Chapter 18
Foundations for Tall Buildings on Alluvial Deposits—Geotechnical Aspects

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18.1 Introduction

With rapid economic development, the urban land cost is skyrocketing. Consequently, construction of tall multi-storeyed buildings has become common in metropolitan cities of India.

As of today, the structural engineer is the primary decision maker in the design process of such buildings in India. Typically, the geotechnical design is done by the structural engineer and there is no involvement of the geotechnical engineer beyond the site investigation stage. Therefore, the approach on geotechnical aspects still remains conventional whereas structural design has seen several innovations.

Geotechnical engineers need to rise to the challenge and come out with innovative solutions that not only ensure safety of the building but also places emphasis on reliability and economy.

The paper discusses the importance of conducting a thorough geotechnical investigation and use of advanced in situ tests to assess geotechnical parameters, particularly for the design of foundations for tall buildings in the Indo-Gangetic Alluvium. Load testing and quality assurance tests shall enhance the reliability of the predictions. Using the soil parameters and pile-load-settlement behavior as inputs into a soil–structure interaction analysis, the design may be optimized to minimize the number of piles and account for contribution of the intervening soil between the piles.

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M. Latha G. (ed.), *Frontiers in Geotechnical Engineering*, Developments in Geotechnical Engineering, https://doi.org/10.1007/978-981-13-5871-5_18

18.2 Design Approach

18.2.1 Definition of Tall Building

IS: 16700-2017 [1] defines "tall building" as a building with height exceeding 50 m. The code is applicable for building height not exceeding 250 m.

The Council of Tall Buildings and Urban Habitat [2] definitions are as follows:

- Building height > 100 m: Skyscraper
- Building Height > 300 m: Super tall
- Building Height > 600 m: Mega tall.

18.2.2 Factors Influencing Foundation Behavior

Foundation behavior is governed by the following aspects:

- strata characteristics,
- foundation type,
- magnitude of the load, and
- distribution of loads.

To limit the extent of geotechnical uncertainty in the foundation design and construction, a realistic and reliable geotechnical model of the stratigraphy, soil strength, and stiffness characteristics should be established.

18.2.3 Design Criteria

The design of the foundations should satisfy the following criteria (Quick et al. [3]):

Ultimate Limit State (*ULS*): The factor of safety against failure of foundation and the supporting soils should be adequate.

Serviceability Limit State (*SLS*): Total and differential settlement of the foundation under the working loads should not affect the serviceability of the building.

In addition, safety and stability of nearby buildings and services should not be put at risk during the construction stage or in the long-term (post-construction)—ultimate limit state and serviceability limit state.

The foundation design of high-rise buildings should be done considering performance-based soil–structure interaction (SSI). It should not limited to traditional empirically based design methods such as a bearing capacity approach with an applied factor of safety (Poulos and Badelow [4]).

18.2.4 From Concept to Construction

The various steps that govern the design and construction of tall buildings include the following:

- Preliminary studies, planning, and data collection.
- Conceptual design.
- Geological and geological investigation—may be in two stages, preliminary and detailed.
- Preliminary foundation analysis based on the geotechnical investigation and selection of suitable foundation type.
- Detailed foundation design.
- Foundation construction
  - In situ testing and
  - pile-load tests and pile integrity tests (if piles are planned) and/or footing load tests.
- Review of design based on the test results and assessment of foundation performance.

The flowchart in Fig. 18.1 describes the steps involved in initial and detailed stages of the design. The geotechnical engineer should be a part of the design team during each of these stages. Presently, for the detailed design, the role of the geotechnical engineer is usually performed by the structural engineer.

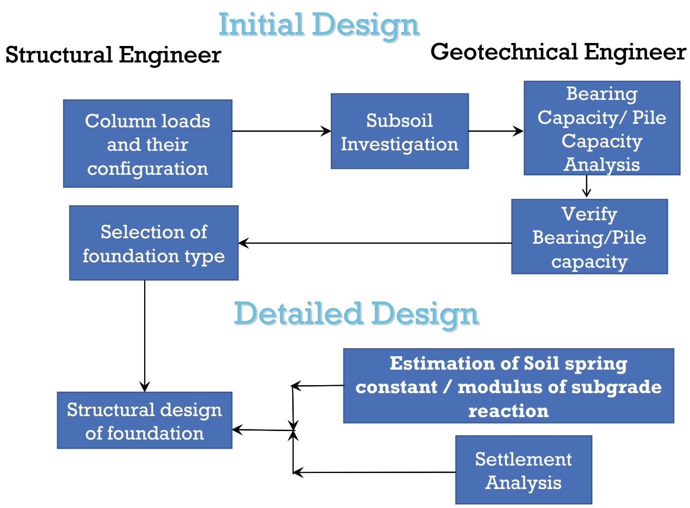


Fig. 18.1 Flowchart explaining the role of structural engineer and geotechnical engineer

18.3 Geotechnical Data

18.3.1 Initial Studies

The geological features at the site that may influence the design and performance of the foundations should be assessed so as to identify any specific measures to be taken. A desk study of the published literature and Internet search is the first step. This should be followed up with site visits to observe the following:

- topography,
- soil type, any rock exposures, and geological mapping (if required),
- groundwater level in wells in the locality, presence of nearby water bodies,
- performance of foundation of nearby buildings, details of any building failures in the vicinity, and
- local experience, etc.

These and any other information that can be obtained can provide valuable pointers that could guide the process of characterizing the ground conditions and quantifying the relevant geotechnical parameters required for foundation design.

18.3.2 Levels of Geotechnical Input

The geotechnical input that the owner/designer gets may be categorized into four levels (Sundaram [5]) as explained in Fig. 18.2.

As one moves up along the pyramid from bronze toward platinum, the factor of ignorance reduces and the reliability of design is enhanced. It also results in the reduction in foundation cost and construction time.

Fig. 18.2 Geotechnical input pyramid

For the design of tall buildings, particularly in thick alluvial deposits that are not underlain by rock within depth range of 50–100 m, it is vital to move toward Gold/Platinum level of investigations, while avoiding bronze level investigations. The soils at shallow depth of recent origin may be loose or prone to liquefaction during earthquakes. The properties of the deeper soils are vital for a good assessment of foundation settlement.

This requires detailed geotechnical and geophysical investigation and in situ testing, developing site-specific design parameters, using advanced design methods, load tests as well as construction monitoring.

18.3.3 Site Investigation

The site investigation should include a comprehensive borehole drilling and in situ testing program. For tall buildings in thick alluvial deposits, the investigation should extend to significant depths, to at least 50–100 m depth. The geotechnical data obtained should be reviewed carefully to select realistic design parameters.

In alluvial deposits, liquefaction analysis is important. It is usually done using SPT data, cone tip resistance and shear wave velocities (IS: 1893-2016[6], Youd and Idriss [7] NCEER Report). The highest level of groundwater should be carefully assessed so as to perform the liquefaction analysis for the worst condition.

SPT and SCPT. Two in situ tests are commonly used in geotechnical investigations are the Standard Penetration Tests (SPT) and the Static Cone Penetration Tests (SCPT). SPT conducted using an automatic trip hammer can give useful results in alluvial soils. SCPT gives a continuous profile of soil resistance with depth and may be used for bearing capacity and settlement analysis.

In the Indo-Gangetic alluvium, refusal (SPT N-value > 100) is usually met below 30–40 m depth. These high SPT values in the refusal stratum do not yield a fair assessment of the soil stiffness. Therefore, the designer may have to conservatively select modulus of elasticity (E-values) for settlement analysis.

SCPT conducted using a 20-tonne capacity penetrometer also encounters refusal, usually around 20–30 m depth. The test cannot be used to assess E-values in the refusal stratum.

Pressure meter Tests. Good quality pressure meter data in soils can provide very useful results which can effectively assess the E-values. The test provides deformation properties at strain levels which are commensurate with those of the ground when subjected to service loads from the building (Haberfield [8]).

However, in sands below water table that may collapse during drilling, the ground may get disturbed and result in oversized diameter of the hole. This could result in reporting lower values of deformation modulus.

Seismic Tests. Cross-hole or down-hole seismic test usually gives a good assessment of shear wave velocities with depth. This may be supplemented with seismic refraction or SASW/MASW tests to assess the lateral variation of the ground characteristics.

But E-value for small strain cannot be applied directly to foundation analysis since ground strains under dead, live and wind/earthquake loads are significantly higher than those experienced during seismic testing. The influence of the strain level should be taken into account in the test interpretation. Haberfield [8] suggests dividing the E-values from cross-hole seismic test by a factor of 5 to obtain the static E-value for settlement analysis.

In Situ Permeability. In situ permeability tests may be required in areas of shallow water table. Since tall buildings may usually have at least 2–3 basements, substantial dewatering could be required in areas of shallow water table. In sands, dewatering could be a challenge due to the high inflow.

Field permeability tests are usually done in boreholes by falling head method or constant head method. Pump-out test can give a more realistic assessment of the hydraulic parameters for the design of the dewatering system.

Experience has shown (Gupta and Sundaram [9]) that the coefficient of permeability and transmissivity measured from pump-out tests in the Indo-Gangetic Alluvium could be 1–2 orders of magnitude higher than that determined from a borehole in situ permeability test.

Laboratory Tests. A suite of laboratory tests to assess the soil classification and index properties is essential to develop a representative soil profile. This should be supported by tests to characterize strength and stiffness properties as well as consolidation properties. Conventional unconsolidated undrained triaxial tests and consolidated undrained triaxial tests (with pore pressure measurement) are usually performed in cohesive soils whereas in granular soils, consolidated drained triaxial or direct shear tests are conducted. Sufficient tests should be done so as to develop a design profile for foundation analysis.

Laboratory tests should also be done to assess the concentration of harmful salts like sulfates and chlorides in soil and groundwater.

18.4 Geotechnical Interpretative Studies

18.4.1 Selection of Foundation Type

The interpretative aspect of the geotechnical data generated is an important overlap zone between the structural engineer and the geotechnical engineer. Selection of appropriate foundation type and depth based on the loading conditions and soil characteristics sets the tone for the foundation design and construction.

The commonly used foundation systems in alluvial deposits in the Indian scenario are:

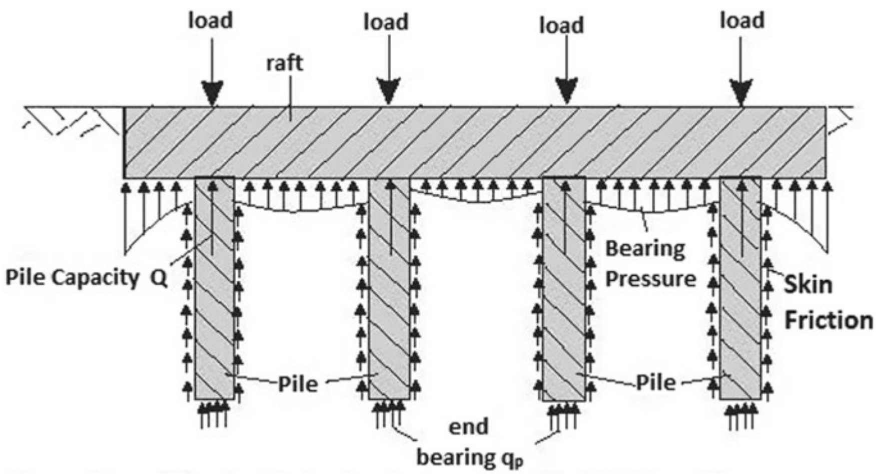


Fig. 18.3 Load transfer mechanism in piled raft

- *Raft foundations*: Loads are transferred to the ground through a raft slab covering the whole footprint of the building.
- *Pile foundations*: Piles below the columns can transfer the loads to the deeper more competent stratum. The pile caps cast over each pile group are usually connected through a beam to give rigidity to the structure.
- *Hybrid piled-raft system*: This is an economical solution for tall buildings with high foundation loads.

The hybrid piled raft is increasingly being advocated and adopted in the design of tall buildings all over the world (Kachzenbach et al. [10], Poulos [11], de Sanctis and Mandolini [12]).

It may be used to transfer the loads to the piles with the intervening soils below the raft also contributing to the load transfer (see Fig. 18.3). As a result, it has potential cost-saving and better control of differential settlement (Amornfa et al. [13]).

The contribution of the intervening soils could be substantial in stiff to hard clays and medium dense to dense sands. However, it may not be significant where the soils immediately below the raft are loose sands prone to liquefaction during earthquakes and soft to firm clays with undrained shear strength less than 50–60 kPa that may undergo consolidation/long-term settlement.

18.4.2 Piled-Raft Foundation

The piled-raft soil–structure interaction study should be done using actual column locations and loads. The single pile capacity and load-settlement behavior should be assessed from initial load tests on test piles. Zoned spring constants should be used and the settlement analysis of the piled raft should be done using PLAXIS 3D or other appropriate software to assess the load share between the piles and raft. This can be effectively used to optimize the number of piles.

In alluvial deposits, total and differential settlement and horizontal displacement, as well as stiffness of soil and pile (serviceability limit state), are the issues of primary concern that govern the design.

Shear strength and bearing capacity (ultimate limit state) should be checked although it may not govern the design.

The advantages of the piled-raft system are:

- The piles act as settlement reducers and limit the total and differential settlement.
- The piles may be loaded beyond the computed safe pile capacity (up to 70–80% of the ultimate pile capacity) since it the behavior as a piled-raft governs.
- The tilt due to eccentric loading and inhomogeneous soil conditions reduces.
- The number of piles required reduces in comparison with the pile system without raft contribution.

Amornfa et al. [13] demonstrated that in Bangkok clays, that with increasing depth of the raft, the load share of the piles reduces. For a three basement case with the foundation at about 15 m depth, the load shared by the piles reduces down to 72% of the total building loads. For rafts at shallow depth however, the contribution of the soils immediately below the raft may be less with the piles transferring substantial part of the load.

18.4.3 Foundation Design Considerations

Based on a detailed geotechnical investigation, a geotechnical model should be prepared. A proper understanding of the constitutive behavior of the soil is required to select a design profile for foundation analysis. The design considerations (Haberfield [8]) in a layered alluvial deposit are illustrated in Fig. 18.4.

Design considerations and parameters of interest in the design of piled raft are summarized in Table 18.1.

The piles are usually used up to 70–80% of the ultimate bearing capacity which is higher than the permissible safe design value for a comparable single pile. Therefore, a thorough understanding of soil–structure interaction is necessary.

18.4.4 Load Tests and Integrity Tests

To validate the design, load tests are important. Initial load tests should be performed on sufficient piles to model the pile behavior and to determine its stiffness. Static load tests traditionally performed by the maintained load method (step loading or cyclic)

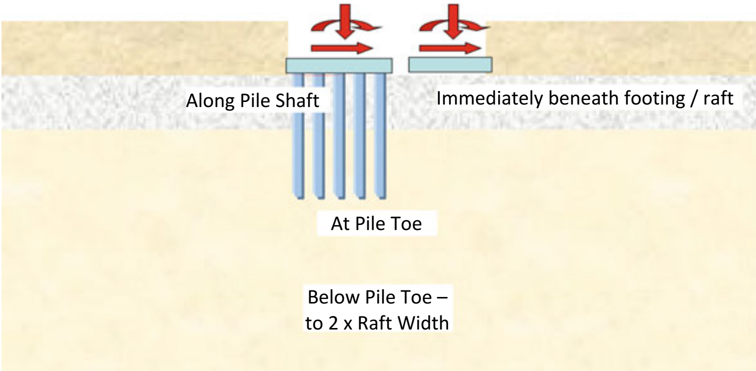


Fig. 18.4 Design considerations for raft and piled raft

Table 18.1 Geotechnical design considerations for piled raft

Zone	Parameters of interest
Immediately below raft	• Strength: bearing capacity • Settlement
Along the pile shaft	• Soil characteristics, c-ϕ values • Piles shaft resistance • Possibility of negative skin friction • Stiffness/modulus of subgrade reaction • Pile installation/drivability issues
At the pile tip	• Presence of hard strata for good end bearing • Soil characteristics, c-ϕ values • Stiffness/modulus of subgrade reaction • Pile installation/drivability issues
Below pile tip to 2 times raft width	• Soil characteristics, c-ϕ values • Stiffness/modulus of subgrade reaction

are a preferred method. But for piles very high ultimate capacity, often exceeding 1000 tonnes, it could be very cumbersome, expensive and time consuming.

High-strain dynamic load test has gained popularity in India as a cost-effective option and is now an acceptable test method in the construction industry. The test can assess the structural integrity as well as load-settlement behavior using CAPWAP analysis (Vaidya and Likins [14]).

Bi-directional static load tests using Osterberg cell has been used on some projects in India and its application is rising. It involves casting a sacrificial jack near the pile tip and can be used to test piles to high loads (Osterberg [15], George et al. [16]).

Footing load tests on 1.5–2 m wide footings can be used to realistically assess the modulus of subgrade reaction of the soil immediately below the raft and also to validate the theoretical settlement estimates (Gupta et al. [17]).

18.5 Provisions of IS: 16700-2017

The Bureau of Indian Standards has recently brought out a new code of practice, IS: 16700-2017 [1] outlining the criteria for structural safety of tall concrete buildings. Section 9.0 of this code covers foundations and has several requirements regarding geotechnical aspects.

18.5.1 Geotechnical Investigation

As per Clause 9.3.1 of IS: 16070-2017 [1], geotechnical investigation for tall buildings should comply with the following requirements:

- Geotechnical investigation should establish the safety of the building and should include liquefaction analysis and estimation of spring constants/modulus of subgrade reaction.
- Boreholes for tall buildings should be spaced about 30 m apart. Minimum of three boreholes should be drilled per tower.
- The depth of the investigation should be at least 1.5 times the raft width in soil and 30 min rock.

The authors advocate the use of static cone penetration tests and pressure meter tests for better assessment of E-values and modulus of subgrade reaction. Cross-hole seismic tests can also give a valuable input for static analysis as well as to assess liquefaction potential.

18.5.2 Foundation Depth

The embedded depth of the building shall be at least 1/15 of the height of the building for raft foundations and 1/20 of the height in case of piled raft excluding pile length (Clause 9.4 of IS: 16700 [1]). Some relaxations are available for foundations on rock and for the case of no uplift loads.

18.5.3 Soil Modeling

While modeling raft foundation through modulus of subgrade reaction, the code recommends that zoned spring constants should be used. For buildings taller than 150 m, soil–structure interaction study should be done to obtain the zoned spring constants (Clause 9.7 of IS: 16700 [1]).

Haberfield [18] highlights the importance of selection of proper values of spring constant which is a function not only of the soil but also the loading conditions and geometry.

18.5.4 Settlement

Clause 9.8 of IS: 16700-2017 refers to IS: 1904-1986 RA 2006 [19] for permissible settlement of foundations on soil and to IS: 12070-1987 RA 2010 [20] for foundations on rock.

Table 18.1 of IS: 1904-1986 RA 2006 [19] specifies permissible total settlement of 75 mm for raft foundations bearing on sands and hard clays. It also specifies a permissible differential settlement of 0.0021L (L = center-to-center distance between columns) and permissible angular distortion of 1/500 for concrete buildings.

The code permits relaxation of the permissible total settlement to 125 mm for rafts and piled rafts provided that angular distortion of the raft does not exceed 1/500.

The authors advise caution in implementing the relaxed settlement criterion and suggest that it should be backed up by the following:

- Detailed and thorough geotechnical investigation is an essential prerequisite.
- It should be ensured that the foundation is safe in shear (bearing capacity).
- Soil–structure interaction using FEM software such as PLAXIS 3D to assess settlement, differential settlement, stresses, etc. should be done.
- Monitoring settlement during the construction period will not only enhance the reliability of prediction but also give advance warning that can help prevent potential failure before it occurs.

On the basis of the review of over 52 case studies, Zhang and Ng [21] suggest limiting tolerable total settlement to 106 mm and angular distortion to 1/500.

18.6 Quality Assurance

The best of designs can fail if the requisite quality is not maintained during construction. Routine static load tests are usually done as per IS: 2011 (Part 4)-2013 [22] to 1.5 times the design safe load. Bi-directional static load test provides time saving and economical option, particularly for high load-carrying-capacity piles.

The current industry practice in India on all major projects is to supplement this with low-strain pile integrity tests (PIT) on 100% of the piles (Likins et al. [23]). Cross-hole sonic logging (CSL) is also being used to locate construction defects in piles.

The results of PIT and CSL should be used to select appropriate piles for static load test and/or high-strain dynamic load tests (HSDLT). This will form the basis of acceptance of the piles and ensure quality assurance.

If any piles fail during load test or HSDLT, coring through the pile is a good option (Gupta et al. [24]) to reconfirm the pile quality. In such case, redesign of the piling system shall be required based on the actual pile stiffness and lower capacities.

18.7 Case Study

18.7.1 Project Details

This case study gives details of foundation design for a 38-storeyed building with two basements under construction in Noida (UP) located east of Delhi, not very far from the River Yamuna. The project, described as mixed land use, offers high-end commercial, retail and residential units.

18.7.2 Regional Geology

The deposits in the area belong to the Indo-Gangetic Alluvium and are river deposits of the River Yamuna and its tributaries. The Pleistocene and Recent deposits in the project area are composed primarily of sands and silts.

The Indo-Gangetic alluvial tract in the nature of a synclinal basin formed concomitantly with the elevation of the Himalayas to its north (Krishnan [25]). It was formed during the later stages of the Himalayan Orogeny by the buckling down of the northern border of the peninsular shield beneath the sediments thrust over it from the north.

The newer Alluvium, locally called *Khadar*, consists primarily of fine sand that is often loose in condition at shallow depths. The older alluvium, locally called *Bhanger*, consists of compact sands and silts and is generally, rich in concretions or nodules of impure calcium carbonate (kankars).

18.7.3 Initial Investigation

The builder got an investigation done consisting of seven boreholes to 30 m depth and three boreholes to 40 m depth. These boreholes were drilled after excavating to 6 m depth.

The boreholes revealed the presence of alluvial sands to 40 m depth with occasional sandy silt zones in some boreholes. Groundwater was met about 1.5 m depth below the excavated level. Typical borehole profiles are presented in Fig. 18.5.

Based on the borehole data, the soil investigation agency had recommended safe axial compressive capacities of 300 and 350 tonnes, respectively, for 30 and 35 m

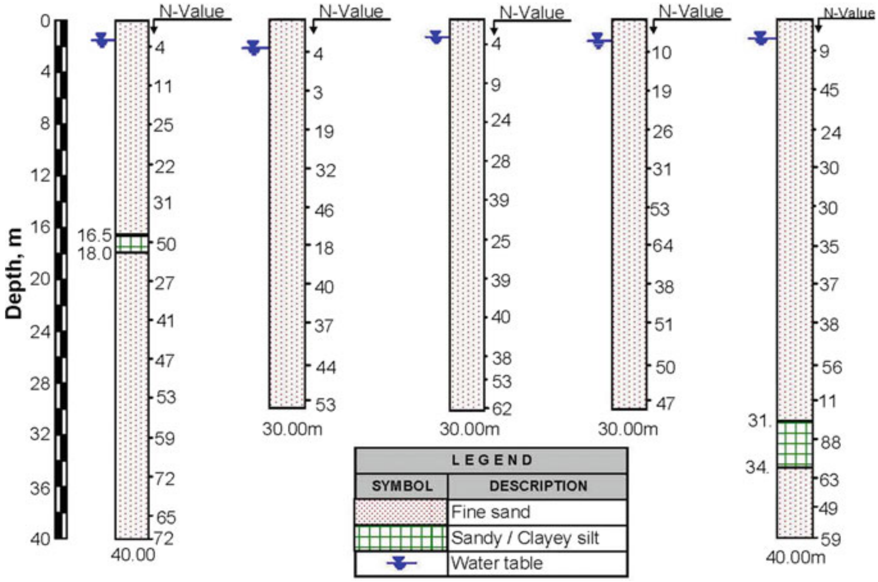


Fig. 18.5 Typical profiles of boreholes drilled for initial investigation

long 1000 mm diameter RCC bored piles. The corresponding values given for a 1200 mm diameter pile were 420 and 480 tonnes, respectively. These values are typical of the capacities used on most other sites in the surrounding areas within a 4–5 km radius of the project.

18.7.4 Initial Load Tests

Initial load tests were performed on 1000 and 1200 mm diameter piles of lengths 30–35 m. A photograph of the load test in progress is presented in Fig. 18.6. Test results are summarized in Table 18.2.

As illustrated in Fig. 18.7, the pile-load test results are fairly inconsistent/scattered with no clear trend. Many piles (especially TP-1, 2, and 3 of 1200 mm diameter piles; and TP-1 of 1000 mm diameter) significantly underperformed, indicating possible structural defects in the pile and/or poor bottom cleaning.

On the contrary, some of the piles (such as TP-2, 3, and 4 of 1000 mm diameter) performed rather well. The safe pile capacities substantially exceeded the computed theoretical capacities.

Fig. 18.6 Pile load test in progress

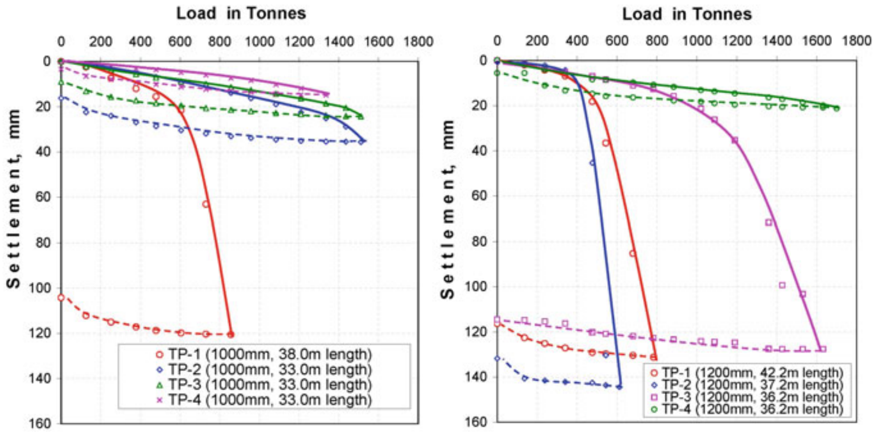
Table 18.2 Results of static load tests on initial piles

Pile no.	Pile dia (mm)	Pile length (m)	Interpreted safe capacity (MT)[1]	Type of slurry used
TP-1	1200	40.0	317†	Bentonite
TP-2		35.0	280†	Bentonite
TP-3		34.0	613	Composite*
TP-4		34.0	1000	Composite*
TP-1A	1000	35.0	353	Polymer
TP-2A		30.0	686	Polymer
TP-3A		30.0‡	860	Polymer
TP-4A		30.0‡	>890	Polymer

‡Gravel pad provided at pile tip
†Possible construction defects resulted in lower capacity
*Polymud + Alfabond (a thickening agent) and bentonite
[1] Safe pile capacity interpreted in accordance with IS: 2911 (Part 4)-2013 [22]

18.7.5 Detailed Geotechnical Investigation

Having gained confidence that well-constructed piles shall be able to carry loads substantially higher than that computed based on the initial geotechnical investigation, it was decided to perform an additional geotechnical investigation which included



(a) 1000 mm diameter pile **(b)** 1200 mm diameter pile

Fig. 18.7 Results of pile load tests

Fig. 18.8 Boreholes and pressure meter tests in progress

three boreholes to 60 m depth, pressure meter tests in two boreholes and a cross-hole seismic test (Fig. 18.8).

Figure 18.9 presents typical borehole data.

Figure 18.10 presents typical pressure meter test data showing the profile of limit pressure and deformation modulus with depth.

Primary and shear wave velocities and dynamic shear modulus and dynamic Young's modulus from cross-hole seismic test are presented in Fig. 18.11.

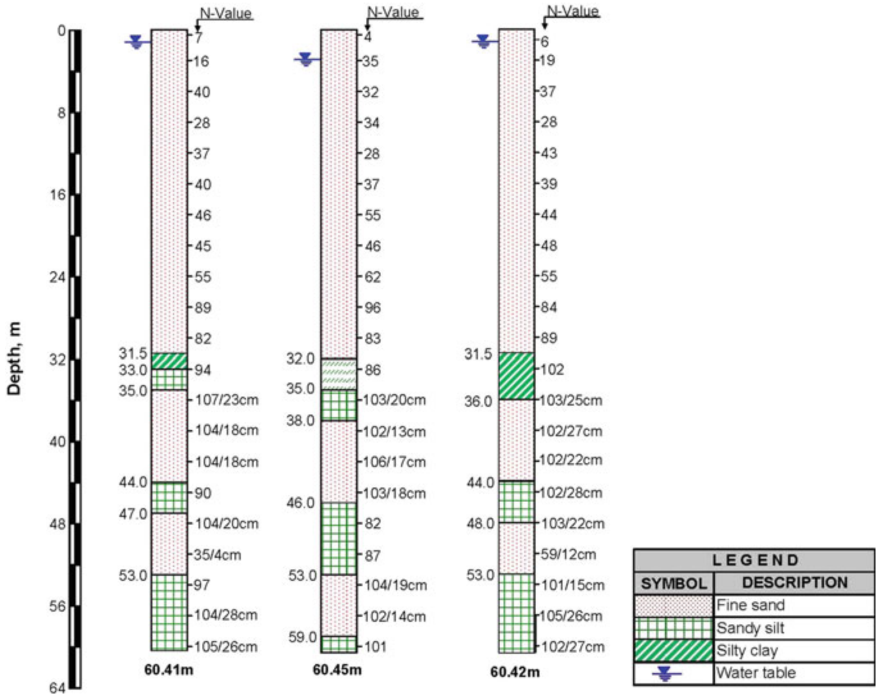


Fig. 18.9 Data from deep boreholes drilled for piled-raft analysis

18.7.6 Design Profile

Reviewing the soil characteristics, SPT values, pressure meter modulus, laboratory tests, etc., soil parameters selected for analysis of statically loaded foundations are presented in Table 18.3.

The design groundwater level was considered at the existing ground level for the worst condition. Poisson's ratio, μ, for the soil was taken as 0.33.

18.7.7 Computed Pile Capacities

Pile capacities under compression loading has been computed based on the c-ϕ values as per IS: 2911 (Part 1 Section 2) 2010 [26]. Pile capacities were also computed using the pressure meter design rules (Clarke [27]). Analysis was done for 1000 and 1200 mm diameter bored piles with cutoff level at 8.0 m depth below average ground level. The computed pile capacities are presented in Table 18.4.

The above values include a factor of safety of 2.5 in accordance with IS: 2911 (Part 1 Section 2) 2010 [26].

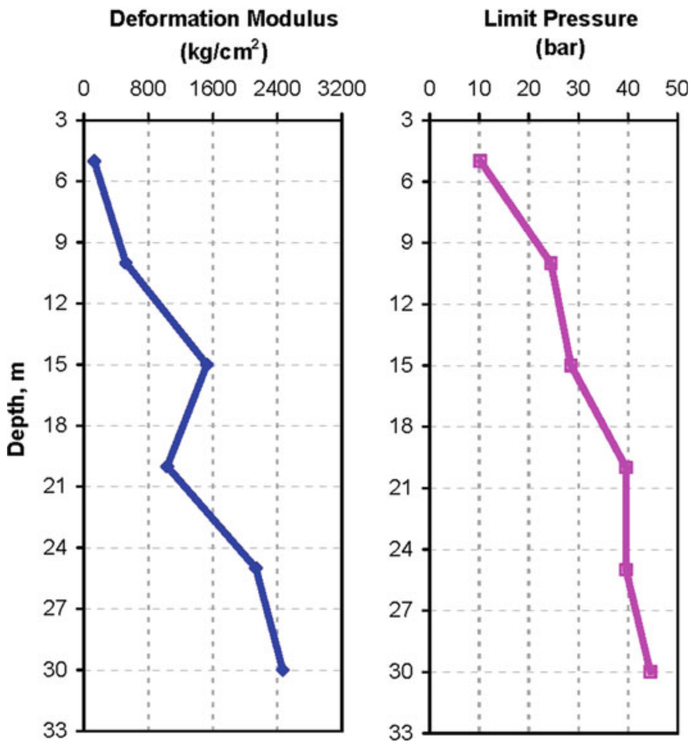


Fig. 18.10 Pressure meter modulus and limit pressure versus depth—typical results from one borehole

18.7.8 Piled Raft

Piled-raft foundation was used to optimize and economize the design. The analysis was done considering 1000 mm diameter pile of length 35 m installed below cutoff level of 8 m. The raft thickness was taken as 3.5 m in consultation with the structural engineer.

Piles were introduced at strategic locations based on the column loads. The pile locations were finalized after several hit and trials to achieve maximum economy ensuring that the load on any individual pile does not exceed 900 tonnes (2/3 of the ultimate pile compressive capacity of 1350 tonnes).

Figure 18.12 illustrates the pile layout finalized for the project.

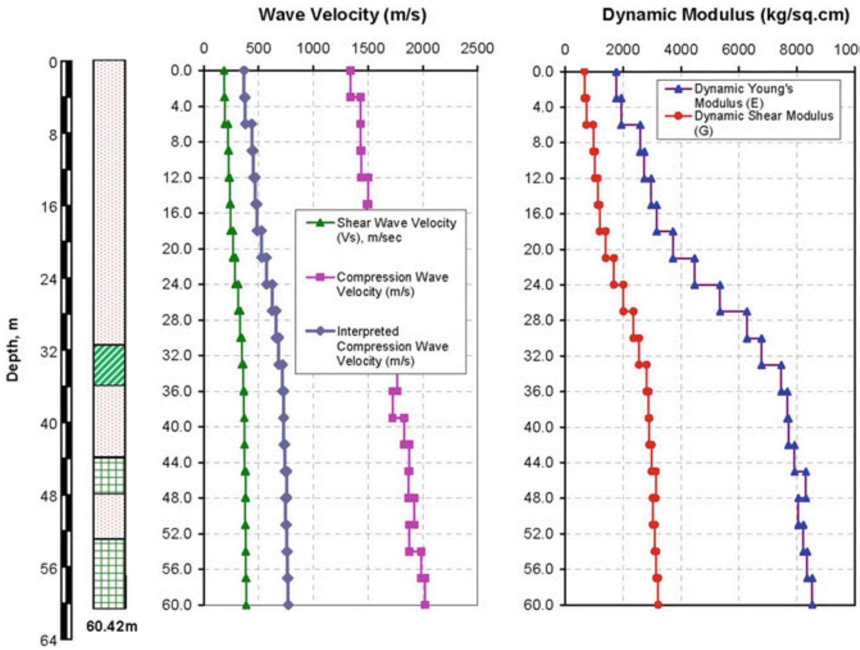


Fig. 18.11 Wave velocities and dynamic moduli from cross-hole seismic test

Table 18.3 Design profile

Depth (m)		Soil classification	γ (kN/m^3)	c (kN/m^2)	ϕ°	E (kN/m^2)	E_{PMT} (kN/m^2)	l_m (kN/m^2)
From	To							
0.0	2.0	Silty sand/fine sand	18.0	0	30	3000	10,000	500
2.0	10.5		19.0	0	31	6200	70,000	2000
10.5	13.0		19.5	0	32	29,600	100,000	2500
13.0	23.0		20.0	0	33	43,700	160,000	3600
23.0	33.0		20.0	0	33	62,400	220,000	4000
33.0	36.0	Clayey silt	21.0	0	33	90,000		
36.0	44.0	Silty sand	21.0	0	33	124,800		
44.0	47.0	Clayey silt	21.0	90	6	124,800		
47.0	53.0	Fine sand	21.0	0	34	124,800		
53.0	60.0	Sandy silt	21.0	110	5	124,800		

where
γ = Bulk density of soil
c = cohesion intercept
ϕ = angle of internal friction
E = Modulus of elasticity of soil
E_{PMT} = Pressure meter (deformation) modulus
l_m = Limit Pressure

Table 18.4 Pile capacities under compression loading

Pile diameter (mm)	Pile length below COL	Safe compression pile capacity (kN)		
		c-ϕ values	Pressure meter	Selected design value
1000	30	4640	4800	4700
	32	4900	5100	5000
	35	5290	5560	5400
	40	5940	6310	6000
1200	30	6960	7390	7000
	32	7340	7750	7500
	35	7910	8540	8100

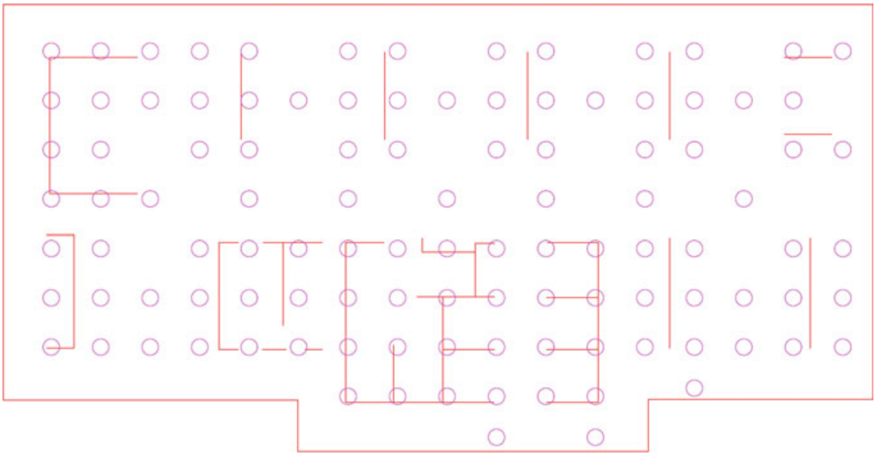


Fig. 18.12 Pile layout

18.7.9 PLAXIS Modeling

Initial analysis using the PLAXIS 3D software was done for raft without piles. However, settlement of a raft foundation was found to be about 330 mm which is higher than the permissible 75 mm total settlement specified in IS: 1904-1986 RA 2006 [19] for raft foundation. Differential settlements were also beyond permissible limits and the angular distortion was about L/400.

Thus, the analysis confirmed that piles are required to be installed as settlement reducers. Piles were therefore introduced in the finite element model below the raft at strategic locations within the high settlement zones.

Pile-load tests conducted at the site were simulated in the FEM model for calibration of the model, using soil profile and soil E-values given in Table 18.3. It was observed that soil E-values had to be reduced by 40% based on single pile simulation,

Table 18.5 Summary of piled raft settlements and pile loads

Load combination	DL + LL
Pile diameter	1000 mm
Pile length	35 m
Maximum raft total settlement	74.3 mm
Maximum pile load	8900 KN
Maximum angular distortion	L/1200

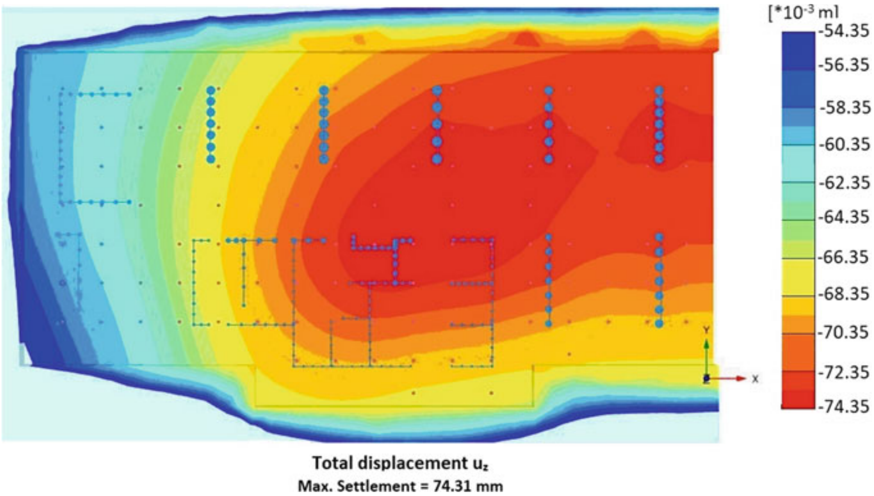


Fig. 18.13 Settlement contours

for the pile settlements obtained by the software to match the field pile settlements. Loads distribution and settlements were assessed to optimize the foundation design.

18.7.10 Results of Analysis

Based on detailed analysis, the proposed building can be supported on a piled-raft foundation as per the configuration in Fig. 18.12. Maximum settlements of the piled-raft foundation and pile loads are summarized in Table 18.5.

The maximum total settlement is less than the permissible. Differential settlement/angular distortion is also less than permissible value of L/500. Figure 18.13 presents the settlement contours. Figure 18.14 presents the pressure distribution at the soil–raft interface.

The spring constants for soil and for piles, computed as the ratio of contact pressure and settlement are presented in Figs. 18.15 and 18.16 which were used by the structural engineer for the design of foundation.

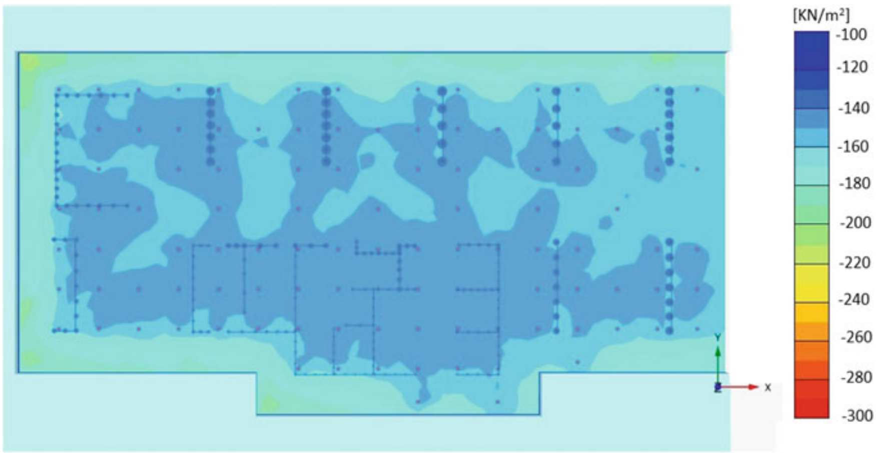


Fig. 18.14 Pressure contours at soil–raft interface

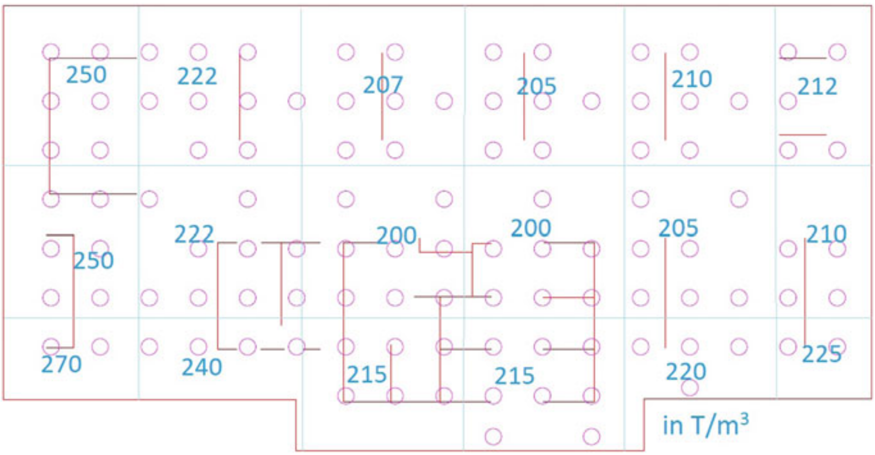


Fig. 18.15 Soil spring constant

18.7.11 Cost-Saving Realized

To illustrate the financial saving achieved by thorough geotechnical investigation and soil–structure interaction analysis, the foundation costs is summarized in Table 18.6.

It may be seen that the cost of the detailed geotechnical investigation and the SSI is significantly less than the savings in the foundation cost. While the number of piles decreases, the reliability of the design increases substantially.

The authors of the opinion that further saving is possible if the following in situ tests are performed prior to starting work on the routine piles:

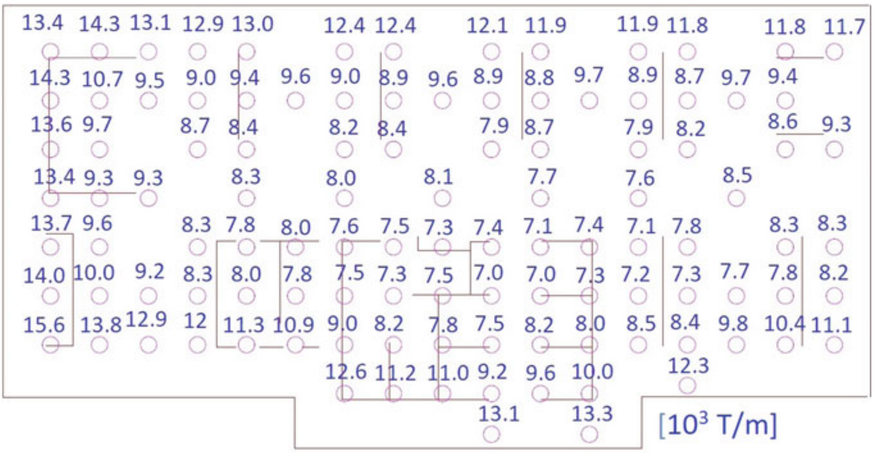


Fig. 18.16 Pile spring constant

Table 18.6 Cost-savings analysis

Design stage	Load on pile* (MT)	No. of piles	Estimated piling cost (in million rupees)	Savings† (in million rupees)	Consultancy cost (in million rupees)
Based on limited geotechnical investigation	300	550	440.0		₹ 0.5
Based on detailed geotechnical investigation	536	500	400.0	₹ 40	₹ 2.0
Piled raft (based on assumed spring constant)	536	358	286.4	₹ 153.6	–
Piled raft system—SSI using PLAXIS	900#	221	176.8	₹ 263.2	₹ 0.25
Direct project cost saving				₹ 263.2	₹ 2.75

*as per the design based on the safe pile capacity
#approximately 67% of ultimate load carrying capacity of pile
†Savings in cost of piling in comparison to that based on limited geotechnical investigation without SSI

- Footing load tests on 1.5–2 m size footing may be used to assess the E-values and spring constant of soil effectively.
- Sufficient number of high-strain pile integrity tests and/or bi-directional static load test on initial test piles used in conjunction with the static pile-load tests can give a better assessment of load-settlement behavior of piles that can be input into the PLAXIS model for the piled-raft analysis.

18.8 Concluding Remarks

In the Indo-Gangetic alluvium in northern India, sands and silts are encountered to substantial depths. These soils are usually loose to medium dense at shallow depths and very dense (SPT N > 100) below 30–40 m depth. Construction of tall buildings in such soils needs a thorough geotechnical investigation. This should be followed up by soil–structure interaction using PLAXIS or other software.

Piled-raft foundation is increasingly being used for the design of tall buildings. It takes advantage of the load sharing of the intervening soils and thus can optimize the foundation design.

With IS 16025-2017 permitting up to 125 mm total settlement for raft foundations and piled rafts, the following issues are critical:

- Detailed geotechnical investigation should be performed.
- Liquefaction potential assessment should be done for highest water table expected.
- Assessment of modulus of subgrade reaction is an important aspect.
- Differential settlement/tilt should be restricted to 1/500.

For piled rafts, the piles are usually loaded to 70–80% of the ultimate pile capacity. Therefore, quality assurance of pile construction is essential to ensure that the piles behave as designed. Conventional static load tests should be supplemented with high-strain dynamic pile-load tests and bi-directional static load test (O-cell) for a realistic assessment of pile-load settlement behavior.

Quality assurance tests such as low-strain pile integrity tests and cross-hole sonic logging should be an integral part of field testing of the piles. This will assist in optimizing the design and ensuring that the piles are capable of supporting the applied loads.

A case study of a 38-storeyed building in Noida presented in this paper demonstrates the effectiveness of the design and the advantage achieved by conducting a thorough geotechnical investigation and soil–structure interaction analysis. Savings of nearly Rs. 260 million were realized on the piling costs in comparison to what the owner would have spent based on a simplistic limited geotechnical investigation.

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