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From the Editors
The editors would like to welcome you to the first edition of The Journal of the Deep Foundations Institute. This publication is a milestone in DFI’s ongoing effort to fulfill its mission as an information resource for design and construction of foundations and excavations. The journal will focus on subjects that are directly relevant to practice and thus will provide a bridge between academia doing applied research, practicing professionals, and the construction industry. The role of The Journal of the Deep Foundations Institute is detailed in the mission statement of the journal (see below). Note that this mission statement includes a special interest to integrate and maintain a diverse range of subjects and to incorporate projects from the international community. Reports on case histories and failures will be particularly invaluable in advancing the practice through lessons from real world experiences. Participation in the journal is open to everyone in the field of interest; however, it is clear that the success of this publication depends on you, the reader of the journal, to contribute by writing articles and sharing your experiences with your peers. Practitioners are continuously looking for improved ways to solve problems, to implement advances in technology, and to enhance reliability and cost-effectiveness. The Journal of the Deep Foundations Institute will provide a means to improve our industry, if we all participate in an open and professional manner in sharing our experiences and lessons learned. As the editors, our promise is to make the review process as painless and efficient as possible while maintaining the high standards of an international technical journal.

Papers presented in this first edition of the journal cover a wide range of interests, including the challenges for design and construction of the Sutong Bridge foundations in China, rapid lateral load testing of deep foundations in the USA, deep diaphragm wall activities at RandstadRail project in the Netherlands, ground movement issues related to deep foundations reported by Harry Poulos from Australia, and design of drilled shafts supporting sound walls in the USA.

The editors would like to acknowledge Theresa Rappaport, the executive director of DFI, for her enormous dedication and tireless efforts in making this publication a reality. Appreciation is extended to the DFI members who supported the inaugural issue of the journal.

We look forward to the success of The Journal of the Deep Foundations Institute as a valuable resource to our industry for many years to come. Comments, suggestions, and submissions are welcome and may be submitted via the DFI website at www.dfi.org.

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DFI Journal Mission/Scope
The Journal of the Deep Foundations Institute publishes practice-oriented, high quality papers related to the broad area of “Deep Foundations Engineering”. Papers are welcome on topics of interest to the geo-professional community related to, all systems designed and constructed for the support of heavy structures and excavations, but not limited to, different piling systems, drilled shafts, ground modification geosystems, soil nailing and anchors. Authors are also encouraged to submit papers on new and emerging topics related to innovative construction technologies, marine foundations, innovative retaining systems, cutoff wall systems, and seismic retrofit. Case histories, state of the practice reviews, and innovative applications are particularly welcomed and encouraged.
Introduction

Detailed design of the Sutong Bridge was performed by The Design Group of Sutong Project. Construction management of the entire project is being performed by the Jiangsu Provincial Sutong Bridge Construction Commanding Department (CCD). COWI A/S and Ben C. Gerwick, Inc. served as special consultants to CCD during design and construction of the bridge.

The width of the Yangtze River at the bridge site is approximately 6 km and the total length of the Sutong Bridge is approximately 8 km. The site conditions for the main towers are extremely challenging. The northern pylon, Pier 4, is located in about 30 m water depth and the southern pylon, Pier 5, is located in about 16 m water depth. The river is subject to both high fresh water run off volumes and tidal effects, creating currents exceeding 3.0 m/sec in the extreme conditions.

Maximum potential wave heights at the site exceed 3.5 m. The river is alluvial and subject to rapid changes in bottom contours due to high erosion and deposition rates. The river bed at the northern pylon consists of sandy materials and at the southern pylon, the bed material is mainly silty loam and silty clay. These site characteristics create a condition where the river bottom will immediately respond to the introduction of any structure such as a bridge or pylon. Hydraulic model studies for the bridge, performed by the Nanjing Hydraulic Research Institute, predicted up to 29 m of scour (100 year return period) at the south pylon with a caisson foundation and up to 24 m of scour (300 year return period) at the same location for a large diameter pile foundation solution (Jensen, 2004).

Bedrock is located at approximately 240 m below the river bottom. The soils in the upper 240 m consists of layered sediments of fine sand, course sand, silty sands and gravels with occasional layers of clay.

The river is the main waterway to the entire Yangtze Basin with heavy barge traffic and up to 50,000-t container ships in the main navigation channel.
Foundation Design

The foundation for each A-shaped pylon consists of 131 drilled shafts, 2.8/2.5 m in diameter. See Fig. 4 for layout of the pile cap and drilled shafts at each pylon. The drilled shafts are capped by a 13 m deep dumbbell-shaped pile cap with plan dimensions of 113.8 m by 48.1 m.

The bottom of the cap tremie seal is positioned at Elev. -10.0 m, approximately 12.0 m below mean sea level. The drilled shaft casings are 2.8 m diameter with a wall thickness of 25 mm. See Fig. 5. The permanent casings extend from Elev. -7.0 to Elev. -53. The drilled shafts beyond the casing tip are 2.5 m diameter and extend to a design tip elevation of -124 at Pier 4 and -121 at Pier 5. Post grouting of the drill shaft tips was performed to increase the total ultimate capacity.

The ultimate load capacity of the drilled shafts was confirmed by four offshore load tests, two at Pier 5, the south pylon, and two at the approach piers. The two tests at Pier 5 confirmed an ultimate capacity of 92 MN (20,700 kips). Testing was performed using the Osterberg Cell Method. Test pile SZ5 was tested twice, before and after tip grouting, to give an indication of the increased capacity obtained through tip grouting. The tests indicated that the bearing capacity of the drilled shaft was increased by 20% or 15 MN (3,757 kips) by the tip grouting.

The load deformation curve after grouting showed a much more rigid behavior than before grouting. This result demonstrated that the tip grouting had a positive effect on not only the tip but also on the side friction on the lower portion of the pile.

Drilled Shaft Design

Drilled shaft design methods have traditionally relied on mobilizing skin friction along the shaft length to resist service axial loads. End bearing, if not discounted, is usually significantly reduced and mainly employed to satisfy extreme load conditions or safety factor requirements. This is mainly due to the concept of strain incompatibility since ultimate end bearing is mobilized at a shaft displacement two or three orders of magnitudes larger than the displacement required to mobilize ultimate skin friction. This is especially true for larger
diameter shafts. Moreover, the displacement needed to mobilize significant end bearing is likely to be larger than estimated due to drilling-induced soil disturbance at the tip of the shaft and debris remaining after cleanout. As a result, developments in drilled shaft construction technology have been mainly focused on increasing shaft diameter or shaft length in order to increase shaft axial capacity.

An effective alternate technique that can be used to increase shaft axial capacity is post-grouting of drilled shaft tips. This technique, although introduced four decades ago, has not been widely used in the US despite its significant potential for cost saving and improvement of quality control of drilled shaft construction. This technique works by effectively preloading and densifying the soil and any remaining debris under the tip of the shaft by pressure grout delivered by a system of pipes pre-attached to the reinforcement cage of the shaft. As a result, larger end bearing capacity can be mobilized at the tolerable displacement limit, thus increasing overall shaft capacity without having to increase its length or diameter.

**Drilled Shaft Axial Capacity**

The maximum demand axial shaft load was determined to be 44.0 MN for the pylon foundation. With an adopted design safety factor of 2.0, the design axial load capacity of the pylon shaft should therefore be at least 88.0 MN.

The Chinese codes which applied to this project determine the ultimate axial capacity of the drilled shaft as the minimum of: 1) the load at which the shaft settlement is 80 mm, 2) the load at which creep is 0.2 mm per hour at the end of 24 hr load application, and 3) the load at which there is a dramatic and sudden change in the load versus displacement curve.

The soils at the pylon site consist mainly of firm to stiff CL clay extending to elevation -45 m followed by layers of medium to very dense fine to coarse sands and silty sands with occasional loam layers. Bedrock is located at approximately 240 m below riverbed. Based on the soil conditions, and the estimated skin friction of a 2.5 m diameter shaft, the design team decided that a shaft tip elevation of -124 m at the northern pylon and -121 m at the southern pylon would be sufficient if a significant percentage of end bearing could be relied on. To achieve this while meeting the settlement and creep criteria, and to minimize the detrimental impact of drilling-induced soil disturbance and remaining debris at the bottom of the drilled hole, post-grouting of the shaft tips was selected as the most economical solution. Also, a 2.8 m diameter permanent steel casing was selected to extend to an average elevation of -53 m to maintain hole stability and to increase lateral stiffness of the foundation in the upper clay layers.

**Drilled Shaft Construction**

Due to the high river currents, all drilled shaft construction was performed from a steel platform constructed over the top of the pier site. In addition, an upstream mooring platform (13 m by 44 m) and a downstream batch plant platform (39 m by 44 m) were constructed immediately adjacent to the main platform. See Fig. 6.

The top elevation of the drill platform was +7 m, approximately 3 m above high water. The main platform was used as both a template to drive the 131 drill shaft casings and to provide a work deck for the drill units. Casings at the northern pylon were driven to grade with a vibratory hammer and at the southern pylon a diesel hammer was used. See Fig. 7. After installation...
of each drilled shaft casing, bracing was added to tie each casing into the work deck, and thereby adding rigidity to the entire work deck. Drilling was performed with 8 rotary-drill units positioned on the top of the work deck. Drill bits varied depending on the formations encountered. See Fig. 8 and Fig. 9. A bentonite slurry with a minimum positive head of 3 m was used to maintain drill hole stability. Both drilling and concreting operation were conducted simultaneously on the platforms. A minimum concrete strength of 5 MPa (725 psi) was required in an adjacent drilled shaft before drilling was allowed. The post tip grouting operation was also performed concurrently with these operations. However, the grouting operations maintained a minimum distance of 50 m from drilling and concrete placement operations in order to avoid hole-instability problems from elevated pore water pressure created by the grouting operation.

Concrete was supplied by a batch plant with a capacity of 100 cubic meters per hour, positioned on the downstream platform. Cement and aggregates were delivered to the platform by barges moored directly to the downstream platform.

Post grouting of the drilled shaft tips was performed with 4 loop-shaped pipes pre-attached to the reinforcing cage. The bottom of each loop turned at the bottom of the cage and extended into the interior of the drill shaft approximately 50 cm. Grout exited the pipes through 6 holes, 8 mm in diameter, drilled in the underside of each loop. A one-way valve was created by encasing the loop in a bicycle tire. To ensure that the system was not plugged during the concrete placement operation, clean water was pumped through the system under pressure to confirm open access to the surrounding tip area. Post grouting was performed with neat cement grout.

**Post-Grouting of Drilled Shaft Tips**

Post-grouting of drilled shaft tips is usually conducted using two techniques; the flat jack, or the sleeve-port (also called tube-a-manchette). In the first technique, grout is delivered by tubes attached to the rebar cage to a steel plate with a rubber membrane underneath at the tip of the shaft. In the second technique, which was used in this project, grout is delivered to the tip through loop-shaped pipes which are pre-attached to the rebar cage. For this project, six tubes were used. The bottom of each pipe, at the tip of the shaft, has a U-shape and extends approximately 50 cm into the interior of the shaft, as shown in Fig. 11. Grout was discharged through eight holes 8 mm in diameter in the underside of each U-shaped pipe, which was encased by a bicycle tire to act as a tight fitting rubber sleeve creating a one-way valve. The

The reinforcing cages were fabricated in four sections and coupled together with threaded mechanical connectors on the work deck over the top of the casings prior to lowering them into the drill hole. See Fig. 10. Concrete was placed with a tremie pipe centered in the drill hole.
advantage of this system is that it allows clean water to be pumped under pressure to ensure that the system is not plugged during concrete placement operations, and confirms that an open access to the shaft tip area is maintained.

The major issues in post-grouting of shaft tips, other than the design of the grout delivery system, are to determine the grout pressure and grout quantity. The work of Mullins et al. (2006) shows that the level of grout pressure is the most important factor affecting the gain in end bearing and the stiffness in its load-displacement relationship. Another secondary factor is the time period of application of grout pressure. In this project, grouting operations have been controlled by both grout quantity and grout pressure. The project criteria require the grout pressure to reach the targeted level for at least five minutes, and the grout quantity to be at least 80% of the design value. Obviously, since grout pressure acting on the shaft tip is resisted by the shaft skin friction, the maximum grout pressure, and therefore the maximum achievable enhancement in end bearing, is governed by the ultimate skin friction resistance. This also means that the process of post-grouting of shaft tips will cause an upward movement in the shaft as the soil-shaft interface is strained. Therefore, field measurements of the uplift movement of the top of the shaft when related to the applied grout pressure can provide valuable information to verify the axial capacity of production shafts. For the Sutong Bridge, the skin friction of the pylon shafts was estimated as 64 MN; therefore, for a 2.5 m diameter shaft, the estimated maximum grout pressure that can be applied at the shaft tip was 13 MPa plus the buoyant weight of the shaft. Practically, a lower grout pressure was used since the maximum pressure that can be applied in the field was 7 MPa.

In addition to the upper limit governed by ultimate skin friction, the grout pressure should exceed the hydrostatic pressure at the shaft tip. The project criteria adopted the following method to determine the minimum operating pump pressure:

\[
P = P_g + \sum \gamma_i L_i \]

where \(P_g\) and \(P_w\) are the pump and hydrostatic pressures at the shaft tip level, respectively, and \(\gamma_i\) and \(L_i\) are the effective unit weight and thickness of each layer \(i\) above the shaft tip, respectively. \(\zeta\) is an empirical coefficient for grout resistance, which is a function of the type of soil material at the shaft tip. For sands, \(\zeta\) ranges from 1.5 to 3.0. Therefore, the estimated minimum operating grout pump pressure in this project was 3 MPa.

Based on the estimated upper and lower grout pressure limits and the required gain in end bearing to meet the design safety factor, the design team decided to use a grout pressure of 5 MPa, which was subject to verification by field tests. Although not used during the design phase of this project, one can estimate the gain in end bearing as a function of the applied grout pressure and shaft settlement using the recent work of Mullins et al. (2006), which suggests the following equation:

\[
TCM = 0.713(GPI)\%D^{0.364} + \frac{\%D}{0.4(\%D)+3.0} \]

where \(\%D\) is the shaft settlement as a percentage of its diameter \(D\), TCM (tip capacity multiplier) is the ratio between the end bearing at a \%D settlement to the end bearing at a settlement equal to 5% shaft diameter. GPI (grout pressure index) is the ratio of the applied grout pressure to the ungrouted end bearing at a settlement of 5% D. The ungrouted end bearing at a settlement of 5% D was estimated as 3.5 MPa for the pylon shafts. Therefore, for an applied grout pressure of 5 MPa, i.e. GPI of 1.43, the estimated TCM from this approach for 1% D, 3% D, and 5% D settlement is 1.3, 2.2, and 2.8, which correspond to an allowable end bearing capacity of 4.6, 7.7, and 9.8 MPa, respectively. Therefore, if this approach was used during the design phase of the project it would also indicate that a 5 MPa grout pressure would be sufficient to obtain at least 25 MN in end
bearing while meeting the project settlement requirement of 80 mm. The design end bearing of 25 MPa was the end bearing targeted to obtain an ultimate axial shaft capacity with safety factor of 2.0.

The design grout quantity was estimated as 100 kN based on the porosity and grout penetration ratio of the soil at the shaft tip. Consideration was also given to the grout going upward around the pile shaft.

**O-Cell Tests with Post-Grouting of Shaft Tips**

To measure and verify the skin friction and end bearing capacities of the design shafts, several onshore and offshore shafts were tested. The results presented herein are from an O-cell test on an offshore shaft constructed at the southern pylon site which was loaded before and after grouting. The 2.5 m diameter test shaft had a tip elevation of -121 m with a 2.8 m diameter 25 mm thick permanent steel casing with a tip elevation of -53 m, as shown in Fig. 12. Six U-shaped grout pipes were attached to the shaft reinforcement cage as shown in Fig. 11. O-cells were placed at two levels. The upper level was 28 m above the shaft tip with two 870 mm diameter O-cells and a total nominal ultimate load of 55 MN, while the lower level was 1.5 m above the shaft tip with two 660 mm diameter O-cells and a total nominal ultimate load of 32 MN. Four LVWDTs (Linear Vibrating Wire Displacement Transducers) were installed at each O-Cell level. Eight (8) levels of vibrating wire strain gauges and four telltales were used as shown in Fig. 12.

The test was conducted by LOADTEST Singapore office. The soil profile at the test site consists of layers of gray silty CL clay down to elevation -53.5 m overlying layers of fine to coarse sands that extend well below the shaft tip elevation. The test was conducted in two phases; before and after grouting of the shaft tip. The first phase consisted of a one-stage load test. In this phase the lower O-cells were pressurized in 17 loading increments, each 0.9 to 1.0 MN and lasting 30 minutes, while the upper O-cells were kept closed. As shown in Fig. 13 and summarized in Table 1, at the end of the 17th increment, the total lower O-cells load was about 16.5 MN with a total expansion of 93 mm, mostly from end bearing settlement, which is larger than the 80 mm limit required by the project design criteria. At the end of the first phase tests, the shaft was unloaded in 5 increments.

**Fig. 12** Test Shaft Layout and Instrumentation

**Fig. 13** Load-displacement Curves Before and After Post-grouting of Shaft Tip from Stage 1

The second phase of the test was conducted 5 days after grouting of the shaft tip. The grouting process was conducted in three cycles to help achieve a uniform treatment of the soil at the shaft tip. In each cycle, the grout pressure was increased in equal increments to the design level, while the grout quantity was distributed equally in the straight grout pipes. In the first cycle 50% of the neat cement grout quantity was extruded, followed by pressure washing the grout pipes with clear water. After at least 1.5 hours of waiting, 30% of the grout quantity was extruded after which the grout pipes were pressure washed again with water. After at least 3.5 hours of waiting, the third cycle was completed by extruding the remaining 20% of the grout quantity. In the first and second cycles, there was more emphasis on controlling the grout quantity, while in the third cycle more emphasis was put on controlling the grout pressure.
Two main stages of load tests were conducted in the second phase. In the first stage the lower level O-cells were pressurized in 28 loading increments, each 0.9 to 1.0 MN, while the upper level O-cells were kept closed to assess the improvement in end bearing after grouting. As shown in Fig. 13, after the final loading increment, an end bearing load of 27 MN was achieved with 44 mm tip settlement (1.8%D), while for a 1%D settlement, the measured end bearing capacity after shaft tip grouting was 5.3 MPa, which agrees well with the value predicted by the Mullins et al (2006) approach. This level of gain in end bearing was satisfactory and showed that the process of post-grouting of shaft tips can be reliably depended on to obtain the required design axial load capacity of the shafts while meeting the project settlement limit and eliminating the risk associated with drilling-induced soil disturbance and remaining debris at the tip of the shaft.

To measure the skin friction response along the shaft after grouting, a second stage of loading was conducted as part of the second phase of the test. This time, the upper level O-cells were pressurized in 1.6 to 1.7 MN load increments while the lower level O-cells were unlocked. The test was stopped when the upper shaft segment ultimate skin friction was reached after moving 106 mm upward, but before reaching the ultimate skin friction of the lower segment, as shown in Fig. 14. From this test, it can also be noticed that the lower 28 m long shaft segment close to grouted tip has a much stiffer skin friction load-displacement response than the upper 77 m segment. This is also evident from the load distribution curves based on strain gauge measurements shown in Fig. 15.

Scour Protection

The conceptual scour design for the two main piers was performed by COWI A/S, Denmark. The detail design was performed by Jiangsu Provincial Communication, Planning & Design Institute. Hydraulic studies and surveys were performed by Nanjing Hydraulic Research Institute.

The hydraulic design parameters for the scour protection were a combination of the current, water level and in some cases, waves acting at the same time. See Fig. 16.
The pile group width perpendicular to the river current is 48 m and the length is about 112 m. The hydraulic model tests showed the extension of the scour around the structure to be essentially equal in all four directions, approximately 60 m.

The ideas presented for scour protection were developed based on COWI's experience in combination with their understanding of the very difficult conditions in Yangtze River with deep water, high currents and high sediment transport.

The major problem associated with the scour protection was its construction. The scour protection in itself had to be made in a way that it was not too difficult to construct, and also, that it would prevent scour during construction. It was assessed that if the bridge piers were made without prior scour protection, the development of scour would be so rapid that it would be difficult to construct the scour protection later on and the bed level would have eroded to such a low level, that the advantage of the existing bed levels would have disappeared. Therefore, the scour protection as presented in Fig. A-7 in the Appendix was designed in such a way as to allow for construction of piles through the center of the temporary scour protection and then later on the final scour protection could be introduced.

In addition, due to the very high flow velocities and high sediment transport, the adopted scour protection scheme would need to be relatively simple and robust and not require very accurate dredging levels before placing of the material in the scour protection.

It was also desirable to construct the protection in smaller sections that together would constitute the total protection. The final protection should also be robust and be able to function with unavoidable inaccuracies.

With these objectives in mind, the designers refrained from the use of large prefabricated mattresses, gabions or large bamboo/willow mattresses. Such solutions could be used but would be difficult to handle and place in the very high currents prevailing at the site.

The principal ideas for the scour protection of the pylons of the Sutong Bridge included the use of three distinct areas or zones.

(1) **The Central Area or Inner Zone**

This zone includes the central area where the bridge piles for the main pylons and temporary structures are present.

The area extends 20 m away from the structures. In this area, the river bed would be temporarily protected by use of layers (3 nos.) of sand-filled geotextile bags. See Fig. 17.

The idea behind this concept is that by this action, the river bed will be protected but it will still be possible to bore the piles through the protection. After completion of the piling, the final protection was constructed with a filter layer of quarry-run and minimum 2 layers of armour stones (rock).

(2) **Outer Area**

Beyond the inner zone, the Outer Area is situated. It extends about 40 m further out from the Central Area. The scour protection consists of one layer of sand bags covered with a layer of quarry-run on top of which was placed the same type of rock armour as for the central area.

(3) **The Falling Apron Area**

Outside the Central and Outer Area is the Falling Apron Area. Its width varies according
With respect to the armor stones, it was essential that the structural integrity be obtained. Therefore, it was crucial that 2 layers of armor stones are present in all areas. In layer thickness, this corresponds to 1.0 m for the Central Area and the Outer Area. For the inner section of the Falling Apron Area, it corresponds to 1.2 m thickness.

The scour protection is a flexible structure that will be subject to some displacement of material. Especially the Falling Apron will be moving during launching when scour occurs at its edges. Therefore a detailed monitoring program was prepared covering the entire bridge alignment.

The solution adopted, with sand bags and stone layers dumped from the water surface, was found to be the most feasible under the given difficult circumstances with water depth up to 30 m, high currents and zero visibility. The future erosion at the edges of the protection will be prevented from progressing close to the bridge piers by the use of the Falling Apron concept for the outer edge of the scour protection.

See Fig. A-6 and A-7 in the Appendix for further details of the scour protection system.

### Pile Cap Construction

The 13.3 m deep pile cap for each pylon is positioned at the water line with the bottom at Elev. -7.0 m and the top at +6.3 m. The caps were constructed by first building a double-walled steel caisson in-the-wet, directly above the final location. The 1.8/2.0 m thick double wall or perimeter wall was constructed as a watertight compartment and served four basic functions. See Fig. 18.

It first served as the perimeter stiffening frame that gave the caisson its rigidity during lowering operations. Secondly, it served as the buoyancy tanks to minimize the deadweight of the caisson as it was lowered into the water and down to final grade at Elev. -10.0 m. Third, it served as a temporary cofferdam and exterior permanent form for the casting of the pile cap. And finally, the perimeter wall acted as a permanent ship-impact protection fender during the service life of the bridge. The perimeter wall was filled with concrete below Elev. -1.0 m.
The first caisson (North Pylon) when initially constructed was 118 m by 52.4 m in plan, 7 m high, and weighed approximately 3050 tonnes. See Appendix Fig. A-3. The bottom of the caisson was a steel plate stiffened by steel trusses that spanned the full width of the caisson and tied into the perimeter walls. The bottom deck of the caisson started out at approximately Elev. +6.0 and was lowered in three basic stages to Elev. -10.0. The first stage lowered the caisson approximately 5 m, at which point the caisson was floating under its own buoyancy. At the end of this stage, the lowering was stopped and the perimeter walls were increased to a height of 18 m and the lowering was completed in two stages to final grade by partial flooding of the cofferdam.

The lowering operation was performed with 16 strand jacks, DL-S418 (See Figs. 19 and 20.) supplied and operated by Dorman Long Technology, Ltd. For the layout of strand jacks see Appendix Fig. A-4.

All 16 jacks were spaced along the perimeter wall of the caisson and sat on support frames positioned over the top of the exterior drill shaft casings. Each jack had a safe working load of 418 tonnes, thus providing a safety factor of 2.2. The entire caisson was quite stiff and relative movements of only 10 mm between adjacent jacking points created a 35% differential loading. The entire lowering operation was controlled with a Dorman Long P40 computer control system which provided communications between jacks, power-pack and control computer. The lowering operation was performed with strokes of 200 mm and a stroke range of only 5 mm to ensure stable balanced loads between jacks.

Once the 13 m high caisson reached final grade at Elev. -10.0, the caisson was locked in position and the annulus between the drilled shaft casings and the steel plate of the caisson bottom deck was sealed. A 3.0 m deep tremie concrete seal was then placed over the entire bottom area of the caisson except for the 2 m
wide exterior wall. After the tremie seal attained specified strength, the caisson was dewatered (See Fig. 22) and the rest of the pile cap was constructed in the dry. See Fig. 23.

For the second caisson (South Pylon), the entire caisson was assembled full height and weighed approximately 5,800 tonnes. This caisson was lowered by a strand jacking system from Tonji University to a self-floating condition, and final grade was reached by partial flooding of the exterior perimeter walls. Once at final grade, the tremie seal was placed and the rest of the South Pylon was constructed in the dry. Both lowering operations worked well and everything went smoothly.

**Conclusion**

The foundation design and construction team on the Sutong Bridge have succeeded in constructing foundations for a world record setting bridge at a very challenging site on the lower Yangtze River. The design team, working in conjunction with their construction counterparts, developed innovative holistic solutions that addressed the very rigorous requirements of the structural design while remaining fully constructible under extremely difficult conditions.

Post-grouting of shaft tips was found to be an effective and economical procedure that was implemented to increase the axial capacity of the drilled shaft foundation of the Sutong Bridge in China. By preloading and compaction of the soil and any remaining debris at the shaft tips, the end bearing capacity can be significantly increased and reliably depended on. This was validated in O-cell tests conducted on shafts before and after tip grouting. Another advantage of post-grouting of shaft tips is the ability to check the axial capacity of each production shaft through measurement of grout pressure and upward movement of the top of the shaft.

**Acknowledgement**

The authors wish to thank the Jiangsu Provincial Sutong Bridge Construction Commanding Department for the opportunity to assist them on this very challenging bridge project.

**References:**


**Appendix:**

1. Fig. A-1 Main Span Side Elevation
2. Fig. A-2 Pylon Details
3. Fig. A-3 Main Pylon Caisson Details
4. Fig. A-4 Caisson Lowering System
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Deep Diaphragm Wall Activities at RandstadRail Project in Rotterdam, The Netherlands

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The Netherlands RandstadRail project makes use of a large range of sophisticated geotechnical techniques to allow a shield driven tunnel to approach the central station of Rotterdam. This paper describes the construction of three deep excavation pits constructed with 1.2 m and 1.5 m thick diaphragm walls. Besides the presentation of the actual diaphragm wall activities special consideration is given to the break-in and break-out situations of the Tunnel Boring Machine (TBM) into the deep excavations pits using sealing blocks and partly glass fibre reinforcement in the diaphragm walls.

The Project
The project RandstadRail in the Dutch province of South Holland will provide a new light rail train connection between the cities of Den Haag and Rotterdam. While the project makes extensive use of the existing track from Den Haag to Rotterdam, it is necessary to construct a nearly 3 km-long tunnelled section for a direct connection to the central station of Rotterdam, where by another contract an interchange station to the Rotterdam's Metro system will be established. The tunnel below Rotterdam represents, from an engineering perspective, the most challenging part of the whole RandstadRail project and is let as a Euro 178M contract known as The Statenwegtracé contract.

The tunnel will be made up of two 2.4 km-long single-track shield driven tunnel tubes, the first bored tunnel below an urban area in The Netherlands, with an outer diameter of 6.5 m. The tunnel will be driven using a hydro shield tunnel boring machine (TBM) with a diameter of 6.78 m. The remaining 600 m will be constructed as a cut and cover tunnel, mainly using construction pits with retaining sheet pile walls and combi walls. The launch shaft, the construction pit for the new underground station Blijdorp in the middle of the bored tunnel track, and the arrival shaft will be realised applying diaphragm walls.

The project contractor SATURN v.o.f. (Samenwerking Tunnelrealisatie Nederland) is a joint venture of two contractors, namely Ed. Züblin AG from Germany and The Netherlands firm Dura Vermeer Group N.V. The client for the project is the public transport company of Rotterdam RET (Rotterdamse Electrische Tram).

The project management in the tender and construction phase, including the principle design and the site supervision is carried out by the engineering office of the municipality of Rotterdam (Ingenieursbureau Gemeentewerken Rotterdam). Ed. Züblin’s subsidiary Züblin Spezialtiefbau GmbH (Züblin Ground Engineering) is in charge of all geotechnical and foundation works on the Statenwegtracé contract. The project was contracted in April 2004 and is scheduled for completion end of 2008.

Particularly complex foundation and geotechnical work is needed for the tunnel launch and reception shafts, the new cut and cover station midway along the tunnel section, and retaining support for the cut and cover tunnel. At the starting and arrival situation of the TBM different types of soil improvement activities such as lime-cement columns, permeation grouting and jet grouting are required to allow the tunnelling.

[Fig. 1] Aerial View on Tunnel Track in Rotterdam
Ground conditions

Ground conditions, although typical for Southern Holland, are particularly difficult for tunnelling and deep excavations. They include deep deposits of soft clay and peat and with the groundwater level just below ground surface, retaining elements need to extend to at least 35 m to reach an impermeable cut-off.

The soil investigation carried out by the client before the tender included around 600 cone penetration tests (CPT) and around 60 borings across the whole tunnel track. Fig. 3 shows a typical CPT at the start situation of the tunnel.

In detail the soil profile consists of between 2 to 5 m of refilled sand overlying geologically young Holocene soils. These are made up of alternating layers and lenses of soft clay and peat to a depth of between 15 to 18 m.

These in turn overly denser Pleistocene sand to depths of between 35 to 38 m. The soil characteristics of the Holocene and Pleistocene are shown in Table 1. The water table in the Pleistocene is pressurised and any exchange with the free water table in the refilled upper sand layers due to building activities has to be avoided. Below this is the impermeable layer of Kedichem, an over consolidated sand, clay and peat, which provides the target layer for retaining elements to achieve a natural seal and prevent the inflow of groundwater into the deep excavations.

<table>
<thead>
<tr>
<th>Table 1</th>
<th>Soil Characteristics</th>
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<tbody>
<tr>
<td></td>
<td>Holocene</td>
</tr>
<tr>
<td></td>
<td>clay</td>
</tr>
<tr>
<td>$\gamma/\gamma_r$ [kN/m³]</td>
<td>14 – 16</td>
</tr>
<tr>
<td>$c$ [kN/m²]</td>
<td>10</td>
</tr>
<tr>
<td>$\phi$ [°]</td>
<td>13 – 20</td>
</tr>
<tr>
<td>$k$ [MN/m³]</td>
<td>3.0 – 3.75</td>
</tr>
</tbody>
</table>

The vertical alignment of the tunnel as can be seen in Fig. 2 was designed to situate the major part of the tunnel in the Pleistocene sand layer, which is advantageous for the tunnel drive and the permanent stability of the tunnel linings.

Deep Excavation Pits

Three deep excavation pits have to be constructed for this project: the launch shaft, the cut and cover pit for a new underground station and the arrival shaft. The retaining walls for all three excavation pits are carried out with diaphragm walls up to a depth of 42 m and a thickness of 1.2 m and 1.5 m.

The largest of these three excavation pits is the mid-way along the twin 2.4 km long tunnel drives positioned pit for the new Blijdorp Station, that will be excavated before the TBM passes through. Including the sealing blocks the excavation pit has a length of nearly 150 m and a width of around 25 m. The plan view of the pit with the surrounding buildings is shown in Fig. 4. The excavation depth of the pit is 22 m below ground level and the diaphragm walls reach to a depth of 42 m giving an embedded length of around 4 m in the impermeable layer of Kedichem. This way the pit is sealed against inflow of groundwater from beneath.
The thickness of the diaphragm walls varies between 1.2 m and 1.5 m, depending on the distance to the surrounding pile founded buildings. In the smaller part of the pit, where the adjacent buildings are only 7.2 m from the construction pit, the client applied the 1.5 m thick diaphragm walls, to reduce the bending of the retaining walls and the influence on the foundation piles of the close building. In this part of the construction pit the panel length was also restricted to 3.0 m by the client. For the rest of the diaphragm walls the structural analysis of the trench stability carried out by Züblin’s geotechnical design office allowed a panel length of 8.0 m. This calculation had to be carried out according to the German standard DIN 4126 with an increased safety factor of 1.3 instead of 1.1 and by close adjacent buildings 1.5 instead of 1.3. This regulation was resulting in L-shaped guide walls for the trench excavation that had to reach 1.0 m above ground level.

For the improvement of the impermeability the steel joint elements will be provided with rubber waterproof sealing strips, that will stay in the concrete of the primary panel while detaching the steel element.

As the station is excavated, internal support will be provided by massive tubular struts and walings in four layers. The diaphragm walls will be part of the final structure of the station and are used as foundation elements of the station. They will form together with a second reinforced concrete wall, which will be cast after the TBM passes through the final walls of the station. After removal of the steel struts and walings the station walls will be supported permanently just by the concrete base slab and roof slab. The thickness of this combined final station wall will be 2.15 m. This allows a maximum span of 16 m between the base and roof slab.

This design resulted in unusually heavy reinforcing cages reaching the full depth of the diaphragm walls. The cages have two layers of
40 mm reinforcement bars at 175 mm centres on both faces over nearly the full length of the cages. Additional screw joints had to be installed in the reinforcing cages to connect the diaphragm walls with the internal walls. Because of space limitations of the inner city construction site, the cages are fabricated off-site in three sections, transported to the site and joined together as they are installed in the trench panel. Once assembled each cage for a 42 m-deep by 3 m-wide panel weighs up to 45 tons. The overlap of the reinforcing bars for the two layer reinforcement had to be shifted, resulting in a total overlap length of 3.5 m. To be able to connect the three parts of the heavily reinforced cages without difficulty, it was necessary to crank the overlapping bars.

To form the box structure of the station and the sealing blocks L-shaped and T-shaped panels were used. Also the reinforcement cages had to follow the form of these panels in one piece in their horizontal layout. These special panel cages weighed over 50 tons.

For the construction pit of the Blijdorp Station a total of 4,500 tons of reinforcement for a diaphragm wall area of 15,000 m² had to be installed. For all three deep excavation pits in total around 7,500 tons of reinforcement is installed into the diaphragm walls.

**Sealing blocks**

For the three deep excavation pits the break-in and break-out situation of the TBM is handled by creating sealing blocks in the underwater concrete construction method. The underwater excavation is carried out, after pits of about 9 m length and 22 m width are constructed using 1.2 m or 1.5 m thick diaphragm walls. During the excavation the water table is not lowered and after reaching the final excavation depth the sealing blocks receive an underwater concrete slap of 1.2 m thickness. Above that the sealing block pits are filled with unreinforced low strength concrete about 3.5 m above the later tunnel drive. The remaining pit until the ground level will be filled with sand.

Because the cutting wheel tools of the TBM cannot cut through reinforcement bars in concrete structures, the area through which the machine has to pass must be cleared of any reinforcement before the start of the TBM drive (Fig. 11). The sealing block will prevent water and soil entering the excavation pit while creating the opening in the diaphragm walls for the shield passage. For the circular openings,
that are made out of the excavated construction pits, the concrete and the reinforcement bars of the diaphragm walls will be demolished within a distance of 500 mm around the tunnel diameter. In front of the diaphragm wall then a precast rc-wall will be installed, on which the starting seal construction is fixed (Fig. 12).

The diaphragm walls situated on the opposite side of the sealing block also have to be penetrated by the TBM and were therefore designed to be unreinforced in the area of the TBM passage. To prevent the earth pressure acting on this unreinforced diaphragm walls during the underwater excavation of the sealing blocks different means were taken at the three deep construction pits.

For the launch shaft the sealing block is excavated and refilled at the same time as an adjacent excavation pit. This excavation pit, surrounded by retaining sheet pile walls and tied back above groundwater level using single bar grout anchors, is needed to replace the soft Holocene layers by sand, because the soft layers would not provide sufficient stiffness to take the forces from the tunnel lining. After the sand refill has reached

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**[Fig. 10]** Cross Section through Low Strength Concrete Sealing Block at Blijdorp Station with Indicated Tunnel Tubes

**[Fig. 11]** Opening of Diaphragm Wall

**[Fig. 12]** Sealing Construction

**[Fig. 13]** Longitudinal Section through Sealing Block at Blijdorp Station with the Transition Zone that was Replaced by GFRP Reinforcement in the Left Sided Diaphragm Wall
above the groundwater table it is compacted using the deep vibro compaction method. Just before the arrival shaft a monolithic block of jet grouting of about 15,000 m³ is installed beneath and next to the railway embankment of the central station. The jet grout block is reaching until the sealing block, protecting the unreinforced diaphragm wall against earth pressure during the underwater excavation of the sealing block.

The original proposal according to the client’s design for the Blijdorp Station was to form additional transition zones of lime-cement columns, 4 and 6 m long, just before the two sealing blocks for the break-in and break-out of the TBM (shown in Fig. 13)

The method of lime-cement columns, also known as dry deep mixing, refers to in-situ mixing of soil with the addition of binders in a dry form. The mixing is carried out by means of mechanical equipment, typically using rotating single mixing tools. The mixing tool is first rotated into the soil down to the final depth of the column. The binder is fed through a nozzle in the mixing tool with compressed air from a separate binder tank and is then mixed with the soil during retraction of the mixing tool. The dry deep mixing method was developed in Scandinavia for the soil improvement of soft soil such as clay and peat. It is not suitable for the improvement of sand layers of larger extent, especially when the columns are applied in an overlapping grid. High strength of the lime-cement-columns in sand cause problems during intrusion and retraction of the mixing tool and bear the risk of loosing the mixing tool due to breakage.

The client’s design asked for transition zones made of lime-cement columns that had to reach until 8 m into the Pleistocene sand.

Züblin came up with an alternative option, to apply glass fibre reinforcement in the diaphragm walls, which had originally been unreinforced and had to be driven through by the TBM. The glass fibre reinforcement (GFRP = glass fibre reinforced polymer) can easily be crushed by the cutting wheel tools of the TBM. The original transition zone of lime-cement columns could completely be abolished by applying the “soft eye” option.

Besides technical advantages it was a faster and cheaper option and was therefore accepted by the client.
Glass fibre reinforcement is manufactured by a process whereby high-strength glass fibre is drawn through a form and immersed in synthetic resin. The impregnated fibres are then drawn through a mold and toughened. The result is a material with a very high tensile strength, even much higher than conventional steel. Perpendicular to the load carrying direction of the fibres it can be trimmed very easily, especially when it is embedded in concrete. Together with the concrete the GFRP bars will be crushed without major abrasion to the TBM cutting wheel tools.

For each of the in total four break-in and break-out situations at the Blijdorp Station a trench panel of 7.5 m length was located in the area of the later tunnel drive. In each of these panels three reinforcing cages were installed next to each other. To reduce the amount of the expensive glass fibre reinforcement, the height of the “soft eye” was just 7.5 m. That means that the area of the “soft eye” is just 350 mm larger than the TBM borehead. This measurement was determined by taking into account the tolerance of the TBM drive and the installation of the reinforcement cages.

Above and below the GFRP-cages normal steel reinforcement cages were brought into the trench and joined with the GFRP-cages. Due to the subdivision into pure glass fibre and pure steel cages it was not necessary to use a temporary steel frame for lifting (see Fig. 14). Each glass fibre cage was equipped with two layers of 32 mm GFRP-bars 160 mm centres at the earth side and one layer at the side of the sealing block. The shear forces in the diaphragm wall are taken by GFRP double-head anchor-bolts.

Summary

The execution of the diaphragm walls for all three excavation pits was successfully finished in spite of difficult ground conditions and extraordinary dimensions. By now the launch shaft is excavated, the base slab is cast and the opening in the diaphragm wall and the installation of the sealing construction for the shield passage is completed. The TBM drive successfully started at the of end 2005.

Besides the large-scale diaphragm wall activities this project is very special in regard to the high concentration of different types of soil improvement works at the starting and arrival
point of the TBM. At the starting point, where the tunnel is still mainly situated in soft Holocene clay and peat layers a monolithic block of more than 38,000 m³ of lime-cement columns has been created to increase the stiffness of the soil.

Permeation grouting has to be installed underneath a running railway line, where the tunnel crosses the loose packed sand embankment. Soft gel grouting is used at this position to prevent liquefaction of the sand and to prevent the communication of the ground water tables in the Pleistocene and in the sand deposit. Furthermore, a block of hard gel grouting has to be created in order to stabilise existing wooden piles that will be cut off during the tunnel boring process and to transfer the toe load of these piles to a higher level so they do not penetrate the tunnel lining.

Before the TBM reaches the arrival shaft next to the central station it has to cross the main railway embankment of the central station. Underneath and next to this embankment a monolithic block of jet grouting of around 15,000 m³ has to be installed. One third of these jet grout columns, that have to be installed below the embankment, will be produced with an inclination of less than 45° and a length of 24 m from besides the embankment. Furthermore, ground freezing is going to be used to build in total five cross passages between the twin tunnel tubes.

This wide range of complex geotechnical methods at the RandstadRail project in Rotterdam leaves still a lot of interesting aspects to be presented in future.

References


Design of Drilled Shafts Supporting Sound Walls

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Drilled shafts are widely adopted as the foundation for sound walls. However, there has been a lack of uniformity in design and analysis methods and design criteria, in terms of factor of safety against ultimate capacity failure as well as the allowable deflection. In order to establish a uniform design methodology for the drilled shafts supporting sound walls in cohesive and cohesionless soils, respectively, a database of full-scale lateral load tests on fully instrumented drilled shafts was collected. Based on the compiled database, existing design methods and design criteria of laterally loaded drilled shafts were evaluated. Broms method and COM624P (or LPILE) are suggested as the design methods for drilled shafts supporting sound walls in both cohesive and cohesionless soils. Additionally, the corresponding design criteria, including factor of safety and permissible deflection, for both design methods are recommended. Two full-scale lateral load tests on fully instrumented drilled shafts were subsequently conducted in Colorado to further verify the design recommendation. A comprehensive geotechnical investigation program was also carried out at the two new lateral load test sites that included pressuremeter test, SPT, as well as laboratory triaxial consolidated undrained tests and direct shear tests on the soil samples taken from the lateral load test sites. The test results obtained at these two load test sites were employed to validate the recommended geotechnical design and geotechnical testing methods for the drilled shafts supporting sound walls.

Introduction

Sound walls are frequently constructed along portions of highways nearing residential areas. Due to economy and simplicity, drilled shafts have been widely used as foundations for sound walls. The primary loadings to sound wall foundations are lateral loads and moments from wind loads. The most common methods used by most of state Department of Transportation (DOT) for design of drilled shafts under lateral loads from wind loads are divided into two categories:

- Ultimate capacity based design methods to ensure adequate margin of safety (e.g., Brinch Hansen method 1961, Broms method 1964a and 1964b, Davidson et al. 1976, Bang & Shen 1989);
- Serviceability design methods to ensure that shafts lateral displacement is smaller than a predefined tolerable displacement (e.g., NAVFAC DM-7 1971, COM624P or LPILE).

Different engineers even in the same DOT use different design methods. The factor of safety (FS) and permissible deflections used with these methods are not uniform, sometimes even for the same design method. These methods are not intended to provide comparable levels of safety because they are for different purposes; if they provide comparable FS then one method will be adequate. Methods for determining pertinent soil parameters needed in both types of analysis (ultimate capacity and deflection prediction) also differ and have not been consistently evaluated for their applicability and accuracy. All these factors make the design very conservative and lack uniformity that could lead to high construction costs for these shafts.

Several past research studies were conducted to identify the appropriate design approach for sound walls. Boghrat (1990) discussed and compared four other design methods (TRR 616 method, Woodward and Gardner method, New York Department of Transportation Method, and North Carolina Method) by using hypothetical cases; however, no design recommendation was made due to the lack of full-scale field test data for verification. Helmers et al. (2000) conducted lateral load testing on model drilled shafts having diameters of 203 mm (8 inches) or 229 mm (9 inches) in partially saturated silts and clays at five sites in Virginia. A comparison between measured lateral capacities and predicted values by using Broms method (1964b) and Brinch Hansen method (1961) was performed. Based on the comparison, the use of Brinch Hansen method (1961) was recommended for the design of drilled shafts to support sound walls in partially saturated silts and clays, in which a reduction factor of 0.85 on the predicted capacity was also suggested.
It can be seen that previous studies have not been validated with prototype load test data. In this study, therefore, full-scale lateral load test data is used to identify the suitable design methods and acceptance criteria for the drilled shafts to support sound walls in cohesive and cohesionless soils, respectively. The current practice including analysis methods and acceptance criteria are critically reviewed. The correlations between SPT N values and soil parameters are reviewed and summarized in this paper. A lateral load test database on the drilled shafts supporting sound walls was collected to consist of seven full-scale field tests of fully instrumented drilled shafts in clay conducted in Ohio (Liang, 1997) and five tests in sand from open literature (Bhushan et al. 1981). Based on the selected database and soil parameters determined from Liang’s SPT correlation charts (Liang, 2002), the most common design methods presented before were evaluated by comparing the predictions from these design methods with results from lateral load tests. Based on this evaluation, appropriate design methods and acceptance criteria for drilled shafts to support sound walls in cohesive soils and cohesionless soils are identified. A design approach incorporating both strength limit based design and the serviceability based analysis was recommended that ensures a more consistent design outcome with comparable margin of safety from both capacity and deflection viewpoints. Furthermore, two lateral load tests of fully instrumented drilled shafts constructed at a sand soil deposit and a clay soil deposit, respectively, near Denver, Colorado were conducted by Nusairat, et al. (2004). A comprehensive geotechnical investigation program was also carried out at the two new lateral load test sites that included pressuremeter test, SPT, as well as laboratory triaxial consolidated undrained (CU) tests and direct shear tests on the soil samples taken from the lateral load test sites. This also allowed for evaluation of the accuracy of various testing methods for determining the soil parameters for the design methods for sound walls. The overall test results were used to verify the recommended design and testing methods.

Review of Current Practices

Analysis Methods

In current practice, both allowable deflection based design methods and strength limit based design methods described in the following are used for sound wall foundation design (Nusairat et al., 2004).

Brinch Hansen method (1961) is based on earth pressure theory for c-w soils. The method is only applicable for short piles (drilled shafts), and a trial-and-error procedure is needed in the calculation to locate the point of rotation. Broms method (1964a, 1964b) is only suitable for homogeneous soils, either cohesive soils or cohesionless soils. However, it can be applied to short drilled shafts or long drilled shafts; and the shaft head can be free or restrained. In AASHTO “Guide Specifications for Structural Design of Sound Barriers, 1989”, the Sheet Piling Method is suggested for the design of piles supporting sound barrier walls. The Sheet Piling Method was initially developed for sheet piles embedded in cohesionless soils. For cohesive soils, assumption of friction angle has to be made and the cohesion is assumed to be zero. Since it is rather difficult to make any rationale assumption about the equivalent friction angle, the Sheet Piling method is not used in this paper for drilled shafts embedded in clay.

NAVFAC DM-7 method (1971) is based on Reese and Matlock’s (1956) non-dimensional solutions for laterally loaded piles with soil modulus assumed proportional to depth. The limitations of NAVFAC DM-7 method (1971) are that the lateral load cannot exceed about 1/3 of the ultimate lateral load capacity and only elastic lateral response can be predicted. COM624P (Wang and Reese, 1993) program based on p-y method (Reese et al., 1974 and 1975), or the equivalent commercial program, LPILE, has been widely used for decades. This method treats soil as Winkler foundation which may introduce a small amount of inaccuracy because it ignores the interactions between the discrete springs.

Design Criteria

For ultimate capacity based design methods, an appropriate FS has to be determined. In Colorado Department of Transportation (DOT) practice, 2.5 to 3 have been adopted as an overall FS (Nusairat et al., 2004). In this paper, the FS in the range of 2 to 3 will be investigated. For service limit based design methods, the allowable deflection at ground level needs to be known for the design of drilled shafts to support sound walls. According to Colorado DOT practice (Nusairat et al., 2004), 0.25 to 0.5 inch (6.4 to 12.7 mm) of deflection at the ground line is considered to be acceptable. Most engineers cited 1 inch
At the ground under service loading conditions as a maximum, and some stated that deflections greater than 1 inch (25.4 mm) may be acceptable in some situations. Deflections at the bottom of the shafts are normally checked to ensure that it is a very low number to be nearly equal to zero. A tilting of the sound barrier walls of 0.833% was established for a big project in Colorado. This was selected based on aesthetic not structural concerns. In Ohio DOT practice, the allowable wall top deflection is 1% - 1.5% of wall heights (Liang, 2002).

A relationship between the wall top deflection ($\Delta_w$) and the shaft head deflection ($\Delta_p$) can be derived as follow, if rigid body rotation of shaft and wall along shaft tip is assumed.

$$\Delta_p = \Delta_w \frac{L}{H_w + L}$$  \hspace{1cm} (1)

in which, $H_w$ is the distance between wall top and ground line; and $L$ is the length of embedded drilled shafts. Considering typical drilled shaft length of 9 to 15 feet (2.7 to 4.6 m) and wall height of 14 to 18 feet (4.3 to 5.5 m) in Colorado DOT and Ohio DOT practice, then 0.6 to 1.5 inch (15 to 38.1 mm) of permissible shaft top deflection could be calculated from Equation 1 based on Colorado DOT and Ohio DOT permissible deflections at wall top. Therefore, a range of permissible deflections at drilled shafts head (0.6", 1.0", and 1.5" or 15, 25.4, and 38.1 mm) will be investigated.

### Soil Parameters Determination Methods

The soil parameters for p-y analysis and capacity based design methods could be obtained from geotechnical laboratory tests, SPT tests, and pressuremeter tests. Because the soil information given in the selected test database (to be introduced later) are basically in the form of SPT boring logs; the soil parameters in this study are determined from correlations with SPT N values for consistency. Anderson and Townsend (2001) evaluated several existing SPT correlations (such as Terzaghi, 1955) against 24 SPT test data in cohesionless soils based on p-y analyses. They concluded that little difference exists and the correlations are conservative. The SPT correlations for clay (such as Hegedus and Peterson, 1988) and aforementioned SPT correlations for sand were investigated against 21 lateral load tests in sand and 37 lateral load tests in clay by Liang (2002).

<table>
<thead>
<tr>
<th>Table 1</th>
<th>Correlations of SPT for Cohesive and Cohesionless Soils (After Liang, 2002)</th>
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<tr>
<td><strong>Cohesive Soils</strong></td>
<td></td>
</tr>
<tr>
<td>SPT-N$_{60}$</td>
<td>0 to 2</td>
</tr>
<tr>
<td>$S_u$ (psi)</td>
<td>0 to 1.88</td>
</tr>
<tr>
<td>$\varepsilon_{50}$</td>
<td>&gt; 0.02</td>
</tr>
<tr>
<td>$k_s$ (lb/in$^3$)</td>
<td>&lt; 30</td>
</tr>
<tr>
<td>$\gamma_{sat}$ (pcf)</td>
<td>100 to 120</td>
</tr>
</tbody>
</table>

| **Cohesionless Soils** |  |
| SPT-N$_{60}$ | 2 to 4 | 4 to 10 | 10 to 20 | 20 to 30 | 30 to 50 | 50 to 60 |
| $\phi$ (°) | 28-29 | 29-31 | 31-34 | 34-37 | 37-42 | 42-45 |
| $k_s$ (lb/in$^3$) | A. W. T. | 20-25 | 25-60 | 60-90 | 90-160 | 160-240 | 240-260 |
| B. W. T. | 15-20 | 20-40 | 40-60 | 60-90 | 90-130 | 130-150 |
| $\gamma_{moist}$ (pcf) | Min. | 104 to 108 | 108 to 112 | 115 to 120 | 120 to 125 | 124 to 128 | 128 to 130 |
| Max. | 114 to 118 | 120 to 124 | 122 to 130 | 128 to 132 | 130 to 145 | 140 to 145 |

Note: $S_u$ = undrained shear strength; $\varepsilon_{50}$ = strain at 50% of maximum deviatoric principle stress; $k_s$ = coefficient of subgrade reaction modulus; $\gamma_{sat}$ = saturated density of soils; $\phi$ = friction angle; $\gamma_{moist}$ = wet density of soils; A.W.T. = above water table; B.W.T. = below water table.

1 psi = 6.9 kPa; 1 lb/in$^3$ = 27.7 g/cm$^3$; 1 pcf = 16 kg/m$^3$. 
The correlation study and extensive sensitivity study have led Liang (2002) to propose modified SPT correlations shown in Table 1, which could provide best match with p-y analysis results. Because the correlation suggested by Liang (2002) was intended for deriving soil parameters for COM624P (or LPILE) program, it was adopted in this study to obtain necessary soil parameters from SPT N values.

**Lateral Load Test Database**

There are quite a few lateral load test data available in the literature, such as Florida Department of Transportation’s database compiled by the University of Florida. However, only a small part of the existing test data is related to the shaft diameter between 20 inch (508 mm) and 36 inch (914 mm) and shaft length between 6 feet (1.8 m) and 20 feet (6.1 m), which are commonly found for sound wall foundations. After examining the available test data, only 7 lateral load tests in clay were selected from load tests in Ohio (Liang, 1997), and 5 load tests in sand by Bhushan et al. (1981) was selected. To enlarge the database for drilled shaft tests in sand, drilled shafts with 42 inch (1.1 m) and 48 inch (1.2 m) diameter were also included. The undrained shear strength of cohesive soils and friction angle of cohesionless soils were determined using Liang (2002) correlation. The average weighted strength values for the entire soil layer are summarized in Table 2. The information of drilled shaft dimension and the moment arm (the distance between load points and ground level) are also included in Table 2.

Usually, lateral load tests do not reach the stage of complete soil failure; therefore, the ultimate lateral capacity is not directly available from test results. There are two kinds of failures: one is the drilled shaft structure failure, the other one is the failure of soils which is defined as the appearance of excessive deflection under very small increment of load. Kulhawy and Chen (1995) developed a hyperbolic curve fit technique to simulate the non-linear load-deflection behavior and to predict the ultimate capacity of piles (drilled shafts). The hyperbolic equation in terms of the lateral load (H) and the lateral deflection (δ) can be expressed as follows:

\[
H = \frac{\delta}{a + b\delta}
\]

where a and b are curve fitting constants. The ultimate lateral load capacity is defined as the deflection δ become infinite large and is calculated as 1/b.

**Evaluating Results and Recommendations**

**Ultimate Capacity Based Design Methods**

For drilled shafts in cohesive soils, Fig. 1 presents the comparison of measured over predicted

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Testing Shaft</th>
<th>Undrained Shear Strength (psi)</th>
<th>Friction Angle (degree)</th>
<th>Embedded Length (ft)</th>
<th>Moment Arm (ft)</th>
<th>Diameter (inch)</th>
</tr>
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<tbody>
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<td>39</td>
<td>17</td>
<td>0</td>
<td>42</td>
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<td></td>
<td>Pier 5</td>
<td>0</td>
<td>41</td>
<td>18</td>
<td>0</td>
<td>36</td>
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<td></td>
<td>Pier 6</td>
<td>0</td>
<td>40</td>
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<td>0</td>
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<td></td>
<td>Pier 7</td>
<td>0</td>
<td>40</td>
<td>18</td>
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<td>48</td>
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<tr>
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<td>23</td>
<td>0</td>
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<tr>
<td></td>
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<td>23</td>
<td>0</td>
<td>9.5</td>
<td>0</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td>I-90P100</td>
<td>18.7</td>
<td>0</td>
<td>10</td>
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<td>36</td>
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<tr>
<td></td>
<td>I-90P101</td>
<td>18.7</td>
<td>0</td>
<td>12</td>
<td>0</td>
<td>30</td>
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<tr>
<td></td>
<td>I-90S1</td>
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<td>0</td>
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<td></td>
<td>I-90S3</td>
<td>22.6</td>
<td>0</td>
<td>12</td>
<td>10</td>
<td>30</td>
</tr>
</tbody>
</table>

*Moment Arm is the distance between load point and ground line.
1 psi = 6.9 kPa; 1 ft = 0.305 m; 1 inch = 25.4 mm.
capacities using Broms and Brinch Hansen methods. The I70S2 test is not evaluated because the measured capacity could not be obtained. For the cases with zero feet of moment arm shown in Table 2, Broms method provides very close estimates with test results, except for one case. On the other hand, Brinch Hansen method provides aggressive and unsafe predictions in these cases. It seems that Broms method provides better prediction capability than Brinch Hansen method for these cases. For the three cases with 10 feet (3 m) moment arm shown in Table 2, Broms and Brinch Hansen method provide similar results. The ratio of the measured over the predicted capacities ranges from 2.0 to 2.5. Thus, it is found that Broms method and Brinch Hansen method provide much more conservative predictions on drilled shafts subjected to combined lateral load and moment than just lateral load. In general, Broms method provides more accurate and safer prediction than Brinch Hansen method.

For drilled shafts embedded in cohesionless soils, Fig. 2 provides the comparison of measured over predicted capacities by various methods. All methods provide conservative predictions for most of the cases. Broms method provides more conservative estimates than others; while Brinch Hansen method appears to provide unsafe prediction in one case. It is prudent to adopt a relative conservative method since load tests for evaluation were selected from one source. Therefore, Broms method (1964b) is suggested for estimating the ultimate capacity of drilled shafts in sand.

Service Limit Based Design Methods
The performance of COM624P (Wang and Reese, 1993) and NAVFAC DM-7 (1971) were investigated against the load test results in database. For drilled shafts in cohesive soils, comparisons among test results and analysis results of COM624P and NAVFAC DM-7 of drilled shaft I70S2 and I90S3 provide representative results as shown in Fig. 3 (a) and (b). It can be seen that NAVFAC DM-7 overpredicts deflections; while COM624P provides good agreement with measured load-deflection curves at initial portion and provides safe results in the non-linear portion.

![Graph 1: Measured Over Predicted Capacities of Drilled Shafts in Clay](image1)

For drilled shafts embedded in cohesionless soils, Fig. 2 provides the comparison of measured over predicted capacities by various methods. All methods provide conservative predictions for most of the cases. Broms method provides more conservative estimates than others; while Brinch Hansen method appears to provide unsafe prediction in one case. It is prudent to adopt a relative conservative method since load tests for evaluation were selected from one source. Therefore, Broms method (1964b) is suggested for estimating the ultimate capacity of drilled shafts in sand.

Service Limit Based Design Methods
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![Graph 2: Measured Over Predicted Capacities of Drilled Shafts in Sand](image2)

(a) Shaft I70S2

![Graph 3: Load-Deflection Curves at Head of Drilled Shafts](image3)

(b) Shaft I90S3

(1kip = 4.448 kN; 1 inch = 25.4 mm)
For drilled shafts in cohesionless soils, the result of Pier 5 provides typical comparisons among test results and analysis results of COM624P and NAVFAC DM-7 as shown in Fig. 4. Similar to the cases in clay, it can be seen that the NAVFAC DM-7 method provides larger deflection than measured in the initial portion; while COM624P provides good predictions in the initial portion of load-deflection curves but overpredicts deflection at high load levels.

(1kip = 4.448 kN; 1 inch = 25.4 mm)  
[Fig. 4] Load-deflection Curves at Head of Drilled Shaft Pier 5

Based on above evaluation results, COM624P (or equivalent program LPILE) is recommended for the service limit based design of the drilled shafts supporting sound walls in cohesive and cohesionless soils, since NAVFAC DM-7 cannot predict the load-deflection behavior well and the prediction is linear.

Factor of Safety and Permissible Deflection

To establish a sense of linkage between the shaft deflection and shaft capacity, the normalized ratios of measured over predicted capacities using COM624P according to different permissible deflection criteria (e.g., 0.6 inch, 1 inch, and 1.5 inch or 15, 25.4, and 38.1 mm) are presented in Fig. 5(a) and (b) for drilled shafts in clay and sand, respectively. From Fig. 5(a), one can see that the normalized ratio ranges from 1.2 to 1.8, for the cases with zero moment, and from 3.3 to 4.7 for the cases with combined lateral load and moment, respectively. From Fig. 5(b), it can be seen that the factor of safety ranges from 3.3 to 7 for 0.6 inch (15 mm) permissible deflection, from 2.7 to 4.5 for 1 inch (25.4 mm) permissible deflection, and from 2.3 to 3.4 for 1.5 inch (38.1 mm) permissible deflection. All of the normalized ratios are larger than 1, which seems to suggest that all three permissible deflections are acceptable from a geotechnical capacity viewpoint.

[b]Fig 5] Measured Over Predicted Capacities of Drilled Shafts at Various Permissible Deflections

The lateral wind loads applied to sound walls usually produce the accompanying moments. From Fig. 1, it appears that Broms method prediction is about ½ of the measured ultimate capacity for combined lateral load and moment. If a FS of two is applied to Broms method, the actual margin of safety is about 4. In considering both capacity and deflection, then a FS = 2 and allowable shaft head deflection of 1.0 inch (25.4 mm) seem to be appropriate for sound walls. It should be emphasized that this conclusion was derived from drilled shaft geotechnical response, not from structural consideration of sound walls.

Recommended Design Methodology

The design methodology for drilled shafts supporting sound walls is suggested as follows. First, Broms method and a FS of two are recommended to be used to determine the
required drilled shaft length with known shaft diameter. Then, COM624P computer program (or LPILE) shall be used to check whether the deflection at drilled shaft head under the design load exceeds the permissible deflection of 1.0 inch (25.4 mm) or a deflection value designated by a structural engineer. If the deflection is less than or equal to the permissible deflection, the drilled shaft length designed by Broms method is acceptable. Otherwise, if deflection criterion controls, then COM624P computer program (or LPILE) should be run to determine the shaft length such that the design load would not result in deflection which is more than the permissible deflection.

**Validation of Recommended Design Methodology**

In order to validate the recommended design methodology for drilled shafts supporting sound walls, two full-scale field lateral load tests on fully instrumented drilled shafts have been conducted in Colorado (Nusairat et al., 2004). The test drilled shafts with diameter of 30 inch (762 mm) and length of 16 feet (4.9 m) were originally designed for supporting sound walls in a clay deposit and a sand deposit. A comprehensive geotechnical investigation program was also carried out at the two new lateral load test sites that included pressuremeter test (PM), SPT, as well as laboratory triaxial consolidated undrained (CU) tests and direct shear tests on the soil samples taken from the lateral load test sites (See Nusairat et al. 2004 for complete details). Pressuremeter test results were employed to indirectly estimate the soil strength values using the FHWA (1989) equation for clay site. Liang (2002) correlations were used for SPT test data interpretation. Interpreted soil strength parameters for clay and sand sites are tabulated in Tables 3 and 4, respectively. Table 5 provides the calculated capacity and ratio of measured capacity over predicted capacity of test drilled shafts in clay and sand site using Broms method and various soil parameter determination methods. It can be seen that the soil parameters from laboratory tests (e.g. triaxial CU test or direct shear test), provide the most accurate capacities as compared with those measured for clay site. For sand site, soil parameters interpreted from Liang’s (2002) SPT correlation provides the best estimate on capacity of drilled shafts.

### Table 3: Undrained Shear Strength of Colorado Clay Interpreted from Various Soil Testing Methods

<table>
<thead>
<tr>
<th>Soil Layers (ft)</th>
<th>SPT</th>
<th>Lab Test</th>
<th>Pressuremeter</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>N values</td>
<td>$S_u$ (psi)</td>
<td>$S_u$ (psi)</td>
</tr>
<tr>
<td>0-2.5</td>
<td>12</td>
<td>11.3</td>
<td>18.3</td>
</tr>
<tr>
<td>2.5-4.5</td>
<td>12</td>
<td>11.3</td>
<td>15</td>
</tr>
<tr>
<td>4.5-6.5</td>
<td>15</td>
<td>14</td>
<td>14.4</td>
</tr>
<tr>
<td>6.5-10</td>
<td>9</td>
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<td>13.7</td>
</tr>
<tr>
<td>10-12.5</td>
<td>4</td>
<td>3.75</td>
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</tr>
<tr>
<td>12.5-16</td>
<td>8</td>
<td>7.53</td>
<td>11.7</td>
</tr>
</tbody>
</table>

$S_u =$ Undrained Shear Strength; 1 ft = 0.305 m; 1 psi = 6.9 kPa.

### Table 4: Cohesion and Friction Angle of Colorado Sand Interpreted from Various Soil Testing Methods

<table>
<thead>
<tr>
<th>Soil Layers (ft)</th>
<th>Pressuremeter</th>
<th>SPT</th>
<th>Direct Shear Test</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>c’ (psi)</td>
<td>$\varphi’$ (degree)</td>
<td>N Values</td>
</tr>
<tr>
<td>0-4</td>
<td>9.7</td>
<td>34</td>
<td>13</td>
</tr>
<tr>
<td>4-6</td>
<td>9.7</td>
<td>34</td>
<td>8</td>
</tr>
<tr>
<td>6-9</td>
<td>5.6</td>
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<td>9-15</td>
<td>11</td>
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<td>7</td>
</tr>
<tr>
<td>15-15.7</td>
<td>11</td>
<td>27</td>
<td>7</td>
</tr>
</tbody>
</table>

$c’ =$ Drained cohesion; $\varphi’ =$ Drained friction angle; $\varphi =$ Undrained friction angle; 1 psi = 6.9 kPa; 1 ft = 0.305 m.
Table 5 Calculated Lateral Capacity of Drilled Shafts for CDOT Test Sites

<table>
<thead>
<tr>
<th>Soil Parameters</th>
<th>Broms Method (kips)</th>
<th>Measured/Predicted</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay Site</td>
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<tr>
<td>SPT Liang</td>
<td>71</td>
<td>1.9</td>
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<tr>
<td>Lab Test</td>
<td>108</td>
<td>1.3</td>
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<tr>
<td>PM (FHWA)</td>
<td>98</td>
<td>1.4</td>
</tr>
<tr>
<td>Sand Site</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SPT Liang</td>
<td>91</td>
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</tr>
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<td>Direct Shear</td>
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</tr>
<tr>
<td>PM</td>
<td>84</td>
<td>1.14</td>
</tr>
</tbody>
</table>

(1kip = 4.448 kN)

Fig. 6 shows the comparison of measured and predicted load-deflection curves using soil parameters interpreted from laboratory test for clay site and SPT test for sand site. It can be seen that the predicted load-deflection curve matches the load test in clay site, especially in the initial portion of the curves. Although COM624P overpredicts the deflections for drilled shafts in sand, the prediction is still reasonable and is on the safe side.

![Load-deflection Curves at Head of Drilled Shafts Tested in Colorado](image)

Conclusions

Various existing methods for predicting ultimate capacity and deflection of drilled shafts supporting sound walls were evaluated. Based on this evaluation, a design methodology for the drilled shafts supporting sound walls is recommended. Broms method (1964a, 1964b) with a FS of two is suggested for ultimate capacity based design for drilled shafts in cohesive and cohesionless soils. The deflection at the drilled shaft head designed by Broms method should be checked to be less than a permissible deflection of 1 inch (25.4 mm) or a deflection value designated by a structural engineer, by using COM624P (or LPILE). If it exceeds permissible deflection, COM624P (or LPILE) should be used to determine the appropriate drilled shaft length.

Appropriate geotechnical test methods are recommended for obtaining relevant soil parameters for various design methods. For clay, the most appropriate soil testing method is lab test, e.g. triaxial unconfined undrained test, consolidated undrained test or direct shear test. For sand, SPT with Liang (2002) correlation provides the best soil strength interpretation. Pressuremeter test would provide reasonable soil strength interpretation as well.

The recommendations provided in this paper will result in more uniform, consistent, and cost-effective design and testing methods for the drilled shafts supporting sound walls. This uniformity ensures that fewer manhours are needed in deciding on analysis methods. Rather, engineers can focus more on the determination of high quality soil parameters for input into the analysis. This paper recommended lower FS than often used by the design engineers and geotechnical tests that will generate higher strength values than what is often assumed in the design. This will lead to significant savings in future sound wall projects. The proposed design/analysis approach for I-225 project in Colorado has been shown to reduce the required drilled shaft length by 25% compared to original Colorado DOT design approach. For a project that involves a large quantity of drilled shaft construction, or when a unique soil condition and complex loading combination exist, the lateral load test prior to final design decision could potentially offer cost saving to the project.

Acknowledgements

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support of Mr. Dennis Hanneman of Geocal Inc., Aurora, Colorado and Mr. Shuyu Liu of The University of Akron.

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Ground Movements – A Hidden Source of Loading on Deep Foundations

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Ground movements can arise from a large number of sources and can have a significant effect on nearby piles and deep foundations. The loading of the piles by ground movements is a different mechanism to that arising from direct applied loading to the pile head, and consequently it is not generally possible to adequately analyze the effects of ground movements simply by applying some type of equivalent loading to the pile head. The main effects of ground movements are the development of additional movements, axial forces and bending moments in the piles, and thus the key design aspects are related to movements and to the structural integrity of the pile. However, the ultimate geotechnical load carrying capacity is generally not affected by the ground movements themselves.

This paper will describe an approach to the analysis of ground movement effects on piles, considering axial and lateral movements separately. Some of the main features of pile response will be discussed for three specific problems involving ground movements:
1. Piles near and within embankments;
2. Piles near an excavation for a pile cap;
3. Piles subjected to seismic ground motions.

Introduction

There are many circumstances in which pile foundations may be subjected to loadings arising from vertical and/or lateral movements of the surrounding ground. Fig. 1 illustrates a number of these circumstances. In such cases, at least two important aspects of pile foundation design must be considered:
1. the movements of the piles caused by the ground movements
2. the additional forces and/or bending moments induced in the piles by the ground movements, and their effect on the structural integrity of the piles.

Problems involving the effects of ground movements on piles may be analyzed in at least two ways:
1. Via a complete single analysis (generally numerical) involving modelling of the pile, the soil and the source of the ground movements. This will give a complete solution for the behaviour of both the soil and the pile.
2. Via a simplified approach involving initial separation of the soil and the pile (“sub-structuring”) so that the soil movements are first computed and then imposed on the pile. In this approach, the focus is generally placed on the behavior of the pile.

This paper summarizes a consistent theoretical approach to the analysis of ground movement effects on piles, for both vertical and horizontal movements, which falls into the second category. Two distinct stages are involved in this analysis:
1. Estimation of the “free-field” soil movements which would occur if the pile was not present;
2. Calculation of the response of the pile to these computed ground movements.

Some specific cases of ground movement are then considered, and in each case, a discussion is given of the general features of pile behaviour revealed by the theory, and typical applications to practical field cases are described.
**Analysis of Pile Response to Externally Imposed Soil Movements**

**Axial response**

The analysis used for axial response of the pile has been described by Poulos and Davis (1980) and has been used to analyze problems of negative friction of piles in consolidating soil, and of tension and uplift of piles in expansive soil. It employs a simplified form of boundary-element analysis, in which the pile is modelled as an elastic column and the surrounding soil as an elastic continuum.

The pile is divided into a series of cylindrical elements. The vertical movement of each element is related to the applied load, the pile-soil interaction stresses, the pile compressibility, and the pile tip movement. The vertical movement of each supporting soil element depends on the pile-soil interaction stresses, the modulus or stiffness of the soil, and also on any free-field movements that may be imposed on the pile. To simulate real pile response more closely, allowance may be made for slip at the pile-soil interface, i.e., the pile-soil interaction stresses cannot exceed the limiting pile-soil skin friction.

The above analysis has been implemented via a FORTRAN computer program, PIES (Poulos 1989).

**Loading Via Ground Movements Versus Direct Applied Loading**

There is a widespread misconception that the effects of externally imposed ground movements on piles can be estimated by the application of equivalent loadings at the pile head. To illustrate the consequences of this procedure, the case in Fig. 2 has been analyzed. A single pile in a two-layer soil profile is considered, and the pile is subjected to the following sources of loading:

- An applied vertical load of 1.0 MN
- An applied lateral load of 0.1 MN at the pile head
- Vertical ground movement profile which decreases from 100 mm at the ground surface to zero at a depth of 12 m

The analysis has been implemented via FORTRAN computer programs, including a proprietary program called ERCAP, and in an alternative approach, via the program PALLAS (Hull, 1996).

**Group Effects**

The analysis of ground movement effects on groups of piles has been reported by several authors, for example, Kuwabara and Poulos (1989), Teh and Wong (1993), Chow et al (1990), Xu and Poulos (2001). All these authors have found that under purely elastic conditions, group effects tend to be beneficial to the pile response as compared to single isolated piles, i.e. the group effects tend to reduce the pile movement and the forces and moments induced in the piles. This is especially so for the inner piles within a group, which, because of the pile-soil-pile interaction are, in effect, “shielded” from the soil movements by the outer piles. Experimental work reported by Chen (1994) indicates that the ultimate lateral pile-soil pressures are affected to some extent by grouping and that the group effect may either increase or decrease the pile response, depending on the pile configuration and spacings.

From the viewpoint of design, it is generally both convenient and conservative to ignore group effects and analyze a pile as if it were isolated. Thus, in the remainder of the paper, attention is concentrated on ground movement effects on single isolated piles.
- A lateral ground movement profile which also decreases from 100 mm at the ground surface to zero at 12 m depth.

![Diagram](image)

[Fig. 2] Typical Problem Analyzed

Fig. 3 shows the computed axial load distributions for the applied load acting alone, the vertical ground movement acting alone, and the applied load and the ground movement acting together. It can be seen that the distributions of axial load in the pile due to applied loading are very different from those induced by the ground movements. In the latter case, the maximum axial load occurs near the bottom of the upper soil layer which is subjected to movement. It can also be seen that the addition of the two profiles of axial load gives axial loads which are less than those computed for the combined loading and ground movement case.

![Graph](image)

[Fig. 3] Comparison of Axial Responses

Fig. 4 shows the corresponding distributions of moment computed for the lateral response of the pile. Again, it can be seen that the distribution of induced bending moment is very different for applied loading and for lateral ground movement. In the latter case, the maximum moment occurs well below the pile head, near the bottom of the zone of ground movement. The maximum bending moment under the combined loadings is also at the latter location, since the moment due to the applied loading is virtually zero where the moment due to the lateral ground movements is largest.

![Graph](image)

[Fig. 4] Comparison of Lateral Responses

In terms of practical pile design, the above example demonstrates the following important points:

- The effects of ground movements can not be simulated accurately by the application of a load to the pile head;
- The superposition of axial load distributions due to axial applied loading and vertical ground movements may underestimate the maximum axial load in the pile;
- The maximum load in a pile subjected to lateral ground movements may occur well below the pile head. In this particular case, having the pile reinforced only to resist applied lateral loading (for example, in the upper 6 m or so) will be inadequate to resist the ground-movement induced moments. The pile may well fail structurally at a considerable depth below the pile head.

Thus, it is important to consider the possibility of ground movements in pile design, and to allow for reinforcement to resist deep-seated moments that may be induced by these movements.
Generic Design Charts For Piles Subjected To Soil Movements

Vertical Soil Movements
Design charts for the settlement and maximum axial force in a single end bearing pile on rock and subjected to vertical soil movements, have been published by Poulos and Davis (1980), while similar charts for a single floating or friction pile have been presented by (Poulos, 1989; Poulos and Davis, 1980). Such charts will tend to give upper bound values of both pile settlement and induced pile force, because there is no limit to the pile-soil shear stress that is developed between the pile and the soil. In reality, the existence of an ultimate skin friction will result in a limit to the axial force and pile movement that can be generated within a pile. The use of elastic solutions therefore tends to be conservative when applied to practical cases. Corrections for pile-soil slip and other practical effects are presented by Poulos and Davis (1980) and Nelson and Miller (1992).

Lateral Soil Movements
If the distribution with depth of free-field lateral movements can be simplified, it is possible to develop useful design charts to enable approximate assessment of the pile head deflection and the maximum bending moment in the pile. Chen and Poulos (1997, 1999) have presented such charts, both for a pile in soil subjected to a uniform movement with depth (to a depth zs below the surface), and for a soil in which the horizontal movement decreases linearly with depth, from a maximum at the surface to zero at a depth zs. These solutions assume that the soil remains elastic, and they therefore generally give an upper bound estimate of the pile moment and deflection. The extent of the possible over-estimation increases with increasing lateral soil movements, due to the progressive departure from elastic conditions which results from the development of plastic flow of the soil past the pile.

From a study of several examples, Chen and Poulos (2001) have suggested the following preliminary guidelines for the determination of soil movements in making theoretical predictions via the generic design charts:

1) For unstrutted excavations or relatively small slope movements, a linear soil movement profile, with a maximum value at the ground surface and zero at a certain depth below the surface, may be adopted. The maximum value may be estimated from measured ground surface movements or via appropriate empirical approximations which relate movement to the height of the retained soil, for example, Peck (1969).

2) For landslides involving relatively large soil movements (for example, up to about 0.4 pile diameters), a uniform soil movement profile may be adopted.

The above study by Chen and Poulos also shows that the elastic design charts can give reasonably good estimations of the lateral pile response, provided that the ground movements are not very large, for example, less than about 30-40% of the pile diameter.

Piles Near Embankments
Introduction
The construction of embankments on clay can result in the development of substantial immediate and time-dependent vertical and horizontal movements of the soil beneath and adjacent to the embankment. In situations where bridge abutments adjacent to such embankments are supported on piles, these piles may experience significant axial and lateral loads which are induced by the soil movements. The design of abutment piles therefore requires a proper consideration of the pile response to the externally imposed soil movements, including an assessment of the consequent bending moment and lateral deflection profiles of the piles.

Marche and Lacroix (1972), Heyman (1965), and Heyman and Boersma (1965) have presented field data of the lateral pile response to embankment-induced soil movements. Poulos and Davis (1980) have compared the results of a boundary element analysis with this field data, and have found fair agreement. Stewart et al (1991, 1992) have conducted centrifuge model tests of piles bridge abutments and have developed an empirical procedure for estimating the lateral deflection and bending moment in a pile or pile group. Poulos (1996) has compared various methods of analyzing lateral pile behaviour, including some design methods based on estimation of pressures developed between the pile and the soil. Such methods appear to be generally less reliable than methods based on a proper pile-soil interaction analysis.
Estimation of Ground Movements

Settlements beneath and near embankments can, in principle, be predicted by conventional methods of settlement analysis, whether they are based on one-dimensional analysis, elastic-based analysis, or numerical analyses, such as the finite element method or the program FLAC. An important requirement for such settlement predictions is to make allowances for non-linear soil behaviour, including differences in compressibility in the normally- and over-consolidated states, local yielding or failure within highly-stressed zones, and the effects of consolidation and creep.

In contrast, horizontal movements below and near embankments are difficult to predict accurately (e.g. Poulos, 1972) and it is often more appropriate to make use of empirical information on such movements. Bourges and Mieussens (1979) have evaluated the results of a number of field observations and recommended a practical design approach which is summarized here. They have found (see Fig. 5a) that the significant parameters of the horizontal displacement profile are:

- The ratio of layer depth (D) to mean embankment width (B);
- The ratio of distance (X) of a point from the crest, to the horizontal extent of the slope (Lw);
- The undrained shear strength su;
- The ratio of undrained shear strength of surface crust and underlying clay (M = su/su).

They identified three general distributions of horizontal displacement, which are illustrated in Fig. 5b. These distributions apply at or near the toe of an embankment, and can be expressed by the following equations:

- Curve 1 (Overall mean curve):
  \[ Y = 1.83Z^3 - 4.69Z^2 + 2.13Z + 0.73 \]  
- Curve 2 (where the compressible layer lies several metres below the surface):
  \[ Y = -2Z^3 + 1.5Z + 0.5 \]  
- Curve 3 (where the soil compressibility is reasonably uniform with depth):
  \[ Y = 3.42Z^3 - 6.37Z^2 + 2.14Z + 0.81 \]

The magnitude of the maximum horizontal displacement, \( \rho_{hmax} \), is given as the sum of the immediate and consolidation components, \( \rho_i \) and \( \rho_c \) respectively. \( \rho_i \) is related to the position of the point \( (X/L) \) and the safety level \( F \), where:

\[ F = 5.14s_{uav} / \gamma H \]

where \( s_{uav} \) = average undrained shear strength in the clay layer
\( \gamma \) = unit weight of embankment fill
\( H \) = embankment height.

From measured data, Fig. 6 plots the dimensionless maximum immediate horizontal displacement \( (\lambda, \text{ where } \lambda = \rho_{hmax} / D) \) against \( F \) for three values of \( X/L_w \). As would be expected, \( \lambda \) increases as \( F \) decreases, and becomes relatively large as failure is approached (i.e. \( F \) approaches 1.0).

The maximum consolidation lateral displacement at any time, \( \rho_{t max,t} \), is correlated with the consolidation settlement \( S_c \) of the centre of the embankment, as follows:

\[ \rho_{t max,t} = 0.16 S_c = 0.16 (S_t - S_i) \]

where \( S_t \) = total settlement at time \( t \)
\( S_i \) = immediate settlement.

Thus, at any time \( t \), the maximum horizontal displacement \( \rho_{hmax} \) can be approximated as:

\[ \rho_{hmax} = \lambda.D + 0.16(S_t - S_i) \]
Poulos (1994) has undertaken a study of the behaviour of piles near and through embankments on clay, using a two-stage procedure:

1. assessment of the horizontal soil movements due to the embankment, using the empirical approaches developed by Bourges and Mieussens (1979), described above;
2. analysis of the response of piles to these movement, using the program ERCAP to compute lateral response and PIES for axial response.

Comparisons were made between the analysis results and the results of centrifuge tests reported by Stewart et al (1992). Fig. 7 compares the relationship between maximum horizontal movement of a pile and embankment height. The centrifuge tests show that, beyond a certain embankment height (which corresponds to an average pressure of about 3s\textsubscript{uday}), the rate of pile displacement with increasing embankment height accelerates. This behaviour is reproduced quite well by the theoretical analyses. Fig. 8 compares the measured and computed distributions of bending moment along a pile, and shows fair agreement in both the magnitude and distribution of moment.

A study of the importance of a number of parameters was also made by Poulos (1994), and the factors which have the most significant influence on pile response were found to be: pile position – relative to the embankment toe, undrained shear strength of the clay, thickness of clay layer, embankment height, pile size, and delayed installation of the pile. The latter is particularly important, and it is found that, if the installation of the pile can be delayed until after embankment construction is completed, the maximum bending moments in the pile can be reduced to 10-15% of the values which would otherwise occur.

As mentioned previously, group effects are generally beneficial when piles are subjected to soil movements, leading to a “shielding” effect and reduced moments and shear forces in the piles, and it is therefore generally conservative to consider a single pile for design purposes.

**Some Characteristics of Pile Behaviour**

Comparison Between Measured and Theoretical Maximum Moments – Deep Clay Layer

Typical Design Charts

For the idealized problem shown in Fig. 9(a), a series of design charts has been developed for a range of values of embankment height, soil layer thickness, soil strength and pile size. For a typical case of a vertical pile located at the crest of the embankment, Fig. 9(b) to 9(d)
shows the maximum positive and negative moments and the pile head deflection for a 400 mm square precast concrete pile, for an embankment height of 8 m. It is assumed that the pile head is restrained from rotation and that the pile is installed after construction of the embankment has been completed. Such charts have the potential to provide a more rational design approach for piles within embankments. Successful use of such charts demands judicious selection of the necessary geotechnical parameters and appropriate modelling of the real problem to reduce it to the idealised cases for which the design charts are derived.

[Fig. 9] Design Charts for Pile at Embankment Crest – 4 m Embankment Height

Application to Case Study

De Beer and Wallays (1972) reported a field test in Belgium that aimed to study the influence of embankment construction on adjacent pile foundations. Measured results were presented for a steel pipe pile and a reinforced concrete pile. The steel pipe pile was 28 m in length, 0.9 m in diameter, and 1.5 cm in wall thickness, while the reinforced concrete pile was 23.2 m in length and 0.6 m in diameter. The pile heads were restrained from lateral displacement. The soil deposit consisted mainly of sand, with a Young’s modulus $E_s$ of about 30 MPa and the limiting soil pressure of about $2p_p$ (where $p_p$ is the Rankine passive pressure) (see Chen & Poulos, 1997). The measured free-field lateral soil movements, shown in Fig. 10(a), generally decrease with depth, after reaching a maximum at a relatively shallow elevation.

Chen & Poulos (1997) have shown that a full analysis via the computer program PALLAS can give estimations of pile bending moments and deflections very close to those measured, using the measured soil movement profile shown in Fig. 10(a). The pile bending moment and deflection profiles estimated using PALLAS are shown in Fig. 11, together with those measured, and a fairly good agreement between the estimated and the measured values can be observed.

Fig. 10 Test at Zelzate (De Beer & Wallays, 1972)

The soil movement profile has also been simplified to a linear profile for two cases, to assess whether simplified soil movement estimates can give adequate predictions of pile response. One case has a surface displacement ($s_0$) of 20 mm, while the other has a $s_0$ value of 40 mm, with both cases having a zero value occurring at a depth of about 18 m. It has been found that the measured profiles are encompassed by those estimated for the above two $s_0$ values.
Piles Near a Pile Cap Excavation

Introduction

When an excavation is carried out for a new pile cap in the vicinity of existing piles, there is often little or no support provided for the excavation, since pile cap thicknesses are typically 1-3 m and the excavation is therefore relatively shallow. However, under conditions in which the ground is highly stressed (for example, within the footprint of an existing building), even such modest excavations deserve careful consideration as ground movements will inevitably be generated by the excavation process. In addition, it is possible that dewatering may also be necessary, in which case additional ground movements (both vertical and lateral) will be generated by the process of groundwater lowering.

Ground Movements

It is now common for the ground movements around excavations to be estimated via detailed numerical analyses such as the finite element method. When numerical analyses cannot be carried out, it is possible to use approximations developed by Clough and his co-workers to estimate vertical and horizontal distributions of ground movements. The distributions of movement with depth are difficult to estimate without some form of analysis, as they depend on wall flexibility and excavation support conditions, but it may sometimes be adequate to assume a linear distribution with depth (Chen and Poulos, 2001).

Common design practice employs two-dimensional analyses, and near the centre of an excavation, two dimensional analyses can give reasonable soil movement estimates (for example, Yong et al, 1996). Thus, in the following examples, a two-dimensional analysis, employing the computer program FLAC, has been used to estimate the ground movements due to excavation for a pile cap. The case examined is shown in Fig. 12, and involves an excavation in medium-soft clay for a 3 m deep pile cap, 10 m in width, with no lateral support provided for the excavation. Figs. 13 a-d show typical distributions of the vertical and lateral movements with depth, at various distances from the excavation. Two different values of the surface pressure are considered, 0 kPa (a “green-field” situation) and 50 kPa, a typical situation that may arise beneath an existing building. It can be seen that, as would be expected, the movements for the 50 kPa surface pressure are considerably larger than those for zero pressure, and that the movements tend to decrease with increasing distance from the excavation. It is further assumed that the excavation is carried out relatively rapidly, and that no drop in the level of the water table arises from the excavation.

Pile Response to Ground Movements

For the case as shown in Fig. 12, Figs. 14 and 15 summarize the computed maximum bending moment at various locations.
moment and shear in an adjacent pile, as a function of the distance from the excavation and the surface pressure. It will be seen that the induced maximum bending moment is very large when the pile is close to the excavation. Indeed, for a 0.6 m diameter reinforced concrete pile with 1% reinforcement, carrying a working axial load of 800 kN (corresponding to a factor of safety of about 2), the maximum design moment capacity is about 0.56 kNm. Thus, Fig. 14 implies that piles within about 10 m of the axis of the excavation could have induced moments that exceed the design capacity of the pile, if the surface pressure is 50 kPa.

Fig. 13(a) Computed Vertical Ground Movements Due to Cap Excavation – $p_a = 0$

Fig. 13(b) Computed Vertical Ground Movements Due to Cap Excavation – $p_a = 50$ kPa

Fig. 14 Computed Pile Moment Due to Cap Excavation

Fig. 15 Computed Pile Shear Due to Cap Excavation
Fig. 16 summarizes the computed additional movement of an existing pile adjacent to the excavation. In this case, if there is zero surface pressure, the adjacent pile tends to move upwards slightly because of the excavation, but it settles if the surface pressure is 50 kPa. In the latter case, the additional axial force induced in the pile by the vertical ground settlement is small, even if the pile is relatively close to the excavation.

Thus, it would appear that the issue that may cause most concern is the induced bending moment and shear in the pile due to the lateral ground movements.

Comparisons Between Computed and Measured Pile Response in Centrifuge Tests

There do not appear to be any measurements of pile response to cap excavation-induced ground movements. However, Leung et al (2000) have presented results from centrifuge model tests on a single pile adjacent to unstrutted deep excavations in dense sand. The model pile was fabricated from a hollow square aluminum tube and instrumented with 10 pairs of strain gauges protected by a thin layer of epoxy. The model pile simulated a prototype concrete bored pile of 0.63 m in diameter, and 12.5 m in total embedded length. The retaining wall supporting the excavation had an embedment depth of 8 m. The Young’s modulus of the sand, $E_s$, was estimated to increase linearly with depth, $z$ (in metres), and expressed approximately as $E_s = N_h z = 6z$ MPa.

Several tests were carried out in which the pile was located at different distances from the retaining wall. The free-field soil movements, pile bending moments and deflections were measured for different depths of excavation. The measured free-field soil movements at different distances from the wall and corresponding to an excavation depth of 4.5 m were found to decrease almost linearly with depth, from a maximum value at the surface.

The generic elastic design charts (Chen and Poulos, 1997) were used to calculate the pile response, with the soil movements simplified to linear profiles. The estimated results are shown in Fig. 17, together with those measured. It can be seen that, in this case, the elastic generic design charts using the simplified soil movements generally give fairly good estimates of the pile response.
of statically-induced ground movements discussed above, but there are additional complexities that must be recognized. In particular, there are two sources of loading of the pile by the ground movements:

1. “Inertial” loading at the pile head, caused by the lateral forces imposed on the structure by the earthquake and which are then imposed on the piles;
2. “Kinematic” loading along the length of the pile, caused by the lateral (and to a lesser extent, the vertical) ground movements developed by the earthquake.

Traditionally, many foundation designers have considered only the effects of inertial loading, but kinematic loading can also be very important.

A relatively simple pseudo-static approach for analyzing this problem, taking into account both inertial and kinematic effects, has been developed by Tabesh and Poulos (2001) and is described below.

**Pseudo-Static Approach**

The pseudo-static methodology developed by Tabesh and Poulos (2001) involves the following steps:

1. The superstructure is modelled by a single degree of freedom system whose natural frequency is equal to the fundamental frequency of the superstructure. The simplest way is to reduce the superstructure to a cap-mass. The designer should be careful about such a crude approximation; the mere eccentricity of the superstructure mass may have a profound effect on the response.

2. The natural lateral period $T$ of the pile head is estimated through published formulae and charts, or in the case of a pile-cap-mass system, via the approximate relationship:

   $$ T = 2\pi(\text{Cap Mass}/K_x)^{0.5} $$

   where $K_x$ = lateral pile head stiffness.

3. A free-field site response analysis is performed to obtain both the time history of the motion at the surface and the maximum displacement of the soil mass along the pile. In this analysis the well-known SHAKE (Schnabel et al (1972)) program, or similar computer codes such as the ERLS program used herein, may be used. As the moment and shear depend on the curvature of the pile, the points whose maximum displacements are to be obtained must be closely spaced, especially near the surface.

4. The maximum values of the displacements along the pile obtained in step 3 are treated as a static soil movement profile, although the displacement at each point may have occurred at different times.

5. The surface motion obtained in step 3 is used in an ordinary spectral analysis of a single degree of freedom system whose period is equal to the period obtained in step 2. The spectral acceleration is calculated.

6. The lateral force to be applied to the pile is obtained from multiplication of the spectral acceleration obtained in step 5 and the cap-mass or the mass of the single degree of freedom model of the superstructure calculated in step one.

7. A static analysis, in which the pile is subjected to the simultaneous application of a lateral force at its head equal to the force obtained in step 6, and a soil movement profile formed in step 4, is performed and the maximum pile moment and shear are obtained.

**Verification of The Pseudo-Static Method**

In order to examine the performance of the proposed pseudo-static methodology, Tabesh and Poulos (2001) considered a soil mass consisting of two layers. Various ratios of layer stiffness were considered, and a range of pile diameters was analysed. The Newcastle 1994 earthquake was used as the excitation source. Eighty different pile-soil configurations were considered for which the envelopes of the positive and negative moment and shear along the pile were obtained via a more complete dynamic analysis, and the shear and moment distributions along the pile were also calculated from the proposed pseudo-static analysis with the maximum computed free-field soil movements as input. The cap-mass (and hence the vertical applied load) was assumed to be zero. Without any exception, excellent agreement was obtained between the dynamic analysis and the pseudo-static methodology. These comparisons suggest that, regardless of the soil non-homogeneity, the static methodology gives very good results for the maximum values of the moment and shear along the pile. Thus, when the response of the pile is dictated by the free-field ground movements, the internal response of the pile can be easily estimated by a very simple static analysis. When the effects of cap-mass were
taken into account, it was observed that, while in many cases the agreement between the proposed pseudo-static method and dynamic analysis was close, while in some others the pseudo-static approach overestimated the maximum moment and shear.

Significance of Inertial and Kinematic Effects

The influence of inertial effects (via vertical loading and/or cap-mass) on the seismic response of pile foundations depends on the frequency content of the earthquake and the natural period of the pile-soil-cap-mass system. Mylonakis et al (1997) have identified the following characteristics:

1. Inertial bending can be significant, especially in the upper part of the piles, when the dominant period of the earthquake is similar to the fundamental period of the soil-pile-structure system.
2. Kinematic bending can be significant when the dominant period of the soil motions are similar to the natural period of the soil strata.
3. The three most likely areas of damage of a pile are the pile head, interfaces between layers of different stiffness, and the pile toe. Pile head damage is most likely in homogeneous strata while damage at strata interfaces is most likely when there is a marked stiffness contrast between the layers. The kinematic bending strains at the pile toe may be significant when the toe is restrained.

To facilitate an understanding of the relative importance of inertial and kinematic effects, analyses have been performed on the fixed head single pile shown in Fig. 18. The analysis has been carried out via the pseudo-static approach described above, so that the results provide an envelope of maximum bending moments and shears along the pile. It is assumed that the site is subjected to the 1994 Northridge earthquake with a maximum bedrock acceleration of 0.2 g. Three cases have been considered:

- A pile with no vertical load/cap mass;
- A pile with a lateral inertial load of 0.2 MN
- A pile with the same lateral inertial load as in the second case, but where the kinematic ground movements are not included in the analysis.

Fig. 19 shows the computed distributions of bending moment along the pile. Two key points emerge from this figure:

1. If kinematic effects are ignored, and only inertial (lateral load) effects are considered, the maximum moment at the pile head can be seriously under-estimated.
2. If only inertial effects are considered, the moment at depths in excess of about 7m becomes insignificant, but with the kinematic effects incorporated, there is a significant moment between depths of about 7 to 10m, i.e. in the vicinity of the interface between the softer upper layer and the stronger lower layer.

The importance of considering both kinematic as well as inertial effects is clearly emphasized in this example.

![Graph showing bending moment distribution](image)

[Fig. 18] Example Analyzed

![Graph showing effects of kinematic and inertial loading](image)

[Fig. 19] Effects of Kinematic and Inertial Loading of Pile on Moment Distribution
Design Charts

Tabesh and Poulos (2006) have attempted to provide a simple means of making preliminary estimates of maximum bending moment and shear in single piles embedded in homogeneous clay layers and subjected to seismic excitation (Fig. 20). To develop these design charts, a time domain method has been employed in which the earthquake motion has been input, and the moment, shear and relative displacement of the pile obtained at all time steps during earthquake. The maximum values at any time during the earthquake have then been extracted and used for the design charts.

![Fig. 20] Idealized Case for Design Charts

Eight earthquakes were selected to develop the design charts, covering a wide range of prominent earthquake frequencies. The piles were assumed to have a fixed head (i.e. zero head rotation), and to be embedded in homogeneous clay layers with Young’s modulus values of 25, 50 and 100 MPa. The pile diameters covered the range of values between 0.2 to 1.5 metres in 0.1 m increments. Two pile modulus values, 10000 MPa and 30000 MPa were considered, being representative of timber and concrete respectively.

Fig. 21 shows typical design charts for the maximum moment and shear developed in a 20 m long pile with an axial load equivalent to a factor of safety of 2. As would be expected, both the moment and shear increase with increasing pile diameter d.

Case Study

The pseudo-static methodology is used to estimate the maximum moment developed in the Ohba-Ohashi bridge in Japan. This bridge is located in Fujisawa city, Kanagawa prefecture, near Tokyo. The seismic observations at this bridge and one of its pile foundations were conducted between 1981 and 1985 by Shimizu Corporation. During this period, 14 earthquakes hit the region. Among them, the twelfth earthquake induced the largest peak horizontal surface acceleration which was 0.11 g.

The bridge was supported by 11 piers and is 484.8 m long and 10.75 m wide. The girder is continuous from pier 5 to pier 8. Piers 5, 7, and 8 are equipped with movable bearings, and pier 6 is of the fixed-shoe type. Fig. 22 shows the details of the bridge and the pile foundation of pier 6 where the strain metres were installed. The soil profile shown in Fig. 22 was obtained from a borehole near pier 6. The top soil layers were extremely soft with SPT-N values being almost zero. The shear wave velocity of the top layers was between 40-100 m/s. The tips of the piles were embedded in a stiff layer of clay with a shear-wave velocity of 400 m/s. The length of the vertical piles was 22 m, with 2 m in the stiff clay.

The piles were steel pipes with a diameter of 0.6 m; the thickness of the vertical piles was 9 mm, and the wall thickness of the battered piles was 12 mm. A total of 32 wall strain metres were installed on one vertical and one battered piles at four different elevations (sa1-sa4, sb1-sb4 in Fig. 22). A total of 11 units of accelerometers of
servo type were installed, with one unit (GS1) at the ground surface, four units (GB1-GB4) at the bearing substratum, three units (BS1-BS3) at the footings, two units (BR1 and BR3) at the piers and one unit (BR2) at the girder.

For this analysis, the eighth earthquake was considered and the pseudo-static methodology was used to estimate the maximum moment developed in the vertical instrumented pile. Group effects were ignored. In the Ohba-Ohashi bridge measurements the free-field displacements along the length of the pile were not measured and only the base and surface motions were monitored. A free-field analysis was therefore required for which ERLS program was employed and the motion monitored at GB1 was used as the input motion. The soil was modelled as a system of 7 horizontal layers, and the mass of the superstructure was concentrated in two points, as shown in Fig. 23. The maximum value of the free-field response was obtained at 48 points corresponding to the centre of 48 pile elements. The spectral acceleration corresponding to the pile natural period and based on the surface motion was calculated to be 0.092 m/s². All piles were assumed to carry equal loads. The pier was very stiff and was considered to be rigid. The eccentricity was calculated to be 16.3 m.

With these assumptions the pseudo-static approach was used to obtain the response of the pile to the combination of the following disturbances:

- a soil movement profile formed from the maximum free-field soil movement obtained at each pile element,
- a head force equal to $F = m_s A_{spec}$ in which $m_s$ = mass of superstructure and $A_{spec}$ is the spectral acceleration obtained for a period equal to that of the pile and based on the surface motion,
- a moment equal to $M = F \cdot e$ in which $e$ is the eccentricity.

The profile of the moment along the pile obtained from the pseudo-static method, along with the maximum moments measured at 4 locations along the pile are shown in Fig. 24.
In this figure the envelope of the positive and negative moment along the pile obtained from an independent dynamic analysis (SEPAP) is also shown (Tabesh and Poulos, 2001).

The proposed static methodology gives good estimates of the measured values, despite the fact that the Ohba-Ohashi bridge was very complicated and was over-simplified for the analysis. Similar results were obtained for several other measured earthquakes (Tabesh, 1997).

It is believed that the pseudo-static analysis can be used by practicing engineers to obtain a reasonable estimate of the maximum moment and shear which is likely to be developed in a pile when it is subjected to earthquake excitation. This analysis is likely to give good results in many practical cases, but it may overestimate the maximum moment and shear in certain other cases, especially when the period of the pile is close to the natural period of the soil mass in which case significant interaction may occur between the pile and soil. One reason is that the maximum free-field effects and maximum inertial effects have been assumed to act simultaneously, which does not occur in a dynamic analysis. More importantly, the assumption behind using spectral acceleration is that the cap-mass is excited by the surface motion. In reality, the cap-mass is excited not through the soil surface, but via the pile head whose motion is different from the surface motion. The pile head and surface motions are very close for a homogeneous soil mass, but when the soil is strongly layered, the pile head motion is often less severe.

Conclusions

There are several circumstances in which ground movements may influence significantly the behaviour of piles. This aspect of pile behaviour has often been overlooked or not recognized, often leading to excessive foundation deformations and possible structural damage of the foundation system. This paper has described a consistent procedure for analyzing the response of piles to ground movements, via simplified boundary element analyses. For such analyses, it is necessary to be able to predict the distribution of the ‘free-field’ ground movements, and then to use these in the analyses together with the pile-soil parameters which are required for the normal analysis of piles subjected to applied head loadings.

Three specific examples of piles subjected to ground movements have been considered:

1. piles near an embankment.
2. piles near a pile cap excavation.
3. piles in ground subjected to seismic action.

Some of the features of behaviour have been discussed, and some typical design charts have been presented to assist in the practical estimation of the forces, moments and displacements induced in the piles by ground movements.

Examples of comparisons between theoretical and measured behaviour are described, and these generally show a reasonable measure of agreement.

Attention has been concentrated on single piles, but it has been found that, in general, the consideration of a single pile is conservative from a design viewpoint, as interaction among piles in a group subjected to ground movements usually has a beneficial effect and reduces the induced deflections, forces and moments as compared with a single isolated pile. In addition, in contrast to the case of piles subjected to direct head loading, elastic analyses tend to give conservative estimates of pile responses to ground movements because there is no limit to the pile-soil pressures developed by the moving soil.

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Rapid Lateral Load Testing of Deep Foundations

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This paper describes a method of conducting rapid lateral loading tests of deep foundations in which the load is applied to the foundation with a time duration of less than one second. For purposes of this paper the loading is applied using a pyrotechnic loading system commonly referred to as Statnamic. However, the method should apply to any lateral loading system which produces a force time history with a loading duration of 20 msec to 1 second. The test provides a measure of the dynamic and static response of the soil/foundation system and must be evaluated as a dynamic loading. Interpretation of the test includes a simple single degree of freedom (SDOF) model which can account for the inertial effects of foundation mass, system damping, and static foundation stiffness. The results of the test measurements are compared with four case histories in which comparative static load test data are available, including pile group foundations and large diameter drilled shaft foundations.

KEYWORDS: load testing, pile foundation, drilled shaft foundation, lateral loading, dynamic loads, pile groups

Many civil engineering works include deep foundations designed for lateral loading. Highway bridges, transmission towers, offshore structures, and buildings are subject to lateral loads from wind, waves, vessel impact, and seismic loadings. Although designers most frequently design for such loadings using static analysis techniques, the actual loadings are most often transient and dynamic, and characterized by rapidly applied loads of short duration. Where site-specific load testing has been performed in support of design for lateral loading, tests have most often been conducted as static jacking tests using hydraulic cylinders and sustained loads.

This paper describes a relatively new method of conducting field lateral loading tests on deep foundations in which the load is applied rapidly, i.e. with a load pulse of less than 1 second. For this paper, the device used to apply this loading uses pyrotechnic combustion to generate gas pressure and is commonly referred to as a Statnamic device. However, the methods used to evaluate the test measurements and interpret the results apply to any rapid test method which is so fast that inertial and damping forces are important, but slow enough that the foundation system moves in a displacement pattern which is similar to a static loading rather than as a bending wave propagation mode. An example of the latter has been described by Briaud and Ballouz (1996) in which a transient impact loading was used to measure the dynamic lateral stiffness of a pile. The Statnamic device produces a much longer wave pulse and has been used to conduct axial loading tests (Janes et al, 1991). The differences in axial loading pulse between the rapid loading produced by the Statnamic and the stresswave produced by a hammer impact have been well documented. The use of the Statnamic device for lateral loading represents an adaptation of the loading device for this purpose.

The use of a rapid loading device for lateral load testing offers several potential advantages and limitations. The test can be conducted quickly and efficiently to loads of large magnitude and without the need for a reaction system. Some tests conducted by the author have achieved lateral load magnitudes of 10 MN. Safety is actually enhanced, because there is no hydraulic system loading between two objects with pent-up strain energy and the potential for breakage and quick release due to failure of some component. The dynamic nature of the test provides the additional benefit of measuring the dynamic component of soil resistance, so long as the static component of resistance can be determined reliably. The dynamic nature of the test also represents a limitation, in that rate effects can vary with soil type. Soil creep or displacement due to sustained loading cannot be measured with a rapid test. The dynamic nature of the test requires that inertial components of the system be accounted for and thus the analysis is less straightforward than for a conventional static test. Finally, the measurements must be made using a high speed data acquisition system.

Some early test results have been described by El Naggar (1998), and that paper included his use of a relatively sophisticated Winkler model of a pile on nonlinear spring/dashpot system which operates in the time domain. While the
use of such a model provides the means for a very complete description of the problem, the inherent complexity of a dynamic model with many variables makes it difficult to use for routine testing and data interpretation. The techniques used in this paper emphasize analysis using simple dynamic models which can be readily applied and understood and easily compared with static test results. In general, engineers are familiar with static test data and have developed judgement based on such measurements.

The objectives of this paper are to describe the loading system and test measurements and to outline a simple and reliable method for interpretation of the results. Several case histories which include both rapid loading and conventional static loading tests are presented to provide a means of evaluating the interpretation method and of comparing and contrasting the rapid loading system with conventional static tests.

**Description of Test Method and Measurements**

The Statnamic device, illustrated on Fig. 1, is composed of a reaction mass, a piston in which gas pressure builds to initiate the load, and a connection to the test foundation which includes a load cell and hemispherical bearing to accommodate rotation of the foundation. For lateral testing, the reaction mass resides on a sled which allows the mass to slide across the ground surface or the surface of a barge in the case of over-water tests. The horizontal thrust against the test foundation is produced as the gas pressure builds and accelerates the reaction mass away from the foundation. After the reaction mass has moved some distance off the cylinder, an exhaust port opens and the gas pressure is vented. Typical load pulse duration is around 100 msec, although this can be varied somewhat depending upon the magnitude of the reaction mass and exhaust port location. The magnitude of the load is controlled by the amount of fuel placed within the combustion chamber.

The loading test is conducted typically with four progressively increasing load intervals, each of which is a separate rapid loading pulse. Roughly one hour elapsed time is required between load intervals for the device to be reloaded, re-assembled and repositioned against the test foundation. The use of four successive and increasing load pulses has been observed to provide the best means of reproducing the nonlinear load vs displacement relationship with which engineers are familiar from static testing.

Measurements at the top of the foundation include load, displacement, and acceleration. The calibrated load cell provides an accurate and reliable measure of the force applied to the foundation as a function of time. Accelerometers are typically placed at several locations on the test foundation and provide a measure of the acceleration time history and, by twice integration, a displacement time history. Capacitor-type accelerometers are used rather than piezo-crystal type because of their greater stability and reduced tendency for drift over the several seconds of time for which data are gathered. Displacement transducers consisting of long-travel LVDT= s or linear potentiometers are typically mounted on a reference beam to provide a second and redundant measure of movement. It is often difficult to avoid ground-induced vibration of the reference system, although such transient motions are often very small relative to the foundation motion. However, the use of displacement transducers provide the most reliable measure of permanent displacement and the use of these transducers together with the integrated accelerometer measurements provides needed redundancy. An additional accelerometer mounted upon the reference beam is typically used to monitor any vibrations in the reference system.

Measurements are also made below the top of the foundation in order to determine the displaced shape of the pile or shaft and the location of maximum bending stresses. The determination of the displaced shape is made using recoverable, downhole accelerometers, an example of which is shown on Fig. 2.
These are mounted on a guide which is lowered into place at the prescribed elevation within an inclinometer casing. An adjustable mount allows the device to be oriented in the proper direction to align with the load and displacement direction of the test foundation. Double integration of the downhole accelerometer signals allow determination of the displacement time history at each instrument location. For most of the tests performed to date, a string of 8 downhole accelerometers (along with the above ground displacement measurements) have proven adequate to define the displaced shape of the test foundation and the point of plastic hinge formation beneath the surface. Strain gauges are typically used to monitor bending stresses within the pile or shaft. Note that resistance-type strain sensors rather than vibrating wire instruments are required in order to obtain data at the frequency needed for dynamic testing.

All of the instrumentation must be monitored using a high speed data acquisition system. A sampling frequency of 1000 samples per second has proven sufficient for the rapid lateral testing of drilled shafts used in this study, although higher frequency sampling may be needed for small piles. Most commonly the system is set to trigger from the load cell and record data from a pre-trigger time of 0.5 seconds to about 4 seconds post-trigger. Most deep foundation systems have a resonant frequency well above 3 Hz, and the data of interest occur generally within the first second after trigger. Some large group foundations over water have produced data of interest for several seconds after trigger.

**Test Foundation Response and Data Interpretation**

**Example Measurements**

Although several case histories will be presented later in this paper, it is instructive to utilize some actual data in the process of describing the measured foundation response and proposed method of analysis. Presented on Fig. 3 are the four load time histories for a load test recently conducted in Charleston, SC (Brown and Camp, 2002). This shaft was a 2.6 m diameter by 46 m deep cast-in-place concrete drilled shaft with a permanent steel liner some 25 mm thick in the upper 17 m of the shaft. The soil conditions consisted of soft organic clay within the upper 15 m underlain by a very stiff calcareous clay known locally as the Cooper Marl Formation.

![Load and Displacement Time Histories for Charleston Test Shaft C-2](image)

Also shown on Fig. 3 are the displacement time histories from these four loadings, measured at the point of loading approximately 1 m above the ground surface. This foundation had large damping, as the oscillations damped out very quickly after the initial peak.

Shown on Fig. 4 are the peak displacements from the downhole accelerometer measurements, plotted as a function of depth below the ground surface, for load events two through four. These measurements indicate very reliably the point of rotation near the top of the Cooper Marl at a depth of around 16 m below grade.

**Derived Static and Dynamic Load-Displacement Response**

Using a simple single degree of freedom system, an equivalent static and damping response may be derived from the rapid loading lateral test
measurements. This model includes a nonlinear static spring resistance, inertia of the shaft rotating about a hinge point below ground, and a viscous damping component. The forces acting on the foundation may be described as follows:

\[ F_{\text{meas}} = F_{\text{inertia}} + F_{\text{damping}} + F_{\text{static}} \]  

where,

- \( F_{\text{meas}} \) = measured force on the load cell
- \( F_{\text{inertia}} \) = inertial resistance from effective mass of the foundation
- \( F_{\text{damping}} \) = effective viscous damping resistance
- \( F_{\text{static}} \) = effective static soil resistance

The inertial resistance is roughly that of a cylinder rotating about its base, with a diameter equal to that of the test shaft and a height taken as approximately 18 m, based on the observed displacement pattern (16 m below grade plus approximately 2 m above). For such a cylinder of radius \( r \), height \( h \), and mass \( m \), the mass moment of inertia about the base, \( I_y \) is:

\[ I_y = m \left( \frac{r^2}{4} + \frac{h^2}{3} \right) \]  

The rotational acceleration of such a cylinder in relation to a displacement \( x \) at the loading point \( z \) would be \( \ddot{x}/z \) and thus summing moments about the base,

\[ (F_{\text{inertia}})z = (I_y)(\ddot{x}/z) \]  

Therefore, \( F_{\text{inertia}} = (I_y)(\ddot{x}/z^2) = m_\text{e} \ddot{x} \) where \( m_\text{e} \) may be thought of as the effective mass of the foundation. For this test, \( m_\text{e} \) would be calculated to be around 85,000 kg. It is normally necessary to increase this value somewhat for analysis purposes in order to include some mass from the passive earth pressure wedge of surrounding soil (which is also suggested by the data) and this value is increased by 20% for this example.

The damping resistance is presumed to be represented by a viscous damper in which the force \( F_{\text{damping}} \) is proportional to the velocity, \( \ddot{x} \), by a constant, \( c \) (which is in units of force-sec./length). In order to relate this more meaningfully to a system damping parameter, the damping constant is expressed as a percent of the critical damping, \( c_c \), by

\[ D = c/c_c = c/[2(km_e)^2] \]  

where,

- \( k = \text{static stiffness} \)

thus,

\[ F_{\text{damping}} = c = D \times [2(km_e)^2] \]

The static resistance is modeled as a function of displacement, \( x \), using a spring with stiffness \( k_s \). Because the soil response for lateral loading at large strains is known to be highly nonlinear, this spring may be modeled as a nonlinear stiffness which decreases as a function of displacement. For routine analyses, it is normally sufficient that the stiffness has been taken as a constant which is derived independently for each static loading (and is smaller with each successive increased load). It is also possible to assume a stiffness which decreases according to some prescribed mathematical way as a function of displacement.

The model is backfitted to the results of the four test measurements to obtain the nonlinear spring and viscous damping parameters which best match the observed behavior, using the measured load vs time as input. This procedure is thus a signal matching process, and the solution is not unique. However, the relatively few parameters constrain the model very well. The static stiffness primarily controls the initial peak displacement with little influence from other parameters. The static stiffness and mass control the frequency of oscillation, and the mass is constrained by the physical attributes of the problem. The damping controls the decay of the peak displacements after the initial peak.

For the Charleston test shaft example, the match of the backfitted model and the measurements is illustrated on Fig. 5. The four test loads are matched using an effective mass of 102,000 kg for each case, a stiffness which is constant for each load case but which decreases from 80 MN/m for the first load to 35 MN/m for the last, and a viscous damping component which is 52% of critical damping for each case. This is a relatively high damping ratio.
compared to many similar tests, but reflects the large damping which was observed for the test conditions at this site.

The measured, inertial, damping, and static forces are plotted as a function of time for load 4 of this example on Fig. 6. This plot illustrates the development of soil resistance to lateral loading forces during the test. As the rapid loading is applied, this energy is initially used to mobilize the inertia of the shaft. The viscous damping is mobilized as the velocity of the shaft approaches a maximum, then the static soil resistance is mobilized as the displacement reaches a large value. At the maximum displacement, the velocity of the shaft goes to zero, the applied force is already over and the static soil resistance is mobilized to a maximum in order to stop the inertia of the shaft.

As a result of the construct of a model for the test result, the static stiffness can be used to produce a derived static load vs. displacement response as illustrated on Fig. 7. The points shown on that plot indicate the total static soil resistance (static spring force) which is mobilized at the maximum displacement during each of the four loading events. Because the damping is zero at this point, these points are not sensitive to the damping. The points plotted on this figure which are labeled “Total (Static + Damping)” represent the maximum sum of the static + damping forces plotted at the displacement for which this maximum occurs. These forces represent the maximum soil resistance force which was mobilized during the loading event, including the contribution which is attributed to viscous damping. The static resistance is comparable to a static loading test of short duration for which inertial and damping forces are not significant, and for which long term creep from sustained loading is not a major component. The damping contribution represents the effect of the high rate of loading of this soil during the rapid load testing event, which mobilizes a substantial amount of rate-dependent soil resistance in this case. Note also that this dynamic soil resistance is mobilized at a large displacement and a frequency of around 2 to 3 Hz, the damped resonant frequency of the test shaft. This frequency is thought to be reasonably close to that of a seismic load event on a large bridge. Interpretations of dynamic response at other similar but slightly differing rates of displacement or frequency might be inferred from these measurements. Some additional degradation due to gapping and/or reductions in soil shearing strength might be anticipated for many cycles of loading.

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**Fig. 5** Signal Match from SDOF Model for Charleston Test Shaft C-2

**Fig. 6** Forces as a Function of Time from SDOF Model, Load No. 4

**Fig. 7** Derived Soil Resistance vs. Displacement
Case Histories

It is instructive to examine those case histories for which the rapid lateral load testing method can be compared with conventional static tests. Several such examples are now available and are reviewed below.

Charleston Test Shafts

The Charleston, SC test shaft C-2 referenced above was part of a major load testing project for a proposed new bridge over the Cooper River. Another identical test shaft, C-1, was located only about 10 m away in nearly identical soil conditions and was also subject to lateral loading. Shaft C-1 was first subject to lateral loading using a hydraulic system so as to produce a slow cyclic lateral loading with a period of around 2 minutes/cycle. After the completion of the cyclic loading, this shaft was subsequently subjected to a rapid loading test in the opposite direction using the Statnamic device.

The results of the slow cyclic and the derived response from the subsequent rapid loading on test shaft C-1 are shown on Fig. 8, along with the rapid loading test results from the identical test shaft C-2 discussed previously. The solid line on this figure representing the cyclic test is seen to match very closely with the derived static response from the two rapid loading tests. The cyclic loading was performed using 10 cycles of constant amplitude force at loads up to about 2.7 MN, followed by a monotonic loading to around 4.5 MN (at which point the pump failed and the test was rapidly concluded!). The derived static response matches quite well with the first cycle loadings (although cyclic degradation was relatively small) and for this monotonic loading to the maximum applied static force. Note that these soft clay soils might be expected to creep under sustained load, but the static (cyclic) test in this case was of short duration. So, the derived static response in this case matches well with a static test of short duration for which long term creep is not a factor.

Rock Socket Shafts at Auburn NGES

A series of lateral static loading tests were performed on short drilled shafts at the Auburn University National Geotechnical Experimentation Site (NGES) in Alabama (Kahle, 2000). Tests on several similar shafts were performed nearby using the rapid loading method with the Statnamic device. The geotechnical conditions at this site are composed of weathered metamorphic rocks, characterized as hard, fractured quartzite. The overburden soils were stripped away so as to leave the fractured rock formation present immediately below the ground surface. Several shafts of 0.9 m and 1.5 m diameter were constructed into this fractured rock to embedded lengths of up to 2 m.

Conventional geotechnical borings indicate the rock to be hard, but intensely fractured. Coring RQD values were near zero with % recovery generally around 20% or less. Standard penetration test resistance values for the rock typically indicate refusal. Large intact samples of rock taken from the shaft drilling excavation were cored in the lab so as to provide specimens for unconfined compression tests (no suitable samples were obtained from core borings). Unconfined compressive strengths from seven samples ranged from 75 to 185 MPa, with an average value of 130 MPa (19 ksi). This very high strength is virtually irrelevant, as the strength during load testing was dominated by the macrostructure of the formation. The weathered rock at this site represents the type of material for which site-specific field loading tests are most appropriate, as geotechnical characterization of the weathered rock for foundation design purposes is extremely difficult.

Presented on Fig. 9 are data from two shafts which were pushed apart using a hydraulic loading system (shafts E5 and D5), along with the derived static and dynamic soil resistance from a rapid loading test on a shaft of similar size (B5). These shafts were each 0.9 m diameter and embedded approximately 2 m deep. Presented on Fig. 10 are similar data.
from shafts which were each 1.5 m diameter and embedded approximately 1.5 m deep. The static loading tests were performed by applying load in increments and holding at constant displacement for periods of around 10 minutes at each increment. The fractured rock did not exhibit a large amount of creep, but did exhibit a strong nonlinear response as a passive earth pressure failure was achieved in each test. Subsurface displacements from integrated accelerometer measurements during the rapid loading tests are provided in Fig. 11, which reveal that this shaft rotated as a nearly rigid-body rotation about a point just above the shaft toe. Inclinometer data from the static tests were similar, indicating a point of rotation about 0.3 to 0.5 m above the toe.

The comparisons between the rapid loading and static loading tests for this site suggest that the derived static response agrees quite well with the measured static response from tests on similar shafts, within the range of variability from the site. Most notable was the similarity in the load which produced passive earth pressure failure in this fractured rock formation.

Pascagoula, MS Pile Group

A third and quite different case history is provided by the results of static and rapid lateral loading tests on a group of six prestressed concrete piles. This test was a part of a field test pile program intended to provide guidelines for deep foundation design for a new bridge over the Pascagoula River as well as for other future bridges along the Mississippi Gulf Coast. The site for the testing program is adjacent to the alignment for the new bridge in an area with approximately 5.5 m water depth. The soils are predominantly alluvial deposits of soft to stiff clays above elevation -18 m, with dense sands interbedded with stiff clays below that elevation.

The static lateral load test setup consisted of two foundations to be loaded by jacking each against the other as shown on Fig. 12; additional details of the static load test results are provided by Brown and Crapps, 1998. The static test was performed in increments of load, with the load at each increment maintained for a period of around 60 minutes. The pile group consisted of six square prestressed concrete piles, 0.76 m in width, which are arranged to
have two vertical piles, two batter piles designed to act in compression, and two batter piles designed to act in tension. The batter piles were installed on a 1:4 batter. The piles were spaced at 3 pile widths at the cap, center to center. The driven piles were embedded 1.5 m into the 2.4 m thick concrete cap to provide sufficient development length on the prestressing strands so that the full moment capacity of the pile was available at the base of the cap. The piles had 24 - 12.5 mm (1/2 inch) diameter strands with 75 mm cover and each strand was prestressed to 144 kN (32.3 kips). A 0.46 m diameter void within the center of each pile was filled with concrete after driving.

After completion of the static lateral load tests, the Statnamic loading was applied from the deck of a barge so as to load each foundation in a direction opposite to that of the static test. The Statnamic testing was performed by applying 5 progressively increasing magnitude loadings using the 14 MN capacity device shown on Fig. 13. Care was taken to ensure that large permanent plastic deformations were not induced in the piles or shafts during the static loading which would affect performance during Statnamic testing. Measurements of permanent displacements taken using the inclinometer device suggest that a permanent lateral displacement after static testing was about 6 mm.

The pile group motions were dominated by lateral translation with very small rotations (as was the case for the static loading). The derived static and damping resistances were computed using a mass equal to the mass of the pile cap plus the contribution of the piles above the mudline. A damping ratio of approximately 30% proved effective in matching the observed displacement time history. Derived static and dynamic resistance as a function of displacement are provided on Fig. 14, along with the static load test measurements. Note that although the static load testing included sustained loading and took about ½ day to complete, the derived static response is seen to match the measured static response quite well. Note also that the derived static curve exhibits the nonlinearity observed in the static test, which was expressed as decreasing foundation stiffness at increased amplitude of motion. The resonant frequency was observed to decrease as the stiffness decreased, as expected.

The peak strain gauge data for the statnamic test results generally appeared quite similar to the patterns of strain from the static test. An example of these measurements is provided on Fig. 15 for one of the more well instrumented of the prestressed piles, pile 5 (vertical). This figure illustrates the strains in the pile from the
first peak during the statnamic loading; i.e., this is a "snapshot in time" corresponding to that peak strain. For this loading, the total static + damping soil resistance at this point in time is approximately 4.5 MN. The maximum static load applied was 3.9 MN and is provided for comparison. The static and statnamic loads were applied in opposite directions, thus these are "out of phase" with opposite signs. The pattern of strains is seen to be identical, with an offset from the 0 axis of the average between the west and east sides of the pile which reflects the axial load strains superimposed upon the bending strains. Pile 5 for the statnamic loading is put into compression while this pile was in tension during the static loading. The strain gauge data suggest that the piles performed quite similarly during the statnamic and the static lateral loadings.

Summary and Conclusions
A method of conducting rapid lateral loading tests of deep foundations has been described, along with a simple procedure for interpretation of the data. The rapid lateral testing procedure is seen to have some advantages in terms of testing efficiency, the capability of inducing very large lateral loads, and the capability of observing dynamic behavior. Several case histories are provided which allow comparisons of the derived static and dynamic response with that of conventional static tests. The simple analytical model described in this paper appears to be capable of providing a reasonable interpretation of the static lateral load response from the rapid loading test and also provides additional information relating to damping. The damping resistance derived from the statnamic loading suggests that the deep foundations tested may have some additional soil capacity available to resist very transient dynamic loading events such as seismic, maximum wind gust, or vessel impact.

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