02 Jun 1993, 9:00 am - 12:00 pm

Discussions and Replies Session 1

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DISCUSSIONS AND REPLIES

SESSION I

Discussion by T.P. Smith
University of Missouri-Rolla, USA

on
Case Histories: Geology, Value Engineering and Deep Foundations in New York Area

Paper No. 1.18

Geological and subsurface conditions in the New York City area are both variable and complex. Based on these factors and the significant amount of construction in the area, New York City has an extensive building code defining, among other things, bearing capacity based on geological history and engineering characteristics for specific soil classifications. The authors present three case studies where this variability in geology and application of Value Engineering lead to different deep foundation choices. The first study is on the choice of pile foundations for a new industrial facility in Staten Island. The authors describe a subsurface of fill and deposits over compact glacial soils. Application of Value Engineering changed an initial design recommendation for pre-cast pre-stressed concrete (PPC) piles to a proprietary Tapered Point Tip (TPT) pile for several of the facilities. TPT piles are particularly suited for sandy soils. Options in addition to the proprietary TPT piles are not presented. The second study was on a pile foundation for a bus depot in the Bronx. Satisfactory bearing surfaces existed at 20-30 ft depth with a GWT at 10 feet depth. Steel H piles were chosen because of their durability and suitability for bearing on rock. The third study concerned underpinning of existing structures using caissons as part of tunnel work for train facility consolidation. The geological profile had variable rock hardness along the construction route. Value Engineering of the problem lead to a choice of 30" diameter caissons bearing on 20 tsf rock (construction problems developed with a 24" diameter caisson bearing on 40 tsf rock). Understanding of the site geology influenced each design decision. Similarly, Value Engineering analysis optimized designs, saving time and money. Value Engineering is a significant point of this paper. The authors could elaborate on how it is applied and why, for example, it was not used for preliminary design (Case 1 and 3) or how it resulted in the choice of more expensive pile (per ton) in Case 2. Based on the complexity of the geology and the construction environment a discussion of the variables and criteria in Value Engineering would also benefit the reader, particularly if there are variables generic to metropolitan area construction.

Discussion by Y.IWASAKI
Geo-Research Inst., Osaka

on
Ground Movements and Pore Pressure Variation Caused by EPB Shield Tunneling - Shanghai(China)Sewage Tunnel

Paper No.1.19

The settlement of the ground by shield driving has been one of the problems in geotechnical construction in urban area and the reported case gives important contribution.

QUESTIONS Please describe the stress condition at the boundary BCE in Fig.11. The authors shows the observed settlement of 100mm at T=0day. Please indicate which portion of the observed settlement in Fig.5 corresponds to the initial settlement, in terms of the above condition.

ON CONSOLIDATION PROCESS The authors conclude the settlements of the surface and the third layer become the same value of 75mm at the final stage as sum of the different contributions from initial settlement and consolidation.

Discusser's opinion; The initial settlement increases with the measured depth, which means the ground above the crown is expanded(void increased) when the crown portion settled 45mm at initial settlement. However, the ground settled later to the initial state (initial void) after a while (delayed settlement), there is no actual consolidation process at all. This process is rather rebound and recompression stage, if you like to describe it as consolidation.

From Fig.6	depth(m)	initial sett. Si(mm)	final settl. S(mm)	change of layer thickness
surface	0.0	29	75	
first layer	0.9	40	75	no change
second layer	1.8	42	75	no change
third layer	2.5	45	75	(compressed by)30mm
shield crown	3.5			

What is important is the settlement of 35mm which took place in a zone of 1m thick below the third layer and above the crown of the machine. Based upon discusser's opinion, this is <u>the real consolidation</u> taken place at the destructured disturbed zone.

DISTURBED ZONE may be less than one meter thick.The authors estimated the disturbed zone as 4.5m by the distribution of the excess pore pressure. However, the discusser considers that these pore pressure changes are mainly due to the change of total stress given by movements of machine and grouting pressure. The pore pressure change caused by total stress change has nothing to do with disturbed zone. Since it is difficult to estimate total stress change, residual porepressure, which may indicate disturb zone, is hard to be separated from the measured pressure.

Discussion by D. Sielbach
Graduate Student, University of Missouri Rolla
on
Foundations Failures of Bridges and Geotechnical Investigations

Paper No. 1.26

This paper presents case histories of two bridge collapses caused by inadequate foundation material for bridge piers.

The brick masonry arch Palakmati Bridge was constructed in 1910. The piers were founded on shallow, brick masonry wells located 3 meters below the lowest bed level in a yellow soil layer. The structure collapsed when a heavy tanker tried to cross the bridge following heavy flooding in 1989. An investigation indicates that the failure was caused by loss of support of pier 3 due to scouring. A replacement structure founded 5.5 meters below stream bed has been proposed.

The concrete slab Mand bridge was built in 1945-1946. Pier P-1 was founded on a gravel layer which was underlain by a layer of silt. A layer of rock is located at a depth of 15 meters. Pier P-1 settled 0.4 meters in August of 1991 causing major damage to the structure and its subsequent closing. An investigation attributes the failure to consolidation of the silty layer and scour in the silty layer. Further investigations revealed that pier P-2 is also not reliable.

The authors indicate that these failures along with others have taught the following lessons: 1) Confirmatory borings must be taken at each pier location and if the piers are founded on rock, the rock should have a minimum thickness of twice the width of the foundation or 5 meters, whichever is greater. These requirements are similar to the current practices recommended by the American Association of State Highway and Transportation Officials for use on public roads in the United States. 2) The type of substructure to be used should be determined by the type of foundation material present, 3) Arches and continuous type structures should not be used where the large settlements are anticipated, 4) Conglomerates and soil with boulders are not good spread footing foundation material where scour is anticipated, and 5) Protective scour countermeasures should be provided where necessary and old bridges should be monitored after floods to determine the presence of scour around piers.

Recent scour related bridge failures in the United States have served to focus attention on the importance of: 1) Designing new bridges for the anticipated scour and 2) Evaluating existing bridges for potential adverse effects from scour and taking corrective actions as necessary. The most notable recent bridge collapse caused by scour was the New York State Throughway bridge over Schoharie Creek on April 5, 1987. As a result of the collapse of the center span during a near record flood, one semi-trailer and four automobiles fell nearly 80 feet into the river. A total of nine bodies have been recovered and one person is still missing. The bridge piers were founded on spread footings bearing on a glacial till which was considered a highly satisfactory bearing material.

To assist with the scour design and evaluation process, the U.S. Department Of Transportation, Federal Highway Administration (FHWA), has developed the Hydraulic Engineering Circulares (HEC) No. 18, "Evaluating Scour at Bridges", and HEC 20, "Stream Stability at Highway Structures". These publications were released in February, 1991 and have publication numbers FHWA-IP-90-017, and FHWA-IP-90-014, respectively.

Major points that are discussed in these publications which may be applicable to bridges over water such as the Palakmati and Mand bridges are discussed below.

NEW CONSTRUCTION OR RECONSTRUCTION

It is important that the bridge location be established in a stretch of the river that is stable either naturally or through the use of countermeasures. In some instances it may be more economical to relocate the bridge to a more stable location.

The site specific subsurface conditions must be considered when evaluating the scour potential at a bridge. Different materials scour at different rates and the anticipated depth of scour must be compatible with the expected service life of the bridge. In general, scour will reach its maximum depth in sands and gravels in a matter of hours; cohesive soils in a matter of days; glacial tills, sandstones and shales in months; limestones in years; and dense granite in centuries. An interdisciplinary team of engineers with expertise in hydraulic, geotechnical, and structural engineering should be used for all foundations designs.

The bridges should be designed to withstand the effects of the estimated total scour which would occur during its expected service life. Total scour is comprised of 1) Aggradation and/or degradation of the stream bed, 2) Contraction scour, and 3) Local scour. Methods and guidance for estimating total scour are given in the HEC publications.

New bridges should be designed to withstand the effects of total scour for a specified superflood with little or no risk of failure. A structure's importance and its potential consequences of failure should be considered when determining which superflood to design for.

EXISTING BRIDGES

The HEC publications suggest the following steps be followed to determine the vulnerability of existing bridges to scour:

1) Compile a list of those bridges with actual or potential problems due to scour.
2) Prioritize the scour susceptible bridges by conducting a preliminary office and field examination.

3) Conduct field and office scour evaluations of the bridges on the prioritized list using an interdisciplinary team of hydraulic, geotechnical, and structural engineers.
4) For bridges identified as being scour critical by the interdisciplinary team, develop a plan of action for correcting the scour problems.
5) The remaining waterway bridges not initially identified as scour susceptible should be evaluated.

This process of identifying the current status of the bridges regarding their vulnerability to scour is currently being undertaken in the United States.

Also, to meet the requirements of the National Bridge Inspection Standards (NBIS), all bridges on public roads must be inspected at least every two years. As part of these inspections the condition of the substructure, channel, channel protection, and waterway adequacy are evaluated.

To assist with the identification of scour critical bridges, a program of underwater inspections has recently been established. This program requires all public bridges over water to have underwater inspections on a regular basis. The underwater inspections may consist of wading and probing where possible, and the use of divers and/or electronic scour detection equipment where necessary.

The reader is referred to the HEC 18 and HEC 20 publications for additional details and information regarding the principles and practices recommended by the FHWA for evaluating the scour potential at bridges. The NBIS and related Bridge Inspection Manual should be consulted for additional information on bridge inspections and monitoring.

Discussion by T.P. Smith
University of Missouri-Rolla, USA

on
Foundation Problems at a Residential Complex-A Case History

Paper No. 1.27

Supplementing building construction by adding additional stories to a facility, particularly a residential complex, can pose significant foundation problems in terms of settlement and bearing capacity. In this paper, Sinha describes a testing and investigation approach used to investigate the adequacy of a foundation and foundation soil for modifying an existing single story building to a double storied configuration. The testing plan for the building addition included plate load tests (PLT), standard penetration tests (SPT), and dynamic cone penetration tests (DCPT). Lab analysis included particle size distribution and Atterberg Limits. Using the data from PLT results, Sinha computed bearing capacity and settlement using the tangent method and log-log method. From these calculations Sinha determined that certain areas of the complex were adequate for a second story, certain areas were adequate for a second story of lightweight construction material, and other areas required strengthening. Observation of wall and roof cracks inside the facility were consistent with testing conclusions. Suggestions incorporated into the building upgrade based on the investigation worked satisfactorily. Unstated are specific foundation recommendations and problems, if any, encountered in implementing these recommendations. It is in the investigative portion of the paper that additional information would benefit the reader. For example, what were the existing foundation design and soil conditions, by standard classification, for the areas in question? Also, one area of the building was judged to have an insufficient safety factor for its current single story loading. Did this require any specific, immediate actions to remediate the existing design? In the interest of developing lessons learned from the successful investigation and recommendations, general issues are worth discussing. What role did the initial building design plan and soil investigation, particularly for the footings, play in the investigation. Was the foundation design physically verified? Sinha provides the results of SPT and DCPT tests but does not address their role in the investigation. For future investigations, in the interest of economy, would the author recommend these tests or would he primarily use the PLTs on which most of the conclusions are based? The paper is clearly beneficial and discusses a common approach to improving use of existing facilities in congested areas. Much can be learned from extrapolating procedures and findings to a more general approach to the problem.

Discussion by C. Mirza
Strata Engineering Corp., Don Mills (ON) CANADA

on
Behaviour of Laterally Loaded Drilled Shafts in Stiff Soil

Paper No. 1.38

The use of the prebored pressuremeter and dilatometer in predicting the lateral load-deflection response of cast in situ concrete piles in a stiff varved clay deposit is presented in a most concise and compelling manner. Unfortunately, the spacing between the tested piles and their sequence of arrangement in plan view has not been provided. The authors stress the need to properly interpret p-y curves for valid predictions, and suggest that the side friction component from the F-Y curve can be an important component of the total resistance.

Although the shape of the load-deflection curves obtained from the in situ tests do match the actual load-deflection curves rather well (Figure 3), the question of why a shaft with a diameter of 0.51 m showed greater resistance than a similar length shaft of 0.61 m diameter does not appear to have been sufficiently scrutinized, the difference in observed performance being attributed to natural soil variations. Figure 4, based on the load test data of Figure 3, indicates clearly that the load-deflection behaviour is shaft length dependent. The almost identical behaviour of Shafts 1 and 3 (1.52 m length) suggests that even with a 20 per cent difference in shaft diameter (0.51 m vs. 0.61 m) there is little difference in performance.

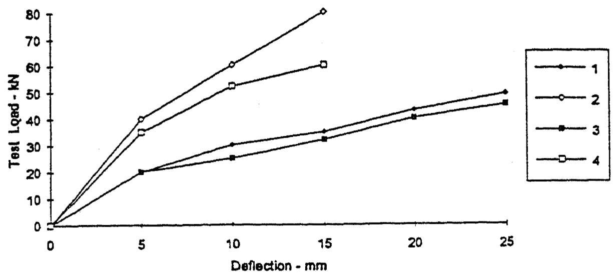

Figure 4. Load vs. Deflection, Shafts 1-4.

The question as to why the 0.51 m diameter pile (Shaft 2) performed better than the 0.61 m diameter pile (Shaft 4), when both are of the same length (2.44 m), needs to be answered. Figure 5 shows the relationship of the L/D ratio for the four test piles versus the load at 10 and 15 mm deflections at the pile head. Clearly, there is a trend for the lateral load capacity of these shafts to increase with the L/D ratio. It would therefore be interesting to see these tests extended to larger L/D ratios, while ensuring the shafts remain within the desiccated zone.

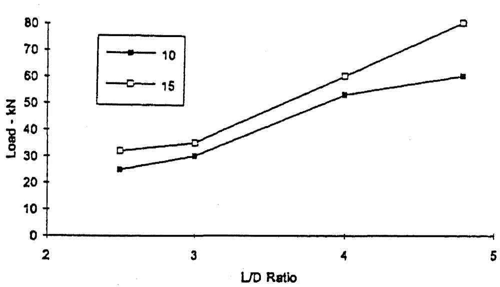

Figure 5. Test Load vs. L/D at 10 and 15 mm Deflections

For first approximations, the lateral load of drilled shafts can generally be estimated to be some percentage of the computed static vertical load capacity. The measured lateral loads for the tested piles, at 10 mm deflection, range from about 8.6 per cent (Shaft 3) to 15 per cent (Shaft 2) of the vertical shaft resistance (calculated ignoring end bearing and a concrete-soil adhesion of 100 kPa), and average about 12 per cent, which matches closely values of 12-15 per cent obtained from lateral load tests on 2 m diameter unlined concrete caissons installed within a cohesive stiff glacial till in Toronto.

It is hoped the research will be continued to include vertical compression and tension load tests, and lateral load tests both before and after each vertical load test. The writer's personal experience indicates that the application of lateral loads subsequent to vertical loading (such as after a bridge has been opened to traffic) tends to reduce the vertical load capacity in serviceability limit states. In situ tests of the type reported in this paper may provide valuable insights into this phenomenon.

Discussion by C. Mirza
Strata Engineering Corp., Don Mills (ON) CANADA

on
Taj Mahal - An Appraisal of Foundation Performance

Paper No. 1.70

It is always a pleasure when geotechnical information is obtained and interpreted for historic structures, especially those which have suffered no apparent foundation problems. That pleasure is enhanced immeasurably when the structure is a world renowned monument such as the Taj Mahal, where to any visitor, the monument to love located above the ground all but erases from enquiry the wonderment of how it may be supported below the ground. The authors are therefore to be applauded for sharing their findings with the international geotechnical community, which can now claim the enviable privilege of knowing secrets hitherto buried, literally speaking.

The paper provides subsurface information which indicates the presence of a 10 m thick sand, in a probable compact to dense state, sandwiched between an upper and a lower clay deposit of soft consistency. The authors presume the foundations of the structure were probably taken down below scour depth and are likely situated within the sand deposit. It is doubtful that the builders of that time had access to information on calculating scour depths to the degree of preciseness implied. It is however, quite likely that they were knowledgeable enough to avoid the upper clay deposit for their foundations.

The pressure-void ratio relationship of the lower clay deposit (Table 1) shows void ratios which are somewhat lower than those computed from the average reported 23.4 per cent moisture content and a specific gravity of 2.67. The lower clay soil may therefore in fact may be a slightly plastic silt (ML). The low undrained shear strength of about 50 kN/m^2 reported for the lower clay stratum is inconsistent with the average moisture content of less than 25 per cent. Such low undrained shear strengths are often associated with normally consolidated highly plastic clays having moisture contents normally in excess of 40-50 per cent. It is unfortunate that plasticity data (Atterberg limits) have not been provided for this lower deposit.

In consideration of the historical, national and world-wide significance of the Taj Mahal, it would seem to be only natural that no effort should be spared in a thorough assessment of the subsurface conditions when any change to the status quo is being contemplated. The authors conclude that the foundations of the structure have remained submerged for some 350 years, and that 99.4 per cent of the calculated settlement (1.40 m) has already occurred, and therefore there is little danger to the monument when the status quo is changed.

Any structure which settles (or is calculated to have settled) a total of 1.4 m must exhibit differential settlements, since, to paraphrase Terzaghi, nature is never so obliging as to provide uniform thickness and properties of compressible material below large and massive foundations. The stratigraphic cross-sections of Figures 2 and 3 suggest clearly the possibility of differential settlement due to the variable thickness of the sand stratum. Yet, the authors cite "flawless performance" of the structure, and how well it has withstood the test of time. Thus, either the settlement has been computed on the basis of extremely conservative values for the lower clay (which is more likely a silt, given the nature of river deposits), or in fact the structure has not settled 1.4 m. In either case, questions arise, which require a more thorough examination of the facts as well as the assumptions. For example, the reported average undrained shear strength of 50 kN/m^3 at depths of 50 m or more below river bed level defies all credulity. Hence, the proper characterization of the soils below the expected foundation level will do for starters (Mirza, 1982).

The risks associated with the several assumptions regarding foundation type, depth, stress distribution, nature of the subsoil and impacts from likely changes in hydrostatic heads are just too great for a structure of the stature, significance and majesty of the Taj Mahal. One hopes that the impacts of any proposed changes to the status quo will be examined and approved by an independent review panel of experts before such changes are implemented.

Mirza, C. 1982. A case for the extension of the unified soil classification system. Canadian Geotechnical Journal, vol. 19, No. 3, pp: 388-391.

Replies by S. K. Jain
New York City
Department of Transportation
Bureau of Bridges Design

on

"Case Histories: Geology, Value Engineering and Deep Foundations in New York Area"

Paper No. 1.18

The writer would like to thank Dr. T. P. Smith, of University of Missouri-Rolla, U.S.A. for his comments and interest shown in the paper. Dr. Smith has asked the following questions:

1. Why foundation types selected through value engineering for case histories 1 and 3 were not considered during final design?

2. How value engineering resulted in the choice of more expensive (per ton) H Pile for case history No. 2?

<u>Response</u>

1. The use of TPT Piles was considered during final design but the option was discarded due to the presence of discontinuous layers of silt (Stratum M). The tip of end bearing piles sitting above or in the silt stratum was thought of potential source of settlement. During construction, the contractor performed elaborate subsurface investigation and testing to define the silt layers and agreed to drive the piles below the silt stratum which required modification or revision to the pile driving criteria established by pile load tests. The owner accepted the value engineering proposal offered by the contractor and saved almost one million dollars in foundation cost. Franki pile (expanded base) was an alternate proposal. In case history No. 3, the caisson size had to be revised due to unanticipated poor rock conditions encountered during construction. This poor rock condition was limited only to a small portion of the site. The contractor presented a cost analysis or value engineering study to justify his scheme and to avoid construction related delays.

2. In case No. 2, high capacity (150 ton) H Piles were chosen because it was determined that less number of piles would be needed to resist specified structural column loads. Considering the savings in foundation construction and pile caps, etc., H Piles appeared more economical.

Generally, the most important parameter in the selection of optimum foundation system is the understanding of local geology and foundation experience in the area.

Replies by K. M. Lee
Department of Civil & Structural Engineering
The Hong Kong University of Science & Technology
Clearwater Bay, Hong Kong

on

"Ground Movements and Pore Pressure Variation Caused by EPB Shield Tunnelling - Shanghai (China) Sewage Tunnel"

Paper No. 1.19

In the Finite Element Modelling of the problem, we did not simulate the outward pressure (and the associated ground heave) generated by the positive bulkhead pressure of the earth pressure balance (EPB) shield. Excavation of the tunnel was simulated by first deducing the equivalent nodal forces that would be acting around the tunnel boundary prior to excavation (without considering the outward pressure) and then removing these nodal forces to create a stress-free surface.

Comparison between the measured and the calculated surface settlements with disturbed zone was given in Fig. 12 of the paper. The observed settlement of 100 mm at Time=0 at Fig. 12 was obtained from the vertical difference between 5 and 4 in Fig. 5*a* (i.e., we are comparing the net amount of ground settlement but not the absolute magnitude of ground movement). Obviously, if the heaving force can be simulated under plane strain cross-sectional conditions, we would be able to directly compare the absolute ground movement. The positive face pressure ahead of the tunnel face, however, is a three-dimensional phenomenon. It is our interest to understand the three-dimensional continuous face support mechanism associated EPB shield, and to develop simplified techniques to simulate this 3D face pressure by means of 2D plane strain approximation.

The so-called "disturbed zone" of soil around a tunnel opening would be induced by two possible mechanisms: firstly, the change in in-situ stresses and pore water pressure associated with tunnel excavation; secondly, the mechanical remoulding associated with alignment problems (pushing, pitching, yawing, etc.) of the tunnel shield. A correct simulation should consider both types of mechanisms. Because of the 3D nature of stress changes and pore pressure changes around the tunnel opening associated EPB shield, it is very difficult to correctly simulate this form of ground disturbance in a 2D plane strain manner. In the paper, this was approximately simulated by enlarging the "disturbed zone" to a much bigger value of 4.5 m to reflect this type of ground disturbance. We agree with the discusser's opinion that the actual thickness of "mechanical disturbed zone" is in the order of 1 m. Unless we have a better understanding of 3D total stress and pore pressure changes, it is very difficult to separate these two types of disturbance. It is our opinion that this problem would be subject to further research investigation.

Replies by U. N. Sinha, Scientist,
Geotechnical Engineering Division,
Central Building Research Institute,
Roorkee - 247 667.

on

Foundation Problems at a Residential Complex- A Case History

Paper No. 1.27

The author would like to offer sincere thanks to Mr. T. P. Smith for showing keen intereset and seeking some of the clarifications on above paper and finding it a beneficial study for raising the existing buildings to double storeyed configuration. The case study as published is based on field and laboratory investigations (Tables I, II and III). It is mentioned that the user's organisation constructed a residential complex of various type of accomodations having single storeyed row type construction. No soil investigation report was available except width and depth of foundation adopted for various type of accomodations (Table III). The depth of foundation of 1.35 m in all cases was reported. Having no information on soil investigations the author undertook soil investigations adjacent to existing buildings (Fig. 1). Though field investigations for PLT, SPT and DCPT were carried out but the recommendations were made based on bearing capacity of foundation soils and settlement criteria. However author considered beneficial to undertake SPT and DCPT for assessing the general strength characteristics of soil layers upto around 10 m depth [Fig. 2(a-e)]. The results of SPT and DCPT as obtained [Fig. 2(a-e)] reveal low N-values upto 3.5 m depth indicating presence of soft deposit of soils. Fig. 2(e) reveals higher N-values and is probably because of medium stiff layers. Below 3.5 m the improvement in N-values was observed. The classification of soils were carried out according to IS 1498 - 1970 and are also mentioned in Fig. 2 (a-e) and soil layers were of ML, CL-ML, CI, CL and SM groups and are considered to be low to medium compressibility.

In the paper the conclusions were drawn on the basis of Plate Load Test results considering bearing capacity of foundation soils and settlement criteria and accordingly recommendations were made for raising the existing buildings to double storeyed configuration wherever it was found adequate. As pointed out by Mr. Smith, the author would like to suggest to include SPT and DCPT alongwith PLT to get general condition of soil stratification with strength characteristics. The use of SPT and DCPT results were also indirectly made for recommendation. Hope this clarifies most of the points raised by Mr. Smith. The interest and valuable suggestions are highly appreciated.