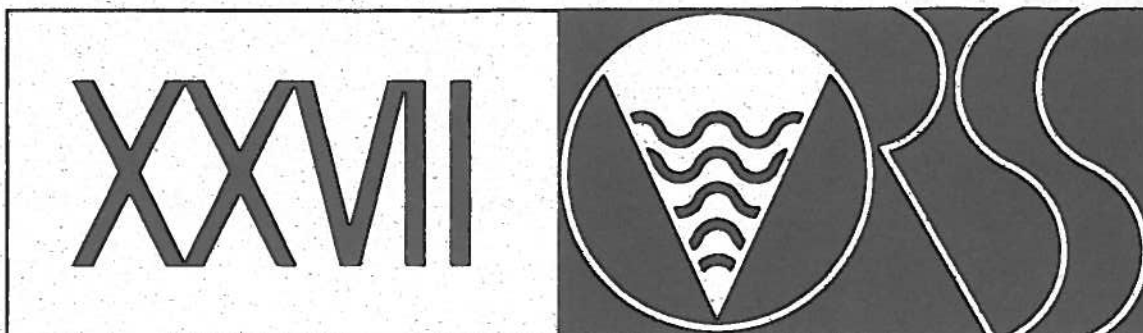




FORENSIC STUDIES IN GEOTECHNICAL ENGINEERING



Cincinnati, Ohio
October 11, 1996

ORVSS XXVII

FORENSIC STUDIES IN GEOTECHNICAL ENGINEERING

OCTOBER 11, 1996

AGENDA

- 7:00 Registration - Coffee / Pastries
- 8:00 Welcome & Opening Remarks - *Doug Batt, Chairman*
- 8:10 Seminar Moderator - *Clyde N Baker, Jr.*
- 8:30 Calculated Risk - *Fred H. Kulhawy*
- 9:45 Break - Coffee
- 10:00 Geotechnology in Dispute Resolution - *James P. Gould*
- 11:15 Dispute Avoidance and Piling Specifications - *Bengt H. Fellenius*
- 12:30 Lunch - provided to registered attendees/speakers/exhibitors
- 1:30 Collapse of Geogrid-Reinforced Retaining Structure - *Gerald A. Leonards*
- 2:45 Break - Coffee/Soft drinks
- 3:00 Foundation Failures : Primary, Secondary and Tertiary Liabilities
- *Jorj O. Osterberg*
- 4:15 Panel Discussion - Moderator - *Clyde N. Baker, Jr.*
- 5:15 Social Hour
- 6:30 Closing

PROCEEDINGS
of the
TWENTY-SEVENTH
OHIO RIVER VALLEY SOILS SEMINAR
(ORVSS XXVII)

FORENSIC STUDIES IN GEOTECHNICAL ENGINEERING

October 11, 1996
Sharonville Convention Center
Cincinnati, Ohio

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PROGRAM SPEAKERS

Clyde N. Baker, Jr., P.E., SE

Dr. Bengt H. Fellenius

Dr. Jorj O. Osterberg

Dr. Gerald A. Leonards

Dr. Fred H. Kulhawy

Dr. James P. Gould

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OCTOBER 11, 1996

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APPENDIX

Past Ohio River Valley Soils Seminars

**"EARTH RETENTION DESIGN, GROUND MOVEMENT
MONITORING
AND LIABILITY - A CASE HISTORY"**

**Clyde N. Baker, Jr.¹
STS Consultants, Ltd.**

ABSTRACT

The role of the geotechnical engineer in earth retention design to control ground movements is reviewed. The different perspectives of the design geotechnical engineer working for the owner and the geotechnical engineer consultant working for the contractor on temporary earth retention are discussed. The role of the owner in assessing the cost implications resulting from different geotechnical perspectives is reviewed, using a major downtown Chicago three-level basement excavation as a case history.

The case history involved protection of city streets and utilities, modification in the design to increase safety against slope failure, monitoring of ground movements with inclinometers, monitoring of water loads on the earth retention system with strain gauges on representative rakers, and a comparison of predicted loads and deflections versus actual.

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Ltd., Northbrook, Illinois 60062

INTRODUCTION

A. Our increasingly litigious society

Most of us are well aware that we are living in an increasingly litigious society. Those of us in the geotechnical engineering profession see it most directly in the increasing cost of our professional liability insurance. We also see it in comparing how often we are sued today with 25 years ago. Twenty-five years ago, there seemed to be recognition that the unexpected could always occur in the underground without necessarily meaning somebody was negligent. Contractors maintained property insurance to cover minor to moderate damage to adjacent structures caused by a contractor's excavation. Today, there appears to be an increasing effort on the part of insurance companies to find someone to blame or at least to share the cost.

B. Excavation, ground movements, property damage and litigation

Theory and experience tell us that when an excavation is made in the ground, some ground movement must occur. The probable minimum amount of movement would be the elastic rebound due to stress relief and the maximum would be total excavation wall collapse. Reliably predicting the amount of movement that will occur for a given excavation sequence and retention system is difficult at best and is often impractical due to the influence of even small changes in construction procedures and in anticipated subsurface condition. In addition, determining the amount of movement which can be tolerated before unacceptable property damage occurs is often even more difficult. In spite of this, if the retention system is constructed in accordance with accepted plans and specifications and still ground movements caused unacceptable damage to adjacent structures, the contractor's insurance company is apt to say "my client did his work in accordance with specifications and therefore someone else must be at fault and should pay the cost of repair".

For a given project, the owner or developer typically feels that he has hired the best design engineers and contractors and has a right to expect a competent design and construction that won't result in lawsuits and extra costs. The structural engineer and the contractor are apt to contend that they relied on the report of the geotechnical engineer to develop the most appropriate design and construction techniques. The geotechnical engineer, for his part, usually points out the possibility of soil occurrences which may be different from what he has tested and indicated in his report. Previous construction, old foundations, sewer lines (etc.) can further complicate a soil profile.

The benefits, compensation, and liability related to cost reductions in a retention system design must also be put in perspective. The owner of the project is usually the beneficiary of any construction cost savings resulting from competitive bidding. Sometimes these cost savings may result in reduced factors of safety against ground movements and adjacent property damage. At the same time, the contractor is receiving the bulk of the money spent on the construction to accomplish the design. While the geotechnical engineer may have developed the soil parameters on which the design and construction is based, he receives only a small fraction of a percent of the cost of the project for his fee, yet often bears a disproportionate amount of the liability.

With the foregoing background, in the remainder of the paper we will review the various roles of a geotechnical engineer having to operate within the conflicting perspectives of the parties involved. The paper will conclude with an interesting case history utilizing a major downtown Chicago deep basement project.

THE ROLE OF THE GEOTECHNICAL ENGINEER IN EARTH RETENTION DESIGN

In the writer's experience, on most projects in the United States today, design engineers separate design of permanent structures from design of temporary structures. Usually, the design of temporary structures is relegated to the contractor. (The work is most often actually performed by specialty subcontractors.) In some cases, in an attempt to limit their liability exposure, the design engineers make certain that their design contracts do not include responsibility for review of the contractors earth retention design for fear that such review would increase their liability. These projects often utilize a performance type specification in order to put the responsibility of the retention system performance totally on the contractor. In an effort to ensure a minimum level of competence, however, the specifications will often include experience qualification requirements and will require the contractor to retain a licensed structural engineer and/or geotechnical engineer to design his temporary earth retention system, either having such people on staff or retained independently.

The perspective and objective of the contractor bidding on and building the project is to accomplish the design and specification requirements at minimum costs. Minimum costs can often mean minimum acceptable factor of safety. However, the question "What is a minimum acceptable factor of safety?" requires knowledge of the price of failure and what is an acceptable risk. If the adjacent property damage potential is not too great, the contractor may be willing to risk a failure and thus operate with a theoretical factor of safety which is close to 1. The owner's perspective may, however, be somewhat different. Even if the actual dollar cost of failure is not huge (cost of repairing sidewalk and streets), the owner may be unwilling to accept a significant risk of failure because of concern for damage to his "community image". In this case, an owner might want a significantly higher factor of safety than the contractor.

Thus, a geotechnical engineer working for a contractor attempting to win a bid for an earth retention design might reasonably work with a relative factor of safety of 1.2, whereas a geotechnical engineer working for the owner might reasonably desire a relative factor of safety of 1.5.

How should the geotechnical engineer operate within the potentially conflicting perspectives of contractor and owner? The answer is: very carefully.

Whether working for the owner or the contractor, the following guidelines apply:

1. Carefully state all assumptions.
2. Be thorough and detailed in the analysis and note any construction sequencing limitations.
3. Have work reviewed by competent peers, either in-house or external.
4. Predict insofar as practical, approximate earth movements expected if design and sequencing are followed, unless the specifications already give movement limitations.
5. Insist on a monitoring program and have contingency plan recommendations in the event movements significantly greater than predicted occur particularly if there is adjacent property that could be damaged if excessive movement occurs.
6. Insist on a field representative to see that design assumptions are met and required construction sequencing is followed. If concerned about construction control, insist on indemnification.

In many projects, the geotechnical engineer has been retained by the owner for exploration and consultation services, and the contractor has been given full earth retention system design responsibilities. Should the geotechnical engineer become involved in the review of the retention system or should he/she maintain a "hands off" policy in an attempt to keep the full liability on the contractor? Will he get sued anyway if failure occurs?

In our opinion, if asked by the owner to review the design, the geotechnical engineer has a professional responsibility to perform the review recognizing the different perspectives. If after review he thinks the contractor's design is unsafe or marginal (involves an unacceptably low factor of safety and a high risk of excessive movements) he should discuss his concerns with the contractor, reviewing assumptions and calculations. If differences cannot be resolved, he should report the differences of opinion and the reasons for them to the owner.

The writer recently had an opportunity to be involved in the above review type role for a major downtown Chicago deep excavation project which will be described briefly in the next section.

QUAKER TOWER CASE HISTORY

1. The Quaker Tower site is located at the northeast corner of the intersection of Clark Street and the Chicago River. The project consisted of a 40-story office tower with three full basement levels, involving a 36-foot to 39-foot cut below lower level street grade except on the Clark Street side where a raised street level resulted in an effective 55-foot deep cut. A 12-foot high pile supported retaining wall plus small berm provided the initial earth retention system for support of Clark Street. Figure 1 shows the cross section through this site from west to east, including initial grade and final excavation levels. Also shown are the assumed soil profiles with the soil cohesion taken as one half the unconfined compressive strength of 2 inch diameter Shelby tube samples. Figure 1A shows the location diagram.
2. The contractor assumed average soil properties based on the site geotechnical engineering report and analyzed for a 36-foot deep cut with surcharge to account for the retaining wall backfill load.

3. The contractor's soldier pile and lagging design assumed an open cut full depth with a berm and with the initial top brace not installed until half of the base mat was poured. Once this top level brace was installed, the next two wale and brace levels were to be installed sequentially with appropriately cut back berms.

4. The geotechnical engineer retained by the owner was asked to review the contractor's design. In his review, he assumed the poorest soil profile from the borings, rather than the average, and considered a 55-foot deep cut when analyzing for the deep seated slide case.

His analysis indicated a high level of risk of slope failure or, at a minimum, excessive movement, before the top raker could be installed. The theoretical factor of safety was less than 1, if the poorest soil profile was assumed and lateral resistance of the piles supporting the existing retaining wall was discounted.

5. The contractor was asked to review his assumptions and to analyze for slope stability. By using average soil properties and taking into account the bending moment capacity of the piling, the contractor was able to show an overall factor of safety very slightly in excess of 1.
6. Meetings were held with the owner and contractor to resolve differences. The contractor felt his system would work as designed, but expected 6 inches to 12 inches of settlement or movement. The owner was concerned that the city would stop the project, or at least have to close Clark Street if movement or settlement exceeded 6 inches. He believed this would have a bad effect on his image regardless of the cost factors.

7. Methods for reducing the risk of excessive movements and increasing factor of safety were discussed with the contractor. The geotechnical engineer recommended partial excavation and then cross lot bracing to permit the entire mat construction, installing the top raker through the cross lot braced excavation to the mat, and then proceeding as before. See Figure 2. The contractor countered with replacing the cross lot bracing system with a tied back wall on one side, using regroutable ties and with an open cut on the other noncritical side of the excavation where there was ample room for open cut. The geotechnical engineer agreed and the owner and contractor negotiated a price for the additional work. See Figure 3.
8. Inclinerometers were used to monitor movement during the various stages of excavation and bracing. Figures 4 and 5 show the inclinometer movement for the center of the Clark Street side of the excavation for two inclinometers located just outside the soldier piles and about 30 feet apart. A third inclinometer was located on Clark Street about 15 feet west of inclinometer I-2. This inclinometer initially showed a similar movement shape but proportionally less, although it ultimately showed maximum movements in the soft clay about equal to inclinometer I-2. See Figure 6. The different curves are dated and show the amount of movement at each of the various stages of excavation and bracing. The amount of actual ground surface settlement that occurred in the area of the inclinometers is shown on Figure 3 and indicates maximum settlement and total volume of settlement approximately equal to the maximum lateral movement and volume of lateral movement in the soft clay or about 6 inches.

In analyzing the observed movements, it is evident that considerable movement occurred during the time that it took to excavate down to the lower tied back wall level and install the soldier beams and regroutable tie backs (approximately 1 to 2

inches). An additional 1 to 2 inches occurred during the length of time it took to excavate and pour the mat foundations and install the top raker level. From the time of the first raker installation until completion of the total excavation and three levels of rakers, an additional 2 inches of movement occurred with an additional approximate 1-inch movement during backfilling and raker removal, for a total movement of approximately 5 to 6 inches. While this movement is greater than was desired, it was not sufficient to cause closing of Clark Street at any time during construction.

9. Vibrating wire strain gages, installed on three rakers (on top, bottom and both sides) located between the inclinometers after the rakers were installed and monitored during maximum loading and again on unloading, showed total loads a little less than projected by the contractor (about 83% of that calculated by the contractor, and significantly less than that calculated by the geotechnical engineer) but with a much different distribution. The contractor's predicted loads and the measured loads at a representative cross section are shown in Figure 7.

It is evident that the top waler carried significantly less load than predicted, whereas the bottom waler carried considerably more load than predicted with the

center waler as predicted. It is likely that part of the explanation for the lower total loads and particularly, the lower top brace load is the horizontal load carried by the pile supported retention wall. Another possible explanation for this tendency would be the beginning of a circular slide rotating around the center brace level, thereby unloading the top brace level and increasing load on the bottom brace level. The magnitude of movement below excavation level indicated by the inclinometers lend some credence to this possibility. This also substantiates the geotechnical engineer's concern for a deep seated slide if an attempt had been made to make an open cut all the way to the maximum cut elevation -30 before installing any retention system and bracing.

On the other hand, the fact that the total loads were less than the predicted loads might indicate that the ground was stronger than assumed and the factor of safety greater than calculated, which would support the contractor's belief in the original design.

10. The contractor and the geotechnical engineer maintain a difference of opinion on the likelihood of the original retention system working. The contractor believes that the retention could have worked, provided the excavation was made quickly enough and the mat poured and top raker installed in the shortest possible time. The geotechnical engineer remains convinced that a major slide would have been likely and the work could not have been completed quickly enough to avoid it. At the same time, the magnitude of movements that occurred before the top raker was installed emphasizes the need to get the top raker in as quickly as possible. Slope movements definitely occurred in the excavation slope above the lower

level tie back system. It is possible that utilizing driven piles for tie backs could have speeded up the installation process and reduced the length of time the excavation needed to be open prior to installing the top raker. However, driven piling would have introduced greater damage potential to existing foundations if the piling were to hit any of the existing retaining wall pile supports during driving. Alternatively, the slope movements and resulting initial lateral displacements could have been reduced further by making a shallower initial cut and utilizing a two level tie back or brace system for the inner tied back wall. This of course, would have been more costly and would have had to be considered in any cost benefit analysis.

CONCLUSION

In conclusion, it is important to recognize the different perspectives of a contractor and an owner and their respective geotechnical engineers in temporary earth retention design and construction. The value of review of the contractor's design by the owner's geotechnical engineer has been emphasized. The willingness of an owner to pay extra for a safer temporary earth retention design helped foster a more cooperative attitude among all the parties involved.

ACKNOWLEDGEMENTS

The writer wishes to express his appreciation to the developer, BCE Development Properties, inc. for permission to use data from their project, and to the architect and building design structural engineer, Skidmore, Owings & Merrill for their technical support. The writer would also like to thank the general contractor, PCL Construction, and earth retention subcontractor, Schnabel Foundation Company, for their cooperation and assistance. Instrumentation of the rakers was authorized and paid for by Schnabel Foundation Co.

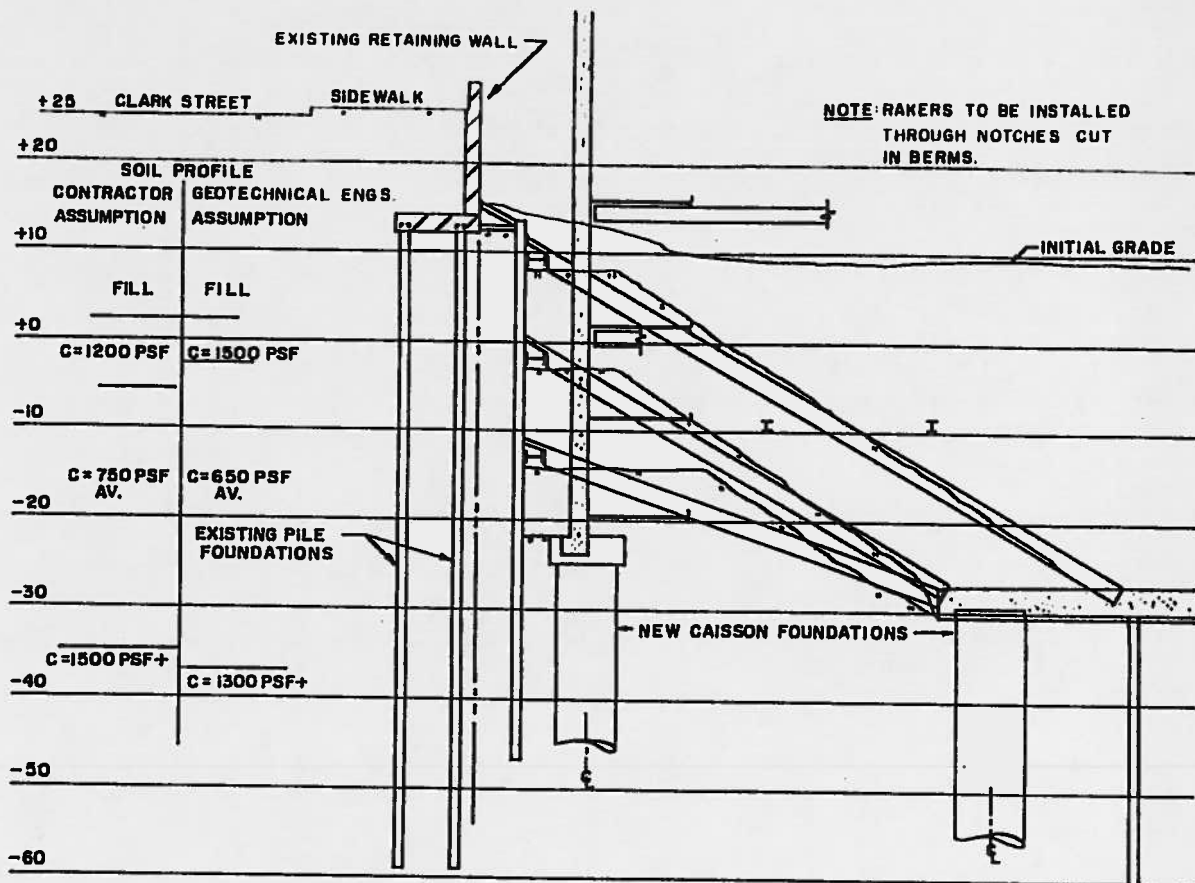


FIGURE 1 - EXISTING & PLANNED CONSTRUCTION

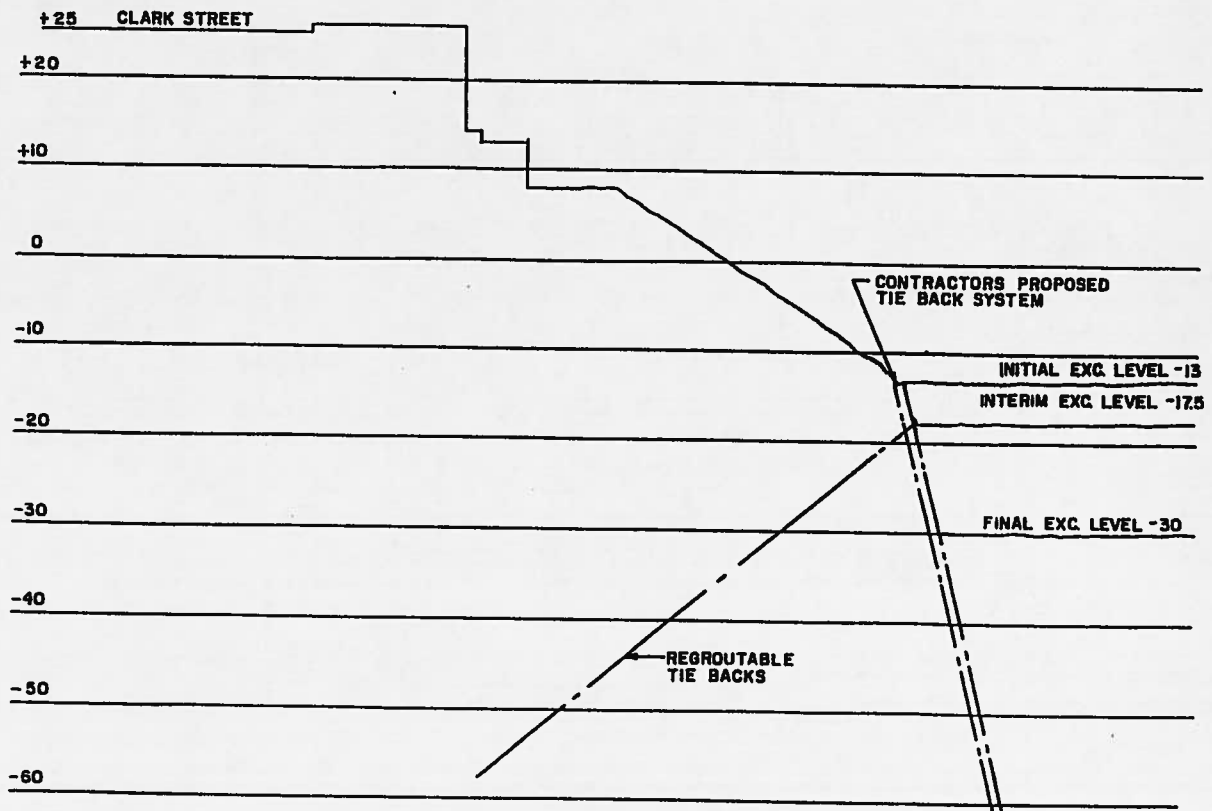


FIGURE 2
PROPOSED LOWER LEVEL TEMPORARY RETENTION SYSTEMS



FIGURE 4



SITE LOCATION DIAGRAM

FIGURE 1A

NOTE:
ALL ELEVATIONS
ARE C.C.D.

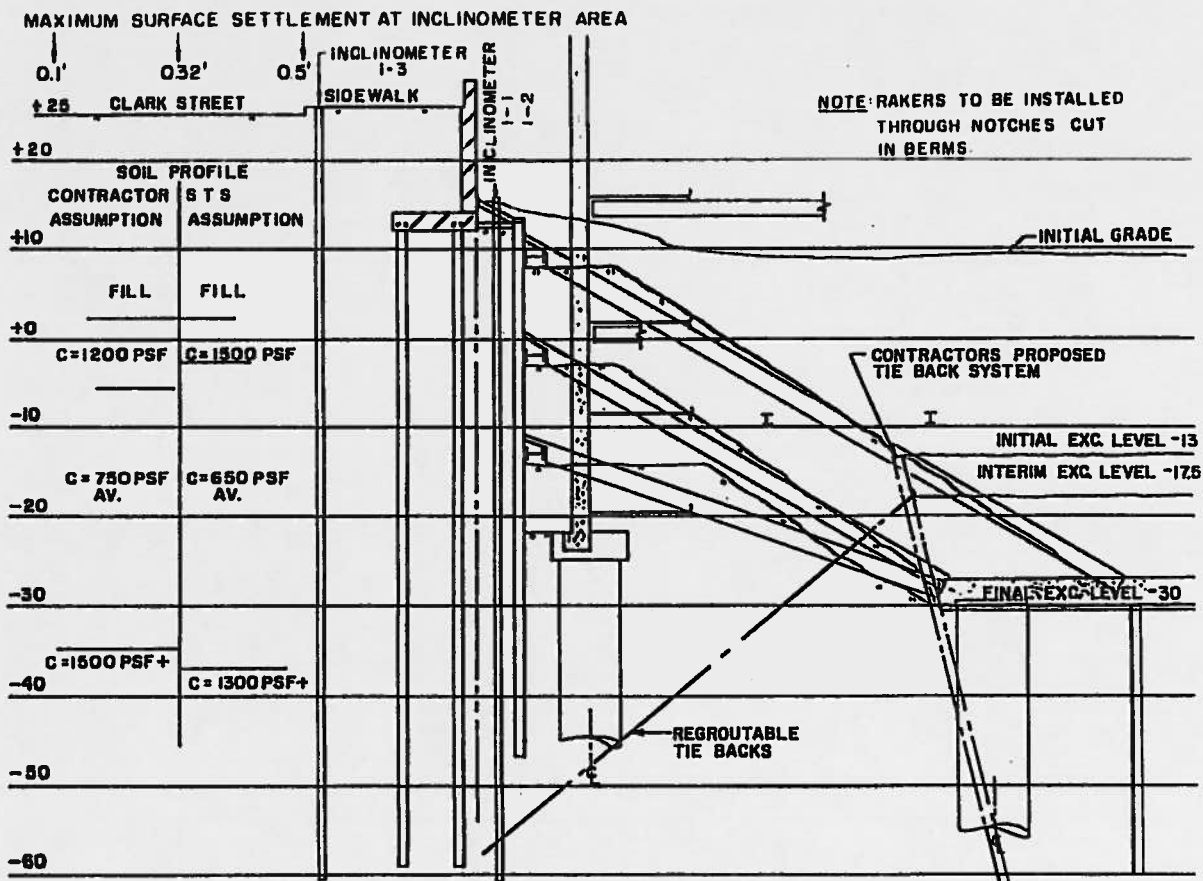


FIGURE 3-FINAL RETENTION SYSTEM

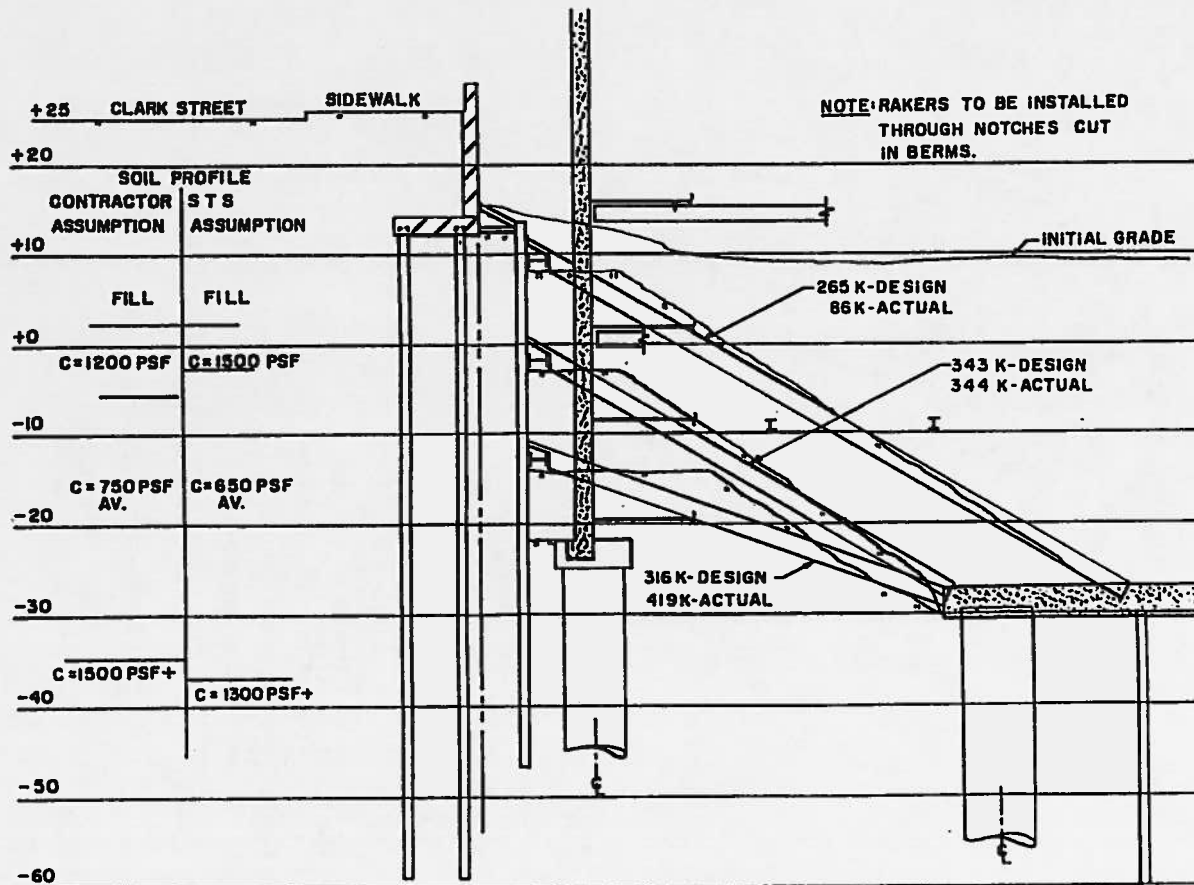


FIGURE 7-RAKER LOADS

OF DRAINS, SHIRTS, AND CODES: LESSONS FROM STUDIES OF FAILURES IN ENGINEERED WORKS

Mark T. Bowers¹

ABSTRACT

The major causes of foundation failure are summarized including general shear failure, erosion and scour, liquefaction, ground collapse, high seepage forces, excessive compression, expansion and heave, ground subsidence, and deterioration of concrete and other founding materials. It is imperative that engineers learn the lessons taught by foundation failures to avoid repeating them. The discussion continues with an examination of the content of the permanent literature concerning case histories of failures. The reasons why the literature contains so few histories are discussed. The difficulties of investigating failures are presented. Case histories from the author's experience are presented to highlight various points. Specific problems that come with expert witness testimony are covered to some extent. An examination of existing codes and their enforcement is considered in light of several cases of failure. The author examines existing sources of information that can be used by consulting engineers and students to explore foundation performance that did not meet expectations. The paper concludes with a proposal for the development of a better mechanism for examination of case histories of failures in both academic and consulting engineering settings.

EXPECTATIONS OF FOUNDATION SUPPORT

Hunt (1) described the purpose of a foundation as being the ability "to transfer the load of a structure to the ground in a manner that provides adequate support for the structure and results in its satisfactory performance." He further indicated that "adequate foundation support requires that there be no danger of ground rupture which would result in collapse or overturning of the structure and that ground compression or heave, which would result in detrimental deflections in the structure, not occur." Bowles (2) points out that "a soil shear failure can result in excessive building distortion and even collapse. Excessive settlements can result in structural damage to a building frame, nuisances such as sticking doors and windows, cracks in tile and plaster, and excessive wear or equipment failure from misalignment resulting from foundation settlements."

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MAJOR CAUSES OF FOUNDATION FAILURE

The major causes of foundation failure were summarized by Hunt (1) in the following manner:

Ground rupture and collapse conditions:

- 1) General shear failure in soils such as soft to stiff clays and organic materials
- 2) Erosion and scour in granular soils from broken water pipes and flowing natural waters
- 3) Liquefaction of fine-grained granular soils during earthquakes
- 4) Ground collapse caused by tunneling, old mines, and limestone cavities
- 5) High seepage forces in rock masses supporting concrete dams

Vertical ground displacement and detrimental structural deflection caused by:

- 1) Excessive compression or consolidation of soils or rock masses
- 2) Expansion and heave from swelling soils or rock and frost penetration in fine-grained soils
- 3) Ground subsidence from lowering of the groundwater table, or excavations for tunnels, retention of adjacent structures, or construction of adjacent structures
- 4) Ground collapse caused by tunneling, old mines, limestone cavities, and saturation of collapsible soils
- 5) Deterioration of concrete or other foundation materials, due to groundwater or other environmental conditions

CASE 1: RESIDENTIAL SETTLEMENT DUE TO A FAILED STORM DRAIN

A thirty-eight year old brick home on poured concrete foundations had developed substantial cracking along the north and east sides of the structure. An addition had been constructed on the north end of the home at some time but damage had not been noticed until recently. A local geotechnical consultant had been engaged and had installed several strain

gauges. I was asked to review the reports to date and to render an opinion. A site reconnaissance was undertaken. At the end of the visit I noted four storm drains in the street in the front of the home, two on each side of the street. I contacted the local sanitation district engineer and determined that the storm sewer did not parallel the street but veered northeast into a drainage swale northeast of the home. In fact, the storm sewer alignment was such that it lay under the north addition to the home, 5 to 6 feet in from the corner of the house! To quote my report, "...it is possible that soil infiltration into the sewer may be occurring and with this, some soil disturbance may be caused. This would require a check by the sanitation personnel." The result was that a television camera was pulled through the storm sewer beneath the home. The sewer was found to be collapsed beneath the home with much soil intrusion. The city closed the sewer, grouted the pipe closed, and monitored the situation in the home for a year. The city also paid all the engineers' fees for my client. Lesson learned...keep your eyes open. Observing the orientation of the storm sewers led to a successful resolution of the problem and at much less expense to the homeowner.

WHAT DO WE MEAN BY "FAILURE"?

Leonards (3) indicated that "the term 'failure' connotes an unacceptable difference between expected and observed performance. Included are rupture or erosion of the ground, unexpected cracking that is unsightly, or reduces load-carrying capacity, or both, movements that impair the function of the structure, and monitoring systems that do not convey intended warnings in time."

CASE 2: A FLOODED BASEMENT WITH AN UNUSUAL CULPRIT

I received a frantic call from a homeowner who indicated that his basement was flooded and he could not figure out why. It should be noted that heavy rains had occurred in the days leading up to the client's call. I went to the address indicated to find a ranch-style home built at the top of a small hill. The home consisted of a partial basement with cinder-block walls that had been paneled over. Indeed, the floor had had about an inch of water on it. The sump pump had a cracked outlet line. Approximately three-fourths of the floor and their belongings had been wet...the water had advanced within a foot of the floor drain. So much for sloping the floor to the drains. I completed a quick site review. A pool was located in the backyard but the water advance on the floor was opposite that from the direction of the pool. I checked the gutters and downspouts. They were brand new and the downspouts led to underground drains. Mulch had been placed around the house. I took a garden hose and checked the underground drains; the owner stood at the street to view any flow through the lines. At one corner flow did not

occur...a blockage was in the buried line. I removed the surrounding mulch, disconnected the downspout, and checked the drain. I took a coat hanger to probe the pipe and found a flannel shirt in the pipe just below the ground line. Apparently the installers of the gutters had placed the shirt in the line to prevent mulch from going down the hole until they had attached the downspout. Unfortunately they forgot the shirt. Further flushing with water showed no further blockage. I went inside the house and with the owner's permission, removed the panelling on the inside wall opposite this drain. Behind the panelling was a massive black stain of fungus, mildew and other growth and a distinct line showing the seepage from the ground line down to the floor. My opinion: The shirt blocked the drain. The runoff from recent rains came down the downspout, backed up above the shirt and spilled out on the ground but below the mulch, thus hidden from view. The seepage then occurred through cracks in the block wall mortar and down the wall face behind the panelling. Once cleared of the shirt and with a repaired sump pump outlet line, no further flooding occurred.

ANALYSIS OF FAILURE

The last case was certainly an interesting and rewarding one for the author yet in many cases it is much more difficult to pinpoint the reasons for a problem. Leonards (3) gave excellent counsel when he wrote:

It is commonly believed that investigation of instabilities offers the ultimate opportunity to gain new insights, to evaluate analytical tools, and to improve the design process. This however, is not always the case, for the following reasons:

- 1) The geometry of the structure, the loading conditions, the soil stratigraphy, and the ground-water regime prior to failure must be inferred from the original site investigation and extrapolation from adjacent ground conditions, from the plans and specifications, and from examination of the post-failure conditions. In many cases, this information is incomplete, and the very fact that adjacent material did not fail implies that conditions in the failure zone may have been different.
- 2) It is always difficult and frequently impossible to reconstruct the sequence of events that led to failure; thus the factors which initiated the instability seldom can be positively identified.
- 3) As the original conditions are irreparably altered (and cannot be fully reconstructed), it is not possible to test different hypotheses of failure on the prototype.

- 4) The time and expense required to investigate all aspects of a failure are often incompatible with the objectives of the parties involved. The aim may be restricted to collecting and interpreting evidence favorable to the parties seeking redress for damages, or the need to reconstruct the facility quickly may take precedence, or politics may interfere with the investigative process.
- 5) There exists no rational methodology for investigating failure. As a result, lessons from past experiences are not cumulative. In each new instance, the investigator is compelled to develop his own strategy to minimize bias in collecting data and in analyzing and interpreting the results.

TRYING TO LEARN FROM PAST PROBLEMS

LePatner and Johnson (4) discuss the dramatic increase in claims against architects and engineers that began in the mid-1950s. They note that it is in the interest of architects and engineers "to learn the lessons such claims teach, in order to avoid repeating them in their own practices." Sound advice, thus why do we continue to see repetition of such problems? LePatner and Johnson continued as regards learning from past problems:

Unfortunately, this is far easier said than done because of a natural reluctance of professionals to publicize their difficulties. As a result, a designer has the opportunity to learn of problems, and the lessons which derive from the consideration of these problems, only from those few cases which attain sufficient notoriety to be published in the media or in trade journals, or where the designer, or friends and associates, are personally involved. Moreover, the quality of such information tends to be poor--accounts in the media are often incomplete and, at times, inaccurate; word-of-mouth accounts are often little more than rumor.

CASE 3: A HOME DAMAGED IN A SLIDE-PRONE HILLSIDE

I was requested to examine a small residence on a steep street along a slope in Cincinnati. The home consisted of wood frame construction over a poured concrete basement foundation. The house was built in 1963 and abandoned in 1976. My client had purchased the home in 1989 under speculation to refurbish it into low-income rental property. To the rear of the house is a slope extending some 325 feet uphill to a major arterial street. In examining this slope with an engineering geologist colleague, we found at least seven scarp faces ranging in height from a few inches to upwards of three feet in vertical offset. Property line fences along the back of the houses were found to be buckled and tilted. A six-inch wide crack was found in the ground just 25 feet behind the house. No toe of the earth movement was

clearly visible but a mass of earth had built up behind the basement wall of the structure. Upon gaining access to the house I examined the basement walls. At the rear of the house the top of the wall had tilted into the basement by about 31 inches. My client asked me, "Can you fix it?" I did some additional geological studies, determining the slope to be in the Kope Formation (a potentially treacherous one in the Greater Cincinnati Area as concerns slope stability) and wrote my report. I also determined the neighboring house just downhill from this site was taken down in 1978 after having experienced distress over a period of several years. In the end, my client lost his investment as the remedial costs were exorbitant and the potential for initiating a slope failure by excavating behind the house far outweighed any positive value from repair of this house. The house was taken down. Lesson learned: When considering the purchase of a property, engage the services of an engineer before signing the purchase agreement. This client did not learn from the fact that the adjacent home was taken down...he never asked why. It did not take long, however, to find an article in the newspaper which discussed the neighbor's plight and the reasons for the demise of the adjacent home.

BUILDING CODES AND THE EFFECT OF THEIR NEGLECT

Article 12 (Chapter 4101:2-12) of the Ohio Basic Building Code (5) concerns foundation systems and retaining walls. The general section which prefaces this section of the Code states its goal:

The provisions of this article shall control the foundation design and construction of all buildings and structures hereafter erected to insure adequate strength of all parts thereof for the safe support of all superimposed live and special loads, in addition to their own dead load, without exceeding the allowable stresses or design capabilities.

Section 1223 specifically deals with the design of retaining walls. Here it is stated:

Retaining walls shall be designed to resist the pressure of the retained material, including both dead and live load surcharges to which such walls are subjected, and to insure stability against overturning, sliding, excessive foundation pressure and water uplift.

CASE 4: FAILURE OF A NEWLY-CONSTRUCTED STONE RETAINING WALL

I was requested to investigate the reasons why a section of a nine-month old six-foot high stone retaining wall had failed in a residential neighborhood. The retaining wall consisted of trimmed limestone in mortar with a near-vertical exposed face and a sloped backface. The wall had been constructed as a gravity-

type wall. I spent about 2.5 hours at the site and left with a series of disturbing notes which showed that the construction appeared in many respects to violate current standards of practice (6,7).

I found the following items in my investigation of this failed wall:

- 1) Since the maximum exposed height of the wall exceeded 4'6", the bottom of the wall footing was to be placed at least 2'6" below the finish grade at the face of the wall. Since the gravity wall consisted of stone, the wall was supposed to rest on a concrete footing not less than 12" in thickness. Not only was there no concrete footing associated with this wall, but the base of the wall was within inches (less than four inches based on several trial excavations) of the ground surface. The stones were placed directly on the soil.
- 2) A minimum of 12" of clean, free-draining granular backfill is required by code to be placed behind all walls and is to extend from an elevation coinciding with the finished grade at the face of the wall to within 8" of the top of the wall. No such drainage material existed behind this wall...it was simply clay and rubble against the stone.
- 3) The stone is required to be laid up in Portland cement mortar. I found that only the front 8" to 9" of the exposed face of the wall had any mortar between the stones. The rest of the wall consisted of loose stones, brick, and chunks of concrete. The mortar was so weak that I was able to dismantle the wall at the broken cross section by hand without the assistance of tools.
- 4) For the maximum exposed height of 6'1", and not accounting for the footing at this time, the wall should have had a minimum base thickness of 3'0" if drainage had been provided. The Code indicates that this dimension is to be doubled in those cases where nonporous (clay) backfill is used. I found the wall to have a thickness of 2'1" at its base.
- 5) Weep holes did exist according to Code dimensions and spacing though they terminated behind the wall in clay backfill.
- 6) The backfill placed behind the wall was apparently placed with little or no compaction. I found settlement of the surface, erosion, and much soil loss behind the wall. Some of the holes were large enough for a man to reach his arm in and others were at least 2.5' deep.

Other items could be mentioned but the point is clear. Why do we

write Codes if they are not going to be enforced? Each of us could tell similar stories. In this case the homeowner was emotionally torn as he had only moved in the home two months prior to this failure. He had two small children who played in the shadow of the wall in their front yard. The damage could have been much worse, yet it was totally unnecessary if the plans for the wall and its construction had been reviewed for compliance to Code.

THE PERILS OF PEER REVIEW AND EXPERT WITNESS TESTIMONY

Speaking of the engineer as an expert witness, Bockrath (8) counseled:

When in the courtroom, a witness should conduct herself with dignity and confidence. It is clearly desirable for an expert witness to give the impression that she knows her business, that she is scrupulously honest, and that she is confident as to the accuracy of her testimony. She should be thoroughly competent and able to defend her opinions. ...she should not attempt to be an expert witness for a plaintiff (or defendant, as the case may be) whose general position--at least on the matter about which expert testimony is desired--she cannot wholeheartedly endorse and support.

...the witness should not allow opposing counsel to wrest control of the situation from him or demoralize him to the extent that he will make some untrue, inconsistent, or self-contradictory statement.

A prospective expert witness should not be reticent about appearing in court and expressing himself before a group of spectators. He should be mentally prepared for his task and able to think clearly and speak effectively under difficult circumstances.

This last sentence deserves repetition. Serving as an expert witness is a "task" and the circumstances surrounding the preparation and giving of testimony can indeed be "difficult." The lawyers for the opposition are doing their best to win their case, too. Sometimes, major errors get set aside in a fight over a detail. The case is won or lost on the abilities to maintain a focus on such issues. Principles that the engineer strongly believes in are often ignored and set aside by the attorneys in these squabbles.

CASE 5: SEWERS, SETTLEMENT, AND VIDEOTAPES

I was requested by a homeowner to review a lengthy set of documents relating to a settlement problem in his home. At the same time I was asked to conduct a condition survey of the damage

his home had sustained. I was requested to "render an opinion as to why this damage had occurred." I reviewed all the information made available to me and conducted my site reconnaissance. The principal damage to the structure was along one side of the house but concentrated near one corner. The home had a basement. Two sewer lines are located immediately adjacent to the house exterior face and within a few feet below the footing base. The lines consist of an 8" sanitary sewer and a 24" storm sewer. Previous records showed many requests from the homeowner to the municipal offices to investigate the settlement problem. The sewer lines had been probed and a man was sent to crawl through the storm sewer. He found it damaged and was unable to complete the circuit. I suggested that television cameras be used to obtain a better picture of the condition of the sewers, believing that soil erosion was occurring beneath the footing into the sewers and the soil was being swept away. I wrote my report and heard nothing for about 2 years. At that time I received a summons to appear for a deposition of my testimony. After that, several trial dates were established but cancelled for various reasons. A second deposition date for my testimony was scheduled but cancelled at the last hour.

Nearly four years after my initial visit to the site I was called to testify in the jury trial. I had continued to follow the case, gathering information both before and after my initial deposition, and had received copies of the videotapes of the sewer lines in their as-was condition. The 8" sanitary sewer had been relined with a plastic product. The 24" storm sewer was videotaped and showed extensive damage in the form of cracking and missing sections right beneath a sinkhole that had appeared in the surface immediately adjacent to the most damaged corner of the home. I was not allowed to admit this information as it came to light after my initial report written nearly four years previously. I was not allowed to discuss any information garnered after my first deposition. My testimony was disregarded. I received the second set of tapes from my client's lawyer with instructions to review them in light of my original suggestion that the lines be videotaped.

My client has not received due process nor justice. One sewer line had been repaired but the line which clearly shows damage and soil infiltration has not been repaired and the municipality disclaims any responsibility for damages to my client's home. To date the storm sewer remains unrepaired despite owner requests and site visits to inspect the sewer dating back to 1977.

Challenging, frustrating, potentially demoralizing, and emotionally unsettling. These words just begin to describe how the engineer can feel in the midst of the cross-examination and at trial's end. A review of Bockrath's chapter entitled "The Engineer As an Expert Witness" would serve all of us well.

AVAILABLE RESOURCES ON ANALYSIS OF FAILURE

Recent books by Piesold (9) and Petroski (10,11) add important information in our understanding of engineering by examination of failure. Piesold, for example, gives a synopsis of his text in the following:

[He]...describes specific projects or events...where design and construction would undoubtedly have derived great benefit from an earlier similar experience. [He] ...covers a wide range of topics, including design, specifications, construction, economics, legal aspects, environmental, and many technical matters which recur in the life of nearly every practicing engineer, often causing difficulties to a greater or lesser degree. [The book] ...is intended to bring to the attention of students, engineers, architects, quantity surveyors and related professionals what can go wrong, what appears to continue to go wrong from generation to generation, and how project analysis can contribute towards the avoidance of serious error. [It]...is dedicated to the avoidance of error, and shows how adverse consequences can, in most circumstances, be avoided by a process of project design and/or construction analysis before implementation.

ASFE, Professional Firms Practicing in the Geosciences, offers much to the geotechnical engineer in the form of case histories, counsel, and a number of bulletins and guides. A recent ASFE publication is entitled EXPERT: A Guide to Service As a Forensic Professional and Expert Witness.

The American Society of Civil Engineers (ASCE) publishes a most interesting journal entitled Journal of Performance of Constructed Facilities. This journal is now in its tenth year of publication. The February issue contains an annotated bibliography of forensic engineering developed by Nicastro (12).

WHERE DO WE GO FROM HERE?

Petroski (13) notes that "the proper use of the concepts and realities of failure is essential for successful design practice, which involves proper engineering judgment. Among the most valuable sources of good design judgment are case studies of how great engineers designed against failure."

Baer (14) asks if civil engineering graduates are adequately informed on failure. Baer states that "to competently teach a course on construction failures, an instructor must have adequate experience with fabricating processes, erection procedures, mixtures of compounds, and construction tolerances and be familiar with principles of constructability." He concludes by saying that "it is through the understanding of failures that we

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learn the limits of new theories and materials. The ideal situation for students is for the curriculum to include at least one course on failures. If that is not possible, failure case histories should be incorporated into senior concrete, steel, and foundation design courses."

Rendon-Herrero (15) reports on a survey sent to 175 civil engineering department heads to assess the status of and inclusion of failure information in civil engineering programs at schools in the United States. The results of that 1989 survey identified 11 courses as specifically dealing with the failure of engineered facilities. Bosela (16) reports on those identified courses and made the following recommendations:

It is the opinion of the TCFE [Technical Council on Forensic Engineering] Education Committee that students must be continually reminded of the lessons learned from failures of engineered facilities. This must be done through a variety of methods throughout their education, including:

- 1) Introductory courses
- 2) Incorporating failure case studies into design and analysis courses at the undergraduate level
- 3) Graduate courses specifically dealing with failure case studies
- 4) Short courses and seminars
- 5) Increased availability of case study information, utilizing resources such as the ASCE Journal of Performance of Constructed Facilities.

CONCLUSION

This paper has provided information on the major causes of foundation failures. It has been shown that there is a natural reluctance among engineers to publish information concerning their failures. The difficulties involved in investigating failures have been reviewed. Case histories of failures from the author's consulting background have been presented to illustrate various points of discussion. Sources of information concerning forensic engineering practice, litigation and expert testimony, failure mechanism theory, and performance of constructed facilities have been identified. Finally, ideas concerning how we can better inform students of failures in engineered works have been presented. It is this author's hope that this summary will serve as a springboard to other professors in how we teach our students and prepare them for their professional careers. It is also hoped that this paper will serve as impetus to consulting firms to involve their employees in continued education and to share their case histories through appropriate media.

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THREE CASE HISTORIES OF SETTLEMENT FAILURE

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INTRODUCTION

Case histories of foundation failure are interesting because they provide insight into real life events that were not supposed to happen. Settlement failures, while not as dramatic as bearing capacity, seepage, or slope failures, can still cause heartburn because they are expensive to repair. After all, the geotechnical engineering process is supposed to provide comfort to the owner and assure him that his structure will be properly supported; therefore, it is important to find out why settlement failures occur and what can be done to prevent them.

This paper examines three case histories where unanticipated settlement failures occurred in very different geological environments and under seeming unrelated circumstances. Individually, they may appear unrelated but upon closer examination, there are some denominators that are common to all of them.

These case histories have also been simplified so that they can be presented in a paper short enough for an interested reader to tolerate. This was done intentionally to keep the paper focused on the subject matter. However it should be noted that some of these case histories, the last one in particular, are far more complex than what could be described in a couple of pages.

As these cases develop, the reader may wish to contemplate how he would pursue the problem, beginning with the limited information initially available. The realities involved in projects like this usually involve clients who have limited budgets available and various third parties who were involved at the time of design or construction, but just cannot seem to remember very much about the job on the day you call. Construction notes, files, and photographs disappear from the face of the earth, especially when there are lawyers lurking about. Finding a cause of the problem usually starts with bits and pieces of known information. By working systematically through (or around) the unknown, eventually, like a crossword puzzle, the answers begin to emerge. At that point, the next questions are what to do to fix it, how much it will cost, and who will pick up the tab.

CASE HISTORY NO. 1 - TILTING SAND SILOS

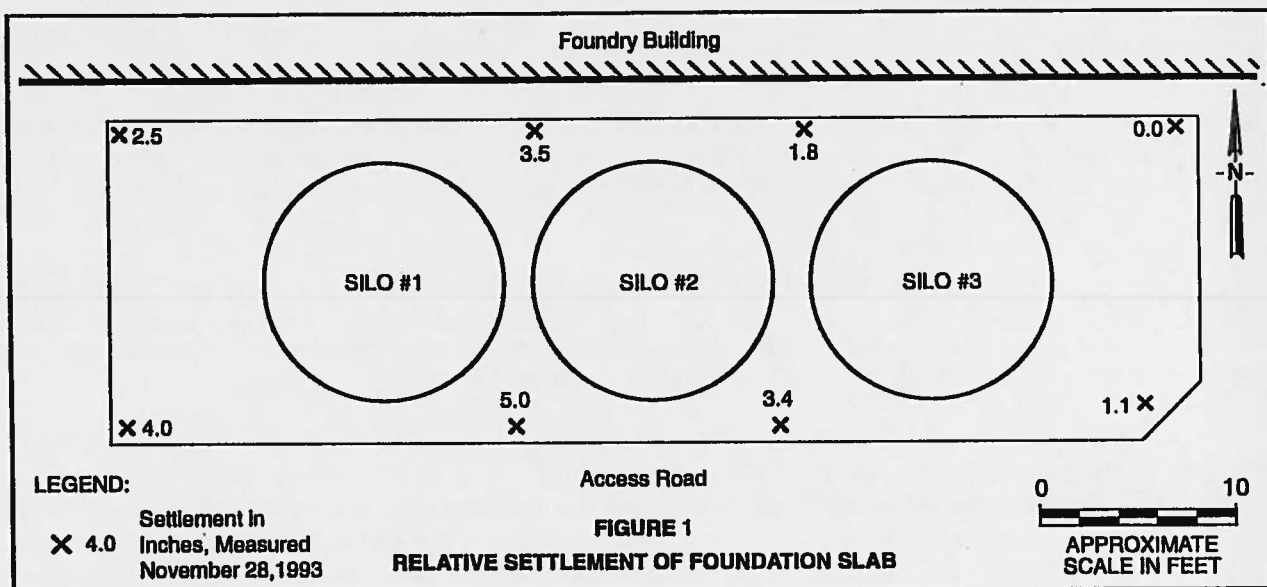
This case began as a fast-track project to build three sand silos at a foundry in southern Ohio. The site is located on a flood plain of the Ohio River. Subsurface conditions were known to include:

- Loose surficial fill
- River alluvium (silty clay)
- Glacial outwash (sand and gravel)
- Bedrock (sandstone and shale)

Bedrock was expected to be deep, so a foundation system was established consisting of a common, reinforced concrete slab founded on soil. However, within a year after being completed, the silos tilted. Had all of them tilted uniformly, the movement may have gone unnoticed, but two of the silos tilted to the east and one tilted to the west; Silos No. 1 and No. 2 came together at the top where they rested against one another. This eventually attracted the attention of the owner, who called for an investigation.

The immediate concern was to establish if movement was continuing and when or if the situation could get critical. The owner arranged for a survey, which indicated that the top of the common foundation slab had settled up to 5 inches relative to its northeast corner (see Figure 1). The absolute settlement is not known as no elevation data were recorded when the foundation slab was constructed. Tilting of the silos, as calculated by the surveyor, is shown in Figure 2.

The failure investigation began by developing a chronology of events and an analysis of the information that affected design. As shown in Figures 1 and 2, all three silos shared a common concrete slab foundation, 17 feet wide by 56 feet long, and 3 feet thick. Each silo was 12 feet in diameter, 71 feet high, and had a total sustained live load of 295 tons. The approximate date each silo was initially loaded is shown in Figure 2. The common concrete foundation slab was founded on compacted structural fill 8 feet deep, constructed over stiff silty clay. So what went wrong?



Geotechnical information concerning the project was provided by the owner, who had hired the drilling contractor directly and specified that he drill four borings to a depth of 15 feet (B-1 through B-4). The preconstruction drilling pattern is shown in Figure 3. The preconstruction subsurface profile, as interpreted from the borings, is shown in Figure 4, indicating about 8 feet of soft fill, then stiff silty clay to about 13 feet, and medium to dense sand and gravel below that to the bottom of each hole. During the design process, the owner provided these logs to Geotechnical Consulting Firm A, who in turn sent them to Geotechnical Consulting Firm B, who performed the analysis. Firm B tested the soil samples provided and offered several recommendations, which are summarized as follows:

- Remove all loose fill to a depth of 8 feet and discard the material excavated. The width of the excavation should be 1.3 times the footing width.
- Replace excavated soil with appropriate backfill material, such as a well-graded mixture of durable sand and gravel, compacted to 98 percent of the Standard Proctor Value.
- The compacted structural fill may be designed to support a working load of 3,000 pounds per square feet (psf) in shallow spread footings.

Weight Data per Silo
 Dead Weight of Silo = 18 Tons
 Product (Live) Load = 277 Tons
 295 Tons

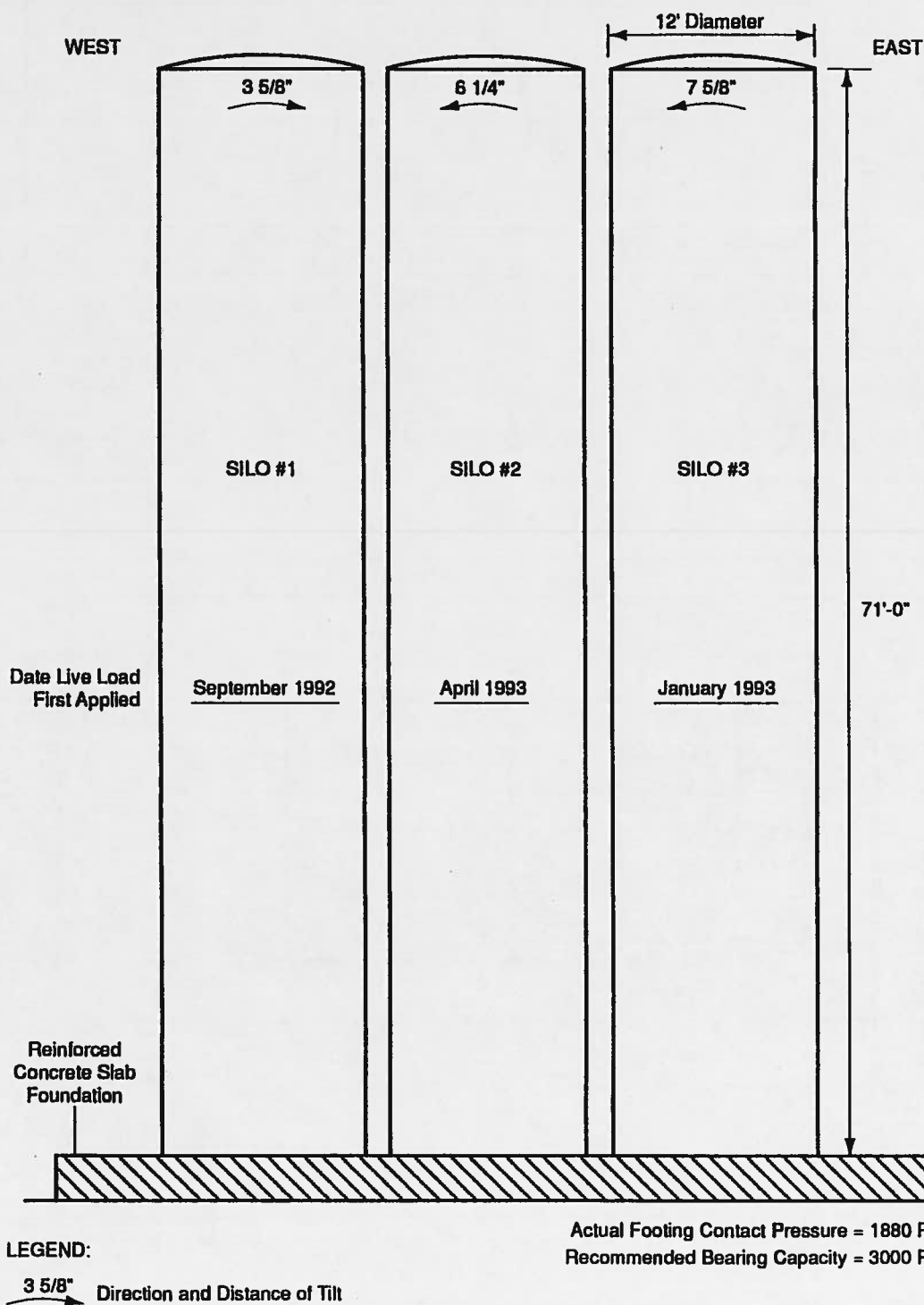
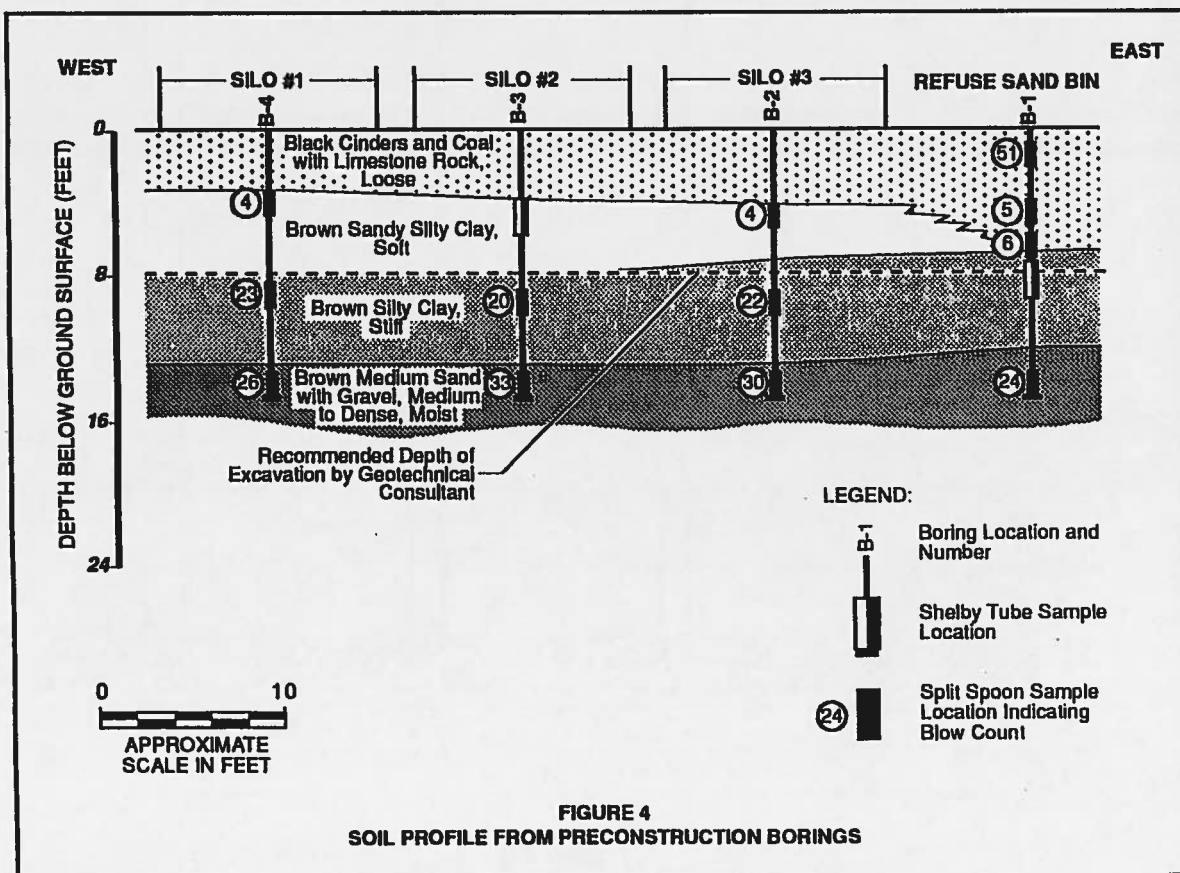
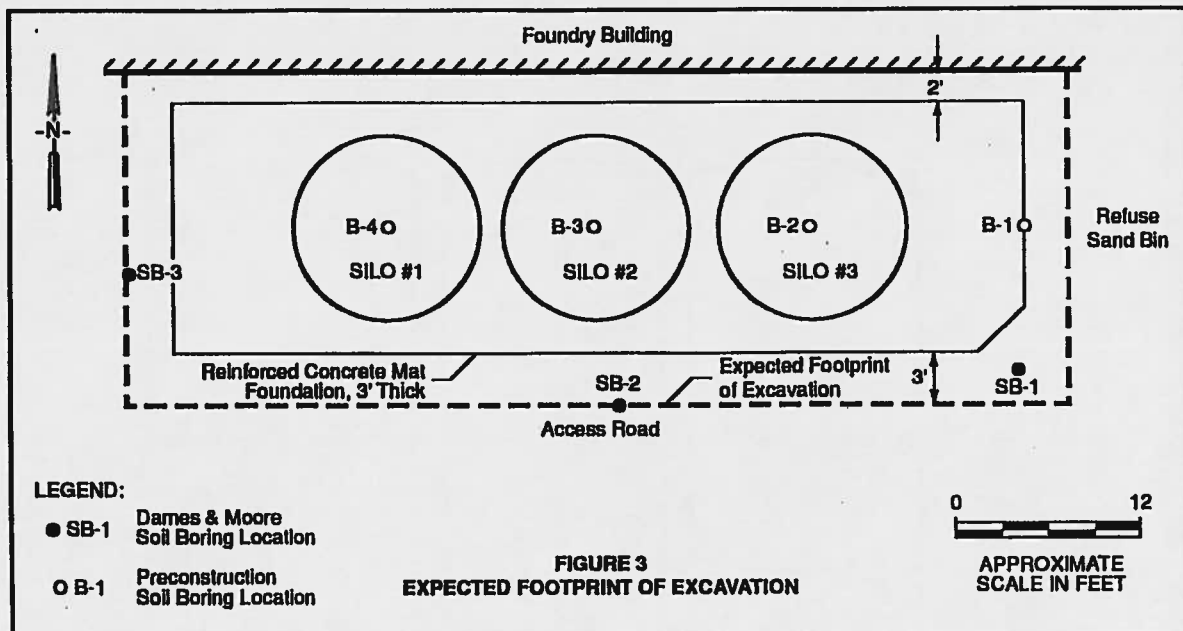


FIGURE 2
 VERTICAL PROFILE OF SAND SILOS



The only other record that the owner could provide was a report by the geotechnical inspection firm, indicating that silty clay was used as the structural fill and that the only compaction inspection report available was from 4.0 to 1.5 feet below grade. All of the in-place density test results showed that the compaction criteria were achieved, but --- who authorized the use of silty clay instead of sand and gravel, and what kind of soil was used between 8.0 and 4.0 feet? The geotechnical inspection firm was contacted, along with the on-site inspector (who is currently employed elsewhere). The contractor's representative (who is also employed elsewhere) was contacted as well. No other records seemed to exist and nobody could remember much, except that the hole appeared to be about 8 feet deep. Due to physical constraints at the site, the recommended excavation footprint of 1.3 times the footing width could not have been achieved. The excavation footprint may actually have been less than the footprint of the foundation because the excavation wall was probably sloped inward to protect the continuous footing of the adjacent building wall (see Figure 3). It is not likely that Geotechnical Firms A or B were aware of these constraints.

Subsurface conditions were investigated directly by Dames & Moore. Consideration was given to drilling directly through the concrete foundation slab in order to examine the structural compacted fill beneath it. Due to the many physical constraints (which are not portrayed in Figure 3), it was concluded that drilling into the concrete foundation slab would be very difficult and extremely expensive. Drilling an angle hole beneath the slab was also considered, but it was decided to drill vertical holes initially due to limitations on schedule and budget.

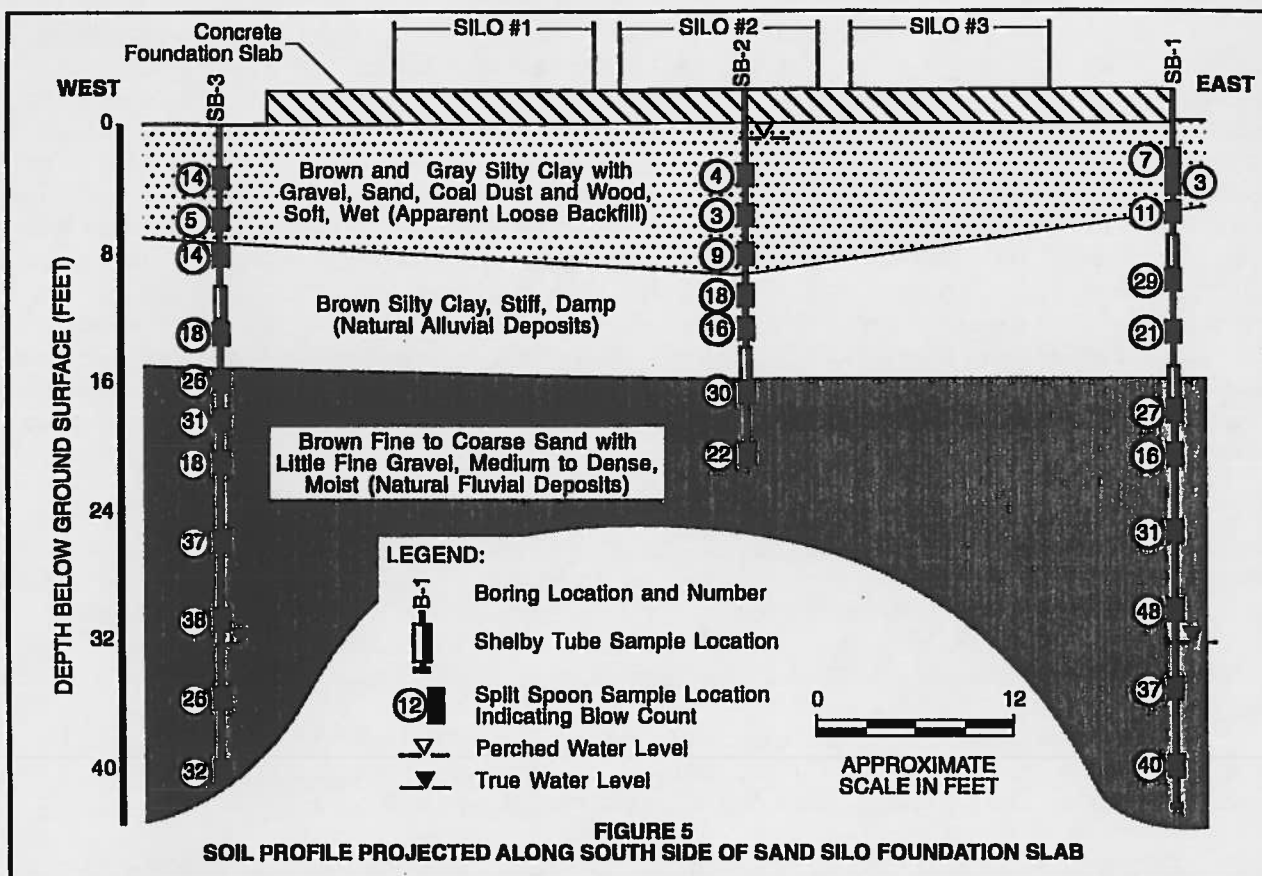
The decision was made to drill three vertical holes around the concrete foundation slab, and to carry them deep enough to look for any zones of soft, compressible soil that may lie within its zone of influence. These borings (SB-1 through SB-3) are shown in Figure 3.

The subsurface profile developed from the post-construction borings showed that no soft material was present beneath the stiff silty clay to a depth of 40 feet (see Figure 5). Most of the sand and gravel zone is classified as dense, except for a zone around 20 feet deep, which is classified as medium, based on the blow count values. The water table in the sand and gravel zone was at 32 feet. Since no soft strata in the deep part of the soil profile were identified, the focus of investigation turned back to the shallow zone.

The post-construction borings did reveal the presence of soft fill soil to a depth of 9.5 feet in Boring SB-2, just south of the middle of the concrete foundation slab. This is deeper than the recommended depth for removing the soft soil and replacing it with structural fill, as illustrated in Figure 6. It was also documented that the shallow fill soil surrounding the concrete foundation slab was saturated with ground water perched on the underlying silty clay layer. The silty clay layer itself was also examined and laboratory consolidation tests were performed. It was concluded that a small amount of settlement could be attributed to the silty clay layer, but nowhere near the magnitude of what was observed on top of the concrete foundation slab.

At this point, the smoking gun pointed back at construction of the compacted structural fill, below a depth of 4 feet. While those who witnessed this part of the job had very poor recall of what actually happened, it was our conclusion that the job must have been very difficult and the probability of having layers of soft, compressible fill material left below the structural fill seems to be a virtual certainty. Several observations support this conclusion:

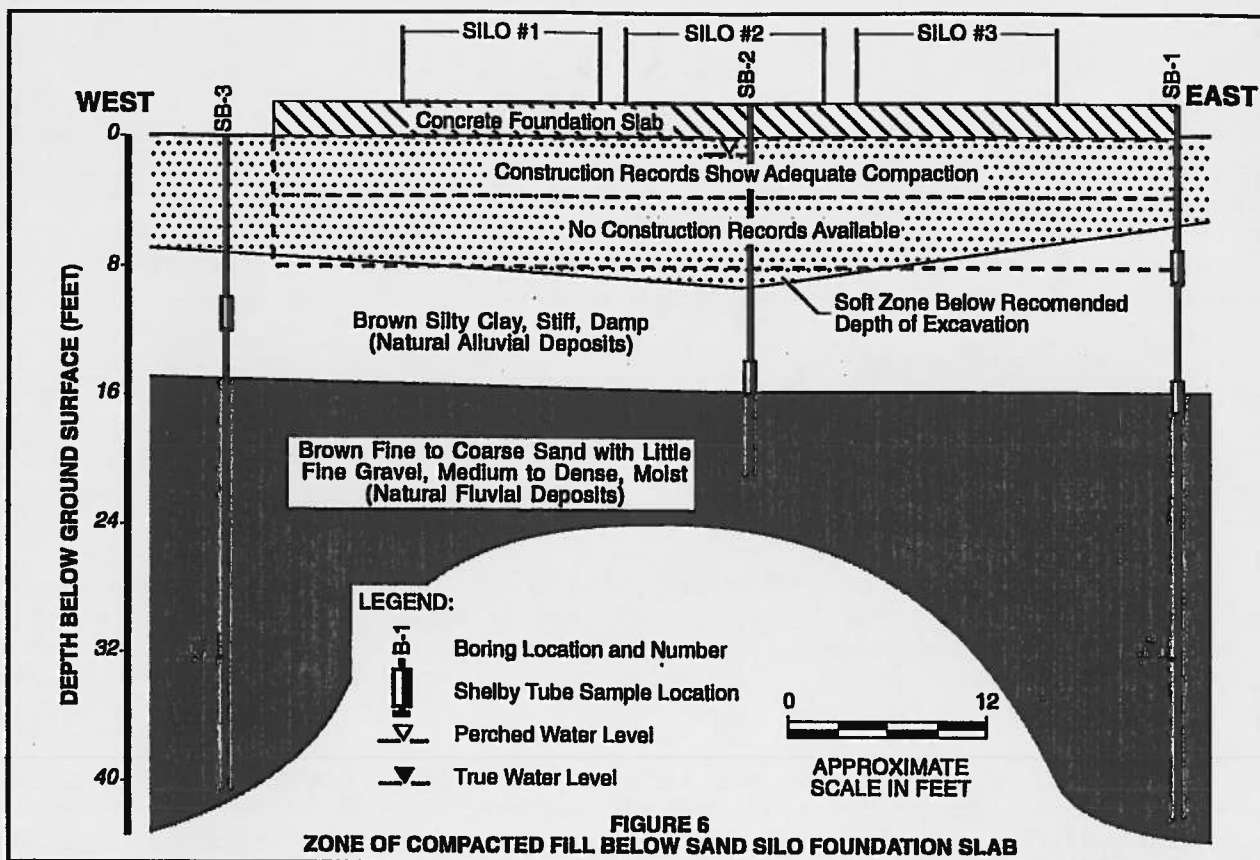
- Post-construction borings showed that soft, compressible fill soil was encountered at depths greater than 8 feet immediately to the south of the foundation slab.
- Physical constraints of the site did not allow the footprint of the excavation to be any larger than the footprint of the concrete foundation slab. This was further confirmed by the absence of compacted structural fill in any of the post-construction borings, which were drilled in close proximity to the concrete foundation slab.



- No inspection records existed documenting the depth or condition of the excavation bottom, prior to placement of the compacted structural fill.
- No construction records existed for the compacted structural fill placed below a depth of 4 feet. Records also indicated that compaction test data were not even available prior to the date that compacted structural fill was placed from 4.0 to 1.5 feet.
- The indigenous fill soil is saturated with perched ground water, making it very unstable in the walls of an unsupported excavation. Sloughing problems were probably severe at the time the excavation was made. Nobody who witnessed the construction job had any recall of shoring being used.

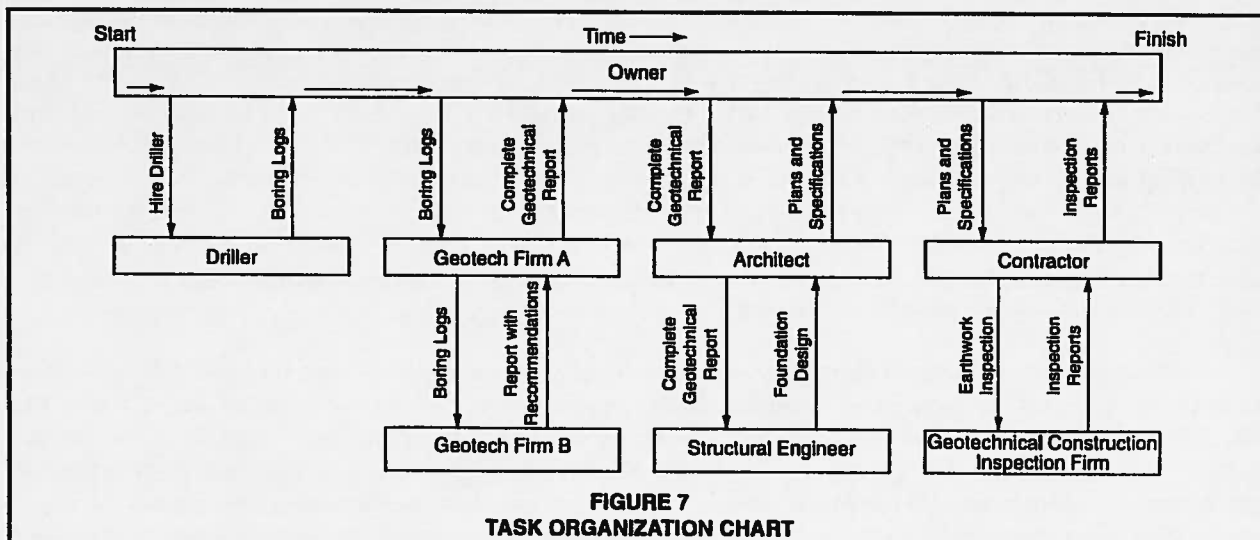
Subsequent elevation surveys conducted on behalf of the owner indicated that movement of the concrete foundation slab had ceased. This was a further indication that thin zones of wet, compressible soil were probably left between the compacted structural fill and the stratum of stiff silty clay beneath it.

Although the cause of the failure seemed to be readily apparent, placing blame for the problem was more difficult. This was a fast-track project so the owner's representative managed and directed each phase of the project. This is graphically illustrated in Figure 7. The biggest problem is that there was no geotechnical engineering firm placed in direct charge of the job from its inception, through design and construction. Each task was parceled out by the owner so there was no responsible professional available when the most critical decisions were made or construction operations took place.



Additional elevation surveys were recommended to demonstrate that no further movement was taking place. The concrete foundation slab could be releveled by a grouting technique known as "slab jacking," and the silos could be replumbed using shims. However, the owner elected not to incur this cost or the consequences of taking the silos out of service for the time needed to make the repairs.

The entire concept of using a shallow foundation system was probably inappropriate for this application. The limited data provided to the geotechnical consulting firms and the design firms by the owner made it infeasible to provide an appropriate design. However, the project may have been inadvertently successful if the contractor had been lucky enough to encounter a little less loose fill and/or dryer conditions at the time of the excavation.



CASE HISTORY NO. 2 - SINKING STAMPING PRESSES

A boom in the manufacturing of automobiles led to a demand for stamped metal parts during the mid 1980s. A parts manufacturing company in central Tennessee decided to increase its production capacity by making an addition to its existing facility and adding a line of stamping presses. The building addition was 250 feet long and 100 feet wide, with a poured concrete floor. The exterior walls were supported on continuous strip footings founded on silty clay soil. The general subsurface profile at the site is described as follows:

- Silty clay residual soil
- Limestone bedrock

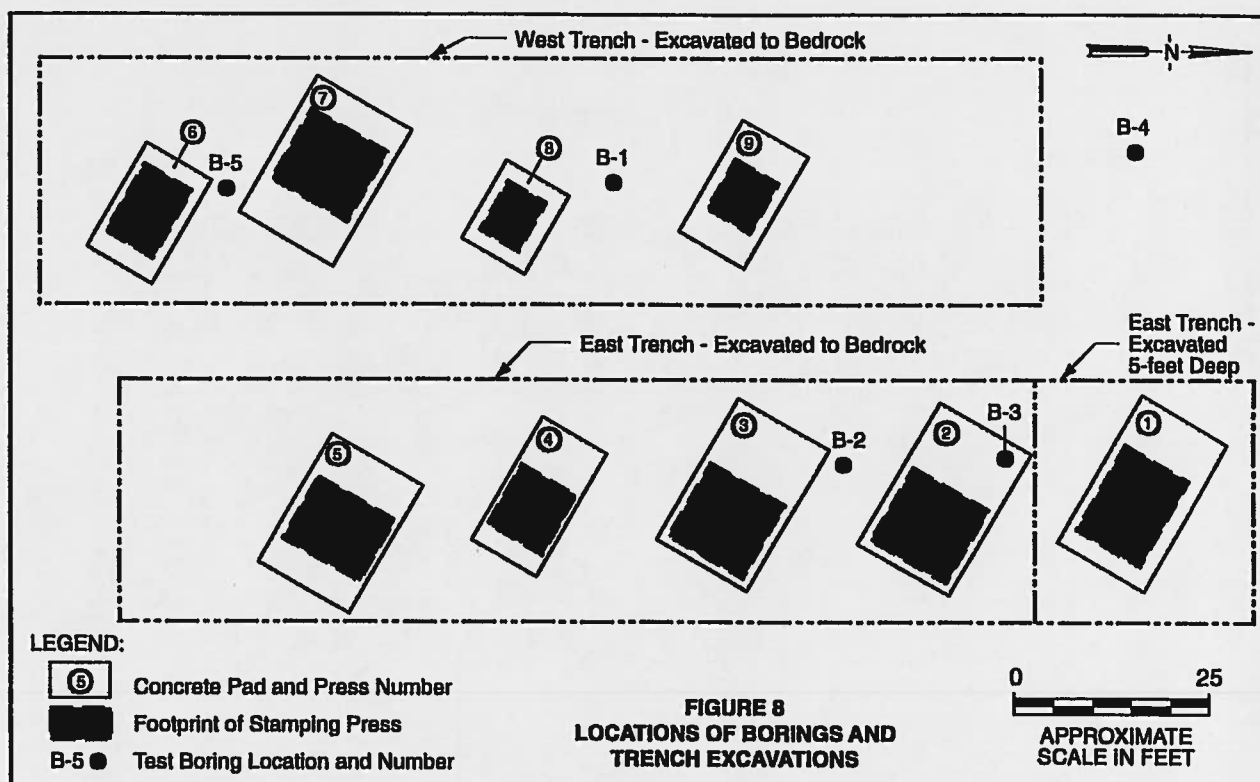
Each of the stamping presses was set on an individual foundation of reinforced concrete, 2-foot thick. The nine stamping presses range in size from 150 tons (gross weight) to 500 tons, with most in the 300-ton and 400-ton range.

Within a few months after production began in the addition area, the stamping press foundations began to sink, ultimately settling 2 to 4 inches on one side (generally toward the east). Two of the presses not only settled, but tilted so badly that the western end of the foundation slabs protruded above the floor level. In all, seven of the nine stamping press foundations experienced some degree of settlement that required repair.

The failure investigation began with a review of the original geotechnical report prepared for the project by a local geotechnical consulting firm. Soil borings indicated 11 to 13 feet of medium to stiff silty clay overlying a flat, hard bedrock surface. No mention was made of karst activity in the report and no karst features were evident at the site. One soil sample in one boring was associated with a blow count of three, indicating the local presence of very soft, wet soil. The geotechnical engineer recognized this as a possible problem and recommended that zones of soft soil be removed, if encountered during site preparation. If excavated areas were to occur under the machine foundations, the geotechnical consultant recommended that the excavation be filled with crushed stone compacted to at least 98 percent of its Standard Proctor maximum dry density. The consultant also recommended that all soil removal and backfilling operations be monitored on a full-time basis by an experienced technician responsible to the engineer and the owner. Unfortunately, the design/build contractor declined to have an inspector present onsite during these critical operations.

The failure investigation next led to interviews with the design/build contractor. It was discovered that two trenches were excavated as a consequence of encountering soft soil during site preparation. Each trench was 30 feet wide and encompassed the stamping press foundation areas as shown in Figure 8. The western trench was 135 feet long and was excavated to bedrock, 11 to 12 feet deep. The eastern trench was 145 feet long; most of it was excavated to bedrock, except for 30 feet on the northern end, which was only excavated to a depth of 5 feet. The trenches were backfilled with coarse crushed limestone, up to 3 inches in maximum dimension. The contractor claimed to have used a vibratory roller and to have compacted the granular backfill as specified in the geotechnical report—but, there were no compaction test results or inspection records available. However, photographs did show the presence of a vibratory roller working on the structural backfill, just as the contractor had claimed!

This turned into one of those puzzles you think about at night. Everything that should have been done appeared to have been done correctly, yet the stamping presses continued to settle and tilt. The available literature was of little help because not much is written in textbooks about impact loading under machine foundations. The manufacturer of the presses provided some information on dynamic loading, but this was of little help because the sum of the static plus dynamic loads was well within the safe bearing capacity values calculated by the geotechnical engineer, even for the eccentric loads that were placed on the footings.

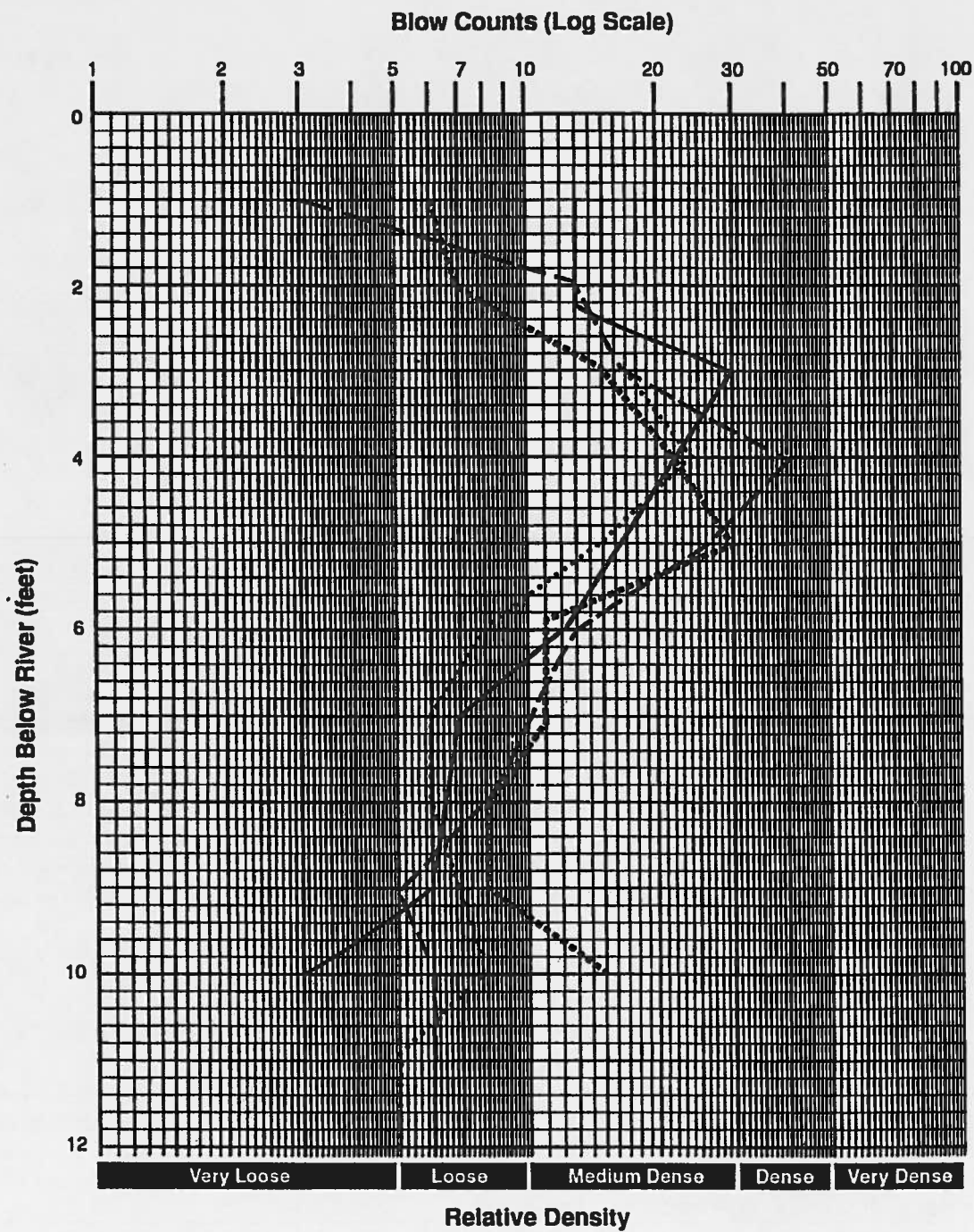


Haunted by the fact that no compaction records existed for granular backfill beneath the machine foundations, it became apparent that the most likely reason for the problem was that the granular backfill had not been properly compacted. A program to investigate this possibility was developed using the Standard Penetration Test (SPT) to determine relative density from the blow count data. There was no practical method for collecting soil samples from the sampling tool because pieces of the crushed rock were up to 3 inches long. So the objective was to investigate relative density of the backfill, and to collect soil samples from the auger. Bedrock would be indicated by auger refusal and the water level indicated by water depth inside the augers.

Five test borings were drilled, two in the east trench (B-1 and B-5), two in the west trench (B-2 and B-3), and one outside of the trench area (B-4), as shown in Figure 8.

All four of the test borings drilled in the excavated and backfilled trenches showed a dramatic decrease in blow count with depth. The bottom 3 to 5 feet of soil was loose, with relative density increasing towards the surface. Except for one boring (B-3) that was drilled directly through a machine foundation, the soil became looser near the surface. When the drill holes were started, gaps several inches thick were observed under the concrete floor, especially adjacent to the machine foundations. It soon became apparent that the damage due to settlement could affect the entire floor slab, as well as the machine foundations.

Blow count profiles shown in Figure 9 proved that the granular backfill was never compacted, except for perhaps its surface before the machine foundations were poured. At this point, the design/build contractor "fessed up" and took responsibility for repairing the failure. The stamping presses could not be operated out of plumb for very long, and it was clear that the subgrade was not dense enough to sustain more impact loading without inducing additional settlement. The focus then shifted on how to permanently repair the problem at a minimum cost.

**LEGEND:**

———— Boring B-1
 - - - - Boring B-2

..... Boring B-3
 Boring B-5

FIGURE 9
STP BLOW COUNTS
IN TEST BORINGS

Several treatment modalities were considered. The design/build contractor favored equipping the stamping presses with shock absorbing press mounts to reduce impact loading to the underlying soil. Other ideas included supporting the machine foundations on piers or minipiles, or the use of portland cement grout. Our concerns extended not only to the machine foundations and the floor slabs, but to the interior spread footings, which were also founded on the loose granular backfill. If the backfill was subjected to impact loading in the future, even at low levels or if it was subjected to seismic forces from an earthquake, would the interior columns also settle? From the owner's viewpoint, the corrective action had to include a solution that strengthened the entire mass affected by the loose granular backfill.

The solution that emerged was to inject Class C fly ash in the form of a grout slurry. The mixture used was 1.5 volumes of Class C fly ash to one volume of water. Samples of grout were tested and showed a laboratory unconfined compressive strength greater than 600 psi after 28 days. Both direct observation in test pits and test borings showed that the fly ash effectively filled voids in the crushed limestone backfill. Blow counts from test borings drilled after remediation indicated that the grouted crushed stone was very dense. Since the machine foundations have been replaced and the stamping presses have been placed back in service, no additional settlement has been observed.

CASE HISTORY NO. 3 - COLUMN SETTLEMENT IN A HIGH RISE BUILDING

This case history involves a Federal Courthouse and Office Building in Philadelphia, Pennsylvania. These buildings occupy an entire city block. The 10-story, reinforced concrete-framed Federal Office Building is located over the northern third of the site. The steel-framed Federal Courthouse was intended to include a tower 22 stories in height; it is located in the southern portion of the site. Below grade, there is a common basement approximately 320 feet by 630 feet in plan dimensions. The basement is approximately 10 to 20 feet below the level of the adjacent streets.

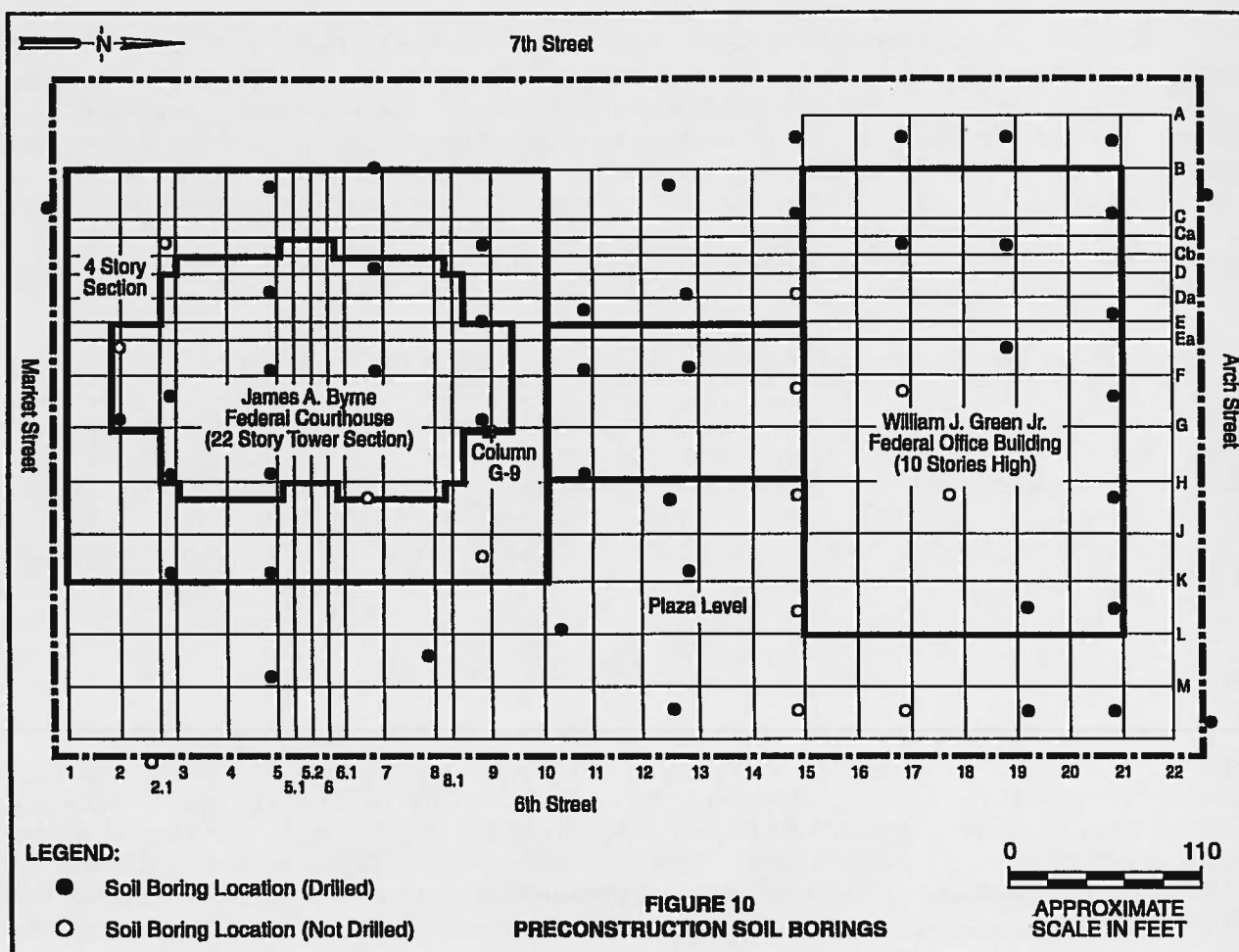
Subsurface conditions prior to construction included the following units:

- Miscellaneous fill soil
- Alluvial soil
- Weathered bedrock (soft mica schist)
- Sound bedrock (hard mica schist)

The bedrock formation, known locally as the Wissahickon Schist, is intruded by veins of pegmatite and coarse granite. The upper portion of the schist has been altered by chemical weathering. The weathering profile is quite complex and variable throughout the area.

The site was originally investigated by a local geotechnical consulting firm to develop foundation design criteria for the proposed buildings. Fifty-nine borings were originally planned but 47 were actually drilled. The locations of these borings are shown in Figure 10.

Originally, the design engineers concluded that steel H-piles should be used to support the buildings. At a later date, caisson foundations were introduced as an alternative. The geotechnical consultant provided recommendations for the founding elevations of proposed foundation caissons. In that report, a bearing capacity of 8 tons per square foot was recommended for foundations established in the dense alluvium, and 40 tons per square foot was the recommended bearing capacity for foundations established in the mica schist bedrock.



The building layout was established to accommodate drilled-and-belled caissons designed for a bearing capacity of 40 tons per square foot in certain areas, and a bearing capacity of 8 tons per square foot in other areas. A drawing designated the planned bottom elevations of the caissons. The higher capacity caissons, which support column loads ranging from 2,500 kips to 4,500 kips, were originally intended to be constructed on relatively sound bedrock. One hundred eighty six caissons in the office building and courthouse were designed for a bearing capacity of 40 tons per square foot. These were designed in sizes ranging from 2-1/2 feet to 7-1/2 feet in shaft diameter with 3-foot to 10-foot diameter bells.

Problems started to appear while the courthouse building was under construction. Deflections were observed at six column locations. Downward movements of up to 1-7/16 of an inch at Column G-9 were observed. Other columns, designed for a bearing capacity of 40 tons per square foot, were also observed to be experiencing downward movements. Signs of downward movement had also been noted in the vicinity of Column M-15 at the basement level, beneath the completed and occupied office building. Evidence of distress (cracks and spalling of the concrete) was clearly evident in some of the grade beams overlying certain caissons.

Construction of the courthouse tower was discontinued to prevent increasing the load on Caisson G-9. At this time the structural engineers reported that the columns in this general area were supporting about 60 percent of the total design column load.

Dames & Moore was retained to study and evaluate the problem. The project began with an investigation of subsurface conditions in the proximity of certain caisson foundations and to evaluate the load-supporting characteristics of the materials beneath them. Test borings were drilled to depths ranging from 89 feet to 109 feet below the basement slab.

The investigation also included a study of the use of caisson foundations in the Philadelphia area. Where load tests or soil analyses have not been made, the City of Philadelphia Building Code assigns minimum bearing capacities as follows:

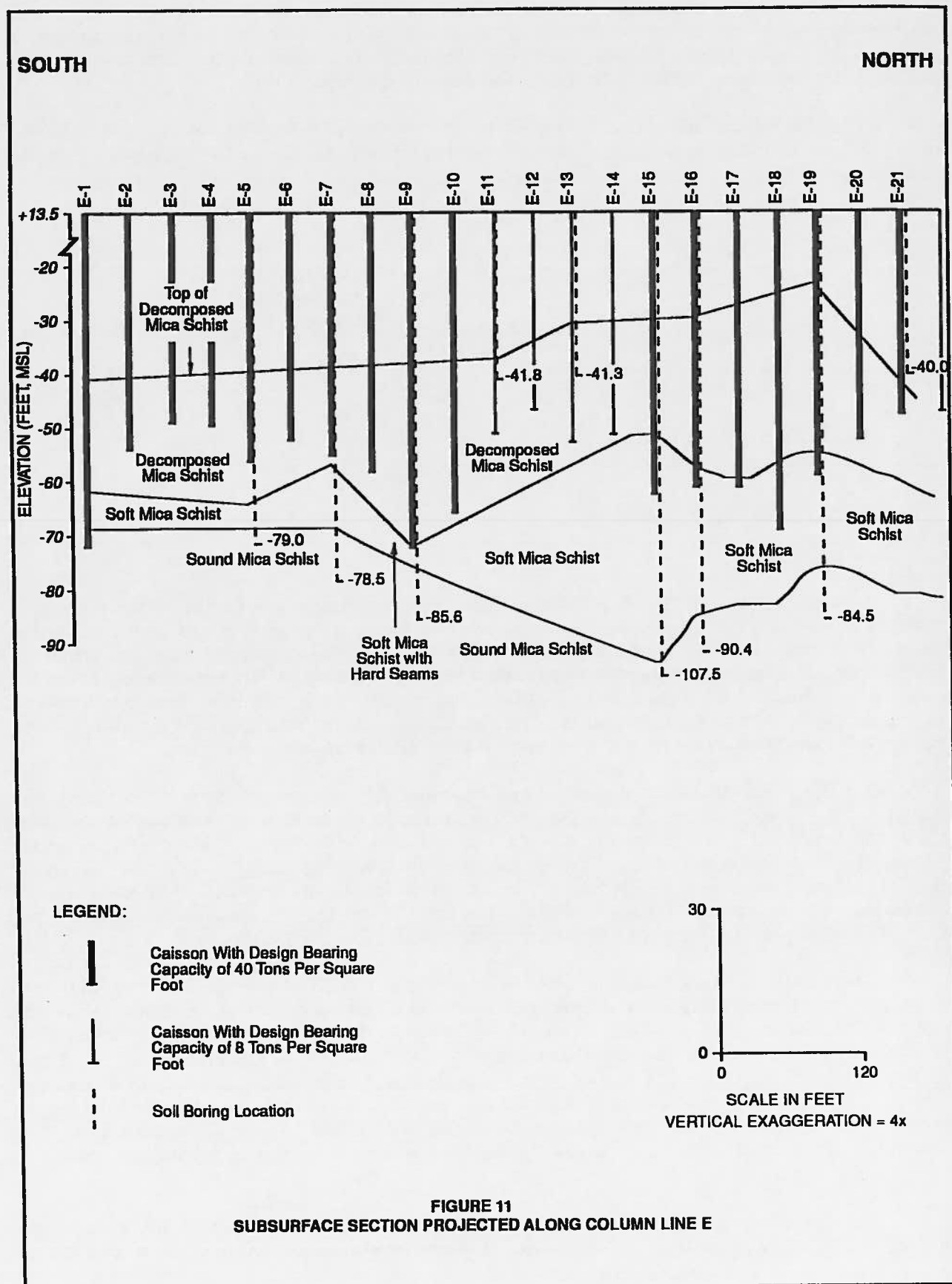
Soil	Bearing Values in Tons per Square Foot
Hard sound Mica Schist or Gneiss rock	20
Hard broken or seamy Mica Schist or Gneiss rock	15
Soft Mica Schist or Gneiss rock	10

Representatives of the Philadelphia Building Department could not identify a single building in Philadelphia where a bearing capacity of 40 tons per square foot had been used for the design of caissons installed on decomposed mica schist. However, foundation engineers in the Philadelphia area have recommended and used a bearing capacity of 40 tons per square foot for caissons established on sound mica schist. For example, the Federal Reserve Bank of Philadelphia, located immediately to the north of the site, is one example of a structure having caissons supported on sound rock at a bearing capacity of 40 tons per square foot.

It was learned that local engineers had designed caisson foundations to be supported on decomposed mica schist with bearing capacities in the range of 8 to 20 tons per square foot. However, none of the foundation engineers contacted knew of a single building constructed on caisson foundations in the Philadelphia area, designed for a bearing capacity of 40 tons per square foot on decomposed mica schist. It was also interesting to learn that the design and construction of the Federal Courthouse and Office Building was exempt from the City of Philadelphia Building Code, because these buildings were being constructed by the Federal Government.

The post construction borings were drilled adjacent to 28 of the 186 caissons that were designed for a bearing capacity of 40 tons per square foot. Using results of these borings, it was estimated that of these 28 caissons, 20 were established on decomposed mica schist, seven were founded on soft to medium hard mica schist, and one was founded on sound mica schist. It was further estimated that of the 158 remaining caissons designed for a bearing capacity of 40 tons per square foot, 100 were established upon decomposed mica schist, 40 were founded on soft to medium mica schist, and 18 were founded on sound mica schist. Using these data, Dames & Moore developed subsurface sections in the building areas. One typical subsurface section is presented on Figure 11.

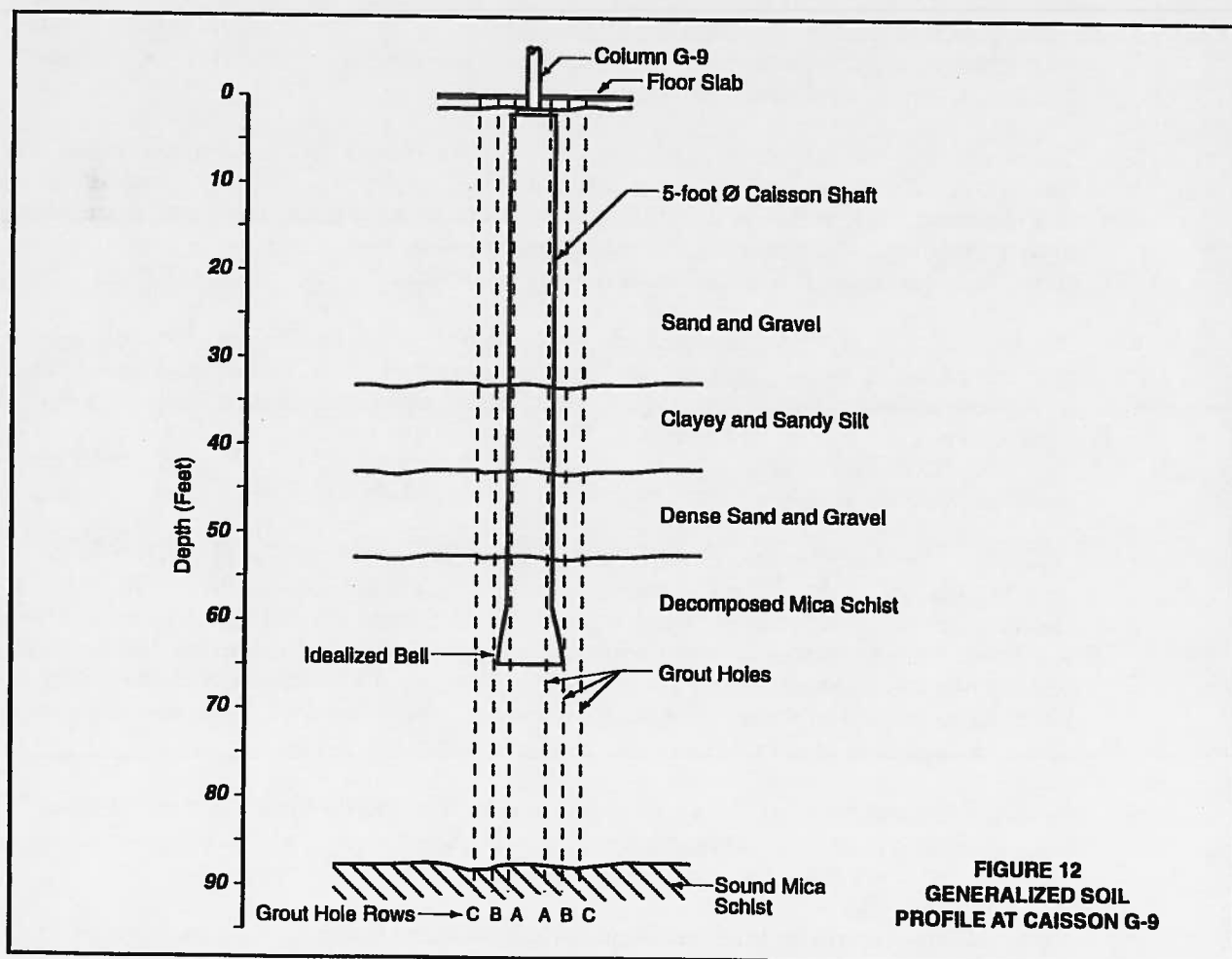
The accuracy of interpretations between borings varies because of the erratic and unpredictable nature of the weathering profile. However, it was concluded from the available data



that most of the caissons were established 10 to 20 feet above the originally proposed elevation and that caisson G-19 was apparently installed 45 feet above the originally proposed elevation. Amendments introduced into the original job specification permitted the contractor to determine the elevation of each caisson bell at the time of construction, based on a field and laboratory testing program. Errors in conducting the laboratory testing program translated into improper placement of the caisson bells because the rock was much weaker than what the testing results had indicated.

This combination of events caused many of the heavily loaded caissons to be founded on material other than sound mica schist. Marginal bearing capacity and potential downward movement of caisson foundations should have been considered when it was decided to establish the caissons designed for a bearing capacity of 40 tons per square foot, especially when the bell elevations were moved to a level above those originally recommended by the geotechnical engineer.

Three treatment modalities were considered to underpin or support the columns, including use of H-piles, drilled-in caissons, or grout to increase the bearing capacity of the supporting strata. Jacking of Column G-9 after completion of a remedial grouting program demonstrated the ability of the weathered schist to carry the design load with negligible settlement. An illustration of the grouting pattern at Caisson G-9 is presented in Figure 12. Subsequently, a combination of resin and cement grouting was used, both to repair poor concrete in the caissons and to strengthen the weathered schist below them.



This project is an example of what can go wrong when too much emphasis is placed on cost savings and not enough consideration is given to sound engineering practice. If the original geotechnical consultant was permitted to drill test borings into sound bedrock and establish proper tip elevations for either steel H-piles or belled caissons, the job could have been competitively bid and would probably have performed just fine. However, excessive efforts by the owner to reduce cost created an environment for all the wrong things to happen. In the process of trying to save a few thousand cubic yards of concrete, the entire project was put in jeopardy and the cost of repair greatly outweighed the cost of doing the job correctly from the onset.

CONCLUSIONS

Upon review of these three case histories and other similar projects, a few common threads seem to emerge. Settlement failures that are caused by design or construction flaws generally appear within the first year after the structure is completed. They may manifest themselves as inconspicuously as a broken pipe connection on a tank, or a bump on a concrete floor. Many times settlement failures are so insignificant that they simply go unnoticed. However, some settlement failures result in expensive repairs so the following conclusions are offered in recognition of the problem.

1. It is in the owner's best interest to retain a good geotechnical engineering consultant and have him involved with the project throughout both the design phase and the construction phase. Settlement problems seem to creep into a project whenever the geotechnical engineering consultant does not have continuity with the job, or have adequate control to prevent them.
2. Some owners (or engineers who represent the owner) have a nasty habit of specifying the number and depth of borings to be drilled for a subsurface investigation. While this is usually done in the name of controlling costs, the owner or his engineer should realize that he now bears the liability for limiting the subsurface information available and for any consequences that may result.
3. Occasionally, engineers put relatively little emphasis on settlement concerns when bearing capacity values appear to be more than adequate. This may not seem important—especially in today's highly competitive environment where time spent on a job equates to money—but it can be a trap that snares the careless consultant every now and then. The next time you read a geotechnical report and there is hardly any mention of settlement, be cautious. The author may not have given it much thought.
4. When engineers are faced with having to solve a forensic problem related to any kind of a foundation failure, they sometimes experience a reflex reaction to look for a solution that is highly technical and sophisticated. Some may immediately see a need for finite element modeling or applying complex theory to explain the failure. Experience has shown that most settlement failures result from simple causes, which are usually related to events that occurred at the time of construction. Even when a complex approach may be necessary, don't ever abandon the basics.
5. Grouting is generally the most useful treatment for strengthening loose granular soil, poor quality concrete and decomposed bedrock. Special grouting techniques, such as slab jacking or jet grouting, are also applicable under some circumstances.
6. Proper design, construction, and quality control practices in the initial stages of a project are generally much cheaper than post-construction grouting.

DISPUTE AVOIDANCE AND PILING SPECIFICATIONS

Bengt H. Fellenius¹

Abstract

Surprises costing money and causing delays occur frequently during the construction of foundation projects, and in particular for piling projects. The contract specifications often fail to spell out the responsibilities for such events and this omission invariably results in disputed claims that sometimes only can be resolved by litigation. Much of this can be avoided by careful wording of the specifications, expressing all quality requirements in quantifiable terms and anticipating difficulties setting out beforehand who is responsible. Examples are given of pitfalls to avoid and unsuitable phrasings used in actual specifications are quoted. New master specifications are presented and specific aspects are discussed, such as how to specify a pile driving hammer, quality of equipment, tolerance limits for bending and location, and terms to avoid. Example case histories of contract disputes are presented.

When the unexpected occurs at a site and costs escalate and delays develop, the Contractor feels justified to submit a claim that the Owner may see little reason to accept. When the parties turn to the technical specifications for the rules of the contract, these often fuel the dispute instead of mitigating it, because the specifications are vague, unclear, unbalanced, and containing weasel clauses that help nobody in resolving the conflict. Rarely do specifications prepare for that deviations from the expected can occur.

Surprises occur frequently during the construction of foundation projects, and in particular in the case of piling projects. The surprises take many forms, but one aspect is shared between them: they invariably result in difficulties at the site and, more often than not, in disputes between the parties involved.

For example, the soil conditions sometimes turn out to differ substantially from what the contract documents indicate. On other occasions, the piles do not go down as easily as anticipated by the Owner's design engineers and/or by the Contractor's estimator. Or, they may go down more easily and become much longer than anticipated. Or, a proof test shows that the pile capacity is inadequate. Or, the piles do not meet a distinct "refusal" and, consequently, the stringent termination criterion in the specifications results in a very prolonged driving causing delays and excessive wear on the Contractor's equipment.

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Quite often, the Contractor's equipment fails to do the job. Perhaps, the equipment required by specifications is "misdirected". Perhaps, the Contractor is inexperienced and can not perform well, or the equipment is poorly maintained and difficult to use. Whether or not the Contractor honestly believes that the subsequent delays, the inadequate capacity, the breakage, etc. are his fault, he will submit a claim for compensation. Often, when the claim is disputed by the Owner, the Contractor nevertheless is awarded compensation by the court, because the contract specifications do not normally contain any *clear* requirement for *quality* of equipment.

Or, the Contractor's leads are not straight and the helmet occasionally jams in the leads. However, are the leads out of the ordinary—after all, they are the same as used on the previous job—and, besides, although they are not straight can they really be called bent, or crooked?

Or, on looking down a pipe pile, the bottom of the pipe can not be seen. Well, is then the pile bent and is it bent in excess?

Or, when the use of a water jet is required to aid the pile penetration, the pile does not advance or advances too quickly and drifts to the side or a crater opens up in the soil next to the side of the pile. The pump pressure and volume are usually detailed in the specifications, but the size and length of the hose and the size of the nozzle are rarely indicated. Yet these details are vital to the performance of the jetting system.

Frequently, sentences are used such as "in the Engineer's opinion", but with no specific reference to what the opinion would be based on. Such general "come-into-my-parlor" clauses do not hold much water in court, but they are the root of much controversy.

Be careful of the meaning of the terms used. For example, 'allowable load capacity' is a totally confusing set of words. A few years ago, I worked on a litigation case where the Engineer used the words 'allowable load capacity' to indicate the required working load of the piles. Unfortunately, the Contractor interpreted the words to refer to the capacity to which he had to drive the piles. A similar confusions appeared on a more recent project (nothing really changes in this regard), where the engineer deliberately reduced the pile lengths to about half the usual length in order to avoid driving into a boulder layer existing at depth at the site. He also, appropriately, reduced the desired capacity and pile working load (50 tons), requiring a "capacity" of only 100 tons on piles normally accepted and installed to a 200-ton capacity, which is what the specs required. However, someone—it was never determined who—thought that plain 'capacity' sounded too casual and changed it to 'load capacity'. At the outset of the pile driving, the contractor asked what loads he was to drive to and was told that the loads were 100 tons. So, naturally, he drove to a capacity of twice the load, which meant that the piles had to be longer and, as the designer had expected, the piles were driven into the boulder layers. The results was much breakage, problems, delays, and costs. The claim for extra was \$300,000. Jargon terms can be very dear.

On the topic of using jargon: The word "set" is not a synonym for "blow-count" (the blows per a certain penetration length). Set is the penetration for one blow or, possibly for a series of

blows. In one case, the specifications stated that the Contractor was to drive the piles (concrete piles of limited strength concrete) "to a very small set and he was cautioned not to overdrive the piles". Of course, the Contractor took care not to damage the piles by driving them too hard, which is what "overdriving" means. In fact, the driving turned out to be very easy and several of the piles drove much deeper than the plans and drawings indicated. Unfortunately, in writing the sentence I just quoted, the spec-writer meant to warn the Contractor that the blow-count was expected to be very small and that the piles, therefore, could easily drive too deep. Talk about diametrically opposed interpretations. And predictable surprises. In this case, the Engineers insisted that their intended interpretation was the right one and a costly claim and litigation ensued.

Most specifications only identify a required pile driving hammer by the manufacturer's rated energy. However, the rated energy says very little of what performance to expect from the hammer. The performance of hammers varies widely and depend on pile size, choice of helmet and cushions, soil behavior, hammer age and past use, hammer fuel, etc. Whether or not a hammer is "performing to specs" is one of the most common causes of discord at a site. The reason is that the specifications are very poor in stating the hammer.

A Contractor undertakes to complete a design according to drawings and documents. Amongst these are the Technical Specifications, which purport to describe the requirements for the project in regard to codes, stresses, loads, and materials. Usually, however, only little is stated about the construction. Yet, in the case of a piling project, the conditions during the construction are very different to those during the service of the foundation, and the latter conditions depend very much on the former. When the project is similar to previous projects and the Contractor is experienced and knowledgeable, the technical specifications can be short and essentially only spell out what the end product should be. Such specifications are Performance Specifications. However, these are very difficult to write and can easily become very unbalanced, detailing some aspects and only cursorily mentioning others of equal importance. The text must spell out what is optional to the Contractor and what the Contractor must comply with. Even if the intent is that the specifications be Performance Specifications, and even if they so state, most specifications are actually written as Compliance Specifications (sometimes called Detailed Specifications). Government specifications are almost always Compliance Specifications.

When surprises arise and the Contractor as a consequence is slowed down, has to make changes to procedures and equipment, and generally is losing time and money, then, disputes as to the interpretation of the specifications easily develop. Therefore, the writer of Specifications must strive to avoid loose statements when referring to quality and endeavor to quantify every aspect of importance. Do not just say that a pile must be straight, but define the limit for when it becomes bent! Do not just say that the pile shall have a certain capacity, but indicate how the capacity will be defined! Do not forget to give the maximum allowable driving stresses and how they will be measured, if measured! In short, take care not to include undefined or unquantified requirements. One of the most non-constructive situation is when the Engineer says that a pile is damaged, or bent, or too short, etc. and the Contractor says "no it ain't". The Engineer answers

"it is, too!", and before long whatever communication that existed is gone, the lawyers arrive, and everybody is a looser (well, perhaps not the lawyers).

You may enjoy the following direct quotes from contract specification submitted by a US Government agency earlier this year, yes, in 1996.

1. Piles shall be driven to reach the design bearing pressures.
2. The minimum allowable pile penetration under any circumstance shall be 17 feet.
3. The Contracting Officer will determine what procedure should be followed if driving refusal occurs.
4. The hammer shall have a capacity equal to the weight of the pile and the character of the subsurface material to be encountered.
5. The hammer energy in foot-pounds shall be three times the weight of the pile in pounds.
6. Inefficient diesel, air, or steam hammers shall not be used.
7. Each pile shall be driven until the bearing power is equal to the design piles pressure.
8. All piles incorrectly driven as to be unsuitable as determined by the Contracting Officer shall be pulled and no payment will be made for furnishing, driving, or pulling such piles.
9. All piles determined to be unsuitable by the Contracting Officer shall be replaced by and at the expense of the Contractor.

I promise you that the above quotes are real and not made up by me for the occasion. I am sure that many of you have similar and worse examples to show. However, when you stop smiling, you should ponder what depths of ignorance and incompetence the nine quotes represent. And the consequence to Society of our industry having to function with such players in charge of the purse strings.

Public Works Canada (PWC), recently commissioned the author to prepare new master construction specifications for piling in collaboration with the PWC. It is the intent that upon formal approval, the specifications will become new National Master Construction Specifications.

The prime objective in the new specifications has been to obtain specifications which will reduce the incidence of *disputed* claims in the construction process. Therefore, in writing the specifications, much effort has been placed on quantifying the requirements. It is expected that the specifications will significantly reduce the incidence of disputed claims in the piling construction work.

Notice, a undisputed claim is not necessarily a negative aspect of a project. When the Contractors know that should a difficulty arise that lies outside what one normally would expect, incurred costs will be compensated, then, their price will reflect the removal of the contingency

charge otherwise necessary. If, on the other hand, they must consider that, should the difficulty appear, their claim for compensation may be rejected, they will include a contingency for this eventuality. Then, should the difficulties occur, the Contractors often try to claim for the incurred costs anyway, and they might win the claim, collecting twice, as it were. Or because of their including the contingency, their bid was high and they lost the contract to the less experienced competitor, who indeed, claimed compensation for the difficulty along with a few more of his own doing. The continual loser is the Owner and the Society.

The new Master Specifications follow the standard format and numbering of the current US and Canadian National Master Specifications. That is, they include a general section, a section on the static loading test, and separate sections for specific pile types, such as pipe piles, socketed piles, wood piles, H-piles, precast concrete piles, and expanded-base piles. Of course, the system of measurements is SI.

Spec Notes are frequently interspersed in the text to provide explanation and advice to the designer adapting the master specifications to a project. The Spec Notes and any irrelevant clauses are to be deleted by the Spec-writer before the particular project specifications are completed.

For example, instead of specifying a pile driving hammer by its rated energy, the new specifications specify a hammer by the energy transferred to the pile and the impact force delivered to the pile, which are well defined quantities. In the design phase, energy and force values are to be obtained by means of a wave equation analysis. The wave equation analysis will "marry" the hammer to the pile and soil and to the particular drivability conditions and desired capacity. Naturally, the Owner has the obligation to provide correct values and the Contractor has the right to expect that the values specified are correct. If the values are incorrect, and the Contractor suffers as a consequence, the Owner must compensate the Contractor.

More often than not, the analysis will show that theoretical analysis alone is not able to sufficiently accurately determine the hammer requirements. This is then not an argument against specifying the values. It is an argument proving the inadequacy of simply picking a rated energy, which puts the risk onto the Contractor. It is also an argument for the Owner's obligation to find out ahead of time, or at the outset of a project, what the correct values are. For example, by means of taking dynamic measurements with the Pile Driving Analyzer (PDA). PDA measurements are since many years routinely used to finalize a pile design in connection with test driving or during the Contractor's installation of index piles.

When the potential use of the Pile Driving Analyzer is included in the specifications, then, if during the course of the piling work, reasons arise to question the hammer performance, the PDA can quickly and with a minimum of fuss be brought to the site and the hammer can be accepted or rejected as based on the agreement of the measurements with the specified values. Opinion may differ with regard to the adequacy of the specified values, but such differences are technical in nature and easily resolved without involving the lawyers.

The dynamic measurements may interfere with the Contractor's work, therefore, the general specification section contains a clause that outlines how the measurements are performed and what the responsibilities are for the parties involved.

Dynamic measurements are also commonly carried out to determine pile capacity and integrity. Notice, the PDA measurements need analysis to be useful. Also, the data must be combined with conventional records of the pile installation. The new PWC specifications include a list of what records to take that may look long, but the logging effort costs little and the records may save a good deal of cost in the end, perhaps even making more elaborate measurements and testing redundant.

Further, bending and doglegging of a pile have been defined by a specific bending radius, and straightness and out-of-location have been given specific tolerances. For example, the specifications require that piles must not be bent more than to a radius of 200 m (600 ft) after driving (bending limits are also given for conditions before driving). For pipe piles, this is readily determined by means of an inspection probe, also detailed in the specifications, designed to jam in the pipe at this radius. A pipe pile for which the bottom can not be seen, but into which the probe reaches the bottom, is then by definition straight and acceptable.

The new specifications have increased the requirements on how to perform a static loading test. Details are given on the arrangement of the test, choice of gages, and the method of testing (the quick method is the preferred method). The use of a load cell for determining the applied test load is mandatory.

Requirements particular to each pile type are given in separate sections. For example, requirements are given for long precast prestressed concrete piles pertaining to splices, placing center tubes in the piles, and for measuring and documenting strand slippage on release of strands into the pile.

In the following, a selection of direct quotes are given from the general section of the new specifications, Section 02351. The full text is available in a Continuing Education Guidelines booklet published by the Deep Foundation Institute.

Notice, the numbers given in brackets are suggested values, but may be changed by the spec-writer to fit the project. Sometimes, the brackets only enclose the term "VALUE", or similar, to remind the spec-writer to insert an appropriate value.

3.1 Equipment**.1 Equipment information**

.1.1 Prior to commencement of pile installation operation, submit to [Engineer] for [approval] [review], details of equipment for installation of piles.

SPEC NOTE: *The information requested in 3.1.1.2 is necessary as input to a detailed wave equation analysis of the pile driving.*

.1.2 For impact hammers, give manufacturer's name, type, rated energy per blow at normal working rate, mass of striking parts of hammer, mass of driving cap, and type and elastic parameters of hammer cushion.

.1.3 For non-impact methods of installation, such as drilling, jacking, jetting, use of vibratory hammers, or other means, give full details of characteristics necessary to evaluate performance.

.2 Impact Hammer

SPEC NOTE: *Do not exclude a hammer type unless for very specific reason.*

.2.1 For final driving of [TYPE] piles, provide a hammer capable of delivering to the pile a non-erratic impact stress not smaller than [VALUE] Pascal and transferring a non-erratic energy not smaller than [VALUE] Joules per blow to the pile head at normal working rate.

SPEC NOTE: *Bounce-chamber gages in double-acting diesel hammers are frequently inaccurate.*

.2.2 For air/steam hammer and double-acting diesel hammer, provide a calibrated pressure gage and position the pressure gage on the hammer side of all valves, with no more than 30 m of hose away from the hammer intake, and so it can be easily observed by [Engineer]. Provide a calibration certificate of the pressure gage that is current within 6 months.

SPEC NOTE: *For driving of concrete piles, where tensile stresses are of concern in easy driving, include 3.1.2.3. For both ordinary precast concrete piles and prestressed precast concrete piles, the limiting*

driving tension is governed by the yield strength of the reinforcement with no allowance for tensile strength of the concrete. Therefore, the maximum tensile resistance in a driven concrete pile is equal to the axial tension capacity of the reinforcing bars or of the prestressing strands. When applying a factor of safety of about 1.4, the maximum allowable tensile stress is 70 % of the steel yield. Coincidentally, this percentage is about equal to the net prestress in a prestressed pile.

.2.3 The hammer shall be adjustable to delivering a reduced impact so that during easy driving of the pile, when tensile stresses are of concern, the largest reflected tensile stress is at all times smaller than [VALUE] Pascal.

.2.5 Approval of the hammer equipment [will] [may] be subject to [Engineer's] assessment of the results of the dynamic monitoring of the pile driving using the Pile Driving Analyzer, as described in Clause [3.13].

.2.6 Remedial action due to failure of the Contractor's hammer equipment to satisfy the requirements in Clause [3.1.2] will be at the Contractor's own expense. Such remedial action may consist of, but need not be limited to, adjustment or replacement of hammer cushion, or of pile cushion, or to adjustment or replacement of hammer.

.4 Leads

.4.1 Provide leads that will enable the hammer to deliver impacts concentrically and in alignment with the pile longitudinal axis without inducing bending moments in pile.

.4.2 Provide fixed leads held in position at or near the top and at or near the bottom except where swinging leads are authorized by [Engineer].

.4.3 Provide leads that are parallel and not bent beyond a 15 mm deviation from a straight line over any 5.00 m length (0.3 %).

.4.4 Provide leads of sufficient length to accommodate the combined length of pile and hammer.

.4.5 Performance of the leads will be subject to assessment of [Engineer]. Any remedial action required will be at the Contractor's own expense.

.5 Follower

.5.1 Do not use follower without [Engineer's] permission.

.5.2 When permitted, provide a uniform follower with constant dynamic impedance (EA/c) equal to that of pile and of shape and length enabling driving of pile in specified location to required depth and resistance. Provide follower with socket or hood carefully fitted to the pile head to minimize loss of energy and prevent damage to pile.

.5.3 Where follower is to be used, drive applicable test piles using similar follower.

.5.4 Proper design and performance of follower will be subject to the assessment of [Engineer] which may include dynamic monitoring using the Pile Driving Analyzer as described in Clause [3.13] and any remedial action will be at the Contractor's own expense.

SPEC NOTE: Include 3.1.6 for long, closed-toe, pipe piles and for concrete piles equipped with center tubes. The probes should be designed to react for 200-metre and 100-metre bending radii as described by the Canadian Foundation Engineering Manual (1985), Chapter 24, Section 24.3.3.

.6 Inspection probes

.6.1 Provide two inspection probes for use for pile inspection. The probes shall be made from heavy steel pipe and have dimensions length [] metre ("Probe Radius 200 m") and [] metre ("Probe Radius 100 m") and outside diameter [] mm ("Probe Radius 200 m") and [] mm ("Probe Radius 100 m").

.6.2 After initial driving of pile, lower the probe marked "Probe Radius 200 m" into [pile] [center tube] to the toe of the pile. If the probe fails to reach the toe, record depth of stop, and proceed to lower the probe marked "Probe Radius 100 m".

.6.3 Pile in which the "Probe Radius 200 m" fails to reach the pile toe may be rejected subject to the assessment of [Engineer].

3.3 Field Measurements

SPEC NOTE: Careful records are very important for assessing the unexpected occurrence of any event at a piling project. Delete or modify to suit project.

.1 Maintain accurate and daily records of driving for each pile, including:

- .1.1 Type, make, and rated energy of hammer.
- .1.2 Other installation equipment including details on use of pile cushion, follower, and water jet.
- .1.3 Pile size and length, location of pile in pile group, and location or designation of pile group.
- .1.4 Time for start and finish of driving pile and sequence of pile driving for piles in group.
- .1.5 Penetration for own weight and own weight and weight of hammer, number of blows per 300 mm of penetration from start of driving, and penetration for [four] consecutive series of [] blows when approaching termination of driving of pile.
- .1.6 Observed stroke and blow rate (blows/minute) of hammer.
- .1.7 Toe elevation on termination of driving pile and final toe and cut-off elevations on completion of pile group.
- .1.8 On termination of the driving of open-toe pipe piles, record depth from ground surface outside pile to soil surface inside pipe.
- .1.9 Records of restriking.
- .1.10 Result of inspection of pile by means of inspection probe, as described in [3.1.6].
- .1.11 Other pertinent information, such as interruption of continuous driving, observed pile damage, etc.
- .1.12 Records of elevation of adjacent piles before and after driving of pile.
- .1.13 Record all information on forms provided by [Engineer]
- .2 Provide [Engineer] with three copies of the records.

3.4 Pile Installation

- .1 Use driving helmet to protect pile head.
 - .2 Do not use any loose inserts in the helmet. The Engineer is sole judge of the acceptability of the helmet.
-
-

.6 Advance pile to [toe elevation as indicated on drawing] [penetration resistance specified in Clause [3.9]].

.10 Restrike piles which have settled or heaved during driving of adjacent piles. No additional compensation will be made for pile restruck due to such settlement or heave.

.11 Restrike piles as directed by [Engineer].

***SPEC NOTE:** Include Clause 3.4.14 for closed-toe pipe piles and precast concrete piles equipped with center tubes.*

.14 After driving, pile must be accessible for inspection of integrity and bending through the full length of pile. Presence of any foreign material and water is a cause for rejection of the pile at the discretion of [Engineer].

3.6 Obstruction

.1 Where obstruction is encountered that results in sudden, unexpected change in penetration resistance and deviation from specified tolerances, the Contractor may be required to perform one or all of the following:

.1.1 Removal of obstruction.

.1.2 Extraction, repositioning, and redriving.

.1.3 Addition of extra piles.

.2 If in the opinion of [Engineer] work done as per Clause [3.6.1] could not have been reasonably anticipated by the Contractor, additional compensation for work done will be considered for payment.

3.9 Penetration Resistance

.1 Installation of each pile will be subject to [approval] [review] of [Engineer], who will be sole judge of accept-ability of pile with respect to penetration resistance at end-of-initial-driving as well as at restriking, depth of penetration, or other criteria. [Engineer] to [approve] [review] final penetration resistance of all piles prior to removal of pile driving equipment from site.

.2 [Engineer's] assessment of penetration resistance [will] [may] necessitate dynamic monitoring using the Pile Driving Analyzer, as described in Clause [3.13].

SPEC NOTE: Select and modify 3.9.3 through 3.9.8 to suit project. Notice, Clause .3 is intended to minimize the risk for excessive and prolonged driving, while still ensuring the pile capacity; all three alternative termination criteria are meant to be included.

.3 Drive each pile to a final penetration resistance during initial driving of at least [120] blows for [three] contiguous penetrations of 300 mm, or to a penetration resistance of at least [200] blows for a penetration smaller than 300 mm, or to a penetration smaller than 25 mm for [two] contiguous series of [50] blows, whichever occurs first.

.4 Drive each pile to a final resistance measured in penetration for [three] consecutive series of [20] blows. The required penetration will be established by [Engineer] from results of [dynamic monitoring] [and] [static loading test] as described in Clause [3.13] and Section [02356], respectively.

.5 Prior to taking final penetration resistance, drive piles without interruption for a sufficient interval to break or prevent development of soil set-up.

.6 Drive each pile to a minimum penetration of [VALUE] metre.

.7 Drive each pile to a minimum [toe penetration of [VALUE] metre] [toe elevation of [+ ELEV]] [as indicated].

.8 When required by [Engineer], restrike piles to [the same criterion as applied in initial driving—Clause [3.9.3]] [to a penetration of [VALUE] mm for [three] consecutive series of [20] blows]. No additional compensation will be made for restriking.

3.10 Tolerances

.1 Pile heads, at cut-off elevation, to be within [75] mm of locations indicated, [as measured immediately after termination of initial driving, and [150] mm as measured after all piles have been driven]. [To achieve pile installation within tolerances specified, the Contractor may have to resort to using temporary bracing and templates, as specified in Clause [3.5].

.2 Pile, at cut-off elevation, to be no more than [2] % of length out of alignment.

SPEC NOTE: Clause 3.10.3 applies to pipe piles and concrete piles equipped with a center tube.

.3 Piles must not be bent beyond a bending radius of 200 m, as determined by means of the inspection probe in Clause [3.1.5], which is to be provided and used by the Contractor, or, alternatively, determined by means of inclinometer measurements performed by [Engineer].

***SPEC NOTE:** Include 3.10.4 for soldier piles on specific occasions.*

.4 Pile rotation to be limited to [VALUE] degrees.

.5 Maintain piling within tolerances specified throughout execution of work.

***SPEC NOTE:** Clause 3.10.6 addresses location of an individual pile immediately after its placement. Mislocation of groups of piles and individual piles due to drifting after placement should be addressed separately, as it often is due to causes beyond the Contractor's control.*

.6 If in the opinion of [Engineer] piles are placed beyond tolerances specified, the Contractor may be required to remove such piles and install new piles to the specified tolerances at his own expense.

3.13 Pile Driving Analyzer

***SPEC NOTE:** It is not possible to say beforehand that no problems or disputes will occur on a specific site, and for that reason, it is not possible to say that monitoring is not going to be helpful or required at piling project. Therefore, dynamic monitoring should be considered for all projects involving driven piles and all projects should include specifications on dynamic monitoring albeit with option not to use it or to use it only to a limited extent. Notice, dynamic monitoring is not just to determine pile capacity, it is also an important part of the pile inspection and required for verifying that the pile driving hammer is working correctly and consistently, that driving stresses are adequate, and the soil response as anticipated in the design.*

***SPEC NOTE:** Dynamic monitoring should always be contracted separately with a specialist monitoring company and not be a responsibility of the piling contractor.*

***SPEC NOTE:** Dynamic monitoring includes static analysis and wave equation analysis performed before the monitoring work and appropriate to the site (actual soil, hammer, and pile data to be used in the analyses). Further, selected records obtained in the monitoring work are to be subjected to detailed CAPWAP computer analysis. These aspects must be included in the separate instructions to the monitoring company. ASTM D4945 contains information and recommendations for the performance of dynamic monitoring.*

SPEC NOTE: The Contractors costs for dynamic monitoring at initial driving differ from costs during restriking, which may require special movement of the equipment. Therefore, it is necessary to indicate the monitoring occurring during initial driving and restriking, separately.

.1 Dynamic monitoring of the Pile Driving using the Pile Driving Analyzer will be used on [[ten] piles during initial driving and on [ten] piles during restriking] [15 % of all piles] as selected and directed by [Engineer].

.2 The dynamic monitoring will be used to determine the performance of the hammer, the driving stresses in the pile, the mobilized capacity of the piles, and the required final penetration resistance in initial driving and in restriking.

.3 The Contractor shall co-operate to ensure that the schedule for these specific piles is adhered to by providing all related access and assistance to enable the testing company to perform the monitoring.

.4 The dynamic monitoring consists of attaching strain transducers and accelerometers to the pile, usually close to the pile head, and connecting these to a monitoring station on the ground by means of a cable. Care must be taken to ensure that no damage is done to the transducers, cables, or equipment.

.5 The preparation of the pile head will be done by the testing company, usually while the pile is on the ground. The preparation consists of drilling and tapping holes in the pile. The transducers are attached after the pile is secured in the leads. The Contractor shall assist the testing company in providing a man to climb the leads carrying the transducers and cable and attaching the transducers by bolting them to the holes. The testing company will instruct the Contractor in the details of this work. Attaching the transducers to the prepared pile head requires installation of a total of seven threaded bolts into the prepared holes. Before commencement of pile driving, the testing company will check that all connections are functioning. Occasionally, the driving may have to be temporarily interrupted for the transducers to be adjusted or replaced, or the monitoring results assessed. Such interruptions do not constitute a new monitoring event.

.6 No additional compensation will be made for dynamic monitoring specified in Clause [3.13.1].

SPEC NOTE: It is frequently necessary to perform dynamic monitoring beyond the amount anticipated in the design. Payment must therefore separate between compensation for monitoring according to

Clause [3.13.1] and compensation for additionally requested monitoring. Such monitoring may be carried out during initial driving and/or during restriking.

.7 Monitoring beyond the amount specified in Clause [3.13.1] will be measured for additional compensation.

3.14 Pile Integrity Measurements

***SPEC NOTE:** Questions about the soundness of piles often arise, be the piles driven or bored. When questions do arise, it is important to have the means to perform integrity measurements with option not to use it or to use it only to a limited extent. The option of performing such measurements should, therefore, be included in the specifications.*

***SPEC NOTE:** For driven piles, assessment of pile integrity is included in the high-strain dynamic monitoring provided by the Pile Driving Analyzer. The integrity can also be checked by means of low-strain measurements applicable to both driven and cast-in-situ piles. Precast concrete piles can be checked for hidden defects before driving.*

***SPEC NOTE:** Low-strain integrity measurements should always be contracted separately with a specialist testing company and not be a responsibility of the piling contractor.*

***SPEC NOTE:** Low-strain integrity measurements includes analysis of the soil profile given in the borehole records to correlating the information to the measurements. The testing also includes measurements on many more piles than normally tested in high-strain tests using the Pile Driving Analyzer. The records from several piles need to be compared in order to group the typical responses to the low-strain measurements at the site. Further, selected records should be subjected to detailed computer analysis, so-called PITWAP analysis. These aspects must be included in the separate instructions to the monitoring company.*

***SPEC NOTE:** Low-strain integrity measurements are frequently indecisive and require comparison to other means of testing, such as coring and cross-hole testing methods. The low-strain testing then serves to identify the piles in need of special additional testing.*

.1 Low-strain integrity measurements [may] [will] be performed after the concrete has cured for [72 hours] on [all] [75 % of the] piles as selected and directed by [Engineer]. The measurements will be performed by an independent firm. The Contractor is to provide access to the pile head and to finish the pile head with a smooth upper surface onto which the accelerometer will be placed.

The need for some of the clauses is illustrated by the following summary of four cases of project disputes that went to litigation.

1. Overdriving of a group of steel piles. Several steel piles were to be driven into a dense sand to a predetermined embedment depth of 85 feet. Already at a depth of about 30 ft, penetration resistance values began to exceed 200 blows/foot. The 'Engineer' insisted that the Contractor drive the piles to the specified depth despite that blow-counts reached and exceeded 1,000 blows/foot! A "post mortem" review of the records makes it quite clear that although the heads of the about 90 feet long piles were beaten into the 5-ft stick-up above the ground surface, the pile toes probably never went past a depth of 60 feet. The Contractor had planned for a two-week project in early Fall. In reality, it took almost three months. As the project was located north of the 60th parallel, one can perhaps realize that the subsequent claim for \$6,000,000 was justified. Incidentally, the Contractor could not get out of his obligation to drive the piles. His bond saw to that. However, he won the full amount of his claim from the Owner. The Owner later sued the Engineer for negligence and won.

2. Complete breakdown of communications between Contractor and Engineer. A Contractor got permission to use a heavy diesel hammer at an energy setting lower than the maximum which, according to the hammer manufacturer's notes would be equal to the rated energy for a smaller hammer given in the specifications for the project. At the outset of the piling, it became obvious that the piles drove very slowly at that setting, requiring more than 1,000 blows before the specified termination criterion (minimum depth and penetration resistance) was reached. Static testing showed that the capacity was insufficient. The specifications included provision for jetting and the Engineer required this for all piles. Yet, it was clear that the pile could be driven down to the depths and capacities quickly and without jetting if the hammer was set to work at the maximum energy setting. Of course, this meant that the hammer energy was to be set at a values higher than that given in the specifications. The Engineers were willing to accept this change. However, the Contractor required extra payment for the deviation from the contract to do this, which the Engineer did not want to grant. One thing led to another. The Contractor continued to drive at reduced hammer setting and diligently worked to adhere to the smallest detail of the specification wordings. The Engineer refused to budge and required jetting and recorded everything the Contractor did to ensure that, as the Contractor now wanted to follow the specs to the letter, he was not to deviate from any of the details. Incidentally, the specifications called for outside jetting (rather than interior jetting) in silty soil, which resulted in drifting, bending, and breaking of piles. The final suit involved claims for compensation of more than \$10,000,000.

3. Specification for a near-shore piling project required piles to be driven flush with the sea bottom by means of a follower and stated that the follower should have 'sufficient impedance', but did not explain what this was and nobody checked the impedance of the follower. The Contractor drove the piles with a follower consisting of a steel pipe filled with wood chips. As the driving proceeded, the follower deteriorated and it became harder and harder to drive the piles. This was thought to be caused by densification of the sand at the site and the Contractor stated that the soil report failed to show that densifiable soils existed at the site and claimed compensation for changed soil conditions. The contract required that dynamic measurements be performed at the project, and they were. However, the results of the PDA measurements were not looked at by anyone! Eventually they were, of course, and it became obvious to all that the root of the problem was with the inadequate follower. Well, better late than never, but the delay certainly cost the parties a bundle of money.

.2 The low-strain integrity measurements will be used to provide information for analysis of pile integrity. They involve attaching an accelerometer to the pile head and connecting the accelerometer with a cable to a data acquisition unit. Measurements are initiated by striking the pile head with a small hand-held hammer.

.3 The Contractor shall co-operate to ensure that the schedule for the low-strain integrity measurements is adhered to by providing all related access and assistance to enable the testing company to expeditiously perform the measurements.

.4 No additional compensation will be made for low-strain integrity measurements specified in Clause [3.14.1].

SPEC NOTE: It is frequently necessary to perform integrity measurements beyond the amount anticipated in the design. Payment must therefore separate between compensation for low-strain integrity measurements according to Clause [3.14.1] and compensation for additionally requested measurements.

.5 Low-strain integrity measurements beyond the amount specified in Clause [3.14.1] will be measured for additional compensation.

4. Long prestressed piles were required to support a new dock for a port extension. The soil profile consisted of an about 35 m to 40 m very soft soil deposit with some dense sand layers of varying thickness and depth, followed by very dense gravel and sand with boulders. The depth to the bearing soil layer required piles of such length that they became heavier than the available equipment could handle. The piles problem was solved by building the piles as composite piles, the upper about 30 m long solid concrete section and a lower about 15 m long H-pile section. The penetration into the dense soil was expected to vary because of the presence of the boulders. During driving through the soft soils, care was taken not to drive too hard as this would have induced damaging tension in the pile. However, when the pile toe reached the dense soil and the penetration resistance increased, the hammer was set to hit harder to build up capacity and to advance the H-pile end into the bearing layer. Several piles broke already at moderate blow-count and others a few feet further into the very dense bearing layer during hard termination driving. Expressed reasons for the breakage ranged from poor quality of the piles through sudden barge movements and inadequate equipment and/or use of wrong pile cushions. Not until the case was before the courts was it established that the H-pile extension was so light that the impact wave on reaching the end of the concrete section, which was in the very soft soil, a large portion of the wave was reflected as a tension wave. Because the pile toe was in the dense soil, when the remainder of the wave reached the pile toe, a strong compression wave was reflected. The low blow-count and good toe response made the hammer ram rise high and provide a strong impact to the pile. The tension from the end of the concrete section being proportional to the impact force, therefore, reached damaging levels. A study compiling driving logs showed that the breakage correlated well with the presence of soft soil at the bottom end of the concrete section. Dynamic measurements had been conducted for determining capacity early in the project. The 'post mortem' study of the records established that when the bottom end of the concrete section was in soft soil, tension reflections occurred that exceeded safe levels.

It is not possible to give too many details on projects that went to dispute, because space limitation precludes giving an adequately impartial background to the cases. An account giving some of the details could easily appear slanted toward one or the other of the various players, who may then be justified in feeling slighted. Therefore, only the above cursorily information is presented in these notes. The oral presentation will be illustrated and include more details.

Lucid, comprehensive, and equitable specifications are necessary for successful projects. However, even when the specs are good, if the communication lines break down, the project may still end up keeping our fellow professionals in the legal field living well. However, it is my experience that rarely are the initial 'surprises' and difficulties such that the parties really need to go the full way of the courts. Instead of posturing and jockeying for legal position, if the parties show a bit of good intent and willingness to understand each other and make some effort toward finding out what really is happening and why so, litigation can often be avoided. When people keep talking to each other, an understanding can usually develop that the specs are unclear or special technical difficulties have indeed arisen, and that some common sense 'horse trading' may settle the money issues. Going to court should be a last resort.

* GEOTECHNOLOGY IN DISPUTE RESOLUTION

By James P. Gould,¹ Honorary Member, ASCE

(The Twenty-Sixth Karl Terzaghi Lecture)

ABSTRACT: This paper explores the factors that contribute to claims of changed subsurface conditions in heavy-construction contracts. It reviews methods of resolving the attendant disputes and describes a group of cases to illustrate the issues. These cases are concerned chiefly with mined tunnels, which are the most drastically affected by the unexpected. The purpose is to summarize the special requirements for preparing geotechnical information for construction and to suggest procedures for avoiding claims or bringing about their equitable resolution.

INTRODUCTION

The paper is based on the October 1990 Terzaghi Lecture, which was titled "Inventive Changed Conditions Claims," presented at the 1990 ASCE annual convention, in San Francisco. I have recast the paper to take into account rapid developments in dispute resolution since 1990.

Increasingly, construction contract litigation intrudes into the practice of geotechnical engineering. This occurs particularly through changed conditions claims that respond to the provision in federal-aid contracts intended to relieve the contractor of the effects of unforeseen underground conditions. These disputes center on the subsurface, and the geotechnical engineer may be drawn into a forensic role or even be collared among the defendants. The point of view of the paper is that of engineers serving the project owner, since the geotechnical activity undertaken for the owner is larger than that for the contractor. Their work product should be a realistic site characterization to serve the design, bring economy in construction, and avoid disputes.

This paper is not intended as a reference for the standard of care to be applied in specific cases. Those familiar with U.S. heavy construction know that geotechnical practice responds to local geology and industry usages and that it may be inappropriate to impose general standards on the project engineer, who is the best arbiter. No attempt is made here to introduce opinions on legal issues. Decisions of administrative judges or review boards are included solely on factual points. Omitted entirely are environmental considerations, job-safety issues, and consequential damages derived from changed conditions. The paper concentrates narrowly on conventional soil-mechanics problems.

THE DIFFERING SITE CONDITION CLAUSE

The federal contract provision recognizing a "differing site condition" (DSC) originated between the World Wars and appeared in its present form in February 1968. The clause expresses the concept that the risk (in effect the additional cost) of an unexpected underground condition will be borne by the owner. The two changes for which the contractor is entitled to an equitable adjustment are:

- Type I: "Subsurface or latent physical conditions at the site differing materially from those indicated in this contract" or
- Type II: "Unknown physical conditions at the site, of an unusual nature, differing materially from those ordinarily encountered and generally recognized as inherent in the work of the character provided for."

Type I is a conflict with those features portrayed in the contract or reference documents. This difference can arise from a wide variety of imperfections in the exploration, interpretation, or site characterization. It can involve an error or a deficiency of the technique or scale of the exploration. Type II is a surprise to all, not a discrepancy in the documents but an unusual physical condition beyond that reasonably expected. By the nature of construction litigation and soil mechanics practice, a discrepancy or defect in the prebid data is easier to demonstrate than a "surprise," which is judged subjectively. The differing condition must be "material," having a significant impact on cost, time, or methods of construction. The more extensive a competent site exploration, the less chance of a surprise (Type II) or a difference between real and predicted conditions. However, greater detail and wider speculation in the geotechnical study can increase the chance to prove a defect (Type I) by offering a larger target to an aggrieved contractor.

Explanation of the DSC Clause

The DSC clause is intended to decrease the contingency that bidders must otherwise include in their proposal. Additionally, it is in the public interest to achieve equity in a contract whose ultimate object is a public benefit. The same motive would not necessarily be incumbent on a private owner. While there is an intent to include a DSC provision in federal-aid contracts undertaken by local public agencies, this is not automatic.

Without a DSC clause the aggrieved contractor may claim misrepresentation to recover the cost of a defect, a surprise, or a simple loss. For some agencies, this alone is not sufficient for recovery since fraud or negligence must also be demonstrated. An example of such negligence could be a boring made without meeting the "standard of care." Specialists know that there is no absolute standard of care in soil mechanics, much less in a commercial boring log. Standard of care is elastic, depending on the project, budget, venue, industry or agency usage, engineering judgment, etc. One or more forensic "experts" may be needed to set to rights the issue of standard of care.

Though hardly the Eldorado represented by environmental remediation, changed condition litigation has created a burgeoning market for geotechnical services both in asserting

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and defending claims. However, there is a clear negative aspect that puts the geotechnical engineers of a project at risk for failing the standard of care in their site characterization. With only a modest amount of imagination an attorney can convert what is truly a DSC claim into a suit against the project owner and design professionals. The owner's geotechnical engineers should recognize that it is greatly in their own interest to produce a prebid investigation that will avoid a claim. If a claim eventuates, then the engineers should strive to resolve that claim equitably and promptly and not encourage it to escalate to the next level. Note that federal backers will often contribute to settlement of a DSC claim, but they certainly will not choose to share the cost if the finding is of negligence or defective design.

Absence of a DSC Clause

To evaluate the effect of the DSC provision, Halligan et al. (1987) studied the handling of claims on a large number of projects. Traditionally, in the absence of a DSC clause the responsibility for unexpected site conditions rested with the contractor. Typically, the engineers' site investigation was intended for planning and design and not much concerned with constructability. Halligan et al. pointed out that regardless of the contract provisions, contractors do not and cannot assume sole responsibility for unexpected site conditions, and hence they expect owners to share the risk. An independent subsurface investigation is usually too costly for contractors to use in preparing bids. Overriding market pressures to reduce all costs greatly restrict the contingencies for uncertainty that can be placed in bids. Halligan et al. concluded that it was difficult to distinguish the recoveries for contractors working under liberal DSC clauses compared to those under more restrictive language. In a majority of cases in which there were discrepancies or surprises, the contractor convinced the courts that these were in part the responsibility of the owner.

Interpretive reports on site conditions can alert bidders to conditions where a large degree of uncertainty exists. This enables contractors to focus on specific risks and encourages contingency planning for potential problems. Halligan et al. concluded that as claims were considered at progressively higher levels in the owner's organization, payment to contractors increased. The longer a claim remained unresolved, the more likely the contractor will attribute cost overruns or delays to the claims situation. Their study demonstrated that contractors expected owners to share in the risk and could, through the courts, enforce that premise.

IMPROVED CONTRACTING PRACTICES FOR UNDERGROUND CONSTRUCTION

U.S. National Committee on Tunneling Technology (USNCTT)

In response to a flood of costly litigation, USNCTT began pioneer studies in 1974 to improve contracting practices. Owners were urged to accept more equitable allocation of the risk of unknown underground conditions and to include contract provisions that would define that risk. USNCTT recommendations are detailed in "Avoiding and Resolving Disputes during Construction," prepared by Underground Technology Research Council (UTRC 1991). In summary, the recommendations are:

- Include a changed-condition clause in the construction contract.
- Disclose fully in contract documents all geological data and interpretation.

- Eliminate disclaimers that discount the value of the included geotechnical data.
- Escrow bidding documents that show assumptions and pricing of the successful bidder.
- Include a geotechnical design summary report (GDSR) in contract documents.
- Establish in the contract a dispute review board (DRB) for expeditious review and settlement of disputes.

Increasingly, these recommendations are utilized by owners on major underground construction. However, placing successful bidding documents in escrow remains a matter of contention. Contractor groups argue that bidding documents are trade secrets and that absolute confidentiality of escrowed material cannot be assured. For example, Caltrans withdrew this requirement and sought an acceptable alternative in their efforts to improve risk allocation in their contracts (ENR 1993).

Experience with Dispute Review Boards

One of the earliest applications of a DRB was for the second bore of the Eisenhower Tunnel, in Colorado, 1975 to 1979. By late 1991, 100 projects, or one-half of the tunnels bid in the United States in 1990, had dispute review boards—on work that amounted to about 70% of the year's dollar value (Rubin et al. 1991). In 21 completed projects, 63 recommendations for settlement had been made and accepted without subsequent litigation. Typical costs of maintaining the DRB have ranged from 0.1% of total construction cost on large projects to 0.3% on smaller jobs, with fees split between owner and contractor (UTRC 1991). Basic advantages are as follows: using specialists familiar with the project work, close contact with job progress and problems, and rapid resolution of disputes using on-site personnel.

Effect of the Scale of the Geotechnical Investigation

Midway in the 20 years of attempted improvement since 1974, USNCTT (1984) examined the effect of scale of the geotechnical investigation on claims on 87 major underground projects in the prior 20 years. The study concentrated on tunnels, chambers, and shafts; and omitted earthwork, foundations, and general civil engineering. Their study endorsed USNCTT recommendations for improved contracting, with these conclusions:

- Of the studied projects 60% involved claims, of which 95% were for substantial amounts. Sixty-four percent of these claimed amounts was ultimately awarded. The impact was to add 12% to overall construction costs, not including legal extras.
- Typical exploration was equivalent to four borings per 1,000 route-feet of tunnel; that is, 250 ft average spacing of borings along the route, equivalent to about 0.4 lineal ft (lf) of borings per route-foot.
- Claims defense starts to improve at a quota of about 0.6 lf of borings per route-foot of tunnel; corresponding to the level at which exploration cost exceeds about 1% of project cost.
- Ideally, exploration cost may be justified up to about 3% of project costs, producing roughly the equivalent of about 1.5 lf of borings per route-foot of tunnel.

USNCTT (1984) also recommended development of exploration techniques in several areas:

- Improve investigation methods and predictions for three particular topics: in-situ stress state, "standup" time in

mined excavations and tunnels, and ground-water factors—effects on stability in tunneling and influence of water chemistry.

- Expand the capabilities of geophysical and other indirect sensing methods to extrapolate boring information to some useful distance beyond the borehole. Subsurface information could be greatly enhanced for depths too great for economical borehole penetration.
- Improve horizontal drilling techniques that can recover rock core and penetrate long distances without wandering off line. The need is great for tunnels beneath mountainous terrain, where borings to planned grade can be prohibitively expensive.

ALTERNATIVE DISPUTE RESOLUTION (ADR) PROCEDURES

The dispute review board recommended by USNCTT-UTRC is contract-stipulated, composed of three construction specialists chosen to represent the parties' interests evenly. It holds periodic site meetings and has hands-on involvement with job problems. Its decisions are immediate, or at least expedited, and although nonbinding, are available as evidence in further proceedings.

Independent of the USNCTT-UTRC initiatives, the American Arbitration Association has long offered an alternative to litigation. Arbitration may be contract-stipulated, establishing one to three arbitrators to settle claims. Hearings may be formal, with counsel present, using rules of evidence, and producing a binding final decision. Another alternative is mediation, a negotiation process facilitated by a single professional mediator. Usually mediation is voluntary, undertaken in an early stage of dispute and does not force a decision that binds the parties.

Each of these three resolution procedures is an effort to avoid the cost, time, and abrasion of litigation. Attorneys (Ostrower 1991) have complained that ADR stresses an equitable solution, rather than a legal interpretation of the contract and does not offer the guarantees provided by a formal legal proceeding. In fact, arbitrators' decisions may be contrary to both common law and contractual provisions. A dispute review board may have this fault, but its effect is less egregious than in binding arbitration. The specialists of a DRB should be able to reach a consensus that does not depart from the contract and that will simply be given appropriate weight if there is a subsequent legal appeal.

"Partnering" has recently been developed (Perini 1993) to meld the owner's and contractor's teams to solve problems at the working level before they can escalate to claims. The key feature is to work out the optimum solution without assigning costs to the parties, leaving that to whatever further proceeding has been mandated. Partnering can be coupled readily with the work of a DRB. Attorneys comment that partnering creates potential liability for the owner's engineers by burdening them with a duty to the contractor. In the absence of partnering, the engineers may be protected by privacy, that is, the lack of a contract between them and the contractor.

The Observational Method to Control Disputes

The observational method of limiting potential claims was introduced on the first New Austrian Tunneling Method (NATM) contract for Washington Metro Section B010, in 1984 (Martin 1987). For the contractor's value engineering change proposal the rock tunnel was separated into three ground types each with its associated specific mining and support requirements, paid for on a unit price per lineal foot of tunnel basis. The length of tunnel in each ground type was

estimated from the geotechnical information, and inclusive unit prices for each type were agreed upon. The decision to change from one type of ground to another was made at the heading by the contractor's superintendent and the owner's resident engineer. Thus the dispute narrowed to the straightforward matter of assigning the appropriate length of tunnel in each pay category based on conditions actually revealed in-situ. In effect, this is an application of the observational method to suppress disputes. The method has been used with marked success on several other recent U.S. rock tunnels with complex geology.

In a similar manner, current tunneling for the Mid-City E Route of the Washington Metro has included owner-designed minimum programs for dewatering and pregrouting to stabilize potential running ground in the tunnels. Lump sums for these minimum programs are included in the bid. Necessary additions are made at unit prices stated in the proposal and based on continuing measurements of effectiveness of the minimum program. Thus, disputes about necessary stabilization are reduced to a simple decision over the add-on quantity, eliminating the question of entitlement and shrinking the scale of extra compensation.

THE GEOTECHNICAL DESIGN SUMMARY REPORT

If the owner is persuaded to follow improved contracting practices the geotechnical engineers' role is enlarged and enhanced. They must carry out exploration to inform both designer and builder with the care appropriate for the project and setting. "Factual" information (boring logs, survey data, observations of ambient conditions, laboratory test results, and the like) should be assembled in an objective geotechnical data report free from speculation. The engineer must then author or contribute significantly to a geotechnical design summary report (GDSR) and should play a decisive role in problem-solving and dispute resolution.

Basic recommendations on the content of the GDSR are given in Appendix E, UTRC (1991). Suggestions for care in the writing are given by Austin (1994). Respecting the GDSR, R. B. Peck, writing for the Washington Metro Board of Engineering Consultants (BOEC 1992), fashioned the following advice:

The concept of geotechnical design summary reports as developed for this project upon Board recommendation, has proven generally useful throughout the underground construction industry as a means to convey to all concerned parties the considerations upon which the design is based. . . . Geological features pertinent to the project at hand should be presented in detail and their significance pointed out with respect to design, construction and future performance. The reasons for selected designs and construction procedures and specification requirements should be given in detail. . . . However, many sections in some of the present drafts tend to repeat common or textbook-style knowledge, such as criteria for allowable settlement or descriptions of instrumentation. Such paragraphs tend to turn off the serious and knowledgeable reader. . . . In general, the Board suggests that the importance of the GDSR justifies its preparation by engineers of substantial, broad experience in both design and construction. The preparation is not a task for junior professionals.

The GDSR's function is not to repeat factual information given in the geotechnical data report but to describe conclusions used in design and in framing specifications. It is particularly important that bidders understand the factors that have guided specifications relating to means and methods, temporary construction, and potential hazards. In framing

the GDSR, engineers should recognize the issues that lead to claims and the pitfalls in executing their investigation. This paper hopes to aid in that recognition.

Bidder's Interpretation of the Factual Data

Using the factual geotechnical data report, consultants retained by a bidder will make their own interpretation of underground conditions to be encountered. That consultant might well reach an opinion of the ground that is more favorable than the opinion given in the GDSR. Then when a DSC claim is submitted, the contractor sets forth the consultant's opinion to represent what they anticipated in preparing their bid (Samuels, unpublished memorandum, 1994). If the GDSR is to serve its intended purpose it alone must be the geotechnical baseline for expected conditions. To this end it may be appropriate to dictate such terms as:

In the event of apparent conflicts or discrepancies with any other geotechnical data made available to the Contractor, including geotechnical reports commissioned by the Contractor or obtained during the bidding period and prior to the Contract award, the GDSR shall take precedence.

To emphasize its role as the "baseline," a revised UTRC guideline now in preparation recommends a name change from GDSR to "geotechnical baseline report" (Austin 1994).

FACTORS CONTRIBUTING TO DSC CLAIMS

Procedural Errors

Moorhouse and Millet (1994) have provided a useful summary of the management and communication difficulties that lead to failures in the delivery of geotechnical consulting services. Our concern here is the specific area of site characterization. Errors made in preparing contract geotechnical information that increase the owner's vulnerability to a DSC claim include the following:

- Specially produced background information or samples are not disclosed for contractors' prebid review. These include drillers' field boring logs, which, however untidy, may contain vital information on difficulties during drilling or records of fluctuating ground water.
- Contradictions remain in the "factual data" made available. For example, various boring log versions are not reconciled, the differences are unexplained, and the log of record is not conformed to reflect final laboratory information.
- Vital ancillary data that are available are not collected or clearly presented. These data could include information on former site uses or structures; existing utility locations and conditions; potential interferences, contamination, or restrictions on site use.
- The geotechnical investigation has not been brought up to date to match the character and layout of the structure as finally designed and specified.
- Geotechnical reports confuse "fact" with speculation and offer unnecessary opinion on means and methods that should be left to the bidders' evaluation utilizing a complete set of facts.
- Geotechnical reports contain advice or recommendations contradicting the specifications requirements.
- Conflicting report drafts or contradictory speculations relating to site conditions are allowed to remain in the job file. They should be eliminated at an appropriate early date. It is improper, if not illegal, to do so after a claim has been presented. Before this happens, the job file

should be reviewed as if it were being examined by the contractor's counsel.

- Contract documents contain disclaimers that disparage the geotechnical investigation to such an extent that its results could be disregarded at the bidders' option.

Problems of Site Characterization

For claims related to the site characterization, the most obvious deficiencies have the most troublesome consequences, such as incorrect or omitted ground-water information, failure to record obstructions encountered in borings, confusing or ambiguous boring log descriptions. These elementary errors are not considered further. Our concern is with four groups of problems that elude a competent exploration:

- The claim arises from a contradiction of the familiar local geology or of well-understood job experiences. It is a surprise. Examples include unexpected sources of heavy recharge, impinging man-made structures, and effects of natural processes such as solution features.
- The crucial property is too elusive to be detected by ordinary methods of exploration and sampling. Examples are the common underprediction of the proportion of boulders, soil is not identified as vulnerable to hydro-compaction, and unexpected acidic inflow develops from sulfate inclusions in surrounding ground.
- The controlling geotechnical property is misunderstood or misinterpreted. This is essentially a limitation in the state of the art. Examples include failure to distinguish unstable soil structure that is vulnerable to running or piping, and unrecognized effect of pore pressure and volume changes caused by pile-driving.
- The scale of the discontinuity or anomaly is finer than the precision of the exploration—a "minor geologic detail." Examples are the controlling effects of dip-slip shears, interference by thin cemented layers such as ironpan, and the effect on stability of secondary structure in hard clays.

Case histories of a group of DSC claims and their resolution are described in the following four sections. These are only a few selected examples to illustrate difficulties in site characterization that lead to the unexpected in construction.

CASE A: CHARLESTON SEWER TUNNELS

Background of the Project

Five tunnels have been excavated in the Cooper Marl near Charleston, S.C., since 1927: four for water supply, the fifth for the sewerage system. Their locations are shown in Fig. 1. Except for limited reinforcement at shafts, all the water tunnels are unlined. The sewer tunnels were supported by ribs and lagging with sewer pipes laid in the invert. Tunnels were 7 or 8 ft (2.1 or 2.4 m) high; 60–110 ft (18–33 m) deep below ground or river level. The Edisto water tunnel was mined by local labor in two segments in 1928 and 1937. The sewer tunnels were excavated 8 mi (13 km) around Charleston harbor between 1967 and 1970. The two later water tunnels were constructed in 1978 and 1983.

All were mined in the Cooper Marl, a marine sediment of late Eocene age. The formation tilted seaward during deposition, dipping southeast at 1 in 500. It is heavily overconsolidated, olive-green fine sandy silt, remarkably homogeneous, with little evidence of stratification. Typical test properties are:

- Unified Soil Classification: ML and MH; $LL = 45$; $PL = 25$

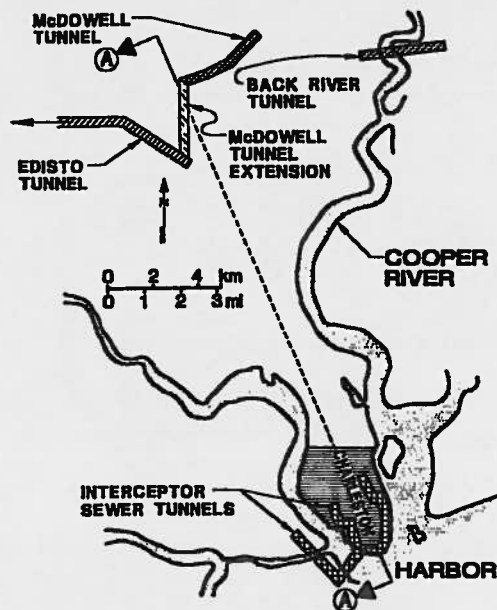


FIG. 1. Location of Charleston Tunnels

- Natural water content = 40–55%
- Dry unit weight = 70–75 pcf (11–12 kN/m³)
- Soluble carbonates = 60–75% of dry weight
- Compressive strength from unconfined tests on 3 in. (7.5 cm) samples = typically 5–10 ksf (250–500 kPa)

The Tunnel Stability Question

Based on the prewar experience with the Edisto tunnels, the city was convinced that the same simple unsupported tunnel, excavated by modern methods, could accommodate the downtown sewer. By elementary elastic theory, vertical compressive stress on the wall of the unsupported sewer tunnel at the springline equals two times the total overburden pressure. For the downtown sewer tunnel, total overburden ranged from 8 to 11.5 ksf (400–575 kPa), with equivalent compressive stress on the tunnel wall of 16–23 ksf (800–1,150 kPa), increasing as the tunnel deepened toward the treatment plant. With test compressive strengths of boring samples at less than 10 ksf (500 kPa), the geotechnical engineer believed the tunnels could not be driven without support.

Because of that concern it was decided that a small hand-excavated test tunnel at a depth of 80 ft (24 m) should be dug to assess stability. From the tunnel excavation, 6 in. (15 cm) samples were hand cut and tested unconfined. These failed brittly at unconfined compressive strengths of 14–16 ksf (700–800 kPa). The test tunnel appeared stable under 8.5 ksf (425 kPa) overburden, and the true compressive strength was judged to be at least 18 ksf (900 kPa). The calcareous cementation responsible for the exceptional strength was perceived as highly vulnerable to breakdown during sampling.

The design team then decided to set up contracts for the tunnel to be unlined, with support only at shafts, portals, and certain safety areas. The test tunnel appeared to be perfect, but 8 months after excavation its walls began to slab off, failing in a classic unconfined compression. By then contracts had been let for major sections of unsupported sewer tunnel. As the tunnel deepened going downgradient compressive stresses at the Springline increased from 18 to 20 to 22 ksf (900 to 1,000 to 1,100 kPa). Concurrently, standup time decreased from weeks to days to hours. Excavation was by road header, and as standup time decreased the installation of horseshoe ribs and lagging became more urgent, placed closer to the heading. This dramatic change in stability was ac-

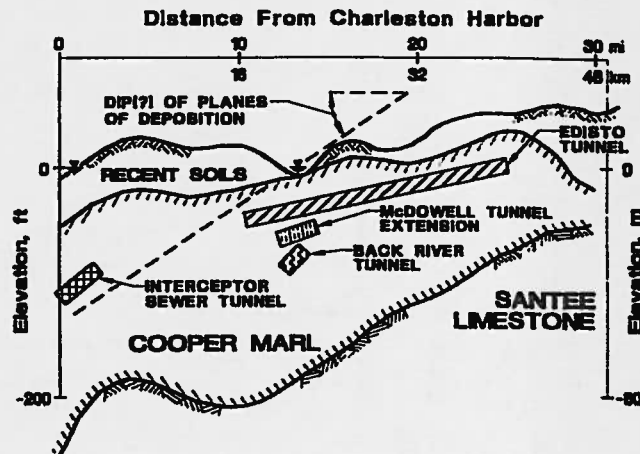


FIG. 2. Section A-A, Down-Dip in Cooper Marl

knowledgeed as an unforeseen condition and the contractor's multimillion dollar support costs became a negotiated extra.

Upland Water Tunnels

In 1978, when the upland water tunnels beneath the Back River and for the McDowell Extension were planned, the same design team was concerned with the conflicting experiences of the past. The recovery and testing of additional "undisturbed" samples could not demonstrate that test strengths exceeded compressive stress at the springline of the new tunnels. Finally it was realized that the upland water tunnels lay substantially deeper in the geological sequence of the Cooper Marl than the downtown sewer tunnels. This is illustrated by the position of the various tunnels with respect to the dipping plane of deposition shown in Fig. 2 on Geological Section A-A, taken down-dip in the Cooper Marl to Charleston Harbor. Thus upland water tunnels were in marl that was more heavily consolidated and stronger than the downtown sewer tunnels.

The real in-situ compressive strengths were probably significantly greater than the test strengths. In the geologically older marl at the water tunnels, true compressive strength was judged to be at least 20 ksf (1,000 kPa). Based on this conclusion, both the new water tunnels were bid with support only at portals and shafts but including unit prices for support as needed. The upland marl was near-ideal for tunneling and both conduits were mined, unsupported, by a Lovat wheel machine. The tunnel boring machine (TBM) set a record for that date, averaging 80–90 ft (24–27 m) per shift, with a maximum speed of about 2 1/2 times as great.

Summary

The design team had been frustrated by its inability to assess in-situ strength reliably. Table 1 compares the test strengths from 3 in. (7.5 cm) thin tube "undisturbed" samples and hand-cut 6 in. (15 cm) samples with probable in-situ strength and stress at the three postwar tunnels. The explanation for the difference in tunnel performance might have been deduced earlier by viewing the tunnels' position within the overall marl deposit. Because the designers and owner accepted the local experience of the past without closely examining the geological setting, mining of the sewer tunnels encountered a costly unforeseen extra.

CASE GROUP B: STANDUP TIME, TUNNELING IN SAND

One of the specters of soft-ground tunneling is the assessment of standup time in sand above the water table. It de-

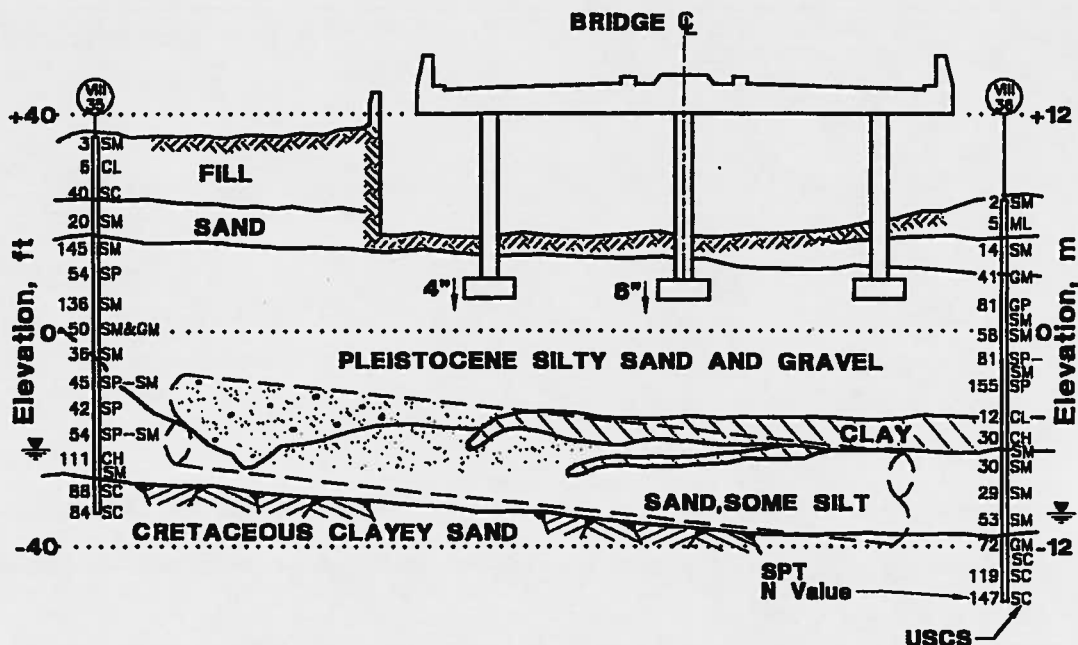


FIG. 3. Metro Tunnel beneath 10th St. Mall Bridge

TABLE 1. Compressive Strength of Cooper Marl Formation

Compressive strength or stress (1)	Charleston sewer interceptor (2)	McDowell extension tunnel (3)	Back River tunnel (4)
From unconfined compression of 3 in. O.D. thin-tube sampled from borings	9.2 ksf 460 kPa	8.6 ksf 430 kPa	5.2 ksf 260 kPa
From unconfined compression of 6 in. O.D. hand-cut samples from tunnels	15.0 ksf 750 kPa	14.4 ksf 720 kPa	13.4 ksf 670 kPa
Probable in-situ compressive strength	18–20 ksf 900–1,000 kPa	22–24 ksf 1,100–1,200 kPa	26–28 ksf 1,300–1,400 kPa
Compressive stress applied at tunnel spring-line	16–23 ksf 800–1,150 kPa	12–20 ksf 600–1,000 kPa	12–18 ksf 600–900 kPa
Average natural water content of Marl	52%	46%	44%

depends on a combination of soil properties: cementation, density, amount and plasticity of fines, gradation. But construction conditions are also important, including ground loads; drying or wetting, which controls capillary stress; construction vibrations; and method of excavation (Deere et al. 1969). It is a fertile subject of Type I DSC claims. The contractor's consultant typically contends that the sand's gradation is the key factor controlling standup time. The prebid gradation data may then be impeached by testing samples taken in-situ during tunneling. The Washington Metro subway provides two examples 15 years apart with different outcomes. The first was Section D004 (worked on 1974–75), in Pleistocene stream terrace sands just south of the Mall, in downtown Washington; the second was Section E005 (1989–90), in Cretaceous Coastal Plain sands beneath Fort Totten Park, in northeast Washington.

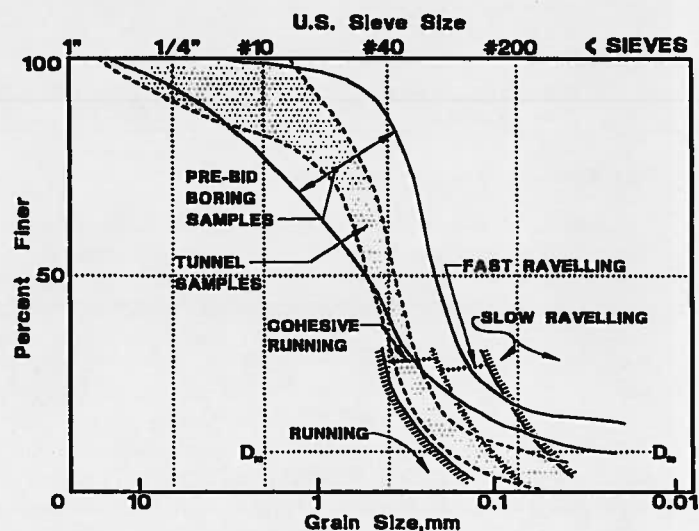


FIG. 4. Gradation of Tunneling Sands

Case B-1: Washington Metro Section D004

Fig. 3 shows a geological profile for Section D004. Twin, 21 ft (6 m) diameter subway tunnels were to cross beneath railroad tracks and the Tenth Street Mall Bridge. That bridge is founded on spread footings in terrace sands that contain scattered clay lenses. The two nearest borings showed chiefly very compact SM sands with 14–20% fines passing the No. 200 sieve. A gradation of prebid samples is shown in Fig. 4.

In the design phase, the Board of Engineering Consultants cautioned that the bridge deserved engineer-designed underpinning, but the designer left the choice of protection to the contractor. The records contained a detailed history of that controversy, eventually discovered and highlighted by the contractor's counsel.

Construction

The contractor tunneled using a digger shield with a single-breasting table. Deep excavations in the Mall had drawn down the water table near the tunnel invert. Then tunnel ventilation thoroughly dried the terrace sands. As a consequence of this

and incomplete breasting of the face, substantial runs occurred in the face, causing settlements of 4–6 in. (10–15 cm) of the bridge footings, distress in the structure, and anxiety to its owner. Settlements were less where the clay interbeds were thickest. The contractor added breasting in the face and performed extensive chemical grouting to stabilize the ground so that the second tunnel pass did not aggravate the experience of the first.

Contractor's Claim

The contractor submitted a claim for their extra costs asserting that prebid data indicated tunneling in slow raveling sands that instead proved unstable. To demonstrate a difference between the gradations portrayed in the documents and those encountered, the contractor's geotechnical consultant analyzed samples that had been obtained in the face during routine inspection by the construction manager. These samples exhibited a gradation in the shaded band in Fig. 4 in contrast to more broadly graded siltier sand represented in the prebid documents. He attributed instability particularly to an absence of fines, showing that the in-situ sand fell in the traditional range of "cohesive running." This difference in gradation reduced standup time from several hours for the expected slow raveling sand to a few minutes for the running ground. The discrepancy may have derived from the fact that prebid tests utilized specimens that mixed the lensed soils in the conventional split-spoon samples; whereas the tunnel samples were taken by scraping the face horizontally to separate individual lenses.

Decision

Claims on the Washington Metro are heard before the Corps of Engineers Board of Contract Appeals, whose members acknowledged a differing site condition (BCA 1981). The Board was impressed that the contractor built his case on tunnel samples taken by the owner to record in-situ conditions. The decision also observed that the owner had gone against their consultants' advice that underpinning of the bridge should be specified and paid for, choosing instead to make it an ancillary obligation of the contractor.

Case B-2: Washington Metro, Section E005

Section E005 was the first U.S. soft-ground NATM transit tunnel. It included complex mining in Cretaceous sands for a 280 ft (84 m) length of center-platform station. This employed five contiguous top drifts to be combined to form the roof arch with a row of center posts. Excavation was above the water table but beneath a park, which facilitated infiltration of rainfall. Specifications called for the first 80 ft (24 m) of these multiple drifts to be protected by a canopy created by overlapping horizontal jet-grouted cylinders. Contract documents now included a GDSR that described the Cretaceous sands to be mined:

The single size gradation and the lack of cohesive fines coupled with cemented sand layers interfering with vertical drainage make these sands vulnerable to piping, erosion and sloughing in the tunnel heading, even with a general drawdown of ground water.

... In summary, the Stratum P2 sand must be regarded as relatively unfavorable tunneling material which requires continuous and careful efforts to prevent movements by supporting the tunnel face and by proper blocking and packing behind temporary supports.

Construction

The contractor appealed successfully to omit the jet-grouting canopy, and instead attempted to stabilize the crown with driven poling boards and shotcrete applied immediately on the tunnel face. In the event, the work proved difficult and slow, with rate of advance only half that anticipated in the bid. The contractor attributed this to unstable sands surrounding the drifts and submitted a Type I DSC claim, asserting that the GDSR portrayed more stable sands than those encountered. This was supported by gradation tests on samples collected by the contractor to contrast with the prebid information.

Decision

The Board of Contract Appeals (BCA 1994) denied the claim, citing the GDSR description of the potentially unstable Cretaceous sands. They attributed much of the sands' mobility to vibrations from driving the overhead poling boards. The BCA expressed a healthy skepticism of the validity of in-situ samples taken in support of litigation:

The evidence associated with the collection of these samples by both parties strongly indicated that they were deliberately taken from areas of the excavation where experience indicated that soils would have been expected to show either a low or a high fines content.

... Finally, Appellant's (the contractor's) post-dispute soil sampling is of dubious probity, as pointed out in Finding 34.

Of course, the BCA denial was influenced by the contractor's use of an alternative method of roof stabilization rather than the specified jet grouting. Compare Metro cases B-1 and B-2 and note that, after long and tortured experience with running sand, the geotechnical engineers finally had the prudence to insert in the GDSR a description of its performance that was so pointed that it could not be safely ignored.

CASE GROUP C: TUNNELING AND EXCAVATION IN SOFT ROCK

Soft sedimentary rocks with unconfined compressive strengths in the range from 0.1 to 1 ksi (0.7–7 MPa) are mined as "soft ground." Their characteristics make it difficult to determine the strength that will impact excavation or tunneling methods. These materials include much Tertiary clay and shale of the West and lower Cretaceous and weathered crystalline rocks in the East Coast. Their test values of undrained strength are influenced by:

- The presence of significant cementation, interparticle bond, or intrinsic cohesion, which is damaged by sampling in test borings.
- An inherited fabric, foliation, or bedding, which has a moderate effect on test strength. The magnitude of its effect depends on the orientation of bedding with respect to the direction of failure loading.
- Frequently also a secondary structure (that is, discontinuities, fractures, and slickensides), which can have a great effect on test strengths (Johnston and Novello 1994). Spacing of discontinuities leads to a further effect of sample size on test results.

In both laboratory and field the degree of confinement can have a drastic influence on strength where bedding or fractures are significant. Thus, it is difficult to obtain a truly undisturbed sample and still more difficult to estimate the strength that controls the force or work needed for excava-

TABLE 2. Compressive Strength for Tunneling in Soft Rock

Case designation, project and formation (1)	LABORATORY RESULTS			
	GDSR Prebid		In-Situ Construction	
	Test value (2)	Procedure (3)	Test value (4)	Procedure (5)
Case C-1, Washington Metro F001, lowest Cretaceous sandy clay	Range 55 to 85 psi; 0.4 to 0.6 MPa	Driven thin-tube samples; tested triaxial with low to moderate chamber pressures	Average 170 psi, 1.2 MPa; maximum 260 psi, 1.8 MPa	Selected hand-cut samples; tested triaxial with high chamber pressures
Case C-2, Washington Metro E001, lowest Cretaceous sandy clay	Range 55 to 85 psi; 0.4 to 0.6 MPa Locally 15 to 25 psi; 0.1 to 0.2 MPa	As for Section F001 Driven thin-tube, tested unconfined	Average 27 psi; 0.19 MPa Average 17 psi; 0.12 MPa	Pitcher samples, tested unconfined California drive samples, tested unconfined
Case C-3, LA Metro A141, Miocene Fernando and Puente	Average 89 psi, 0.6 MPa; maximum 170 psi, 1.2 MPa	California drive samples, tested unconfined	Average 233 psi, 1.6 MPa; maximum 410 psi, 3.2 MPa	Hand-cut samples from tunnel, tested unconfined
Case C-4, LA Metro B251, Miocene Puente	Average 90 psi, 0.6 MPa; maximum 276 psi, 1.9 MPa	California drive samples, tested unconfined	Average 488 psi, 3.4 MPa; maximum 970 psi, 6.7 MPa	Core drill and hand-cut samples from tunnel, tested unconfined

tion. The delicate cementation of Cooper Marl of Case A is a simple example of the problem. Where secondary structure is marked, the assessment of strength is further complicated. The higher values of confined strength will influence the work needed to rip out the material or to fail it by pushing into the soft rock. Conversely, the lower strengths will tend to control stability of an excavated face where confinement is being removed.

Engineers trying to rationalize strengths for the GDSR may be confronted by a range of test values on samples of uncertain quality. They must assess the disturbance in sampling, estimate the different strengths in passive and active failure modes, and, finally, weigh the test data against actual experiences. Claimants have come to recognize that this confused state of affairs can be turned to their advantage in DSC claims. The following four cases illustrate the difficulty of designating an appropriate strength for the GDSR. Table 2 lists compressive strengths obtained by a variety of sampling and testing procedures to represent soft rock in these four examples. Compressive strength stated in pounds per square inch (psi) is the basic parameter used to characterize rock hardness for U.S. tunnelers.

Case C-1, Washington Metro, Section F001

Twin subway tunnels were mined in 1974 south across the Mall, on Seventh Street, in downtown Washington, 1,700 route-feet in length, at a depth of 30 ft to the crown. Subsoils consisted of Pleistocene terrace clay and sand overlying Cretaceous sandy clay. The Pleistocene clay was moderately preconsolidated and medium stiff. The Cretaceous strata appeared below the springline and geotechnical properties given in contract documents were:

- Unified Soil Classification SC, CH, MH: $LL = 60$; $PL = 30$
- Natural water content at or several percent below plastic limit
- Standard sampler penetration resistance 30 to 80; median, 50 blows per foot
- Overconsolidation exceeding overburden: 10–15 tsf (1–1.5 MPa)
- Undrained compressive strength = 55–85 psi (0.4–0.6 MPa)

Construction

The contractor used an ordinary digger shield, expecting to advance 30 ft (9 m) in three shifts, using ribs and lagging

for support and a backhoe in the face. About 10% of the material was assumed to be removed by hand. Because the hard Cretaceous clay appeared at the invert and rose in the heading toward the springline, the shield distorted, flattening the bottom arc. Cretaceous in the invert had to be mined with air tools ahead of the shield's cutting edge. The rate of advance dropped to 12 ft (4 m) a day, and shield distortion necessitated strengthening its interior structure. These conditions continued over about one-third of the tunnel length, after which the Cretaceous dropped back below the invert.

At completion the contractor claimed substantial extra costs and 80 lost work days because of hardness of the Cretaceous. The contractor's geotechnical consultant obtained a series of hand-cut samples from the heading, which yielded compressive strengths averaging 170 psi (1.2 MPa), with a maximum of 260 psi (1.8 MPa) from triaxial tests applying chamber pressure in the range of 40–70 psi (0.3–0.5 MPa). These confining stresses greatly exceeded in-situ lateral pressures and presumably were selected to maximize the test strengths.

Decision

The Board of Contract Appeals agreed that very hard Cretaceous clay had been encountered and that sampling in prebid borings had damaged its cementation, yielding low test strengths. However, they denied the claim on two points (BCA 1980). First, they believed that isolated Cretaceous above the invert was less impediment than inefficient mining and inept operation of the shield. Secondly, the contractor's bidding documents showed no evidence that the prebid test strength values had been given any attention at all in estimating crew size; tunneling rate; or the structural design of the shield, its jacking, or digging capacity. The BCA reached a conclusion that emphasizes the prudence of placing bidding documents in escrow:

Thus when after award (of the contract) a contractor asserts that delays or costs were caused by a differing site condition, we believe that the claimant must, by a fair preponderance of the evidence, persuade us that it actually read and relied on pre-award data to its detriment. Here, in our judgment Appellant has failed to carry this burden.

Case C-2: Washington Metro Section E001a Slurry Walls

Construction in a crowded neighborhood in 1986 required slurry-trench concrete walls at the cut-and-cover transit sta-

tion to protect nearby structures. Overburden consisted of alluvium underlain by basal Cretaceous clay. The slurry wall panels were to be 80 ft (24 m) deep, extending about 30 ft (9 m) into the Cretaceous. The clay was portrayed in the contract document as having properties similar to those in Case C-1 within the Mall tunnels. Sampler penetration resistance at this depth averaged 65 blows per foot. Compressive strengths from triaxial tests on good undisturbed samples averaged about 70 psi (0.5 MPa). However, at the wall's location, borings showed a number of values of unconfined compressive strength as low as 15–25 psi (0.1–0.2 MPa). These were affected by the presence of fractures and slickensides.

Panel excavation was performed by an experienced contractor, making about 60% of their anticipated rate of one 12-ft panel completed per day. Because they failed to reach their expected production, a DSC claim was filed. The contractor's geotechnical consultant undertook a boring and sampling program, presumably to demonstrate that the clay was harder than advertised. His investigation included a few samples taken with a Pitcher sampler, in which a thin tube follows directly behind the coring bit that clears the advance of the tube. This device produces as undisturbed a sample as practicable in these soft rocks. Many more samples were taken with the California drive sampler, 3-in. in diameter with inner liners, providing a 2-in. sample [Hvorslev (1949), page 274]. All samples were tested unconfined, yielding average compressive strengths of 27 psi (0.19 MPa) for the Pitcher samples and 17 psi (0.12 MPa) for the drive samples. Although these represent the best and worst methods of obtaining undisturbed samples in soft rock, slickensides in the clay reduced unconfined strengths to low values for both types of samples.

Decision

Disappointed by his low compressive strengths, the contractor took the position that they were entitled to make their own selection among the prebid data to assess the difficulty of trenching. Since unconfined compressive strength is a standard index of rock hardness, they chose to rely on the low prebid values near the wall location, rather than the high blow counts and medium high triaxial shear strengths. They complained that the owner had not sufficiently warned that fractures and slickensides unduly reduced the unconfined test values. The owner's contracting officer was not convinced by these arguments. Eventually, however, the claim was negotiated within the closeout agreement at the end of the job. The geotechnical engineer recognized that conflicting data in the prebid documents had to be reconciled or explained to the extent that there was a consistent baseline for bid.

Case C-3: Los Angeles Metro Section A141

The first tunnel section for the Los Angeles rapid transit system was mined in 1990 from Union Station to Hill Street, in the downtown area. The twin tunnels passed through coarse alluvium into siltstone, comprising 4,400 route-feet (1,300 m) north and south of the cut-and-cover Civic Center Station. The Miocene siltstone comprised Puente formation in the north tunnels and conformable Fernando on the south tunnels, with the contact actually occurring within the station excavation. These are very heavily overconsolidated, intact, lightly cemented, friable, finely laminated, soft sedimentary rocks. At the tunnel depth, median properties given in the prebid geotechnical report are:

- Unified Soil Classification ML and MH: $LL = 62$; $PL = 39$
- Natural water content = 29%

- Dry unit weight = 91 pcf (14 kN/m³)
- Standard sampler penetration resistance: Puente = 70; Fernando = 64 blows per foot
- Undrained compressive strength: average 89 psi (0.6 MPa); maximum = 170 psi (1.2 MPa)

Construction

The contractor employed a Robbins digger shield with a paddle-shaped hoe having a single ripping tooth, well adapted to the coarse-grained alluvium. Temporary support was by ribs and lagging, but since the Miocene was stable, only a light ground load pressed in on the support. Progress after the learning curve had been expected to be 90–100 ft (27–30 m) a day for two shifts. The shield jacking capacity was 82 kips per circumferential foot (1,200 kN/m), which was more than the reaction against the lightly loaded initial lining could provide. The rock dipped away from the face at a steep angle and the inherent weakness on the bedding planes did not facilitate ripping. Actual progress in the Miocene was 40–80 ft (12–24 m) a day. Maintenance downtime was high, and the contractor submitted a claim for unexpected "hard ground." This was substantiated by unconfined tests on hand-cut samples taken within the heading. These averaged 233 psi (1.6 MPa), 2 1/2 times the prebid average. The contractor attributed the shortfall in their production to the unanticipated strength of the siltstone, and they submitted a total cost claim that, with various other matters, amounted to one-third of the original bid.

Decision

Based on the disparity of compressive strengths, the DRB acknowledged a differing site condition. They noted, however, that advance rates increased with the mining of each successive segment of the four tunnels surrounding Civic Center Station, demonstrating the effect of the miner's increasing efficiency on speed. The DRB agreed with the owner's contention that in weak rock speed of advance is not simply a function of compressive strength but also depends on TBM characteristics (including available reaction to the jacking) plus rock attitude and secondary structures. For example, in a subsequent tunnel contract with the same machine, Puente with the same test strength dipped at a medium angle toward the face and when punctured with the ripping tooth, it simply slumped to the invert on bedding planes. An experienced team advanced as much as 180 ft (54 m) in a two-shift day.

From their detailed analysis of the record of mining progress the DRB recommended an award that was only 15% of the claim. The contractor indignantly rejected the DRB decision and filed suit (Denning 1993). This is a rare and notable example of a challenge to a DRB decision. However, then a wrap-up negotiation folded the claim with other matters into a settlement that diluted the hard rock claim to the extent that the DRB's computation was accepted by the contractor.

Case C-4: Vermont Street Tunnel, Hard Ground Dispute

In the case of Section B251 of the Los Angeles Metro, in the curve south from the Barnsdall shaft into Vermont Street, tunneling was in Miocene Puente, whose unconfined compressive strength was given in the GDSR as an average of 90 psi (0.6 MPa) with a maximum of 276 psi (1.9 MPa). As in case C-3, prebid test samples had been obtained with the California drive sampler with an inner liner. At the time the GDSR was being written, the A141 dispute was in progress. It then became clear that the drive sampler disturbed the Puente and gave low test strengths. Consequently a sentence

was added to GDSR noting that strengths could be as high as 6(N) psi (4.1 MPa).

Construction

In the drive around the 1,000 ft (300 m) radius curve, the contractor utilized a Robbins digger shield. Excavation averaged 57 ft (17 m) per day with 47% maintenance time, compared to the planned 91 ft (27 m) per day. Specifications limited the overcutter to 5/8 in. (16 mm) width. The overcutter is a band placed around the cutting edge perimeter to provide a small clearance to ease passage of the shield skin. In mining the curve the shield tended to oval because of constant pressure between its skin and unyielding Puente. Two breakdowns required major maintenance. The contractor had taken 46 core-drilled or hand-cut samples of the Puente in-situ to determine unconfined compressive strengths, which they suspected were greater than portrayed in contract documents. These averaged 488 psi (3.4 MPa) with a maximum of 970 psi (6.8 MPa). With these results the contractor submitted a claim citing the difference between in-situ strengths and those in GDSR.

Decision

As in cases C-1 and C-3, cited previously, the designers of the TBM were called for testimony before the dispute review board. Soft-ground TBM design concepts are proprietary and are generated by mechanical engineers with little attention to the subtleties of soil mechanics. The TBM designers acknowledged that within the strength range in dispute there was no direct relation between soil strengths and structural design of the shield or selection of the shove jack capacity.

The DRB (1994) recommended a 75-25 division of the extra costs of the hard ground between owner and contractor. They strongly criticized the inclusion of a single sentence in the GDSR that stated a probable maximum strength without explaining its derivation. The DRB agreed with the contractor that this could be construed as a stratagem to protect the owner, not a proper baseline for bid. DBR concluded that the difficulty with the TBM arose from the erroneous anticipation of soft and yielding Puente coupled with a restriction on the overcutter thickness, making damage to the shield inevitable on the tight curve. Their decision was made even acknowledging that so long as the material was considered "soft ground," the discrepancy in compression strength values had no direct role in the shield design.

CASE D: APPROACH FILL, BALTIMORE BRIDGE

A bridge constructed in the 1970s in Baltimore Harbor required a mile-long (1.6 km) approach embankment, filled 25–30 ft (8–9 m) above tide flats. The underlying soils consisted of 70 ft (21 m) of late Pleistocene clay overlying very compact Cretaceous sands. Fig. 5 is a cross section of the embankment and subsoils; Fig. 6 is a longitudinal section. This clay, laid down in initial postglacial drowning of the waterways, is ubiquitous in Mid-Atlantic estuaries. Later fluctuations in sea level exposed and desiccated the crust, overconsolidating the deposit. Its history and properties are similar to Boston blue clay. Its undrained shear strength is 2–3 ksf (100–150 kPa) in the dried crust and 1–1.5 ksf (50–75 kPa) below. When the clay was last exposed, channels were incised in its surface. With the most recent rise of sea level, those channels were filled with soft clay of 0.2–0.5 ksf (10–25 kPa) shear strength.

The designer carried out an extensive boring and laboratory test program focused on the soft channel-filling clays. Three to 4 ft (0.9–1.2 m) of consolidation settlement were predicted

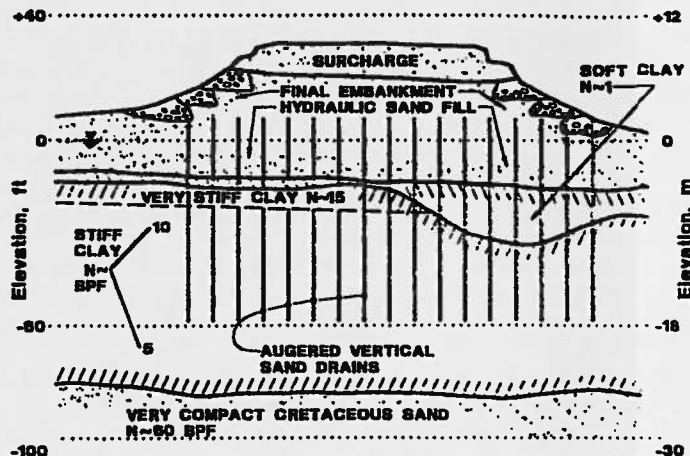


FIG. 5. Approach Fill Cross Section, Baltimore Bridge

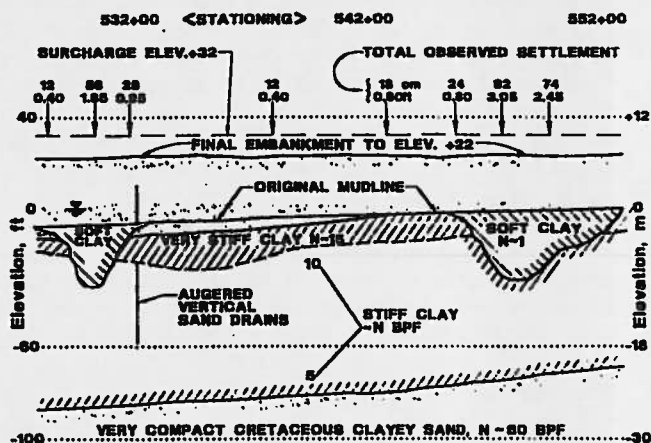


FIG. 6. Approach Fill Long Section, Baltimore Bridge

along with base instability in the soft channels. In the style of that time, the designer concluded that sand-drain stabilization with a 10 ft (3 m) surcharge was appropriate. Initial fill up to the working surface of the sand drains was placed hydraulically with outrigger berms. Specifications permitted drains installed by nondisplacement auger, by rotary drilling, or by using a casing with concurrent cleaning; 4,272 drains, typically 65 ft (20 m) deep, were required along 4,000 ft (1,200 m) of the embankment.

Construction

The experienced contractor intended to use an auger with overcutter with casing slipped down behind. The engineer rejected this procedure as nonconforming because of the small annular space left outside the casing. The contractor's attempt to prove its acceptability set off a protracted dispute with the engineer. Finally, the contractor changed to the hollow-stem auger that had been endorsed by the design team. While the auger worked as intended in the soft channel filling, the Pleistocene clay was so stiff and plastic that it stuck in the flights. If the auger was not cleaned after each use it acted as a displacement mandrel. With much disputation and delay, stabilization work advanced and the embankment was built. When surcharge was complete, pore-pressure measurements showed that 90–95% of primary consolidation was accomplished. The observed settlement was highly erratic, as shown at the top of Fig. 6, with the predicted 3–4 ft (0.9–1.2 m) over the soft channel filling but elsewhere only about 0.5 ft (15 cm).

Decision

The contractors filed a changed-condition claim since they were obviously frustrated by the fact that the soil to be stabilized was not the soft material they had expected through many years of experience with sand drains on the New Jersey Turnpike. Mandated arbitration extended interminably for a total of 30 sessions over a year's time. It was concluded by an agreement that awarded about 20% of the claimed extras. The award equaled only half the expense of arbitration.

This was a classic case of engineers so preoccupied with their design scheme that they failed to consider the difficulties of accomplishing the work. The technical information on soil strength was poorly organized for the contractor's prebid review. Even when the contractors assimilated that information, they could not believe there would be an overriding concern about the details of sand drain installation in so stiff a clay. A modern GDSR would have warned bidders of the interference of the stiff clay crust where a nondisplacement method of drain installation was considered vital.

EXPLORATION TO CONTROL CLAIMS

The preceding case histories illustrate that exploration focused too narrowly on design may be insufficient for construction. Acknowledging an elastic standard of care for subsurface exploration, both the basic geotechnical data report and the GDSR are essential to bidders when they assess constructability. The following guidance should be considered in the exploration. Since they derive from the writer's specific experiences these recommendations are necessarily subjective.

Before exploration, work out an overall geologic pattern sufficient for the field engineer to develop a sense of the stratification. Have the field engineer describe visible features—outcrops, exposures, open-water flow patterns, utilities, and old foundations.

Provide reasonable redundancy in the exploration, utilizing geophysical methods and in-situ testing where appropriate. Examine these results critically to determine whether they are consistent with borehole data. Inconsistencies should be explained.

Provide professional inspection of the exploration. Make the final boring logs of record consistent with laboratory test results. Make available the engineer's field log and the driller's log for bidders' inspection for the ancillary information they contain. Explain clearly the source and status of the various boring log versions and the greater reliability of the logs of record.

Describe on the boring log each sample and individual core run using a reproducible classification based on simple identification criteria, preferably the Unified Soil Classification System. Where details of rock quality are essential, provide color photographs of cores and core sketches illustrating rock fabric and discontinuities.

Insist on complete ground-water observations; explain anomalous readings; include sufficient observation wells or piezometers to provide a baseline of reliable readings. Obtain the background readings necessary to document daily or seasonal variations, as important. Consider that if water-level information is not shown on a boring log, it can be concluded that the water table is below the bottom of that boring.

Make clear to the bidders that almost invariably "perched" water is flowing above the nominal water table. This flow will respond to rainfall, the pattern of surface runoff and leaks in water-carrying utilities. An excavation above the water table will serve as a sump for perched water inflow. Identify sources of recharge supplying the perched water flow.

View skeptically all small-scale field permeability testing

in boreholes. Ordinarily, a rising-head test or a "slug" test with the slug being withdrawn is preferable to a falling-head test since the latter will tend to force fines dispersed in the drilling into the surrounding soil and throttle flow. Where reliable permeability data are essential, include at least a modest pumping test in the boring program.

In making borings, concentrate on interface conditions at a geologic unconformity. In overburden this is almost invariably an old erosion surface, frequently exhibiting weathering below and coarse pervious materials directly above. It can be the locus of a major change in strength, stiffness, and permeability.

Be especially careful when progressing the borehole by soil drilling and sampling methods in the transition zone between residual soil (saprolite) and weathered bedrock. High torque drill rigs are able to rip their way down in the transition zone by "soil drilling" techniques, thereby lowering the apparent top of bedrock in the boring record. Recognize that the energy input of ordinary excavating equipment per unit mass of soil is much less than that concentrated by powerful drill bits. Therefore material that can be removed as soil in the borehole may have to be handled as rock in the excavation or in the tunnel.

For important underground work stipulate "continuous" sampling at critical intervals in the borings. These could be split-spoon samples at the usual 5 ft intervals, with either an additional split-spoon sample or an "undisturbed" sample in the interval between. Avoid using hollow-stem augers in such important borings.

When soil samples are obtained by a thick-wall drive sampler or with a rotary core barrel, make clear in the GDSR that laboratory tests on these samples yield only identification data. If obtaining fundamental engineering properties is intended, then thin-tube "undisturbed" samples should be obtained using Hvorslev's (1949) criteria. A piston-head sampler must be available for use. A Pitcher sampler may be appropriate in hard cohesive material. A Becker hammer is useful in compact very coarse soil.

CONCLUSIONS

The overriding purpose of improved contracting practices is to provide a constructed product at least cost to the owner, with an appropriate profit to both the sponsor and constructor. Success of this process depends greatly on the performance of the geotechnical engineer. Unfortunately in today's market an "appropriate profit" for the contractor may have to be obtained through a changed-condition claim, and it is here that the geotechnical engineer plays a pivotal role. This paper cited progress in dispute resolution that has taken place in the last 20 years with the increasing involvement of the geotechnical engineer. Case histories illustrate the pitfalls that lead to claims requiring engineers to defend their work.

In the present highly competitive heavy-construction market, bidders commonly accept hazards without contingency in their proposal, expecting that "claimsmanship" will turn a marginal job into a profitable one. Bidding circumstances do not afford contractors the time or budget for an independent underground investigation. A contractor who suffers a loss, whether based on unexpected conditions or not, may attempt to recoup through the contract DSC provision. If the owners must pay a significant amount of the claim they may turn on their design team to seek compensation. One purpose of this paper is to assist geotechnical engineers in minimizing that possibility. Not only should the engineers ensure that their work meets the appropriate standard of care; they also must be on guard for special circumstances that can make them vulnerable and think in terms of the strategy of anticlaimsmanship. Finally, it is greatly in the interest of the geotech-

nical engineer to further an equitable adjustment of a changed condition claim rather than allow it to escalate to litigation.

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**ENGINEERING JUDGMENT IN THE EVOLUTION FROM
DETERMINISTIC TO RELIABILITY-BASED FOUNDATION DESIGN**

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ABSTRACT: Engineering judgment has always played a dominant role in geotechnical design and construction. Until earlier this century, most of this judgment was based on experience and precedents. The role of judgment in geotechnical practice has undergone significant changes over the past 50 years as a result of theoretical, experimental, and field developments in soil mechanics and, more recently, in reliability theory. A clarification of this latter change specifically is needed to avoid misunderstanding and misuse of the new reliability-based design (RBD) codes. This paper first provides a historical perspective of the traditional factor of safety design approach. The fundamental importance of limit state design to RBD then is emphasized. Finally, an overview of RBD is presented, and the proper application of this new design approach is discussed, with an example given of the ultimate limit state design of drilled shafts under undrained uplift loading. Judgment issues from traditional approaches through RBD are interwoven where appropriate.

INTRODUCTION

Almost all engineers would agree that engineering judgment is indispensable to the successful practice of engineering. Since antiquity, engineering judgment has played a dominant role in geotechnical design and construction, although most of the early judgment was based on experience and precedents. A major change took place in engineering practice when scientific principles, such as stress analysis, were incorporated systematically into the design process. In geotechnical engineering, in particular, significant advances were made following World War II, primarily because of extensive theoretical, experimental, and field research. The advent of powerful and inexpensive computers in the last two decades has helped to provide further impetus to the expansion and adoption of theoretical analyses in geotechnical engineering

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practice. The role of engineering judgment has changed as a result of these developments, but the nature of this change often has been overlooked in the enthusiastic pursuit of more sophisticated analyses. Much has been written by notable engineers to highlight the danger of using theory indiscriminately, particularly in geotechnical engineering (e.g., Dunnicliff & Deere 1984, Pocht 1994). For example, engineering judgment still is needed (and likely always will be!) in site characterization, selection of appropriate soil/rock parameters and methods of analysis, and critical evaluation of the results of analyses, measurements, and observations. The importance of engineering judgment clearly has not diminished with the growth of theory and computational tools. However, its role has become more focused on those design aspects that remained outside the scope of theoretical analyses.

At present, another significant change in engineering practice is taking place. Much of the impetus for this innovation arose from the widespread rethinking of structural safety concepts that was brought about by the boom in post-World War II construction (e.g., Freudenthal 1947, Pugsley 1955). Traditional deterministic design codes gradually are being phased out in favor of reliability-based design (RBD) codes that can provide a more consistent assurance of safety based on probabilistic analyses. Since the mid-1970s, a considerable number of these new design codes have been put into practice for routine structural design, for example, in the United Kingdom in 1972 (BSI-CP110), in Canada in 1974 (CSA-S136), in Denmark in 1978 (NKB-36), and in the U.S. in 1983 for concrete (ACI) and in 1986 for steel (AISC). In geotechnical engineering, a number of RBD codes also have been proposed recently for trial use (e.g., Barker et al. 1991, Berger & Goble 1992, Phoon et al. 1995).

The impact of these developments on the role of engineering judgment might be analogous to that brought about by the introduction of scientific principles into engineering practice. In this continuing evolution, it must be realized that RBD is just another tool, but it is different from traditional deterministic design, even though the code equations from both methods have the same "look-and-feel". These differences can lead to misunderstanding and misuse of the new RBD codes. For these reasons, it is necessary to: (a) clarify how engineering judgment can be used properly so that it is compatible with RBD, and (b) identify those geotechnical safety aspects that are not amenable to probabilistic analysis. In this paper, an overview is given first of the traditional geotechnical design approach from the perspective of safety control. The philosophy of limit state design then is presented as the underlying framework for RBD. Finally, the basic principles of RBD are reviewed, and the proper application of this new design approach is described with an example of the ultimate limit state design of drilled shafts under undrained uplift loading. As noted prominently in the paper title, engineering judgment is interwoven throughout.

TRADITIONAL GEOTECHNICAL DESIGN PRACTICE

The presence of uncertainties and their significance in relation to design has long been appreciated (e.g., Casagrande 1965). The engineer recognizes, explicitly or other-

wise, that there is always a chance of not achieving the design objective, which is to ensure that the system performs satisfactorily within a specified period of time. Traditionally, the geotechnical engineer relies primarily on factors of safety at the design stage to reduce the risk of potential adverse performance (collapse, excessive deformations, etc.). Factors of safety between 2 to 3 generally are considered to be adequate in most foundation design (e.g., Focht & O'Neill 1985). However, these values can be misleading because, too often, factors of safety are recommended without reference to any other aspects of the design computational process, such as the loads and their evaluation, method of analysis (i.e., design equation), method of property evaluation (i.e., how to select the undrained shear strength), and so on. Other important considerations that affect the factor of safety include variations in the loads and material strengths, inaccuracies in the design equations, errors arising from poorly supervised construction, possible changes in the function of the structure from the original intent, unrecognized loads, and unforeseen in-situ conditions. The manner in which these background factors are listed should not be construed as suggesting that the engineer actually goes through the process of considering each of these factors separately and in explicit detail. The assessment of the traditional factor of safety is essentially subjective, requiring only global appreciation of the above factors against the backdrop of previous experience.

The sole reliance on engineering judgment to assess the factor of safety can lead to numerous inconsistencies. First, the traditional factor of safety suffers from a major flaw in that it is not unique. Depending on its definition, the factor of safety can vary significantly over a wide range, as shown in Table 1 for illustration. The problem examined in Table 1 is to compute the design capacity of a straight-sided drilled shaft in clay, 1.5 m in diameter and 1.5 m deep, with an average side resistance along the shaft equal to 36 kN/m^2 and a potential tip suction of 0.5 atmosphere operating during undrained transient live loading. Five possible design assumptions are included. The first applies the factor of safety (F'S) uniformly to the sum of the side, tip, and weight components; the second applies the F'S only to the side and tip components; the third is like the first, but disregards the tip; the fourth is like the second, but disregards the tip; and the fifth is ultra-conservative, considering only the weight. It is clear from Table 1 that a particular factor of safety is meaningful only with respect to a given design assumption and equation.

Another significant source of ambiguity lies in the relationship between the factor of safety and the underlying level of risk. A larger factor of safety does not necessarily imply a smaller level of risk, because its effect can be negated by the presence of larger uncertainties in the design environment. In addition, the effect of the factor of safety on the underlying risk level also is dependent on the extent of conservatism in the selected design models and design parameters.

In a broad sense, these issues generally are appreciated by most engineers. They can exert additional influences on the engineer's choice of the factor of safety but, in the absence of a theoretical framework, it is not likely that the risk of adverse performance can be reduced to a desired level consistently. Therefore, the main weakness in traditional practice, where assurance of safety is concerned, can be attributed to the lack

TABLE 1. Design Capacity Example (Kulhawy 1984, p. 395)

Design Assumption	Design Equation	Q_{ud} (kN) for FS = 3	Q_u / Q_{ud} ("actual" FS)
1	$Q_{ud} = (Q_{su} + Q_{tu} + W) / FS$	170.7	3.0
2	$Q_{ud} - W = (Q_{su} + Q_{tu}) / FS$	214.2	2.4
3	$Q_{ud} = (Q_{su} + W) / FS$	108.9	4.7
4	$Q_{ud} - W = Q_{su} / FS$	152.4	3.4
5	$Q_{ud} = W / FS$	21.8	23.5

Note: Q_{su} = side resistance = 261.8 kN, Q_{tu} = tip resistance = 184.4 kN, W = shaft weight = 65.3 kN, Q_{ud} = design uplift capacity, FS = factor of safety, Q_u = available uplift capacity = $Q_{su} + Q_{tu} + W = 511.6$ kN

of clarity in the relationship between the method (factor of safety) and the objective (reduce design risk). To address this problem appropriately, an essential first step is to establish the design process on a more logical basis, known as limit state design.

PHILOSOPHY OF LIMIT STATE DESIGN

The original concept of limit state design refers to a design philosophy that entails the following three basic requirements: (a) identify all potential failure modes or limit states, (b) apply separate checks on each limit state, and (c) show that the occurrence of each limit state is sufficiently improbable. Conceptually, limit state design is not new. It is just a logical formalization of the traditional design approach that would help to facilitate the explicit recognition and treatment of engineering risks. In recent years, the rapid development of RBD has tended to overshadow the fundamental role of limit state design. Much attention has been focused on the consistent evaluation of safety margins using advanced probabilistic techniques (e.g., MacGregor 1989). Although the achievement of consistent safety margins is a very desirable goal, it should not be overemphasized to the extent that the importance of the principles underlying limit state design become diminished.

This fundamental role of limit state design is particularly true for geotechnical engineering. The first step in limit state design, which involves the proper identification of potential foundation failure modes, is not always a trivial task (Mortensen 1983). This effort generally requires an appreciation of the interaction between the geologic environment, loading characteristics, and foundation response. Useful generalizations on which limit states are likely to dominate in typical foundation design situations are certainly possible, as in the case of structural design. The role of the geotechnical engineer in making adjustments to these generalizations on the basis of site-specific information is, however, indispensable as well. The need for sound engineering judgment in the selection of potential limit states is greater in foundation design than in structural design because in-situ conditions must be dealt with "as is"

and might contain geologic "surprises". The danger of downplaying this aspect of limit state design in the fervor toward improving the computation and evaluation of safety margins in design can not be overemphasized.

The second step in limit state design is to check if any of the selected limit states has been violated. To accomplish this step, it is necessary to use a model that can predict the performance of the system from some measured parameters. In geotechnical engineering, this is not a straightforward task. Consider Fig. 1, which is the essence of any type of prediction, geotechnical or otherwise. At one end of the process is the forcing function, which normally consists of loads in conventional foundation engineering. At the other end is the system response, which would be the prediction in an analysis or design situation. Between the forcing function (load) and the system response (prediction) is the model invoked to describe the system behavior, coupled with the properties needed for this particular model. Contrary to popular belief, the quality of geotechnical prediction does not necessarily increase with the level of sophistication in the model (Kulhawy 1992). A more important criterion for the quality of geotechnical prediction is whether the model and property are calibrated together for a specific load and subsequent prediction (Kulhawy 1992, 1994). Reasonable predictions often can be achieved using simple models, even though the type of behavior to be predicted is nominally beyond the capability of the models, as long as there are sufficient data to calibrate these models empirically. However, these models then would be restricted to the specific range of conditions in the calibration process. Extrapolation beyond these conditions potentially can lead to erroneous predictions. Ideally, empirical calibration of this type should be applied judiciously by avoiding the use of overly simplistic models. Common examples of such an oversimplification are the sets of extensive correlations between the standard penetration test N-value and practically all types of geotechnical design parameters, as well as several design conditions such as footing settlement and bearing capacity. Although they lack generality, simple models will remain in use for quite some time because of our professional heritage that is replete with, and built upon, empirical correlations. The role of the geotechnical engineer in appreciating the complexities of soil behavior and recognizing the inherent limitations in the simplified models is clearly of considerable importance. The amount of attention paid to the evaluation of safety margins is essentially of little consequence if the engineer assesses the soil properties incorrectly or selects an inappropriate model for design.

Third, the occurrence of each limit state must be shown to be sufficiently impro-

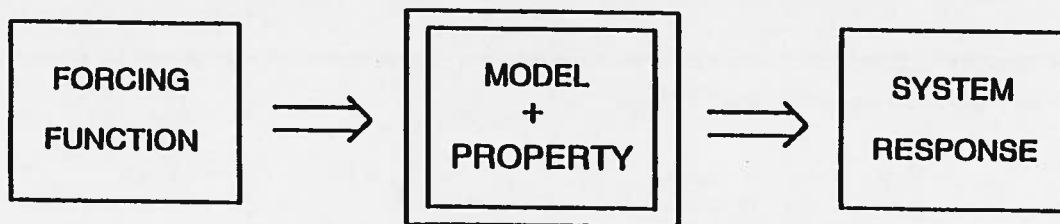


FIG. 1. Components of Geotechnical Prediction (Kulhawy 1994, p. 210)

hable. The philosophy of limit state design does not entail a preferred method of ensuring safety. Since all engineering quantities (e.g., loads, strengths) are inherently uncertain to some extent, a logical approach is to formulate the above problem in the language of probability. The mathematical formalization of this aspect of limit state design using probabilistic methods constitutes the main thrust of RBD. Aside from probabilistic methods, less formal methods of ensuring safety, such as the partial factors of safety method (e.g., Danish Geotechnical Institute 1985; Technical Committee on Foundations 1992), have also been used within the framework of limit state design.

In summary, the control of safety in geotechnical design is distributed among more than one aspect of the design process. Although it is important to consider the effect of uncertainties in loads and strengths on the safety margins, it is nonetheless only one aspect of the problem of ensuring sufficient safety in the design. The other two aspects, identification of potential failure modes and the methodology of making geotechnical predictions, can be of paramount importance, although they may be less amenable to theoretical analyses.

RELIABILITY-BASED DESIGN

Overview of Reliability Theory

The principal difference between RBD and the traditional design approach lies in the application of reliability theory, which allows uncertainties to be quantified and manipulated consistently in a manner that is free from self-contradiction. A simple application of reliability theory is shown in Fig. 2. Uncertain design quantities, such as the load (F) and the capacity (Q), are modeled as random variables, while design risk is quantified by the probability of failure (p_f). The basic reliability problem is to evaluate p_f from some pertinent statistics of F and Q , which typically include the mean (m_F or m_Q) and the standard deviation (s_F or s_Q). Note that the standard deviation provides a quantitative measure of the magnitude of uncertainty about the mean.

A simple closed-form solution for p_f is available if Q and F are both normally distributed. For this condition, the safety margin ($Q - F = M$) also is normally distributed with the following mean (m_M) and standard deviation (s_M) (e.g., Melchers 1987):

$$m_M = m_Q - m_F \quad (1a)$$

$$s_M^2 = s_Q^2 + s_F^2 \quad (1b)$$

Once the probability distribution of M is known, the probability of failure (p_f) can be evaluated as (e.g., Melchers 1987):

$$p_f = \text{Prob}(Q < F) = \text{Prob}(Q - F < 0) = \text{Prob}(M < 0) = \Phi(-m_M / s_M) \quad (2)$$

in which $\text{Prob}(\cdot)$ = probability of an event and $\Phi(\cdot)$ = standard normal cumulative

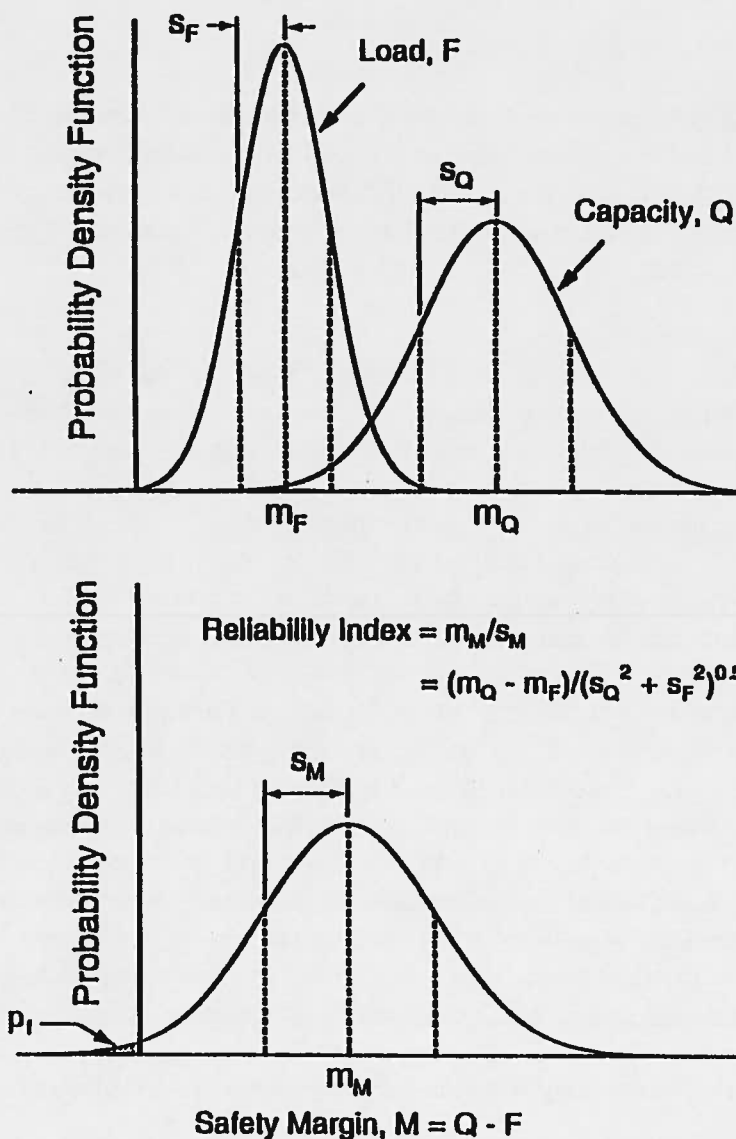


FIG. 2. Reliability Assessment for Two Normal Random Variables, Q and F

function. Numerical values for $\Phi(\cdot)$ are tabulated in many standard texts on reliability theory (e.g., Melchers 1987). The probability of failure is cumbersome to use when its value becomes very small, and this term carries the negative connotation of "failure". A more convenient (and perhaps more palatable) measure of design risk is the reliability index (β), which is defined as:

$$\beta = -\Phi^{-1}(p_f) \quad (3)$$

in which $\Phi^{-1}(\cdot)$ = inverse standard normal cumulative function. Note that β is not a new measure of design risk. It is just an alternative method for presenting p_f on a more convenient scale. A comparison of Eqs. 2 and 3 shows that the reliability index

for the special case of two normal random variables is given by:

$$\beta = m_M / s_M = (m_Q - m_F) / (s_Q^2 + s_F^2)^{0.5} \quad (4)$$

The reliability indices for most structural and geotechnical components and systems are between 1 and 4, corresponding to probabilities of failure ranging from about 16 to 0.003%, as shown in Table 2. Note that p_f decreases as β increases, but the variation is not linear. A proper understanding of these two terms and their interrelationship is essential, because they play a fundamental role in RBD.

Simplified RBD for Foundations

Once a reliability assessment technique is available, the process of RBD would involve evaluating the probabilities of failure of trial designs until an acceptable target value is achieved. While the approach is rigorous, it is not suitable for designs that are conducted on a routine basis. One of the main reasons for this limitation is that the reliability assessment of realistic geotechnical systems is more involved than that shown in Fig. 2. The simple closed-form solution given by Eq. 2 only is applicable to cases in which the safety margin can be expressed as a linear sum of normal random variables. However, the capacity of most geotechnical systems is more suitably expressed as a nonlinear function of random design soil parameters (e.g., effective stress friction angle, in-situ horizontal stress coefficient, etc.) that generally are non-normal in nature. To evaluate p_f for this general case, fairly elaborate numerical procedures are needed, such as the First-Order Reliability Method (FORM). A general description of FORM for foundation engineering is given elsewhere (e.g., Phoon et al. 1995) and is beyond the scope of this paper. At the present time, it is safe to say that most geotechnical engineers would feel uncomfortable performing such elaborate calculations because of their lack of proficiency in probability theory (e.g., Whitman 1984).

All of the existing implementations of RBD are based on a simplified approach that involves the use of multiple-factor formats for checking designs. The three main

TABLE 2. Relationship Between Reliability Index and Probability of Failure

Reliability Index, β	Probability of Failure, $p_f = \Phi(-\beta)$
1.0	0.159
1.5	0.0668
2.0	0.0228
2.5	0.00621
3.0	0.00135
3.5	0.000233
4.0	0.0000316

Note: $\Phi(\cdot)$ = standard normal probability distribution

types of multiple-factor formats are: (a) partial factors of safety, (b) load and resistance factor design (LRFD), and (c) multiple resistance factor design (MRFD). Examples of these design formats are given below for the case of uplift loading of a drilled shaft:

$$\eta F_n = Q_u(c_n/\gamma_c, \phi_n/\gamma_\phi) \quad (5a)$$

$$\eta F_n = \Psi_u Q_{un} \quad (5b)$$

$$\eta F_n = \Psi_{su} Q_{sun} + \Psi_{tu} Q_{tun} + \Psi_w W \quad (5c)$$

in which η = load factor, F_n = nominal design load, Q_u = uplift capacity, c_n = nominal cohesion, ϕ_n = nominal friction angle, γ_c and γ_ϕ = partial factors of safety, Q_{un} = nominal uplift capacity, Q_{sun} = nominal uplift side resistance, Q_{tun} = nominal uplift tip resistance, W = shaft weight, and Ψ_u , Ψ_{su} , Ψ_{tu} , Ψ_w = resistance factors. The multiple factors in the simplified RBD equations are calibrated rigorously using reliability theory to produce designs that achieve a known level of reliability consistently. Details of the geotechnical calibration process are given elsewhere (e.g., Phoon et al. 1995).

In principle, any of the above formats or some combinations thereof can be used for calibration. The selection of an appropriate format is mostly unrelated to reliability analysis. Practical issues, such as simplicity and compatibility with the existing design approaches, are important considerations that will determine if the simplified RBD approach can gain ready acceptance among practicing engineers. At present, the partial factors of safety format (Eq. 5a) has not been used for RBD because of three main shortcomings. First, a unique partial factor of safety can not be assigned to each soil property, because the effect of its uncertainty on the foundation capacity depends on the specific mathematical function in which it is embedded. Second, indiscriminate use of the partial factors of safety can produce factored soil property values that are unrealistic or physically unrealizable. Third, many geotechnical engineers prefer to assess foundation behavior using realistic parameters, so that they would have a physical feel for the problem, rather than perform a hypothetical computation using factored parameters (Duncan et al. 1989, Green 1993, Been et al. 1993).

This preference clearly is reflected in the traditional design approach, in which the modification for uncertainty often is applied to the overall capacity using a global factor of safety (FS) as follows:

$$F_n = Q_{un} / FS \quad (6)$$

A comparison between Eq. 6 and Eqs. 5b and 5c clearly shows that the LRFD and MRFD formats are compatible with the preferred method of applying safety factors. In fact, the load and resistance factors in the LRFD format can be related easily to the familiar global factor of safety as follows:

$$FS = \eta / \Psi_u \quad (7)$$

The corresponding relationship for the MRFD format is:

$$FS = \eta / (\Psi_{su} Q_{sun} / Q_{un} + \Psi_{tu} Q_{tun} / Q_{un} + \Psi_w W / Q_{un}) \quad (8)$$

Although Eq. 8 is slightly more complicated, it still is readily amenable to simple calculations. These relationships are very important, because they provide the design engineer with a simple direct means of checking the new design formats against their traditional design experience.

RBD EXAMPLE

The development of a rigorous and robust RBD approach for geotechnical engineering design, which also is simple to use, is no trivial task. Since the early 1980s, an extensive research study of this type has been in progress at Cornell University under the sponsorship of the Electric Power Research Institute and has focused on the needs of the electric utility industry. Extensive background information on site characterization, property evaluation, in-situ test correlations, etc. had to be developed as a prelude to the RBD methodology. This work is summarized elsewhere (Spry et al. 1988, Orchant et al. 1988, Philippas et al. 1988, Kulhawy et al. 1992). Building on these and numerous studies by others, ultimate and serviceability limit state (RBD) equations were developed for drilled shafts and spread foundations subjected to a variety of loading modes under both drained and undrained conditions (Phoon et al. 1995). The results of an extensive reliability calibration study for ultimate limit state design of drilled shafts under undrained uplift loading are presented in Tables 3 and 4 and are to be used with Eqs. 5b (LRFD) and 5c (MRFD). All other limit states, foundation types, loading modes, and drainage conditions addressed have similar types of results, with simple LRFD and MRFD equations and single

TABLE 3. Undrained Ultimate Uplift Resistance Factors For Drilled Shafts
Designed Using $F_{50} = \Psi_u Q_{un}$ (Phoon et al. 1995, p. 6-7)

Clay	COV of s_u (%)	Ψ_u
Medium (mean $s_u = 25 - 50 \text{ kN/m}^2$)	10 - 30	0.44
	30 - 50	0.43
	50 - 70	0.42
Stiff (mean $s_u = 50 - 100 \text{ kN/m}^2$)	10 - 30	0.43
	30 - 50	0.41
	50 - 70	0.39
Very Stiff (mean $s_u = 100 - 200 \text{ kN/m}^2$)	10 - 30	0.40
	30 - 50	0.37
	50 - 70	0.34

Note: Target reliability index = 3.2

TABLE 4. Undrained Uplift Resistance Factors For Drilled Shafts Designed Using $F_{50} = \Psi_{su}Q_{sun} + \Psi_{lu}Q_{lun} + \Psi_w W$ (Phoon et al. 1995, p. 6-7)

Clay	COV of s_u (%)	Ψ_{su}	Ψ_{lu}	Ψ_w
Medium (mean $s_u = 25 - 50 \text{ kN/m}^2$)	10 - 30	0.44	0.28	0.50
	30 - 50	0.41	0.31	0.52
	50 - 70	0.38	0.33	0.53
Stiff (mean $s_u = 50 - 100 \text{ kN/m}^2$)	10 - 30	0.40	0.35	0.56
	30 - 50	0.36	0.37	0.59
	50 - 70	0.32	0.40	0.62
Very Stiff (mean $s_u = 100 - 200 \text{ kN/m}^2$)	10 - 30	0.35	0.42	0.66
	30 - 50	0.31	0.48	0.68
	50 - 70	0.26	0.51	0.72

Note: Target reliability index = 3.2

corresponding tables of resistance factors. In these equations, the load factor is taken as unity to maintain statistical compatibility with the ASCE loading guide (Task Committee on Structural Loadings 1991), while the nominal load is defined as the 50-year return period load (F_{50}), which is typical for electrical transmission line structures. Note that the resistance factors depend on the clay consistency and the coefficient of variation (COV) of the undrained shear strength (s_u). The COV is an alternative measure of uncertainty that is defined as the ratio of the standard deviation to the mean. The clay consistency is classified broadly as medium, stiff, and very stiff, with corresponding mean s_u values of 25 to 50 kN/m^2 , 50 to 100 kN/m^2 , and 100 to 200 kN/m^2 , respectively. Foundations are designed using these new RBD formats in the same way as in the traditional approach, with the exception that the rigorously-determined resistance factors shown in Tables 3 and 4 are used in place of an empirically-estimated factor of safety.

Target Reliability Index

Before applying these resistance factors directly in design, it is important to examine the target reliability index for which these resistance factors are calibrated. At the present time, there are no simple or straightforward procedures available to produce the "correct" or "true" target reliability index. However, important data that can be used to guide the selection of the target reliability index are the reliability indices implicit in existing designs (Ellingwood et al. 1980). An example of such data for ultimate limit state design of drilled shafts in undrained uplift is shown in Fig. 3, in which a typical range of COV of s_u , mean s_u normalized by atmospheric pressure (p_a), and global factor of safety are examined for a specific geometry. The reliability indices implicit in these existing global factor of safety designs lie in the approximate range of 2.6 to 3.7. A target reliability index of 3.2 is representative of this

UNCERTAINTY IN GEOLOGIC ENVIRONMENT

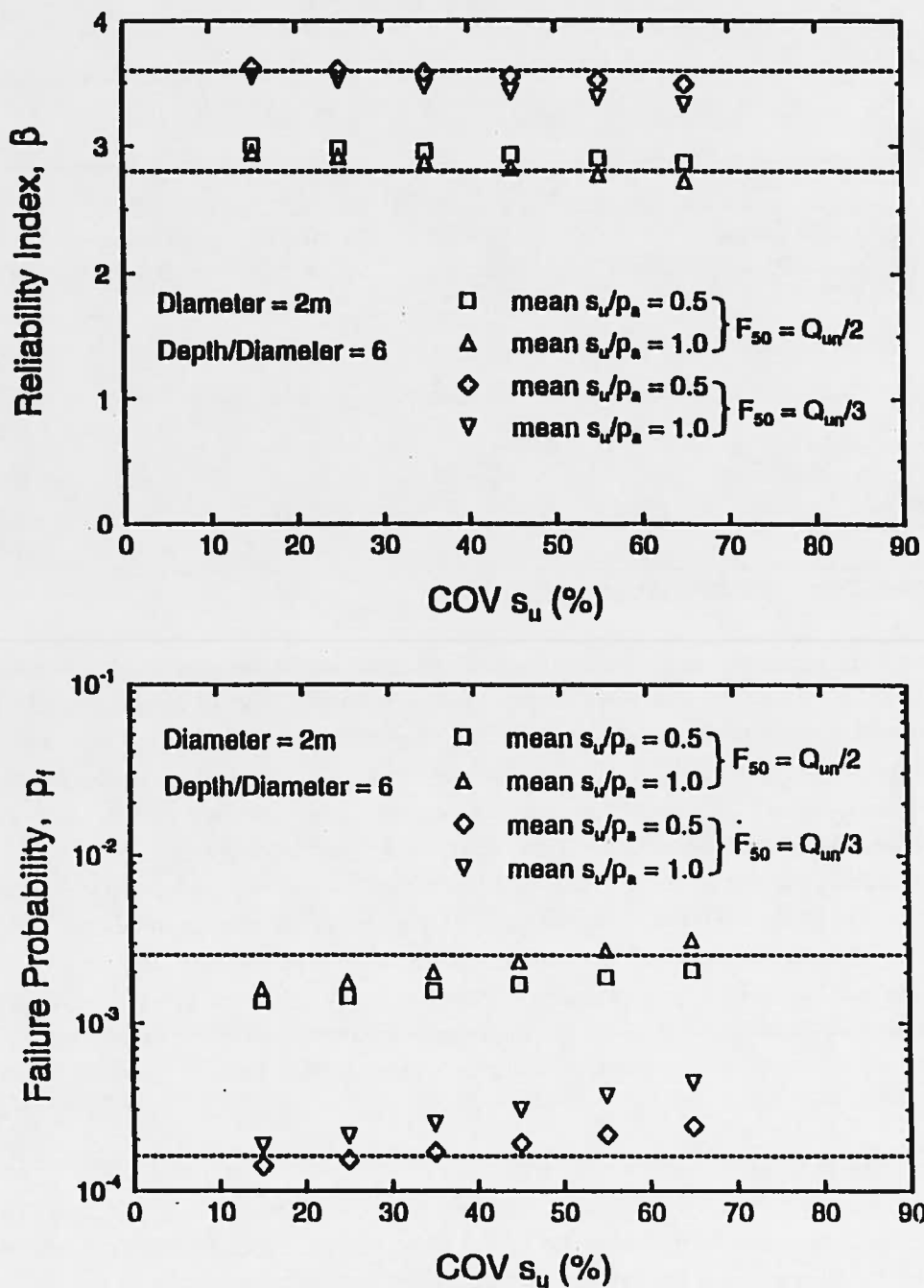


FIG. 3. Reliability Levels Implicit in Existing Ultimate Limit State Design of Drilled Shafts in Undrained Uplift

range. Similar ultimate limit state reliability studies for other parametric variations and loading modes under both drained and undrained conditions also strongly support the use of this target value (Phoon et al. 1995). While this approach is partially empirical, it possesses a major advantage of keeping the new design methodology com-

patible with the existing experience base.

Other important data to consider include the failure rates estimated from actual case histories. However, these failure rates can not be used directly for assessing the target reliability level because the theoretical probability of failure obtained from reliability theory usually is one order of magnitude smaller than the actual failure rate (e.g., CIRIA 1977, Livingstone 1989). This result is not surprising, because the safety of a design is not affected by uncertainties underlying design calculations alone. It also can be compromised severely by factors such as poor construction and human errors. An example of empirical rates of failure for civil engineering facilities and the related costs of failure is given in Fig. 4. For foundations, the empirical rate of failure lies between 0.1 and 1%. This failure rate implies a theoretical probability of failure in the range of 0.01 to 0.1%. Therefore, in terms of the reliability index, the currently accepted risk level is between 3.1 and 3.7. A target reliability index of 3.2 also is consistent with this range.

The above discussion highlights some of the important considerations that are involved in the determination of the target reliability index. It is apparent that the target reliability index can not be adjusted casually without extensive prior calibration with existing practice. Different target reliability indices can be used for design, but specific guidelines always should be given on the conditions for which each target val-

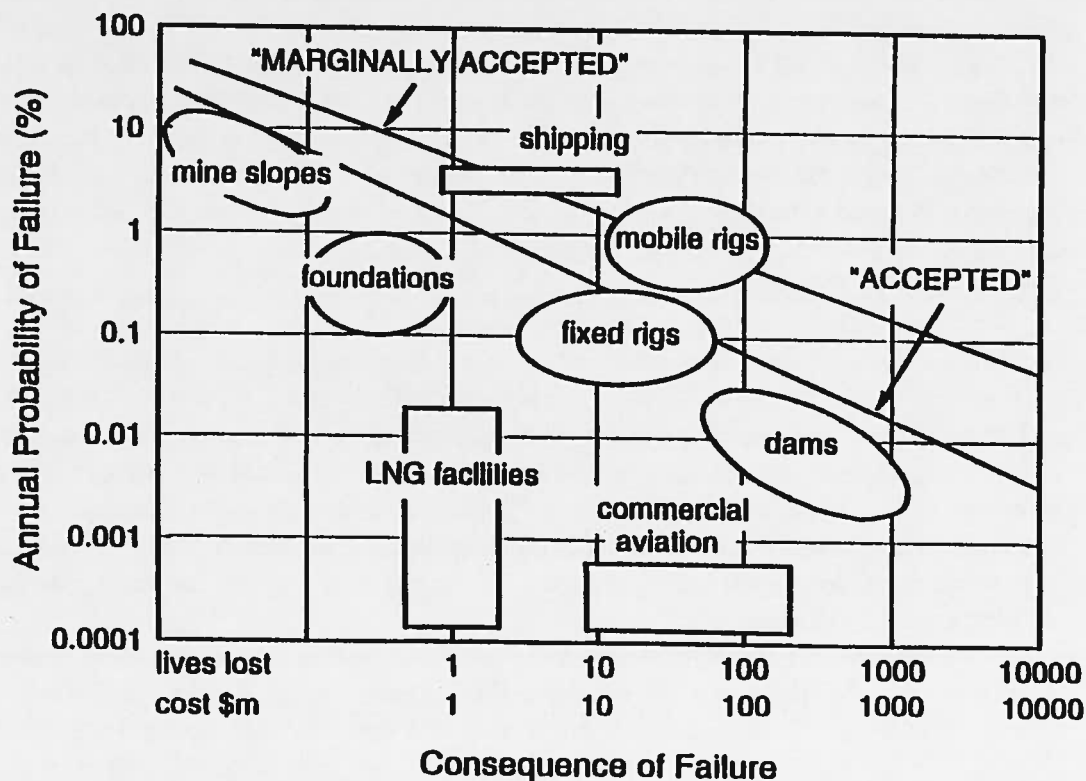


FIG. 4. Empirical Rates of Failure for Civil Engineering Facilities
(Baccher 1987, p. 49)

ue applies. An example of a specific area in which a different target reliability index should be used is for serviceability limit state design (Phoon et al. 1995).

In the absence of specific guidelines, it might be possible for engineers to adjust the target reliability index to reflect some design conditions that already have been accounted for in the calibration of the load and resistance factors. For example, an engineer might be tempted to use a different target reliability index for drained and undrained analysis, because the uncertainty in the in-situ horizontal stress coefficient might be judged to be higher than that in the undrained shear strength. Such intuitive adjustment of the safety level based on judgment alone is normal in the traditional factor of safety design approach and for some rather simplified geotechnical RBD approaches that have been suggested (e.g., Task Committee on Structural Loadings 1991). However, in a rigorous RBD approach, the difference between drained and undrained analyses already has been accounted for rationally in the resistance factors, and further adjustment of the target level would amount to "double-counting". Errors of this type are to be expected in the absence of proper guidance because the typical RBD code user is not familiar with the details underlying the reliability calibration process. A proper appreciation of the target reliability index selection process particularly is important, because the target reliability index often has been mistaken (incorrectly) as the RBD equivalent of the empirical factor of safety.

Definition of Nominal Component

Aside from careful selection of the target reliability index, it also is important to define and understand precisely the nominal components shown in Eq. 5. The level of safety in a design clearly is governed by the product of the load and resistance factors and their respective nominal components. Two foundations can have widely different safety levels, even though the same set of resistance factors is applied, because one design might be based on average soil parameters while another could have accrued additional safety by using highly conservative soil parameters. This important aspect is not sufficiently well-emphasized in the RBD literature (CIRIA 1977, Been & Jefferies 1993).

The definitions of nominal soil strengths in the simplified RBD formats ideally should be consistent with those used in traditional foundation design practice. However, the existing procedures for selecting nominal soil strengths are not well-defined, and they certainly are not followed uniformly by all engineers. Some engineers use the mean value, while others use the most conservative of the measured soil strength (e.g., Whitman 1984). Many other guidelines and rules-of-thumb exist in the literature. For example, Terzaghi and Peck (1948) recommended that the measured strength within a significant depth from each boring should be averaged, and then the smallest average should be used for design.

An alternative definition for the nominal value is based on the concept of an exclusion limit. The definition of a 5% exclusion limit is given in Fig. 5. However, the use of a small exclusion limit probably is not appropriate for foundation design because of several reasons. First, the amount of data required for the reliable determination of a 5 to 10% exclusion limit typically is much larger than the number of measurements taken

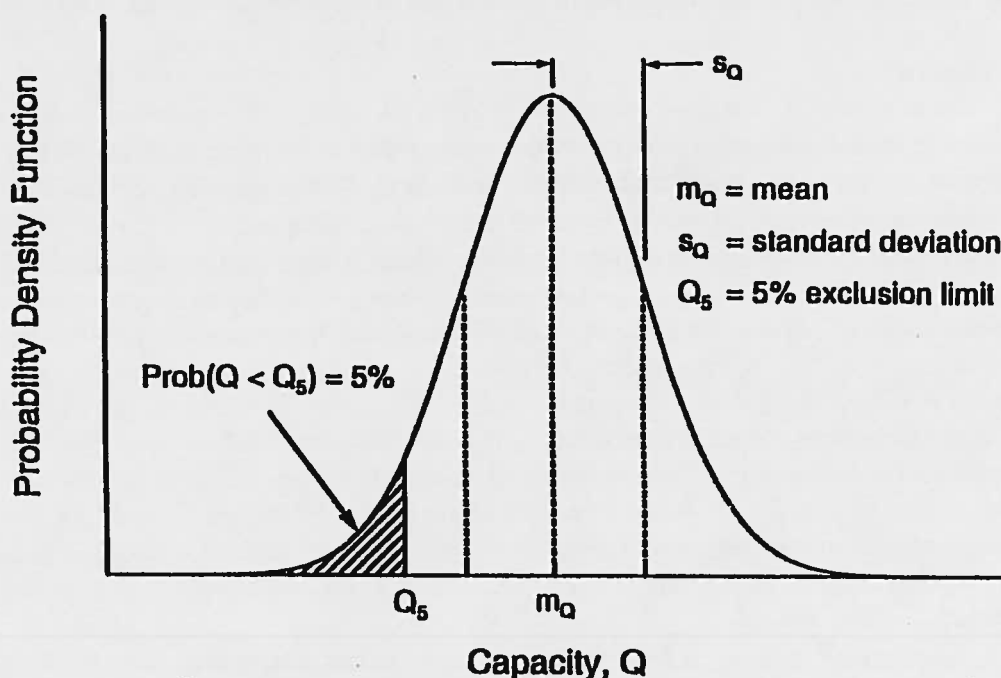


FIG. 5. Definition of 5 Percent Exclusion Limit

for a routine project (Been & Jefferies 1993). Second, the exclusion limit requires probability computations that are not currently performed in most foundation design. The main purpose of using a simplified RBD approach is to relieve practicing engineers from unfamiliar probability calculations so that they can focus on the real geotechnical aspects of the problem. Use of the exclusion limit introduces unnecessary complications and partially undermines the objective of a simplified RBD approach. Third, the exclusion limit concept is less intuitive than that for the mean value.

It is safe to say that most foundation engineers would feel more comfortable using the mean value, because they have a physical feel for that concept from their past experiences of working with actual measured soil strength parameters. Regardless of the choice, it is important to emphasize that the definition of nominal values can not be left to the judgment of each individual engineer, as is the case in traditional practice, if a uniform reliability level is to be maintained. In our opinion, all nominal soil parameters should be defined at the mean because of simplicity and compatibility with most existing foundation design practice.

The other important nominal component is the load, which geotechnical engineers normally do not investigate in detail. Loading agendas can be rather complicated, so it is necessary at least to appreciate what these values mean. In many codes these days, loads are specified using the concept of a return period. As an example, the ASCE loading guide for electrical transmission line structures (Task Committee on Structural Loading 1991) establishes the design loads for wind and other weather-related events at a return period of 50 years. The annual probability of exceeding the 50-year return period load is 1 in 50 or 2%. Other criteria are used by other organizations and for

different types of structures and loads. The resistance factors used in RBD generally are related to the loading model and the definition of the nominal load as well.

Calibrations

Figs. 5b and 5c and the corresponding Tables 3 and 4 look simple and just as easy to use as traditional design practice, which is the intent of any new or alternative design approach. However, there the similarities end. With RBD, rigorous calibrations of the design equations and all the input terms are performed to achieve a target reliability index. Specified within this approach are the nominal loads and resistances, target reliability index, design equation, and resistance factors, all calibrated together rigorously over a range of parameters using the First-Order Reliability Method (FORM). For the cases shown in Tables 3 and 4, the calibration parameter ranges were as follows: wind speed = 30 to 50 m/s, shaft diameter = 1 to 3 m, shaft depth/diameter = 3 to 10, mean s_u = 25 to 200 kN/m², and COV of mean s_u = 10 to 70%. For each combination of parameters, unique resistance factors apply. However, it is impractical to list all of these factors, because to do so would require literally dozens of tables. Instead, the results were scrutinized carefully, and it was found that the resistance factors could be grouped or "averaged" quite effectively over three ranges of s_u and three ranges of COV of s_u , as given in Tables 3 and 4.

The reliability indices for foundations obtained in this manner can not achieve the target reliability index exactly because the same resistance factor is applied to a range of undrained shear strengths. However, with the three groupings selected, it was possible to reduce the average deviation from the target reliability index to a minimum, as shown in Table 5 and Fig. 6. A comparison of the average deviations

TABLE 5. Average Deviation From Target Ultimate Resistance Reliability Index For Drilled Shafts In Undrained Uplift (Phoon et al. 1995, p. 6-8)

Clay	COV of s_u (%)	Average Reliability Deviation	
		LRFD ^a	MRFD ^b
Medium (mean s_u = 25 - 50 kN/m ²)	10 - 30	0.031	0.023
	30 - 50	0.042	0.034
	50 - 70	0.053	0.042
Stiff (mean s_u = 50 - 100 kN/m ²)	10 - 30	0.047	0.037
	30 - 50	0.068	0.054
	50 - 70	0.087	0.063
Very Stiff (mean s_u = 100 - 200 kN/m ²)	10 - 30	0.072	0.051
	30 - 50	0.102	0.074
	50 - 70	0.125	0.082

Note: Target reliability index = 3.2

a - designed using $F_{50} = \Psi_u Q_{un}$

b - designed using $F_{50} = \Psi_{su} Q_{sun} + \Psi_{lu} Q_{lun} + \Psi_w W$

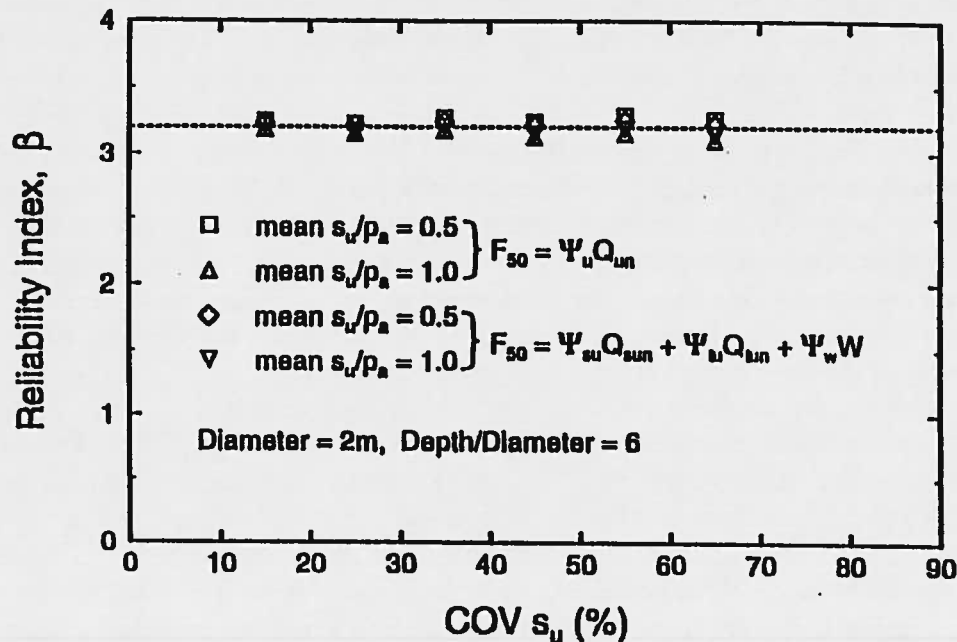


FIG. 6. Performance of Ultimate Limit State RBD Format for Drilled Shafts in Undrained Uplift

produced by the LRFD and MRFD formats also indicates that the MRFD format provides better reliability control. This observation is applicable to the other loading cases as well.

With these calibrations apparently so all-encompassing, one can ask the question whether engineering judgment is being usurped. The answer is an unequivocal "no". Instead, RBD focuses our efforts and judgment on the important issues. First, it forces us to agree on the load model, target reliability index, and design equation to use (at least for the time being). Second, it focuses our energies on evaluating the mean material properties and the variability (COV) in these properties for a given design situation. Specific guidance on these issues is beyond the scope of this paper, but detailed recent discussions by the authors are given elsewhere (Kulhawy 1992, Phoon et al. 1995, Phoon & Kulhawy 1996, Kulhawy & Trautmann 1996). Evaluation of the mean and COV for a particular boundary condition (shear, plane strain, extension, etc.) requires a careful assessment of all site, geologic, exploration, and testing variables. And finally, given that the design engineer knows explicitly what is included in RBD, the design engineer then can enhance or modify the calculation results to include the intangible and/or unforeseen issues noted previously.

SUMMARY

Judgment has, and probably always will, play a critical role in geotechnical engineering design, especially during the evolution from traditional deterministic design to new concepts of reliability-based design (RBD). In traditional geotechnical foun-

dation design, the risk of adverse performance has been controlled by an empirical factor of safety at the design stage. However, this traditional design approach does not ensure a consistent level of safety, because the empirical factor of safety is not well-defined, and its relationship to its underlying uncertainties is ambiguous. To address these problems in a more realistic fashion, an essential first step is to adopt limit state design, which is intimately related to RBD. On one hand, the philosophy of limit states represents a logical and systematic approach to the process of engineering design. Conversely, the formalization of one aspect of this whole process, which is the application of reliability theory to ensure that the occurrence of limit states is sufficiently improbable, constitutes the main thrust of RBD. From this perspective, limit state design represents a more fundamental approach. Undue emphasis on RBD at the expense of the other design aspects clearly must be avoided.

In this paper, a general overview of reliability theory and a simplified RBD design approach have been presented. The load and resistance factor design (LRFD) and multiple resistance factor design (MRFD) formats were shown to be suitable for reliability calibration because they provide the design engineer with a simple direct means of checking the new design formats against their traditional design experience. Generally, the MRFD format is to be preferred. The proper use of these simplified RBD formats was discussed with reference to the ultimate limit state design of drilled shafts under undrained uplift loading. The two important aspects of this new design approach that can not be left entirely to the routine judgment of the design engineer are: (a) the selection of the target reliability index and (b) the definition of the nominal quantities in the design equations.

The applications of these new concepts were explored in some detail, and it was stressed that judgment still has a very important role in the design process. However, the judgment issues shift largely from assessing empirical factors to defining material characteristics and uncertainties explicitly and to judging intangibles and unknowns implicitly. This process puts design within a more rigorous and consistent framework.

ACKNOWLEDGMENTS

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* COLLAPSE OF GEOGRID-REINFORCED RETAINING STRUCTURE

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ABSTRACT: The use of cohesive soils in geogrid-reinforced backfills requires consideration of the performance of these materials under both as-compacted and long-term conditions. Depending on the as-compacted conditions, the long-term performance, as a result of in service saturation, can lead to strength loss and failure of such structures. This paper documents the case history of a geogrid reinforced retaining wall, constructed with cohesive backfill, that failed. Several different failure modes were observed along the wall. The results of extensive field and laboratory testing programs and engineering analyses to investigate the causes of failure are presented. These studies permit the different observed failure modes to be explained. Deficiencies in design and construction quality control are identified. The need for site-specific design considerations rather than generic design procedures for such structures is demonstrated.

INTRODUCTION

The development of the Barren River Plaza Shopping Center in Glasgow, Kentucky, included the construction of a retaining wall along the northeast property line adjacent to the city of Glasgow's Legion Park. Construction of the wall represented a revision from the originally proposed site-grading plan to accommodate expanded lease agreements while avoiding encroachment into the city park area. Following evaluation of a number of alternative wall systems, a composite structure consisting of a compacted clay slope above a stone-faced, geogrid-reinforced retaining wall was constructed. Shortly after completion of the wall, distress was observed on the backfill slope. The condition of the structure deteriorated during the following months, culminating in failures along much of the backfill slope, large deformations of sections of the wall, and collapse of a 21 m (70 ft) section near the eastern end of the wall. The owners retained one consultant to investigate the cause of failure and another consultant to redesign the wall. Objections to the former consultant's report on the cause of the failure were raised by the parties involved. Accordingly, the owners engaged the writers, to make an independent investigation into the causes of the failure. The findings are summarized in this paper.

LOCATION OF PROJECT

Barren River Plaza Shopping Center is located on what was formerly the county fairgrounds on the northwest side of Glasgow, Kentucky. The pre-construction topography at the site consisted of a relatively gentle slope (20H:1V) across the site that dropped from a high of about elevation 253 m (830 ft) at the northeast property boundary to a low of about 238 m (780

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ft) at the south end. Postconstruction topography immediately behind the buildings at the northeast boundary ranged from about 239 m (785 ft) at the west end to about 236 m (775 ft) at the east end. Thus, the established grade for the buildings resulted in the postconstruction ground elevation behind the buildings being 7.5–9.0 m (25–30 ft) below the original ground elevation. The need to accommodate this grade separation in the limited distance between the buildings and the Legion Park property line required construction of a retaining wall.

REGIONAL GEOLOGY

Regional geologic information indicates that the site is underlain by St. Louis Limestone of the Upper Mississippian Series. Depth to bedrock is typically less than 12 m (40 ft) and was encountered in many of the borings advanced at the site at depths of about 7.5–10.5 m (25–35 ft). The overlying soil is typically a mottled yellowish brown silty clay to clayey silt limestone residual soil with some sand. The soil contains up to 40% by volume chert fragments ranging in size from 1.3 to 5.0 cm (0.5–2 in.). The silty clay soils are of medium to high plasticity, with liquid limits in the range of 50–65, plastic limits of 25–35, and natural moisture contents above the water table at or slightly below the plastic limits. Additional site-specific information on subsurface conditions is presented later.

DESCRIPTION OF RETAINING WALL CONSTRUCTION

To provide the necessary grade separation between the rear of the stores on the northeast side of the shopping center and the property boundary with the city park, a geogrid-reinforced retaining wall with a sloping backfill was proposed. A typical cross section, reproduced from the drawings issued for construction, is shown in Fig. 1. A front elevation of the wall showing

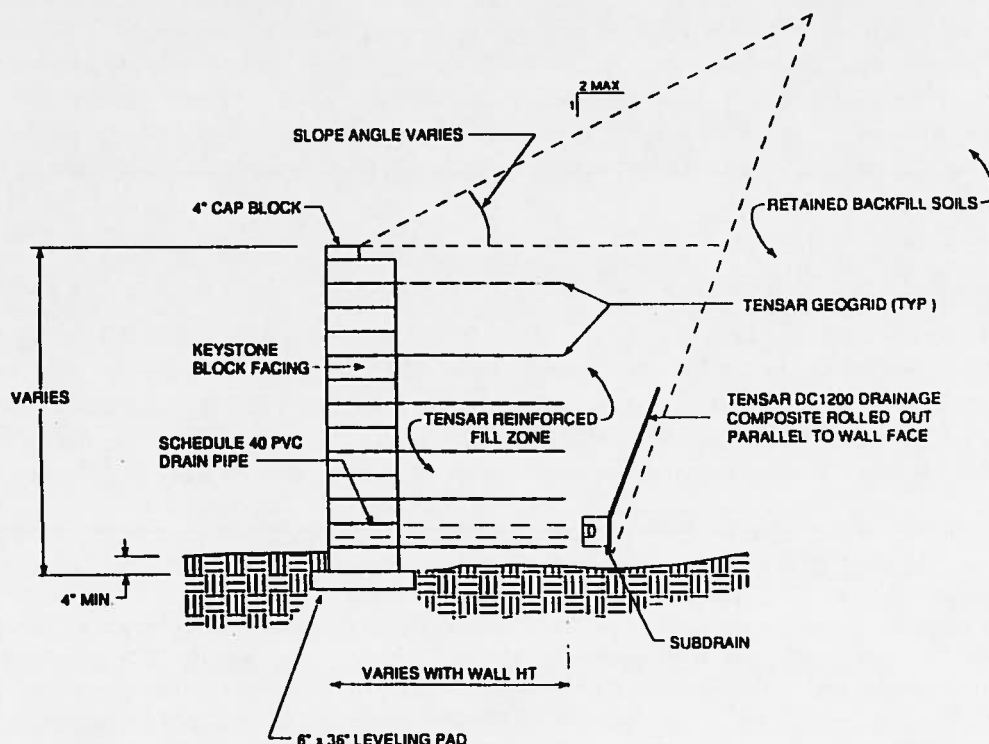


FIG. 1. Typical Section through As-Designed Retaining Structure

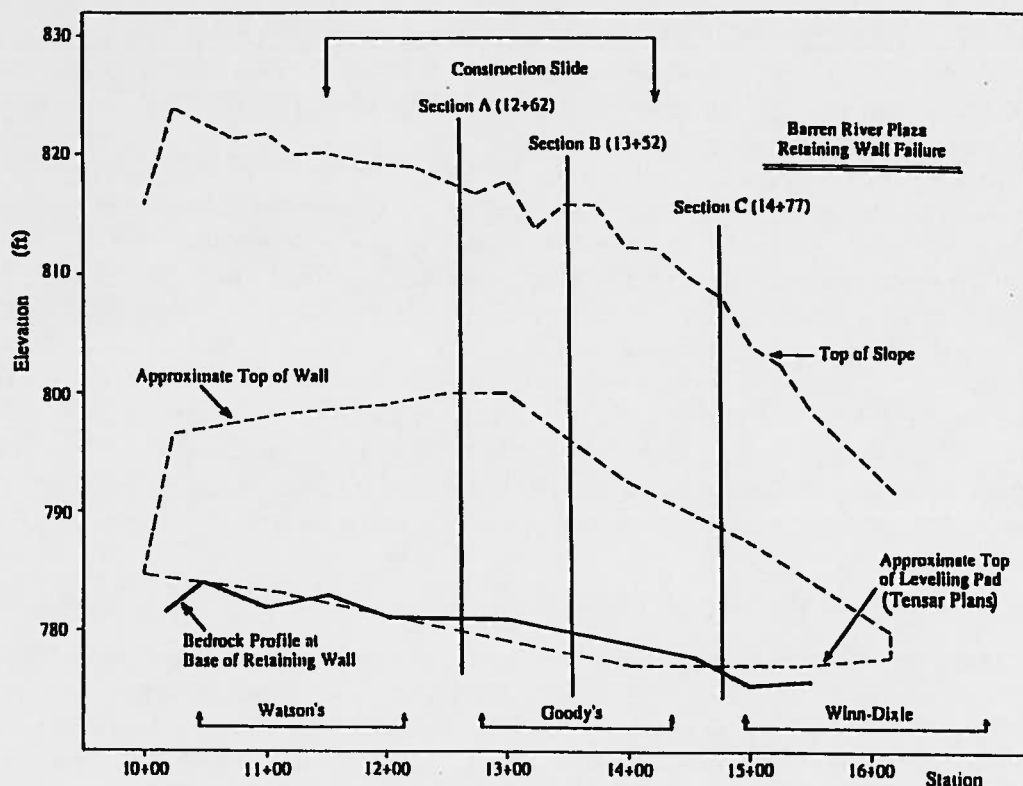


FIG. 2. Front Elevation Profiles of Retaining Structure

leveling pad, top of wall, and top of slope profiles is given in Fig. 2. Bedrock elevations at the base of the wall were obtained directly from borings. For the most part, the wall height varied between 3.0 and 6.0 m (10 and 20 ft), with the maximum height in the vicinity of Station 13 + 00 being 6.4 m (21 ft). The height of the slope above the top of the wall ranged from 8.2 m (27 ft) at the western end to 6.0 m (20 ft) in the center and tapered down to about 3.0 m (10 ft) behind the store at the eastern end. Given the constraints imposed by the location of the wall and the property boundary, this resulted in constructed slopes typically varying between about 1.7H:1.0V and 2.0H:1.0V, as determined from construction surveys. Relevant construction details are summarized next.

Excavation to permit placement of geogrids for the proposed retaining wall was started in late May 1990. The contractor elected to excavate the entire cut before commencing construction of the wall. It can be seen from Fig. 2 that as the excavation approached station 14 + 00, a steep backslope of increasing height was excavated because of the increasing proximity of the wall alignment to the property line and the increasing elevation of the original ground surface. Based on construction photographs, this backslope is estimated to have been up to 10.7 m (35 ft) high at an inclination greater than 1.0H:1.0V. It is understood that concern was expressed about the stability of this temporary slope but no action was taken.

A major failure of the cut slope along the northeast property line occurred in June 1990. The slope failed at a time when blasting operations to remove bedrock to permit placing footings on grade were in progress (near the northwest end of wall) but it is not clear that the slide occurred coincident with a blast. The slide, which encroached into Legion Park, occurred approximately between stations 11 + 50 and 14 + 25 (Fig. 2).

Construction of the retaining wall was initiated in June shortly after the

construction failure. Efforts were made to incorporate the debris remaining from the construction slide into the backfill as the construction proceeded. Construction photos do not confirm that the slide debris was entirely removed and recompacted in the backfill.

In the process of excavating for the leveling pad, bedrock was encountered above the design elevation of the footing pad, primarily between stations 13 + 00 and 14 + 00, as evident from the profiles shown in Fig. 2 and construction photos. Because blasting of rock was no longer permitted, the problem was resolved by relocating the wall to the southwest, where the slope of the bedrock permitted placing the leveling pad at the original design elevation. There is no documentation of how the interference between the sloping bedrock and the required horizontal placement of layers of geogrid was actually resolved in the field, although a letter specifically cautioning against placing the geogrid on the bedrock slope is on file. It is evident that either the geogrid was placed on a slope or the grid layers were foreshortened.

The geogrid-reinforced section of the wall was completed by mid August 1990.

The sloped portion of the backfill was not completed until mid October. The delay in completing this upper portion of the structure resulted from difficulties that arose in constructing the as-designed composite slopes and delays in obtaining approval from the city to proceed with the proposed revised slopes.

The subdrain shown in Fig. 1 was installed at the back of the reinforced zone. The elevations at which this drain was installed will be discussed later. Lateral drains intended to exit at the face of the wall (Fig. 1) were apparently not installed; no one who was on site recalled observing their installation, and they do not appear on any of the construction photographs.

DESCRIPTION OF WALL FAILURE

The first indication of distress in November 1990 was relatively shallow slumping of the sloped backfill with approximately the same main scarp as that of the construction failure. Significant slumping followed shortly after the initiation of seasonally heavy rainfall. Increased slumping of the sloped backfill at about station 14 + 70 preceded the collapse of the facing wall at that location on December 23, 1990. Elsewhere the scarp deepened and was accompanied by varying amounts of lateral wall movement. Surveys of wall movements initiated in January 1991 showed a relative outward movement at the top of the wall in a 4-day period of about 5.0 cm (2 in.) between stations 10 + 50 and 11 + 50 (determined by using a plumb line from the top). Other survey measurements showed outward displacements of facing blocks at ground level of almost 27 cm (10.5 in.) at station 12 + 50 during a 4-month time period.

Fig. 3 shows the pattern for the rate of movement at station 12 + 50 at the base of the wall during the period January to April 1991. It can be seen that there were essentially two periods of accelerated movements, along with extended periods of relative inactivity. Fig. 4 shows the rainfall during 5-day periods between October 1990 and April 1991. The timing of major distress in the backfill slope and the periods of recorded accelerated movements of the facing wall have been noted. The close relation between rainfall and observed performance is evident. Major distress in the backfill slope occurred during the heaviest period of rainfall [>7.6 cm (3 in.) per 5-day period for 2 weeks] and subsequent periods of accelerated wall movements

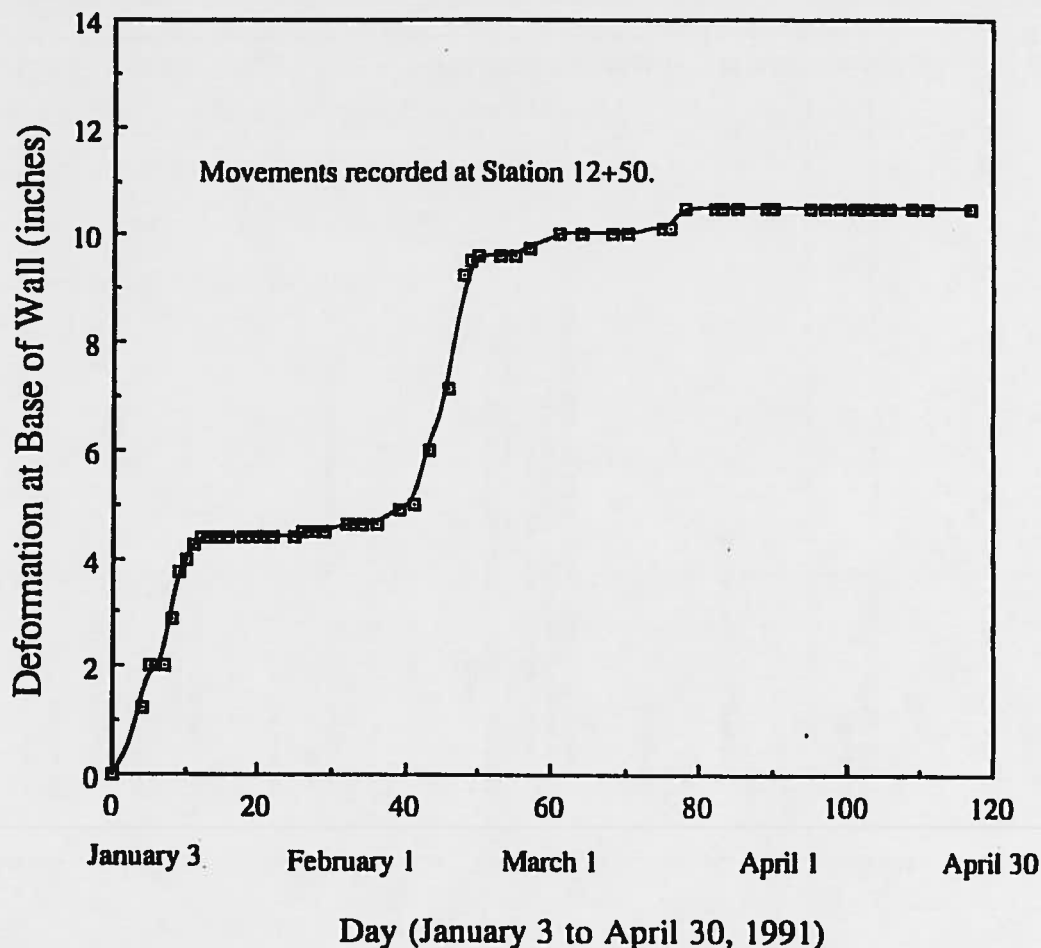


FIG. 3. Deformation Rate during Initial Period of Distress

occurred during the other two periods when >7.6 cm (3 in.) of rainfall per 5-day period was recorded between January and April 1991. Because some 15 days of heavy rainfall occurred prior to initiation of measurements at the base of the wall, the total wall movement is not known.

SUMMARY OF INVESTIGATION

Review of Existing Information

An extensive range of information was available for review in connection with the wall failure. This included geological literature, design records, construction and performance records, and the information generated by various other parties in the form of preliminary failure investigations and for redesign of the wall. An annotated chronological report of the design and construction history of the wall, including photographic documentation, was provided by the owner. The information obtained from these various sources was used in this investigation for a variety of purposes, including: (1) Developing a regional and preliminary site-specific understanding of subsurface conditions; (2) identifying key factors relating to the failure; and (3) planning additional field and laboratory investigations to assist in determining the causes of failure.

Field Investigation Program

A drilling, sampling, testing, and instrumentation program was conducted in September 1991. Five borings (A series) were advanced at locations A1

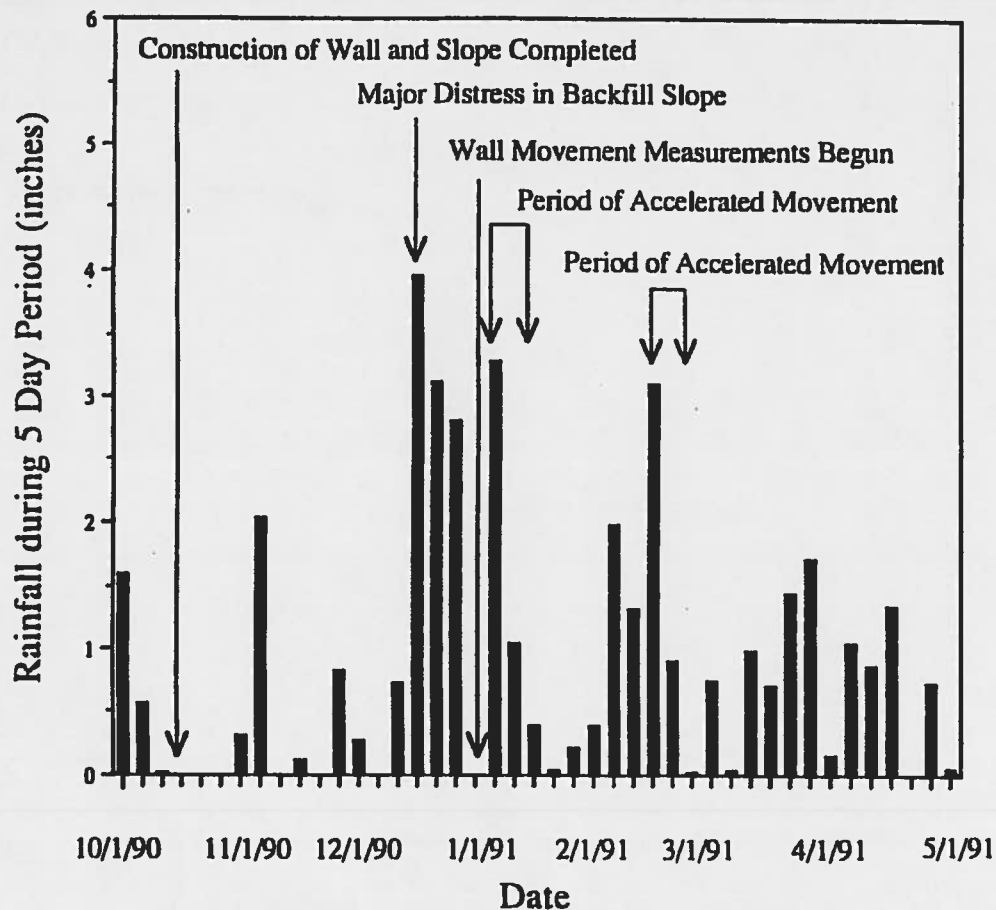


FIG. 4. Rainfall during Initial Period of Distress

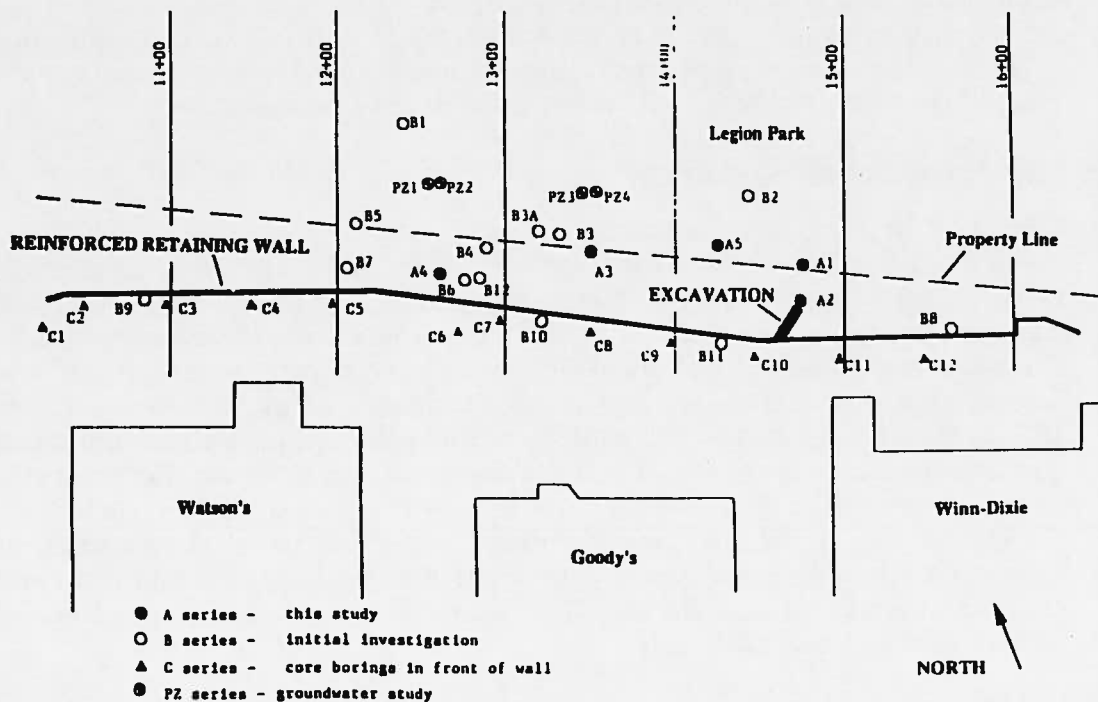


FIG. 5. Boring Location Plan

to A5, as shown in Fig. 5. Also shown in this figure are the locations of borings made by others: B series (borings advanced in January 1991 as part of a preliminary failure investigation); C series (borings advanced in July 1991 to determine bedrock elevation along the proposed alignment of a replacement wall); and PZ series (borings advanced in April 1991 as part of a piezometer-installation program). In general, continuous sampling was performed in all borings performed as part of the present study using a split-spoon sampler. The only exception was boring A3, where a 1.5 m (5 ft) long drive sampler was used. On completion of the drilling and sampling operations, a combined inclinometer-piezometer was installed in each of borings A1 to A4 and a piezometer was installed in A5. Measurements of wall and slope deformations and ground water conditions were made during the period September to December, 1991.

A test excavation was made in November 1991. The excavation was located at station 14 + 70 (Fig. 5), where a section of the retaining wall had collapsed. The excavation extended about 6.0 m (20 ft) back from the face of the wall and exposed the subdrain at the back of the geogrid-reinforced zone. The test excavation was about 1.2 m (4 ft) wide at the base and the height of the cut was about 6.0 m (20 ft) at the back. The main purposes of this excavation were to: (1) Determine the number and locations of geogrids at the failed section; (2) determine the location and condition of the subdrain; and (3) examine the condition of backfill soils within the reinforced zone.

Laboratory Testing Program

Laboratory tests performed included in-situ moisture content and density determinations, Atterberg limits, compaction moisture/density relationships, and as-compacted and soaked compressibilities and strengths of the backfill materials.

Analyses and Evaluations

A range of analyses and evaluations was performed using existing information and the data generated in the writers' investigation, including: (1) Detailed interpretation of subsurface conditions; (2) comparison of as-compacted and postfailure moisture/density conditions; (3) comparison of as-compacted and soaked compressibilities and strengths of backfill; (4) review of compaction control specifications and degree of compliance achieved; (5) review of geogrid placement; (6) review of wall movements; (7) review of inclinometer movements; and (8) stability analyses of wall and backfill. Based on these analyses, the causes of the observed distress and failure were identified.

COMMENT ON FINDINGS

Key Characteristics of Failure

The findings of the extensive analyses and evaluations that were conducted by the writers are presented next. The key characteristics of the failure to be explained were identified.

- Why did the facing collapse at station 14 + 70 even though the wall was higher and the backslope was both higher and steeper at other locations?

- What accounted for the different patterns of movements at varying locations along the wall?
- Why did distress of the wall respond so quickly to rainfall intensity?
- Why did distress of the wall continue to respond to rainfall intensity more than a year after construction was completed?

Subsurface Conditions

Based on the available boreholes (Fig. 5), cross sections were developed for three representative stations along the structure.

- Station 12 + 62—approximate center of construction slide and highest section of wall
- Station 13 + 52—approximate location of sloping bedrock
- Station 14 + 77—center of collapsed section of facing blocks and location of test excavation

The cross section shown in Fig. 6 (station 13 + 52) is typical of the sections examined. Based on these sections, attention is directed to the following factors.

The limits of the fill material were determined on the basis of penetration resistance, density, and moisture content measurement. For example, natural undisturbed soils had blow counts of the order of 15–30 above the water table and 10–15 below the water table. By contrast, compacted fill materials behind the geogrid reinforcement had N values as low as 2 or 3 and generally less than 10 both above and below the water table. Postfailure density determinations further assisted in delineating fill from natural soils. For example, dry densities above the water table in natural soils were typically in the range of 1,600–1,680 kg/m³ (100–105 pcf), while corresponding fill soils had densities ranging from 1,360–1,520 kg/m³ (85–95 pcf) outside the geogrid reinforced zone and 1,490–1,600 kg/m³ (93–100 pcf) within the reinforced zone.

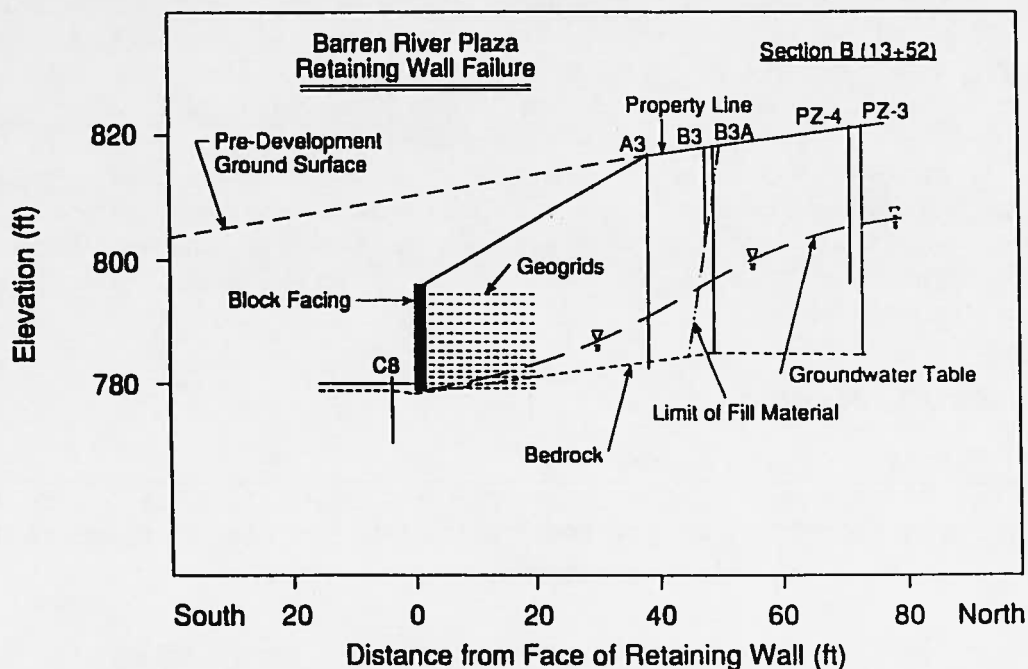


FIG. 6. Cross Section of Retaining Wall at Station 13 + 52

An extensive zone of fill material exists behind the geogrids at stations 12 + 62 and 13 + 52 (in the area of the construction slide) but not at station 14 + 77.

Borings in natural ground showed a thin seam of low strength material immediately above the bedrock.

The sloping bedrock noted earlier in the paper is clearly delineated in Fig. 6. It is apparent that as many as three or four of the lower layers of grids at this location (station 13 + 52) could not have been both continuous and horizontal.

The subdrain and filter cloth as exposed in the excavation at station 14 + 70 was in good condition. The ground-water table was controlled by the subdrain, which was found in the excavation to be above the design location. From the location of the water table at the other sections, it can be inferred that the subdrain at these other stations was also above the design location. This condition could also be inferred from the construction photographs.

Moisture-Density Conditions

An extensive evaluation of as-compacted moisture-density conditions versus postfailure moisture-density conditions was conducted. The results of this study are presented in Figs. 7 and 8. The dashed curves in Fig. 7 show the bounds of laboratory standard Proctor tests based on 11 samples obtained at various locations behind the wall. The range in optimum densities of about 1,600–1,730 kg/m³ (100–108 pcf) and optimum moisture contents of about 18–23% are representative of the variable soil conditions encountered. Also shown are the lines of optimum moisture/density and of 100% saturation. Postfailure moisture/density conditions are shown plotted as open circles and solid dots and are seen to fall within two distinct zones (which are x-hatched in Fig. 7). Tests performed on samples taken from within the geogrid reinforced zone (open circles) showed moisture contents ranging from 18% to 27% and dry densities between 1,490 and 1,650 kg m³ (93 and

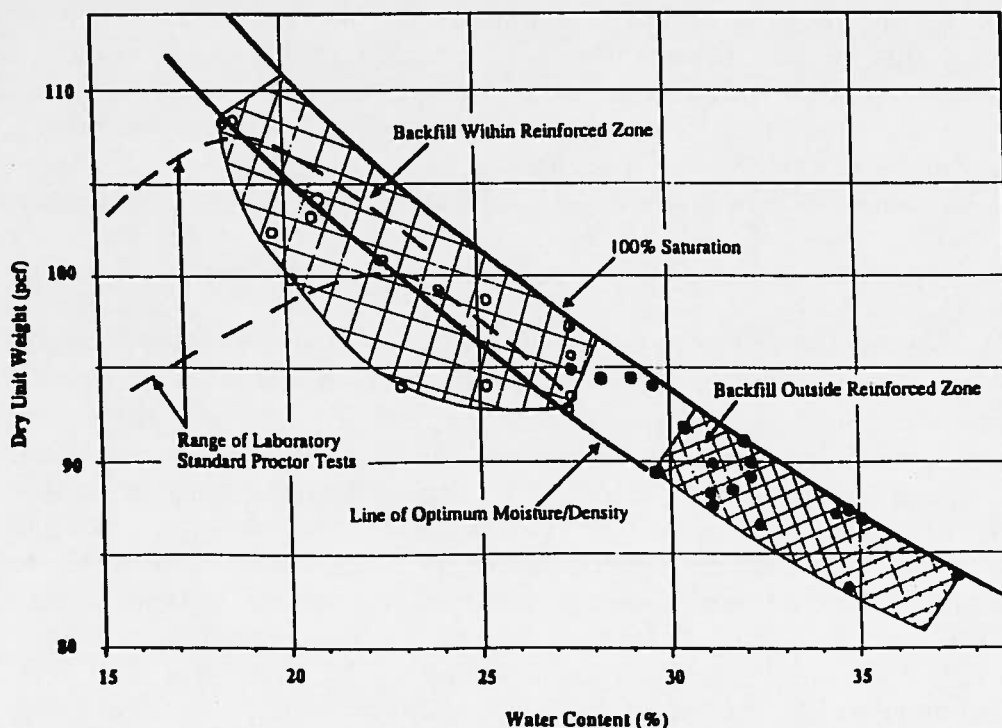


FIG. 7. Postfailure Moisture-Density Conditions

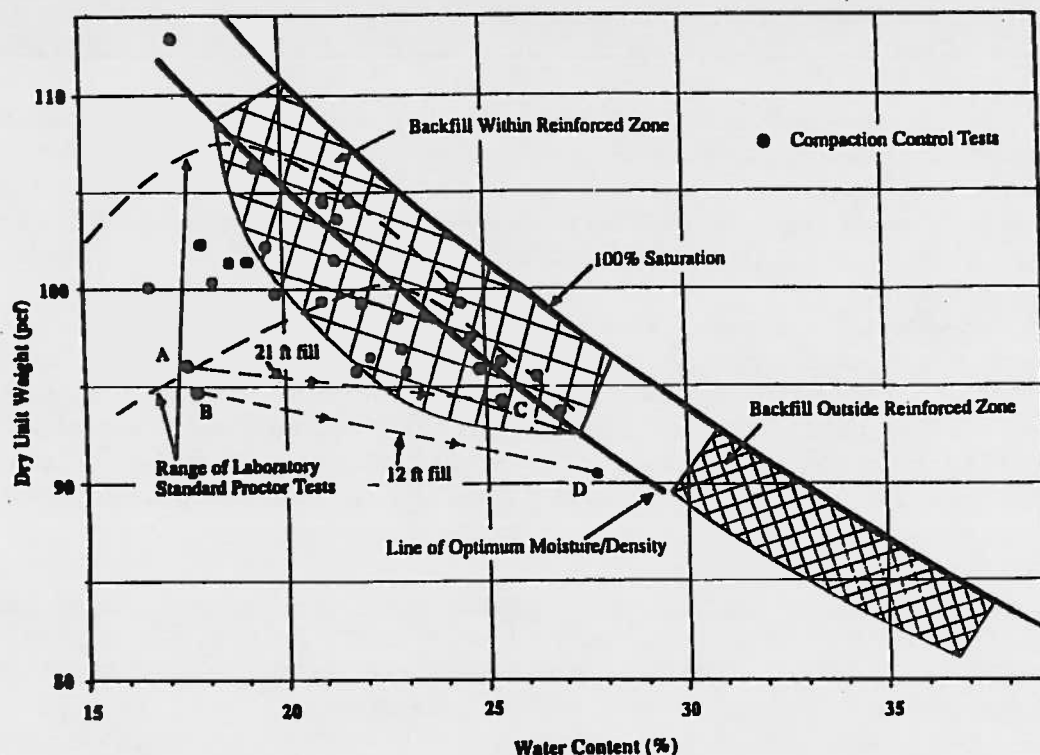


FIG. 8. Compaction Control Moisture-Density Conditions

103 pcf). By contrast, samples obtained from outside the reinforced zone showed moisture contents between 30% and 39% and dry densities between 1,350 and 1,470 kg/m^3 (84 and 92 pcf).

The lines of optimum moisture/density and 100% saturation, and the x-hatched zones in Fig. 7 are reproduced in Fig. 8 along with data points representing the compaction control tests. It can be seen that many of the data points indicate compaction being performed dry of optimum. In addition to the field compaction control data, two additional points (A and B) are shown in Fig. 8. These points correspond to soil samples having the highest plasticity compacted at standard Proctor energy and about 7–8% dry of optimum. When samples A and B were subjected to mean confining stresses corresponding to 6.4 and 3.6 m (21 and 12 ft) of fill, respectively, and were then allowed to imbibe water and swell, the moisture-density conditions changed to points C and D, respectively. The initial conditions A and B correspond to the most plastic samples at the driest recorded as-compacted conditions and hence those likely to undergo the most swell on exposure to moisture. It is clear from Fig. 8 that even under these extreme circumstances the moisture-density conditions of the soil did not reach the x-hatched zone, which represents postfailure values for backfill outside the reinforced zone. Based on this evidence, it is concluded that the only explanation for the measured postfailure moisture-density conditions in the backfill behind the reinforcement is that the soil was not initially compacted to the specified 95% of standard Proctor by a large margin. Using mean values of standard Proctor optimum density of 1,670 kg/m^3 (104 pcf) and postfailure in-situ density of 1,430 kg/m^3 (89 pcf), it is concluded that, on average, soil outside the reinforced zone had a density of about 86% of standard Proctor optimum.

Changes in Soil Strength

A range of triaxial strength tests was performed to evaluate the effects of compaction water content and confining pressure on the shear strength after swell. These tests were performed on specimens compacted to both 95% and 89% of standard Proctor maximum dry density.

The effect of compaction water content on the undrained shear strength after swell at a confining pressure equivalent to a fill height of 7.3 m (24 ft) is shown in Fig. 9. It can be seen that a larger loss in strength occurs as the sample's initial moisture content becomes increasingly drier of optimum. The ratio of as compacted to swelled strength varies from approximately 1.2:1 for specimens with an initial moisture content close to optimum to about 2.5:1 for specimens initially 4% dry of optimum. Moreover, the post-swell strength for a specimen compacted to 95% of standard Proctor maximum and a confinement corresponding to 7.3 m (24 ft) of fill was about 1.41 kg/m² (2,900 psf) if compacted at optimum water content and 0.98 kg/m² (2,000 psf) if compacted 4% dry of optimum.

The effect of confining pressure on shear strength after swell is shown in Fig. 10. It can be seen that, at 5% dry of optimum, a reduction in confinement corresponding to a decrease in backfill height from 6.7 to 2.1 m (22 to 7 ft) reduces the undrained shear strength after swell from about 0.58 to 0.22 kg/m² (1,180 to 450 psf) for specimens compacted at 95% standard Proctor maximum density. A correspondingly lower curve would be obtained for specimens at an initial compaction ratio of about 89% as indicated by the data in Fig. 11, which shows the results of several tests performed under similar moisture-content and initial-confining-stress conditions but at

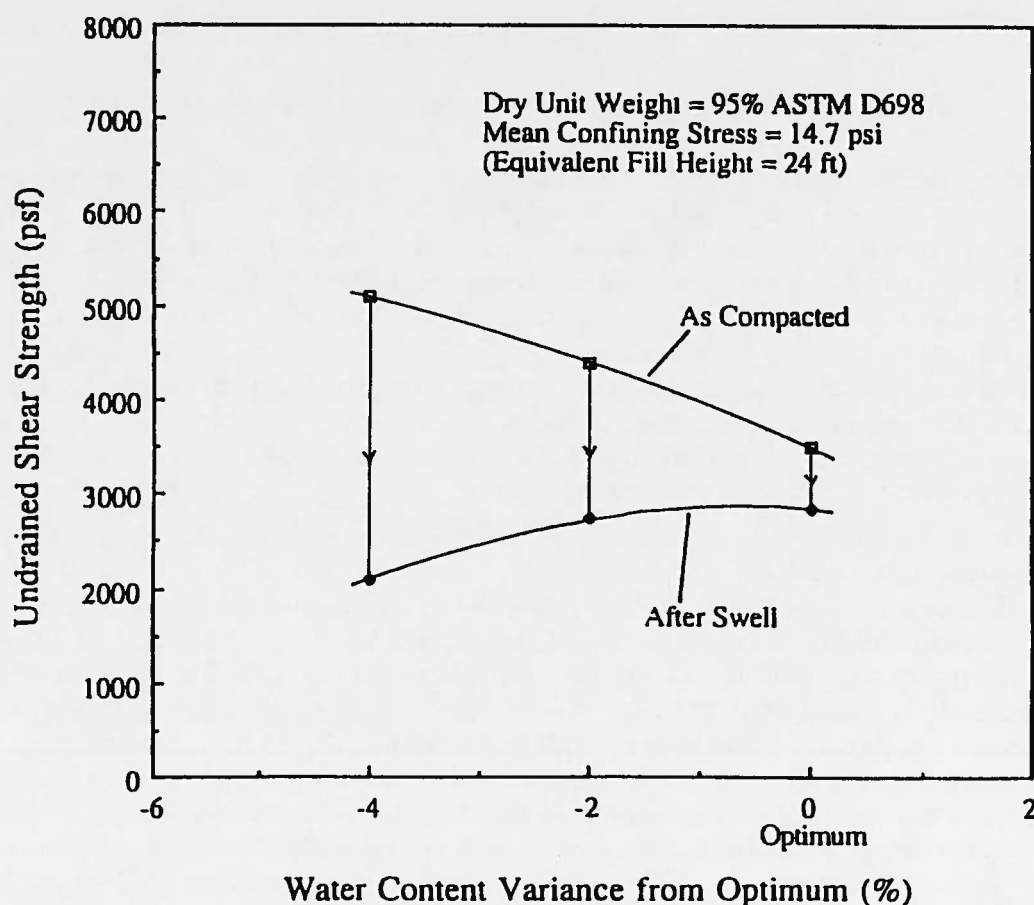


FIG. 9. Effect of Compaction Water Content on Shear Strength after Swell

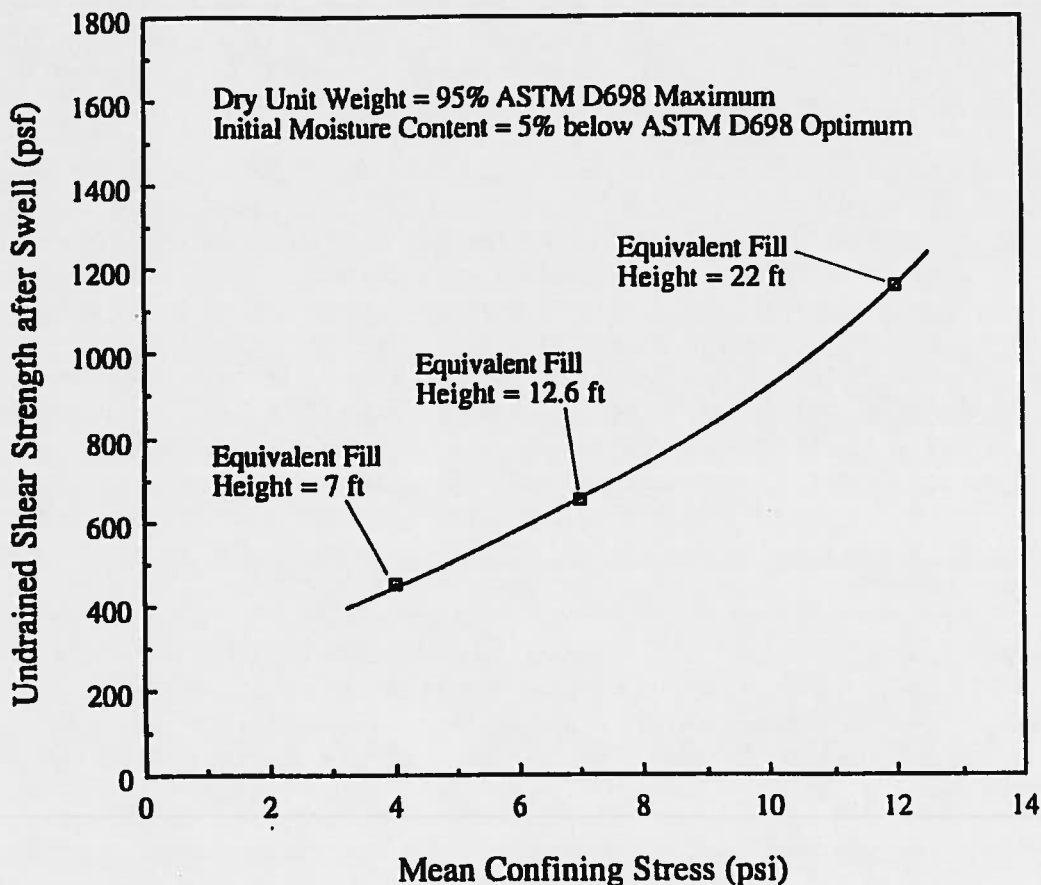


FIG. 10. Effect of Confining Stress on Shear Strength after Swell

different initial dry densities. As expected, the initially looser samples had lower postswell strengths. For example, at a fill height of 7.3 m (24 ft), a sample compacted at optimum moisture content and 95% standard Proctor density would have a postswell shear strength of 1.41 kg/m^2 (2,900 psf) as shown in Fig. 9; on the other hand, at a fill height of 2.1 m (7 ft), a sample compacted 5% dry of optimum and 89% standard Proctor density would have a postswell shear strength of only 0.15 kg/m^2 (300 psf) as indicated in Figs. 10 and 11. The decisive influence of compaction water content, confining pressure, and initial dry density on the strength of compacted clay after imbibing water is evident in Figs. 9–11.

Compaction Control

The project *Site Work Specifications Manual* (unpublished) required one compaction control test every 230 m^2 (2,500 sq ft) of each 0.2 m (8 in.) thick lift in subgrade areas and one test every 930 m^2 (10,000 sq ft) in other areas. It is noted that the *Site Works Specifications Manual* was prepared before the need for a retaining wall was identified and that it contained no requirements for controlling as-compacted water contents. Further, although the number of tests given in the manual is representative of current practice for general fill areas, it does not specifically cover construction of reinforced retaining walls. The construction drawings specified 95% of ASTM D698 (standard Proctor test). Summaries of actual compaction control tests performed during construction of the retaining wall are given in Table 1 and Fig. 12. The number of compaction control density tests performed well below that corresponding to the specifications, and below that corresponding

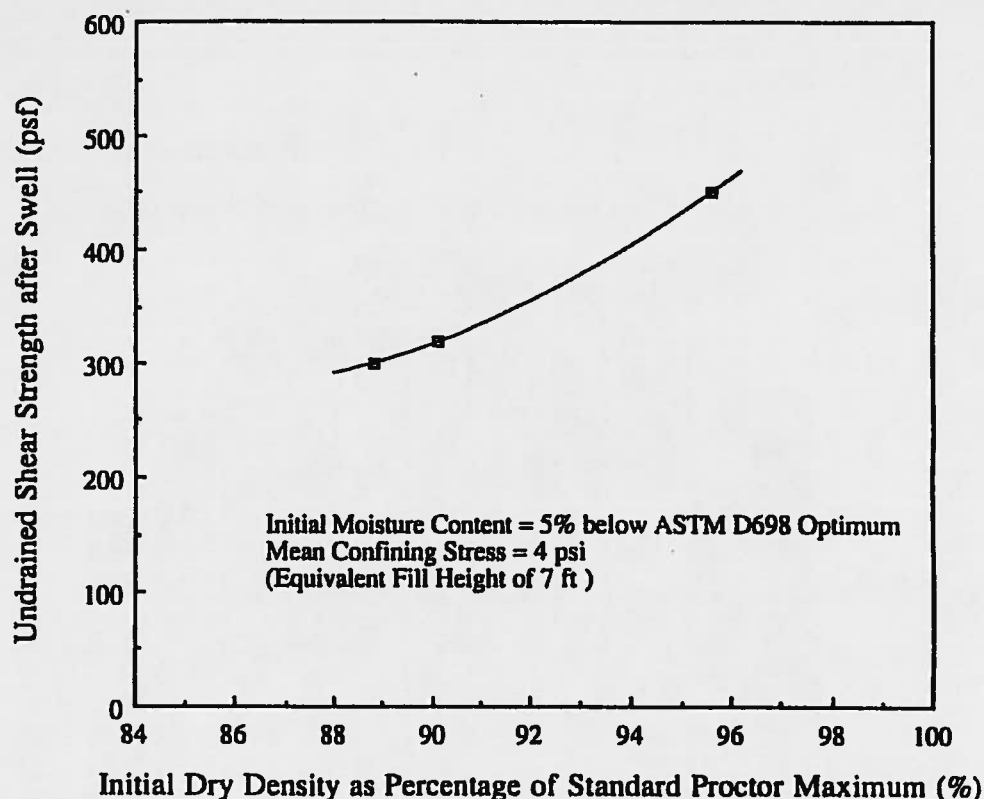


FIG. 11. Effect of Initial Dry Density on Shear Strength after Swell

TABLE 1. Comparison of Compaction Control Performed with Site Work Specifications

Condition (1)	Tests required (2)	Tests performed (3)
(a) Total Volume of Compacted Fill = 10,600 m ³ (375,000 cu ft)		
Subgrade	300	28
Other areas	75	28
(b) Volume of Fill Below Top of Facing Wall = 5,800 m ³ (205,000 cu ft)		
Subgrade	164	23
Other areas	41	23
(c) Volume of Fill Above Top of Facing Wall = 4,800 m ³ (170,000 cu ft)		
Subgrade	136	5
Other areas	34	5

to accepted practice. For example, the U.S. Department of Navy (1982) and the U.S. Bureau of Reclamation (1974) both require one compaction control test every 150 m³ (5,400 cu ft) for backfill around structures. In the volume of fill above and below the top of the retaining wall, this corresponds to 37 and 31 tests, respectively, compared to the 23 and five tests actually performed.

In addition to what is considered to be an insufficient total number of tests, the locations of the tests were not well distributed. The solid and hatched areas shown in Fig. 12 correspond to the volume of fill behind and

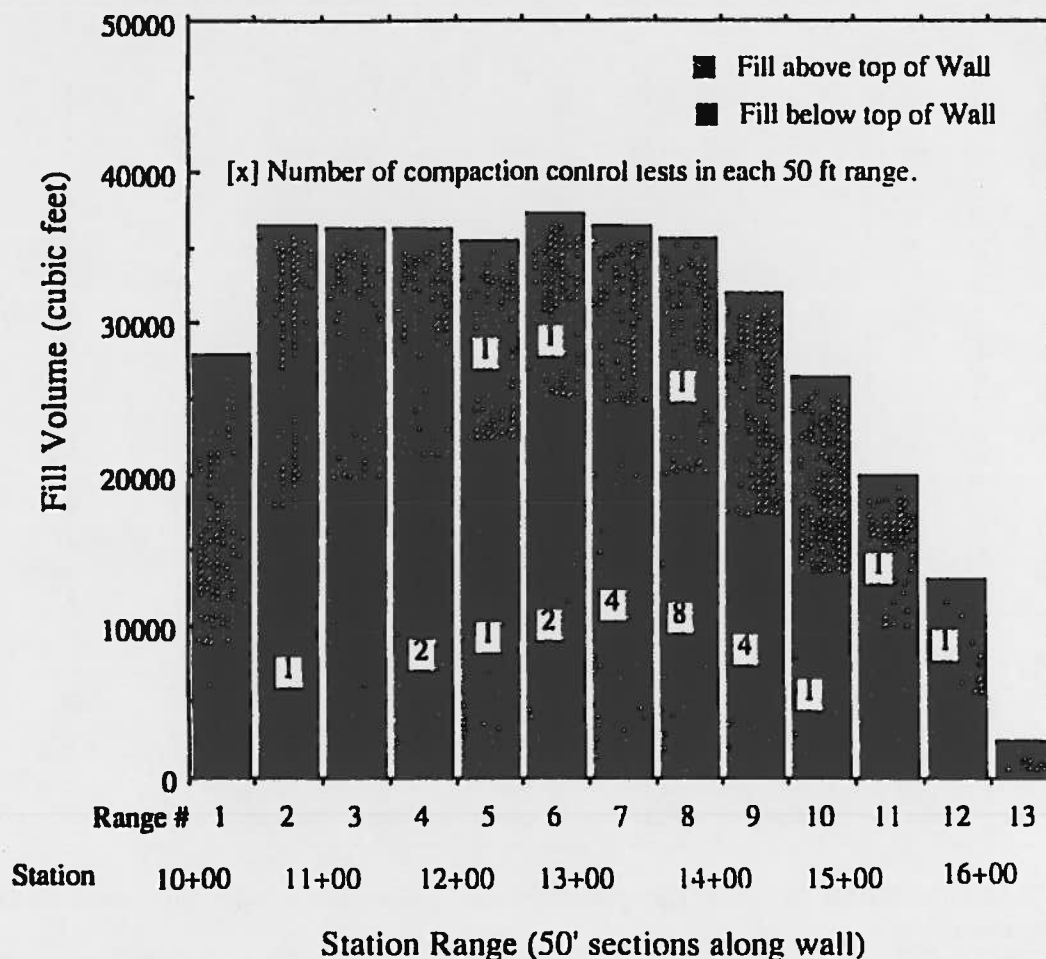


FIG. 12. Frequency and Distribution of Compaction Control Tests

above the wall, respectively, for each 15.2 m (50 ft) section of wall. The number indicated on each area is the number of compaction control tests for that segment. It is evident that many of the tests were concentrated in a small volume of soil behind the wall (lower lifts of fill between stations 13 + 00 to 14 + 50). Large volumes of fill between stations 10 + 00 and 13 + 00 behind the wall (zone of maximum wall deformation), and above the wall along its entire length, received little, if any, compaction control. Insofar as could be determined from the rough sketches available indicating the plan location of the compaction control tests, all of the tests in the fill behind the retaining wall were conducted in the zone reinforced by geogrid.

Geogrid Placement

A review of the actual field placement of geogrids compared to the design drawings issued for construction was performed. This review was based upon visual observation of the grids between the facing blocks and the test excavation performed in November 1991 at about station 14 + 70. The findings of this review of geogrid placement are summarized in Table 2, which shows: (1) The number of grids as designed versus the number of grids located; and (2) the number located but not in the design position, when referenced to the top of the wall. Observations at 15.2 m (50 ft) intervals along the wall are presented. In general, it was not possible to locate all the grids toward the bottom of the wall, because of the presence of soil at the toe of the wall and the hazard associated with its removal. Thus, the presence or

TABLE 2. Review of Geogrid Placement

Station (ft). (1)	Number grids in design (2)	Number grids located ^a (3)	Remarks (4)
10 + 50	7	7	Grids placed as designed
11 + 00	8	7	—
11 + 50	9	8	—
12 + 00	10	9	—
12 + 50	12	11	—
13 + 00	14	11	Three grids at bottom not located due to fill in front of wall; misplaced 1 grid by 16 in. (el. 782 ft)
13 + 50	13	9	Four grids at bottom not located due to fill in front of wall; misplaced 1 grid by 16 in. (el. 782 ft)
14 + 00	7	8	Extra grid behind wall
14 + 70	5	4	Trench excavation; one grid at top not present facing wall cantilevered 4 ft

^aIn general, bottom grid not located, because of fill in front of wall and grids misplaced 8 in. downward with respect to top of wall.

absence of geogrids at the very bottom of the wall could not be verified except at station 14 + 70, where an excavation was made.

Most of the geogrids were started one row of facing blocks 0.2 m (8 in.) lower than designed. More importantly, between stations 13 + 00 and 13 + 50, one layer of grid was misplaced two rows of blocks 0.4 m (16 in.) lower than designed. This occurred at about elevation 238.3 m (782 ft) (four grid layers from bottom). The latter variation in geogrid placement corresponds to zones where the largest outward movement of the bottom of the wall was observed.

The trench excavation performed at about station 14 + 70 confirmed that the top layer of geogrid reinforcement had been omitted at this location. The significance of this omission could be assessed qualitatively by examining the cross section at station 14 + 77 in that approximately the upper 1/3 of the wall had no reinforcement behind it. Further aspects of this omission are discussed later.

Wall Movements

The pattern of initial wall movements was noted earlier in relation to Fig. 3. Measurements of wall movements were restarted on September 17, 1991 and continued until December 12, 1991. In particular, relative movements for the period September to December, 1991 (corresponding to the period when inclinometer measurements were made) were obtained and evaluated. The pattern of wall movements is consistent with those observed during the initial period of distress.

At station 12 + 50 a maximum relative displacement of about 0.18 m (7 in.) was observed, with most of the movement concentrated in the bottom 0.6 m (2 ft). The largest relative displacement recorded of about 0.28 m (11 in.) was observed at station 13 + 50 with essentially all of the movement in the bottom 0.6 m (2 ft). This pattern of movements is similar to that noted earlier for station 12 + 50 but is believed to reflect the added influence of the sloping bedrock interacting with the lower geogrids.

Inclinometer Movements

The pattern and magnitude of movements observed in the inclinometers is consistent with the movements observed at the wall facing. Inclinometers A1 and A2 (Fig. 5) indicated movements concentrated in the upper 1.2–1.8 m (4–6 ft) of the fill, with larger movements close to the wall facing. This is consistent with the surficial instability of the slope and the toppling of the cantilevered portion of the wall at this location. The lower portion of the fill showed little lateral displacement, indicating a stable condition at depth. Inclinometer A3 showed movements of modest magnitude consistent with its location. Inclinometer A4 showed movements occurring over the entire wall height, with the largest movement being close to the bottom of the wall.

STABILITY ANALYSES

The global stability of the structure was analyzed using the limit equilibrium method for a number of different considerations.

Granular Backfill

Using the strength parameters ($c' = 0$; $\phi' = 30^\circ$) apparently assumed in the initial design, the global stability of a 6.4 m (21 ft) high retaining wall was examined for different inclinations of backfill slope using the procedures outlined in the Federal Highway Administration's (FHWA) "Reinforced Soil Structures: Design and Construction Guidelines" (1990). For a slope of 2.0H:1.0V, the factor of safety against sliding is 1.6. Thus, had the wall been constructed according to the initial design configuration and granular backfill material, global stability requirements would have been satisfied. However, there was insufficient room to construct a 2.0H:1.0V backfill slope without encroaching into the park, even after the wall height was increased 0.91 m (3 ft) in some places and moreover, no granular soils were available on site, although the construction drawings indicated that on-site material was to be used as backfill.

Separate studies were initiated during construction to investigate the stability of the wall and of the slope itself. The results of triaxial tests were expressed as effective stress strength parameters [$c' = 0.15 \text{ kg/cm}^2$ (300 psf); $\phi' = 25.7^\circ$]. The slope stability study concluded that a composite slope 1.5H:1.0V and 1.7H:1.0V would be stable, based on the assumption that the clay would behave in a drained condition. The wall stability study concluded that the wall with a 2.0H:1.0V backfill slope would be stable because the strength of the clay would be equivalent to that of a granular soil with $c' = 0$ and $\phi' = 30^\circ$, based on the same assumption that drained conditions would prevail. Revised construction drawings were then issued on this basis.

In the field, the contractor decided to avoid difficulties associated with construction of a composite slope. According to postconstruction surveys, the actual slopes varied between 1.7H:1.0V and 1.9H:1.0V, with the 1.7H:1.0V being typically at the higher wall sections. The factor of safety for a 6.4 m (21 ft) high wall constructed with granular backfill on a 1.7H:1.0V slope (station 13 + 52) is 1.1; for a safety factor of 1.5 the length of the geogrid should be on the order of 12.2 m (40 ft) as opposed to the 6.1 m (20 ft) length that was maintained. Thus, a 6.4 m (21 ft) high wall with a 1.7H:1.0V backfill slope would have been unsatisfactory even if granular backfill were available.

Clay Backfill

A series of stability analyses was performed by the writers for the three cross sections discussed earlier (Fig. 6) in a manner illustrated in Fig. 13, and the results are summarized in Table 3. Analyses were performed to examine both shallow backfill slopes and deep-seated wedges at each of the sections.

Strength parameters were selected on the basis of the laboratory tests that were conducted to simulate the as-compacted conditions of the backfill and the subsequent changes that occurred during the life of the structure (Figs. 9–11). The results of these laboratory tests were confirmed by strength tests on samples obtained postfailure representing in-situ conditions. The strengths adopted are noted directly in Fig. 13, due account having been taken of the relevant height of backfill and of the surface runoff conditions.

The results of the stability analyses summarized in Table 3 show that backfill slope stability is a problem at all sections, which is consistent with the inclinometer movements and the observed distress of the backfill slope. Deep-seated wedge failures are indicated at sections 12 + 62 and 13 + 52 but not at section 14 + 77. These findings are consistent with the observed wall movements. There was little outward movement at the base of the wall at station 14 + 77, which is consistent with a safety factor of 2.48 against a deep wedge failure. The absence of the upper layer of geogrids at station

TABLE 3. Summary of Slope Stability Analyses

Station (1)	Slope angle (2)	Safety Factor	
		Backfill slope (3)	Base wedge (4)
12 + 62	2.0H:1.0V	1.05	0.93
13 + 52	1.8H:1.0V	0.90	0.92
14 + 77	2.0H:1.0V	0.92	2.48

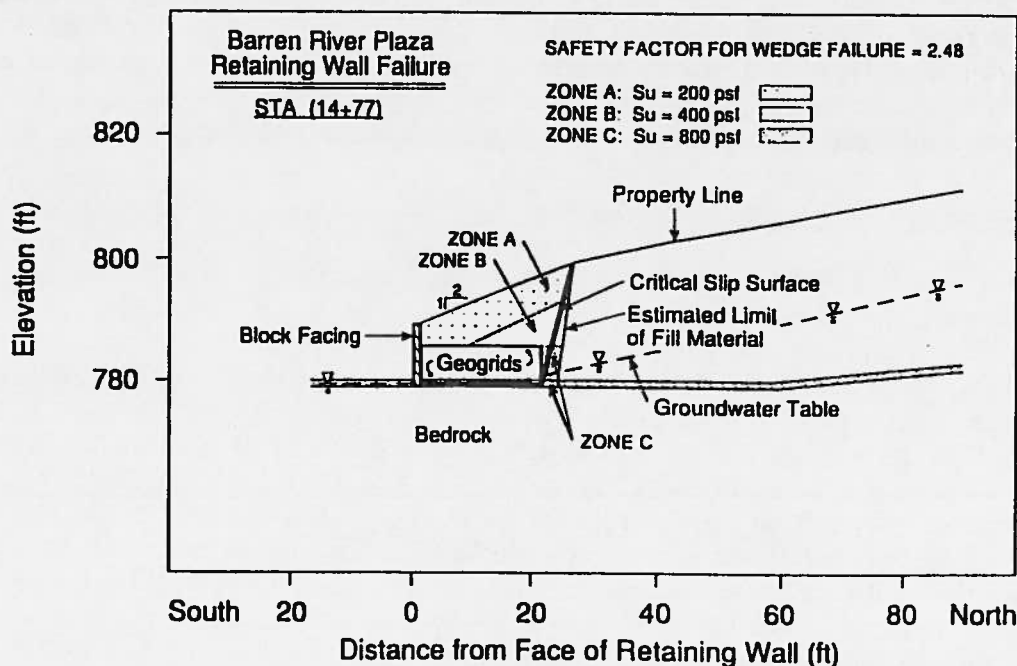


FIG. 13. Stability Analysis of Retaining Structure at Station 14 + 77

14 + 77 is directly responsible for toppling of the facing wall, despite the lower slope height. The topography in the city park aggravated the conditions at the latter section also because much of the rainfall runoff was directed toward this area.

EXPLANATION OF KEY FACTORS

Key factors pertaining to the wall failure were identified earlier. Based on the evaluations and analyses just summarized, these factors are explained as follows.

The collapse of the facing wall in the vicinity of station 14 - 70 was a result of in-service saturation and weakening of poorly compacted clay in the backfill slope combined with the omission of the upper layer of geogrid reinforcement, despite the fact that the wall and slope were not as high at this section as at others.

The pattern of wall movements varied from location to location. Specific correlation between various construction aspects and the movements are possible, including

- Movements at the top of the wall between stations 11 + 00 to 14 + 00 reflect the lack of stability of the backfill slope, which was not accounted for in the reinforcement design.
- Movements at the base of the wall from stations 12 + 00 to 14 + 00 reflect departures of geogrid placements from the construction drawings due to the presence of sloping bedrock at the base of the wall (Table 2), and uncompacted slide debris behind the geogrids.

The structure performed adequately for about 1 month before the onset of the rainy season. Due to the ease of imbibing water, a 1.2-1.5 m (4-5 ft) thick layer of poorly compacted backfill slope slumped and formed a scarp, providing access for water to greater depths. Uncompacted soil behind the geogrid reinforcement, in the area of the construction slide, imbibed water quickly and the low strength of this poorly compacted material was further reduced. Even in the area over the geogrid, compaction dry of optimum resulted in gradual loss of strength upon imbibing water.

SUMMARY OF CAUSES OF FAILURE

A number of factors contributed to the observed poor performance of the wall, including the following.

No specific site investigation was carried out to provide a basis for design of the proposed wall. Had this good practice been followed, the characteristics, properties, and variability of soils available for construction of the backfill, as well as the ground-water and surface runoff conditions, would have been identified initially. Specifying on-site granular backfill when such material was not at hand could have been avoided; setting grade for the leveling pad below rock outcrops could also have been avoided; and the problem with a sloping rock surface could have been dealt with in the design stage, instead of during construction. Drawings were issued for construction in the absence of this needed information.

A geogrid-reinforced wall was recommended without recognizing that it was unsuited to the space available for construction. The specified height of wall and backfill slope could not be constructed without encroaching on

park property. Moreover, the design required the contractor to excavate a steep backslope, 10.7 m (35 ft) high in places, which resulted in the occurrence of a major slide during construction before placement of geogrids was commenced. Warnings to the contractor of potential instability went unheeded.

The consequences of the major construction slide were not appreciated. No engineering review was made to deal with the large volume of loose slide debris, which could not be removed completely without risking further failures. The compaction control program was not designed to verify the contractor's procedures for incorporating and compacting the slide debris into the backfill. Postfailure investigations proved that, in the area of the construction slide, the backfill behind the geogrids did not meet compaction specifications.

Apart from the difficulties associated with the construction slide, the compaction control program did not meet accepted standards from the standpoint of both location and frequency of tests. Moreover, the program took no cognizance of the variability of the soils used for backfill. As a result, failure to meet specified density requirements was not detected during construction.

Collapse of the facing at a section of the wall in the vicinity of station 14 + 70 was a direct consequence of omitting the top geogrid layer at this location, and the large outward movement of the wall between stations 12 + 50 and 14 + 00 was aggravated by the presence of sloping bedrock and the consequent serious misplacement of geogrid layers in this area.

Postconstruction performance of compacted clays, especially those possessing significant swell characteristics, is critically dependent on compaction water content. There were no requirements in any of the specifications to control compaction water contents, although the soils on site are of a swelling nature. Due to the absence of moisture control, the backfill was compacted dry of optimum, which resulted in large losses in strength upon imbibing water.

APPENDIX. REFERENCES

- Federal Highway Administration (FHWA). (1990). "Reinforced soil structures: design and construction guidelines." *Rep. No. FHWA-RD-89-043*, Washington, D.C.
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THE UNIVERSITY APARTMENTS - AN OLYMPIC SETTLEMENT CASE HISTORY

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ABSTRACT

The University Apartments is a complex of 4 mid-rise dormitories constructed for the 1996 Olympic Games in Atlanta, Georgia. The buildings range in height from 5 to 13 stories, and are supported by a variety of shallow and deep foundation systems. During construction, settlements in excess of 4 inches were measured within two of the buildings supported by shallow foundations. As this was far in excess of predictions, a forensic study was commissioned to evaluate the cause and likely effects of the movements on the buildings. Surveys conducted during the initial phase of this study revealed that the larger of the two buildings had, in fact, settled more than 9 inches in the center and 4 inches on either end. The forensic study revealed that the measured structural settlements were consistent with the data obtained during the original geotechnical study. Additional monitoring of the remaining portions of the complex indicated that one wing of a third building, supported by friction piles, was experiencing large settlements as well. This movement was also found to be consistent with geotechnical theory.

INTRODUCTION

In preparation for the 1996 Olympic Summer Games in Atlanta, a major construction program was undertaken in and around the campus of the Georgia Institute of Technology, which served as the Olympic Village. This building program primarily involved the construction of numerous dormitories intended to house over 20,000 athletes, trainers, and support personnel during the course of the games, and up to 10,000 students thereafter. The largest complex of dormitories was designated the "University Apartments" and was constructed at the southeastern corner of the campus during the period of 1994 through early 1996.

The University Apartments complex consists of a total of four mid-rise buildings (A through D) configured as shown on Figure 1. Each of the buildings incorporates a post-tensioned concrete frame and an exterior veneer of brick. Building characteristics are as follows:

Building	Structure	Foundation
A	8 stories	Mat
B	13 stories - 340 feet long by 60 feet wide	Mat
C	5 stories - 170 feet long by 60 feet wide - underlain by 2 levels of parking	Spread footings
D	10 stories with 5 story wings - most over 1 to 2 levels of parking	End bearing auger-cast piles in 11 story section/friction piles beneath 5 story wings

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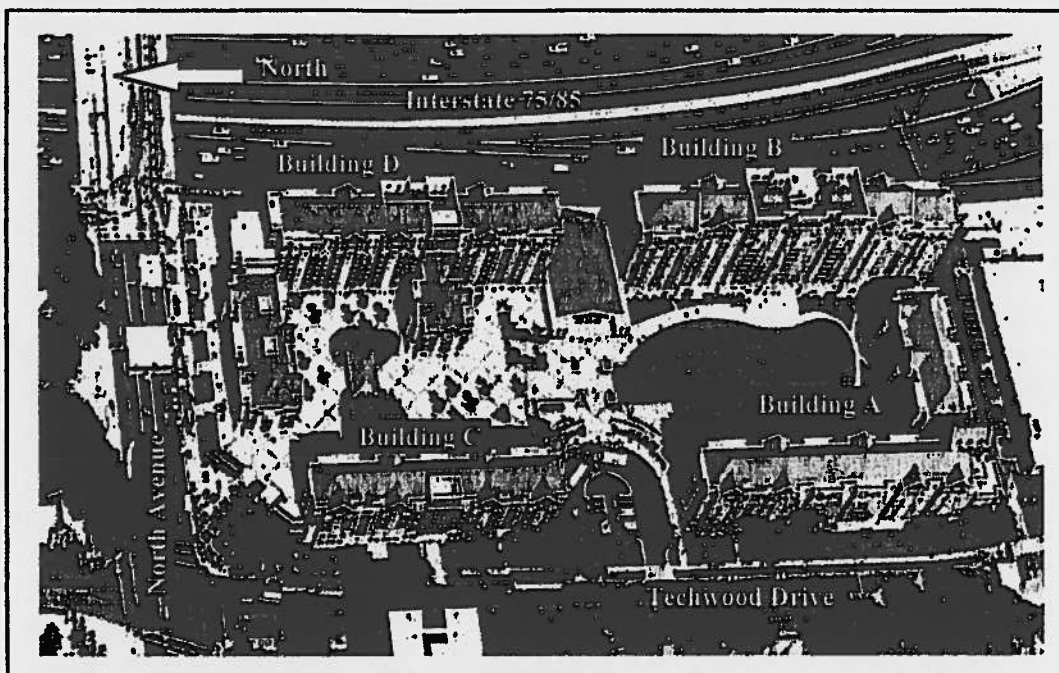


Figure 1: Plan view of building layout

Construction of the complex was begun during the early part of 1994. As construction progressed, some differential settlement of portions of Buildings B and C were noted by construction personnel. Shortly thereafter, a number of survey points were established within these two buildings to gauge the magnitude and trend of the movement. From the summer of 1994 to the summer of 1995, the data indicated that the northern end of Building C had settled almost 4 1/2 inches. Little movement was noted at the southern end of this same building. Points established on the northern and southern ends of Building B indicated movements of up to about 4 inches. Despite the fact that the structural frame of each building had been completed, survey points indicated continuing movement.

As the recorded settlements in the two buildings far exceeded the predicted 1 inch, the state agency responsible for the construction commissioned a forensic study of the geotechnical and structural aspects of the project. The remainder of this paper primarily focuses on the geotechnical portion of this study.

INITIAL WORK

The initial phase of the forensic study consisted of two major tasks. The first of these was to determine the magnitude and extent of the settlement problem in Buildings B and C. The second involved a review of available data and documentation relating to the design and construction of the complex as a whole to seek possible reasons for the observed excessive settlement.

SURVEYS

To evaluate the extent of the problem, elevation surveys were conducted of the first structurally supported floors within both

Buildings B and C. Prior surveys had primarily concentrated on discrete points located at the ends of each structure. The new surveys revealed that Building B, supported by a mat foundation, had a significant sag in the center, measured to be nearly 5 inches lower than the ends in a longitudinal direction. When taken in conjunction with the roughly 4 inches of settlement measured at the ends, a total settlement of about 9 inches was determined at the building center. The survey of the first floor of Building C, which was supported on shallow spread footings, revealed approximately 5 inches of differential settlement between the south and north ends. Combined with previous survey data, a total settlement of 6 1/2 inches was calculated at the north end of the building, verses 1 1/2 inches of settlement at the south end.

DOCUMENT REVIEW

The documents reviewed as part of the study included the original site grading plan, the geotechnical report, foundation plans, and construction site grading reports.

Site Grading Plan Briefly, the site originally sloped gently downward from the east to the west a vertical distance of approximately 30 feet. It had been occupied first by single family homes until the 1930s, then by a two story, low income housing complex and low-rise commercial properties. The new construction was designed to follow site topography, but included substantial excavation for the eastern segment of a below-grade parking deck located between buildings C and D. Grading within the remainder of the complex was generally limited to cuts and fills on the order of 15 to 20 feet.

Geotechnical Study The geotechnical study originally conducted on-site included a total of 30 soil test borings extending to depths of 35 to 85 feet, and a laboratory program consisting of classification tests, one triaxial shear test and nine consolidation tests.

The soil test borings revealed that some fill soils were present in a few areas of the site. However, most of the overburden consisted of residual sandy silt and silty sand soils weathered from the native biotite gneiss and schist bedrock. Standard penetration test values measured in these residual soils were typically on the order of 6 to 12 blows per foot within the upper 20 to 25 feet of the soil profile, gradually increasing to greater than 100 blows per foot with depth. Consolidation tests revealed that the residual materials had a constrained modulus ranging from about 60 ksf to about 200 ksf.

Material sufficiently hard to cause auger refusal was encountered as shallow as 35 feet, but was more typically found at depths of 60 or more feet. The water table was encountered at depths of about 20 feet below grade.

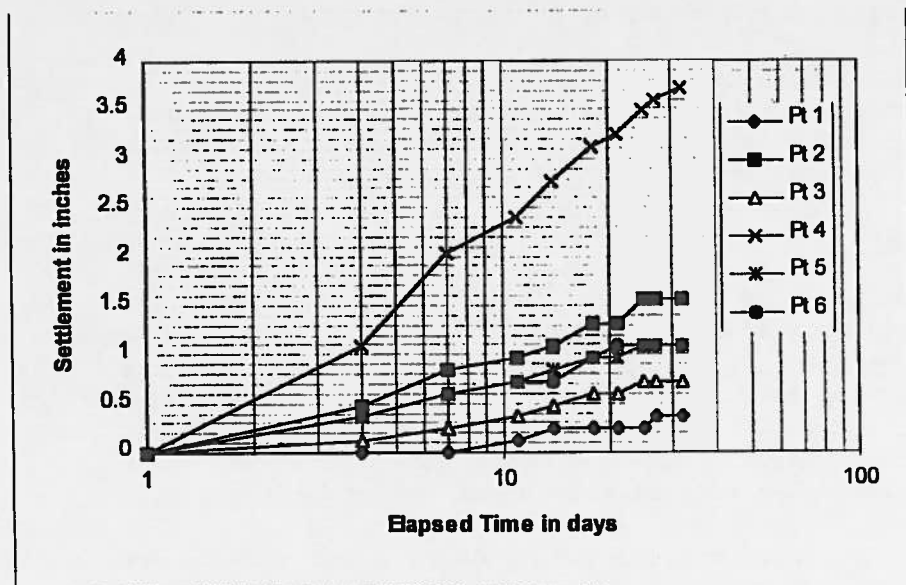
Foundation Design Based upon the results of the field and laboratory studies, it was recommended that the buildings within the southern half of the site (A and B), be supported on mat foundations. Due to the difference in height between the two buildings, the design bearing pressures for the mats varied between 2 ksf for Building A and 3 ksf for Building B. A modulus of subgrade reaction, "k" of 200 pci was

recommended for design. During the course of the design of these mats, the decision was made to place 7 feet of fill between the mat and the lowest floor slab within Building B to ease utility construction. This would increase the total pressure exerted by the Building B mat to nearly 4 ksf. To reduce the post-construction settlements associated with the mats, it was recommended that each of the two sites be preloaded with 8 feet of fill prior to foundation construction.

The geotechnical report recommended that Building C be supported on shallow spread footings designed for a bearing pressure of 4 ksf. To accommodate an entrance to the parking deck, the southern portion of this building was stepped downward one level. A soil supported plaza was included in the design, bearing 20 feet above the lowest level of the building on the north side. To support the retaining wall for the plaza fill and the adjoining columns, a single large strap footing was planned for this area.

As subsurface conditions were found to be variable beneath the footprint of Building D, it was recommended that it be supported by auger-cast piles. The high rise section was designed using piles extending to bedrock, while the northernmost 5 story wing was designed to employ friction piles extending to a depth of 50 feet below grade.

Site Surcharging The surcharging program was performed as outlined in the geotechnical report between the period of April 29 and May 31, 1994. During this period of time, elevation surveys were conducted to gauge the rate of movement (see Figure 2, below).



The surcharge was removed after a period of 32 days, when the geotechnical engineer of record judged that the majority of primary consolidation was complete. As may be noted, settlements of nearly 4 inches were measured as a result of the application of only about 1000 psf of area load. This was clearly not compatible with the predicted settlement of only 1 inch under a mat foundation load of over 3 ksf.

ADDITIONAL STUDY

The next phase of the forensic study involved limited additional data collection, an independent evaluation of predicted settlements based on the field and laboratory data, and the establishment of a long-term settlement monitoring program for each of the buildings within the complex.

FIELD STUDY

To verify the results of the original borings, 2 dilatometer soundings were advanced on the north and south sides of Building C. Representative undisturbed samples of residual soil were then obtained for long-term consolidation testing. Briefly, the two soundings revealed dilatometer moduli ranging from about 100 ksf to 200 ksf within the upper 20 to 30 feet of the soil overburden. Tests conducted on undisturbed samples of residual soil revealed that it had a dry unit weight on the order of 76 pounds per cubic foot and constrained moduli on the order of 100 to 150 ksf. As the settlement of the buildings on-site was continuing several months after completion of the structures, loads were maintained on the consolidation test specimens for up to 2 days each to determine the coefficient of secondary compression. In the tests, C-alpha values generally were found to range from about .1 to .2 percent strain per log cycle of time. All of the test results obtained during the supplemental exploration program were judged to be consistent with the results of the tests conducted during the original geotechnical study.

ANALYSIS

Following the supplemental field and laboratory study, settlement analyses were conducted for Buildings B and C. To gauge the order of magnitude of settlement, the initial analysis employed, a simplified elastic equation. The elastic analysis of the Building B foundation is reproduced below:

$$S = p B \frac{(1 - \mu^2)}{E} I$$

Where

S = Settlement in inches

P = Contact pressure in psi (13 psi considering surcharge and 50% live load)

B = Foundation width in inches (720 inches)

μ = Poissons ratio of soil (assumed 0.35)

I = Influence factor related to foundation shape (1.56 for 5:1 L:W)

E = Young's Modulus of soil (1042 psi from laboratory tests)

The value of "S" resulting from the above calculation was 12.3 inches which compares favorably with the measured settlement of over 9 1/2 inches.

Following the elastic analysis, a more detailed model incorporating the size and location of individual foundation elements and soil strata

was used in a computer program to more accurately predict settlements. This program employed conventional consolidation theory in conjunction with a Westergaard pressure distribution. The results of these analyses were found to closely match the actual measured settlement in each of the two buildings.

In the case of Building B, the use of a Modulus of Subgrade Reaction of 200 pci in design had resulted in a mat only 3 1/2 feet in thickness. Because it was thin, the mat acted as a flexible foundation element and settled more in the center, where the highest concentration of loads was located. Outer portions of the building settled significantly less due to the lower net loading. On-going settlement of this structure was consistent with the 60 foot thickness of residual soil beneath the foundation and the rate of secondary compression determined from the laboratory consolidation tests. Using this data, it was predicted that the building would settle up to an additional 3 inches over the next 20 years in the center and from 1 to 1 1/2 inches at the ends.

The differential settlement noted in Building C was attributed to the significantly different loading conditions on the north and south sides. On the south side, the site had been excavated 10 to 12 feet below original grade. Consequently, the net loading on the footings was far less than the 4 ksf design. The large strap footing on the north side of the building was influenced not only by the structure above, but by the area load induced by the mass of fill placed exterior to the building (see Figure 2 below). When these factors were considered, the magnitude of differential settlement which was actually measured was found to be predictable from soil and structure characteristics.

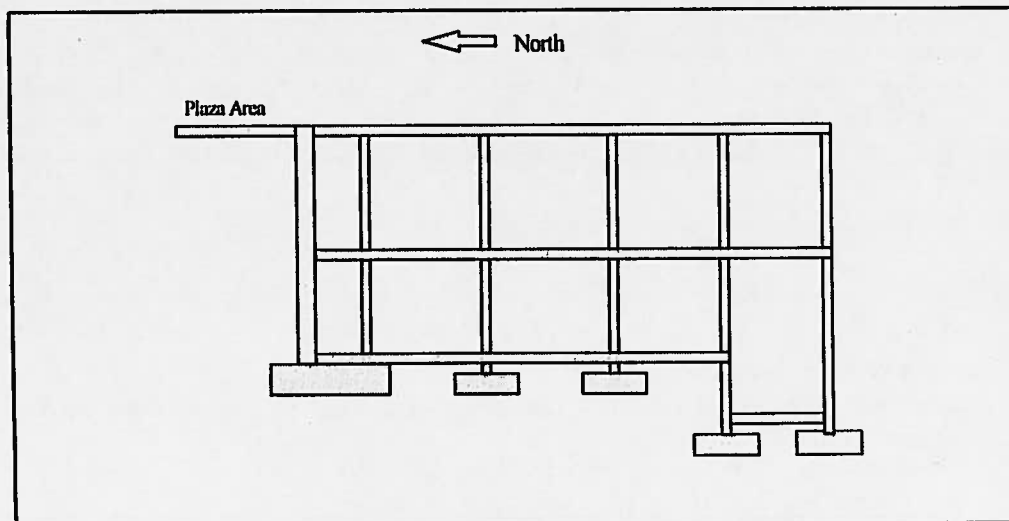


Figure 2: Cross-section through Building C

Following completion of the settlement analysis, the actual and projected future building settlements were given to the project structural engineers for their analysis. Although differential settlements were far greater than anticipated, they were found to transition gradually across each of the two buildings. As a consequence, structural analyses indicated that none of the critical

building components would be overstressed by the deformation. However, recommendations were made to expose and examine utility connections, and to establish a long term monitoring program of the buildings within the complex.

ADDITIONAL SETTLEMENT MONITORING

To confirm the predictions of long term settlement made for the two buildings under consideration in this study and to determine if other buildings were settling, a comprehensive long-term settlement monitoring program was established for the complex. In the program, over 50 permanent survey points were established in the 4 buildings. The elevation of these points were measured on a monthly basis through June of 1996. Following the Olympic Games, monitoring will be conducted quarterly for at least 3 years.

The monitoring program has detected only minimal movement of Building A since points were established in September of 1995. If a live load reduction is considered, this mat only exerts a pressure of about 1500 psf to the underlying soil. As this is only slightly higher than the pressure induced by the surcharge program, it is thought that preloading was effective for this structure.

Building B has shown continued movement consistent with secondary compression since initiation of the monitoring program. Figure 3, presented below, indicates the trends of settlement of this building since the mat foundation was poured in June of 1994. For reference, the structural frame was completed on about day 180 on the graph. Exterior brickwork was completed by day 360.

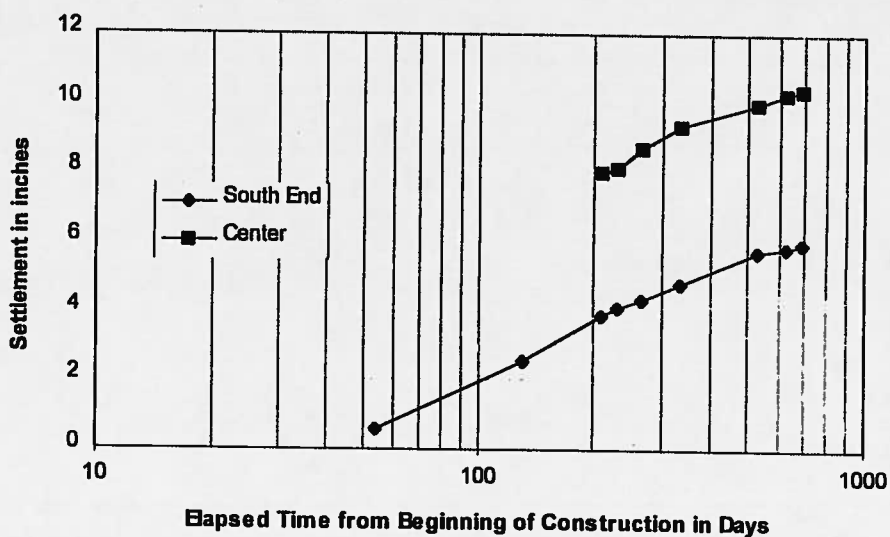


Figure 3: Settlement Trends - Building B

Building C has also shown continued movement during the monitoring period as indicated on Figure 4, below. Again, the rate of movement appears to be consistent with the coefficient of secondary compression determined during from consolidation tests.

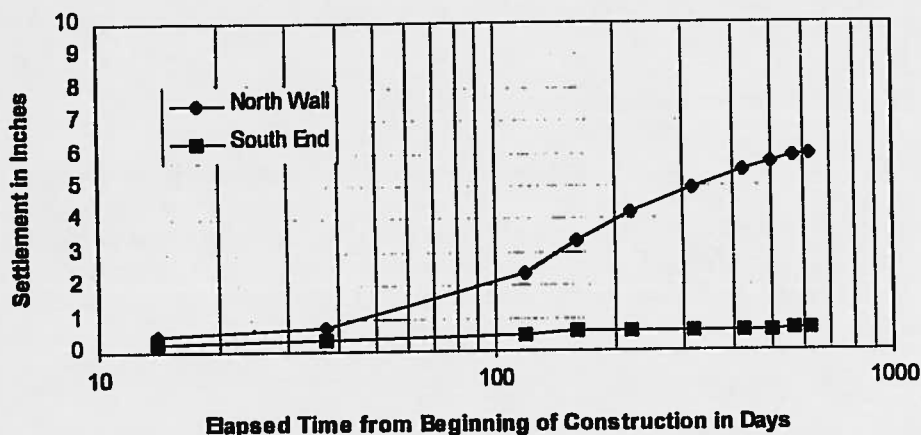


Figure 4: Settlement Trends - Building C

As Building D was supported on piles, little movement was anticipated. However, surveys of the 5-story north wing of the building revealed that the western side was settling at a significant rate when compared with the eastern side (see Figure 5). After researching design and construction documents, it was discovered that up to about 15 feet of new fill had been placed within the western end of the wing immediately prior to pile installation. The piles extended to a total depth of 50 feet below final grade, which left up to about 30 feet of compressible soil beneath the pile tips. Calculations indicated that the magnitude and rate of settlement were predictable as a result of area fill loads and the loads induced to the soil from the pile groups.

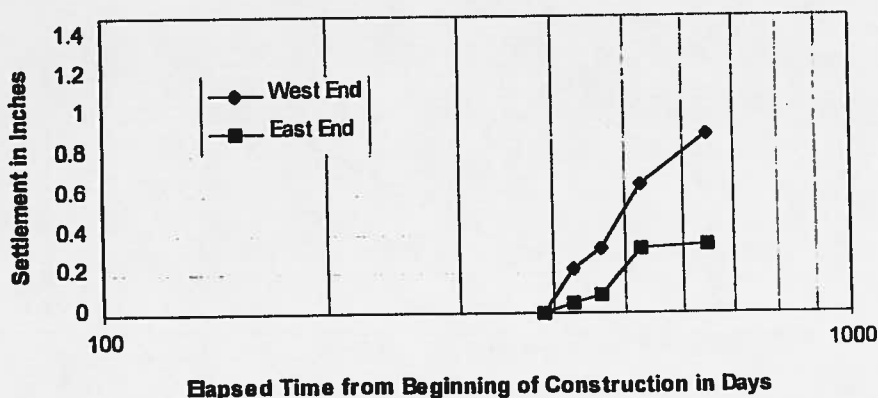


Figure 5: Settlement of North Wing - Building D

LESSONS LEARNED

This project provides a number of important lessons for practicing geotechnical engineers. First and foremost, the classical theories of soil mechanics dealing with stress and consolidation can provide accurate predictions of building settlements, if properly applied. When calculating settlement, it is always advisable to employ simple elastic formulas as a quick check of the order of magnitude of projected settlements.

Second, surcharge programs can and should be used to confirm predictions of settlement as well as to improve in-situ soils. Had the excessive magnitude of settlement of the surcharge been recognized during this project, some adjustment or modification of the foundation design could have been effected prior to construction.

Third, the Modulus of Subgrade Reaction, "k", used in design should consider the size and rigidity of the foundation. For a mat foundation, the design "k" value can be properly determined only through an iterative process involving both the structural and geotechnical engineer. For large mats on thick soil deposits, "k" values should typically be in the range of 10 pci to produce a relatively rigid foundation element.

Finally, when using friction piles to support structures, it is essential that site grading plans be considered in the calculation of predicted settlements. Significant fill, as well as groups of piles can induce far greater settlements than would be expected from the results of a single pile load test.

Geotechnical Failures - Case Histories

An Analysis of the Causes

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Introduction

Detailed analyses and extensive discussions of failures in Geotechnical Engineering are invaluable for learning from these failures and to prevent similar ones from happening again. Yet information concerning failures is often hidden. Even when litigation ensues, the experts analyzing a failure are frequently hampered in obtaining the complete facts and information and are prevented from speaking to the opposing experts. When the litigation is over, no one is willing to spend the money and effort for further more complete studies. Not only is the case forgotten but frequently the experts are forbidden to publish their findings. No one wants to publicize his possible mistakes or even his involvement in a project that failed. While this attitude is natural and understandable, it doesn't advance our profession and help to prevent future failures. The failures which are discussed in this paper are those in which the writer has had personal involvement. Though in most of the cases, extensive information was obtained, the writer cannot guarantee that there hasn't been some deliberate concealment of facts which might render the conclusions as to the causes either invalid or only partially valid. Nevertheless, information, though perhaps not complete, is invaluable for discussion, analysis and learning.

Causes and types of failures

While it is often not possible to pinpoint one single cause of a failure it is worth considering the possible causes. These may be classified as:

Design

Failures in design are those in which the engineer has failed to design adequately for all the known and possible future forces to which the structure will be subjected. This may involve mistakes in analysis and calculations or using incorrect or invalid design methods. While it may seem to many that this

is the major cause of failure, the writer's experience does not lead to that conclusion. As a matter of fact, geotechnical structures are far more frequently overdesigned than underdesigned. This is largely though not wholly due to the use of large factors of safety to allow for lack of adequate information concerning the soil profile and soil properties.

Construction

Failures in construction are far more frequent than design failures. These include failure to build the structure strictly according to plans and specifications; failure to observe and communicate conditions encountered which are different than those assumed in the design; failure to adequately control such forces as ground water flow, soil squeezing, seepage etc. - forces which deteriorate or destroy the structure during construction; failure to have sufficient and adequate supervision and inspection.

Exploration

Failures in exploration are those which fail to reveal the depth, extent and variations and properties of the soil sufficiently or complete enough to form a correct basis for design. This includes insufficient number and depth of borings, failure to observe and record completely conditions encountered while making the soil borings, insufficient in-place and laboratory soil testing, and erroneous results due to sample disturbance. These happen due to ignorance, carelessness, and efforts to save money on "unnecessary" soil exploration. More subtle is incomplete and erroneous information obtained because of failure to understand the geology of the area in question and to use the many tools other than soil borings to make the exploration complete and accurate.

People failures

The term "people failure" is used to denote lack of communication, lack of proper observation, and lack of ability to interpret, appreciate and communicate occurrences, changes, and observations to the proper persons for action and resolution. Even when the observations are communicated, there is often a failure in the chain of command so that the information fails to get to the proper person or persons for action. Drillers are often sent out to make borings without sufficient instructions as to why the borings are being made, what information is needed, what should be observed and reported, how the samples should be taken etc. Inspectors are often sent out to the field without telling them what they are to inspect! Inspectors are often not sufficiently qualified to know the significance of what they are inspecting. Job

superintendents often don't know what the engineers intentions are or why he designed an element a particular way. Design engineers don't communicate with the field personnel to explain their designs and to inform them what to look for - these are all "people problems"

Unknown and unanticipated conditions

If none of the above are the causes of failures we can usually attribute them to unknown and/or unanticipated conditions. However, this cause should never be used as an excuse to not attribute a failure to one of the above causes. For example, we should not attribute a failure due to a soil conditions different than one expected and shown in the borings unless there has been a sufficient number and depth of borings, the borings have been made in the proper locations according to the known geology of the area, that sufficient numbers, type and accuracy of the soil tests have been performed, and that there has been no failure to observe and properly report those conditions if they were observable and revealed. This cause of failure is the most excusable and sometimes is referred to as "an act of God". However, it is more likely due to man's ignorance, and that in spite of not being attributable to the other causes listed, knowledge of the failure can add to our experience and perhaps be preventable in the future.

The remainder of this paper is devoted to case histories and analysis of the causes of failure.

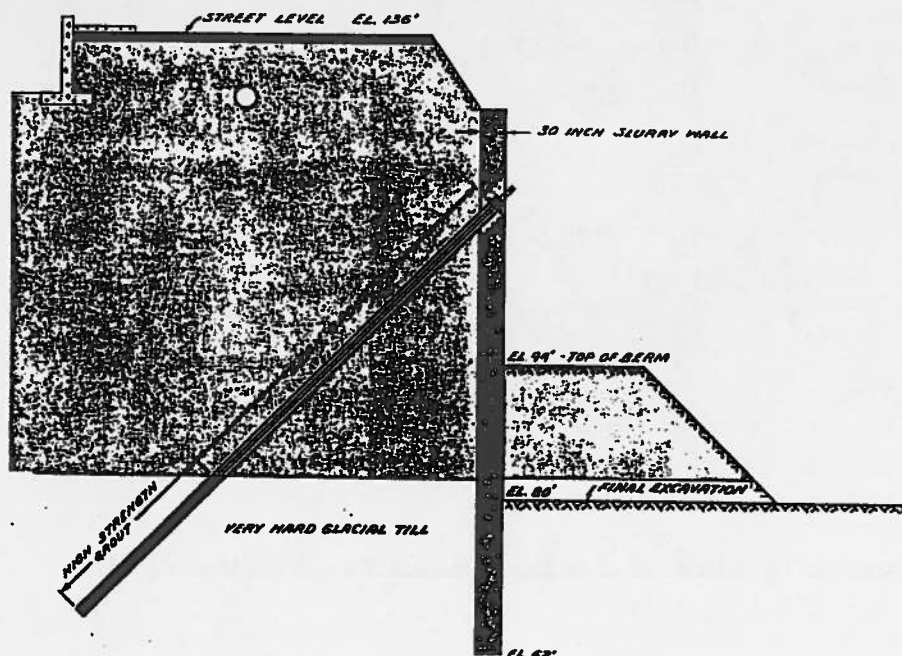


Figure 1 Tieback System before Addition of Support From Permanent Floor Members and Removal of Berm

Case 1. Slurry Wall failure

A deep basement was to be excavated for a new building in the downtown area of a large U.S. midwestern city. Excavations up to 20 meters (65 ft.) were required. The upper 16 meters (53 ft.) consisted of a stiff brittle loess below which is a hard glacial till. To retain the soil a slurry wall was constructed by installing 760 mm. (30 in.) I-beams at intervals in predrilled holes 20 feet into the glacial till and filling with concrete. Each panel was excavated in a bentonite slurry, a reinforcing steel cage lowered and concrete pumped in from the bottom displacing the slurry above. The wall was supported by only one row of anchors as shown in Fig. 1. The anchors were installed after the soil was excavated to 6 m. (20 ft.) below street level. The anchors consisted of high strength wire strand tendons grouted into the hard glacial till over a length of 6 m. (20 ft.).

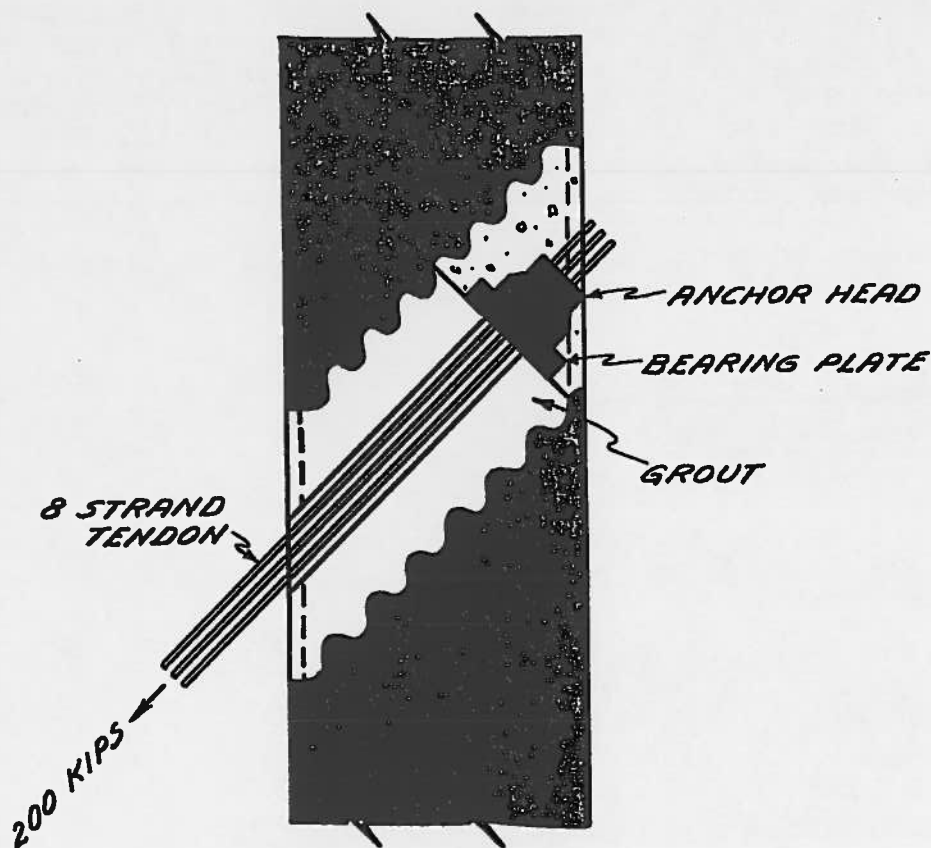


Figure 2 Detail of Anchorage Through Slurry Wall

Details of how the anchorage was held in the wall are shown in Fig.2. Each anchor was designed to hold 100 tons and was prestressed to 125 tons and locked into the wall by a three segment wedge grip. After installation the soil was to be excavated to the final depth leaving a berm as shown in Fig. 1.

After installing two struts extending from the building frame as shown in Fig.3, The berm was then to be removed. The contractor did not follow the specified installation and excavation procedure and instead excavated to the final depth without leaving the berm as shown in Fig.4. Shortly after the wall fell in with the result shown in Fig.5. This broke the sewer and water main, flooding the excavation.

The contractor claimed the failure was due to a break in the water main causing a full hydrostatic head behind the wall for which it was not designed. However, a lawyer who was working in a nearby office at the time, jumped up on hearing a load noise and looked out his window to see the wall actually in the process of falling over. He immediately wrote a very wordy description of what he saw which included a statement that for a minute or so after the wall fell into the excavation, the sewer and water main remained suspended in the opening caused by the failure and then broke suddenly, flooding the excavation.

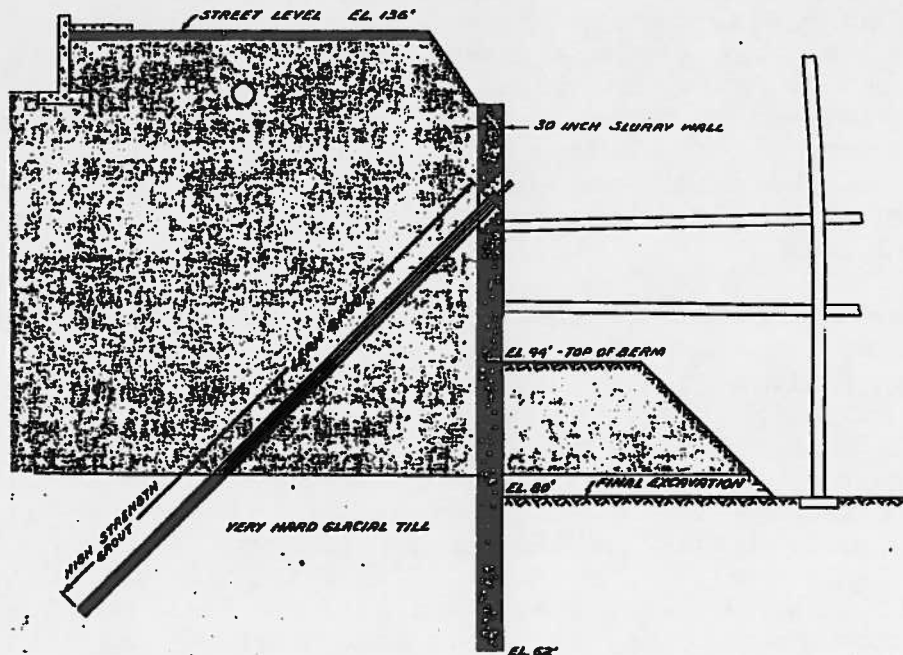


Figure 3 Tieback System with Lateral Support from Floor Members

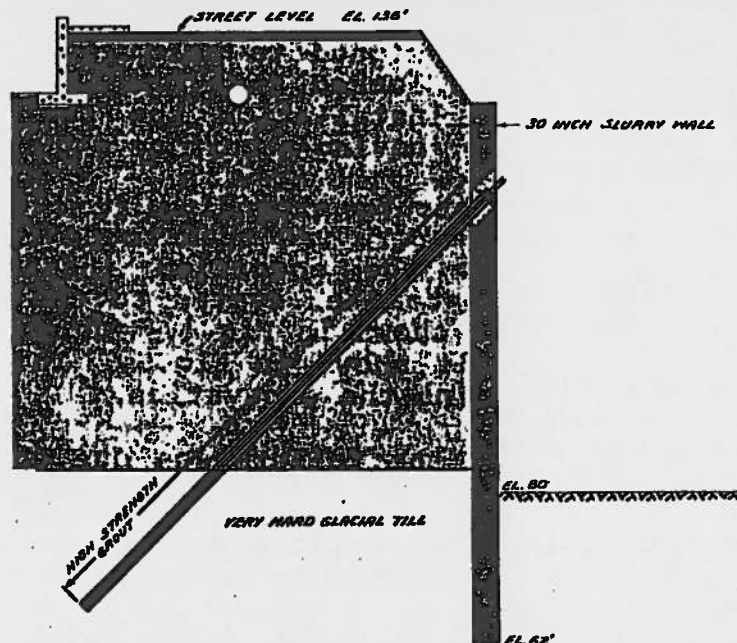


Figure 4 Actual Construction Procedure - Soil Removed to Final Depth Before Lateral Support From Floor Beams Are Placed

Why did the wall fail? Evidence was clear that the grouted anchorage in the glacial till did not fail nor was there any evidence that the tendons failed although they should have been stressed to approximately the failure load. Clean shiny teeth marks on the otherwise partly rusted tendons and shiny marks on the teeth of the three piece wedge grips indicated that the tendons slipped through the grips. The load in excess of that tied in by preloading could not be taken by the wedge grips because they could not slide in the cone shaped grip holders since they were rusted in the holder. Thus the grips could not slip in the holder and dig deeper into the cable tendons to take the increased load.

What was the cause of the failure and how could it have been prevented? Certainly the immediate cause was the contractor's failure to follow the prescribed construction procedure. But why didn't the job inspector prevent the contractor from excavating the berm prematurely? This was a "people failure". And was there any design failure involved? Although the design calculated correctly, there was little margin for unanticipated increases in loading. The anchors should have been designed so that if one failed the adjacent anchors could take the increased load without failing. Instead when one anchor failed they all went like a zipper opening. A second row of anchors would have added more redundancy to the system by distributing the total load to more anchors. The design depended on near perfect execution of the work and near perfect performance of the system and its components. The design, then could also be considered to have contributed to the failure.

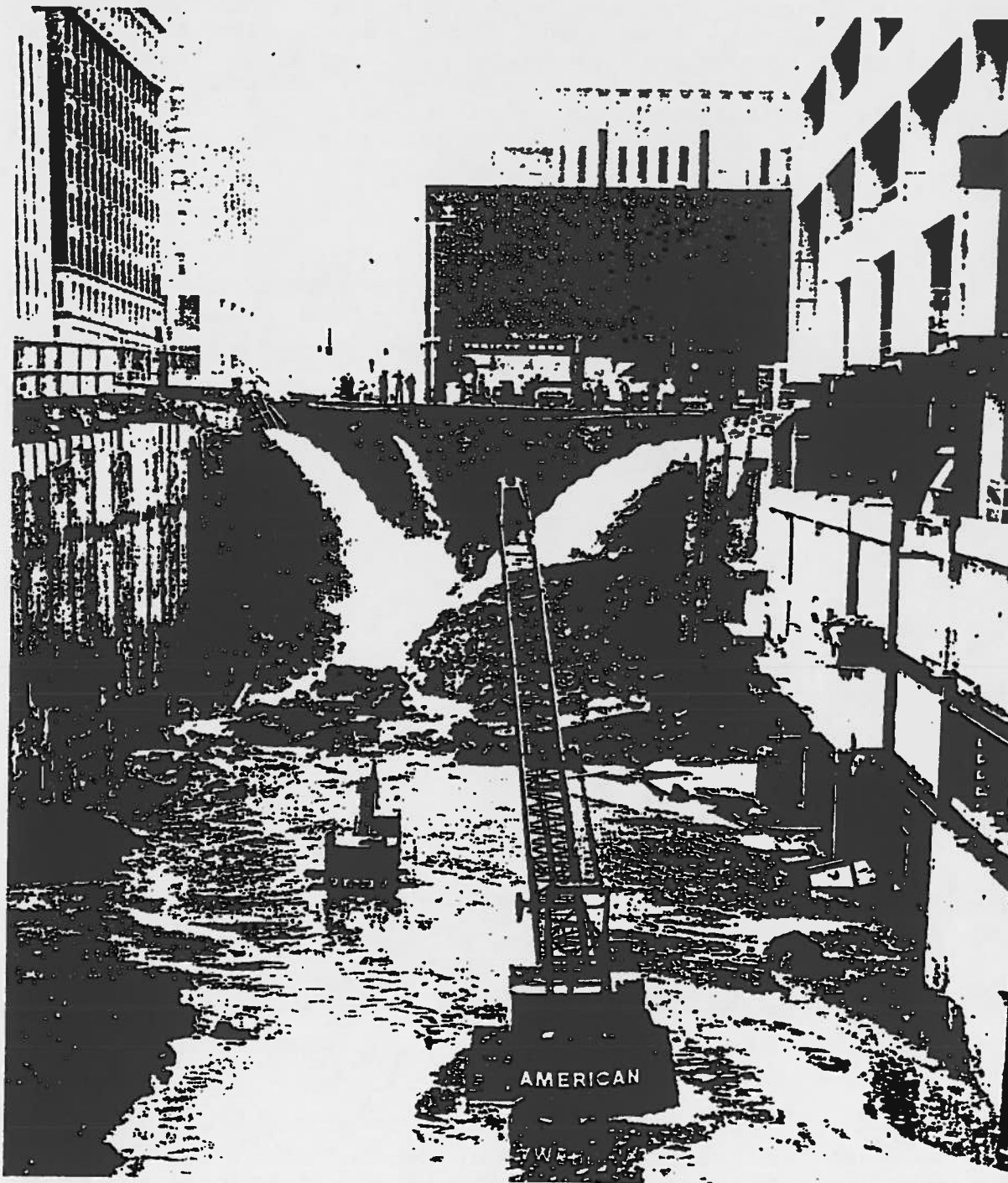


Figure 5 Excavation Shortly After Wall Failure

Case 2. Failure of soldier beam and lagging circular wall

A 35 m. (117ft.) diameter excavation was to be made as an entrance shaft to a tunnel to be constructed. Only one boring was made at the center of the shaft which showed 10 m. (34ft.) of sandy silty clay with an abrupt transition to a chalky claystone which continues to the depth of the boring at 45 m. (150 ft.). The tunnel was to start at 23 m. (76 ft.) in the

claystone. No other borings were made within a hundred meters of the shaft location. Since there was no water bearing sand or free water indicated, the most economical type of bracing system to use is a soldier beam and lagging design with steel ring beams supporting the soldier beams at intervals. This system was partially designed and suggested to the bidders. However, it requires continuous support from the soil. If there is loss of soil from behind, there is danger that the whole system could collapse.

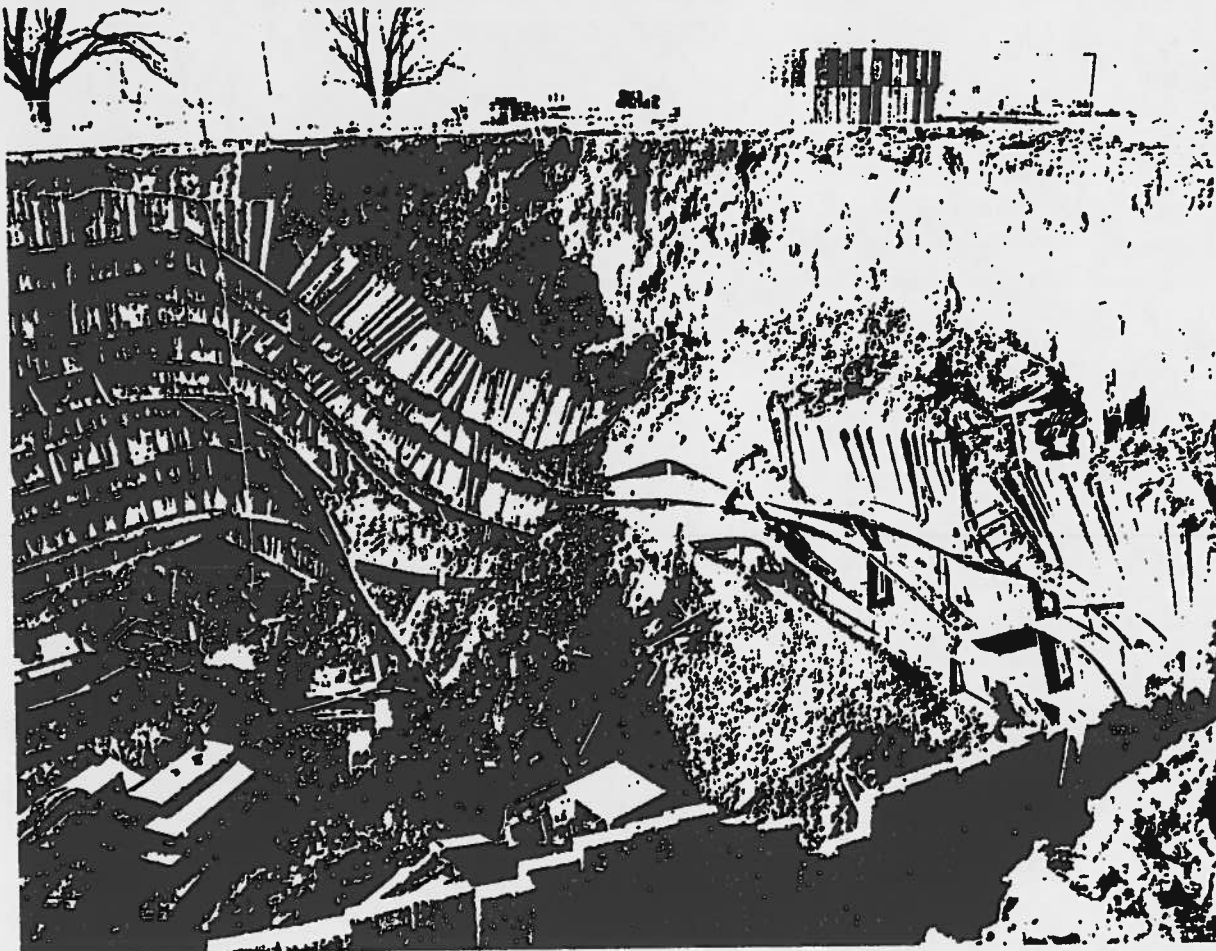


Figure 6 Failure of Circular Wall

When the contractor had almost reached the claystone, and the last section of wood lagging was being installed, water started to flow profusely in the bottom washing sand from behind the lagging. This caused the soil above to collapse and the lagging supporting it to fall in the hole created. This loss of support caused the ring beam to twist and soon virtually the whole wall fell in as shown in Fig. 6. The cost of the loss was about \$2,500,000. The debris was removed and the excavation converted to an open cut, requiring a vast amount of extra soil excavation.

After the debris was removed it was clear that the section which first started to collapse was immediately above a buried stream bed. The otherwise almost flat and horizontal rock surface was carved out by the river bed which was filled with water bearing sand and gravel. It was obvious that the water flowing out of the stream bed from behind and underneath the wall washed the sand and gravel out from under the soil above causing it to cave in, thus triggering the collapse. The profile as indicated by the one boring made at the center and the actual conditions found are shown in Fig. 7.

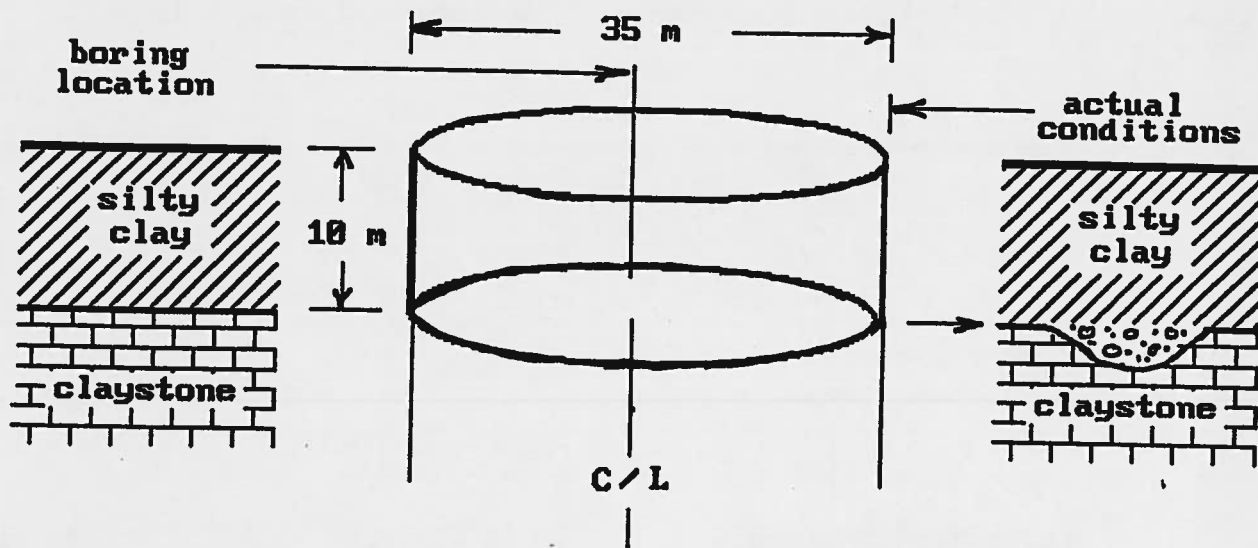


Figure 7 Profile According to Soil Boring and Conditions as Found - Note Buried Stream Bed

Clearly this was an exploration failure. One boring at the center of the large area was hardly enough to indicate or discover the buried stream bed. It is true that three or four additional borings around the perimeter still might not have discovered the buried stream bed. However, geologists familiar with the area knew that buried stream beds on the top of the claystone existed in the general area though some miles away. With this knowledge, some type of seismic exploration would be called for which almost certainly would have discovered the stream bed. Thus we can say that the exploration failure was also a "people failure".

Case 3. The wrong foundation design caused by insufficient and faulty soils information.

Foundations for a high-rise building in the Midwest was designed based only on the boring logs and the SPT (Standard Penetration Tests) indicated on the logs. No water content tests, nor any other laboratory tests were made. The soil

samples were not inspected by any one in the architect's or design engineer's offices. On the left side of Fig. 8 is a composite of the borings with the classifications as given by the driller. A sufficient number of borings were made to indicate that the conditions over the site were quite uniform. It was decided that the structure should rest on piles. A contract was let for mandrel-driven shell piles to be driven an estimated 15 m (45 ft.) into the hard glacial till. The piles could only be driven 4 to 5 meters (14-16 ft.) into the "stiff gray very silty clay" before they met refusal! After two weeks of frustration at not being able to drive the piles to the required depths, the writer was asked to make a geotechnical investigation. Additional borings were made in which undisturbed samples were taken. Water content, unconfined compression and consolidation tests were made. A composite profile of the information obtained is shown on the right side of Fig. 8.

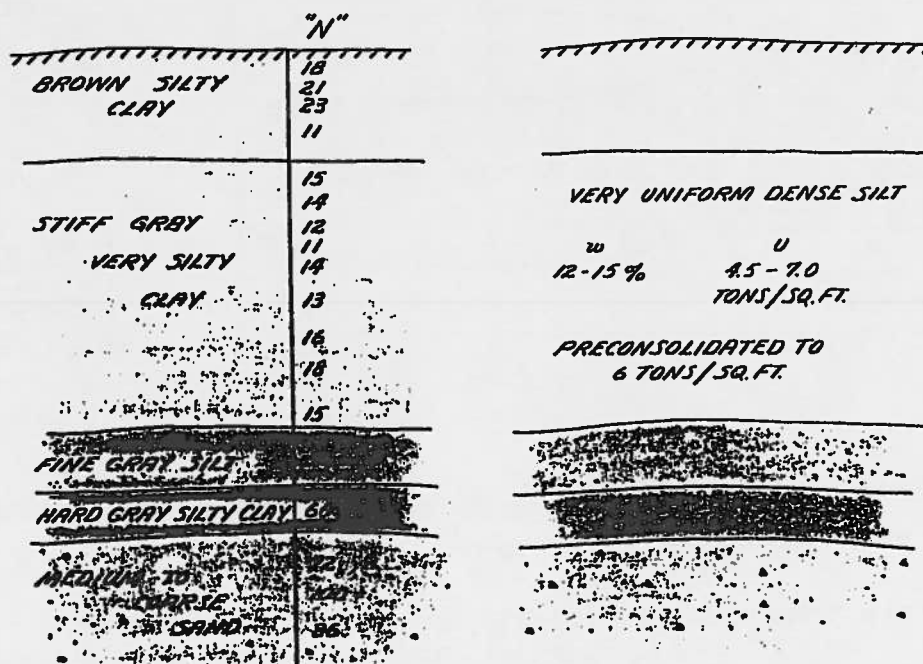


Figure 8 (a) Complete Boring Log on Which Design Was Based,
(b) Profile Determined One Year Later From Undisturbed Samples

It became apparent that the "stiff gray very silty clay" with SPT's of 11-18 was in fact a very uniform dense silt with water contents of 12-15%, unconfined compressive strengths of 440-660 kN/sq m (4.7-7.0 tons/sq ft.). and preconsolidation stresses of about 560 kN/sq m (6 tons/sq ft.) It had been known that the soil had been consolidated by the weight of at least 1,000 m (3,000 ft) of ice during the glacial age. It was obvious that the structure need not rest on piles but could be placed on spread footings resting on the silt using high bearing loads. Though the recommendations of the writer were accepted

by architect, structural engineer, contractor and the superintendent of public works of the city, the only dissenter was the city engineer who was the one who originally ruled that the structure should be placed on piles, and that if it did not, he would resign his job. After considerable discussion, everyone else yielded to his wishes and the structure was placed on piles, but not without difficulty. At each pile location it was necessary to auger a hole through the dense silt, drop the pile in the predrilled hole, and drive the pile the last few feet into the sand!

Was this a foundation failure? Yes, because the foundation as designed could not be executed, but more importantly because it was a far more expensive design than needed. But basically, it was an exploration failure, because the exploration program was not designed to reveal the true nature of the soil. And, of course, it was also a "people failure"

Case 4. A bridge collapse.

A failure occurred on an Interstate highway bridge during construction with a loss of 13 lives. The failure (shown in Fig. 9) was on a ramp leading to the bridge when the last pour of concrete was underway for that section. The ramp was on a grade, a curve and was superelevated so that each leg of the scaffolding supporting the framework for the poured-in-place concrete had a different length. Each scaffold contained four legs (Fig. 10) with each leg resting on a 1.5x1.5x0.3 m (5x5x1 ft.) slab (Fig. 11). The permanent piers for the bridge were on driven piles. The slabs were simply placed on the ground without any soils investigation having been made.

After the collapse, borings and soil tests indicated that the slabs rested on 0.6 to 1.5 m (2 to 5 ft.) of slag under which were organic silts and clays of moderately high water contents. Water contents of the organic soils indicated that settlements of 5 to 8 cm (2 to 3 in.) were possible, enough to cause collapse according to a finite element study made after the collapse. Settlements were critical because of each leg being of a different length and the structure being on a curve.

The settlement, however, could have been less than this due to consolidation caused by drying. Without allowing time for making consolidation tests to determine the preconsolidation stress and obtain a more accurate estimate of the settlement, those in charge of the investigation decided to make a full scale load test on four of the slabs simultaneously. Under suspicion were the slabs themselves because many of the slabs in sections which had not failed showed small cracks in the center of the sides (Fig. 11.). The loading of the four slabs were controlled hydraulically and were kept equal during the test. When the load on each of the four slabs reach 85% of the design load, each slab failed suddenly, with failure of one following

the other in a matter of seconds. The failure is shown in Fig. 12. An examination showed that the slabs failed because they had almost no reinforcement and not because of excessive settlement.

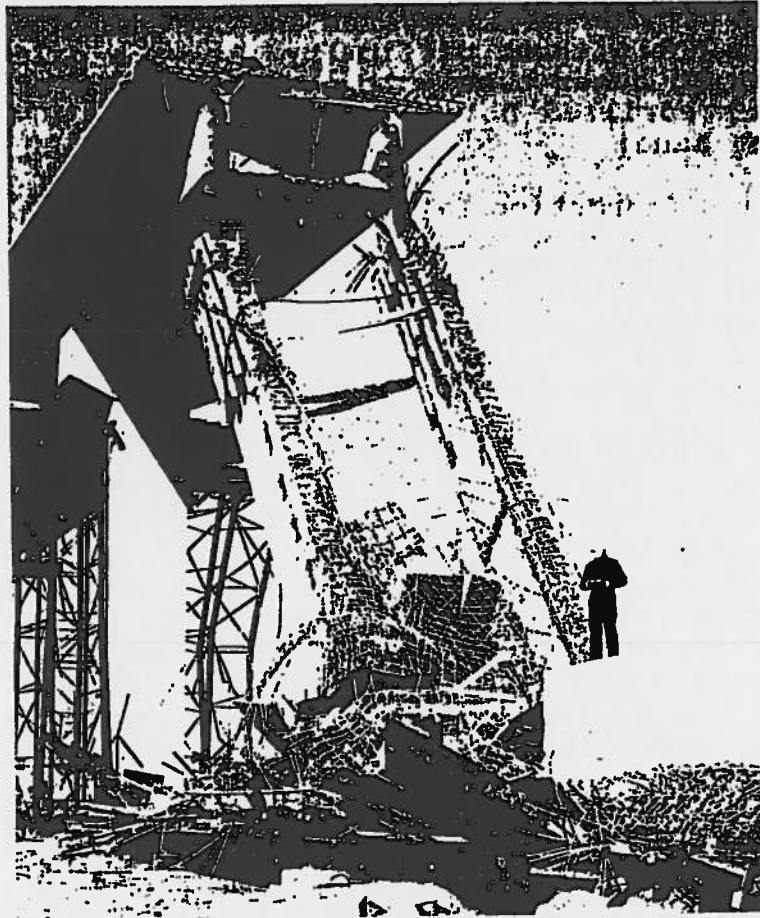


Figure 9 Collapse of Highway Ramp During Concrete Pouring Operations

Was this a geotechnical failure? No, but it could have been. This was clearly a structural failure and clearly the fault of the contractor. The slabs and the scaffolding were temporary structures, which was the contractor's responsibility and thus was not inspected nor approved by the structural engineer who designed the bridge. Though it was a structural failure of a temporary element, it was also a "people failure", since no one had complete responsibility for the entire job. Certainly there should be some mechanism by which the designs and calculations of temporary structures are checked independently of the contractor.

Case 5. Settlement of large bored piles

Fig. 13 shows a large diameter bored pile, one of many to support the columns for a large steel mill on the shore of Lake

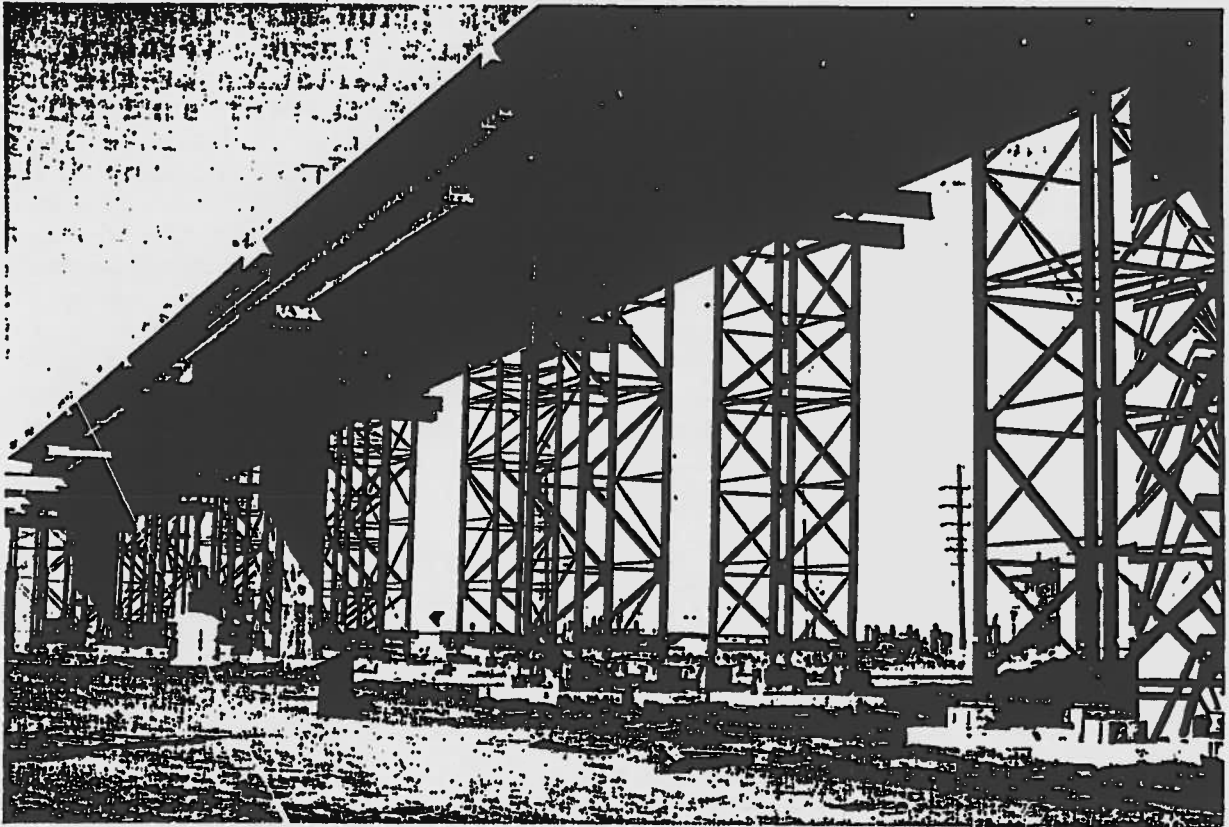


Figure 10 Scaffolding Supporting Concrete Forms

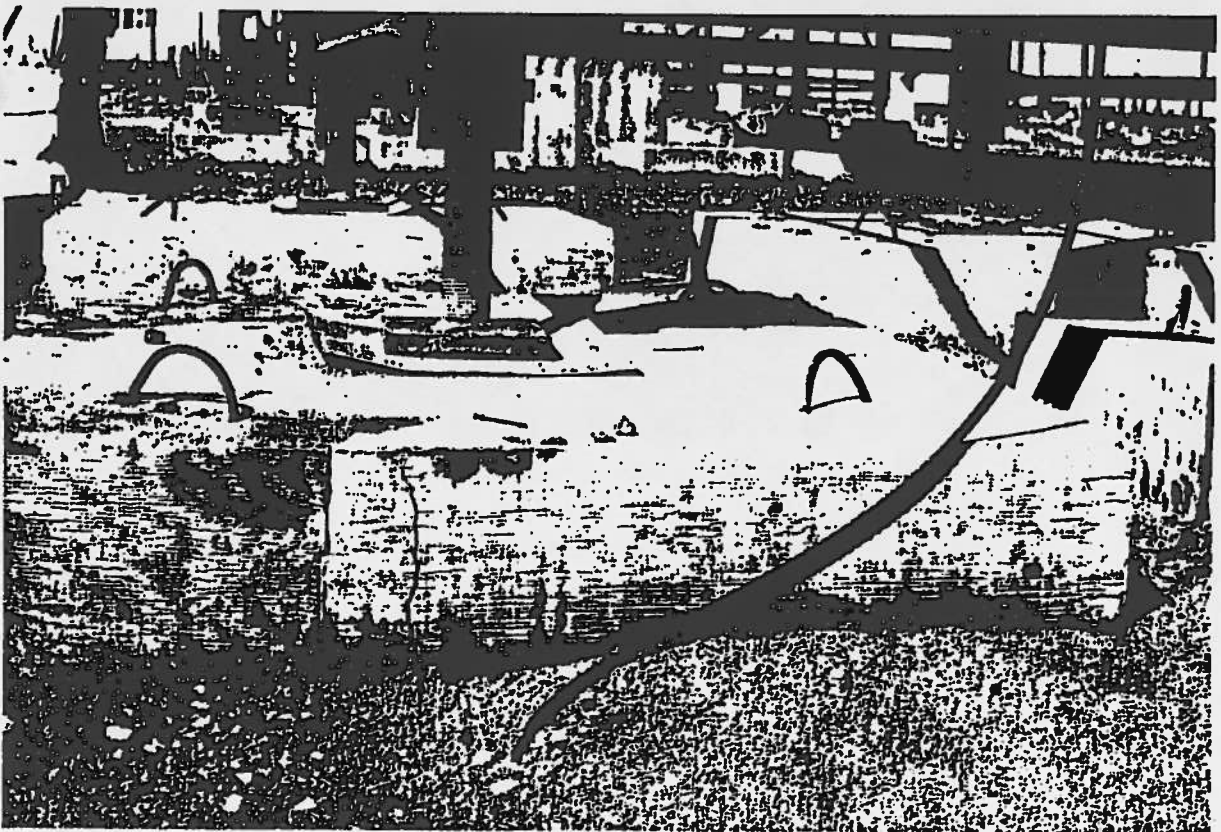


Figure 11 Concrete Slab Supporting Scaffolding Leg

Michigan. The shafts were over 30 m (100 ft.) long and were designed with reinforcement to take large vertical and overturning moments. Shortly before installation of the bored piles, the lake was filled with 8 m (26 ft.) of sand fill to bring the floor level of the steel mill to above lake level. Shortly after completion of the piles and before any column loads were placed, settlements of up to 12 cm (5 in.) were observed. A geotechnical analysis showed that the designer had failed to take into account the large drag down forces due to the consolidation of the soft clay caused by the weight of the sand fill. The estimated drag down amounted to 600 tons which resulted in end bearing unit values far in excess of the design and allowable for the hard glacial till. After considering the alternatives for rectifying the problem, the owner decided to live with the continuing settlement and to jack up the columns and place shim plates as needed.

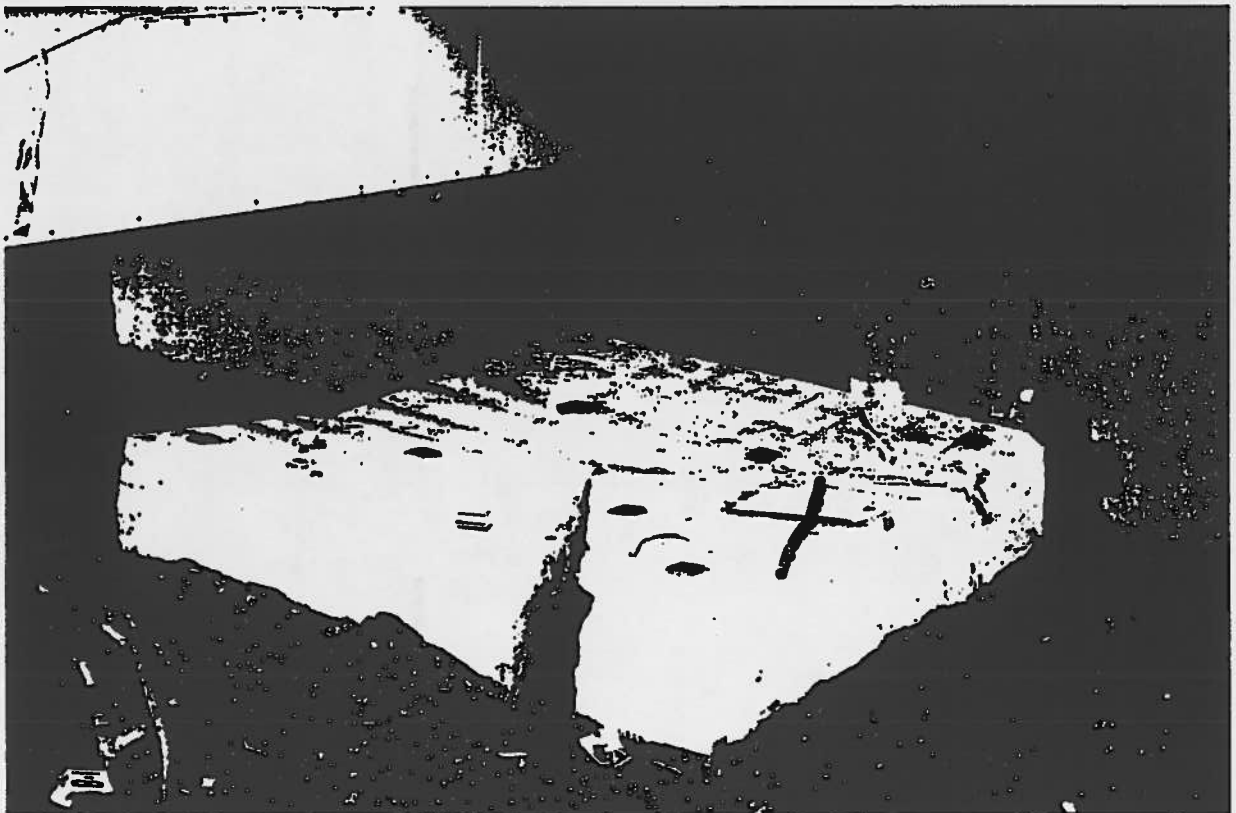


Figure 12 Slab Failure Shown After Load Test

Was this a geotechnical design failure? Yes, since the designer failed to take into account the large drag down force. It is interesting to note that there was not an end bearing capacity failure even though the unit load indicate there should have been ultimate failure. This however was due to the fact that as the shaft moves down, the skin friction tends to reduce, and not until additional settlement causes a tendency

for the friction to increase does the end bearing increase. Therefore the phenomena is that the almost constant force of the drag down punches the shaft into the base soil.

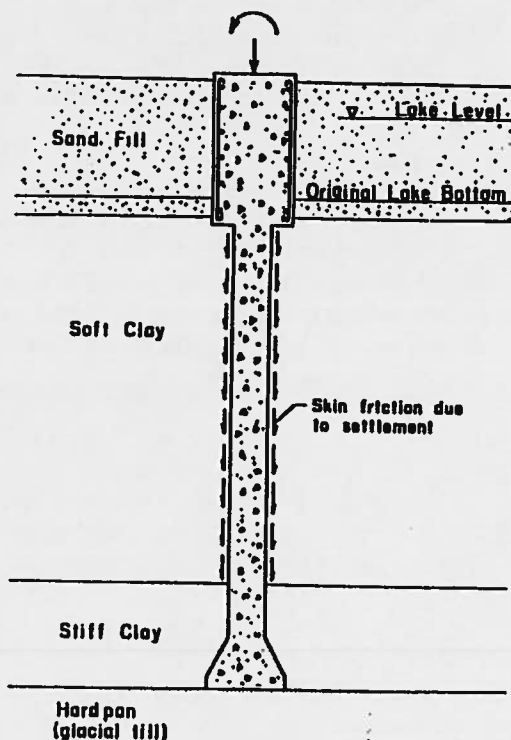


Figure 13 Bored Pile 30M Long

Case 6. Failure of a large bored pile for a 100 storey building

a) The case involves the failure of a bored pile (drilled shaft) for a 100 storey building. The building columns are supported on 3 m (10 ft.) diameter shafts cored into the underlying limestone with the shafts bearing at 42-58 m (137-191 ft.) from the street level. The soil profile and finished shaft dimensions are shown in Fig. 14. When the shafts and concrete caps were completed and the structural steel was finished to the third storey, it was noticed that one of the column had settled 10 cm (4 in.). In order to investigate the cause, the steel frame was removed and the concrete in the drilled shaft was cored. At about 8 m (26 ft.) below the cap the drill bit dropped suddenly. Mud and clay was recovered and it was not until 4.3 m (14 ft.) deeper that concrete was reached (Fig. 13). What caused this cavity or void in the shaft?

To understand the cause it is necessary to understand the method of construction. Referring to Fig. 14, the upper sand was drilled using drilling mud to stabilize the sand. Then an oversized casing 3.6 m (12 ft.) was lowered into the mudded hole

and pushed a short distance into the underlying soft clay. The mud was removed from the hole and drilling proceeded in the dry without a casing to about midway into the stiff clay. A 3 m (10 ft.) casing was lowered into the hole and water added. Drilling then continued until the casing reached the limestone. The water was maintained with a sufficient head in the casing to equal or exceed the hydrostatic level in the sand and gravel. The casing (with tungsten carbide teeth on the lower end) was then turned into the limestone a sufficient distance to form a seal. The water was then pumped out and the limestone was excavated by hand powered tools until sound rock was reached. The casing was then filled with concrete to about 12 m (40 ft.) above the rock. A man was lowered into the shaft to cut the casing at that level with a torch. As concrete was added, the upper portion of the casing was pulled gradually maintaining a head of concrete at least 1.5 m (5 ft.) above the bottom of the cut off casing. The sole purpose of retrieving the upper part of the casing was to save on cost.

When the defective shaft was poured, there was a delay of several hours in concrete delivery by the ready-mix trucks. When the casing was finally filled and pulling resumed, the force required to pull was so high that even the maximum 500 ton pull using a specially constructed A-frame was unable to completely remove the casing. The casing was then cut off and the remainder left in the hole as shown in Fig. 15. The time delay in concrete delivery allowed the concrete to set. Subsequent pulling literally pulled the concrete apart sucking in water and mud in the void created.

To repair the defect, the concrete was removed using jack hammers down to the void. The void space was cleaned and new concrete poured.

Because of the defect, it was feared that other bored piles might be defective. Therefore all of the 55 shafts were cored. Though no large voids were found, weak concrete and seams were found in almost all of them. However, all the concrete below the level at which the casing was left in place was sound. It was subsequently discovered that in spite of the casings being cut and water allowed to drain from behind, that the clay above the cut had squeezed in to completely close the annular space between the hole and the outside of the casing. This trapped water above the cutoff, so that when the casing was pulled, the head of water behind the casing exceeded the pressure of the wet concrete and intruded into the mass, thus contaminating the concrete. All the shafts were cored again and grouted. The cost for repairing the defects and the cost of the delay in the work cost about 30 times the money saved by removing the top sections of the casings.

This was clearly a construction failure. However, like all the others, it was also a "people failure". Properly trained and qualified supervisors and inspectors would have suspected

the concrete had hardened when it was impossible to pull the casing. Use of a retarding agent in the concrete would have most certainly have prevented the large void from occurring. As for the defective concrete in the other shafts, an inspector recorded in his daily logs that he saw and measured the depth of water in the annular space between the soil and the casing when he looked down from above. However, this fact did not come to the attention of those who would have known its significance. This also was a "people failure"

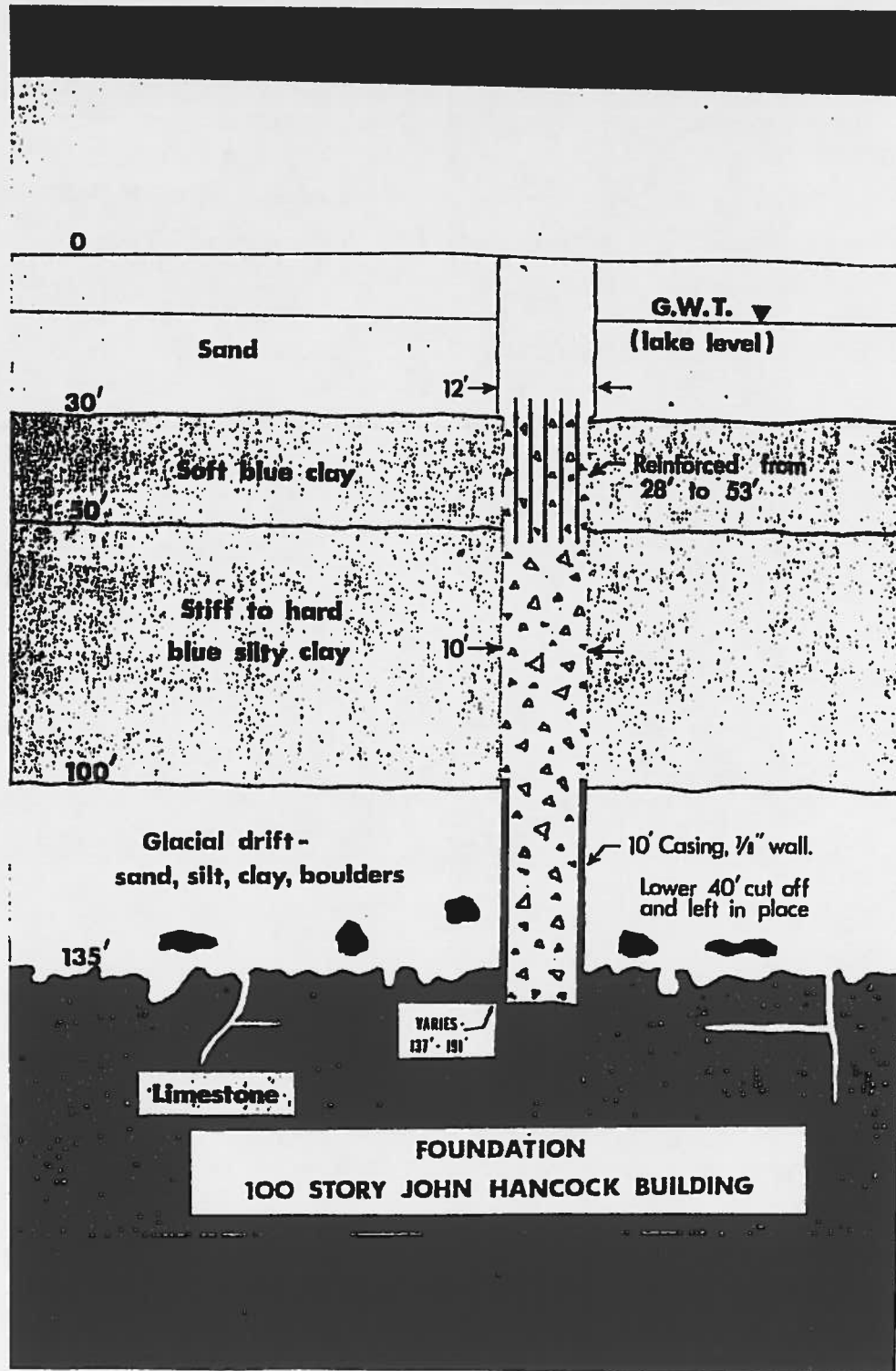


Figure 14 Soil Profile and Completed Bored Pile As Designed

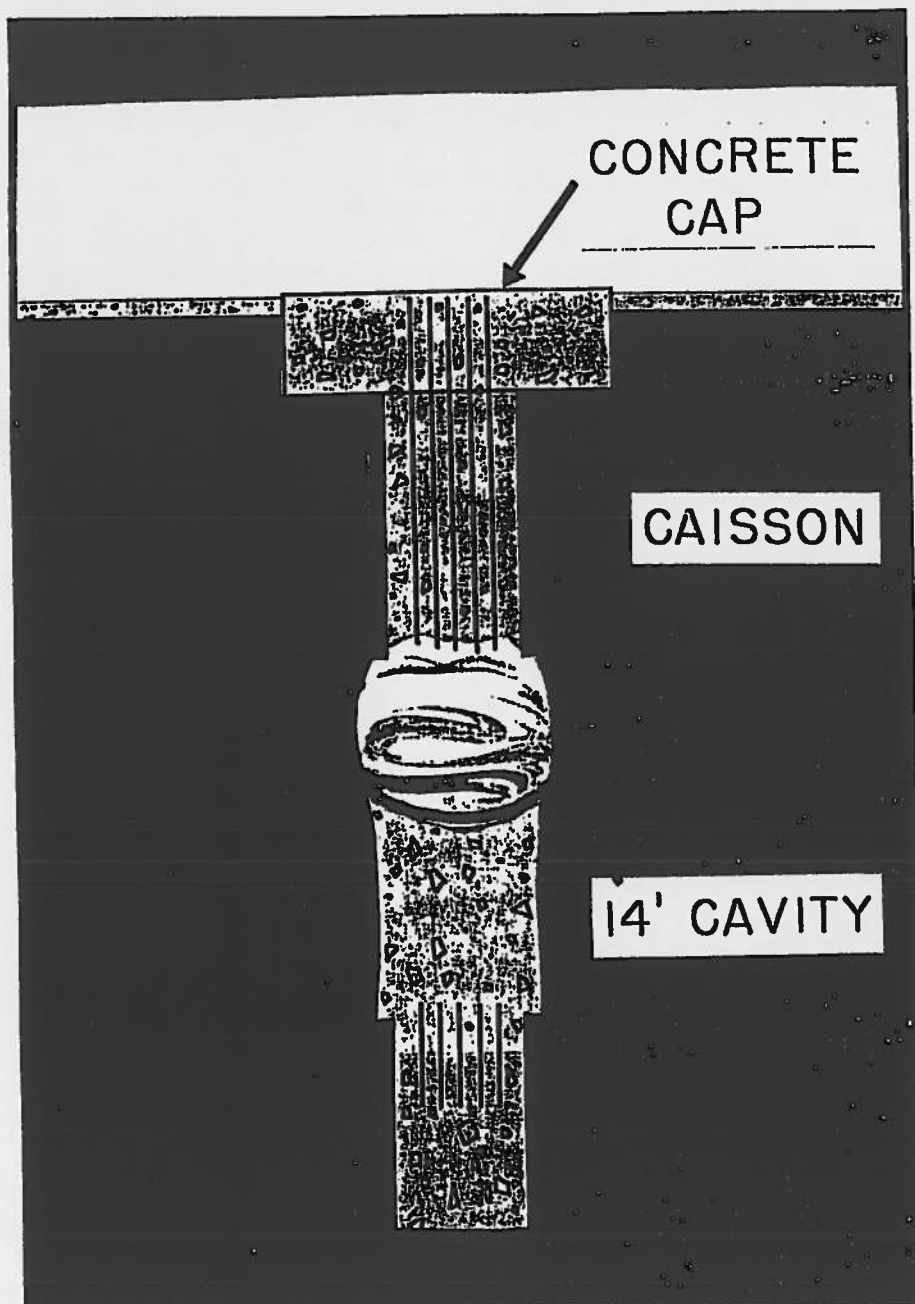


Figure 15 4.3M Void in Bored Pile

Conclusions

The failures discussed in this paper most likely would not have occurred if the measures indicated would have been taken. However, it is easy in hind sight to make this statement. Would we have made the same mistakes? Perhaps if those involved would have thought more about the possible "people problems", and taken steps to avoid them by opening the lines of communication and set up procedures for redundancy and checks at the various stages, perhaps some of the failures would not have occurred. The effort by the writer to bring these cases to your attention will be well worth while if the lesson from only one of the failures will enable only one of you readers to avoid a geotechnical failure.

STABILIZATION OF I-95/U.S. ROUTE 1 RAMP EMBANKMENT

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David P. Shiells⁽³⁾

ABSTRACT: This paper presents the investigation and evaluation of alternatives considered to stabilize a 40-foot high roadway embankment and bridge abutment in Northern Virginia. Immediately after construction, the embankment failed and was reconstructed with the addition of a rock buttress at the toe. Continued movements and minor settlement of the roadway were observed over the next year. On May 23, 1995, during roadway widening at the toe, tension cracks in the embankment widened to an extent that the ramp was closed to traffic. Inclinator data indicated that up to 2 feet of down-slope movement had occurred at a depth of about 32 feet. In addition to the ramp movements, the north abutment of an adjacent bridge had been moving in a down-slope direction, causing expansion joints to close and putting compressive loads in the bridge girders.

Long-term stabilization alternatives, including counter-balance berms, pile walls and lightweight fills, were evaluated. The criteria included not only effectiveness and cost, but also constructability and traffic maintenance. A mechanically stabilized earth (MSE) wall and counter-balance berm were selected to stabilize the embankment and allow replacement of the distressed bridge. Results of slope inclinometer monitoring during and after construction are presented.

INTRODUCTION

HISTORICAL REVIEW

The site is located in Northern Virginia, just north of the Occoquan River and south of Washington, D.C. The area north and east of Occoquan River, where Interstate 95 (I-95) and U.S. Route 1 are connected with a series of ramps, was undeveloped and mostly wooded prior to construction of the ramps. The design of the Shirley G. Memorial Highway (State Route 350 and portions of future I-95) north of the Occoquan River was completed around 1945. Between 1960 and 1964, the ramps connecting Route 1 and I-

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PREVIOUS REPAIRS

Tension cracks were observed near the top of the Ramp B embankment shortly after the embankment was completed in early October 1993. The existing fill was removed (possibly between October 8 and 10) to some depth from the toe to a limit just beyond the north side of Ramp B. An exploration was conducted in October to evaluate the subsurface conditions in order to repair the embankment.

A rock buttress was constructed along the toe of the slope adjacent to the existing Ramp B. The buttress was constructed to a depth of 4 feet below the Ramp A pavement surface and backfilled with 15 feet of surge stone followed by 12 inches of 21B sloped to drain away from the embankment. Construction of the key trench and rock buttress began in mid November 1993 and was completed in May of 1994. Additional tension cracks were observed on both the north and south sides of Ramp B in June 1994. The ramp was opened to traffic on July 7, 1994. Subsequent movements were monitored.

FAILURE CONDITIONS

Accelerated settlement and movements were recorded in the spring of 1995. Tension cracks also appeared in the Ramp B pavement near the east abutment to Bridge 664. In March 1995, cores of the approach slab indicated that 2- to 3-inch thick voids existed directly beneath the slab. Settlement point readings recorded in March and April indicated settlement was occurring in the pavement of Ramp B near the east abutment to Bridge 664.

On May 23, 1995 tension cracks in the Ramp B pavement widened to an extent that the ramp was closed to traffic. Inclometers in Ramp B, near Bridge 664, also indicated that up to 2 feet of lateral down-slope movement had occurred at a depth of about 32 feet. The movement occurred at the time that construction to widen Ramp A near the toe of the embankment was proceeding. The construction included excavating to a depth of about 4 to 5 feet below the existing pavement grade of Ramp A along the length of the embankment toe.

Immediately after the failure, concrete median barriers were stacked four-high along the embankment toe to stabilize movements. Graded stone fill was also placed at the toe from the east end of the median barriers to the east.

In addition to the slope movements, the north abutment of Bridge 663 had been moving in a southeasterly direction over the past recent years. The movement has caused the expansion joints of the bridge to close and put compressive loads in the bridge girders. Construction was underway to demolish the existing bridge deck and construct a new additional north abutment supported on piles to eliminate further movements at the time of the slope failure.

SITE GEOLOGY AND SUBSURFACE CONDITIONS

GEOLOGY

The project site is located in an area near the Fall Line (i.e., contact between the Coastal Plain Physiographic Province and the Piedmont Physiographic Province). The subsurface conditions of this region consist of soils of both marine deposits of the Coastal Plain groups and residual soils of the Piedmont. Beneath the soils, the profile weathers into decomposed rock and rock of the Piedmont.

On the basis of reviewing published geologic maps, the site is located in an area of recent sedimentary deposits underlain by Ordovician Age metamorphosed volcanic rocks. The recent sedimentary deposits consist of the Potomac Formation (Lower Cretaceous) which includes three main units. The upper unit consists of light gray to pink silty to clayey feldspathic quartz sand.

The high plasticity silts and clays of the Cretaceous Age Potomac Group are locally referred to as "Marine Clays". Many areas of slope failure are evident on naturally occurring slopes of marine clay. The clays are typically softened and have slickensides or fissures from the surface to several feet in depth. Deeper unweathered clays are normally hard but may be weakened by intersecting fissures and occasional old shear zones.

Beneath the marine clays are residual silts and clays and weathered schist/phyllite. The rock at this site is mapped as a felsic volcanoclastic member of the Quantico Formation. The felsic volcanoclastic rock is medium gray to pale-green volcanic phyllite with some rare greenstone interbedded with dark gray to black slate.

SUBSURFACE EXPLORATION

The subsurface conditions were explored with 28 soil test borings and four test pits. The soil test borings were drilled to depths of between 18 and 150 feet. Standard penetration tests (SPTs) were conducted in the borings at regular intervals. Undisturbed Shelby tube samples were obtained from several borings for use in laboratory testing. Rock coring was also performed in five of the borings in order to assess the nature and quality of the underlying bedrock at the site.

GROUND-WATER CONDITIONS

Water levels were measured in the borings and in 14 observation wells.

Ground water appears to be perched in many areas above the marine clays and residual clays. On the basis of observation well readings through the rubble (rip-rap) buttress, water appears to be rising up into the rip-rap. This water is most likely coming from surface water infiltration and perching above the clays. A review of moisture content tests also indicates perched conditions near the toe of the Ramp B embankment slope. In general, moisture contents of the soil decreases with depth below the ground surface.

INSTRUMENTATION/MONITORING POINTS

Eleven inclinometer casings were installed at various locations on and around the project to monitor slope movement. The casing depths varied from about 30 feet to 150 feet. Inclinometer readings were taken during design and construction, and bi-monthly thereafter. Depth versus displacement plots were generated for each inclinometer. A typical set of inclinometer readings is shown in Figure 2.

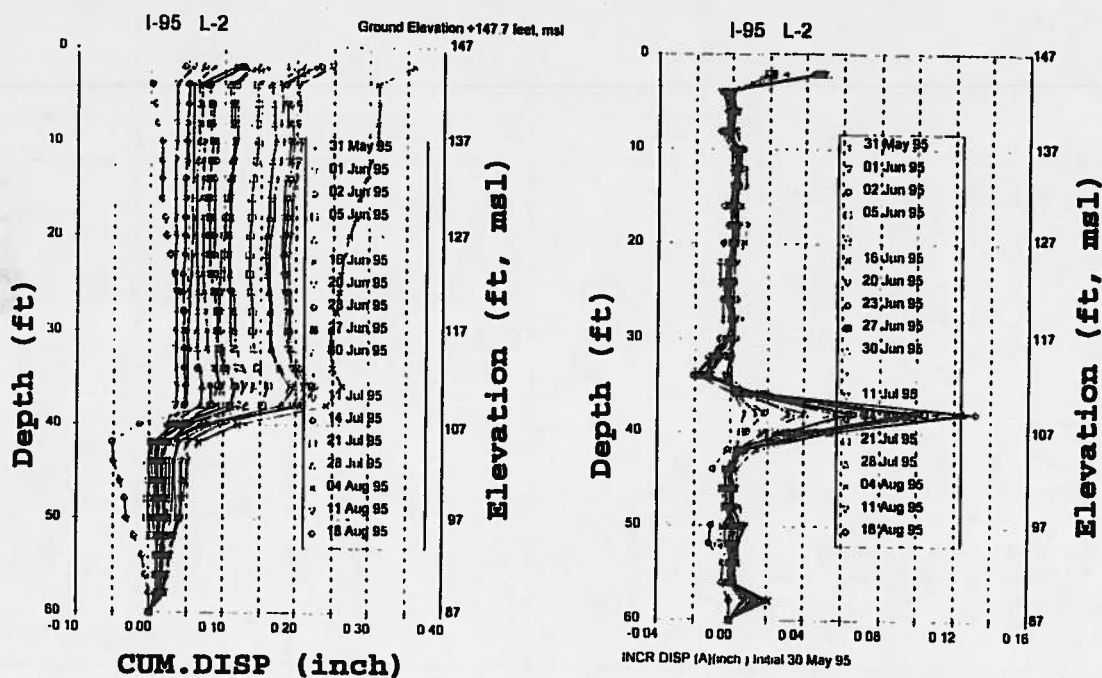


Figure 2. Typical Plots of Inclinometer Readings

LABORATORY TESTING

Laboratory tests consisted of soil classification tests including natural moisture contents, Atterberg limits, and percent finer than the No. 200 sieve. Soil strength tests were also conducted, consisting of direct and residual shear tests and triaxial tests (consolidated undrained and unconsolidated undrained). One-dimensional consolidation tests, moisture density tests, and swell tests were also performed on selected samples.

SUBSURFACE CONDITIONS

The subsurface conditions encountered in the soil test borings generally encountered Fill, soft to firm Marine clays, firm to very stiff residual clays overlying decomposed rock and bedrock. Design soil parameters derived from field and laboratory tests are listed in Table 1.0.

The design soil parameters were verified through back-calculation of a factor of safety of less than 1.0 at the failure condition, and about 1.0 with the temporary stabilization berm and median barriers in place.

TABLE 1
DESIGN SOIL PARAMETERS

Description	USCS Class.	Atterberg Limits		Estimated Soil Properties	
		LL	PI	ϕ	γ
Rockfill Berm	Boulder	---	---	40°	110
Engineered Embankment Fill - sampled as firm to dense silty sand	SM	13 to 21	NP	34.5°	136
Soft to firm low and high plasticity clay	CH CL	53 to 55	21 to 32	12.5° - 13.5°	115
Stiff to very stiff low and high plasticity clay	CH CL	38 to 55	22 to 32	16°	120
Very stiff to very hard low and high plasticity clays and silts	CH CL MH	41 to 54	23 to 36	24°	120

Note: For analysis and design, cohesion (c) was considered as zero for effective strength conditions.

GEOTECHNICAL EVALUATIONS

SLOPE STABILITY

The stability of the Ramp B embankment has been evaluated in two ways: physical monitoring of slope movement using both survey methods and inclinometer readings; and computer slope stability analyses of the existing embankment employing a generalized subsurface stratigraphy with design soil parameters.

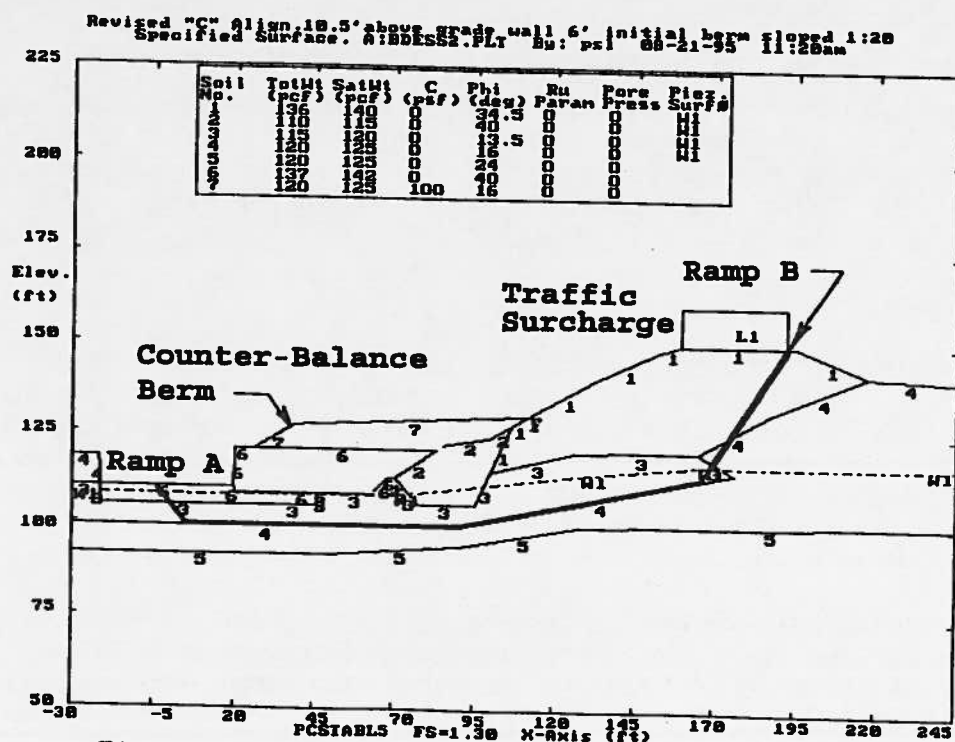
SLOPE INCLINOMETER EVALUATIONS

After the failure and temporary slope stabilization, continued slope movements were recorded to progress at a rate of about 0.02 inches per week. Movement continued to occur under creep conditions, indicating that the slope had a marginal factor of safety. Tension cracks at mid-slope were observed to have widened slightly, and settlement of the Ramp B pavement continued prior to construction of the stabilization system. Figure 2 shows a typical rate of movement at the shear zone for Inclinator L-2.

SLOPE STABILITY ANALYSES

Slope stability analyses were carried out on preliminary cross-sections developed during the early stages of the subsurface exploration. The results of these analyses were used to refine the design soil parameters, stratification, and to check the efficacy of the temporary concrete median barrier wall counterweight. The stability analyses were conducted using the PCSTABL Version 6.0 program developed at Purdue University.

A series of stability analyses were conducted for the various remediation options and, where applicable, different configurations of a particular option. Both circular and wedge failure analyses were conducted. In all cases, the analyses were conducted in order to find the minimum factor of safety for any given cross-section and stabilization option. A discussion of individual results is not pertinent here, since many configurations produced unacceptable factors of safety. However, a typical analysis is reproduced as Figure 3.



LONG-TERM STABILIZATION ALTERNATIVES

Long-term stabilization alternatives were evaluated using the following methodology. First, a wide range of preliminary alternatives were listed on the basis of previous experience in slope stabilization and review of stabilization methods published in the literature. The alternatives were reviewed in open forum discussions with VDOT, FHWA and other transportation design engineers familiar with the project conditions. Once a comprehensive list of potential alternatives was prepared, a list of evaluation criteria was generated. The evaluation criteria included technical analyses, constructibility, cost, use of on-site materials, traffic maintenance, safety and aesthetics. Finally, each alternative was evaluated with regard to the list of evaluation criteria.

A "short-list" of potential stabilization alternatives was then prepared on the basis of preliminary reviews with VDOT and FHWA. A summary of the short-listed, long-term stabilization alternatives are listed in Table 2 below.

TABLE 2
SUMMARY OF LONG-TERM STABILIZATION ALTERNATIVES

Alternative	Est. Cost (in millions)	F.S.
MSE/wall counter-balance berm	\$0.3 to \$0.5	1.35 to 1.8
Rockfill counter-balance berm	\$0.5 to \$0.75	1.1 to 1.24
H-pile wall @ toe	\$0.5 to \$0.8	Not Determined
Type 'A' Insert wall	\$0.5 to \$0.6	Est. to be 1.3
Mid-slope tieback wall	\$1 to \$1.5	Not Determined
Lightweight fill	\$0.9 to \$1.1	1.15 to 1.27

Note: Cost estimates include only unit costs for the specific materials in place and do not include the costs of detours, traffic maintenance, erosion control, bridge replacement, ramp reconstruction, utility relocation, underdrains and other incidental construction.

COUNTER-BALANCE BERMS

Several types of counter-balance berms were evaluated. Each type, however, relies on the same principle of providing static dead load (weight) at the toe of the slope to resist the driving moments of the embankment. The two types of counter-balance berms evaluated to be most feasible included Mechanically Stabilized Earth (MSE) supported berms and rockfill, or a rip-rap berm with key trench.

MSE/Modular Block Wall Counter-Balance Berm. MSE walls have been used extensively for highway retaining structures in the area. Their success relies on several factors including; speed

and ease of construction, relative low cost, and flexibility. An added advantage in the case of slope stabilization by placement at toe is the minimizing of excavation for footings. This could be an important consideration since the probable "trigger" of slope movement was the undercutting at the toe of the slope.

Rockfill/Rip-Rap Berm. Rockfills have been widely used for counter-balance berms in slope stabilization problems. A rockfill berm was also constructed at the toe of embankment B after the original slope failure. Rockfills offer an advantage of being cost-effective and easily constructible. The main disadvantages are that they usually require keys, sloped faces, and are generally lighter in weight. The lighter weights are attributed to the voids (porosity) in rip-rap fills, typically being between 35 to 40 percent by volume.

H-PILE CUTOFF WALL

Driven and pre-drilled H-pile or drilled shaft cutoff walls have been designed and used as slope stabilization and landslide remediation solutions for many slides in marine clays of Northern, Virginia. The walls generally consist of driving steel H-Piles at 4- to 5-foot centers in a line perpendicular to the direction of the slide. In some cases, the H-Piles are placed in concrete-filled, pre-drilled holes to avoid vibrations in sensitive cases. The H-Piles extend to depths sufficient to develop adequate passive and moment resistance to support active pressures of the soil above predetermined or assumed slide planes.

The primary advantages of H-Pile cutoff walls are their relative low cost, short construction times, and minimization of excavation at the toe of slides. One of the most significant benefits is the fact that the H-Piles intercept the shear zone and provide comfortable factors of safety. The difficulty in using H-Pile cutoff walls for stabilization of the Ramp B embankment is in optimizing the location of the wall. The previously placed rubble (rip-rap) and concrete pavements would make installation of the piles difficult near the toe.

LIGHTWEIGHT FILL

Use of lightweight fill to stabilize the slope approaches the problem from a slightly different angle than the alternatives presented previously. The principle of lightweight fill is to reduce the driving moments by unloading the crest of the slope instead of increasing the resisting moments by adding weight to the toe of the slope. The benefit reveals itself where constrained space at the toe is a major issue. An added benefit is that most slope failures beneath roadway embankments (such as Ramp B) require some rehabilitation of the supported roadway. The placement of lightweight fill may then be completed as part of the rehabilitation of the supported roadway.

Lightweight fills for slope stabilization have been extensively used in Europe for about 25 years. Most of the research and applications have been in Norway and other Scandinavian countries. In the past 10 years they have also seen more and more use in Japan. Only recently have lightweight fills beneath roadways and for slope stabilization been recognized as a viable option and employed in the United States.

Most lightweight fill consists of three principal types; lightweight aggregates (e.g., Solite®), lightweight concrete (e.g., Elastizell®) and Expanded Polystyrene (EPS). The unit weights of the materials typically range from as low as 0.75 to 1.5 pounds per cubic foot (pcf) for EPS to 30 to 42 pcf for Elastizell. Unit weights for lightweight aggregates depend entirely on the type of aggregate and compaction but are generally in the range of 45 to 60 pcf (loose weights).

Stability analyses of various lightweight fill options indicate that the majority of the Ramp B embankment would require replacement with lightweight fill. Alternatively, lightweight fill would be required to be used in conjunction with a smaller counter-balance berm at the toe of the slope.

MID-SLOPE TIEBACK WALL

The concept of a mid-slope tieback wall is a variation of the H-Pile cutoff wall. The mid-slope tieback wall simply provides the benefit of allowing the use of lighter piles and supporting the driving moments with tiebacks. The mid-slope tieback wall also allows the installation of the H-Piles up-slope of the rubble buttress. Finally, this concept provided the greatest room at the toe of the slope by not encroaching on the Ramp A alignment.

The difficulties associated with this concept for this site included; construction of temporary ramps to gain access to the mid-slope area, excessively long tiebacks to obtain necessary capacities and adhesion, double corrosion protection for tiebacks and significant monitoring and testing.

TYPE 'A' INSERT^(SM) WALL

Type 'A' Insert^(SM) Walls are a method of stabilization using in-situ earth reinforcements developed by Nicholson Construction Company. The method has been used to control movements of marginally stable slopes using a pattern of vertical and near vertical reinforcing elements (Pin Piles) installed across a slide plane. The Pin Piles^(SM) are tied to a central concrete cap located near the ground surface. The benefits, compared to more traditional methods of stabilization (i.e., counter-balance berms and anchored pile walls), are generally cost and are limited by site constraints. The Type 'A' Insert Wall system is generally least disruptive to sites, may be constructed on steep slopes, and may be installed quickly using lightweight drilling

equipment. The system also requires little or no excavation making it simple to install.

DESIGN AND CONSTRUCTION

All factors considered, including rehabilitation of Bridge 663, VDOT selected the MSE wall and counter-balance berm as the best solution for the site conditions and constraints. The MSE wall and counter-balance berm allowed realignment of Ramp A further from the toe of the embankment by the elimination of unnecessary spans of Bridge 663. A high comfort factor based on experience with MSE walls coupled with the technical, cost, safety and constructibility issues raises this alternative to the top of the list.

Various geometries were evaluated assuming that VDOT 21A crushed aggregate would be placed within and directly behind the MSE wall. The on-site sandy silts and clays from the northwest side of Ramp B and from MSE wall cut areas on the south side of Ramp A would be placed upslope of the 21A aggregate. Use of the on-site soils reduced costs and helped balance site cuts and fills. In addition, the crushed aggregate in the temporary berm was used within the wall to further reduce costs.

The MSE wall was extended from the existing toe of the Ramp B embankment to continue under and to the new west side of Bridge 663 (New Bridge 677). The MSE design option allowed demolition of the north abutment and north pier and constructing a new abutment further south supported by a MSE wall. The center span structural steel was salvaged and used in the new bridge.

A contract was bid by VDOT on September 14 with incentives to complete the construction of the MSE walls and counter-balance berm by December 15. Construction was initiated the week of September 25 and the walls and berms were significantly completed by November 16. Grouting of the embankment and construction of most of the drainage systems were completed about December 16. Construction of the new Bridge 677 and non-critical portions of the MSE walls continued into 1996.

Further monitoring of the slope inclinometers indicate that ongoing movements and settlements in the embankment have been stabilized and the system is functioning as designed. (See Figure 4).

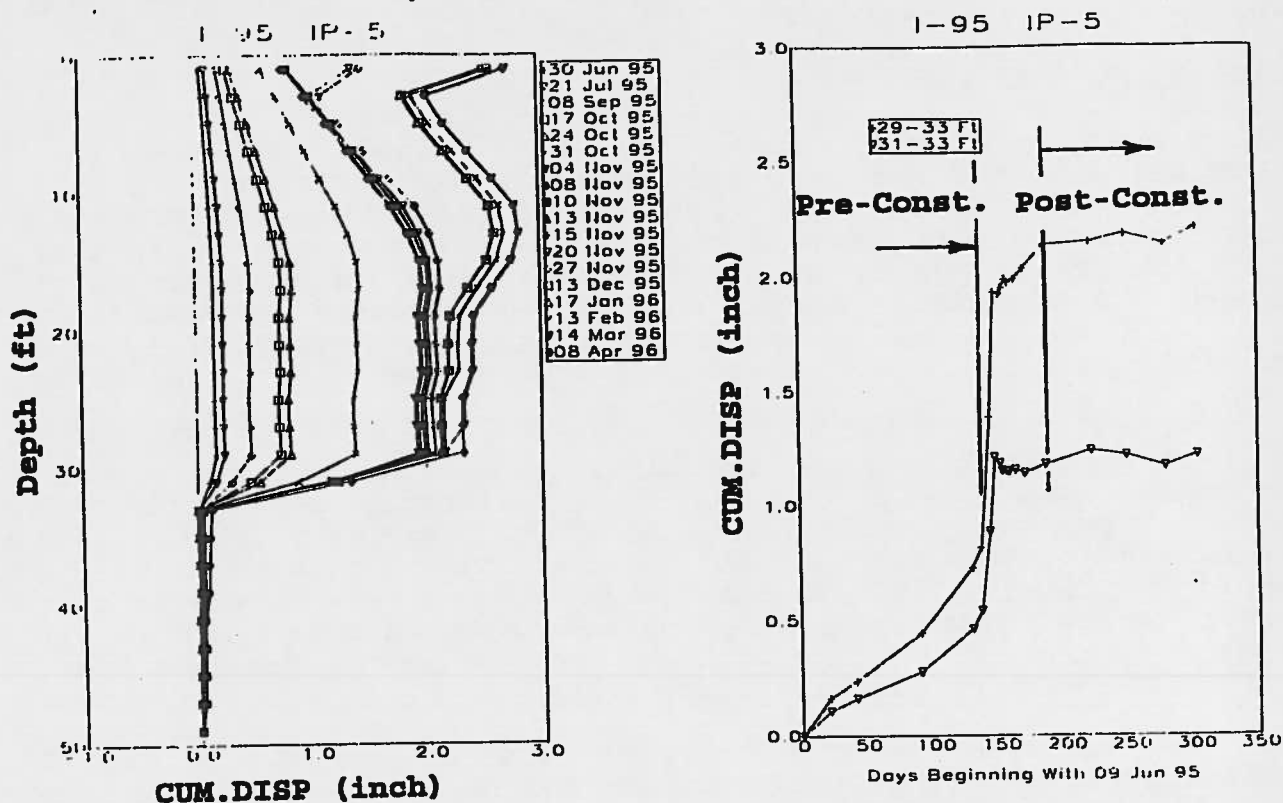


Figure 4. Inclinerometer Displacement vs. Time

SUMMARY AND CONCLUSIONS

1. Complex geology and the presence of low strength Marine clays in the location of a 40-foot high embankment led to the failure of the ramp.
2. An extensive subsurface exploration was conducted to develop a reasonable model of the complex geologic and hydrologic conditions. Slope stability analyses were conducted to determine the location of the failure plane and magnitude of driving forces. Both stability analyses and slope inclinometer results indicated the probable failure surface extended through soft to firm marine and residual clays with low residual strengths. The location of inclinometer movements validated the subsurface model and results of slope stability analyses.
3. Various long-term stabilization alternatives for stabilizing the failed Ramp B embankment were reviewed and evaluated in open forum discussions with VDOT, FHWA and other transportation design engineers familiar with the project conditions. The Mechanically Stabilized Earth (MSE) counter-balance berm alternative was selected by VDOT as the preferred choice.

4. Selection was based on several factors, including: Obtaining a suitable factor of safety for stability; constructibility; maintenance of traffic; cost; providing a solution to movements of Bridge B-663; as well as allowing realignment of Ramp A for safety issues and future widening considerations.

5. The marginal stability of the Ramp B embankment in its current condition dictated that the sequence of construction be followed closely, with particular attention paid to ramp stability during periods of undercutting.

6. Perched ground-water conditions were identified as contributing to the failure of the embankment. A network of blanket drains, underdrains and surface drainage features were designed to direct water away from the slope.

7. Construction of the MSE wall counter-balance berm was completed within 6 weeks of the award of a contract and prior to the onset of winter conditions. Early slope inclinometer readings indicate that ongoing movements have been arrested and the system is functioning as designed.

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The authors wish to express appreciation to the entire design team including; the Virginia Department of Transportation (VDOT), Federal Highways Region 3 (FHWA), Howard, Needles, Tammen and Bergendoff (HNTB) and Legion Design. Any explicit or implicit statements made herein by the authors should not be inferred to be official procedures or standard design by the VDOT, FHWA or HNTB.

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APPENDIX
PAST PROCEEDINGS OF
OHIO RIVER VALLEY SOILS SEMINAR

<u>ORVSS</u>	<u>TITLE</u>	<u>DATE</u>	<u>LOCATION</u>
I	Building Foundation Design & Construction	10/16/70	Lexington, Kentucky
II	Earthwork Engineering, Start to Finish	10/15/71	Louisville, Kentucky
III	Lateral Earth Pressures	10/27/72	Ft. Mitchell, Kentucky
IV	Geotechnics in Transportation Engineering	10/05/73	Lexington, Kentucky
V	Rock Engineering	10/18/74	Clarksville, Indiana
VI	Slope Stability and Landslides	10/17/75	Ft. Mitchell, Kentucky
VII	Shales & Mine Wastes: Geotechnical Properties, Design & Construction	10/08/76	Lexington, Kentucky
VIII	Earth Dams & Embankments: Design & Construction	10/14/77	Louisville, Kentucky
IX	Deep Foundations	10/27/78	Ft. Mitchell, Kentucky
X	Geotechnics of Mining	10/05/79	Lexington, Kentucky
XI	Earth Pressures & Retaining Structures	10/10/80	Clarksville, Indiana
XII	Groundwater: Monitoring, Evaluation, & Control	10/09/81	Ft. Mitchell, Kentucky
XIII	Recent Advances in Geotechnical Engineering	10/08/82	Lexington, Kentucky
XIV	Foundation Instrumentation and Geophysical	10/14/83	Clarksville, Indiana
XV	Practical Application of Drainage in Geotechnical Engineering	11/02/84	Ft. Mitchell, Kentucky
XVI	Applied Soil Dynamics	10/11/85	Lexington, Kentucky
XVII	Natural Slope Stability and Instrumentation	10/17/86	Clarksville, Indiana
XVIII	Liability Issues in Geotechnical Engineering & Construction	11/06/87	Ft. Mitchell, Kentucky

<u>ORVSS</u>	<u>TITLE</u>	<u>DATE</u>	<u>LOCATION</u>
XIX	Chemical & Mechanical Stabilization of Soil Subgrades	10/21/88	Lexington, Kentucky
XX	Construction In & On Rock	10/27/89	Louisville, Kentucky
XXI	Environmental Aspects of Geotechnical Engineering	10/26/90	Cincinnati, Ohio
XXII	Design & Construction with Geosynthetics	10/18/91	Lexington, Kentucky
XXIII	In Situ Soil Modification	10/16/92	Louisville, Kentucky
XXIV	Geotechnical Aspects of Infrastructure Reconstruction	10/15/93	Cincinnati, Ohio
XXV	Recent Advances in Deep Foundations	10/21/94	Lexington, Kentucky
XXVI	Site Investigations: Geotechnical and Environmental	10/20/95	Clarksville, Indiana