Evaluation of WSM, LRFD and FE Methods for Pile Capacity Calculation With Pile Load Test for IGM

Pratik V. Shah[1] and Kandarp K. Thaker[2]

[1]Associate Professor, Dept. of Civil Engineering, RNGPIT, Surat.
[2]Geotechnical Consultant, C. E. O. and Partner, K. C. T. Consultancy Services, Ahmedabad.

Abstract: As many infrastructures projects are going on in Ahmedabad like metro rails and bridges which are being constructed, for that behaviour of pile foundation capacity is must. And in that too the behaviour of pile in overconsolidated soil. Sometimes the overconsolidated soil are considered as Intermediate Geomaterials (IGM) i.e. behaviour comes in between the continuum of soil and rock. There are many analytical methods for evaluating pile capacity in any type of soil. But for overconsolidated soil it may be exhibit the wide range of properties. Using an analytical method 17 problems were analysed and there results were calculated from LRFD method (O‟Neill and Reese) & IS code (IS 2911 P1/S2, 2010) were compared and the FEM modelling of this soil was done using software PLAXIS 2D. The load settlement curve from FEM software and actual load test data were super imposed and the result were quiet matching. Skin friction was found to be over predicted by 37.26%, 212.37%, and 32.46% when determined from static formula (IS 2911 P1/S2, 2010), Cole & stroud method (IS 2911 and IRC 78, 2014) and LRFD method (O‟Neill and Reese) respectively. End bearing was found to be over predicted by 754.31%, 864.89%, and 55.04% when determined from static formula (IS 2911 P1/S2, 2010), Cole & stroud method (IS 2911 and IRC 78, 2014) and LRFD method (O‟Neill and Reese) respectively.

Keyword: Intermediate Geomaterial (IGM), Overconsolidated Soil, LRFD method, IS code method.

1. Introduction:

When the soil is having a low bearing capacity and hard strata is available at larger depth, piles are used to transfer the load to deeper and strong medium. The first pile was used by the romans; Vitruvius (59 A.D.) records the use of such foundations. Evidence were found out that a pile driving device was used in building artificial islands at Oakbank in Scotland as early as 5000 years ago. It is believed that the first pile was used in 1848 in Chicago. Pile foundation are used to carry the load coming over it along the lateral loads without the failure and excessive settlement.

In present research work, the work is done on the soil named IGM i.e. Intermediate Geomaterials. IGM's, is a soil material which is harder and denser than the ordinary soils but which are not cemented to an extent which are found in rock. It comes between the continuum of soil and rock mass. The study shows whether the static method (IS 2911, P1/S2, 2010) or Cole & stroud method (IS 2911 and IRC 78, 2014) is applicable to the pile carrying capacity of IGM material, which is further compared with the LRFD method for IGM and pile load test data were calibrated with the PLAXIS 2D. For soil properties required in PLAXIS 2D N-values were used to calculate using correlations.

2. Pile Carrying Capacity:

The capacity of pile foundation is determined with combination of skin friction and end bearing. The skin friction is related to the shear strength parameters of the soils as well as the friction between the soil and concrete. The end bearing is related to function of a pile displacement, the peak value differs with the displacements. The concept of separate calculation of skin friction and end bearing resistance forms the base of all static calculation of pile carrying capacity.

2.1 IS code method:

Piles in Granular Soil: IS 2911 (Part 1 / Sec. 2): 2010

$$Q_u = (A_p*0.5*D*\gamma*N_\gamma+P_D*N_q) + (\Sigma (K_i*P_{Di}*\tan\delta_i*A_{si})) \quad (1)$$

Where,

A_p = cross-sectional area of pile tip, in m^2, D = diameter of pile shaft, in m, γ = effective unit weight of the soil at pile tip, in kN/m^3, N_γ & N_q = bearing capacity factors depending upon the angle of internal friction, ϕ at pile tip, P_D = effective overburden pressure at pile tip, in kN/m2, Ki = coefficient of earth pressure applicable for the *i*th layer, P_{Di} = effective overburden pressure for the *i*th layer, in kN/m^2, δ_i = angle of wall friction between pile and soil for the *i*th layer, A_{si} = surface area of pile shaft in the *i*th layer, in m^2,

Piles in Cohesive Soils: IS 2911 (Part 1 / Sec. 2): 2010

$$Q_u = (A_p * N_c * C_p) + (\Sigma (\alpha_i * c_i * A_{si})) \quad (2)$$

Where,
A_p = cross-sectional area of pile tip, in m^2, N_c = bearing capacity factor, may be taken as 9, C_p = average cohesion at pile tip, in kN/m^2, α_i = adhesion factor for the ith layer, c_i = average cohesion for the ith layer, in kN/m^2, A_{si} = surface area of pile shaft in the ith layer, in m^2,

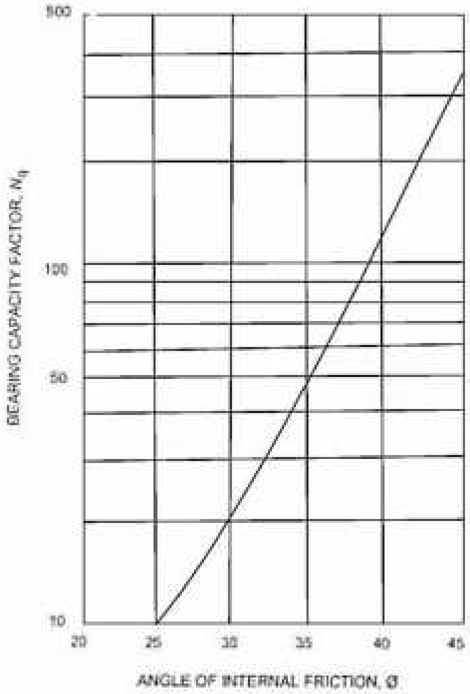

Figure 1. Bearing Capacity Factor N_q (BIS 2010)

Table 1. General Shear Failure Factors for N_γ (BIS 2002)

ϕ	0	5	10	15	20	25	30	35	40	45	50
Nγ	0	0.45	1.22	2.65	5.39	10.88	22.4	48.03	109.41	271.76	762.89

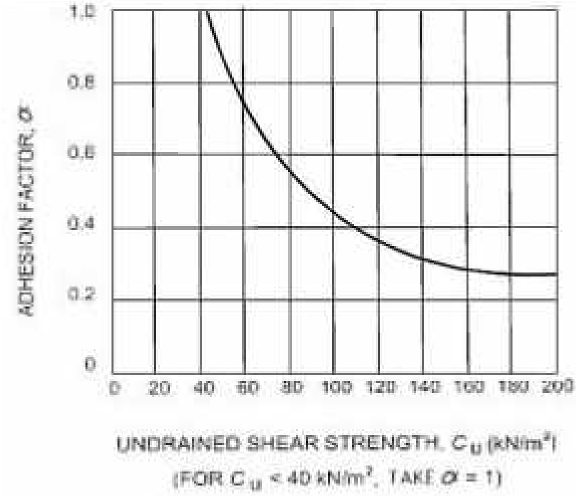

Figure 2. Variation of ~-with C_u (BIS 2010)

Piles in Weather/Soft Rock: IRC – 78 – 2014

$$Q_u = (A_p * \{C_{u1} * N_c/Fs\}) + [\Sigma (\{\alpha * C_{u2}/Fs\} * A_{si})] \quad (3)$$

Where,

A_p = cross-sectional area of pile tip, in m^2, A_{si} = surface area of pile shaft in the ith layer, in m^2, C_{u1} = average shear strength of rock in the socketed length of pile in kN/m^2, C_{u2} = average shear strength of rock in the socketed length of pile, in kN/m^2, N_c = bearing capacity factor taken as 9, α = 0.9, Fs = factor of safety usually taken as 3 for end bearing and 6 for skin friction.

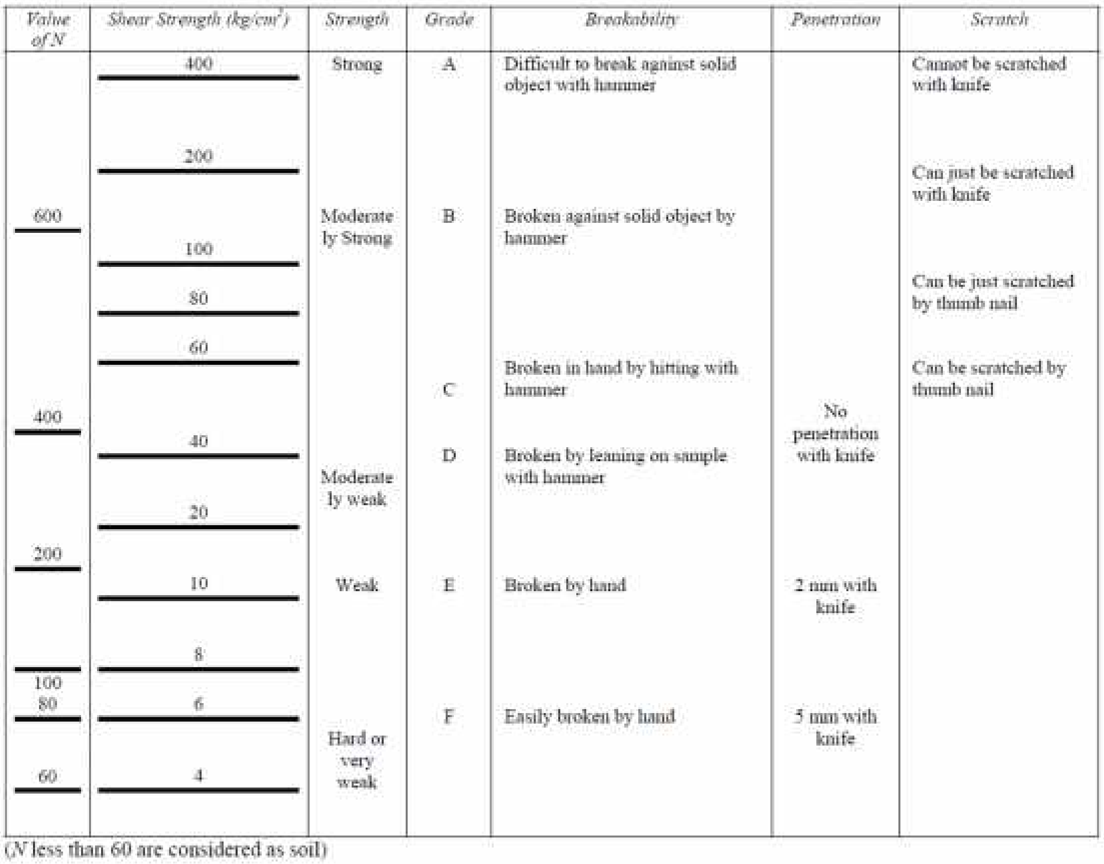

Figure 3. Consistency and Shear Strength Of Weather Rock (B.R. Srinivasamurthy, K.L. Pujar 2009)

2.2 Piles in IGMs: FHWA – IF – 99 – 025

Cohesionless IGMs:

$$q_{pbk} = 0.59*((N_{60}*(P_a/\sigma'_v))^{0.8})*\sigma'_v \quad (4)$$

$$q_{psik} = k_{oi}*\tan\varphi`_i*\sigma'_{vi} \quad (5)$$

Where

q_{pbk} = is the characteristic value of the resistance per unit area of the base, q_{psik} = is the characteristic value of the resistance per unit of the shaft in layer *i*, A_p = cross-sectional area of pile tip, in m^2, A_{si} = surface area of pile shaft in the ith layer, in m^2, N_{60} = hammer efficiency, P_a = atmospherric pressure, in kN/m^2, σ'_v = effective overburden pressure at pile tip, in kN/m^2, σ'_{vi} = effective overburden pressure for the *i*th layer, in kN/m^2, k_{oi} = design value of earth pressure at rest for *i*th layer

$$k_{oi} = (1 - \sin\varphi`_i) * \left(\frac{0.2 \cdot P_a \cdot N_{60i}}{\sigma'_{vi}}\right)^{\sin\varphi`_i} \quad (6)$$

φ`i = design value of internal friction for *i*th layer

$$\varphi`_i = \tan^{-1} * \left[\left\{\frac{N_{60}}{12.3 + 20.3\frac{\sigma_{vi}}{P_a}}\right\}^{0.34}\right] \quad (7)$$

Cohesive IGMs:

$$Q_u = (A_p*q_{max}) + (\Sigma(\alpha_i*q_{ui}*A_{si})) \quad (8)$$

Where A_p = cross-sectional area of pile tip, in m^2, A_{si} = surface area of pile shaft in the ith layer, in m^2, q_{max} = $[s^{0.5} + (m*s^{0.5} + s)^{0.5}]*q_u$, $_i$ = adhesion factor, q_{ui} = unconfined compressive strength at *i*th layer, q_u = unconfined compressive strength at pile tip.

3. Collection of Data and Data Analysis:

Borelog data with the actual pile load test of Ahmedabad city and surat is collected from various consultacy agencies. The borelog data will be used to find out the parameters for constitute model and other soil properties like frictional angle, undrained shear strength, etc.

3.1 Correction for the N – values (($(N)_{60}$):

The N – values have many correlations with relative density (R_d), undrained shear strength (Su), angle of internal friction (ϕ) and other parameters.

$$(N)_{60} = ((N*C_N)*C_d)*C_E*C_B*C_R*C_S \tag{9}$$

Where,

N = measured standard penetration resistance (raw data of N at the site), C_N = depth or overburden correction factor, C_d = dilatancy correction factor, C_E = hammer energy ratio correction factor, C_B = borehole diameter correction factor, C_R = rod length correction factor, C_S = correction factor for samplers with or without liners

3.2 Angle of Internal Friction (¤):

The ability of a unit of a soil or rock to withstand a shear stress.

$$\phi = 0.28*N_{corr} + 27^{o}$$
$$\phi = \sqrt{12 * N_{corr}} + 15 \tag{10}$$

3.3 Saturated Density of Soil (¥$_{sat}$):

It is the ratio of the weight per unit of volume.

$$\gamma_{sat} = \frac{N_{corr}}{390} - \frac{N_{corr}}{11.5} + 19.15 \tag{11}$$

3.4 Undrained Shear Strength (S_u):

For finding out the value of S_u for weather rock a figure 3. given Cole and Stroud will be used. And for other method S_u will be equal to 5.985* N_{corr} and 4.5* N_{corr}.

3.5 Stress Strain Modulus (E_s):

It is the ratio of stress along an axis over the strain along that axis in the range of elastic soil behaviour. The E_s will be found out from the figure no. 4 and 5.

Soil		
	Use the undrained shear strength s_u in units of s_u	
Clay and silt	$I_P > 30$ or *organic*	$E_s = (100 \text{ to } 500)s_u$
Silty or sandy clay	$I_P < 30$ or *stiff*	$E_s = (500 \text{ to } 1500)s_u$ Again, $E_{s,\mathrm{OCR}} \approx E_{s,nc}\sqrt{\mathrm{OCR}}$ Use smaller s_u-coefficient for highly plastic clay.

Of general application in clays is

$$E_s = Ks_u \qquad \text{(units of } s_u\text{)} \tag{a}$$

where K is defined as

$$K = 4200 - 142.54 I_P + 1.73 I_P^2 - 0.0071 I_P^3 \tag{b}$$

and I_P = plasticity index in **percent.** Use $20\% \le I_P \le 100\%$ and round K to the nearest multiple of 10.

Another equation of general application is

$$E_s = 9400 - 8900 I_P + 11\,600 I_c - 8800 S \qquad \text{(kPa)} \tag{c}$$

I_P, I_c, S = previously defined above and/or in Chap. 2

Figure 4. Equations for E_s (Bowles 1996)

3.6 Dilatancy Correction (ψ):

$$\psi = \phi - 30 \tag{12}$$

3.7 Poisson`s Ratio («):

Table 2. value of «

Soil condition	Type of consolidation	Value of μ
Sand	NC OC	0.35 0.30
Clay Unsaturated	NC OC	0.35 0.30
Clay Fully Saturated	NC OC HOC	0.50 0.40 0.30
IGM		0.25

Soil	SPT	CPT
Sand (normally consolidated)	$E_s = 500(N + 15)$ $= 7000\sqrt{N}$ $= 6000N$ — — — $E_s = (15000 \text{ to } 22000) \cdot \ln N$	$E_s = (2 \text{ to } 4)q_u$ $= 8000\sqrt{q_c}$ — — — $E_s = 1.2(3D_r^2 + 2)q_c$ $E_s = (1 + D_r^2)q_c$
Sand (saturated)	$E_s = 250(N + 15)$	$E_s = Fq_c$ $e = 1.0 \quad F = 3.5$ $e = 0.6 \quad F = 7.0$
Sands, all (norm. consol.)	$E_s = (2600 \text{ to } 2900)N$	
Sand (overconsolidated)	$E_s = 40000 + 1050N$ $E_{s(\mathrm{OCR})} \approx E_{s,\mathrm{nc}}\sqrt{\mathrm{OCR}}$	$E_s = (6 \text{ to } 30)q_c$
Gravelly sand	$E_s = 1200(N + 6)$ $= 600(N + 6) \quad N \le 15$ $= 600(N + 6) + 2000 \quad N > 15$	
Clayey sand	$E_s = 320(N + 15)$	$E_s = (3 \text{ to } 6)q_c$
Silts, sandy silt, or clayey silt	$E_s = 300(N + 6)$	$E_s = (1 \text{ to } 2)q_c$
	If $q_c < 2500$ kPa use $E_s' = 2.5q_c$ $2500 < q_c < 5000$ use $E_s' = 4q_c + 5000$ where $E_s' = \text{constrained modulus} = \frac{E_s(1-\mu)}{(1+\mu)(1-2\mu)} = \frac{1}{m_v}$	
Soft clay or clayey silt		$E_s = (3 \text{ to } 8)q_c$

Figure 5. Equations for Es (Bowles 1996)

4. Numerical Modelling:

Finite Element Model is a method of solving continuous problems governed by differential equations by dividing the continuum into a finite number of elements, which are specified by a finite number of parameters. A problem is solved by dividing the larger geometry into small elements, which are interconnected with nodes. Each element is assigned an element property. In solid mechanics, the properties include stiffness characteristics for each element.

4.1 Hardening Soil (HS) Model:

The HS model is a hyperbolic elasto-plastic model and which models the dependence of stiffness moduli on stress level but does not take the viscous effects such as creep and stress relaxation. The HS model is based on the Mohr-Coulomb failure criterion and its yield surface may expand due to plastic strains. In the HS model, the volumetric hardening has been complemented by deviatoric (shear) hardening to overcome this; it is an extension of the hyperbolic model developed by Duncan & Chang (1970).

Advantages:

- More accurately stiffness is defined than the MC model
- It takes into consideration soil dilatancy

Disadvantages:

- The higher computational costs
- It does not include viscous effects

It does not include softening

4.2 PLAXIS 2D Analysis:

In order to perform a numerical analysis with PLAXIS 2D, some general assumptions with regards to material behaviour , stress state, geometry and parameters selection is to be made,

1. Assumption of axisymmetry.
2. The elements used in the model are 15 nodded trianglular element.
3. The geometry in X - direction is fixed to 20.1 m and in Y – direction is aproximately 1.25 times of the depth of pile.
4. The gravitational constant is 9.8 m/s^2.
5. The density of water is taken as 10 kN/m^3.
6. The clay material are undrained type of soil material and granular soils are drained type soil material.
7. The meshes are made two times more refined in the region of three times from the pile for more accurate result.
8. For modelling in HS model the value of m is fixed to 0.6 for HOC and 0.5 for cohesionless soil.
9. The value of R-inter is fixed to 0.9 for all soil models and 1.0 for pile materials.

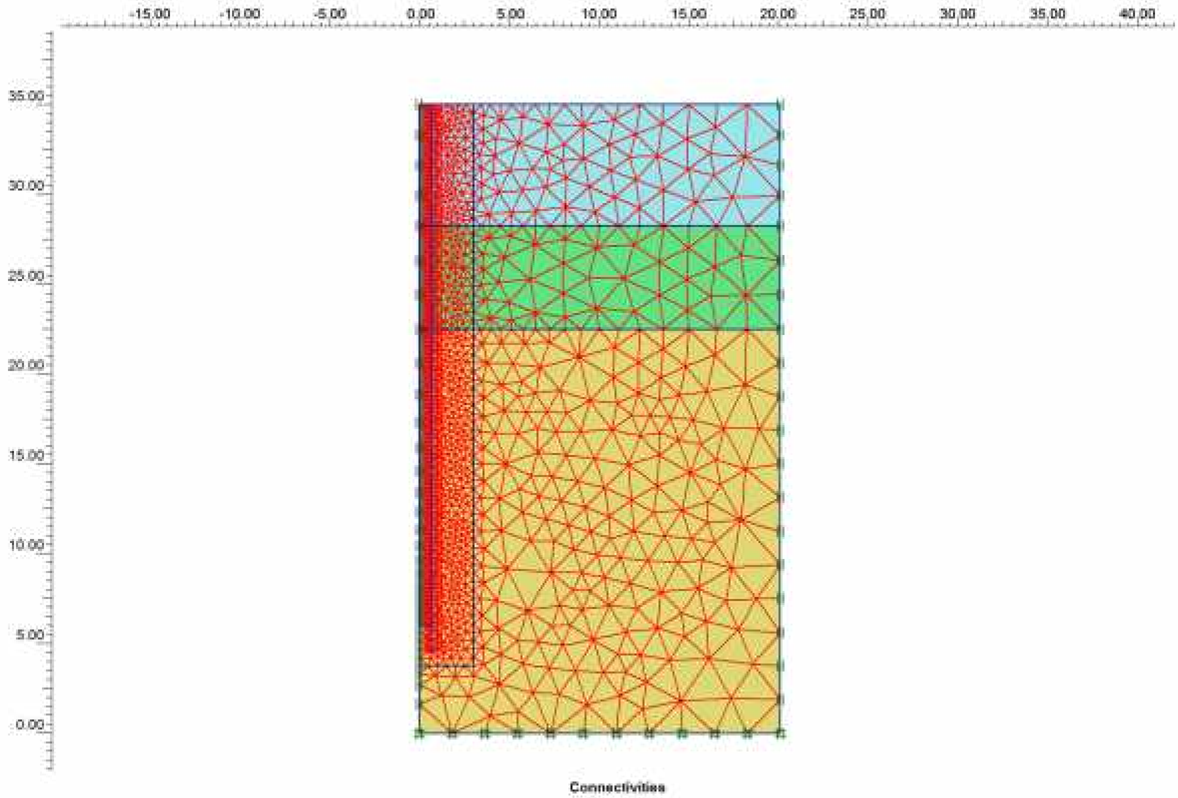

Figure 6. Geometry of model

5. Result:

The study was not limited to Ahmedabad region only, the data for Surat region was also taken into consideration for the thesis. The results obtain from the PLAXIS 2D shows similar behaviour to the actual pile load test data. And the assumption for different values was found to be accurate.

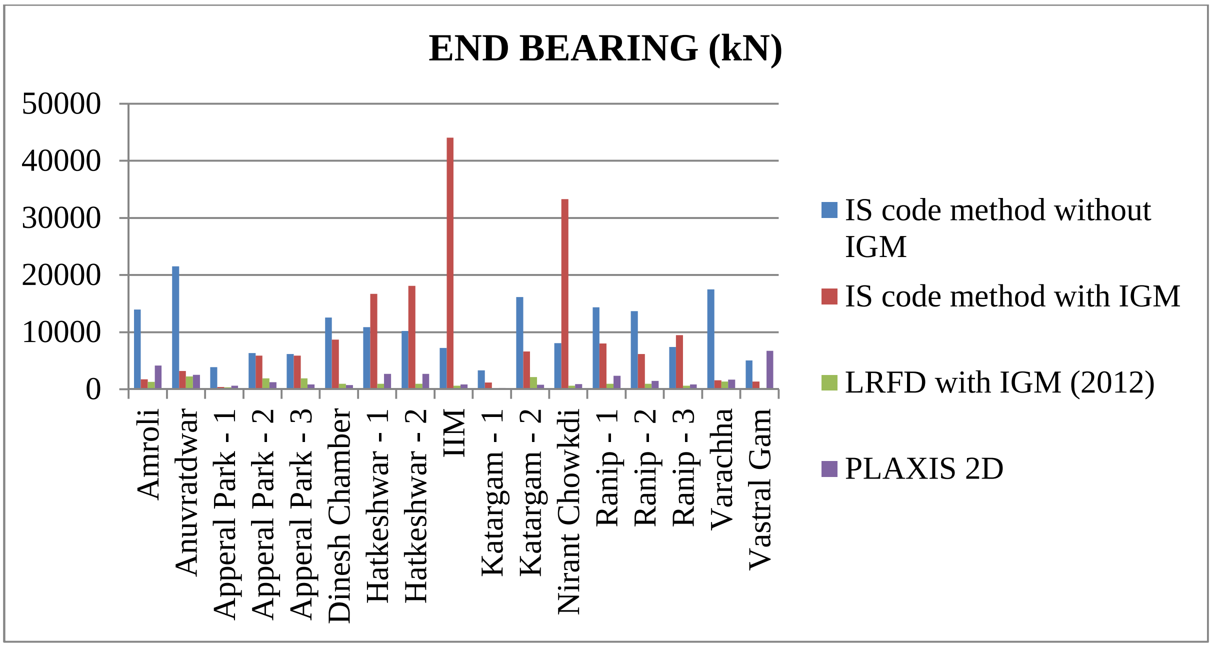

Figure 7. Comparison of End Bearing Capacity

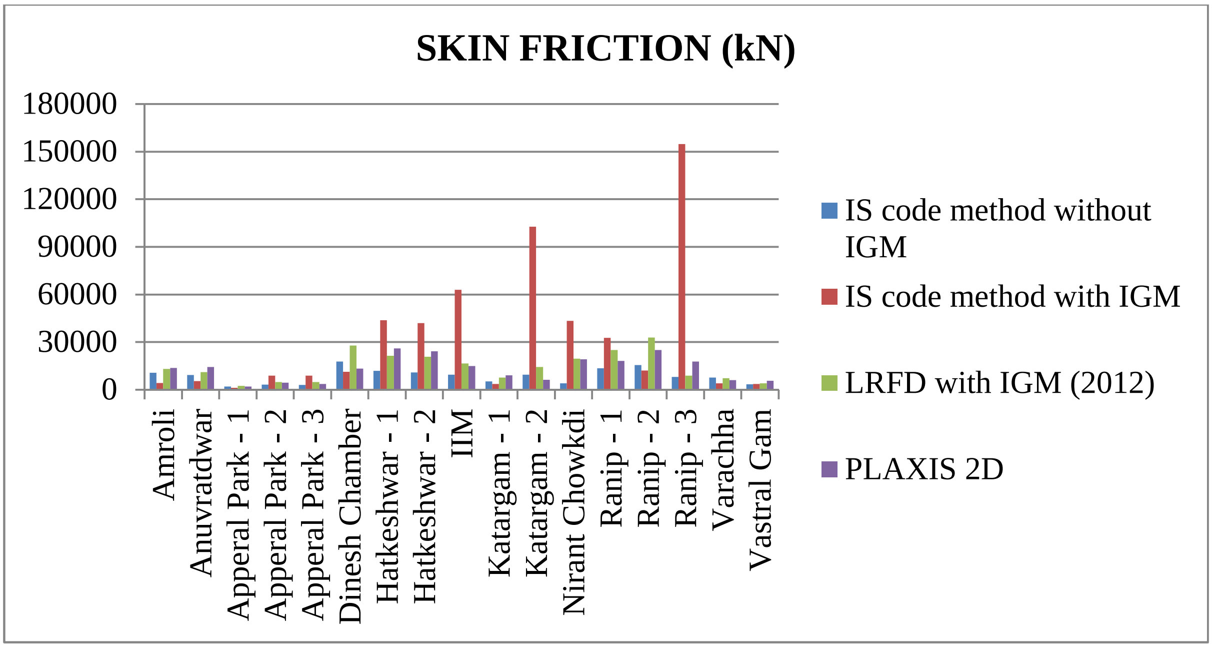

Figure 8. Comparison of Skin Friction

Table 3. Value of Safe Load

Location	IS 2911 Part - 1/ Sec. 2 2010	IRC 78 - 2014	LRFD METHOD
	Total Ultimate Load (kN)	Total Ultimate Load (kN)	Total Ultimate Load (kN)
Amroli	9826.78	2366.31	8575.61
Anuvratdwar	12277.81	3459.84	7931.31
Apperal Park - 1	2350.68	614.92	1613.54
Apperal Park - 2	3827.18	5911.57	3909.64
Apperal Park - 3	3685.84	5911.57	3986.48
Dinesh Chamber	12086.87	7983.18	17207.31
Hatkeshwar - 1	9114.82	24192.59	13401.36

Hatkeshwar - 2	8410.94	24024.83	12974.39
IIM	6679.29	42832.74	10276.78
Katargam - 1	3458.76	1925.57	4669.46
Katargam - 2	10290.65	43713.84	9790.83
Nirant Chowkdi	4857.71	30694.66	12031.84
Ranip - 1	11176.97	16270.30	15509.47
Ranip - 2	11729.68	7287.64	20294.59
Ranip - 3	6198.76	65731.61	5688.15
Varachha	10017.91	2246.95	5117.53
Vastral Gam	3406.25	1951.00	2551.79

6. Conclusion:

Consistent with usual practice, the static formula is used for determination of the safe load on piles in soils. It was observed in many actual load tests that the safe load obtained is reasonably on higher side compared to theoritically determined safe load. This warrants a study to find the most suitable approach to narrow such difference and optimising pile design.

Data from various region was collected for studying the behaviour. Analyses was performed using the borelog, laboratory and filed test results and actual load test data. The study was conducted on both the Ahmedabad and Surat soils and following conclusion were drawn based on that.

1. A reasonable match was found between the result of full scale pile load test and that obtain in FE analyses using axisymmetry model in PLAXIS 2D. Though limited data set was analysed, available findings provide a good insights.
2. In over and heavily over consolidated soils in study area, the static formula gives lower value of safe load comapred to that from the approaches used in the IGM. Safe load on pile is under estiamted when static formula is used.
3. By idealising the over and heavily over consolidated soils as IGM and by using the O’Neill and Reese method, more reliable estimate of safe load can be obtained. It is also confirmed that the LRFD method by O’Neill and Reese gives more realistic safe load compare to the method of Cole and stoud as recommended by IRC – 78 – 2014 for IGM for the piles in the study area.
4. It can be concluded from study that above conclusions hold good even for geographically different locations within study area having over consolidated soils.
5. The skin friction evaluated using PLAXIS 2D modelling matches fairly with the skin friction derived using the approach of O’Neill and Reese (LRFD method for IGM). Skin friction was found to be over predicted by 37.26%, 212.37%, and 32.46% when determined from static formula (IS 2911 P1/S2, 2010), Cole & stroud method (IS 2911 and IRC 78, 2014) and LRFD method (O’Neill and Reese) respectively.
6. The end bearing evaluated using PLAXIS 2D modelling matches fairly with the skin friction derived using the approach of O’Neill and Reese (LRFD method for IGM). End bearing was found to be over predicted by 754.31%, 864.89%, and 55.04% when determined from static formula (IS 2911 P1/S2, 2010), Cole & stroud method (IS 2911 and IRC 78, 2014) and LRFD method (O’Neill and Reese) respectively.
7. PLAXIS provided good estimate of load settlement curve based on soil properties and using HS model when findings were compared with the acutal load test data.

There is a tendency to terminate full scale pile load test once ultimate load is found instead of continuing it till failure, which limits the this study to some extent. It is desirable that pile load tests are continued till failure. For this, if reaction loading is a limitation than test pile diameter may be reduced which does not result into any appreciable change in soil structure interaction.

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