow and fluid characteristics and this has large effect on velocity distribution and hence on the mean velocity. The friction factor (f) increases with increasing concentration of the suspended sediment (Yaseen et al., 2010).

The values of friction factors in sand bed rivers depend primarily on bed-form configuration which may change from plane bed, to ripples and dunes, to upper-regime plane bed and antidunes. The specific effects of bed-forms in terms of classification characteristics and resistance to flow can be found in Simons and Richardson (1963), Engelund and Hansen (1967). Specific studies on the geometry of sand dunes and resistance to flow can be found in Vanoni and Hwang (1967), Engelund (1977) and Van Rijn (1982, 1984).

In studies of flow with suspended sediment two issues often raised are the effect of suspended sediment on velocity distribution and flow resistance. Flow computations in rigid-boundary channels and alluvial channels need information on boundary friction. Accurate flow resistance values may improve the channel design and help in deciding depths of the channels. Proper channel design reduces the overtopping and loss of water in irrigation channels. The overall objective of this study was to enhance the understanding regarding flow resistance due to formation of different bed configuration under flow regimes in small channel and hence improve the design parameters of these channels.

Author α: Centre for Integrated Mountain Research (CIMR), University of the Punjab, Lahore, Pakistan.
Author σ: Centre of Excellence in Water Resources Engineering, University of Engineering and Technology, G.T. Road, Lahore, Pakistan. e-mail: afzelbutt@yahoo.com

II. Materials and Methods

Experiments were conducted in the Model Tray Hall of Centre of Excellence in Water Resources Engineering in the rectangular lined channel. The length of the channel is 40 m and its depth is 0.6 m. Bed width of the channel is 0.75 m and its bed slope is 0.35 percent. To measure the average flow depth, water measuring scales were installed at head, middle and tail ends of the channel. The sediments as bed load were used having the size ranging from 0.5 to 1.2 mm. For measurement of discharge in the experimental channel; a 90^0 v-notch weir of length 2 feet at u/s of the experimental channel was installed. The Francis Formula ($Q=0.0138H^{5/2}$) for measurement of discharge (Q in liter per second and H is in cm) was used. Sieved sand (0.5 mm to 1.2 mm) was spread over the bed of the channel. The thickness of the sediment layer was 10 cm. The bed surface was made plain with the help of a wooden template before starting the experiment. Observed the bed form after 30 minutes water runs. Repeat the above procedure for different discharges.

a) Experimental Scenarios

Various combinations of discharge were used in the present study. A series of experiments were conducted in sediment free (i.e. clear) water in the channel to determine value of the friction factor 'f_0'. With this setup, 9 different flow rates were used for clear water in the channel ranging from 12 to 39 liter per second for each scenario.

b) Computation of Flow Resistance (f)

The ASCE Task Force on Friction Factors in Open Channels (1963) expressed its belief in the general utility of using the Darcy-Weisbach formulation for resistance to flow in open channels, noting that it was more fundamental, and was based on more fundamental research. Darcy–Weisbach equation was used to calculate the friction factor given as:

$$\mathbf{f} = \frac{8g\,R_b S}{U^2} \tag{1}$$

Where f is the friction factor, g is the acceleration due to gravity (m/sec²), S is the bed slope of the channel (in fraction), U is the mean flow velocity of the channel (m/sec), R_b is the hydraulic radius with respect to bed (m). Williams's formula was used to compute the hydraulic radius with respect to bed as:

$$R_b = \frac{h}{\left(1+\dfrac{0.055h}{b^2}\right)} \tag{2}$$

Where h is the flow depth (m) and b is the channel width (m).

c) Method of Bed Forms Prediction

The flow in channels composed of erodible granular material. A strong physical interrelationship exists between the friction factor, the sediment transport rate and the geometric configuration assumed by the surface. The changes in bed forms result from the interaction of the flow, fluid and bed material. Thus the resistance to flow and sediment transport are the functions of the slope and the depth of the stream, the viscosity of the fluid and the size distribution of the bed material. To predict the bed forms following approaches were used.

i. *Simons and Richardson's Approach*

By this approach bed form was predicted in terms of the median fall diameter of bed material in the sand sized range and the stream power from graphical relationship which was developed by Simons and Richardson (1977);

Stream power is the product of shear stress, τ_0 and the mean velocity, U

$$\text{Stream power} = \tau_0 \times U \tag{3}$$

Shear stress can be computed by using the following relation

$$\tau_0 = \gamma DS \tag{4}$$

Where

γ =Specific weight of water (lbs/ft³)

D=Depth of flow (ft)

S = Bed slope (in fraction)

ii. *Athaullah's Approach*

By this approach bed form was predicted in terms of different flow regime based on the Froude number and the relative roughness from graphical relationship which was developed by Athaullah (1968); Froude number was by using the relation

$$F_r = \frac{U}{\sqrt{gD}} \tag{5}$$

Relative roughness is ratio of the Hydraulic radius, R and the median bed- material size, d.

$$\text{Relative roughness} = \frac{R}{d} \tag{6}$$

iii. *Van Rijn's Approach*

By this approach bed form was predicted in terms of a dimensionless particle parameter, d_* and a transport –stage parameter T from graphical relationship which was developed by Van Rijn (1984);

The dimensionless particle parameter was computed as

$$d_* = d\left[\frac{(\rho_s-\rho)g}{\rho\nu^2}\right]^{1/3} \tag{7}$$

Where

D = Median size of bed material (m)

ρ = Mass density of fluid (kg/m^3)

ρ_s = Mass density of sediment (kg/m^3)

g = gravitational acceleration (m/sec^2)

υ = Kinematic viscosity (m^2/sec)

The transport-stage parameter was computed by using the following relation;

$$T = \frac{(U_*^/)^2 - (U_{*c})^2}{(U_{*c})^2} \tag{8}$$

Where

U_{*c} = critical bed shear velocity (m/sec)

$U_*^/$ = bed shear velocity related to grain roughness (m/sec)

The critical bed shear velocity was computed as

$$U_{*c} = \left(\frac{\tau_c}{\rho}\right)^{1/2} \tag{9}$$

The critical shear stress was computed from the shields diagram and the bed shear velocity related to grain roughness was computed by following Chezy-type equation;

$$U_*^/ = \frac{g^{0.5}U}{18\log\left(12R_b/3d_{90}\right)} \tag{10}$$

Hydraulic radius with respect to bed was computed by using the equation (2).

III. Results and Discussion

The computation procedure to predict the bed forms by different approaches and their results under discharges are shown in Table 1. All bed forms which were predicted from prediction approaches have the same results and match with physically observed bed forms.

Values of the friction factor in clear water decreases with increase of discharge as shown in Figure 1. The trend of this relation first decreased up to 18 liter per second discharge. In this range of discharge, a plane bed type was formed and the flow resistance decreased. The value of the friction factor (f) in this range of discharge can be computed by using the empirical relation (f = 0.7201- 0.0189 Q). The plane bed formed at smaller velocity ranging from 0.4 to 0.8 ft/sec and smaller and Froude number ranging from 0.14 to 0.23. The friction factor varies from 0.5 to 0.36 over the plane bed. From 18 to 25 liter per sec discharges, the trend of discharge and friction factor (f) relationship increase and in this range of discharge a ripple bed type is formed and friction factor increases. The value of friction factor (f_0) in this range of discharge can be compute by using the relation (f = 0.245 + 0.0061Q). The friction factor shows erratic behavior at discharge rate of 25 l/s but thereafter it again decreases with increase of discharge from 25 to 40 l/s. But rate of decrease in the value of "f" is smaller than the one observed at the smaller flow rates (05-18 l/s). Thus from these results, it can be safely concluded that "f" decreases with increase of discharge but the rate of decrease may be different at different flow rate. For the flow rates of 25 to 40 l/s, the friction factor decreased as also stated earlier. The value of the friction factor (f_0) in this range of discharge can be computed by using the relation (f_0 = 0.4501- 0.0026 Q). The value of the friction factor (f_0) is 0.498 which is maximum at 12 l/s and the value of 0.351 is minimum at discharge of 39 l/s. The bed form dune formed when the flow velocity and Froude number exceed from 0.8 ft/sec and 0.25 respectively. The flow resistance over the dune bed is proportional to stream power.

Figure 1 : Relationship between discharge and friction factor (f_0) for sediment free water

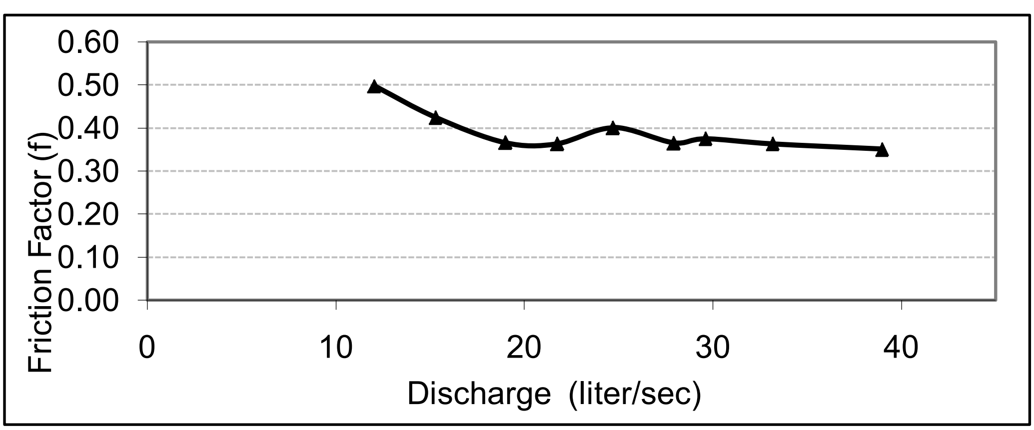

Table 1 : Prediction of Bed-Forms by Athaullah's Approach, Simons and Richardson's Approach and Van Rijn's Approach

							Athaullah's Approach		Simons and Richardson's Approach				Van Rijn's Approach	
Sr. No	Discharge	Depth of flow	Velocity	Shear Velocity	Hydraulic Radius	Froud No	Ratio R/d	Bed Forms	Median Fall Diameter	Bed Shear Stress	Stream Power	Bed Forms	Transport-stage parameter	Bed Forms
	Q (liter/sec)	h (m)	U (m/sec)	U_* (m/sec)	R (m)	Fr	R/d		D_{50} (mm)	σ (ft-lb/sec)	σU (ft-lb/ft-sec)		T	
1	5	0.070	0.117	0.049	0.057	0.14	76	Plane	0.75	0.041	0.016	Plane	-4.55	Plane
2	12	0.089	0.222	0.055	0.069	0.24	92	Plane	0.75	0.049	0.036	Plane	-0.69	Plane
3	15	0.099	0.253	0.058	0.075	0.26	100	Ripples	0.75	0.054	0.044	Ripples	-0.35	Ripples
4	19	0.109	0.285	0.061	0.080	0.28	107	Dunes	0.75	0.058	0.053	Dunes	-0.09	Dunes
5	22	0.119	0.299	0.064	0.086	0.28	114	Dunes	0.75	0.061	0.060	Dunes	-0.02	Dunes
6	25	0.134	0.302	0.068	0.093	0.26	124	Dunes	0.75	0.067	0.066	Dunes	-0.04	Dunes
7	28	0.141	0.324	0.070	0.096	0.28	129	Dunes	0.75	0.069	0.073	Dunes	0.08	Dunes
8	30	0.148	0.328	0.071	0.100	0.27	133	Dunes	0.75	0.071	0.076	Dunes	0.09	Dunes
9	33	0.158	0.344	0.074	0.104	0.28	139	Dunes	0.75	0.075	0.083	Dunes	0.15	Dunes
10	39	0.174	0.367	0.077	0.111	0.28	148	Dunes	0.75	0.079	0.095	Dunes	0.23	Dunes

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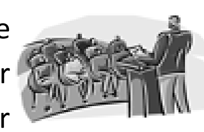

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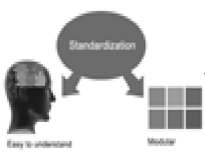

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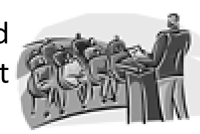

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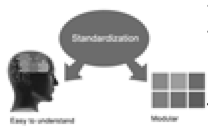

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One must be persistent and creative in using keywords. An effective keyword search requires a strategy and planning a list of possible keywords and phrases to try.

Search engines for most searches, use Boolean searching, which is somewhat different from Internet searches. The Boolean search uses "operators," words (and, or, not, and near) that enable you to expand or narrow your affords. Tips for research paper while preparing research paper are very helpful guideline of research paper.

Choice of key words is first tool of tips to write research paper. Research paper writing is an art.A few tips for deciding as strategically as possible about keyword search:

- One should start brainstorming lists of possible keywords before even begin searching. Think about the most important concepts related to research work. Ask, "What words would a source have to include to be truly valuable in research paper?" Then consider synonyms for the important words.
- It may take the discovery of only one relevant paper to let steer in the right keyword direction because in most databases, the keywords under which a research paper is abstracted are listed with the paper.
- One should avoid outdated words.

Keywords are the key that opens a door to research work sources. Keyword searching is an art in which researcher's skills are bound to improve with experience and time.

Numerical Methods: Numerical methods used should be clear and, where appropriate, supported by references.

Acknowledgements: Please make these as concise as possible.

References

References follow the Harvard scheme of referencing. References in the text should cite the authors' names followed by the time of their publication, unless there are three or more authors when simply the first author's name is quoted followed by et al. unpublished work has to only be cited where necessary, and only in the text. Copies of references in press in other journals have to be supplied with submitted typescripts. It is necessary that all citations and references be carefully checked before submission, as mistakes or omissions will cause delays.

References to information on the World Wide Web can be given, but only if the information is available without charge to readers on an official site. Wikipedia and Similar websites are not allowed where anyone can change the information. Authors will be asked to make available electronic copies of the cited information for inclusion on the Global Journals Inc. (US) homepage at the judgment of the Editorial Board.

The Editorial Board and Global Journals Inc. (US) recommend that, citation of online-published papers and other material should be done via a DOI (digital object identifier). If an author cites anything, which does not have a DOI, they run the risk of the cited material not being noticeable.

The Editorial Board and Global Journals Inc. (US) recommend the use of a tool such as Reference Manager for reference management and formatting.

Tables, Figures and Figure Legends

Tables: Tables should be few in number, cautiously designed, uncrowned, and include only essential data. Each must have an Arabic number, e.g. Table 4, a self-explanatory caption and be on a separate sheet. Vertical lines should not be used.

Figures: Figures are supposed to be submitted as separate files. Always take in a citation in the text for each figure using Arabic numbers, e.g. Fig. 4. Artwork must be submitted online in electronic form by e-mailing them.

Preparation of Electronic Figures for Publication

Even though low quality images are sufficient for review purposes, print publication requires high quality images to prevent the final product being blurred or fuzzy. Submit (or e-mail) EPS (line art) or TIFF (halftone/photographs) files only. MS PowerPoint and Word Graphics are unsuitable for printed pictures. Do not use pixel-oriented software. Scans (TIFF only) should have a resolution of at least 350 dpi (halftone) or 700 to 1100 dpi (line drawings) in relation to the imitation size. Please give the data for figures in black and white or submit a Color Work Agreement Form. EPS files must be saved with fonts embedded (and with a TIFF preview, if possible).

For scanned images, the scanning resolution (at final image size) ought to be as follows to ensure good reproduction: line art: >650 dpi; halftones (including gel photographs) : >350 dpi; figures containing both halftone and line images: >650 dpi.

Figure Legends: Self-explanatory legends of all figures should be incorporated separately under the heading 'Legends to Figures'. In the full-text online edition of the journal, figure legends may possibly be truncated in abbreviated links to the full screen version. Therefore, the first 100 characters of any legend should notify the reader, about the key aspects of the figure.

6. AFTER ACCEPTANCE

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Before start writing a good quality Computer Science Research Paper, let us first understand what is Computer Science Research Paper? So, Computer Science Research Paper is the paper which is written by professionals or scientists who are associated to Computer Science and Information Technology, or doing research study in these areas. If you are novel to this field then you can consult about this field from your supervisor or guide.

TECHNIQUES FOR WRITING A GOOD QUALITY RESEARCH PAPER:

1. Choosing the topic: In most cases, the topic is searched by the interest of author but it can be also suggested by the guides. You can have several topics and then you can judge that in which topic or subject you are finding yourself most comfortable. This can be done by asking several questions to yourself, like Will I be able to carry our search in this area? Will I find all necessary recourses to accomplish the search? Will I be able to find all information in this field area? If the answer of these types of questions will be "Yes" then you can choose that topic. In most of the cases, you may have to conduct the surveys and have to visit several places because this field is related to Computer Science and Information Technology. Also, you may have to do a lot of work to find all rise and falls regarding the various data of that subject. Sometimes, detailed information plays a vital role, instead of short information.

2. Evaluators are human: First thing to remember that evaluators are also human being. They are not only meant for rejecting a paper. They are here to evaluate your paper. So, present your Best.

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6. Use of computer is recommended: As you are doing research in the field of Computer Science, then this point is quite obvious.

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9. Use and get big pictures: Always use encyclopedias, Wikipedia to get pictures so that you can go into the depth.

10. Bookmarks are useful: When you read any book or magazine, you generally use bookmarks, right! It is a good habit, which helps to not to lose your continuity. You should always use bookmarks while searching on Internet also, which will make your search easier.

11. Revise what you wrote: When you write anything, always read it, summarize it and then finalize it.

12. Make all efforts: Make all efforts to mention what you are going to write in your paper. That means always have a good start. Try to mention everything in introduction, that what is the need of a particular research paper. Polish your work by good skill of writing and always give an evaluator, what he wants.

13. Have backups: When you are going to do any important thing like making research paper, you should always have backup copies of it either in your computer or in paper. This will help you to not to lose any of your important.

14. Produce good diagrams of your own: Always try to include good charts or diagrams in your paper to improve quality. Using several and unnecessary diagrams will degrade the quality of your paper by creating "hotchpotch." So always, try to make and include those diagrams, which are made by your own to improve readability and understandability of your paper.

15. Use of direct quotes: When you do research relevant to literature, history or current affairs then use of quotes become essential but if study is relevant to science then use of quotes is not preferable.

16. Use proper verb tense: Use proper verb tenses in your paper. Use past tense, to present those events that happened. Use present tense to indicate events that are going on. Use future tense to indicate future happening events. Use of improper and wrong tenses will confuse the evaluator. Avoid the sentences that are incomplete.

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18. Pick a good study spot: To do your research studies always try to pick a spot, which is quiet. Every spot is not for studies. Spot that suits you choose it and proceed further.

19. Know what you know: Always try to know, what you know by making objectives. Else, you will be confused and cannot achieve your target.

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21. Arrangement of information: Each section of the main body should start with an opening sentence and there should be a changeover at the end of the section. Give only valid and powerful arguments to your topic. You may also maintain your arguments with records.

22. Never start in last minute: Always start at right time and give enough time to research work. Leaving everything to the last minute will degrade your paper and spoil your work.

23. Multitasking in research is not good: Doing several things at the same time proves bad habit in case of research activity. Research is an area, where everything has a particular time slot. Divide your research work in parts and do particular part in particular time slot.

24. Never copy others' work: Never copy others' work and give it your name because if evaluator has seen it anywhere you will be in trouble.

25. Take proper rest and food: No matter how many hours you spend for your research activity, if you are not taking care of your health then all your efforts will be in vain. For a quality research, study is must, and this can be done by taking proper rest and food.

26. Go for seminars: Attend seminars if the topic is relevant to your research area. Utilize all your resources.

27. Refresh your mind after intervals: Try to give rest to your mind by listening to soft music or by sleeping in intervals. This will also improve your memory.

28. Make colleagues: Always try to make colleagues. No matter how sharper or intelligent you are, if you make colleagues you can have several ideas, which will be helpful for your research.

29. Think technically: Always think technically. If anything happens, then search its reasons, its benefits, and demerits.

30. Think and then print: When you will go to print your paper, notice that tables are not be split, headings are not detached from their descriptions, and page sequence is maintained.

31. Adding unnecessary information: Do not add unnecessary information, like, I have used MS Excel to draw graph. Do not add irrelevant and inappropriate material. These all will create superfluous. Foreign terminology and phrases are not apropos. One should NEVER take a broad view. Analogy in script is like feathers on a snake. Not at all use a large word when a very small one would be sufficient. Use words properly, regardless of how others use them. Remove quotations. Puns are for kids, not grunt readers. Amplification is a billion times of inferior quality than sarcasm.

32. Never oversimplify everything: To add material in your research paper, never go for oversimplification. This will definitely irritate the evaluator. Be more or less specific. Also too, by no means, ever use rhythmic redundancies. Contractions aren't essential and shouldn't be there used. Comparisons are as terrible as clichés. Give up ampersands and abbreviations, and so on. Remove commas, that are, not necessary. Parenthetical words however should be together with this in commas. Understatement is all the time the complete best way to put onward earth-shaking thoughts. Give a detailed literary review.

33. Report concluded results: Use concluded results. From raw data, filter the results and then conclude your studies based on measurements and observations taken. Significant figures and appropriate number of decimal places should be used. Parenthetical remarks are prohibitive. Proofread carefully at final stage. In the end give outline to your arguments. Spot out perspectives of further study of this subject. Justify your conclusion by at the bottom of them with sufficient justifications and examples.

34. After conclusion: Once you have concluded your research, the next most important step is to present your findings. Presentation is extremely important as it is the definite medium though which your research is going to be in print to the rest of the crowd. Care should be taken to categorize your thoughts well and present them in a logical and neat manner. A good quality research paper format is essential because it serves to highlight your research paper and bring to light all necessary aspects in your research.

Informal Guidelines of Research Paper Writing

Key points to remember:

- Submit all work in its final form.
- Write your paper in the form, which is presented in the guidelines using the template.
- Please note the criterion for grading the final paper by peer-reviewers.

Final Points:

A purpose of organizing a research paper is to let people to interpret your effort selectively. The journal requires the following sections, submitted in the order listed, each section to start on a new page.

The introduction will be compiled from reference matter and will reflect the design processes or outline of basis that direct you to make study. As you will carry out the process of study, the method and process section will be constructed as like that. The result segment will show related statistics in nearly sequential order and will direct the reviewers next to the similar intellectual paths throughout the data that you took to carry out your study. The discussion section will provide understanding of the data and projections as to the implication of the results. The use of good quality references all through the paper will give the effort trustworthiness by representing an alertness of prior workings.

Writing a research paper is not an easy job no matter how trouble-free the actual research or concept. Practice, excellent preparation, and controlled record keeping are the only means to make straightforward the progression.

General style:

Specific editorial column necessities for compliance of a manuscript will always take over from directions in these general guidelines.

To make a paper clear

· Adhere to recommended page limits

Mistakes to evade

- Insertion a title at the foot of a page with the subsequent text on the next page
- Separating a table/chart or figure - impound each figure/table to a single page
- Submitting a manuscript with pages out of sequence

In every sections of your document

· Use standard writing style including articles ("a", "the," etc.)

· Keep on paying attention on the research topic of the paper

· Use paragraphs to split each significant point (excluding for the abstract)

· Align the primary line of each section

· Present your points in sound order

· Use present tense to report well accepted

· Use past tense to describe specific results

· Shun familiar wording, don't address the reviewer directly, and don't use slang, slang language, or superlatives

· Shun use of extra pictures - include only those figures essential to presenting results

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Choose a revealing title. It should be short. It should not have non-standard acronyms or abbreviations. It should not exceed two printed lines. It should include the name(s) and address (es) of all authors.

Abstract:

The summary should be two hundred words or less. It should briefly and clearly explain the key findings reported in the manuscript--must have precise statistics. It should not have abnormal acronyms or abbreviations. It should be logical in itself. Shun citing references at this point.

An abstract is a brief distinct paragraph summary of finished work or work in development. In a minute or less a reviewer can be taught the foundation behind the study, common approach to the problem, relevant results, and significant conclusions or new questions.

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- Reason of the study - theory, overall issue, purpose
- Fundamental goal
- To the point depiction of the research
- Consequences, including definite statistics - if the consequences are quantitative in nature, account quantitative data; results of any numerical analysis should be reported
- Significant conclusions or questions that track from the research(es)

Approach:

- Single section, and succinct
- As a outline of job done, it is always written in past tense
- A conceptual should situate on its own, and not submit to any other part of the paper such as a form or table
- Center on shortening results - bound background information to a verdict or two, if completely necessary
- What you account in an conceptual must be regular with what you reported in the manuscript
- Exact spelling, clearness of sentences and phrases, and appropriate reporting of quantities (proper units, important statistics) are just as significant in an abstract as they are anywhere else

Introduction:

The **Introduction** should "introduce" the manuscript. The reviewer should be presented with sufficient background information to be capable to comprehend and calculate the purpose of your study without having to submit to other works. The basis for the study should be offered. Give most important references but shun difficult to make a comprehensive appraisal of the topic. In the introduction, describe the problem visibly. If the problem is not acknowledged in a logical, reasonable way, the reviewer will have no attention in your result. Speak in common terms about techniques used to explain the problem, if needed, but do not present any particulars about the protocols here. Following approach can create a valuable beginning:

- Explain the value (significance) of the study
- Shield the model - why did you employ this particular system or method? What is its compensation? You strength remark on its appropriateness from a abstract point of vision as well as point out sensible reasons for using it.
- Present a justification. Status your particular theory (es) or aim(s), and describe the logic that led you to choose them.
- Very for a short time explain the tentative propose and how it skilled the declared objectives.

Approach:

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- Sort out your thoughts; manufacture one key point with every section. If you make the four points listed above, you will need a least of four paragraphs.

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This part is supposed to be the easiest to carve if you have good skills. A sound written Procedures segment allows a capable scientist to replacement your results. Present precise information about your supplies. The suppliers and clarity of reagents can be helpful bits of information. Present methods in sequential order but linked methodologies can be grouped as a segment. Be concise when relating the protocols. Attempt for the least amount of information that would permit another capable scientist to spare your outcome but be cautious that vital information is integrated. The use of subheadings is suggested and ought to be synchronized with the results section. When a technique is used that has been well described in another object, mention the specific item describing a way but draw the basic principle while stating the situation. The purpose is to text all particular resources and broad procedures, so that another person may use some or all of the methods in one more study or referee the scientific value of your work. It is not to be a step by step report of the whole thing you did, nor is a methods section a set of orders.

Materials:

- Explain materials individually only if the study is so complex that it saves liberty this way.
- Embrace particular materials, and any tools or provisions that are not frequently found in laboratories.
- Do not take in frequently found.
- If use of a definite type of tools.
- Materials may be reported in a part section or else they may be recognized along with your measures.

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- Report the method (not particulars of each process that engaged the same methodology)
- Describe the method entirely
- To be succinct, present methods under headings dedicated to specific dealings or groups of measures
- Simplify - details how procedures were completed not how they were exclusively performed on a particular day.
- If well known procedures were used, account the procedure by name, possibly with reference, and that's all.

Approach:

- It is embarrassed or not possible to use vigorous voice when documenting methods with no using first person, which would focus the reviewer's interest on the researcher rather than the job. As a result when script up the methods most authors use third person passive voice.
- Use standard style in this and in every other part of the paper - avoid familiar lists, and use full sentences.

What to keep away from

- Resources and methods are not a set of information.
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- Leave out information that is immaterial to a third party.

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The principle of a results segment is to present and demonstrate your conclusion. Create this part a entirely objective details of the outcome, and save all understanding for the discussion.

The page length of this segment is set by the sum and types of data to be reported. Carry on to be to the point, by means of statistics and tables, if suitable, to present consequences most efficiently.You must obviously differentiate material that would usually be incorporated in a study editorial from any unprocessed data or additional appendix matter that would not be available. In fact, such matter should not be submitted at all except requested by the instructor.

Content

- Sum up your conclusion in text and demonstrate them, if suitable, with figures and tables.
- In manuscript, explain each of your consequences, point the reader to remarks that are most appropriate.
- Present a background, such as by describing the question that was addressed by creation an exacting study.
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- Not at all, take in raw data or intermediate calculations in a research manuscript.
- Do not present the similar data more than once.
- Manuscript should complement any figures or tables, not duplicate the identical information.
- Never confuse figures with tables - there is a difference.

Approach

- As forever, use past tense when you submit to your results, and put the whole thing in a reasonable order.
- Put figures and tables, appropriately numbered, in order at the end of the report
- If you desire, you may place your figures and tables properly within the text of your results part.

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- In spite of position, each table must be titled, numbered one after the other and complete with heading
- All figure and table must be adequately complete that it could situate on its own, divide from text

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The Discussion is expected the trickiest segment to write and describe. A lot of papers submitted for journal are discarded based on problems with the Discussion. There is no head of state for how long a argument should be. Position your understanding of the outcome visibly to lead the reviewer through your conclusions, and then finish the paper with a summing up of the implication of the study. The purpose here is to offer an understanding of your results and hold up for all of your conclusions, using facts from your research and generally accepted information, if suitable. The implication of result should be visibly described. Infer your data in the conversation in suitable depth. This means that when you clarify an observable fact you must explain mechanisms that may account for the observation. If your results vary from your prospect, make clear why that may have happened. If your results agree, then explain the theory that the proof supported. It is never suitable to just state that the data approved with prospect, and let it drop at that.

- Make a decision if each premise is supported, discarded, or if you cannot make a conclusion with assurance. Do not just dismiss a study or part of a study as "uncertain."
- Research papers are not acknowledged if the work is imperfect. Draw what conclusions you can based upon the results that you have, and take care of the study as a finished work
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- Give details all of your remarks as much as possible, focus on mechanisms.
- Make a decision if the tentative design sufficiently addressed the theory, and whether or not it was correctly restricted.
- Try to present substitute explanations if sensible alternatives be present.
- One research will not counter an overall question, so maintain the large picture in mind, where do you go next? The best studies unlock new avenues of study. What questions remain?
- Recommendations for detailed papers will offer supplementary suggestions.

Approach:

- When you refer to information, differentiate data generated by your own studies from available information
- Submit to work done by specific persons (including you) in past tense.
- Submit to generally acknowledged facts and main beliefs in present tense.

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Topics	Grades		
	A-B	**C-D**	**E-F**
Abstract	Clear and concise with appropriate content, Correct format. 200 words or below	Unclear summary and no specific data, Incorrect form Above 200 words	No specific data with ambiguous information Above 250 words
Introduction	Containing all background details with clear goal and appropriate details, flow specification, no grammar and spelling mistake, well organized sentence and paragraph, reference cited	Unclear and confusing data, appropriate format, grammar and spelling errors with unorganized matter	Out of place depth and content, hazy format
Methods and Procedures	Clear and to the point with well arranged paragraph, precision and accuracy of facts and figures, well organized subheads	Difficult to comprehend with embarrassed text, too much explanation but completed	Incorrect and unorganized structure with hazy meaning
Result	Well organized, Clear and specific, Correct units with precision, correct data, well structuring of paragraph, no grammar and spelling mistake	Complete and embarrassed text, difficult to comprehend	Irregular format with wrong facts and figures
Discussion	Well organized, meaningful specification, sound conclusion, logical and concise explanation, highly structured paragraph reference cited	Wordy, unclear conclusion, spurious	Conclusion is not cited, unorganized, difficult to comprehend
References	Complete and correct format, well organized	Beside the point, Incomplete	Wrong format and structuring

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