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INVITED PAPER

Case Studies in Geotechnical Engineering Constructions

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Abstract The enormous growth in the geotechnical activity in the country has resulted in slackening of the quality in all the three aspects of investigation, design and constructions endangering the safety of structures. The geotechnical engineering practice has now reached a stage of scientific maturity where many innovative solutions can be provided to solve field problems which had not been possible earlier. In the following paper case studies pertaining to each of the deficiencies of geotechnical activities have been reported.

Keywords Case history · Investigation deficiency · Innovative solutions · Reinforced earth · Soil nailing · Under-slab drainage

Introduction

Innovative geotechnical solutions to field problems more often than not do not follow the standard codes of practice. In fact the codes may follow after successful implementation of such innovative solutions. One such example is the reinforced earth concept by Henry Vidal [1] which has been identified as the most innovative concept of last century in Civil Engineering Constructions. The recent increase in the rate of urbanization has thrust on the geotechnical engineers many new challenges such as deep excavations, poor and marginal foundation soils and underground constructions. The conventional solutions may be risky and cost prohibitive. Further these challenges have brought in many new and inexperienced players into the fields of geotechnical investigation, design and construction. This has resulted in the quality deficiency of each aspect, prohibitive cost and endangered the safety of the structure. The following article is aimed at highlighting the deficiencies in the above three activities which affected the safety of the structures. Some cost effective and innovative solutions for rectification are presented.

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Deficiency in the Geotechnical Investigations and Poor Selection of Foundations

This is about a multistory hotel building in Chennai. The construction of the hotel building of size 18 m × 36 m in plan with two basements, ground floor, mezzanine floor and nine upper floors was started in 1986 and was to be completed in 1991 (Fig. 1). The lifts were ordered and the lift manufacturer marked plumb lines on the lift well walls. After 3 months, when installation of the cage started a recheck of the plumb lines indicated that the originally marked lines were out of plumb by more than 75 mm. This created panic among all the players. Prof. A. Sridharan and the author were requested to be the Consultants to examine and investigate the problem and to provide suitable solution. The owners were requested to provide the details of soil investigation reports, structural design basis report from the structural consultants and the as built structural drawings.

From the available geotechnical report it was clear that the investigation had been carried out to a maximum depth of 3.0 m and at only two locations and recommendation of the SBC was 12 t/m^2 at 3 m depth. But the actual construction with two basements, the foundation level was at 5.4 m from the original ground level. The structural consultant had extrapolated the depth effect to adopt a safe bearing pressure of 18 t/m^2. The structural design basis report indicated very conservative approach had been adopted for the structural designs. The structural distribution of foundations is presented in Fig. 2. It is totally unconventional with combined strip footings and rafts randomly arranged. The structural design basis report and the drawings indicated that the total load on the foundation will be more than 18 t/m^2 even with complete raft foundation. A detailed analysis of loads on all columns and footings indicated that the intensity of pressure varies from 14 t/m^2 to more than 20 t/m^2 under different footings.

Analysis and Solution

A detailed leveling survey was conducted around the building and plumb measurements were taken on all the four sides. It was noticed that the building has tilted diagonally along DB diagonal of length of about 40 m by more than 250 mm while along the diagonal AC, there was no significant difference. But between D and C corners and A and B corners, the tilting was of about 125 mm. It was decided to get soil investigations done again at four corners of the building to a depth of 16 m from the ground level which could give the ground characteristics to a depth of 10 m below the footing level. Figure 2 provides the locations of the 4 bore holes A, B, C and D with reference to the building. The soil profile indicated the presence of water table at the footing level of about 5.5 m below the ground level. Typically details of two bore holes B and D are presented in Fig. 3. The geotechnical profile details of A and C boreholes were the average of the details of B and D boreholes.

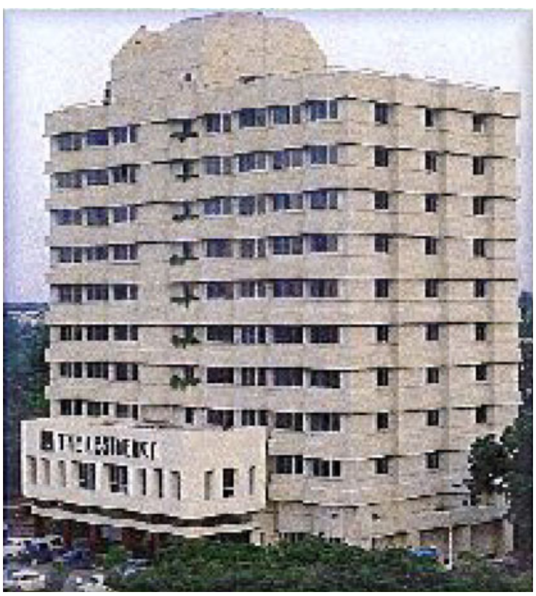


Fig. 1 Completed building

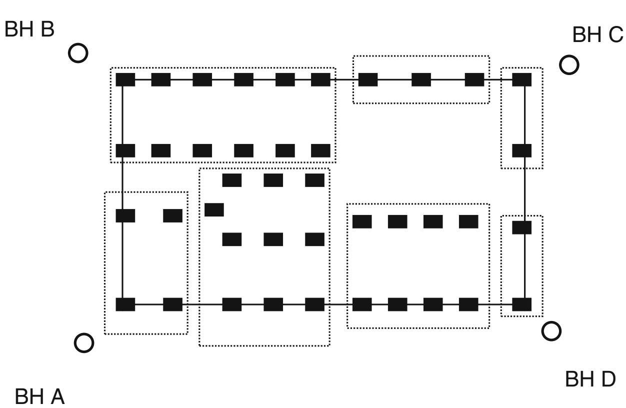


Fig. 2 Foundation plan

The soil profile of borehole B below the footing level indicates the presence of two layers of highly compressible clay, one of thickness of about 1.2 m at 6 m level (within 0.6 m below the footing of more than 10 m wide) and the other of 1.7 m thick at 8.5 m level (within 3 m from the bottom of the footing) indicating their presence within the significant depth below the footing. In contrast the details of bore hole D indicate the presence of one layer of clay at a depth of 4 m below the footing level. The footing width at D corner is about 2 m indicating the presence of clay layer is just below the significant depth and does not have major contribution for the settlement of the building. Hence it was concluded that the raft footing in the region of B which has a higher bearing pressure of 20 t/m^2 and the presence of two clay layers with in the significant depth below the 10 m wide footing has caused excessive settlement when compared to the smaller 2 m wide footing in the D borehole area which has no clay layer within the significant depth and the soil pressure is only about 14 t/m^2. In the bore hole area A and C the thickness of compressible clay layer of about 1.2 m thick is at about 2.5 m below the foundation level which has caused some settlement. The measured settlement of A, B and C corners tallied well with the computed values from the compressibility properties from geotechnical investigations.

Necessary consolidation tests were conducted and analysis was carried out about the time required for complete consolidation and it was found that almost 90 % consolidation has already completed. This may be due to the sandwiched nature of the clay layer between the two layers of silty sand. However continued increase in differential settlement may be also due to lateral squeezing of the soft clay layers below zone of bore hole B. The building had been provided with the same column cross sections for all the 13 floors introducing an increased rigid behavior. The eccentricity of the load on the columns caused due to differential settlement around bore hole B were creating increased stress conditions

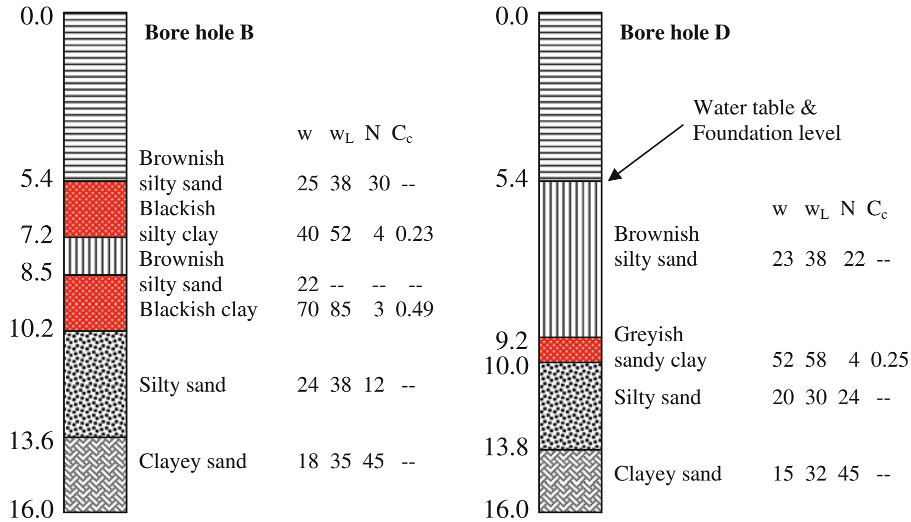


Fig. 3 Soil profile in Borehole B and D

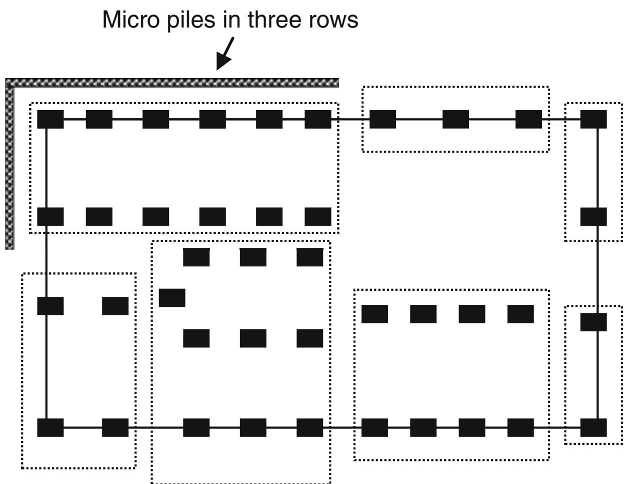


Fig. 4 Scheme of remedial measures

but were within the crushing strength of concrete with reduced factor of safety.

The remedial solutions had to achieve an immediate safe condition for the inauguration of the Hotel. The first of the remedial solutions was to arrest the increase in differential settlement. Since the time was too short it was decided to adopt parallel actions pending the complete analysis. The following sequential measures were adopted (Figs. 4, 5).

1. The lower basement floor in the zone of bore hole D was completely empty and 800 tons equivalent of sand bags was loaded on the floor to produce an intensity of pressure of about 8 to 9 t/m^2 in that corner. This really had some good initial effect of not increasing the differential settlement between D and B corners due to immediate settlement of the silty sand layer of about 4 m thick. Since the clay layer below the corner D was also getting influenced by the sand bag loading and the compressibility of this layer was time dependent and the differential settlement value started fluctuating.
2. The area around borehole B had the soft clay layers sandwiched between silty sand layers up to about 5 m below the footing level. It was decided to increase the density of sandy silt layer and also to provide a curtain to prevent further lateral squeezing of soft clay layers. It was decided to install end closed 100 mm dia. MS pipes as micro piles of 10 m long @ 250 mm c/c in 3 rows with staggered arrangement. The pile tips were planned to reach middle of the 10 m wide raft footing in that zone. For this inclination selected was about 60° with horizontal. The total number of piles driven in this quadrant was 330. The driving arrangement was from the original ground level which itself was about 5.5 m above the foundation level. A suitable tripod and driving dolly with chain pulley arrangement was made to slide the weight from 16 m height. The outer row was installed first and other two rows were installed later such that densification process moves inwards.
3. The pipes were filled with M20 concrete after installation. The top of the micro piles were connected to a designed reinforcement cage and 1 m wide 250 mm thick RCC pile cap was constructed beyond the building edge at 2 m below the ground level embedding the top of the micro piles in the pile cap. The load-settlement curve of the B corner stabilized with a net differential settlement of about 340 mm and there was no increase in settlement over next 4 months. But there was some apprehension about the secondary consolidation effect. At this stage it was decided to make an attempt to reduce the differential settlement by about 50 %.
4. The soil stratum at D corner was indicating the presence of sandy-silt layer to a depth of 4.5 m below the foundation level. The ground water table was present at foundation level and pumping of water from bore well could as well make the sandy silt to flow into the bore-well from the space below the foundation.

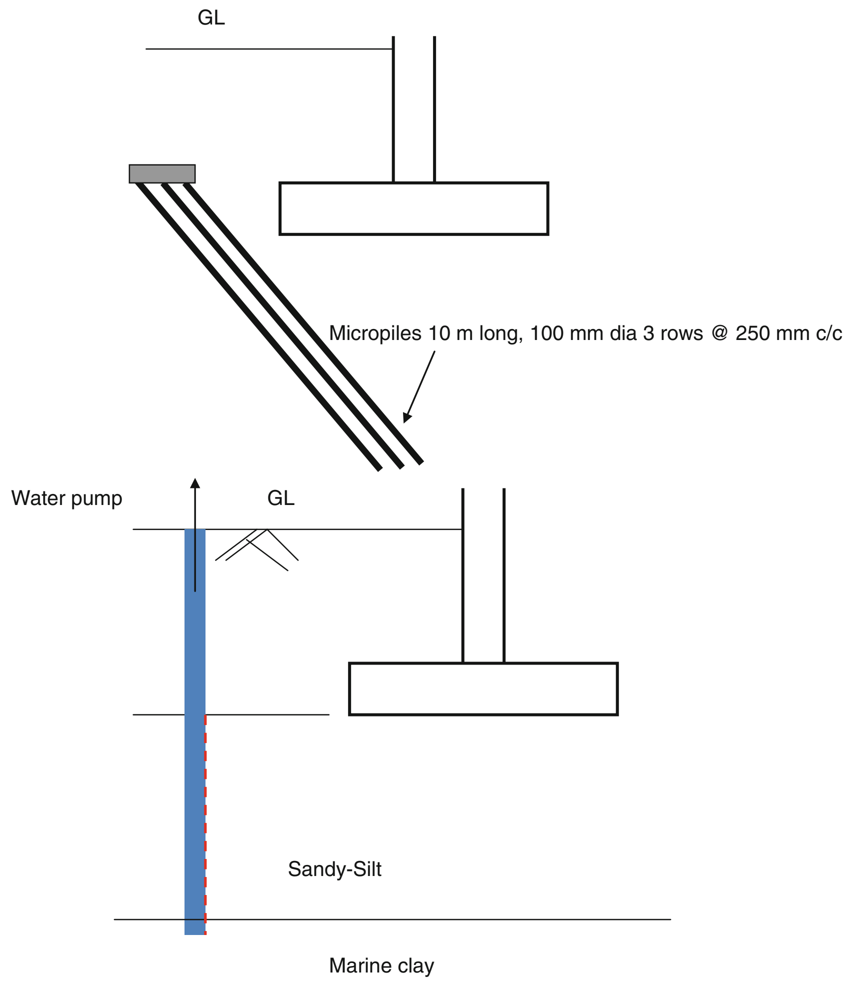


Fig. 5 Details of remedial measures

It was decided to install about 6 bore wells of 300 mm dia. to a depth of 10 m from the ground level. The casing pipes were made of 12 mm wide half slots at 100 c/c in the half of pipe facing the foundation. With slotted casing pipe in position, pumping of water resulted in flow of sandy silt into the bore well from the space below the footings of D borehole area. Weight of about 50 kg of sandy silt removed from each of the bore wells resulted in the settlement of the footings in the area by about 1 mm. Each day about 50 kg of sandy silt was removed from each of the bore holes for about 200 days which caused a decrease in the differential settlement to less than 120 mm.

5. This caused about 200 mm of induced settlement in an area of about 20 m × 12 m. Further action was stopped since the owners were happy with what had been achieved. Further monitoring of the movement was done for more than 3 years on monthly and quarterly basis and found no further settlement.
   The technical detail of the above concept of inducing controlled settlement was reported in a Conference on Case Studies [2] and has been referred to while rehabilitating the Pisa Tower in 2001.

THE GEOTECHNICAL FAILURE ANALYSIS AND REHABILITATAION OF A REINFORCED EARTH WALL

A geo-grid reinforced earth retaining wall for a fly-over along the National Highway No. 4 was constructed a few years back. The wall height is varying from 2 to 12 m. At a location of about 7–8 m height of the wall there was snapping of connection between facing element and the geo-grid reinforcement and there was sudden failure of the RE wall. The failure had taken place over a length of about 10 meters (Figs. 6, 7, and 8).

Analysis and Solution

The soil in the failed zone appeared to be in good condition as required from an RE wall point of view. In fact the failure plane had not gone beyond a meter from the wall face which was the boundary of the drainage zone of granular material. The soil behind the failed zone was standing near vertical indicating the sufficiency of the

Fig. 6 Collapse of RE wall

Fig. 7 Collapse of RE wall

Fig. 8 Exposed friction slab

reinforcement layers and good quality of compaction. The failure was essentially in the form of collapse of the panels due to snapping of the connecting polyester cleats/clamps, perhaps initially of the upper layers. The number of geo-grid reinforcement layers provided was found to be reasonable. There was variation in the grade of geo-grid increasing with depth.

The RE walls generally will have a friction slab of RCC and support crash barrier. In the present case the friction slab was of 400 mm thick and about 2.4 m wide which also formed part of the traffic lane (Fig. 9).

A closer examination of the detailing and the site condition of failure indicated that the failure has initiated from snapping of upper layers of geogrid/connections. This was indicative from larger elongation of the upper layers of geo-grid.

The wheel loads will transmit the load and vibrations differently in the two systems. In rigid pavement portion the vibrations are transmitted down into the granular medium whereas the same is damped/dissipated in the flexible pavement.

The transmitted vibration will tend to compact the granular soil if the density is lower than the density corresponding to critical void ratio of the granular material resulting in the loss of contact of the friction slab and soil. This will in turn make the friction slab to transmit the reaction due to the wheel load to the facing element. As the facing element assembly is not designed to transmit the axial loads the reaction will tend the assembly of facing element to *buckle* causing the snapping of the cleats/joints

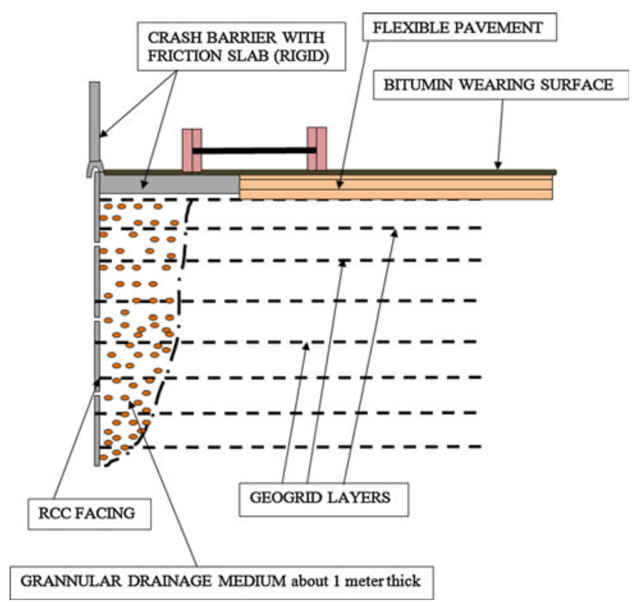


Fig. 9 Schematic diagram of RE wall

between the facing and the geo-grid. This is the primary reason for the failure (Fig. 10).

It was found from the un-failed zone the density of granular free draining soil is much lower than that corresponding to critical void ratio which was determined in the laboratory. The rehabilitation scheme had to be such that the original concept and view needed to be maintained. It was decided to provide sand bag shoring from the service road without disturbing the traffic such that the traffic can be allowed on the fly-over. A small toe wall was built in masonry to restrict the spread of sand bag into the service road lane (Figs. 11, 12, and 13).

Providing a suitable staging, sand bags were removed from top to a convenient depth of 1.65 m (Fig. 14). The failure plane was trimmed to remove loose materials. Six meter long 20 TOR bars were driven at a spacing of 550 mm c/c (three rows) in both directions. 200 gsm Geotextile non-woven fabric was fastened to the excavated surface by punching holes for the nails and the same was protected by fixing 50 × 50 × 2.8 mm MS weld mesh to the nails in front of the fabric.

The steps were repeated to reach the bottom level in increments of 1.65 m. Further the old facing elements were incrementally erected and with additional steel cleats connected to the driven nails. The gap was filled with sand and compacted by vibration. Finally core-cutting the friction slab from top the last layers of sand were filled (Figs. 15, 16, 17, and 18).

To avoid similar buckling failure of the facing elements for the entire length of RE wall, it was decided to provide additional frictional and tensile strength for the upper layers of the RE wall along the entire length by driving six number of 6 m long 20 mm dia. bars per facing element in the top 3 meters.

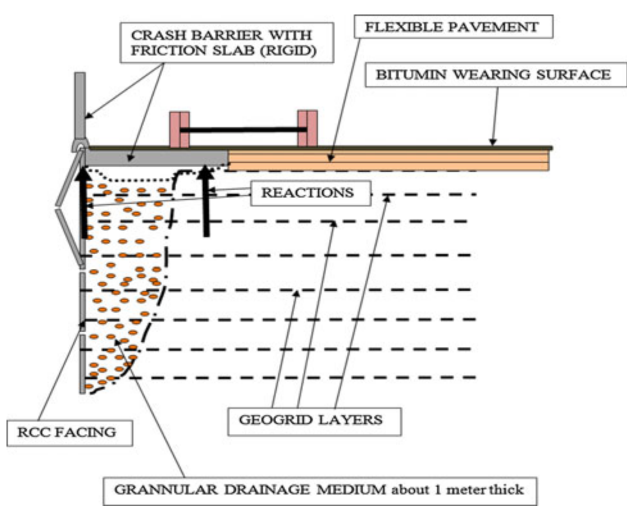


Fig. 10 Schematic diagram of failure condition

CONSTRUCTION OF 108 VILLAS ON 6 M THICK FILLED UP SOIL

The case of a filled up soil over soft clay where more than 100 villas of 3 floors were planned had the

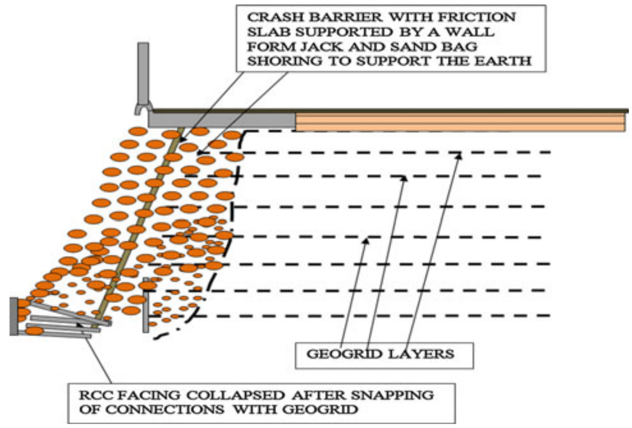


Fig. 11 Schematic diagram of sand bag shoring

Fig. 12 Toe wall support at base for sand bags

Fig. 13 Completion of sand bag shoring

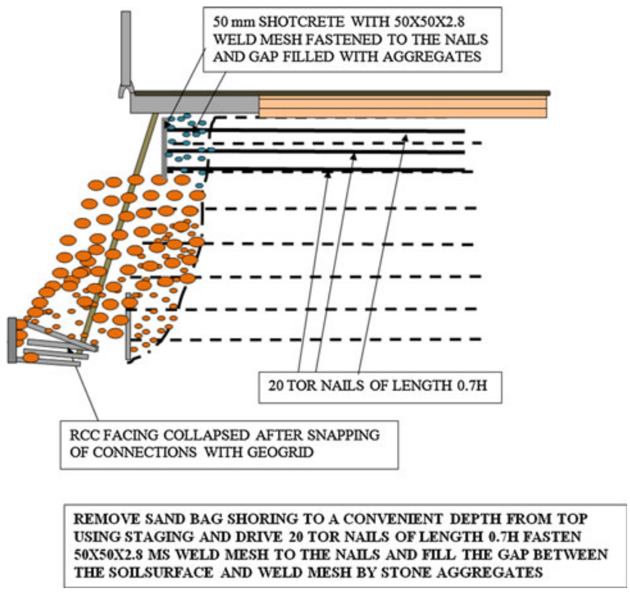


Fig. 14 Incremental removal of sand bag shoring and soil nailing

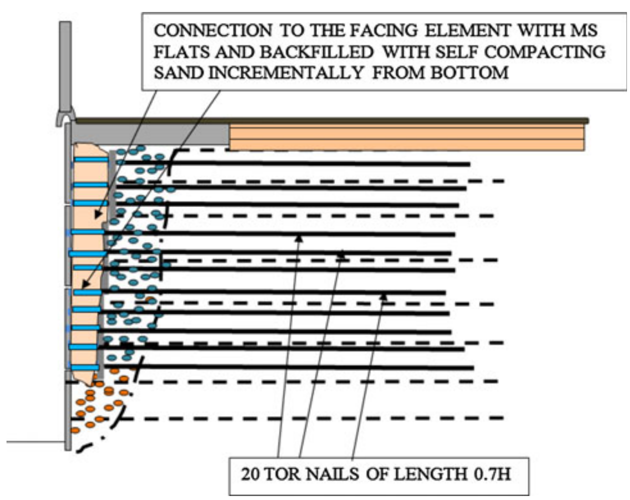


Fig. 15 Schematic diagram of completed rehabilitation

recommendations from the structural consultant to install more than 4,000 number of piles of 12 m long which involved prohibitive cost and time delays. The case was referred to the author for an alternative solution.

Analysis and Solution

The geotechnical report indicated 4–6 m thick filled up soil by just dumping over years. The sandy silt had *N* randomly varying from 3 to 20. Below this layer there was 4–5 m thick tank bed fat clay with *N* varying from 3 to 10. The water table was also reported to be between 3 and 4 m. The *N* value of the weathered rock stratum below the clay layer was varying from 20 to 50. The recommendation was to

Fig. 16 Filling of sand in the gap behind the facing element

Fig. 17 View of replacement of facing elements

Fig. 18 Finished view of RE wall with additional driven nails

place the end bearing piles on the weathered rock. The sandy silt soil had liquid limit of 30–38 % while the plasticity index was between 12 and 18 %. The clay had the LL of 48–64 %.

The details of the proposed Villas indicated the foot prints to be about 160 m^2. The total load of the building was about 650 t indicating a required bearing pressure of just less than 5 t/m^2 for raft slab. The filled up soil could provide this level of bearing pressure provided the *N* value was uniform. The *N* value was varying quite randomly both horizontally and vertically across the site. Hence the following scheme which can produce uniform soil parameters for design was adopted.

1. Assuming the entire footprint of the site as inverted flat slab with capitol which will be stepped footing for the columns was proposed in the design.
2. Assuming the width of the footing as the size of the capitol, the depth of the soil to be treated below the footing level to obtain the consistent design parameters was taken as the width of the capitol. The capitol size was adopted as a maximum of 1.5 m × 1.5 m.
3. The locally available soil was sampled up to a depth of 2 m and was tested for its compaction characteristics. It was indicative from laboratory tests the soil in general has maximum dry density values of more than 1.85 and 2.05 g/cc for standard and modified Proctor's energy. Hence it was decided to Proctor density conditions. The liquid limit of the blended soil was restricted to be within 30 % and PI within 15 %.
4. On a sample compacted test embankment section, standard penetration and plate load tests were conducted to evaluate the modulus of sub-grade reaction for the design of the raft slab. The plate load test results indicated the modulus of sub-grade reaction value to be about 1.5 kg/cm^3. From *N* value of more than 30, the modulus of sub-grade reaction could be estimated in the same range. In the design, it was decided to adopt the variation in the value of K. The variation is made by dividing the entire foundation slab into strips and the K varied ±20 %, variation of experimentally obtained K value in the adjoining strips. Further the slab was divided into strips in the perpendicular direction and K varied as earlier. The slab was assumed as 250 mm thick and the capitol thickness as 400 mm. With this the finite element analyses were carried out for all the combinations of loads with variation in the K. The envelope of stress was obtained for every point and the structural system was modified to restrict the stresses to be within the permissible limits. The modified structure was again analyzed and by iteration the final sections were obtained.
5. The field implementation started with removing the soil to a depth of 2 m from the existing level for each building and stacking the same for blending to get the desired properties (Fig. 19). Quarry dust was also used in some cases to reduce the liquid limit. The blended soil was checked again for the compaction characteristics.
6. The blended soil was spread in layers of 250 mm and compacted to modified standard Proctor density using heavy vibrocompaction equipment (Fig. 20). The field control was carried out as per standard practice. Finally SPT and plate load tests were conducted which indicated the corrected weighted average *N* value of more than 15 at 3 m depth below the footing and an allowable soil pressure of more than 12 t/m^2 had been achieved.
7. The footing was designed as an inverted thickened raft slab with modulus of subgrade reaction value in the range of 0.4–0.6 kg/cm^3 which resulted in a slab thickness of less than 200 mm and pedestal of 400 mm thick. The entire work has been completed (Figs. 21, 22, 23, 24, and 25).

A CASE OF BUILDINGS ON HIGHLY COMPRESSIBLE ORGANIC PEAT

This is about buildings of 3 floors built for defense personnel along the west coast. The geotechnical investigations for the project had been carried out by a well-known group. There were seven boreholes taken to depths varying from 5 to 8 m. The soil profile had been identified as dark sandy soil with *N* varying from 10 to 25 within the investigated depths. The general ground level was at about low tide level. Based on the test results, the geotechnical

Fig. 19 The surface soil stripped to a depth of 2 m

Fig. 20 Field compaction

Fig. 21 PCC below the raft

Fig. 22 Reinforcement for raft

Fig. 23 Thickened foundation slab

Fig. 24 Typical house under construction

Fig. 25 Interior of the house

investigating agency recommended independent column foundations at 1.5 m below the existing ground level with an allowable soil bearing pressure of 12 t/m^2. Further it was recommended to raise the ground formation level to 600 mm above the high tide level which was about 2.5–3.0 m above the local ground level. The plinth level would be another 450 mm above the formation level. This necessitated providing two levels of stiff tie beams one at foundation top and the other at plinth level. The locally available sandy soil was recommended to be the backfill material.

When the excavation for column footings started in the first of the 20 blocks there was no problem except in the farthest footing in which some local soft patch was met. But when excavation was in progress for the second block in the second column pit black soft organic material with foul smell was met at about 1.2 m below the local ground level (Figs. 26, 27, and 28). An attempt was made to assess the thickness of this stratum. It was observed that a 20 mm bar could easily be pushed to a depth of more than 6 m.

Other foundation pits were also examined. The presence of organic sandy peat soil with relatively high water content was observed in all the borehole locations for the 20 blocks. It may be noted that in none of the boreholes soft organic peaty soil had been reported to be present within the investigated depths. This called for the revision of the designs. The first choice recommended by the structural consultants was pile foundation with piles of more than 12 m length. The project authorities could not accept this proposal because of the prohibitive cost. The problem was entrusted to the author.

Fig. 26 Organic peat in excavation

Analysis and Solution

The 1.2 m depth of top soil had N value varying from 4 to 6 and with overburden correction it could be 6–9. Since nearly 2.5 m thick embankment banking had been proposed from the local ground level to reach the formation level in the initial DPR itself, it was decided to use the engineered fill and to locate the footings within the fill such that the pressure bulb will be with in the top stiff layer. For this it was recommended to adopt RCC strip footings of minimum width. It was found that with an allowable bearing pressure of about 12 t/m^2 the width of strip footings required was about 1.5–1.8 m. Locating the footing at 0.9 m below the general formation level the depth of engineered fill below the footing was about 1.5 m in addition to the 1.2 m thick natural soil. For this proposal there was concurrence of everybody since there were no new items and there would be saving from the original proposal because the two level tie beams were not required.

The following scheme was proposed with the specified sequence of construction (Fig. 29).

1. Remove the vegetation and roots by scarifying a depth of 0.3 m. Mark the strip footing width based on the allowable bearing pressure of 12 t/m^2.

Fig. 27 General ground condition at low tide level

Fig. 28 Typical pit showing organic peat

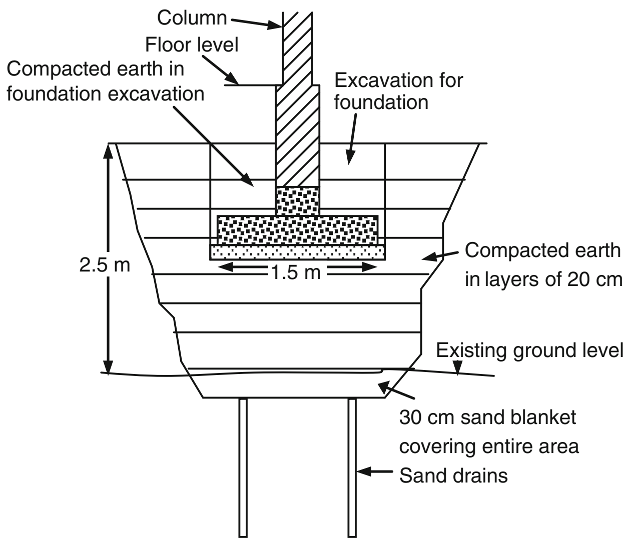


Fig. 29 Schematic diagram of the proposed scheme

2. Introduce 3 m deep 200 mm dia. sand drains/piles from the existing ground level at 1.5 m c/c in two rows within the width of the strip footing using bore well drilling rigs.
3. Spread 300 mm thick sand layer for the entire width and length of the site.
4. Construct engineered fill in layers of 200 mm compacted to standard Proctor maximum dry density to a height of 0.6 m above the high tide level using the locally available sandy soil.
5. On the compacted soil mark the footing plans and excavate to a depth of 900 mm and provide PCC and RCC footing as per structural designs.

All the buildings have been constructed as per the original plans and there was saving from the initial cost. The settlement monitoring is continuing and the buildings are performing satisfactorily without any observable settlement or distress (Figs. 30, 31, and 32).

The entire road net work and utilities have been placed on/within the engineered soil fill. The system has been subjected to severe rainfall conditions for a couple of seasons and all the services are functioning satisfactorily.

DEEP EXCAVATION PROTECTION SYSTEM

This is about a building in Delhi where the deep excavation of 11 m was required with in 6 m from the boundary with neighboring 3 floor brick masonry buildings on the compound wall. The geotechnical investigations revealed the water table at about 12 m below the ground level and cemented sandy silt extends up to a great depth with intensity of cementation increasing with depth. The

Fig. 30 Construction of strip footing in the engineered fill

Fig. 31 Extent of the engineered fill

Fig. 32 Completed buildings and roads on the engineered fill

structural designers had proposed installation of 1,100 mm dia. shoring (touch) piles of length 20 m at 1,200 c/c. During trial installation it was observed that the pile bore hole was collapsing while crossing the water table level. It became too cumbersome to install piles and hence for alternative approach the author was consulted.

Analysis and Solution

The *N* value varied from 20 to more than 40 at about 12 m depth. The original proposal of touch piles were supposed to act as part of the retaining wall. The alternative excavation protection scheme also should be designed to act as part of the permanent retaining wall. Generally just below water table very large piles with narrow gap between the two adjoining piles will have the problem of collapse especially in sandy silt stratum. Hence the first choice is to design smaller diameter piles and this can be done by providing additional props which can do away with large diameter of cantilevered piles. The deformation of the excavation protection system should be very minimal to protect the brick masonry structure on the common compound. The active earth force which will be near constant irrespective of the size of piles need to be balanced by the anchor force instead of passive force in the embedment zone as was originally proposed for 1,100 mm diameter piles. With these constraints the following scheme was proposed.

The water table is just below the final excavation level and it may not be required to provide touch piles. Hence it was decided to adopt 450 mm dia. piles of 10 m length spaced at 1,350 mm c/c.

A sloped excavation to a slope of 1(H): 2(V) at a distance of 1.5 m from the boundary to a depth of 3 m was made. A 40 mm thick shotcrete layer was provided with 50 × 50 × 2.8 mm MS weld mesh fastened to the excavated sloped surface with U hooks. The 450 mm dia. 10 m long RCC bored and cast-in situ piles were installed at −3 m level such that the tips of the piles are at 2 m below the bottom of excavation. The piles would be just touching the rear face of the proposed retaining wall. The piles were connected by a stiff RCC capping beam of size 600 × 450 mm such that the top level of the capping beam merged with the benching from which the piles were installed (Figs. 33, 34, and 35).

Now the excavation has to be done incrementally and the increment is decided by the bending capacity of the piles. With 6 of 20 TMT reinforcement bars, the moment of resistance of the pile is 7.5 t-m. For the first row of grouted nail to be placed at −4.5 m level the horizontal force required to be mobilized from the grouted nails will be 10 t for a pile spacing of 1,350 mm. The distance to the boundary is only 6 m and hence it was required to place the grouted nails inclined. The inclined component of the force will be a function of the angle of inclination of the grouted

Fig. 33 Installation of piles

Fig. 34 Piles ready for capping beam

Fig. 35 Capping beam on piles

nail itself. Further the interfacial shearing resistance between the grouted nail and the soil could be computed by the reported N value. The frictional resistance can be considered to be equal to N in kPa and by trial and error the optimal inclination/or spacing could be obtained. The following scheme is successfully implemented (Figs. 36, 37, 38, 39, 40, and 41).

Design Calculations for the Grouted Nails

Because of infrequent use of this technique, design details of the scheme are provided below.

From N values ϕ can be estimated to be more than 33° and $K_a = 0.30$. Unit weight of soil is assumed as 1.8 t/m^3. To account for the back slope an equivalent surcharge pressure of 2.0 t/m^2 is estimated. The interfacial frictional

Fig. 38 Installation of grouted nails with shotcrete

Fig. 36 Building on the compound and excavation up to first benching

Fig. 39 Installation of grouted nails with shotcrete in third row

Fig. 37 Installation of grouted nails

Fig. 40 Protection scheme completed

Fig. 41 General view of protection

resistance between the grouted nail and the soil, f_s will be equal to N in kPa. Then the horizontal active earth force at the first anchor level can be computed assuming an equivalent uniform surcharge of 2 t/m^2.

Horizontal force = of the wall.

The axial force in the nail of 30° inclination with horizontal at a spacing of 0.9 m (2D spacing where D is the diameter of the piles) with a FS of 1.25 will be 3.5 t.

The effective length of the 150 mm dia. grouted nails within the zone of $N = 22$ will be:

$$L_e = \frac{3.9}{(\pi \times 0.15 \times 2.2)} = 3.75\,\text{m}$$

Though the rigid pile being restrained as the excavation is progressed the potential failure plane is considered as the usual Coulomb's failure plane. Then the total length of the grouted nail required will be 7 m. The tensile force in the nails is 3.9 t and 16 TMT bars are provided as nails.

Similarly for the second row of grouted nails at 3.75 m level from the top of the pile, the inclined nail force will be 7.1 t for a spacing of 0.9 m. The average N value is 32 over the nail zone. The effective length of the grouted nail for 7.1 t will be 4.7 m and the required total length will be 7 m. The tensile force in the nail is 7.1 t and hence 20 TMT bars are provided. For the third row at 6 m level from the top of the pile the inclined force will be 9T for a nail spacing of 0.9 m c/c. The average N value is 38. The effective length required will be 5 m and the total length will be less than 7 m. Adopted nail length is 7 m. For the nail force of 9 t, 25 TMT bars are provided (Fig. 42).

ALTERNATIVE DESIGN TO RESIST UPLIFT PRESSURE IN A BUILDING WITH FOUR BASEMENTS

This is about a building near Delhi built on cemented sandy silt over an area of about 40,000 m^2 and basement excavation was up to a depth of 16.5 m. The geotechnical Investigations had been carried out in two stages by two agencies covering the entire 4 hectares of the site. The water table is reported to be at about 4.5–6 m below the existing ground level in the first report while the same is reported to be at about 8 m in the second report. The design team had finalized the structural system in the form of 10 m long double under-reamed tension piles of 400 mm dia. with 900 mm dia. under reams, below

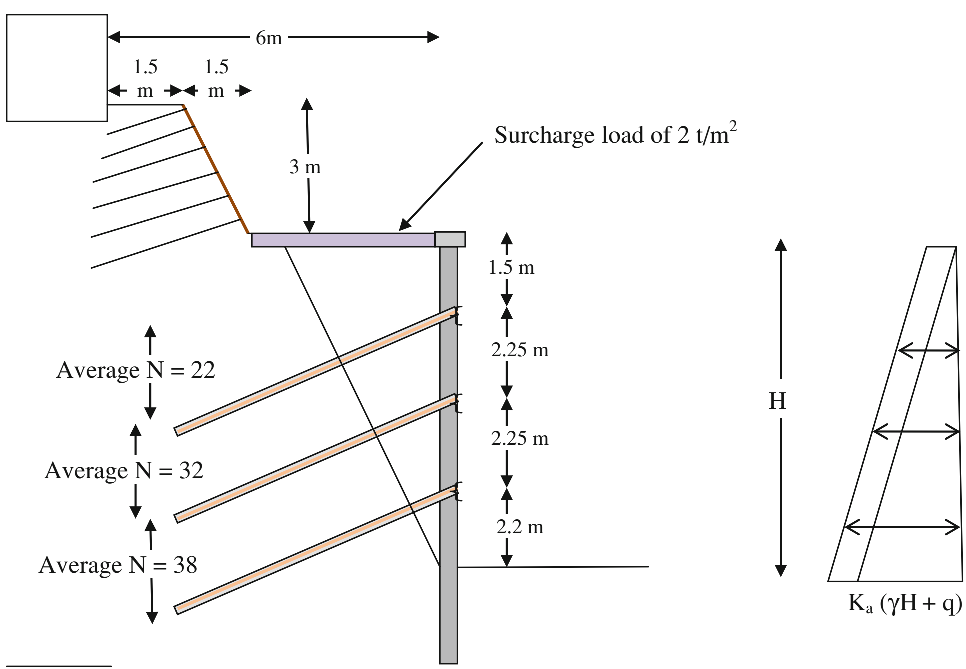


Fig. 42 Design details for the grouted nails

the grade slab to take care of the net uplift pressure of about 10 t/m^2 due to the possible high water table. The spacing of the piles was in a grid of 1.5 m $\times$ 1.5 m. The total number of piles required was in the order of 20,000, the installation of which from the excavated level of -16.5 m itself was a task. Further the stability of the bore hole to accommodate 900 mm dia. 2 reams below water table was causing delay in the implementation of the project. The author was requested to examine the possibility of providing the under slab drainage system as an alternative to relieve the uplift pressures in-lieu of the structural system with under-reamed piles.

Extensive discussions were made with all concerned about the design basis and the effectiveness of the system to clear the apprehensions of all concerned. The Architects and the Structural Consultants were specific that the under slab drainage system should take care of all the exigencies. Hence the following designs have been made little more conservatively which can effectively function with 100 % efficiency without compromising the factor of safety.

The design inputs required for the under slab uplift relieving system are assumed/arrived as follows.

1. The depth to the foundation level below the water table: Since it is reported to be variable with season as a conservative measure the water table level has been assumed to be at 1.5 m from the ground level. The corresponding water head acting below the grade slab will be 15 meters.
2. The coefficient of permeability of the soil: The geotechnical reports have not indicated the permeability values. Since the grain size distribution values have been given for all the layers attempt was made to obtain the coefficient of permeability values based on Hazen's equation. This was also compared with the presumptive values reported in literature for specific idealized soil groups of this range. Again conservative values have been selected for the designs. The coefficient of permeability values have been chosen in the range of 10^{-3}–10^{-5} cm/s. Also field permeability tests were requested by fresh tests and the values obtained were in between the range assumed.
3. Design Philosophy:

    a) The design philosophy adopted is to provide a drainage system to dissipate the hydro-static pressure head. There will be two levels of drainage system. First the under slab drainage system which are laid below the raft/grade slab between the column footings. The drains are essentially French drains with drain pipe clad with non-woven geo-textile filter cloth embedded in an aggregate drain of size 3d $\times$ 3d which is wrapped with filter cloth. The second level is garland drains which are again French drains with larger size to take care of the increased discharge with distance located at a depth below the under slab drains to provide the necessary driving head. The under slab drains are connected to the garland drain below the retaining wall foundation.
    b) The peripheral garland drain will be linked to sumps located at a depth below the garland drains with a required driving head.
    c) The water from the sump will be pumped using submersible pumps with auto-levelcontrollers.
    d) The pumps should be designed to pump out the water collected over 1 h in 20 min.

Design Outputs are obtained as follows.

1. The garland drain behind the retaining wall will have several segments with sump wells pumps. The top-phreatic line generates and equilibrates with time as the water from the sumps is pumped. The parabolic shape of the top phreatic line is controlled by the equation $R_0 = 3{,}000$ H (k)$^{0.5}$ where k is in m/s.
2. The yield per running meter of the pipe will be $q = \frac{kH^2}{2R_0}$ with compatible units. The yield per running meter of the wall will be about 1,000 l/day. For 25 meters it will be 25,000 l/day. Pipe drain of 100 mm wrapped with 250 gsm non-woven filter fabric embedded in an aggregate drain of 300 $\times$ 300 mm again wrapped with filter fabric of 250 gsm is proposed with a driving head of 1 m. For every subsequent 25 m the size of the pipe has been changed to 150 mm with aggregate drain of 450 $\times$ 450 mm and 200 mm with aggregate drain of 600 $\times$ 600 mm. At this point after draining a distance of 75 m a sump well with pumping system is provided. The sump also gets the discharge over a distance of 75 m from the pipe on the other direction.
3. The capacity of the sump well shall be to store 20 min of water draining into the sumps from a distance of 75 m each from both the directions. This requires a size of about 2,500 lit of live storage which should be pumped in about 7–8 min. For this the pump capacity required to pump over a head of about 25 m is about 2.5 HP. A standby pumping system with differential level of auto-controllers will be required to take care of malfunctioning of the pump if any.
4. The under slab drain net work will have a grid of 50 mm porous/slotted pipes wrapped with 250 gsm non-woven filter cloth and embedded in an aggregate drain of 300 $\times$ 400 mm again wrapped with 250 gsm filter fabric. The spacing in both the directions will be 4 m c/c maximum.
5. The under slab drains get connected suitably to the garland pipe net work. The difference in level between the under slab pipe drain invert level and the garland pipe drain invert level shall be 500 mm minimum.

6. The difference in level between the invert level of the garland pipe and the maximum water level in the sump shall be 1,000 mm.

The schematic details of the proposal are presented in the Figs. 43 and 44. The details of the French drain is shown in Fig. 45. Also as implemented scheme is shown in the photos (Figs. 46, 47, 48, 49, 50, 51, and 52). The scheme is performing satisfactorily.

A CASE OF UPLIFT FORCE DAMAGE

The adverse effect of uplift pressure not accounted in the design on the entire basement of a building in Chennai is shown here (Figs. 53, 54). The building had two basements and the ground water level rose up to the ground level causing extensive damage to the basement floors in non-tower area lifting the building by almost 1.3 m in the farthest point. The external drainage system with dewatering bore-wells at 10 m c/c all along the periphery brought the system to some equilibrium. Permanent passive anchors are being installed to prevent the future uplifts after partially dismantling the building.

CASE OF PROTECTION OF 18 M DEPTH OF EXCAVATION CLOSE TO A 42 M TALL CHIMNEY

This is the case of protection of the existing 42 m tall Chimney next to which 18 m deep excavation has been made for 4 basement construction of retail mall. The Chimney has a base diameter of 3.5 m and foundation diameter of 7.5 m. The depth of chimney foundation is about 2 m below the local ground level. Initially it was proposed to have 7.5 m of excavation for two basement floor. It was reported that the structural consultants had proposed and installed 12 m deep 600 mm dia. bored cast insitu RCC piles at 750 mm c/c at 1.5 m away from the

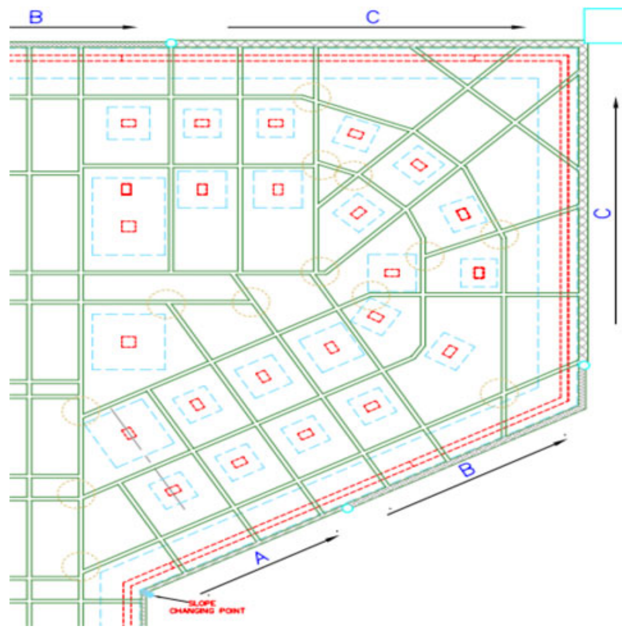
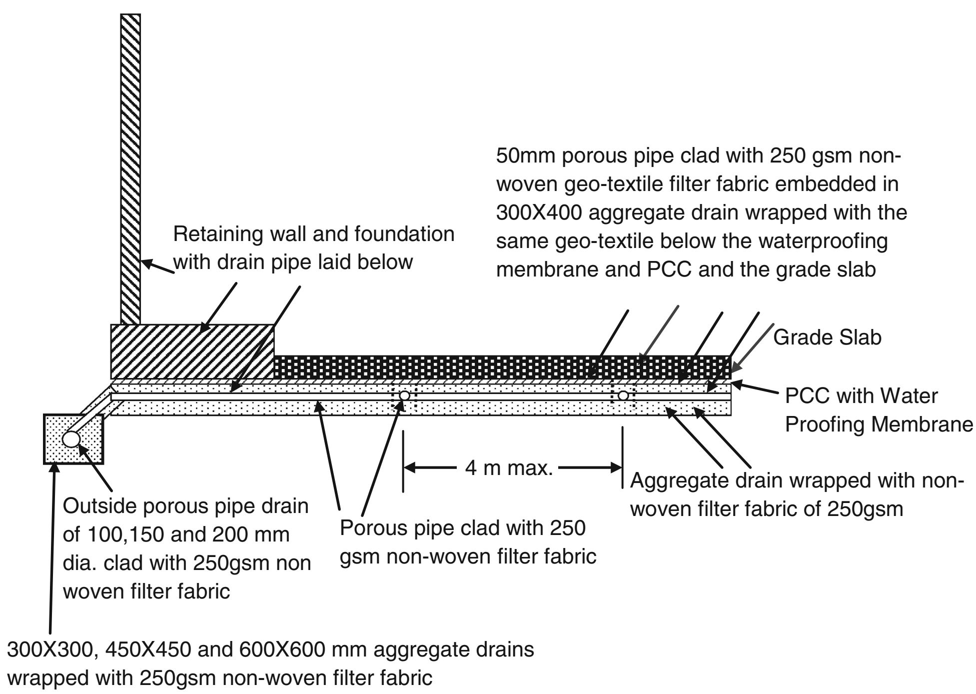


Fig. 44 Plan of the under slab drainage system

Fig. 43 Schematic details of the proposal



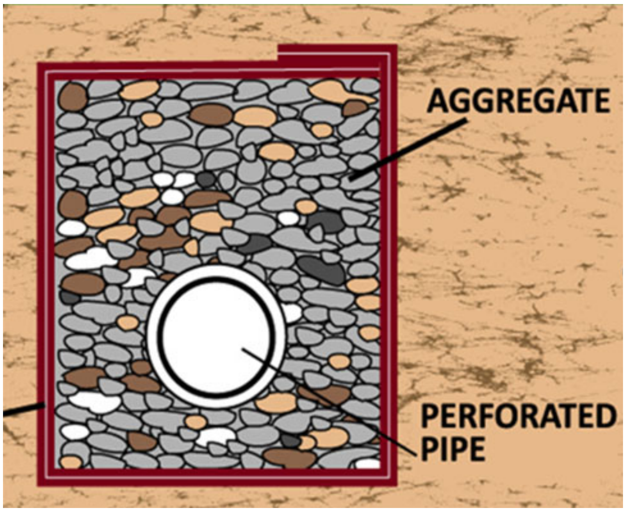


Fig. 45 Typical French drain

Fig. 46 View of the under slab drains

Fig. 47 View of the under slab drains

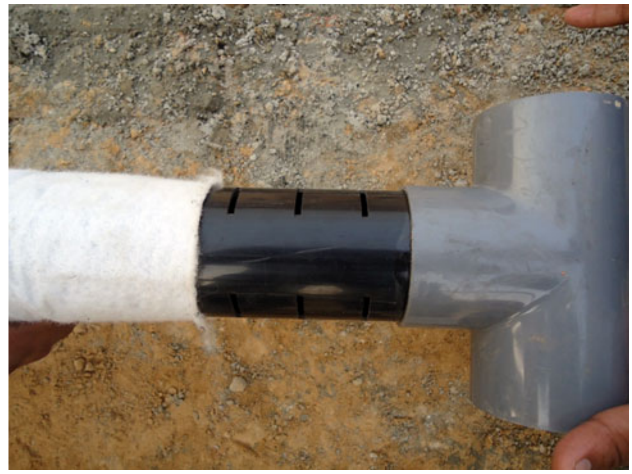


Fig. 48 Slotted pipes covered with geotextile

Fig. 49 Under slab drainage junction

chimney foundation to protect the excavation which was to be at about 6 m away from the chimney footing edge. Since the present excavation is much deeper than the original depth it was necessary to protect the Chimney and the excavation. The following scheme involving soil nailing, micro-piling and grouted nails were in combination proposed (Fig. 55).

Step 1 Leaving sufficient working space behind the proposed retaining wall install 20 m long 200 mm dia. MS pipes of 6 mm thick as micro-piles in predrilled holes of 300 mm dia. drilled using the bore-well drilling rigs (Figs. 56, 57). Grout the annulus space with cement slurry with water cement ratio of 0.42 adding non-shrink grout compound. Fill the pipe with M20 concrete. Weld a capping beam of ISMC200 connecting all the piles.

Step 2 Excavate a depth of 2 m and drive 8 m long 20TOR bars at 100 with horizontal as nails @ 400 mm c/c in the space between the micropiles (Fig. 58). Weld

Fig. 50 Backfill over garland pipe

Fig. 51 Backfill over garland pipe (another view)

Fig. 52 Backfill over garland pipe (another view)

Fig. 53 Uplifted basement

Fig. 54 Crushing of the column due to uplift

75 × 75 × 4.2 mm MS weld mesh to the pipe surface and to the nails with cross bars and provide 50 mm thick shotcrete as per IS:9012.

Step 3 Repeat the step 2 three more times to reach −8 m level.

Step 4 Weld 2 × 100 ISMC wailer beam to the face of the micro pile at −9 m level. Drill at the center of the space between the micro-piles 150 mm dia. 15 m long bore holes inclined at 30° at 800 mm c/c and place 25TMT bars with spacers and grout with pure cement slurry with water cement ratio of 0.42 and adding non-shrink grout compound under a pressure of 1.5 bar. Fasten 75 × 75 × 4.2 mm MS weld mesh with U hooks to the soil surface and by welding to the wailer beam and micro-piles and provide 50 mm thick shotcrete.

Step 5 After the grout is set (minimum 3 days) excavate further 2 m and repeat the step 4 twice with nails inclined at 15° to reach −14 m with grouted nails wailer beams at −11 m and −13 m levels.

Step 6 Repeat step 5 twice with 32TMT bars with grouted nails and wailer beams at −15 and −17 m levels. Complete the excavation to the required level and construct the retaining wall as per structural designs with no earth pressure on the walls since the grouted nails together with micro-piles and wailer beams are taking care of the earth pressure. Water proofing could be done on the shotcrete surface (Figs. 59, 60, and 61).

The work was progressing satisfactorily and grouted nails were being installed at third level when the civil contractor wanted to hasten the raft and wall construction requested the excavation and protection work contractor to complete the work in 3 days the remaining three rows. This made the excavation protection contractor to install every day one row of nails without allowing the grout to set and stabilize the system. The unstabilized 6 m depth of excavation in the bottom caused the cracks to appear on the surface very close to the previously installed RCC piles. At the time of appearance of the crack, work on last row of grouted nails had just started. Using total station the verticality of the Chimney which was being monitored from the beginning indicated that the Chimney has tilted by more than 85 mm in just 2 h. Immediately earth and rock boulder shoring was provided before deciding further action (Fig. 62).

A detailed investigation revealed that the earlier installed piles had not been provided with capping beams. The piles had become floating system. Also it was possible that while installing the grouted nails at −11 m level the angle of the grouted nail not being properly maintained might have hit the bottom tip of the RCC pile. It was indicative from the bulging at −13 m level that the unset grouted nails had not offered any resistance. The cracks had developed to quite large depths of more than a meter. Immediately cement slurry grout with non-shrink grout compound was pumped into the cracks. The amount of grout consumed was quite extensive and it was more than 100 bags of cement and it was not possible to ascertain where all it has gone (Fig. 63).

The RCC piles were interconnected at ground level with ISMB encased in concrete as capping beam (Figs. 64, 65). Further the structural shoring in the form of struts was provided from the all ready cast raft and foundations at a distance more than 12 m from the micro piles to the wailer beam at −11 m level (Figs. 66, 67, and 68). With this earth shoring, grouting at the top, structural shoring and strutting, it was observed from the total station survey that the movement of the top of the Chimney

Fig. 55 Schematic diagram of the design

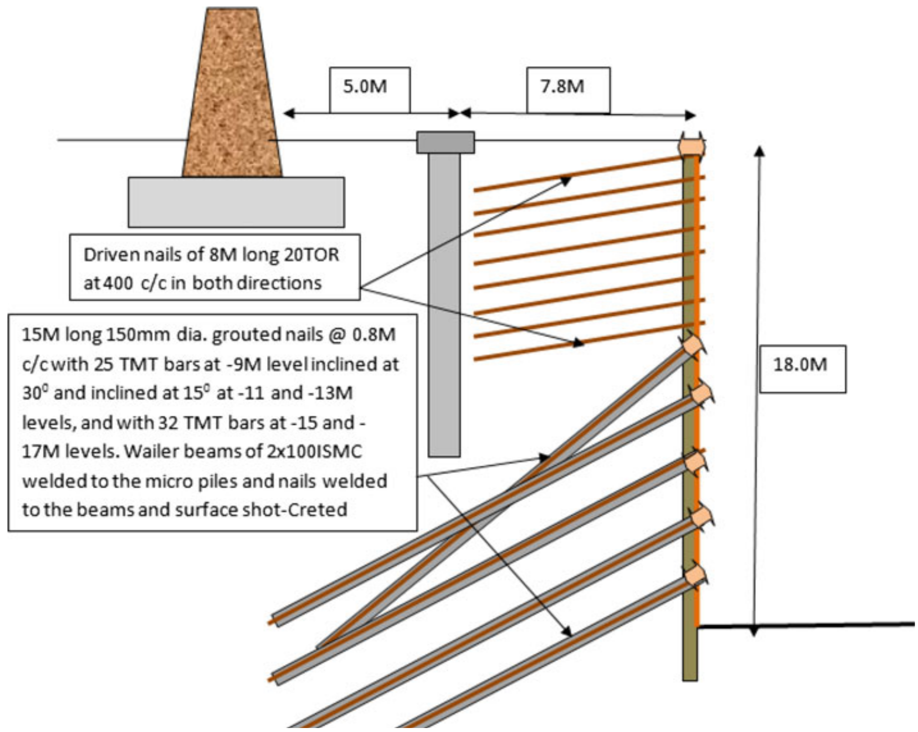


Fig. 56 Installation of micro piles

Fig. 57 Installation of capping beam and first row of grouted nails

had stopped. By this time the total deflection of the top of the Chimney had crossed 135 mm. It was further advised to incrementally remove the earth shoring and introduce the remaining grouted nails and wailer beams along with the required grouted nails to reach the bottom of the excavation.

CASES OF EXCAVATION PROTECTION WITH DRIVEN AND GROUTED NAILS

A number of projects (more than 200) have been successfully implemented throughout the country and a few of them are discussed.

Design of Nailed Wall

This is about the construction of driven nailed walls at Indian Institute of Science Campus, Bangalore. While constructing an underpass across the National Highway in front of IISc, by box-jacking technique, the approach ramps were initially designed to be of RCC walls. For this about 16 well grown trees have to be cut for which IISc had granted permission. Just at that time in another project the author had developed a method of driving the nails using compressor driven percussion Jack-Hammer. Earlier the driven nails used to be installed using conventional sludge hammer, which was not practicable for large size projects

Fig. 58 Installation of capping beam and first row of grouted nails

Fig. 59 Excavation in progress

Fig. 60 Excavation in progress

Fig. 61 Excavation in progress

Fig. 62 Earth shoring to control tilting of the chimney

Fig. 63 Grouting of the cracks

[2]. However the full potential of the methodology had not been explored.

The nailed wall at IISc was designed based on the principles Reinforced Earth Walls, assuming the potential failure plane proposed by [1]. The length of reinforcement was adopted as 0.7H. The interfacial friction coefficient tanϕ_{μ} is tan(2ϕ/3). With this the nail spacing was obtained as 400 $\times$ 400 mm from the pullout failure criterion for a FS of 1.5. As part of research program an intensive parametric study was made on layers of different *N*, varying the capacity of the

Fig. 64 Capping beam on the RCC piles

Fig. 65 Capping beam on the RCC piles

Fig. 66 Structural shoring

compressor which runs the Jack-Hammer and conducting pull out tests on the driven nails. The N varied from 15 to 40 for different layers over a depth of 6 m. The compressor capacity was varied and the adopted capacities were about 165, 200 and 330 cfm with delivery pressure rating of 4.5–6 kgf/cm^2. Then a correlation was attempted to link the N value with rate of driving the bars with a given compressor

Fig. 67 Removal of earth shoring and building of RCC wall with structural shoring in position

Fig. 68 Removal of earth shoring and building of RCC wall with structural shoring in position

capacity which directly measures the penetration strength of the ground. By adopting the correlation between N and ϕ the spacing was linked to the rate of driving with a given capacity of the compressor.

It was found that the rate of penetration of 20 TOR bar of length less than 6 m with a standard compressor of capacity 165 cfm can be directly linked with N as $N = 0.8\,\mathrm{R}'$ where R' is the time in seconds for penetration of 1 m. Similarly correlations have been generated for other sizes of the nail and compressor capacities. Further the N value has been correlated directly with interfacial shear strength, τ_s by many people for piles and anchors. It is related as $\tau_s = N$ to $2\,N$ where N is in kPa for small and large displacements respectively. Adopting this relationship of $\tau_s = N$, the spacing has been directly linked with rate of penetration. From the study it was clear that the spacing can be related to N and hence to R' as $S_x = S_y = 1.25\,N = \mathrm{R}'$. The correlation is not valid for $N > 50$ since in the test site all the N values were less than 50 (Figs. 69, 70, 71, 72, 73, 74, and 75).

In Chennai where soft clay and sandy soil layers were alternating, for an excavation of 7.5 m deep next to the

Fig. 69 Driven nails for the ramp side walls at IISc

Fig. 70 Driven nails for the ramp side walls at IISc

Fig. 71 Concreting of the facing element at IISc

existing building the nails were introduced in both upward and downward inclination from the clay layers to penetrate into the sandy layer to provide better fictional resistance (Figs. 76, 77).

Fig. 72 Finished ramp side walls for the underpass at IISc

Fig. 73 Protection of deep excavation at Air force station by driven nails

Fig. 74 Protection of deep excavation at Air force station by driven nails

Fig. 75 Finished ramp side walls at Air force station

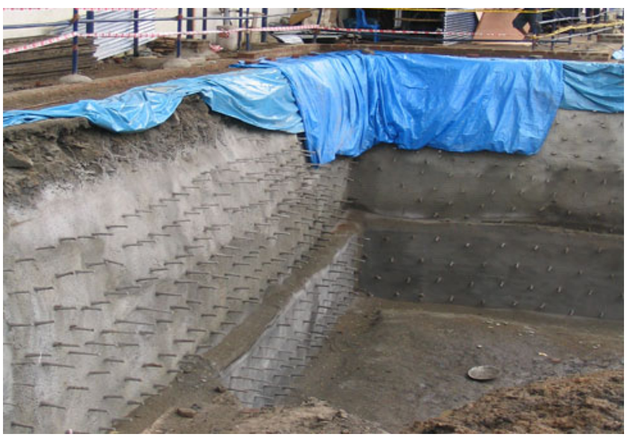


Fig. 76 Driven nails at a Chennai Project

Fig. 77 Driven nails below the existing foundation at Chennai

Fig. 78 Excavation for grouted nails at Bangalore

Design of Grouted Nails

Similarly in drilled and grouted nails also the rate of drilling with a driller rig using a standard compressor can be linked with the interfacial frictional resistance between the grout and the surrounding geological material. A study made with a 650 cfm compressor with a delivery pressure of about 12.5 kgf/cm^2 could drill at different rates in different strata. In soils up to $N = 50$ the rate varied from 1 min to 5 min per meter. The correlation is $N = 12\ \mathrm{R}''$ where R'' is the drilling time in minutes per meter. In strata with N more than 50 and up to rebound condition (which range identifies soft rock), the speed of drilling varied from 5 min to 15 min. The equivalent N values as applicable to soft rock can be estimated from $N = 15\ \mathrm{R}''$. The interfacial frictional resistance between the grout and the soft rock is given by the equation $\tau_f = 15\ \mathrm{R}''$ where τ_f is in kPa and R'' is time in minutes per meter of drilling with 650 cfm compressor under a delivery pressure of 12.5 kgf/cm^2. In hard rock drilling speed varied from 15 min to more than 30 min per meter. Using the same logic the interfacial frictional resistance τ_f can be estimated from the same equation in the absence of data from unconfined compression strength of the rock cores.

It is cautioned that while adopting this relationship for designing, a field trial could be made for a given case since the efficiency of the compressor, drilling rig and the Jack-Hammer and the skill of driving and drilling personnel enormously influence the rate of penetration of the nail.

The sequence of steps of a grouted nail installation scheme for an excavation depth of 14 m in Bangalore is presented. Excavate a depth 2 m. Install grouted nails of 11 m long with 25 TMT bars @ 1.6 m c/c at −1 m level. Fasten MS weld mesh of size 75 × 75 × 4.2 mm with U hooks to the excavated surface and provide 50 mm thick shotcrete. After curing of the shotcrete fix the wailer beam

Fig. 79 Excavation and two rows of grouted nails

Fig. 80 Shotcreted surface after two rows of grouted nails

Fig. 81 Deep excavation and grouted nails as finished

to the nails by welding. Fill the gap between the shotcrete surface and wailer beam with concrete to achieve effective contact between wailer beam and the shotcrete surface.

Fig. 82 Slope protection at NMDC project -Combination of driven and grouted nails

Fig. 83 Deep excavation with driven and grouted nails as finished

Fig. 84 Deep excavation with driven and grouted nails as finished

Fig. 85 Touch piles of 300 mm size with grouted nails

Fig. 86 Touch piles of 300 mm size with grouted nails

Repeat the steps with dewatering and with bars of higher diameter as per design (Figs. 78, 79, 80, and 81).

Design of Combination of Driven and Drilled Grouted Nails

The combination of driven nails and drilled grouted nails can be adopted for steep slopes and footings with no embedment.

In a project at Whitefield a 19 m deep excavation, where water table was not met, has been carried out with a combination of driven and drilled-grouted nails with shotcrete and wailer beams. At locations close to the boundary 200 mm dia. MS micro-piles filled with M20 concrete in combination with the driven-grouted nails have been installed. In another project small diameter RCC touch piles were installed and as excavation progressed grouted nails with wailer beams have been installed. No shotcrete is provided.

A hillock of about 22 m height was to be cut through to install a conveyor system to run in a RCC duct. The

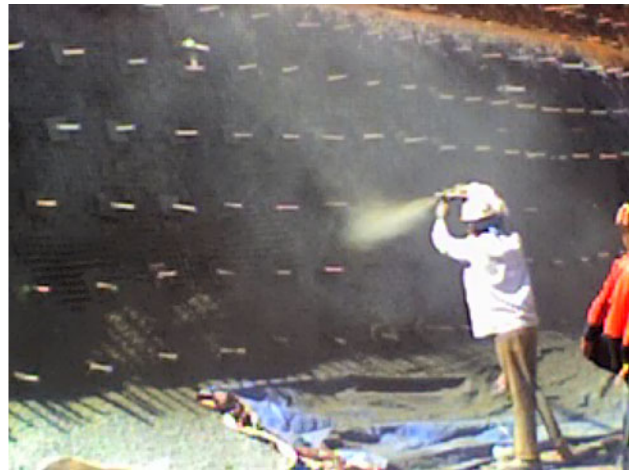


Fig. 87 Slope protection at NMDC project

temporary protection system with driven nails was imple-

Fig. 88 Slope protection at NMDC project

Fig. 89 Slope protection at NMDC project

Fig. 90 Driven nails to strengthen RE walls

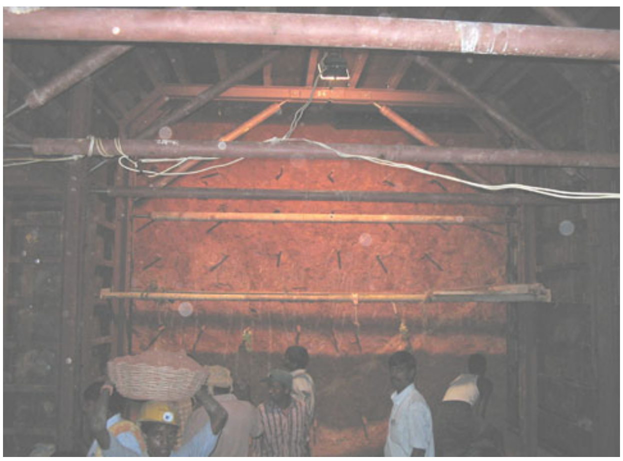


Fig. 91 Driven nails to prevent collapse of excavation face

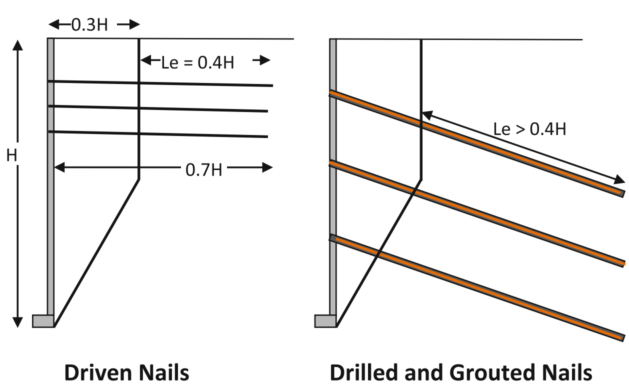


Fig. 92 Design principle of driven and grouted nails

mented to facilitate construction of RCC duct. The temporary system was so effective that the authorities decided to regard the temporary protection as a permanent one and abandoned the proposed RCC duct (Figs. 82, 83, 84, 85, 86, 87, 88, and 89).

Driven nails have been used to improve the stability of the geo-grid reinforced RE wall which had failed at a few locations (Figs. 90, 91).

In an underpass construction below National Highway at Bangalore nails driven on the excavation face prevented the collapse of the soil from facing due to passage of heavy vehicles on the surface.

Acknowledgments The author places on record the valuable inputs received from Professor A. Sridharan and other colleagues in the Department of Civil Engineering, Indian Institute of Science, Bangalore.

Appendix

Design of Driven and Grouted Nails

Based on the principles of reinforced earth soil nailing has emerged as one of the best and economical alternatives for stabilizing the excavations and protection of cut slopes. Off late depth up to 25 m of excavation in urban areas and deep cuts in hilly terrains are being handled by this technique (Fig. 92). The main design features of such system are.

1. Evaluating the proper earth pressure distribution expected with-in and behind the stabilized earth mass.
2. Estimation of the probable interfacial shearing resistance using the properly evaluated interfacial friction value between the soil and reinforcement in driven nails and that between the grout and the soil in grouted nails.
3. Suitable facing system which effectively transfers the active force into the nails.
4. The potential failure surface beyond which the reinforcement in the passive zone resists the pull out of the reinforcement due to active force on the facing element.
5. The design requirements of satisfying the basic mechanics equations of equilibrium in terms of overturning and sliding are valid. In addition both frictional failure (pull out) and tensile failure (tension) criteria need to be satisfied. A check on the bearing capacity which has been made mandatory during initial stages of development of the technique is not being insisted.
6. The interfacial friction between the driven nail with the TOR or TMT bars and the soil could be taken as ϕ of the soil. This is because the volume displacement due to driving of the bars will increase the density locally to provide effective contact in addition to the surface striations on the bar will hold the soil particles

resulting in the failure plane to occur between soil and soil.

7. Generally since the driven nails are of 20 mm in diameter, twice the projected area which is twice the diameter multiplied by the effective length ($2\ d\ L_e$) need to be considered instead of ($\pi\ d\ L_e$). In case of large diameter nails like grouted nails which are formed from drilling the hole and grouting the hole under some pressure after placing the reinforcement in position the perimeter surface area $\pi\ d\ L_e$ may be considered.
8. The interfacial shearing resistance at every point along the nail could be computed as $\gamma\ H \tan\phi$
9. Now the governing equation for the design against pull out failure of a driven nail will be $S_x \times S_y \times K_a \times \gamma \times H$ is the active earth force on the elemental area of $S_x \times S_y$.

The resisting force from the effective length of the bar embedded beyond the potential failure surface will be $L_e\ \gamma\ H\ 2d\ \tan\ \phi$. With a factor of safety of 1.5 the two forms of the forces may be equated to get the equation as

$$S_x S_y = \frac{L_e \gamma H 2d \tan\phi}{1.5 K_a \gamma H} = \frac{2dL_e \tan\phi}{1.5 K_a}$$

From the basic mechanics equations of sliding and over turning it can be shown that the length of nails should be about 0.7H for a factor of safety of 1.5 for both the conditions up to a height of 12 m. The potential failure surface for incremental excavation and stabilization condition can be shown to be a log spiral with initial angle at toe being (45 + ϕ/2) and emerging perpendicular to the horizontal back surface at a distance of about 0.3H. With this it can be assumed to follow the bilinear failure surface as shown in the figure. If the bar is driven inclined by about 10° the effective length will be little more than 0.4H. In the drilled and grouted nails the equation will be

$$S_x S_y = \frac{\pi d L_e \tan\phi}{1.5 K_a}$$

It is possible to compute the interfacial frictional resistance by considering the corrected average N value over the effective length of the bar in KPa as the shearing resistance. Then the governing equation against pull out failure will be

$$S_x S_y = \frac{\pi d N L_e \tan\phi}{1.5 K_a \gamma H}$$

Generally in the case of driven nails tension failure will not be an issue since drivability requires minimum of 16 or 20 mm dia. bars which will far higher tensile capacity. However in the case of grouted nails where the spacing will be in the order of 2 m × 2 m the tensile force need to be checked and the reinforcement diameter need to be changed. The governing equation will be

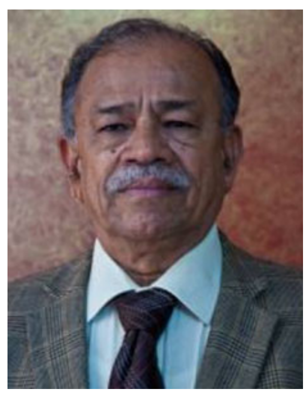
$$1.5\ S_x S_y K_a \gamma\, H = A_s f_s$$

where A is the cross sectional area of the steel bar and f is the allowable tensile strength of the steel bar. 1.5 is the factor of safety.

References

1. Vidal H (1968) La terre armée. Annales de l'institute technique du bâtiment et des travauvx publics, Série Matériaux 30, Supplement No. 223-4, July–August
2. Sridharan A, Srinivasa Murthy BR (1993) Remedial measures to a building settlement problem. Proceedings of third international conference on case histories in geotechnical engineering, St. Louis, Missouri, pp 221–224
3. Nagaraj TS, Sridharan A, Paul Alexander MV (1982) In situ reinforced earth—an approach for deep excavation. Indian Geotech Jl 12(2):101–111

Author Biography



B. R. Srinivasa Murthy (b.1943) graduated civil engineering in 1966 Mysore university and obtained M Tech in Soil Engineering from IIT Powai in 1968. He was SRF under CSIR Scheme in IIT Delhi during 1968-69 and then served as Lecturer in Civil Engineering at SIT Tumkur during 1969-70. Later he joined PWD as Junior Engineer and served for two and half years designing Major Irrigation Project Structures. His passion for teaching brought him back to the academic line and joined the University Visweswaraya College of Engineering of Bangalore University as Lecturer in September 1973 and became Reader in 1981. He obtained his Ph.D. degree from the Indian Institute of Science in 1983 under the guidance of Prof. T S Nagaraj and his Ph.D. thesis was awarded with Prof G. A. Leonard's Prize of IGS and Prof. P.S. Narayna Medal of IISc. In 1984 he joined Indian Institute of Science, Bangalore and promoted as Associate Professor in 1991 and as Professor in 1998. He has served as Registrar of Indian Institute of Science for over three years. He retired from the services of the Institute in 2005. In over two decades of service at IISc he has guided 14 Ph.D. and 7 M Sc (Engg) theses in addition to scores of ME dissertations. He has published/presented over 100 Technical papers in international refereed journals, International and National Conferences. He has also co-authored a book titled "Prediction of soil Behavior" with Prof. T S Nagaraj and Dr. A Vatsala. He has also contributed a Chapter in an international Hand Book. He has handled three major sponsored research projects during his stay at IISc. The areas of research interest of Dr. Murthy were Constitutive Modeling of Fine grained soils including cemented and unsaturated soils, Cam-Clay models, Reinforced Earth and Reinforced Soil Beds and Ground Improvement Techniques like Micro-piling and Grouted anchors.

Dr. Murthy has very widely travelled abroad both East and West. He was an International Researcher under NRC Canada for 6 Months at University of Laval and J S P S Fellow and INSA-JSPS Fellow for two terms (Nine Months) at Gifu University, Japan. He was a visiting faculty for one Semester at AIT Bangkok. He was Visiting Professor for six months at Institute of Low Land Technology at Saga Japan. He has delivered Lectures at Rice University and University of Florida at USA, Kyoto University, Nagoya Institute of Technology and Institute of Ports and Harbor at Japan, Queens University in Canada. Dr. Murthy has been conferred with many honors including “The Fellow of Institution of Engineers”, “Honorary Fellow of ACCE”, “Life Fellow if IGS”. He has been awarded the IGS-AFCONS KUECKLEMAN AWARD in 2004, Distinguished Engineer Award of the year 2006 by Institution of Engineers (India) and Outstanding Civil Engineer’s Award of ACCE in 2008. In addition to being an excellent Geotechnical Engineering Consultant, he has been a very good Structural Consultant. Rehabilitation of old buildings, fire damaged buildings and damaged sewage digester domes and old arch bridges under distress conditions are his strong areas of consultancy. Projects of large size pipe line design for power projects and water supply schemes have been handled as interdisciplinary area. He has successfully constructed several vehicular and pedestrian under passes below high ways by Box-Jacking technique without disturbing the road or the traffic. Protection of deep excavations (up to 25 m) with soil nailing techniques, in most of the major cities in India has been his current field of activity over the last decade. Alternative foundations on filled up and week soils are being handled regularly. The number of consultancy projects handled so far in the fields of Geotechnical Engineering, Structural Engineering and Pipeline Engineering by Dr. Murthy is far more than 500. He is on the several committees of the state government either as chairman or as member. He is the Chairman of Technical Advisory Committees of Bruhat Bangalore Mahanagara Palike, Karnataka Health Services System, Karnataka State Police Housing Corporaration and CMTI. He is Member of the Taskforce on Quality Assurance in Public works Govt. of Karnataka, Project Management Group of IISc. Karnataka Power Corporation and Karnataka Slum Clearance Board.