

The Eighteenth Spencer Buchanan Lecture
Friday December 3, 2010
College Station Hilton
College Station, Texas, USA
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Uncertain Geotechnical
Truth and Cost Effective
High-Rise Foundation
Design

The 2009 Karl Terzaghi Lecture
By Mr. Clyde Baker



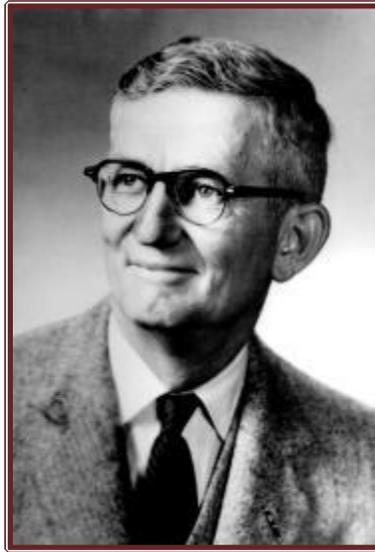
Forensic Diagnosis for
Site-Specific Ground
Conditions in Deep
Excavations of Subway
Constructions

*The 2010 Spencer J. Buchanan
Lecture*
By Professor Kenji Ishihara

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SPENCER J. BUCHANAN



Spencer J. Buchanan, Sr. was born in 1904 in Yoakum, Texas. He graduated from Texas A&M University with a degree in Civil Engineering in 1926, and earned graduate and professional degrees from the Massachusetts Institute of Technology and Texas A&M University.

He held the rank of Brigadier General in the U.S. Army Reserve, (Ret.), and organized the 420th Engineer Brigade in Bryan-College Station, which was the only such unit in the Southwest when it was created. During World War II, he served the U.S. Army Corps of Engineers as an airfield engineer in both the U.S. and throughout the islands of the Pacific Combat Theater. Later, he served as a pavement consultant to the U.S. Air Force and during the Korean War he served in this capacity at numerous forward airfields in the combat zone. He held numerous military decorations including the Silver Star. He was founder and Chief of the Soil Mechanics Division of the U.S. Army Waterways Experiment Station in 1932, and also served as Chief of the Soil Mechanics Branch of the Mississippi River Commission, both being Vicksburg, Mississippi.

Professor Buchanan also founded the Soil Mechanics Division of the Department of Civil Engineering at Texas A&M University in 1946. He held the title of Distinguished Professor of Soil Mechanics and Foundation Engineering in that department. He retired from that position in 1969 and was named professor Emeritus. In 1982, he received the College of Engineering Alumni Honor Award from Texas A&M University.

He was the founder and president of Spencer J. Buchanan & Associates, Inc., Consulting Engineers, and Soil Mechanics Incorporated in Bryan, Texas. These firms were involved in numerous major international projects, including twenty-five RAF-USAF airfields in England.

They also conducted Air Force funded evaluation of all U.S. Air Training Command airfields in this country. His firm also did foundation investigations for downtown expressway systems in Milwaukee, Wisconsin, St. Paul, Minnesota; Lake Charles, Louisiana; Dayton, Ohio, and on Interstate Highways across Louisiana. Mr. Buchanan did consulting work for the Exxon Corporation, Dow Chemical Company, Conoco, Monsanto, and others.

Professor Buchanan was active in the Bryan Rotary Club, Sigma Alpha Epsilon Fraternity, Tau Beta Pi, Phi Kappa Phi, Chi Epsilon, served as faculty advisor to the Student Chapter of the American Society of Civil Engineers, and was a Fellow of the Society of American Military Engineers. In 1979 he received the award for Outstanding Service from the American Society of Civil Engineers.

Professor Buchanan was a participant in every International Conference on Soil Mechanics and Foundation Engineering since 1936. He served as a general chairman of the International Research and Engineering Conferences on Expansive Clay Soils at Texas A&M University, which were held in 1965 and 1969.

Spencer J. Buchanan, Sr., was considered a world leader in geotechnical engineering, a Distinguished Texas A&M Professor, and one of the founders of the Bryan Boy's Club. He died on February 4, 1982, at the age of 78, in a Houston hospital after an illness, which lasted several months.

The Spencer J. Buchanan '26 Chair in Civil Engineering

The College of Engineering and the Department of Civil Engineering gratefully recognize the generosity of the following individuals, corporations, foundations, and organizations for their part in helping to establish the Spencer J. Buchanan '26 Professorship in Civil Engineering. Created in 1992 to honor a world leader in soil mechanics and foundation engineering, as well as a distinguished Texas A&M University professor, the Buchanan Professorship supports a wide range of enriched educational activities in civil and geotechnical engineering. In 2002, this professorship became the Spencer J. Buchanan '26 Chair in Civil Engineering.

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Spencer J. Buchanan Lecture Series

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1994	G. Geoffrey Meyerhof	“Evolution of Safety Factors and Geotechnical Limit State Design”
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1999	J. Michael Duncan	“Factors of Safety and Reliability in Geotechnical Engineering”
2000	Harry G. Poulos	“Foundation Settlement Analysis – Practice Versus Research”
2001	Robert D. Holtz	“Geosynthetics for Soil Reinforcement”
2002	Arnold Aronowitz	“World Trade Center: Construction, Destruction, and Reconstruction”
2003	Eduardo Alonso	“Exploring the Limits of Unsaturated Soil Mechanics: the Behavior of Coarse Granular Soils and Rockfill”
2004	Raymond J. Krizek	“Slurries in Geotechnical Engineering”
2005	Tom D. O’Rourke	“Soil-Structure Interaction Under Extreme Loading Conditions”
2006	Cylde N. Baker	“In Situ Testing, Soil-Structure Interaction, and Cost Effective Foundation Design”
2007	Ricardo Dobry	“Pile response to Liquefaction and Lateral Spreading: Field Observations and Current Research”
2008	Kenneth Stokoe	“The Increasing Role of Seismic Measurements in Geotechnical Engineering”
2009	Jose M. Roesset	“Some Applications of Soil Dynamics”
2010	Kenji Ishihara	“Forensic Diagnosis for Site-Specific Ground Conditions in Deep Excavations of Subway Constructions”

The text of the lectures and a videotape of the presentations are available by contacting:

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AGENDA

The Eighteenth Spencer J. Buchanan Lecture
Friday, December 3rd, 2010
College Station Hilton

- 2:00 p.m. Welcome by Jean-Louis Briaud
- 2:10 p.m. Introduction by Jean-Louis Briaud
- 2:15 p.m. Introduction of Clyde Baker by Jean-Louis Briaud
- 2:20 p.m. “Uncertain Geotechnical Truth and Cost Effective High-Rise Foundation Design”
The 2009 Terzaghi Lecture by Clyde Baker
- 3:20 p.m. Introduction of Kenji Ishihara by Jean-Louis Briaud
- 3:25 p.m. “Forensic Diagnosis for Site-Specific Ground Conditions in Deep Excavations of Subway Constructions”
The 2010 Buchanan Lecture by Kenji Ishihara
- 4:25 p.m. Discussion
- 4:40 p.m. Closure with Philip Buchanan
- 5:00 p.m. Photos followed by a reception at the home of Jean-Louis and Janet Briaud.

Clyde N. BAKER Jr.



Mr. Baker received his BS and MS degrees in Civil Engineering from the Massachusetts Institute of Technology and joined the staff of STS Consultants, Ltd. (formerly Soil Testing Services) in the fall of 1954. Over the past 50 years, he has served as geotechnical engineer for many high rise built in Chicago. He has also served as geotechnical engineer or consultant on seven of the sixteen tallest buildings in the world including the three tallest in Chicago (Sears, Hancock, and Amoco) and the current two buildings in the world, the Petronas Towers in Kuala Lumpur, Malaysia and 101 Financial Center in Taipei, Taiwan.

As a result of his experience, Mr. Baker has developed an international reputation in the design and construction of deep foundations. He has been a leader in using in-situ testing techniques correlated with past building performance to develop more efficient foundation designs. In the Chicago soil profile this has facilitated economical use of belled caissons on hard pan for major structures in the 60 to 70 story height range (such as Water Tower Place, 900 North Michigan, and AT&T) which normally would have required extending caissons to rock at significant cost premium.

Mr. Baker has shared his knowledge and experience with his peers through numerous Conference and University lectures, technical articles, papers and

publications. He is the recipient of the Deep Foundation's Institute Distinguished Service Award, the ADSC Outstanding Service Award, ASCE's Thomas A. Middlebrooks and Martin S. Kapp awards and of three Meritorious Publication Awards from SEAIO including the "History of Chicago Building Foundations 1948 to 1998". He is the author of "The Drilled Shaft Inspectors' Manual" sponsored jointly by the Deep Foundation Institute and the International Association of Foundation Drilling (ADSC).

Mr. Baker has been very active professionally on both the local and national scene. He is an Honorary Member of ASCE. He is a past President of SEAIO and of the Chicago Chapter of ISPE. Nationally he has served as Chair of the Geotechnical Engineering Division of ASCE, Editor of the Geotechnical Engineering Journal, and Chair of ACI Committee 336 on Footings, Mats and Drilled Piers. He is a member of the National Academy of Engineering and was the recipient of the ASCE Ralph B. Peck Award for the year 2000, the Terzaghi Lecture in 2009, and was selected as a Hero of Geotechnical Engineering by the Geo-Institute in 2009.

Mr. Baker is a past Chairman of STS Consultants, Ltd., a 550 person consulting engineering firm, headquartered in Vernon Hills, Illinois and currently serves as Senior Principal Engineer and Senior Vice President.

Uncertain Geotechnical Truth and Cost Effective High-Rise Foundation Design

Clyde N. Baker Jr.¹, P.E., S.E

¹Senior Principal Engineer, AECOM, 750 Corporate Woods Parkway, Vernon Hills, IL 60061

ABSTRACT: Uncertain geotechnical truth affects our ability to produce cost effective high rise foundation design. Virtually no amount of subsurface exploration and laboratory testing can ever provide the complete geotechnical truth. However, structural engineers and geotechnical engineers working as a team balance the available knowledge, including site geology and performance prediction history using in-situ empirically developed geotechnical parameters, with the level of acceptable risk and cost. A brief history of the author's Chicago experience and development with the Menard pressuremeter for in-situ testing is presented. Advice to follow for best results is outlined together with case histories to illustrate the following:

- 1) Maximizing allowable bearing pressure
- 2) Use of mixed foundation systems
- 3) Use of highly stressed piles for settlement reduction
- 4) Reuse of existing foundations
- 5) Construction surprises requiring remediation

INTRODUCTION

This paper, which is based on the author's 2009 Terzaghi Lecture, will share experience gained over a 57-year career, the major portion of which dealt with developing cost-effective foundations for high-rise buildings, including some of the tallest in the world, when geotechnical "truth" was uncertain. For reasons of cost, scheduling and logistics, virtually no subsurface investigation and laboratory testing program can ever provide the complete geotechnical truth. Consequently, foundation designers must balance the best available knowledge with the level of acceptable risk and cost. This paper includes case histories that highlight the contributions that judgment and innovation have made to performance-based design, using empirical methods.

BRIEF HISTORY AND PREAMBLE

The author has practiced in the Chicago area for the majority of his career. A brief

history of the development of foundation engineering in the city is presented in the following paragraphs to show the influences on the author as a geotechnical and foundation engineer.

Pre-World War II and Pre-Depression

Ralph B. Peck provided the authoritative history of building foundations in Chicago for the period before 1948, in the University of Illinois Bulletin, Volume 45, No. 29, published January 2, 1948 (Peck, 1948).

A typical soil profile for downtown Chicago is shown in Figure 1, with the various foundation types that have been used in Chicago. Prior to approximately 1883, most of the downtown Chicago structures were in the three to six-story height range, and were supported on footing foundations. In the 1890's taller buildings, up to 20 stories in height, such as the Monadnock Building and the Auditorium Building, were constructed on footing and mat foundations supported on the stiff clay crust which is encountered above the soft clay deposits.

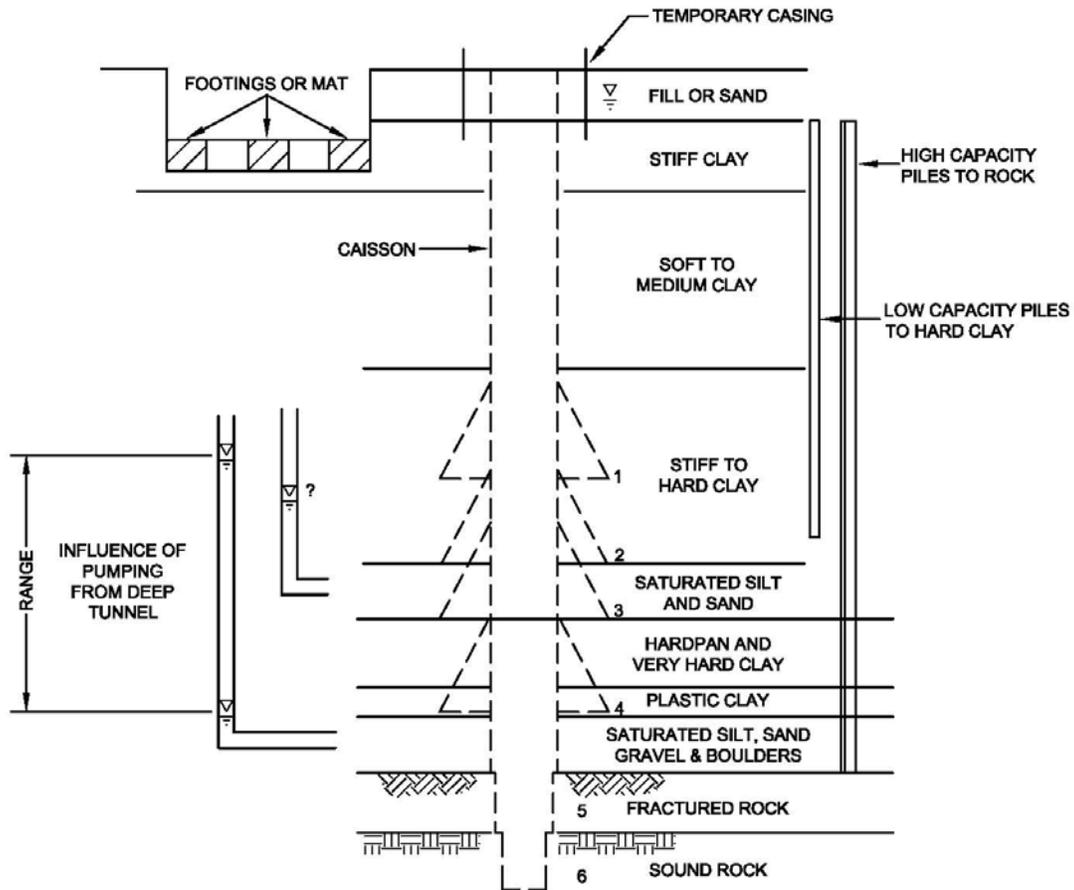


FIG. 1. Typical Soil Profile and Potential Foundations for Downtown Chicago

There was some recognition of the problem of settlement, as it was common practice to design buildings higher than planned to allow for the expected settlement. This added design height amount might vary from 60 mm (2.5 inches) to as high as nine 230 mm (nine inches). However, this allowance was not always sufficient. By 1942, settlement in excess of 760 mm (30 inches) had been reported under the tower section of the Auditorium Building, and settlement in excess of 510 mm (20 inches) was measured at the Monadnock Building (Peck, 1955).

DEVELOPMENT OF THE CHICAGO CAISSON

The Chicago caisson was developed in 1894 by General Sooy Smith to avoid the vibration problems inherent in the pile driving techniques of that time. The method involved hand digging a circular shaft with side walls maintained open by vertical lagging and steel circumferential rings. At first, the Chicago caisson method was used for underpinning or support of only part of the structure adjacent to existing buildings. Eventually consistency in the construction improved, and by approximately 1899, hand-dug caisson foundations were used under entire buildings. Figures 2 and 3 show as open, ringed and lagged straight shaft caisson at the City Hall Building, circa 1910, and workers manually digging a large diameter shaft at the Daily News Building, circa 1927. The role of the business-suited observers in Figure 3 is unknown but might be the design architect and engineer.

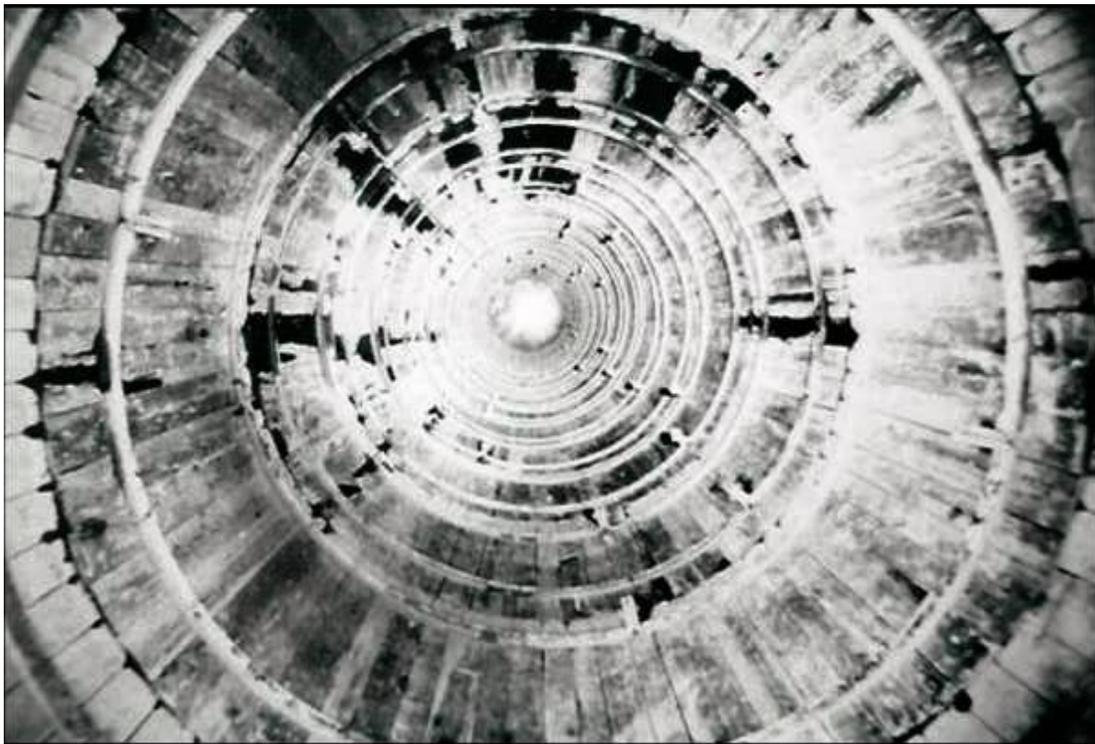


FIG. 2. Chicago City Hall caisson, circa 1910



FIG. 3. Daily News Building hand dug caisson, circa 1927

Chicago Union Station Caisson Load Test

In 1922 a very comprehensive caisson load test program was conducted by J. D'Esposito for the construction of Union Station (D'Esposito, 1924). At that time, the design bearing pressure being used on hardpan (a very dense glacial till requiring special spades to dig) was 0.57 MPa (six tons per square foot of bearing area, or 6 tsf), including the weight of the caisson concrete, with no reduction for friction.

D'Esposito performed the full scale load test program to confirm design assumptions and determine ultimate capacities. Figures 4 and 5 show this load test and the impressive results. Even though this was before the advent of modern soil mechanics, the engineers recognized that part of the test load was being carried by friction in the softer soils above the bearing stratum.

In a second test, a tunnel was advanced in the hardpan underneath the caisson shaft to remove any end bearing capability. The shaft was then reloaded to determine the side friction. The mobilized bearing resistance capacity of the hardpan was calculated to be more than 6.2 MPa (65 tsf), or more than 10 times the allowable bearing pressure they were trying to confirm. As remarkable as the results were, the preference for the use of caissons to rock at the time delayed widespread use of belled caissons bearing in hardpan.

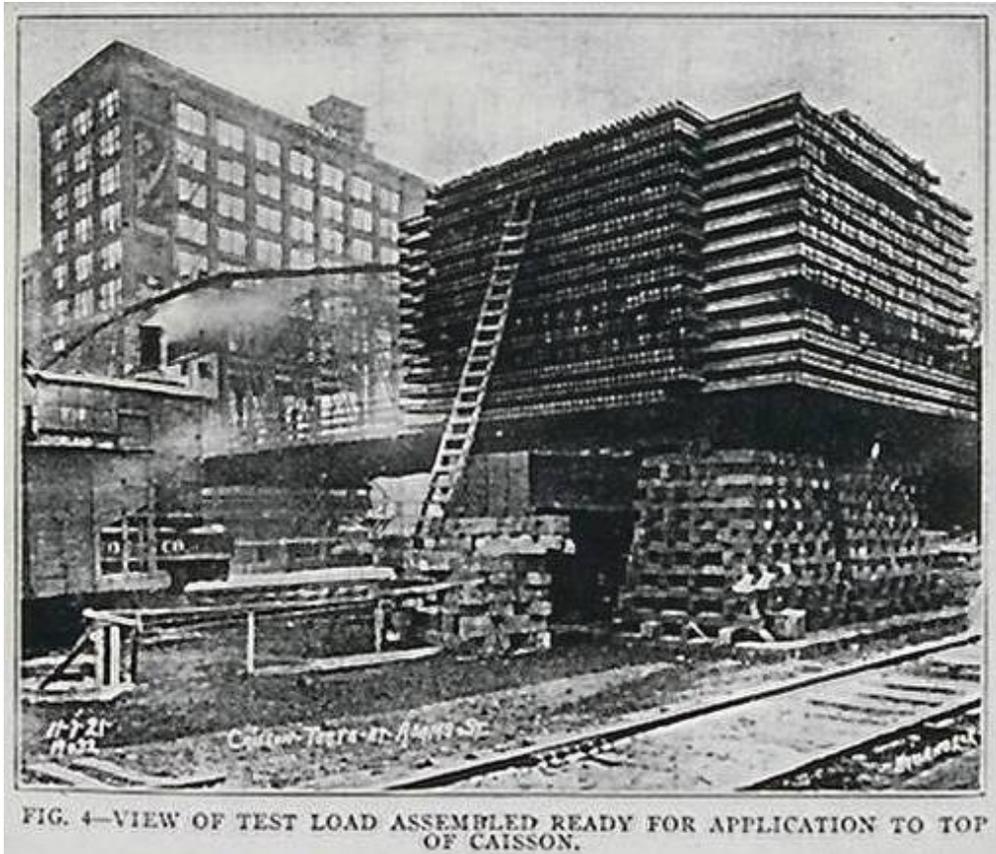


FIG. 4. View of test load assembled ready for application to top of caisson

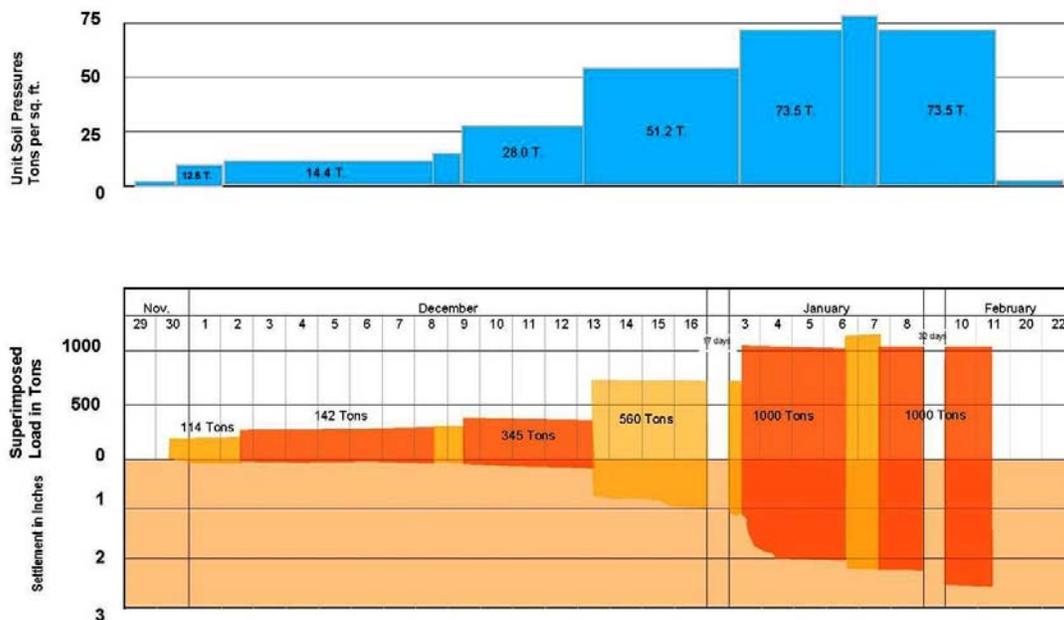


FIG. 5. Union Station caisson load test results

Between World War I and the Great Depression of the 1930's, high rise buildings in the Chicago Loop were constructed primarily on rock caissons. Belled caissons were limited by the fear that water problems associated with adjacent rock caisson construction could cause ground loss and settlement of structures not supported on rock. The conventional line of thought at the time was that a building supported on hardpan caissons could be undermined by adjacent rock caisson construction.

The design bearing pressures used for rock caissons were relatively low, in the 2.9 to 3.8 MPa (30 to 40 tsf) range, rather than the 9.6 to 28.7 MPa (200 to 600 ksf or 100 to 300 tsf) used today, so concrete stresses were also low. It is uncertain when the preferred units for foundation bearing pressure in Chicago changed from tsf to kilopounds per square foot (ksf), but ksf have become the standard, and will be used in the text of this paper henceforth.

Rock caisson construction often encountered large water flows which required extensive pumping. Tremie and rapid concrete placement methods were not yet in common use, and there are many anecdotal reports of projects where concrete was badly washed out, yet the caisson still had sufficient strength to support the structural loads safely. During construction of the foundations for the new Harris Bank building, the hand dug caissons which supported the original Harris Trust Bank building completed in 1911 were partially exposed. The caisson concrete was friable enough that the author could easily break it apart with his bare hands even though the caissons had supported this 20-story building for more than 60 years. Even with modern techniques, including larger, more powerful drilling equipment, and permanent steel casing seated in the dolomite bedrock, large water inflows made construction difficult on this site for the new Bank building built in 1975.

In contrast, where no water problems were encountered during construction, the concrete in hand-dug caissons has been found to greatly increase in strength with age. Coring performed on caissons at the Morrison Hotel, which was the tallest hotel building in the world when constructed in 1925, and which was later demolished to make space for the 1st National Bank building, indicated concrete strengths as high as 75.8 MPa (11,000 psi).

Post-World War II High-Rise Construction Boom in Chicago

The highlights of high-rise foundation construction from approximately 1948 to the present are described in the Chicago Committee on High Rise Buildings Task Force Report "History of Chicago High Rise Building Foundations, 1948 to 1998" (Baker *et al*, 1998a).

Chicago Experience In Maximizing Bearing Pressures For Foundation Design

Prior to 1969, foundation design soil bearing pressures were typically based upon unconfined compression tests. The recommended maximum allowable bearing pressure on "good" Chicago hardpan had gradually increased from 0.57 MPa (12 ksf)

to a maximum of 1.4 MPa (30 ksf) by 1965, when the 65-story Lake Point Tower was built. This increase was based upon Skempton's theory, published in 1951, that the ultimate tip capacity for a deep foundation in clay was nine times the cohesion (Skempton, 1951).

Since unconfined compression tests sometimes failed to demonstrate an adequate factor of safety due to the silt, sand, and gravel content in the hardpan, triaxial compression tests were performed to confirm the design bearing pressure. Triaxial testing could demonstrate significant strength in the hardpan, and very high theoretical bearing capacity values at depth, with actual values dependent on the bearing capacity factors selected. It was realized that foundations could not be effectively designed using the allowable bearing pressure derived from the triaxial test results, because of lack of confidence in our settlement prediction capability using the triaxial test results.

The development of the in-situ pressuremeter test offered distinct advantages in that it avoided the potential sample disturbance inherent in sampling and testing in the laboratory. It was seen as analogous to an in-the-ground load test, and in a very short time frame it was well correlated with building performance. By the 1980's the allowable bearing pressures on good hardpan increased from 1.4 MPa to 2.4 MPa (30 ksf to 50 ksf).

DETERMINATION OF PRE-CONSOLIDATION PRESSURE

Early research by Lukas and DeBussy (1976) indicated that the creep pressure determined during the performance of the in-situ pressuremeter test compared favorably to the preconsolidation pressure determined from high quality consolidation tests. One of the difficulties of determining preconsolidation pressure from consolidation tests on glacial till is the lack of a sharp break in the load deformation curve, which leaves considerable room for interpretation. The creep pressure from the pressuremeter tests appeared to be a simpler and more reliably consistent means for determining the preconsolidation pressure.

The creep pressure marks the transition from higher to lower soil stiffness. For this reason, the applied pressures were typically limited to these creep values. The applied pressure as computed can be the total dead load and effective long term live load stress values plus effective overburden stress. If these stresses exceed the creep pressure, some unpredictable additional settlement beyond the pseudo elastic compression can be expected.

SETTLEMENT THEORIES USING PRESSUREMETER TEST DATA

The two most common approaches for predicting settlement using pressuremeter data are the Ménard semi-empirical procedure described by Ménard (1975) and Briaud (1992), and elastic theory, in which the pressuremeter is utilized to determine an equivalent Young's modulus. The question here is how best to determine the effective Young's modulus. Since the modulus undoubtedly varies with the stress and strain level, there is no single answer to this question. A theoretically correct approach would include utilizing sophisticated numerical methods that vary stiffness depending

upon the computed stress and strain level, and would involve special tests at the stress/strain level anticipated in each soil strata below the bearing level.

At the time, such tests were not available outside the research environment. However, the semi-empirical settlement equation proposed by Ménard showed to be an excellent design tool since it was based on correlation with actual load tests. The Ménard settlement equation is defined as follows:

$$S = (1.33/3E_b) p R_0 (\lambda_2 R/R_0)^\alpha + (\alpha/4.5E_1) p \lambda_3 R \quad (1)$$

Where:

p = Net bearing pressure

E_d = Pressuremeter initial load test modulus

E_+ = Pressuremeter reload modulus

E_b = Effective pressuremeter modulus below bearing level, as calculated in Figure 2 of Ménard (1975)

E_1 = Pressuremeter modulus for 1 radius below bearing

λ_2, λ_3 = shape factors (1.0 for circle)

R = Bell or bearing area radius

R_0 = Reference radius (30 cm or 1 ft)

α = Empirical factor that varies depending on local geology, but often taken as E_d/E_+ .

The α factor introduces the empiricism in the equation since it was estimated or back calculated by Ménard to obtain the best match between the predictive foundation and the observed performance. The results of Ménard's recommendations for the values of α are summarized in Table 1.

Table 1. Menard's α Factors (Menard 1975)

Soil Type	Peat		Clay		Silt		Sand		Sand and Gravel	
	E/P_1	α	E/P_1	α	E/P_1	α	E/P_1	α	E/P_1	α
Overconsolidated			>16	1	>14	$\frac{2}{3}$	>12	$\frac{1}{3}$	>10	$\frac{1}{3}$
Normally Consolidated	For All Values	1	9-16	$\frac{2}{3}$	6-14	$\frac{1}{2}$	7-12	$\frac{1}{3}$	6-10	$\frac{1}{4}$
Weathered and/or remolded			7-9	$\frac{1}{2}$		$\frac{1}{2}$		$\frac{1}{3}$		$\frac{1}{4}$

Although the values of α suggested by Ménard provide generally good estimates of settlements, the author considers that it is most appropriate for the engineers to estimate their own values of α for the different geologies depending on local

experience, since this adjustment factor does not only strictly depend on the soil/rock type, but also encapsulates the local state of practice. The author's observations on projects in the Chicago area have shown that α coincided reasonably with the ratio of the initial pressuremeter modulus E_d and the reload modulus E^+ , except that values of less than 0.5 for clay, or less than 0.4 for silt, or less than 0.33 for sand are not used in Chicago geology.

For this reason, when running the pressuremeter test, the author recommends running an unload/reload cycle between a point slightly below the creep pressure P_f to a level slightly above P_o , the in-situ horizontal earth pressure as shown in Figure 6.

Pressuremeter Data Reduction (BX)

E_d = Deformation Modulus
 E_o = Rebound Modulus
 E^+ = Recompression Modulus
 P_f = Creep Limit
 P_l = Limit Pressure
 $\alpha = E_d/E^+$

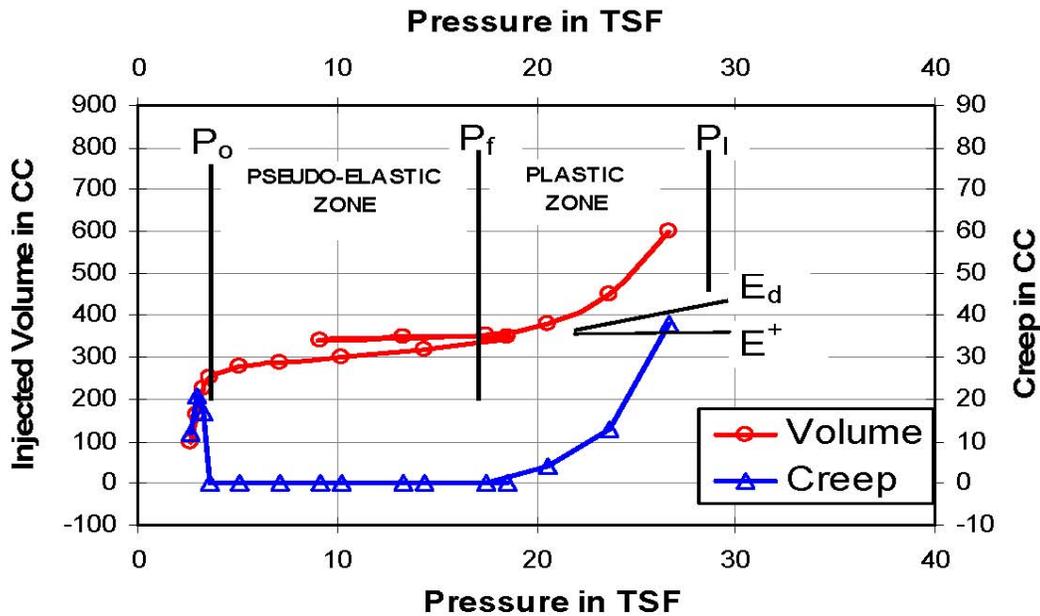


FIG.6. Pressuremeter Data Reduction Outcomes

In elastic theory, the formula for settlement is:

$$\text{Settlement } s = \frac{\mu_0 \mu_1 B q}{E} \quad (\text{Janbu, Bjerrum and Kjaernsli, 1966}) \quad (2)$$

where:

q = bearing pressure

B = foundation width

E = modulus of elasticity

$\mu_0 \mu_1$ = geometry influence factors

In both prediction theories, it is assumed that the stress level is within the pseudo-

elastic range. In pressuremeter terminology, this means that the total stresses must be below the creep pressure.

The author's confidence in the pressuremeter increased following construction of Water Tower Place, which, at 75 stories, was the tallest reinforced concrete building in the world at the time (1976). The predicted settlement of 50 mm (two inches) matched closely the actual post-construction measured settlement of 43 mm to 56 mm (1.69 inches to 2.19 inches). Good correlation was also achieved when back-calculating a predicted settlement using pressuremeter data on the relatively few full-scale caisson load tests performed in Chicago (Baker, 1985).

CURRENT SITUATION

Pressure for More Cost Effective Foundations

Because the results cannot be seen, the desire not to spend any more money below ground than is absolutely necessary has increased over the last 30 years. Private developers can sometimes base the viability of the project on the cost of the foundations. Geotechnical engineers are being increasingly pressured to provide higher foundation performance at lower cost without sacrificing the factor of safety.

In the author's experience, there have been few, if any, major high rise foundation collapses but there have been significant cases where the foundations have not met the anticipated performance in terms of settlement.

A reasonable explanation is that conventional design practice utilizes relatively high factors of safety in the supporting foundation materials. Normal factors of safety range from two when full scale foundation load tests are performed, to three when the capacity analyses are based on laboratory tests only. An intermediate value between two and three is often applied when appropriate in-situ soil testing is performed. In contrast, factors of safety of 1.5 and even lower are applied to the stability of large earth dams, where there have been some well known failures, as discussed by Casagrande in his Terzaghi Lecture "Role of the Calculated Risk in Earthwork and Foundation Engineering" (Casagrande, 1965).

Judgment Required in Evaluating Geotechnical Uncertainty and the Risk Involved in Reducing Foundation Costs

In general, foundation costs often can be reduced by:

- Maximizing allowable bearing pressure
- Using mixed foundation systems
- Using highly stressed piles as settlement reducers, and
- Re-using existing foundations.

In all cases, it is advisable to have a remediation plan in mind for construction or building performance surprises.

When considering ways to reduce foundation costs, knowledge of the local geology and building history is invaluable. However, we may not know as much as we think we do, and things are not always what they seem. The foundation design procedures followed on two of the best known and world's tallest buildings, the Petronas Tower and Burj Dubai, provide an excellent example of not knowing as much as we might think.

Both of these projects involved a mat or raft on long bored piles or barrettes. The elastic modulus used for predicting settlements was based on local experience - correlated with laboratory tests, pressuremeter tests, and pile load tests. However, the assumed elastic modulus in each case was based on different local experience. On Petronas Towers, the elastic modulus was assumed approximately equal to the average pressuremeter reload modulus, whereas on the Burj Dubai project, it worked out to about 0.2 times the pressuremeter reload modulus.

The Petronas Towers foundations were supported on a Kenny Hill formation which consists of highly folded and severely weathered layers of sandstone, siltstone and shale, that is weathered to soil, but retains a remnant rock relic structure. The Burj Dubai bearing stratum, however, consists of low density, poorly cemented, relatively horizontal layers of sandstone and siltstone with varying calcium carbonate concentration. The typical ratio of reload modulus to the initial load modulus was in the range of 2.5 to 3 in the Kenny Hill material, and as high as 10 in the Burj Dubai material. Using these values, the predicted settlement for the Petronas Towers was approximately twice that observed, whereas at Burj Dubai it appears to be closer, but was less than what was observed. This should be proof positive that local experience correlated with local geology is essential. Using the same procedures for both projects could have either significantly under-predicted or over-predicted settlement. Good judgment, or perhaps fortunate judgment, was used for such adverse conditions.

Requirements and Advice

Over the years, the author has developed several special rules to apply when required to be innovative, but geotechnical truths are uncertain.

Requirements

The structural engineer (S.E.) and geotechnical engineer (G.E.) must serve as a team. They must have confidence in each other and communicate well with each other and with the Architect and the Owner.

Advice

To reduce risk when dealing with geotechnical uncertainty:

1. Know and understand your local geology.
2. Compare and calibrate with known case histories in similar geology.
3. G.E. and S.E. should develop the geotechnical exploration program together, including in-situ testing and special laboratory testing. Type and extent may

vary widely, depending on geology, project scope, and complexity, as well as budget and risk constraints.

4. Perform simple bearing capacity and settlement analyses before more sophisticated finite element programs.
5. Perform instrumented load test program (to failure when practical).
6. When operating at the limit of past experience, take small steps. If a large step is necessary, be more conservative.
7. Require preconstruction contractor conference or consultation – Design must be buildable.
8. Go through the “what ifs” and have a “Plan B” ready if poor performance exceeds acceptable limits.
9. Have adequate construction monitoring and quality control programs, with effective communication between field personnel and the design office.
10. Assume but verify. If not possible to verify, then steps 6, 8 and 9 are especially important.

CASE HISTORIES

The following section presents several case histories that demonstrate how judgment, innovation and cost effectiveness have worked in the following categories:

1. Maximizing allowable bearing pressure
2. Use of mixed foundation systems
3. Use of highly stressed piles as settlement reducers
4. Re-use of existing foundations
5. Construction surprises requiring remediation.

In all of these cases, an essential factor was that the structural engineer and geotechnical engineer worked as a team.

Case History No. 1: AT&T Building

AT&T was the first building in Chicago to utilize a bearing pressure of more than 1.9 MPa (40 ksf) on the Chicago hardpan. The foundation engineering for this project was described in a presentation to the Foundation Engineering Congress held in Chicago in 1989 (Baker *et al*, 1989). The project was complicated by the presence of existing caissons and an adjacent 10-story building supported on footing foundations above the Chicago soft clay. A logical but expensive solution was to support the tower on rock caissons. Rock caissons were priced at three times the cost of belled hardpan caissons. Because of the presence of the existing caissons it was necessary to extend the very heavily loaded new caissons, about 14,000 kips (7000 t), that would support the building core, through the hardpan and into a very dense water-bearing silt, sand and gravel, as shown in Figure 7.

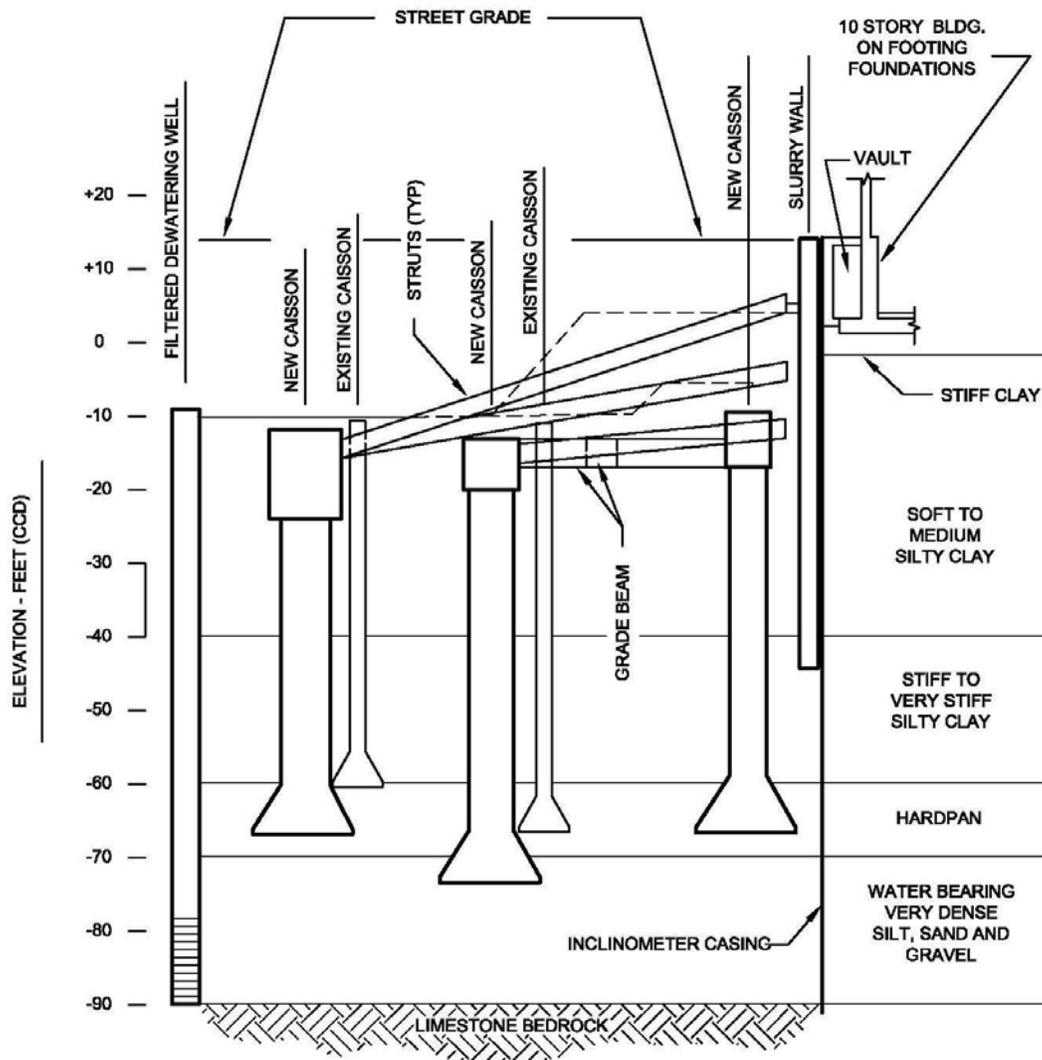


FIG. 7. AT&T Corporate Center Substructure Profile

Pressuremeter testing indicated that 2.25 MPa (45 ksf) bearing could be used in the hardpan and as much as 2.4 MPa (50 ksf) bearing could be used in the underlying very dense silt, sand and gravel if bells could be constructed. This required the use of filtered dewatering wells to lower the water table so that the bells could be constructed. However, it was not known whether the excavated bell walls would remain stable in the silty sand and gravel even when dewatered. A contingency plan was developed as shown in Figure 8 which involved constructing an oversized bell that would be filled with low-strength grout so that a design sized bell could be constructed within the grout bell the next day. This approach had been used successfully on other projects when bell excavations penetrated water-bearing sand (Baker *et al*, 1989). The owner's representative was made aware of this contingency plan and the potential costs. Fortunately for this project, the surface tension developed by dewatering provided enough apparent cohesion to the very dense silt, sand and gravel to keep it stable during bell excavation.

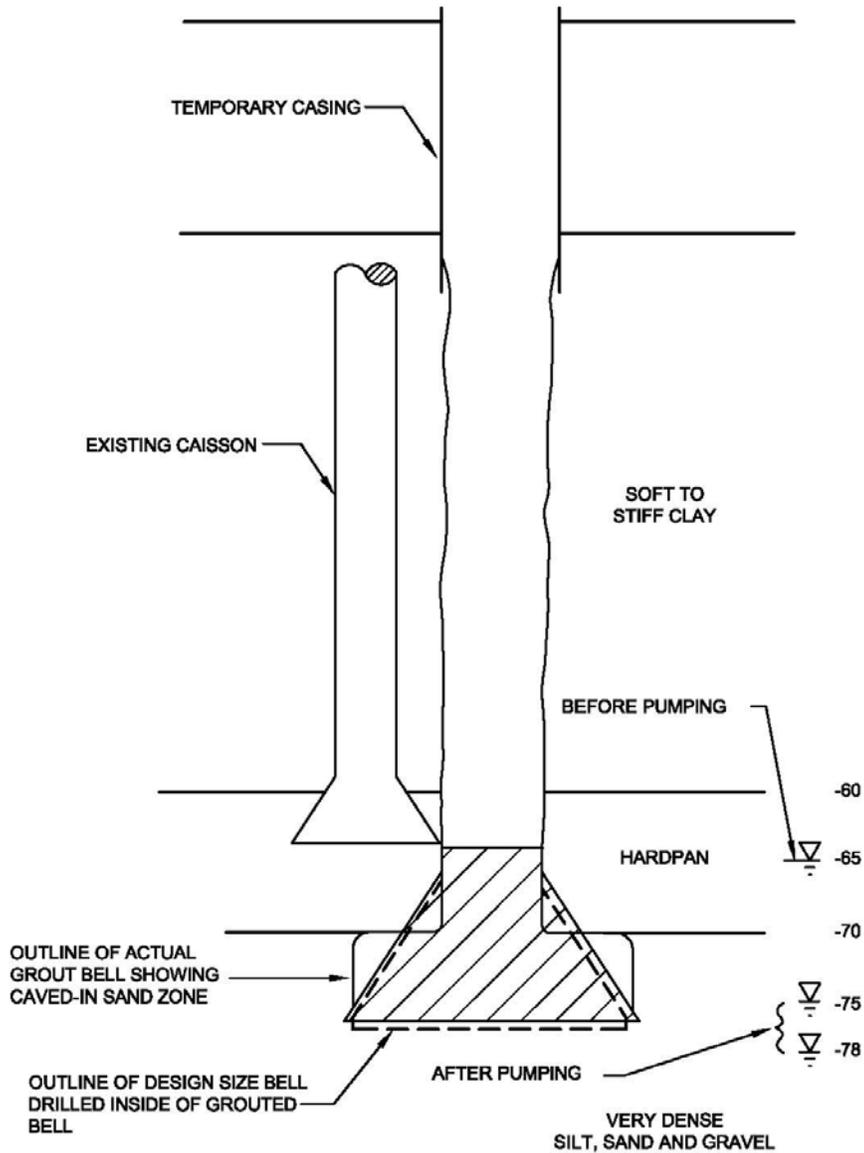


FIG. 8. AT&T “Grouted Bell” Method of Caisson Construction in Caving Soil

Prediction Versus Performance

Measurements taken during construction at the AT&T building demonstrated settlement less than that predicted by the pressuremeter testing even at the unprecedented bearing pressures of 45 and 50 ksf.

- Settlement – Predicted and Observed

Table 2. Predicted vs. Observed Settlement

Predicted vs. Observed Settlement			
Location	Predicted at full load	Observed at % load	Extrapolated to full load
Core (on very dense soils)	24 mm	14 mm at 80%	17.8 mm
Perimeter (on clayey hardpan)	25.4 mm	14.2 mm at 70%	20 mm

Risks versus Cost Evaluation

The innovative aspect of the design was the use of the filtered wells to temporarily lower the water table and enable construction of the large diameter bells in the silt. What was the fallback position if this approach did not work? Since the owner is balancing the cost benefits against the risk of the innovative solution, he needs to know what the down side could be if the procedure does not work. On this project the options were limited to either adding wells until sufficient dewatering had occurred, or redesigning the core caissons as straight shafts socketed into bedrock. Because an extended construction delay to add wells was considered to be a greater risk than the potential different settlement between rock and belled caissons, the dewatering wells were installed early enough so that a design switch could have been made if the dewatering was not successful.

Although three dewatering wells were installed, most of the water was pumped from one highly productive well. At the author’s direction, the productive wells had been installed near the location of a preconstruction borehole that had reported loss of drilling water in the desired extraction zone indicating high permeability.

The risk of bell cave-in in the dewatered silt was offset by the remediation plan described above. Nearby piezometers were used to determine whether the water table had been drawn down sufficiently for caisson construction to begin.

Assuming successful dewatering, the only remaining risk was greater settlement than predicted. Based on experience developed with using the pressuremeter results for settlement prediction on other nearby projects, this risk was considered small.

Case History No. 2: Dearborn Center

Dearborn Center is a 38-story office building constructed on the existing caisson foundations of an 11-story building that had been demolished down to street level.

The geotechnical program for this project consisted of performing seven new soil borings. These borings supplemented ten earlier borings performed for an 85-story tower on rock socketed caissons that was never built. Nine of the 10 historic borings were performed outside the existing building perimeter. Five of the new borings were

performed from the existing lowest basement elevation at -23 ft. Chicago City Datum (CCD) with two borings performed at the first basement level at elevation -4 ft. CCD. A location plan showing all borings, as well as the existing caissons, is included as Figure 9. Borings were performed adjacent to existing Columns 36, 56 and 125 to confirm the presence of the bells and to access the soil immediately below the bells for testing. These borings were advanced to the top of the caisson bell, which was cored with diamond-bit tools to confirm concrete integrity. Coring of the shafts was not considered necessary, since experience has indicated that the poorest concrete is usually found in the bells, as it does not get the same compaction and consolidation effect as the shaft concrete. These three borings were then extended below the bottom of the caisson bell to elevations ranging from -79 to -85 ft. CCD. Pressuremeter tests were performed below the caisson bell in all three of these borings.

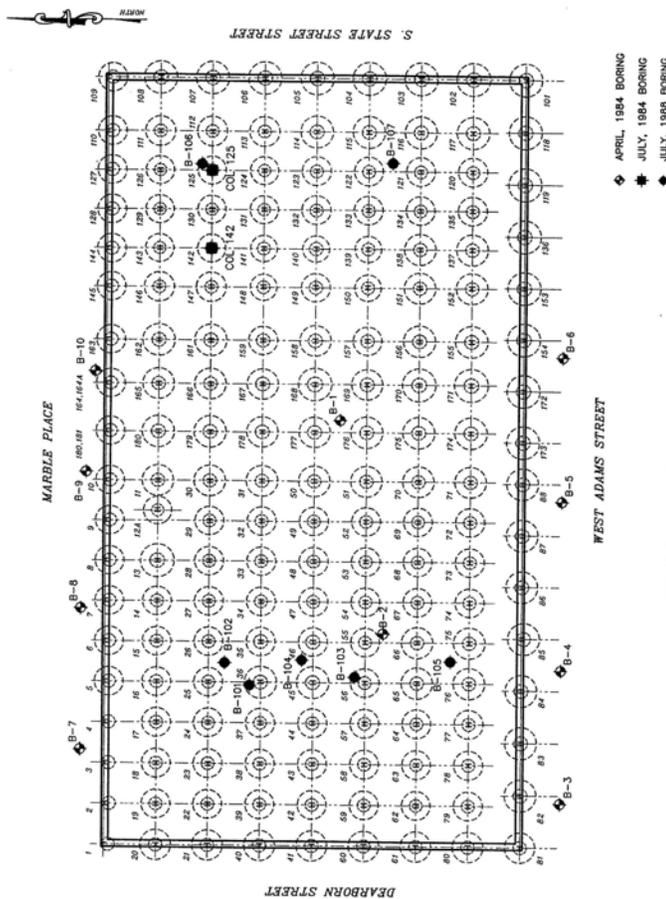


FIG. 9. Dearborn Center Caisson Foundation Plan

Unconfined compression tests were performed on representative samples of the caisson bell concrete and indicated strengths ranging from 44 MPa to 54.5 MPa (6300 to 7800 psi). These results were similar to those obtained in an earlier investigation performed by others in 1984.

A typical soil profile and a graphical plot of the key pressuremeter test results are shown on Figure 10 and 11. A complete tabulation of pressuremeter test results was presented at the 5th International Conference on Case Histories (Baker *et al*, 2004a).

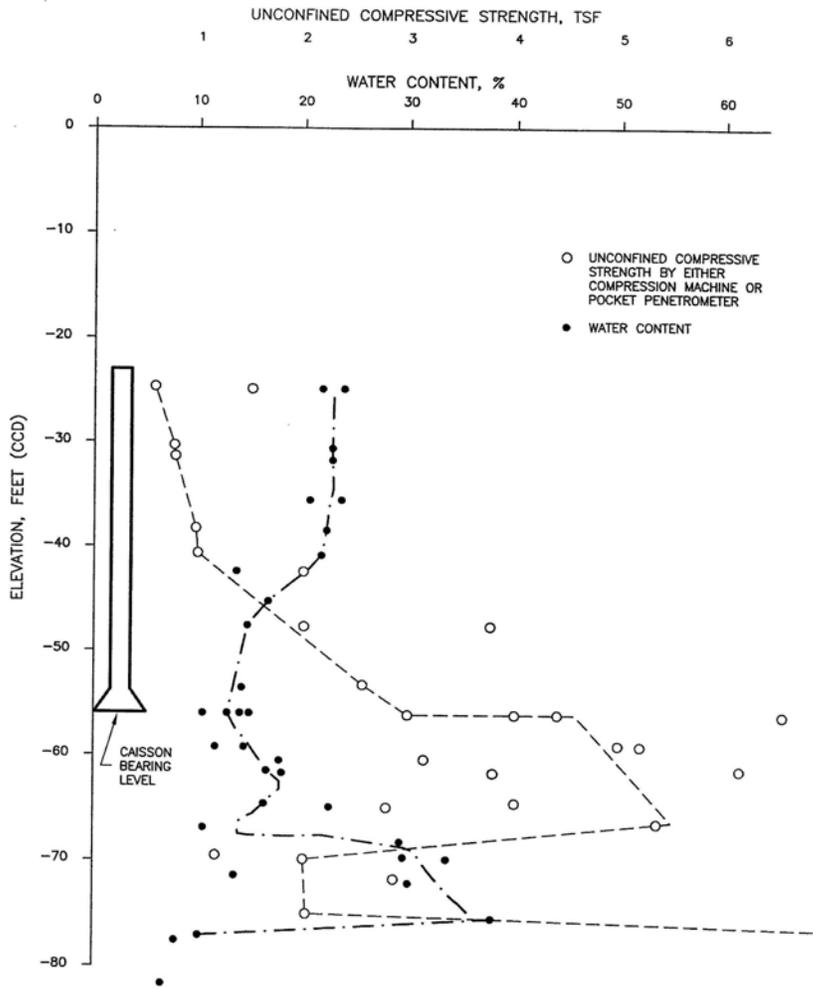


FIG. 10. Dearborn Center Unconfined Compressive Strength and Water Content Evaluation

Geotechnical Analysis

The design concept for the Dearborn Center project was to make cost effective use of the existing substructure at the site, while at the same time permitting development of the maximum practical number of office floors above the existing substructure, and two proposed retail levels on the first and second floors. Substructure levels would be utilized primarily for car parking. To accomplish this, the existing belled caisson foundations were re-used and additional load carrying capacity was developed by using a mat placed on top of the bottom basement slab. Because of the stress unloading of the subsoils below mat level due to the 11.3 m (37 feet) of basement excavation, it was anticipated that significant loads, up to the weight of soil removed, could be applied at the mat level with only a modest settlement.

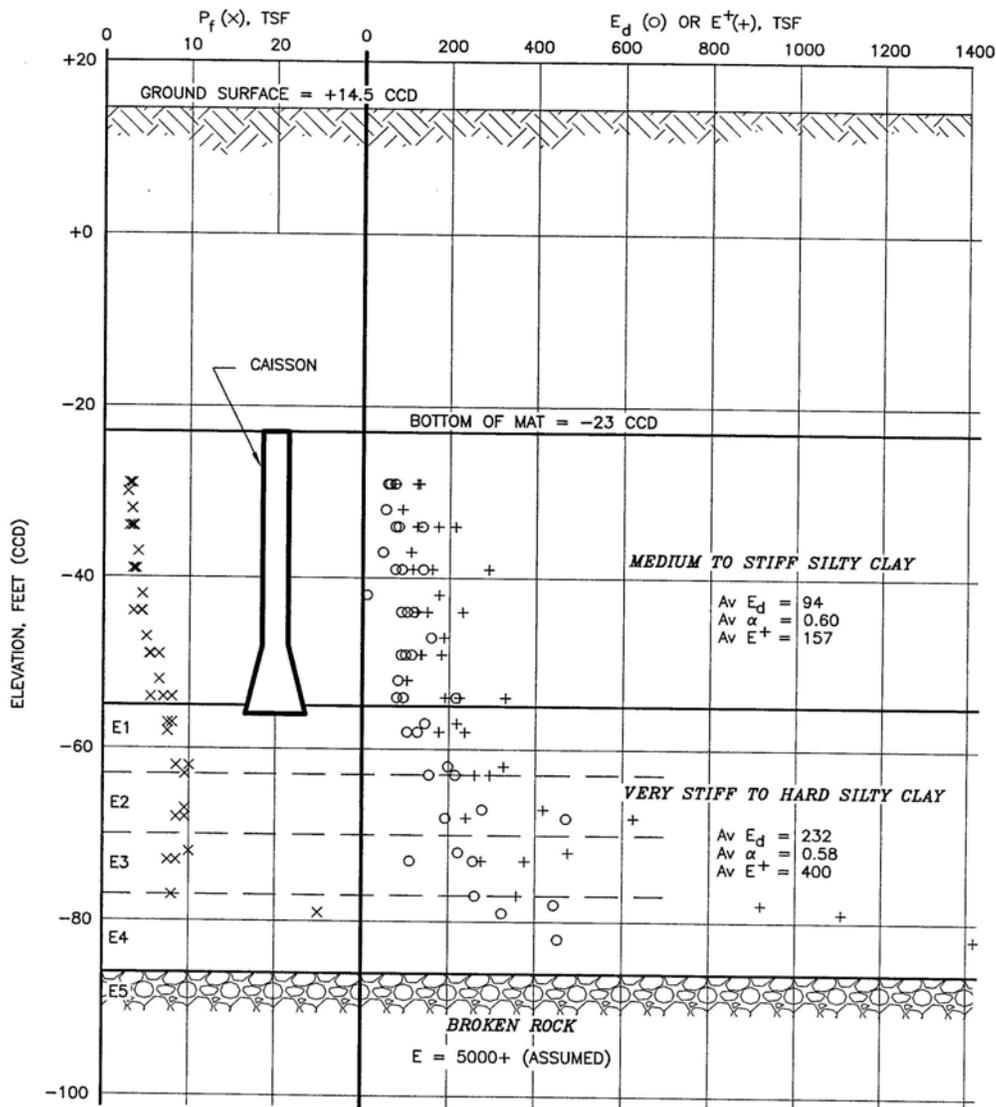


FIG. 11. Dearborn Center Pressuremeter Profile

The pressuremeter tests indicated an average creep pressure of approximately 862 kPa (18 ksf) in the very stiff to hard silty clay zone beneath the caissons. The drop-off in unconfined compressive strength and increase in water content noted in the zone from -68 to -75 ft. CCD (Figure 10) did not result in significantly reduced modulus or creep pressure value, indicating a fairly consistent preconsolidation pressure. It is likely that the higher water content indicates greater plasticity and moisture retention under comparable loads. In order for the settlement to be within the pseudo-elastic zone, the dead load and long-term live load bearing stress plus the overburden pressure should not exceed the average creep pressure. Thus, allowing for an existing overburden pressure of approximately 192 kPa (4 ksf) relative to top of mat level, the maximum dead load pressure could not exceed 670 kPa (14 ksf) to keep the combined

total less than the average creep pressure of 862 kPa (18 ksf). If the bearing pressure under the caissons exceeds this value, there would be a tendency towards increasing settlement and load transfer back to the mat. Caisson springs for use in a mat finite element analysis were developed assuming approximately 25 mm (1 inch) deflection under a pressure of 862 kPa (18 ksf) on a representative 4.3 m (14 foot) diameter belled caisson. Illustrative calculations were presented at the 5th International Conference on Case Histories (Baker *et al*, 2004a)

With regard to the mat, utilizing the pressuremeter data obtained in the subsoils beneath the mat, the average mat pressure required to produce a 25 mm (1 inch) settlement comparable to the caisson settlement is approximately 96 kPa (2000 pounds per square foot (psf)). This data can be used to calculate spring constants under the mat for use in a finite element analysis. This pressure/deflection estimate is based upon an elastic analysis using a Young's modulus for the soil zone beneath mat level of two times the pressuremeter reload modulus. This is an empirically derived relationship based upon monitoring of the performance of another heavily loaded pile supported mat, for the Petronas Towers, described in a presentation to the 4th International Conference on Case Histories (Baker *et al*, 1998b). At that time a value of Young's modulus approximately equal to the reload modulus was used. Since settlement of the Petronas Towers was over-predicted by a factor of two, the foundation design team on the Dearborn Center project decided to use a value of two times the reload modulus for Young's modulus. The author and his colleagues also had observed on other projects, such as the AT&T Building (Case History No. 1), that, while there were good performance checks when using the Ménard rules to predict settlement, using the elastic theory approach with Young's modulus equal to E+ typically over-predicted settlement in the Chicago glacial till soils.

Foundation Performance

Settlement reference marks were set on the building walls and mat at the start of construction. Those that were not covered, and could be found during construction, were checked periodically through construction and tenant occupation, almost to completion of the project. At the time of the last settlement data, it was estimated that approximately 70% of the full design load was being supported by the foundations. The reported settlements varied from zero at the north wall, which was reported to be supported on rock caissons, to 13 mm (½ in) at the west wall. A settlement of 15.9 mm (5/8 in) was reported at the south wall and the interior mat. Allowing for survey accuracy of 3.2 mm (1/8 in), settlements were estimated to range from 3.2 mm (1/8 in) at the rock supported caissons to 19 mm (¾ in) elsewhere. This agrees with predictions used in the design and confirms the adequacy of the basic assumptions made and analyses performed, even though foundation support under the 38-story building varied from moderately hard rock to medium to stiff clay.

Risks versus Cost Evaluation

In this project the primary risk was under-predicting the anticipated settlement. The author's experience with the local Chicago glacial tills was important here. If settlements had been greater than predicted, the existing caissons would have been

pushed further into the underlying soil, exceeding the soil creep pressure and making them act more like settlement reducing piles rather than required structural members. In this case the differential settlement between the corner of the mat supported on rock caissons and the rest of the mat would have been more significant.

If, as construction of this building progressed, differential settlement became intolerable, supplementary micropile support and/or mat stiffening walls would have been required at appropriate locations to reduce the differential to acceptable levels.

Case History No. 3: Chicago Southside Office Building

Construction of a 10-story combination parking structure and office building, with provision for two more stories, was completed in 1996 at 1911 South Indiana Avenue in Chicago, Illinois. This project was described in a presentation to the 5th International Conference on Case Histories (Baker *et al*, 2004b). The new structure was of reinforced concrete design with 7.3 x 12.2 m (24 ft x 40 ft) bays. The lower floor levels are for parking, the upper floors are office space. The lowest floor over half of the structure is at grade, while the lowest floor on the other half is depressed approximately four feet below grade.

Because of the potential for squeezing of soft clays and the relative thinness of an adequate bearing layer at depth, a preliminary geotechnical report declared construction of conventional belled caissons for this project to be too risky and expensive. The author's firm was retained to further evaluate a shallow foundation solution and provide cost effective methods for reducing the anticipated settlement. A supplementary field exploration program was performed consisting of five (5) borings, including in-situ pressuremeter tests conducted within the upper sands just below anticipated footing level, pressuremeter testing within the lower silty sands just below potential deep caisson bearing level, in-situ vane shear testing within the soft clay below footing level and selective, undisturbed three-inch diameter piston sampling of soft clay for consolidation testing, as well as shallow and deep water table measurements. The soil profile, together with the final instrumented foundation system selected is shown in Figure 12.

Shallow Foundation Analysis

Because a dense sand layer and stiff clay crust provided a separation from the soft clay for support of the structure, it was believed that the footings and sand would effectively behave like a mat equivalent. Because of the stress spreading effect, the actual bearing pressure design of the footings would have less influence on the ultimate settlement since it is primarily the average stress increase in the underlying soft clay that causes the settlement. Even allowing for a small amount of preconsolidation in the underlying soft clay due to partial post glacial desiccation, calculated maximum settlement for this equivalent mat case was 200 mm (eight inches), with about 50-75 mm (two to three inches) occurring during construction and 130-150 mm (five to six inches) thereafter. This was considered excessive and ruled out shallow foundation alone solutions.

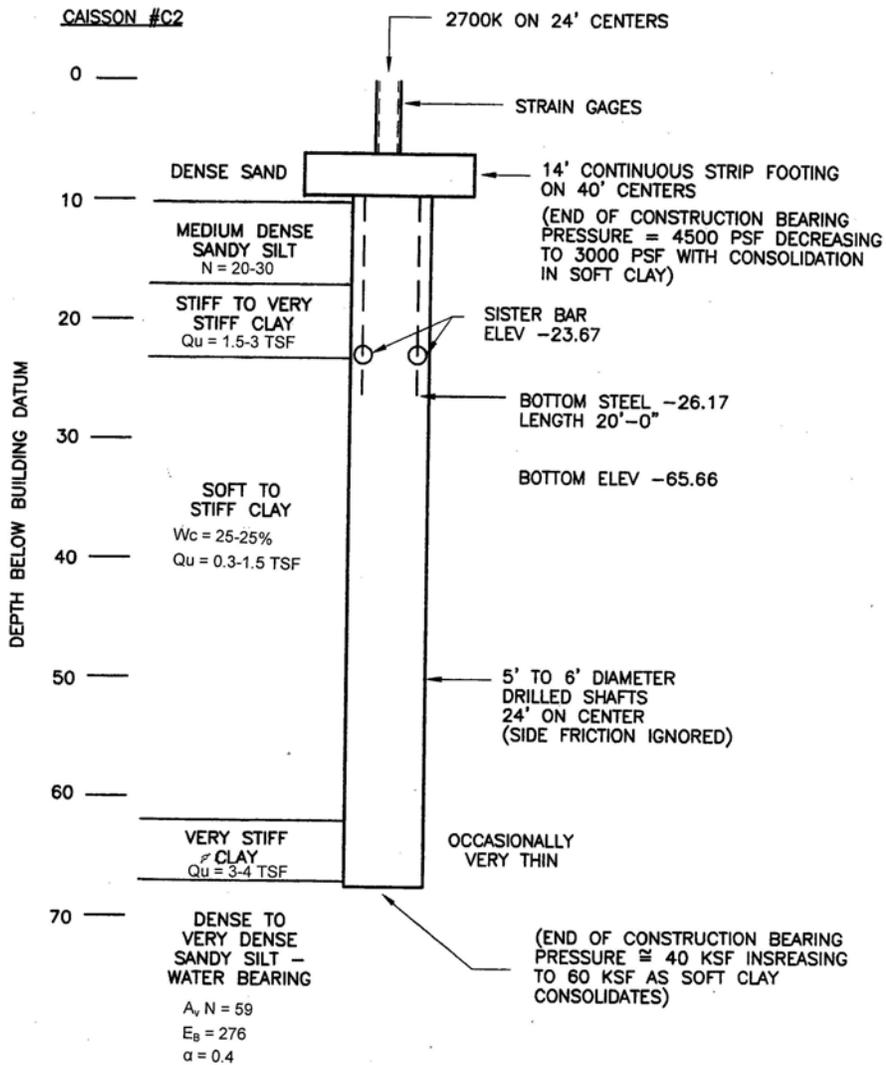


FIG. 12. 12-Story Parking Garage and Office Combined Strip Footing Drilled Shaft Interaction Instrumentation

Deep Foundation Analysis

Various deep foundation solutions were considered including rock caissons, piles, and straight-shaft caissons with normal allowable bearing pressures, but cost estimates on all solutions exceeded the project budget.

Combination System Analysis

To reduce the settlement to an acceptable range, a combination system was designed, consisting of 4.3 m (14 foot) wide continuous strip footings on a 14.2 m (40 ft) spacing supported on the surface dense sand layer, and 1.5-1.8 m (five to six foot) diameter straight-shaft caissons extended down to the dense water-bearing sand and silt layer. Based on local experience, it was anticipated that the straight-shaft caissons could be excavated and filled with concrete before water seepage became a problem. This was not considered possible for belled caissons.

Approximately 60% of the building load was assumed to be initially supported by the strip footings with 40% carried by the straight shafts. This ratio was expected to reverse with long-term consolidation of the soft clay due to the strip footing pressures. The combination foundation reduced the projected settlement to less than one-third of that predicted for either the strip footing or mat foundation solution alone. Since the straight shafts were considered primarily as settlement reducers, a higher than normal bearing pressure could be accepted, consistent with the desired settlement limitation.

The design approach was relatively unique in the sense that the settlement reducing elements carry the load primarily in end-bearing rather than in side friction, which is the common system where a mat combined with settlement reducing piles is normally utilized. This approach is only practical if end bearing is in strain hardening material that offers increased resistance to increasing penetration without a sudden plunging failure point such as a dense frictional material at depth, as opposed to a hard cohesive layer over softer material, which could allow plunging if it failed.

The structural engineer designed the strip footing to withstand higher than anticipated soil pressure, since it is not possible to guarantee the exact load distribution between footing and shaft, particularly with time, as the underlying soft clay consolidates. Ultimate projected settlement for the fully loaded combination system was on the order of 50 mm (two inches) compared to 175 to 200 mm (seven to eight inches) for the strip footings only.

In order to determine how load actually is distributed between the shafts and the strip footing, strain gages were placed in two representative shafts and first floor columns as shown on Figure 12.

Settlements measured since completion of building ranged from 23.9 mm (0.9 in) at Column C2 to 32 mm (1.25 in) at Column B6. Column B6 had the greatest percentage of the load carried by the caisson shaft as compared to the strip footing. This was probably due to the fact that the column was at the end of the footing and did not get the same stress spreading influence that the massive footing provides for interior columns. The B6 caisson appeared to be carrying 76% of the column load whereas the C2 caisson appears to be carrying 59% of the column load. It should be noted that the structure was designed for two additional floors so the loading corresponding to the instrument readings was only approximately 83% of the ultimate design loading. A summary is shown in Table 3.

From the data provided, it appeared that the caissons were behaving slightly stiffer than anticipated and the ultimate settlement would be slightly less than predicted. In making the original calculations for load sharing between footing and shaft and settlement of footing and shaft, some adjustments were made to the parameters used in the bearing stratum below the shafts.

Table 3. 1911 South Indiana Instrumentation Results

1911 South Indiana Instrumentation					
Results as of April 14, 1998 (4 years)					
Column No.	Column Gage Avg. (microstrains)	Calculated Column Load (kips)	Shaft Gage Avg. (microstrains)	Calculated Shaft Load (kips)	Measured Settlement (inches)
B6 (16"x36" col. 8000 psi conc. over 5' diam. 4000 psi shaft)	687	1978	147	1496 (76 ksf)	1.25
C2 (16"x48" col. 8000 psi conc. over 6' diam 4000 psi shaft)	606	2327	94	1377 (48 ksf)	0.9

Conversion Key: 1 inch = 25.4 mm
1 kip = 4.45 kN

1 ksf = 47.9 kPa
1 psi = 6.9 kPa

Since deep pressuremeter tests, unfortunately, were performed in only one boring, there was concern the data might not be representative and could be unconservative. To check for this, the test data was compared to the average Standard Penetration Test values adjacent to the pressuremeter tests and to the overall average N value at the test level. A proportional downward adjustment was made in the design parameters to reflect the fact that the overall average N value was less than the average N value at the pressuremeter test boring. In addition, the modulus values were conservatively adjusted to account for possible disturbance and loosening upon shaft excavation. Modulus values were reduced in half to allow for this possible seepage loosening effect.

However, it would appear from the settlement data that no such loosening effect occurred, and that a better correlation of prediction and performance would have been obtained by using the pressuremeter data without the loosening adjustment. In fact the settlement predicted from calculations then agrees very well. At 2874 kPa (60 ksf) bearing, the calculated settlement is 24 mm (0.9 inches). The observed settlement under shaft C-2 was approximately 24 mm (0.9 inches) at 2825 kPa (59 ksf).

Considering the limited amount of pressuremeter data, the close correlation was a little surprising. However, when working with limited data and making engineering judgment decisions, it is best to stay on the conservative side with any assumptions.

Risk versus Cost Analysis

Engineering judgment played an important role here in evaluating risks and cost. The approach of correlating the pressuremeter modulus at one location with the more prevalent Standard Penetration Test was based on the local experience of the author. The automatic hammers used today in performing Standard Penetration Tests provide much more consistent and reproducible data than the earlier hammers operated with a drill-rig cat-head and two loops of the manually controlled lifting rope.

The reduction in average calculated modulus to allow for some bottom loosening at the bearing level was also based on judgment and a desire to be more conservative. Some consideration was given to coring through a shaft and running additional pressuremeter tests below it, but, since the predicted settlements using the conservatively reduced modulus were acceptable, the additional testing was deemed unnecessary.

Case History No. 4: Chicago Southside Condominium Building

Value Engineering

One of the risks in the geotechnical engineering business is value engineering. The phrase “We can save you money.” has a strong appeal to clients who have to lay out the money for a project. However, there is a need for caution and thoroughness when a value engineering idea is proposed. There must be an adequate exploration and testing program. Good communication is essential, with understanding by all parties. Any assumptions involved in the value engineering must be confirmed and/or monitored with instrumentation.

It is necessary to remember that “things are not always what they seem,” and “we don’t always know as much as we think we do”. This case history illustrates some of the points made earlier and the results that happened when these points were not borne in mind. This case history concerns a 19-story high rise residential building, comprised of five levels of parking and 14 levels of condominiums, located just south of the Loop in Chicago. The original foundation design consisted of rock socketed and steel cased caissons into sound dolomite. The contractor proposed a value engineering option that consisted of straight shaft caissons, constructed under polymer slurry without casing, to the top of rock, and designed for 100 ksf bearing. The soil profile is shown in Figure 13. It indicates surface sand to a depth on the order of 15 feet (4.6 m), underlain by a thin stiff clay crust, followed by soft to stiff clay, down to a very stiff to hard silty clay till, which extended to a thin boulder till just above the dolomite.

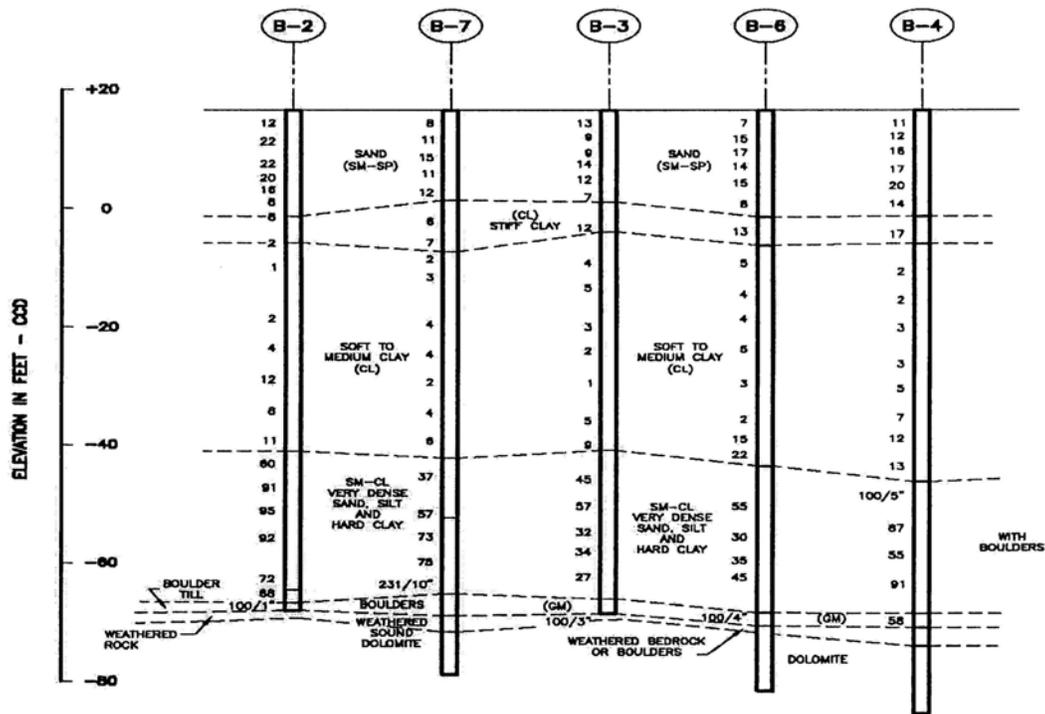


FIG. 13. Soil Profile

The borings performed around the perimeter of the site indicated fairly flat lying soil and rock deposits.

Supplementary exploration with pressuremeter testing to confirm the 4.8 MPa (100 ksf) bearing pressure was also performed around the perimeter of the site since the center of the site was still not accessible. The contractor’s proposed procedure involved drilling under slurry to refusal of earth augers, on the assumption that he would be able to advance the shaft excavation to the surface of bedrock using only earth auger tools. The author met with the caisson inspector on the site and went over procedures with him for the first two caissons. The first two caissons installed were fairly close to existing borings, and the inspector reported that refusal depth for the earth auger was somewhat above the bedrock depth indicated in the borings. The contractor switched to a rock auger, and, with an additional 20 minutes of excavation, was able to advance the excavation to the depth of the bedrock surface indicated in the records of the adjacent borings. The bottoms were cleaned with a cleanout bucket and sounded with a weighted rod so that both the caisson inspector and the author would have a feel for the bottom condition.

Based on the author’s experience from other projects, the revised caisson drilling procedure was accepted and the caisson inspector was advised to ensure that the bottom felt at least as clean as the two that were sounded. The remaining caissons were installed without any noticeable incident until late in the project, when the author again talked with the caisson inspector and was advised that everything went as it did

for the first couple of days except that the rock auger 20 minute refusal elevation seemed to rise over the center portion of the site. Since there were no borings in that part of the site, the inspector just assumed there was a corresponding rise in the bedrock level.

The quality control procedures involved crosshole sonic logging (CSL) of every caisson, and selection of two caissons for coring to confirm both the concrete integrity and the bottom condition. Two caissons were selected in the center of the site, where the highest bottom elevations were recorded. The cores revealed that the bottoms of these caissons were, in fact, about 1.5 m (five feet) above bedrock. Pressuremeter testing in the boulder till indicated insufficient bearing for the 4.8 MPa (100 ksf) design bearing pressure. However, the values were high enough that if adequate allowance were made for side friction taking up part of the load, the total bearing capacity would be acceptable. However, proof of this would require significantly more pressuremeter testing, as well as some type of load testing, since the City of Chicago does not accept side friction without load testing. A number of additional borings with pressuremeter tests were performed, as well as dynamic load testing of three selected caissons.

With all of this data, a spring model was developed for the structure and settlement of the as-constructed caissons was predicted. The main concern was the differential settlement between those caissons that did get to rock and those caissons sitting on the boulder till. The structural engineer had set 10 mm (0.4 in) as the maximum differential settlement tolerable, and the developed spring model indicated that settlements would be less than this, except at one possible location which was marginal. This particular caisson was supplemented with three micropiles. The building was then constructed, and settlement measurements were recorded. The results are shown in Figure 14.

The results appear to indicate that initially, up to construction of about nine floors, the load is taken in friction with very little settlement. Above nine floors, there is a sudden increase in settlement up to about 12 floors and then very little settlement thereafter up to the total 19 floors. It is likely that the sudden increase in settlement from nine to 12 floors where the friction has been exceeded is due both to squeeze or compression of any small sediment left at the base of the shafts, and to tightening up of the underlying boulder till until there is a boulder-to-rock stress path.

The technical conclusions from this case history are given in the paper “A Deep Foundation Surprise, Engineered Response, and Foundation Performance (Baker *et al*, 2004c). However, some additional conclusions are listed here as follows:

1. Things are not always what they seem.
2. Incomplete communication between field and office can lead to major problems.
3. A normally adequate subsurface exploration program may prove insufficient for a changed design based on value engineering.
4. Key assumptions must be verified.

5. A quality control system that includes physical verification of assumptions can find mistakes or defects before foundations are loaded.
6. Active involvement of the geotechnical engineer in evaluating problems and developing successful solutions can reduce the ultimate costs, facilitate equitable cost sharing and avoid litigation.

If a problem develops, give full attention to solving it quickly, not to assigning blame. Cooperative spirit saves everyone money in the long-run. In the foregoing case, the cost was resolved without going to court.

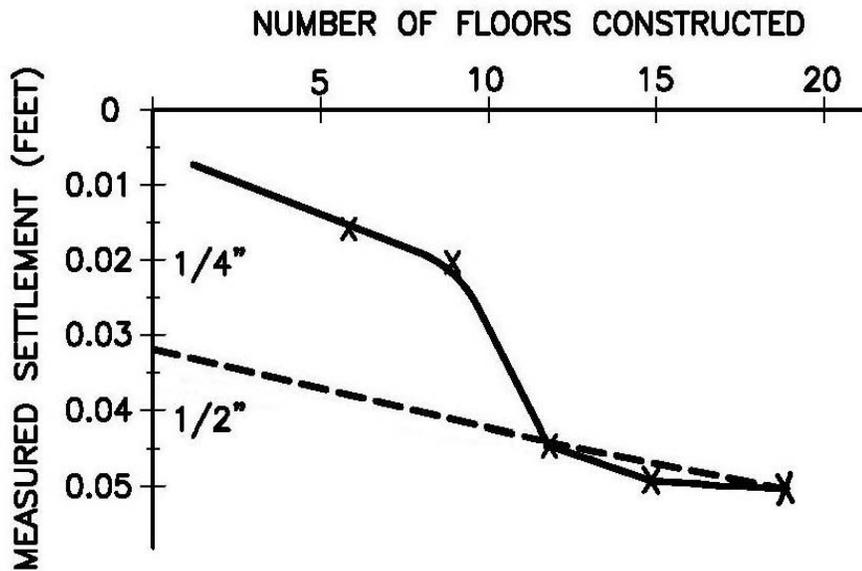


FIG. 14. Average Building Settlement Versus Number of Floors Constructed

On reflection, what were the primary mistakes that were made on this project? Obviously, there should have been at least one boring at the center of the site so that the caisson inspector would have a baseline for comparison with the caisson excavations. The second mistake was not having sufficient ongoing communication with the caisson inspector on a value engineering project of this type, with which the caisson inspector was unfamiliar. On the positive side, everything else on the project worked out well.

Case History No. 5: 111 South Wacker Drive

This case history involves all five categories. 111 South Wacker is a 51-story high rise building completed in 2005, with a structural height of 208 m (681 feet). The foundations for this project were discussed in a presentation to the Deep Foundation Institute conference in Chicago in 2005 (Walton *et al*, 2005). The structure had a full height concrete core and steel exterior frame. The site was formerly occupied by the 18-story US Gypsum (USG) building which was completed in 1962. The building was founded on a forest of belled caissons bearing in a hard silty clay deposit. The

foundation caissons were designed using an allowable bearing pressure of 770 kPa (16 ksf), based on unconfined compression tests. In 1996, the USG superstructure was razed, and the three-level basement was converted to an underground parking structure with ramps used as diaphragms for lateral earth support.

Geotechnical Evaluations

The author's firm had provided construction monitoring and caisson inspection services for the USG foundations in 1961. The historic records included documentation of the as-built caisson dimensions and bearing soil information, as well as the results of concrete compressive strength tests. The author's firm was able to provide the structural engineer and City of Chicago building officials with "real" design properties for all the caissons designated for reuse. As the building design evolved, the structural engineer was able to include, or exclude, existing caissons as required, to support the changing foundation loads.

A limited number of the existing caisson shafts were cored as part of the subsurface investigation program in 2002 to verify the as-built bearing elevations, and to obtain representative concrete samples for strength testing. Because of the amount of reliable information available, it was possible to reduce the scale, and consequently the cost, of the investigation, and to accelerate the design and foundation permitting process for the new structure.

The normal boring data was supplemented with in-situ Ménard pressuremeter tests (PMT) at several locations. The PMT tests were performed immediately below the bells of the cored caissons, and at approximate 1.5 m (5-ft) intervals extending down to bedrock.

The PMT tests indicated that the bearing pressure under the existing USG caissons could safely be increased from 766 to 1,150 kPa (16 to 24 ksf), and that the new caissons, founded in the Chicago hardpan at elevation -65 ft CCD, could be designed for a net allowable bearing pressure of 2,155 (45 ksf). The net limit pressure, creep pressure, and interpolated modulus data are plotted versus elevation in Figure 15. The units are shown in tsf, which is roughly equivalent to kg/cm^2 . The bearing elevation for the new caissons is indicated by the dashed line. The typical design envelope for settlement calculations is depicted on the deformation modulus plot.

Overlap of Caisson Bearing Pressures

As shown in Figure 16, the reinforced concrete building core is supported on an approximately 30.5 m (100-ft) square mat, by 12 new large-diameter belled caissons and 22 existing USG caissons. Because of the unique building architecture, the perimeter building loads for the entire 51-story structure are transferred to the foundation by two exterior columns on the sides facing Monroe Street to the north, the public alley to the south and South Wacker Drive to the west. These non-redundant columns are supported by either a single new caisson, or a combination of reused and new caissons connected with grade beams.

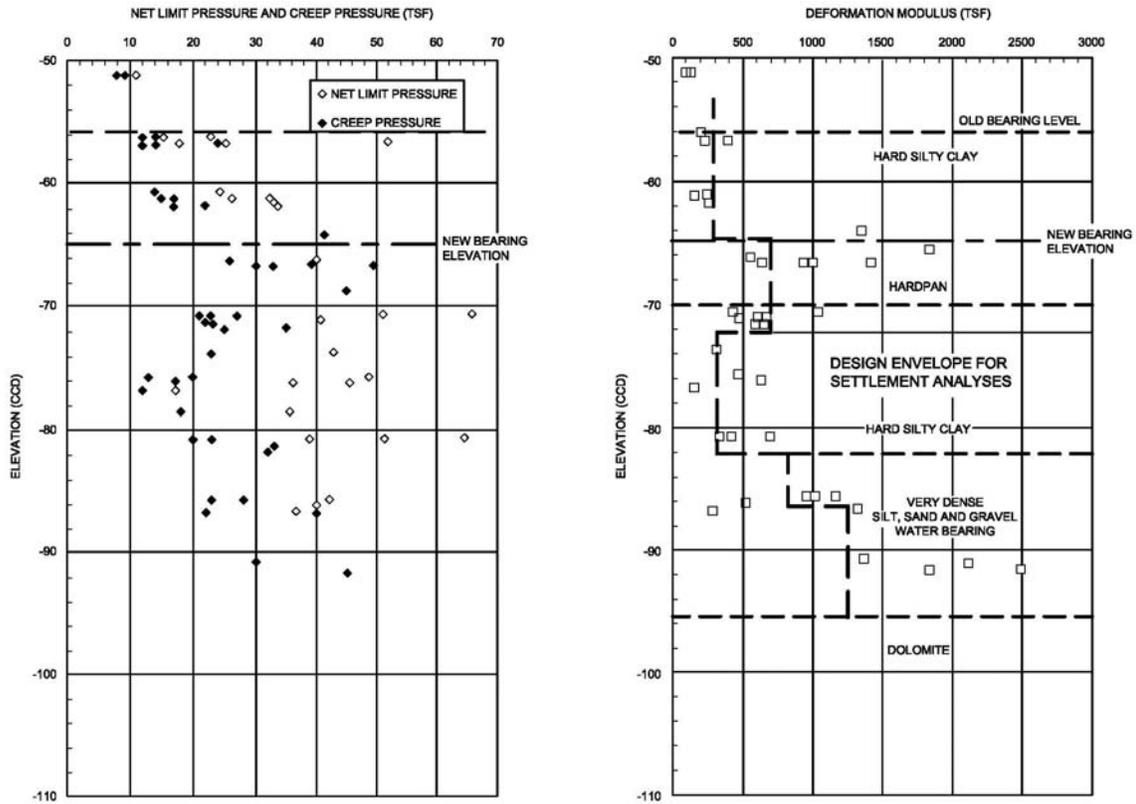


FIG. 15. Pressuremeter Test Profile

For the proposed layout and the maximum caisson loads provided by the structural engineer, the increase in bearing pressure beneath the new caissons, due to the surrounding existing caissons, was determined using Boussinesq theory.

The vertical stresses from the Boussinesq analyses were used to estimate elastic foundation settlements. Both classic elastic theory and Ménard's method were applied at representative locations. The core mat and perimeter caisson foundation settlements were predicted to be less than 25 mm (1 inch).

Vertical soil springs were developed for the structural engineer, based on the settlement predictions. As part of their design, the structural engineer performed a finite element analysis of the building core mat, and an integrated superstructure and foundation frame analysis.

Corrective Action

Unanticipated water problems were encountered at two caissons, D.5-5A and B.5-5A. The corrective measures used are described in detail in the reference paper. Caisson D.5-5A was supplemented with 14 rock socketed micropiles because of excessive water seepage and concern about completed bell bottom softening. Caisson B.5-5A was re-drilled as a steel-cased rock socketed caisson, as bell construction was not possible due to excess water inflow and cave-in.

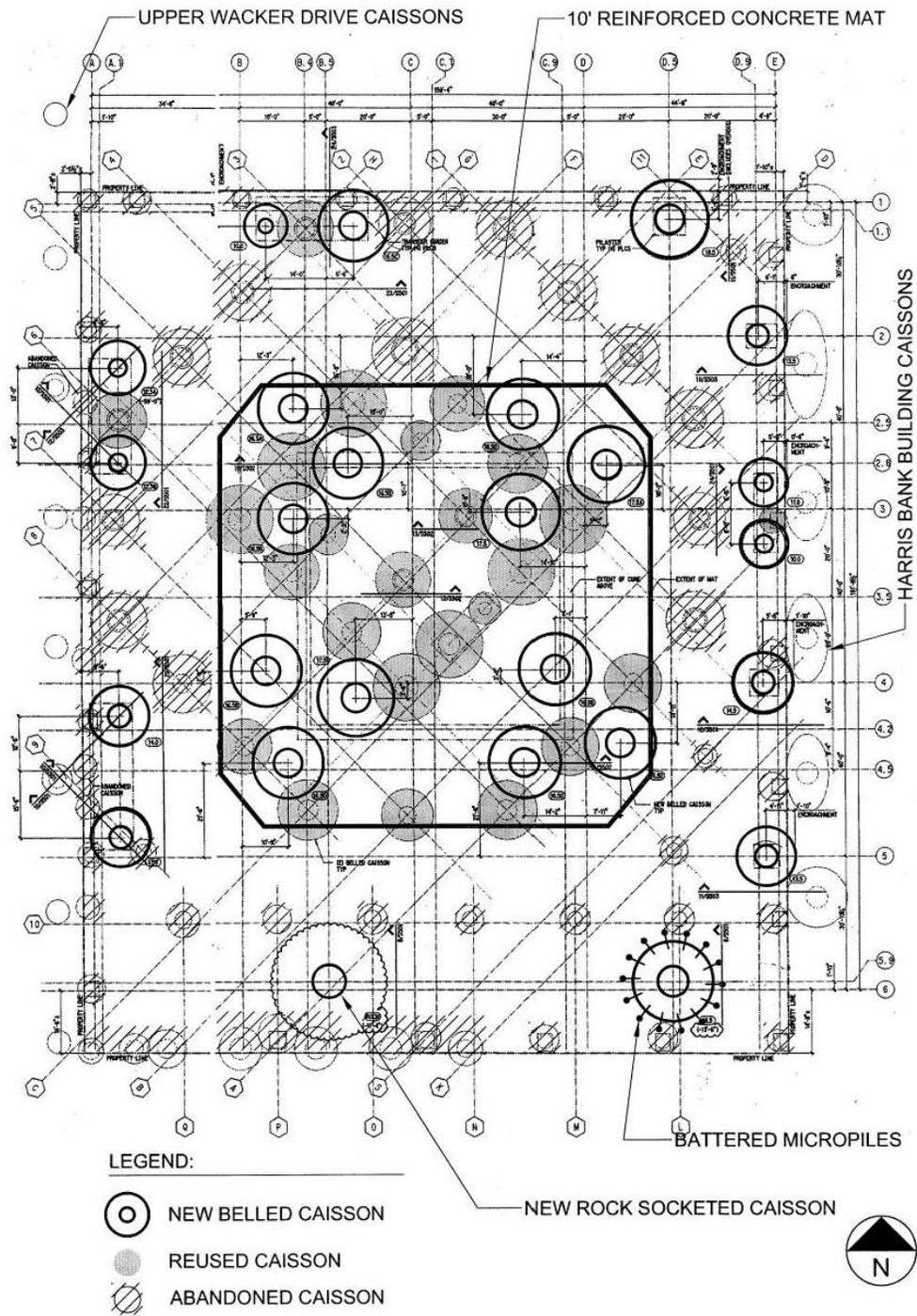


FIG. 16. Foundation Plan for 111 South Wacker Drive

Performance Monitoring

Because the micropiles installed at Caisson D.5-5.A were intended to reduce

settlement, rather than act as primary foundation support members, load testing was not performed. Instead, with the consent of the owner and City of Chicago building officials, building settlements were monitored to demonstrate design adequacy. Survey points were established on top of the two repaired caissons and the core mat. Readings were taken periodically during construction by the building contractor. Readings were terminated when the building reached its design height under sustained dead load conditions. The settlements measured at the end of monitoring are summarized in Table 4 for various key building elements along with the predicted settlement. Elastic theory typically over-predicted settlement but Ménard’s method was close.

Table 4. Summary of Predicted and Surveyed Settlement for 111 S. Wacker

<i>Location</i>	<i>Element Geometry</i>	<i>Dead & Live Load DL + LL</i>	<i>Est. Settlement DL + 0.5LL</i>	<i>Measured Settlement</i>
<u>Elastic Theory</u>				
Core Mat on 34 Caissons	100' x 101.5' (30.5 m x 31 m)	135,360 kip (602,110 kN)	0.75 to 1.0 in (1.9 to 2.5 cm)	0.5 in (1.2 cm)
Isolated Caisson (B.5-5.9)	Converted to 7.5' (2.3 m) Dia. Rock Socketed Caisson	15,000 kip (66,725 kN)	0.5 to 0.75 in (1.2 to 1.9 cm)	0.88 in (2.2 cm)
<u>Ménard’s Method</u>				
Core Mat on 34 Caissons	100' x 101.5' (30.5 m x 31 m)	135,360 kip (602,110 kN)	0.5 in (1.2 cm)	0.5 in (1.2 cm)
Isolated Caisson (D.5-5.9)	~20' Dia. Belled Caisson Retrofitted with 14 – 9.625” Dia. Micropiles	11,800 kip (52,490 kN)	0.75 to 1.0 in (1.9 to 2.5 cm)	0.63 in (1.6 cm)
NW Column on 3 Caissons	12.5' x 32.5' (3.8 m x 9.9 m)	11,110 kip (49,420 kN)	0.75 to 1.0 in (1.9 to 2.5 cm)	0.63 in (1.6 cm)
N Column on 3 Caissons	13.0' x 33.0' (4 m x 10 m)	14,970 kip (66,590 kN)	0.75 to 1.0 in (1.9 to 2.5 cm)	0.81 in (2.0 cm)

Risk versus Cost Evaluation

What were some of the risks versus cost savings considered on this project? The risk of re-using existing caissons at a higher capacity was considered manageable, since all of the as-built records were available, and the author was involved in the original design and construction. Considering the dense packing of the existing belled caissons, the only foundation alternative would have been to fully support the building on new rock caissons. Preliminary pricing indicated that the project would not be

financially viable if this option was required.

The risks were considered small compared to the cost savings, so the project went forward as designed.

Case History No. 7: Petronas Towers, Kuala Lumpur, Malaysia Revisited

The Petronas Towers have been well described in previous publications by the author and others (Baker *et al*, 1994, Baker *et al*, 1998b), and do not really fit particularly well in any of the five categories listed. However, they do fit using engineering judgment when geotechnical truth is uncertain and the final design does include the mixed foundation concept (mat on variable length barrettes) and a remediation plan for the karst limestone sub-foundation.

The Petronas Towers are believed to have the world's deepest building foundations. The Petronas Towers barrette foundations extend to a maximum depth of 130 m (426 ft) below grade in soil and weathered rock, plus ground improvement cement grouting to depths up to 162 m (531 ft). Thus, measured from the bottom of the deepest foundations to the top of the building, Petronas Towers would measure either 582 m (1909 ft) or 614 m (2014 ft) depending upon whether the ground improvement was considered part of the foundation system.

Soil and Bedrock Conditions

A generalized soil and bedrock profile below the towers is shown in Figure 17. The geologic profile consists of 12 to 20 m (39 to 66 ft) of medium dense, silty and clayey alluvial sand. The alluvium is underlain by a medium dense to extremely dense, sandy and gravelly silt and clay material which is a residual soil and weathered rock deposit known locally as the Kenny Hill Formation. The bedrock below the Kenny Hill is of Silurian age and consists mainly of calcitic and dolomitic limestone and marble. The rock surface is very irregular and has been weathered by solution activity creating numerous joints and cavities. As a result of the solution activity, isolated zones of the Kenny Hill have eroded into the bedrock cavities creating soft or loose zones referred to as slump zones. The hard Kenny Hill material above arches over these slump zones, so they do not feel the full weight of the overlying formation.

The rock surface dips steeply from northwest to southeast, such that the tower bustles are situated over bedrock located 80 to 90 m (260 to 295 ft) below street grade. The towers themselves are situated with rock at 100 to 180+ m (330 to 590+ ft) below street grade. As shown in Figure 17, there is also a valley feature in the bedrock surface between the towers, extending deeper than 200 m (658 ft).

Foundation Requirements

Due to the height, slenderness and structural interconnection of the towers, the developer and the designer aimed for predicted differential settlement as close to zero as practical - about 13 mm (less than 1/2 in) across the base of the towers.

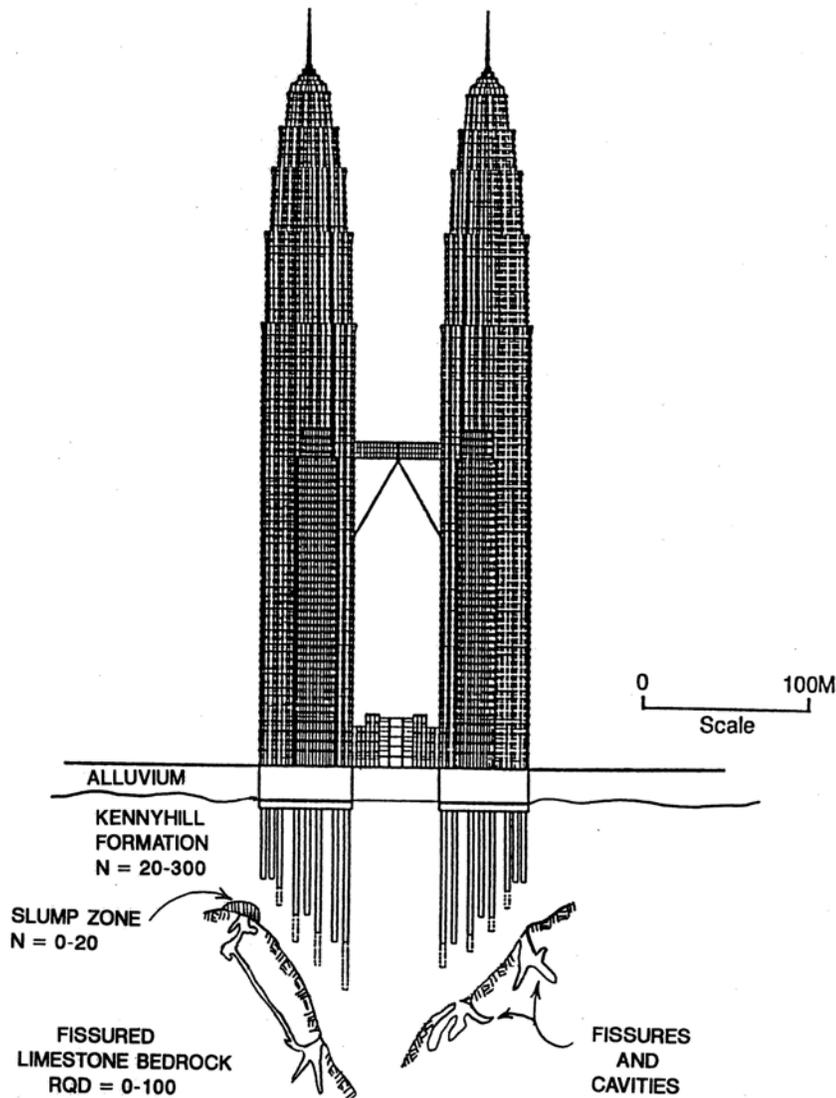


FIG. 17. Tower Foundation Profile

With the anticipated geology and the goal of minimizing differential settlement, foundation alternatives studied included a “floating” raft, a system of bored piles socketed into limestone below any significant cavities, and a raft on friction piles located in the Kenny Hill formation well above the limestone, with grouting of cavities and slump zones as necessary, and with pile lengths varied to minimize differential settlement. The large size and great strength and stiffness requirements of a “floating” raft precluded its use. The great depth to bedrock made socketed bored piles impractical. Therefore, the friction pile scheme was used. During the preliminary design and soil exploration phase, it was found that bedrock elevation at the initial tower locations varied so greatly that rock actually protruded into the proposed basement on one side of the tower. This made control of differential settlement impractical. The tower locations were then shifted approximately 60 m (196.9 ft) to where the thickness of the Kenny Hill formation was sufficient to support

a raft on bored friction piles. There the required differential settlement limitation could be achieved by varying the length of piles or barrettes. A representative Standard Penetration Resistance profile is shown in Figure 18.

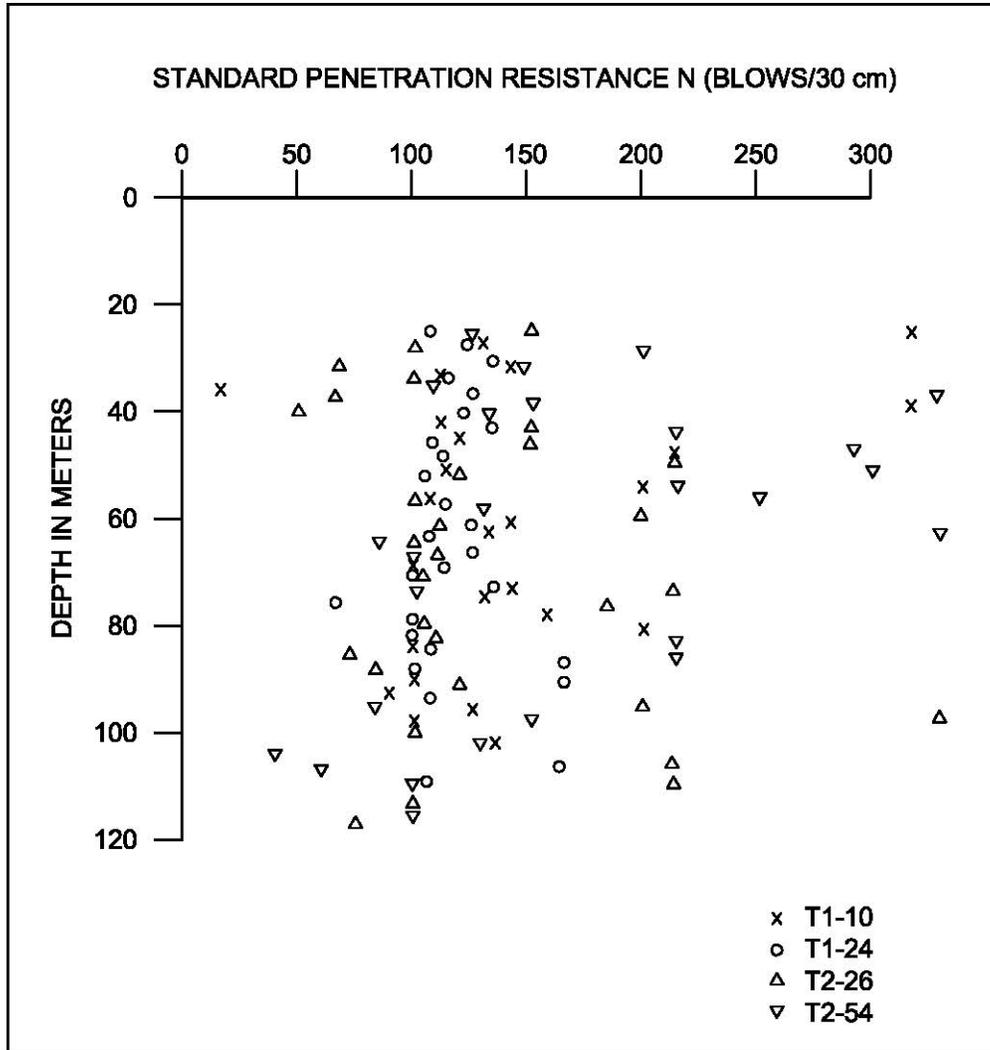
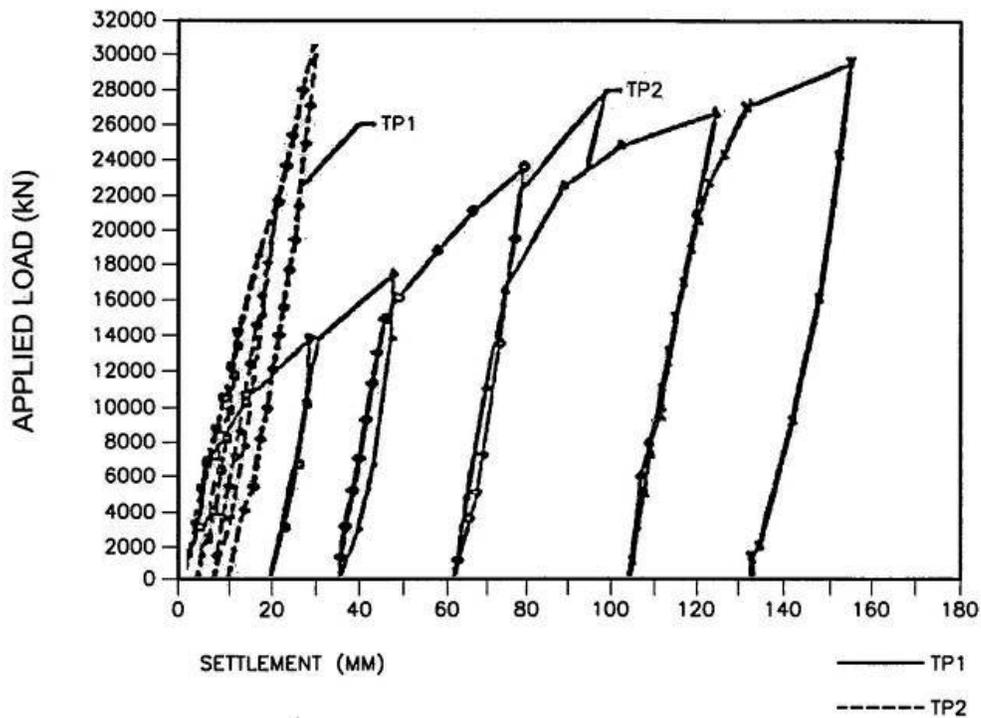


FIG. 18. Standard Penetration Resistance Profile

Load Test Program

Kentledge load tests were performed, using house-high blocks of concrete to provide the reaction. The results of the load tests are shown in Figure 19. Both test piles were 75 meters (245 feet) long and constructed under bentonite slurry. One test pile was post-grouted to break through any filter cake development. Further details are in the reference papers. The post-grouted pile could not be failed with a maximum developed side friction of 330 kPa (6.86 ksf). This led to a recommended design value of 110 kPa (2.28 ksf), and a requirement that all piles be similarly post-grouted.



APPLIED LOAD vs SETTLEMENT
TEST PILE 1 AND TEST PILE 2

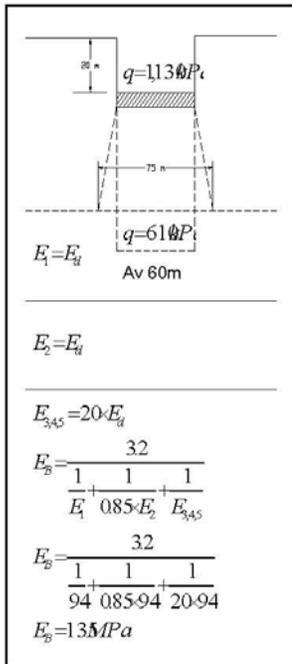
FIG. 19. Applied Load vs Settlement Test Pile 1 and Test Pile 2

Settlement Analysis and Assumptions

Settlement analyses were performed, using the equivalent footing method and simple hand calculations, as shown on Figure 20. Extensive settlement analyses were also performed, utilizing the SAP 90 program and the Plaxis 3-D program, using soil modulus estimates based on back calculation from the test pile program, and from averaging the reload modulus slopes of the in-situ pressuremeter tests. Pile lengths were varied until calculated maximum differential settlement goals were achieved.

Based on bearing capacity considerations only, barrette lengths of 33 m (108 ft) would have been sufficient to support the design loads, but final pile lengths under the main towers varied from 40 m (130 ft) to 105 m (345 ft), based on settlement considerations. Figure 21 shows the FEM predicted settlement, and ground deformation for the final design case from Baker *et al* (1994).

Calculated average settlement, from the equivalent footing method and average uniform conditions, ranged from 41 mm (1.6 in) using the Ménard rules, to 73 mm (2.9 in) using elastic theory. This brackets the computer-generated values, using actual pile length and rock slope geometry.



Pressuremeter Data

$$E_{d,Av} = 94.3 \text{ MPa}$$

$$E_{Av}^+ = 267 \text{ MPa}$$

$$\alpha = \frac{E_d}{E^+} = 0.35, \text{ Use } 0.4$$

Settlement Calculation – Menard Empirical Method

$$s_{Menard} = \frac{1.33}{3 \times E_B} q R_0 \left(\lambda_2 \frac{R}{R_0} \right)^\alpha + \frac{\alpha q \lambda_3 R}{4.5 E_1}$$

$$\lambda_2, \lambda_3 = 1 \text{ for a circle}$$

$$R_0 = 30 \text{ cm}$$

$$s_{Menard} = 0.55 \text{ cm} + 2.16 \text{ cm} = 27.1 \text{ mm}$$

Settlement Calculation – Elastic Theory

$$s_{Elastic} = \frac{\mu_0 \mu_1 q B}{E}$$

$$s_{Elastic} = \frac{0.35 \times 0.92 \times 6,100 \times 75,000}{250,000} = 59 \text{ mm}$$

Elastic Compression of Shaft Down to Equivalent Footing Level

$$\Delta \ell = \frac{\sigma L}{E_{conc}}$$

$$\sigma = \frac{2,680,000 \text{ kN}}{82 \times 1.2 \times 2.8} = 9,727 \frac{\text{kN}}{\text{m}^2}$$

$$E_{conc} \cong 27,000,000 \text{ kPa}$$

$$\Delta \ell = \frac{9,727 \times 40,000}{27,000,000} = 14.4 \text{ mm}$$

Total Predicted Settlement

By Menard Empirical Method

$$S = s_{Menard} + \Delta \ell$$

$$S = 27.1 \text{ mm} + 14.4 \text{ mm} = 41.5 \text{ mm}$$

By Elastic Theory

$$S = s_{Elastic} + \Delta \ell$$

$$S = 59 \text{ mm} + 14.4 \text{ mm} = 73.4 \text{ mm}$$

FIG. 20. Equivalent Footing Method for Settlement Calculation, Petronas Towers

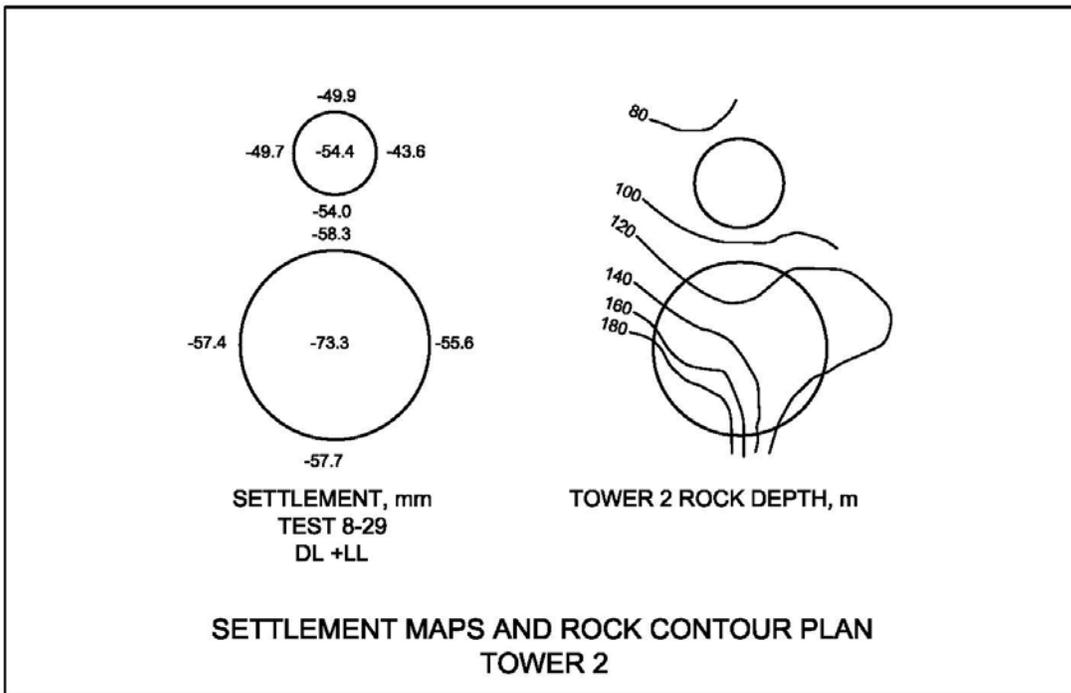
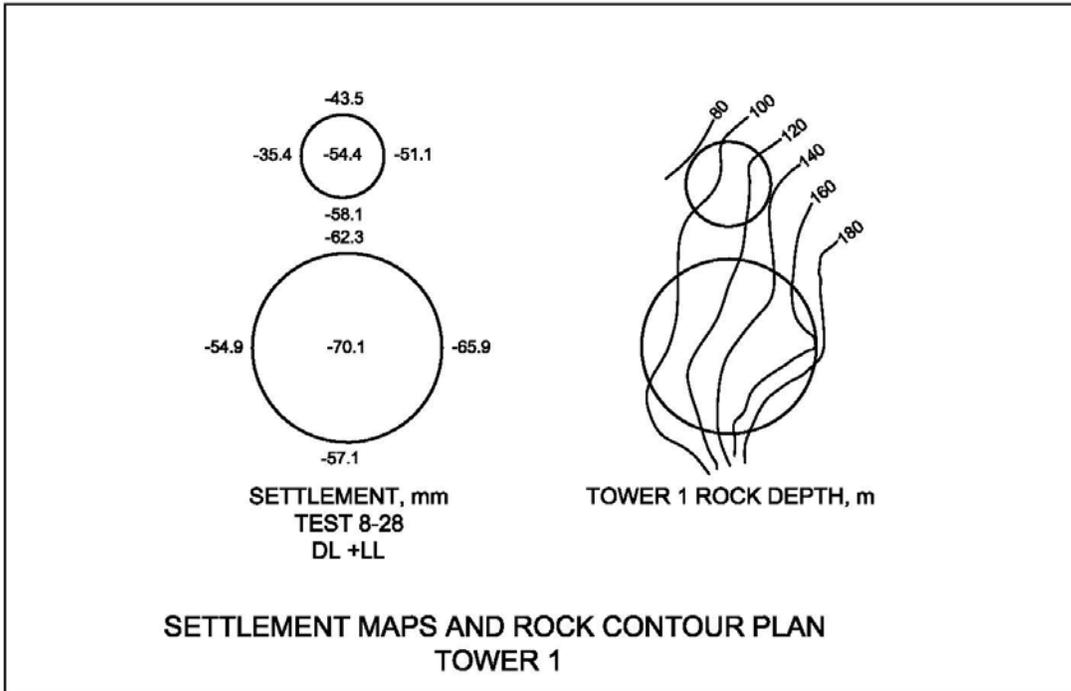


FIG. 21. Predicted Settlement based on FEM Procedures

Details of the soil property information obtained, design parameters developed, and settlement analyses performed, are given in Baker *et al* (1994).

Required Ground Improvement, Foundation Installation and Instrumentation

The boring and probing program uncovered a number of significant cavities in the limestone and slump zones at the limestone interface beneath the tower footprints. There was concern that there may be unpredictable future settlement unless these zones were treated. The goal was to fill the voids in the limestone to make it relatively incompressible, and to improve the slump zone areas so that they could be considered to act similar to the intact Kenny Hill formation. This could be considered remediation in advance, and the judgment versus risk question was ‘How deep into the limestone should the cavities be filled with grout?’ Details of the grouting program, foundation installation and instrumentation program are described in Baker *et al* (1998b).

To stay conservative, a judgment-based decision was made to grout to a depth below which the net building stress would be less than 5% of the overburden pressure.

Performance Evaluation

Predicted maximum settlement for the completed towers was about 70 to 73 mm, (2.8 in), with maximum differential across the mat of 11 mm (0.5 in). Based on settlement measurements taken during construction, it appears that both total and differential settlements of the towers were less than predicted, indicating that the goals of the deep ground improvement program were met.

The time/settlement record through completion of Tower 1 and partial occupancy up to March 19, 1997 is shown in Figure 22. The maximum reported average settlement for the core is about 35 mm (1.4 in) with maximum reported differential settlement of 7 mm (1/4 in). This is approximately half of that predicted in Baker *et al* (1998b), where a maximum settlement of 73 mm and differential settlement of 12 mm was predicted, using finite element analyses based on an assumed modulus for the Kenny Hill formation of 250 MPa (2610 tsf). As depicted in Figure 20, the predicted settlement, following the Ménard rules and equivalent footing method, is only slightly more than that experienced to date, which is 41 mm vs. 35 mm (1.6 in vs. 1.4 in).

From the less than anticipated differential settlement, it appears that the mat, barrettes and soil between the barrettes are acting as one massive block, with the barrettes serving to knit the mass together.

In evaluating the foundation design and performance, the question needs to be asked as to why the settlement is only approximately one-half of that predicted by elastic theory, when extensive in-situ testing was performed including two full scale instrumented load tests and 260 in-situ pressuremeter tests. In the original paper, (Baker *et al*, 1998b) the explanation was given that the erratic very hard layers were more effective in carrying load without deformation than that calculated using proportional modulus assumptions. However, there are other possible explanations for the observed settlement behavior at Petronas Towers.

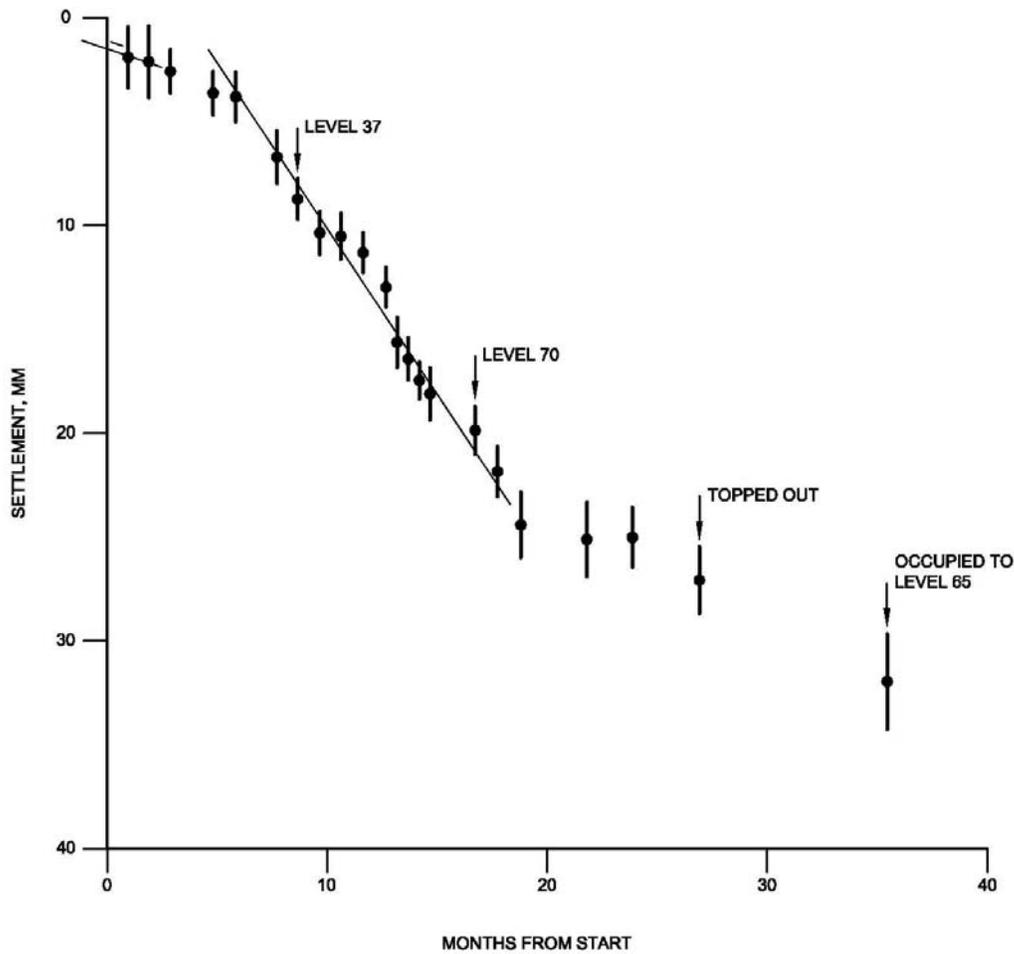


FIG. 22. Time/Settlement Record through Completion of Tower 1

It should be noted that correlation of prediction and performance would be improved if the prestressing effect of the barrette installation from the four-meter level, with the basement level at -20 m (65.6 ft), had been considered in making the prediction. Sixteen meters of soil excavation depth represents approximately 25% of the weight of the building. If this weight had been omitted, the predicted settlement would have been proportionately less.

The possible prestressing effect assumes that the effective composite soil and concrete block modulus is stiffer on initial unloading, compared to initial loading. This appears to be true, based on the pressuremeter tests.

It is also possible that the high friction values indicated in the load test data, and the sedimentary nature of the weathered rock, resulted in faster stress spread with depth than was obtained by assuming Boussinesq elastic conditions. If a rigid block analysis

is assumed, with developed perimeter block friction equal to the maximum load-test value divided by a 1.5 factor to allow for some creep, the calculated settlement including composite block compression coincidentally works out to a total average settlement of 35 mm (1.4 in), or very close to what was observed.

Another possibility is that the non-linear nature of the soil is not adequately expressed by the elastic model used and that the actual soil is stiffer at smaller strains, leading to less settlement compared to using an assumed average modulus.

A Foundation Cost Saving Idea

Since settlement performance was so good, and significantly less than design criteria, the question that can be asked is whether a major foundation cost saving would have been possible if the piles or barrettes had been considered to be settlement reducers, where higher than code allowable concrete stresses could be tolerated, rather than as structurally required foundation elements that must meet code criteria. In considering this concept, the first thing to note is that reduction in the variable pile length design would not have been possible, since the differential settlement predicted by finite element analysis just barely made the design criteria.

However, quite possibly the size of the piles or barrettes could have been decreased, for a large savings in concrete and drilling or excavation costs. The resulting increase in concrete stresses might be acceptable, but the increased elastic shaft compression might not. For this very critical design and level of geotechnical uncertainty, it is likely that engineering judgment would have precluded use of this money saving concept on this project, but perhaps not on the next, with this now well-documented case history.

CONCLUSION

Innovative cost saving foundation design for high rise buildings was achievable, with acceptable risk, in the case histories selected for discussion in this paper, even though geotechnical truth was uncertain. The recommended guidelines given earlier in this paper were generally followed and, in the author's opinion, helped achieve satisfactory foundation performance. The key requirements were:

- Geotechnical engineer and structural engineer teamwork.
- Knowledge of and experience with local geology and past building foundation performance history, and, where any of these were not available, being more conservative.
- Experience with in-situ pressuremeter testing for predicting ground deformation under load in a given geology.
- Having a plan for addressing geotechnical surprises.
- Being more conservative when assumptions cannot be verified.

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Professor Kenji ISHIIHARA



Professor Kenji Ishihara graduated from the University of Tokyo in 1957. After a period of research at the University of Illinois under the advice of Professor R.B. Peck, he returned to the post of Associate Professor at the University of Tokyo in 1977 he took up the position of Professor of Civil Engineering and remained at the University of Tokyo for 40 years. During this time he was active in the International Society for Soil Mechanics and Foundation Engineering (ISSMFE). He was particularly effective in advancing the work of Technical Committee TC4 – Earthquake Geotechnical Engineering and in establishing the new discipline of earthquake geotechnology. His work with ISSMFE culminated in his appointment as President of the Society for the term 1997-2001. On his retirement from the University of Tokyo in 1995 he took up the post of Professor of Geotechnical Engineering at the Tokyo University of Science and then at Chuo University since 2001.

Professor Ishihara was the 1993 Rankine Lecturer delivering “Liquefaction and Flow Failures During Earthquakes” and the Terzaghi orator at the time of the 14th International Conference on Soil Mechanics and Geotechnical Engineering in Hamburg in 1997. He was the recipient of H.B. Seed Medal in 1998 from A.S.C.E. and several other eponymous lectureships. He received the Japan Academy prize in 2000 and was elected in 2010 to a Foreign Associate of the United States Academy of Engineering. In 1996, he published his textbook “Soil Behaviour in Earthquake Geotechnics” (published by Oxford University Press), which summarizes his life’s work. This book was translated in Russian in 2006.

Forensic Diagnosis for Site-Specific Ground Conditions in Deep Excavations of Subway Constructions

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ABSTRACT:

In an attempt to address important but unheeded issues of soil conditions associated with deep subway construction, three cases of large-scale collapse that occurred in recent years are taken up for consideration. One of them is the failure of 31m deep open excavation in Singapore which was underway for subway construction by utilizing the diaphragm wall-strut retaining structures. Amongst several reasons, unaccustomed presence of buried valley was pointed out in the forensic investigations. Thus, shortage of available information regarding deep-seated soil strata was found to be fatal for making conservative design and safe operation of the construction. Thus, scrutiny of ground conditions by closely spaced boring and logging and precise soil property characterization were a reminder to be seriously recognized as a lesson of supreme importance. The second case in this study is the large-scale cave-in at the site of subway construction in Kaohsiung, Taiwan, which occurred as a result of piping in a silty sand deposit at the bottom of the excavation. In view of the circumstances that, nowhere has the piping been observed more conspicuously than in the deposits in Kaohsiung, particular nature of the local silt was suspected to be a cause leading to the occurrence of the piping. The results of pinhole tests and specific surface measurements revealed peculiar characteristics of the local silt which is highly vulnerable to internal erosion subjected to seepage flow. Thus, as a result of forensic type study, the importance of exploring unknown nature of local soils was recognized as an important lesson to be paid attention particularly when construction projects are to be executed in regions of few experiences of deep excavations. The large-scale cave-in which occurred at the subway construction site in Shanghai is taken up as the third example of forensic consideration. The incident is reported to have occurred as a result of mal-functioning of equipments when the subway construction was going on at a depth of about 30m. Although de-tailed information was not made available, it is considered likely that shortfall of investigations on the nature of locally prevalent sandy silts might have been an underlying reason behind the occurrence of unfortunate distress in Shanghai.

1. INTRODUCTION

The term Forensic Engineering is a relatively new word which has become known in the area of geo-technical engineering during the past 30 years. From the cases of this word being used, it is vaguely understood that its main aim is to unearth facts associated with occurrence of accidents or damages and to address them in a simple yet rational manner so that even non-professionals could understand and exercise a fair judgment. The outcome of the forensic investigation is often submitted to the court of law to make a decision or sentence by a judge. The needs for the forensic investigation in geotechnical

engineering have arisen from frequent occurrences of trouble or dispute associated with shortcomings or defects in land development for residential use or accidents during construction works (Day, 1998). In the discipline of the geotechnical engineering, the state-of-the-art knowledge currently available is generally incorporated in the design and practice of construction. However, there are always shortcomings in many facets of soil investigations and misinterpretation of their outcome. Although small troubles are of occasional occurrence, construction works are executed in general without any vital mishap. However, incidents or collapse of fatal importance could occur sometimes inevitably in the foundation works involving deep excavations. Detailed investigations are carried out after the incident from the forensic standpoint. In the course of such investigation, various aspects of geotechnical importance hitherto unknown are unearthed regarding site specific soil characteristics, soil-structure interaction, detailed process and control of construction.

The experiences of calamity as above leave important lessons to be profoundly remembered and to be reflected on future development of the state-of-the-art. However, it has been a general inclination that these precious experiences were neither exposed to eyes of professionals nor well-documented. In fact, to the authors' knowledge, there has been no single case of the big accidents associated with subway construction that was reported in the long list of literature in the discipline of geotechnical engineering. In view of the circumstances as above, an attempt was made in this paper to collect as many information as possible from the investigation of big incidents that occurred in recent years and to arrange them in a precise manner focusing on unforeseen phenomena and circumstances existing in local soils and ground conditions.

2. COLLAPSE OF BRACED EXCAVATION IN SINGAPORE

2.1 Outline

At a site of braced excavation for a cut-and-cover tunnel construction in Singapore, a large scale collapse occurred on April 20, 2004 over the length of 100m section of a carriage way called Nicoll Highway which is located in the southern coast of the Singapore Island as shown in Figure 1. A closer map of the location is shown in Figure 2. The construction south of Nicoll Highway Station was underway involving deep braced excavation flanked by diaphragm walls. The depth of excavation at the time of the failure was 30.8m. The diaphragm wall 0.8m thick was braced by the steel struts with the help of walers and splays. The collapse took place when excavation was underway below the 9th step of struts already in place.

As a result, the soil mass moved towards the excavated space involving large slides on both sides accompanied by the subsidence in the surrounding area. The site is underlain by thick deposits of soft marine clay to a depth of about 35m. Following the incident, detailed studies have been made including borings, sounding, sampling, lab. testing and numerical analyses simulating conditions at the time of the failure. The outcome of the studies was reported by Tan (2006), Yong (2006) and Yong and Lee (2007). Although the probable causes of the incident were pointed out, the present paper will focus on one of the likely causes which is associated with poor understanding of soil conditions in the extremely soft soil area in Singapore.

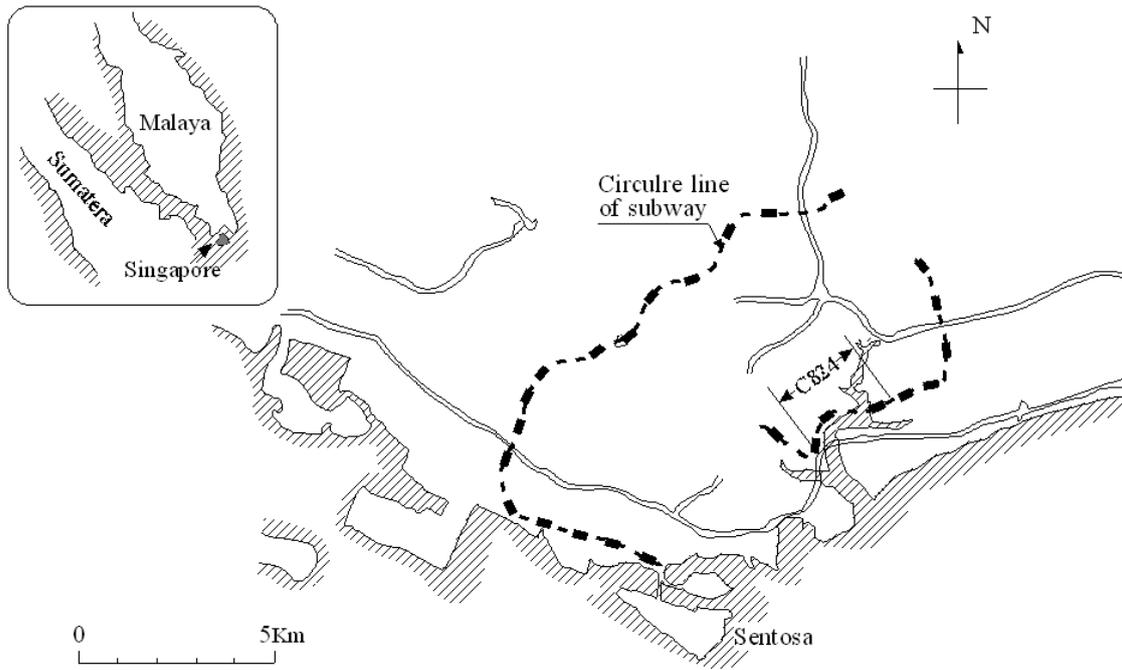


Figure 1. South part of Singapore

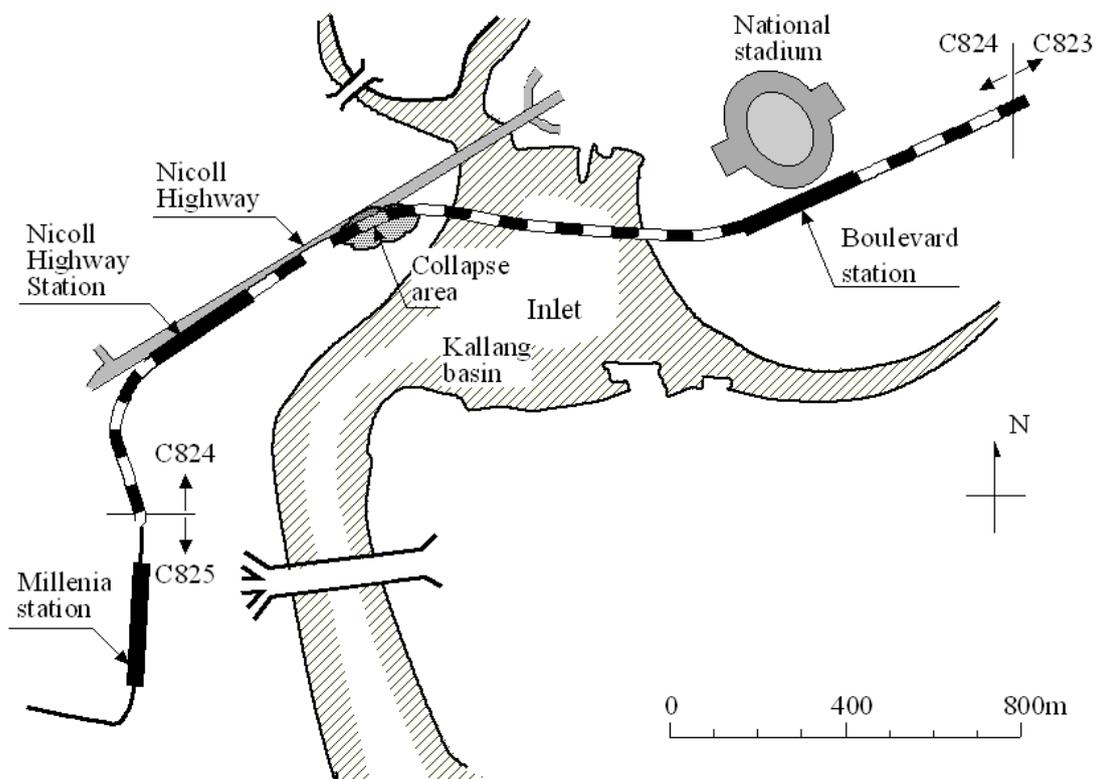


Figure 2. Place of the collapse in the subway construction line



Figure 3. Oblique view of the site to the east after the collapse

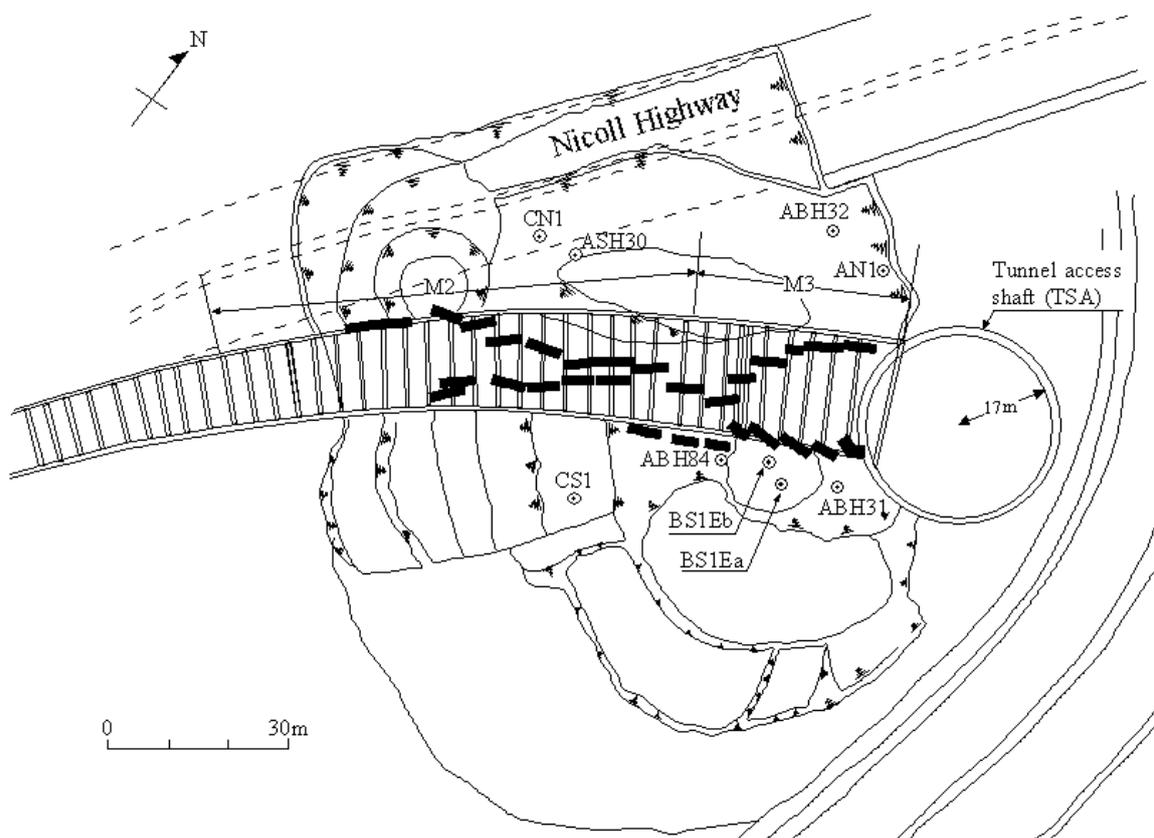


Figure 4. Plan view of the area of the failure and braced wall system

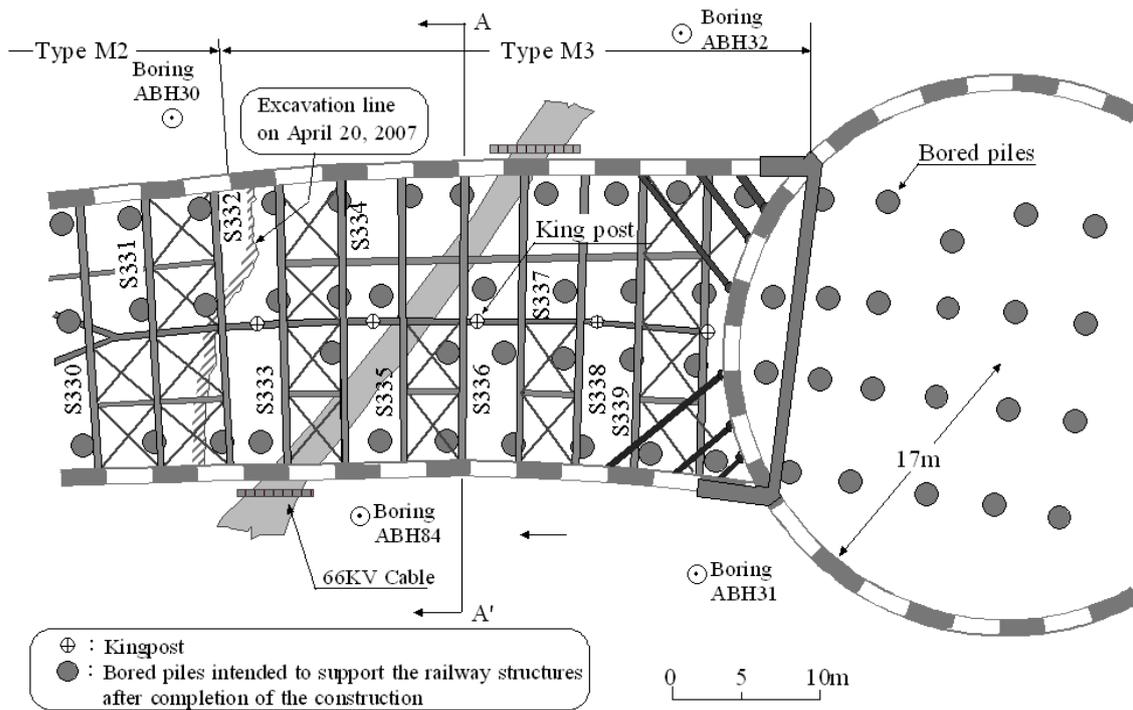


Figure 5. Plane view of the Type M3 section due west of the access shaft

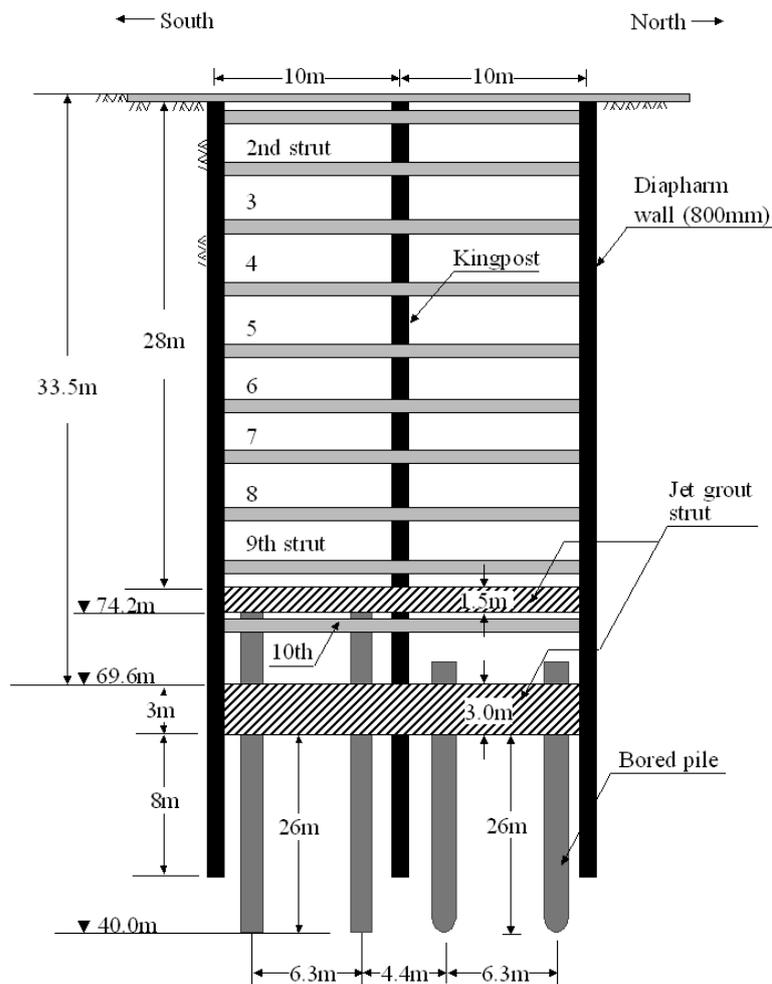


Figure 6. A typical side view of the strutted system for the A-A' cross section

2.2 Collapse of the strut-supported excavation

A section of temporary retaining wall structure adjacent to Nicoll Highway collapsed almost suddenly within a matter of a few minutes at 3:30 p.m. on April 20, 2004. An overview of the site of failure is shown in Figure 3. This is located at the seaward side of the land reclaimed around 1970. The features of the braced excavation and collapse are roughly shown in the plane view of Figure 4 where it is noted that a large tunnel access shaft (TSA) 17m in radius had been constructed to facilitate transport of structure elements and materials down to the floor of the excavation.

2.3 Method of construction

The section of the cut-and-cover tunnels were under construction by means of the bottom-up method with two lines of diaphragm walls supported by strut-waler-splay system. The thickness of the diaphragm wall was 1.0m at the location of cable crossing and at the end section next to the access shaft. Otherwise the thickness was 0.8m. One element of the wall each 3 to 6m wide was excavated in slurry by means of a basket-type digging machine in two or three bites. After lowering the reinforcement cages, concrete was poured in by Tremic pipes. Joints of neighboring elements of the wall was sealed against water penetration, but not rigidly connected as structures. The diaphragm walls, over the section Type M3, were for the curved stretch closest to the access shaft and constructed in May 2003. The walls were designed to be extended 3m into the stiff base layer called Old Alluvium (OA-layer). A detailed plan view of the M3 section is shown in Figure 5 and a typical side views of the strutting system for the cross section A-A' is displayed in Figure 6.

The diaphragm walls were supported by the 10 levels of steel struts with discontinuous walers except at the top strut. To reinforce the strutting action, two soil layers at the depths of 28 to 29.5m (sacrificial strut) and of 33.5m to 36.5m were stabilized as shown in Figure 6 by means of the jet grouting which had been installed from the ground surface before the excavation began. At the same time, with an aim of providing support for the railway tunnel structures to be placed later, four lines of bored piles each 1.4 to 1.8m in diameter were installed from the ground surface at a spacing of 4m or 6m as shown in Figure 5. The depth of the pile embedment was 34.2m on the north side counted downwards from the elevation of 74.2m RL and, it was 29.6m RL on the south side from the elevation of 69.6m RL as shown in Figure 6, where RL (Reduced Level) means a local measure of elevation with a zero point taken at a depth 100m from the mean sea level.

2.4 Situations around the time of the collapse

Excavation with strutting had gone without any hitch until the 9th strut was installed. It was envisioned at this stage that the entire lateral load near the bottom of the excavation was surely transferred to the 9th strut. Then, the excavation to the 10th level of strut was started, thereby taking out the jet-grouted portion 1.5m thick. At this stage, the struts located in this stretch were S333 to S340 with a horizontal spacing of about 4m. The width of the excavation in plan was about 20m and the depth was 30.8m. When the excavation had progressed to the location below the strut No. S332 as indicated in Figure 5, the collapse started to occur. That was at 3:33 p.m. on April 20, 2004. The pictures after the collapse, at 3:34 p.m. and at 3:41 p.m. are displayed in Figure 7. These photos looking over to the east

were taken from the window of the apartment building just north of the Nicoll Highway.

In the morning of April 20, from 9:00 a.m. to 10:00 a.m. engineers on the spot is said to have heard sounds of metallic collision near the north side of the strut S338 and observed the inner flange of H-beam waler at the 9th strut having buckled and shifted about 5 to 6cm downward. This shift aggravated to about 20cm by 2:30 to 2:45 p.m. It was also witnessed around this time that the inner flange of the waler at the 9th strut S335 on the south side had also buckled. Between 2:00 p.m. and 3:00 p.m. the sounds of metallic collision were heard more frequently and eventually the collapse took place at 3:33 p.m.



Figure 7. Photos showing the incident at 3:34 p.m. and 3:41 p.m.

2.5 Conceived causes of the failure

After the occurrence of the collapse, the Committee of Inquiry (COI) was established and probable causes to were investigated and pointed out. In May 2005 the COI report was released, thereby indicating the main reasons as described below.

- (1) Under-design of the diaphragm wall using an inappropriate method of analysis.
- (2) Under-design of strut-waler connection, particularly in the curved section due west of the SA shaft.
- (3) Incorrect back analysis and problems with instrumentation and monitoring for conducting the observation-led construction control.

These would have been combined to induce the failure and detailed account is described for each of the above items in the papers by Tan (2006), Yong (2006), and Yong and Lee (2007). It is not the intention of this paper to debate on these items. The purpose herein would rather be to focus on features of soil conditions which have not been thoroughly discussed, but unearthed more recently through further investigations in details.

2.6 Soil conditions at the site of the incident

In the design stage before the incident, soil conditions were investigated by means of boring, sounding, and recovering undisturbed samples and testing in the laboratory. The locations of borings, standard penetration tests (SPT) and cone penetration tests conducted at the design stage in the vicinity of the failure zone are shown in Figure 8 with symbols ABH. Also designated by AC in Figure 8 are the places where the cone penetration tests (CPT) were carried out. Cone tests were also performed at the locations denoted by MC2026, 3006 and 3007.

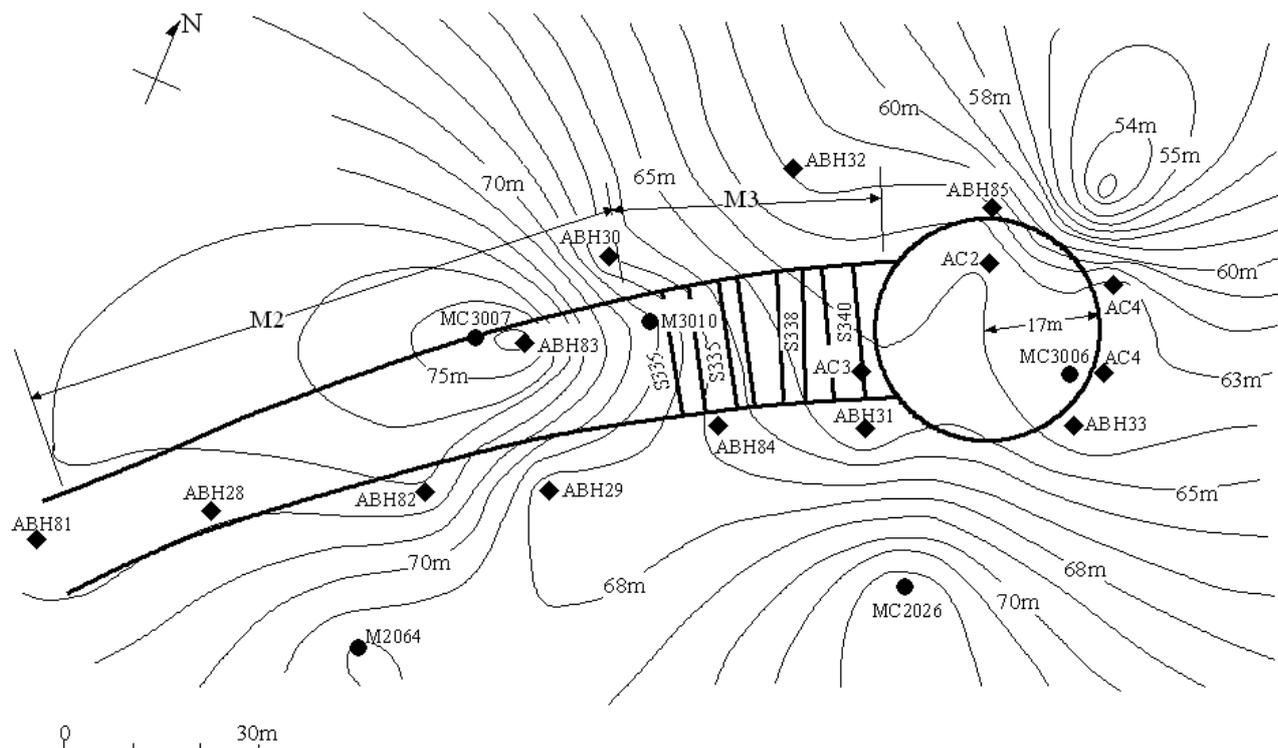


Figure 8. Contours of elevation at the top of the old alluvium (OA-layer) based on boring data at the design stage

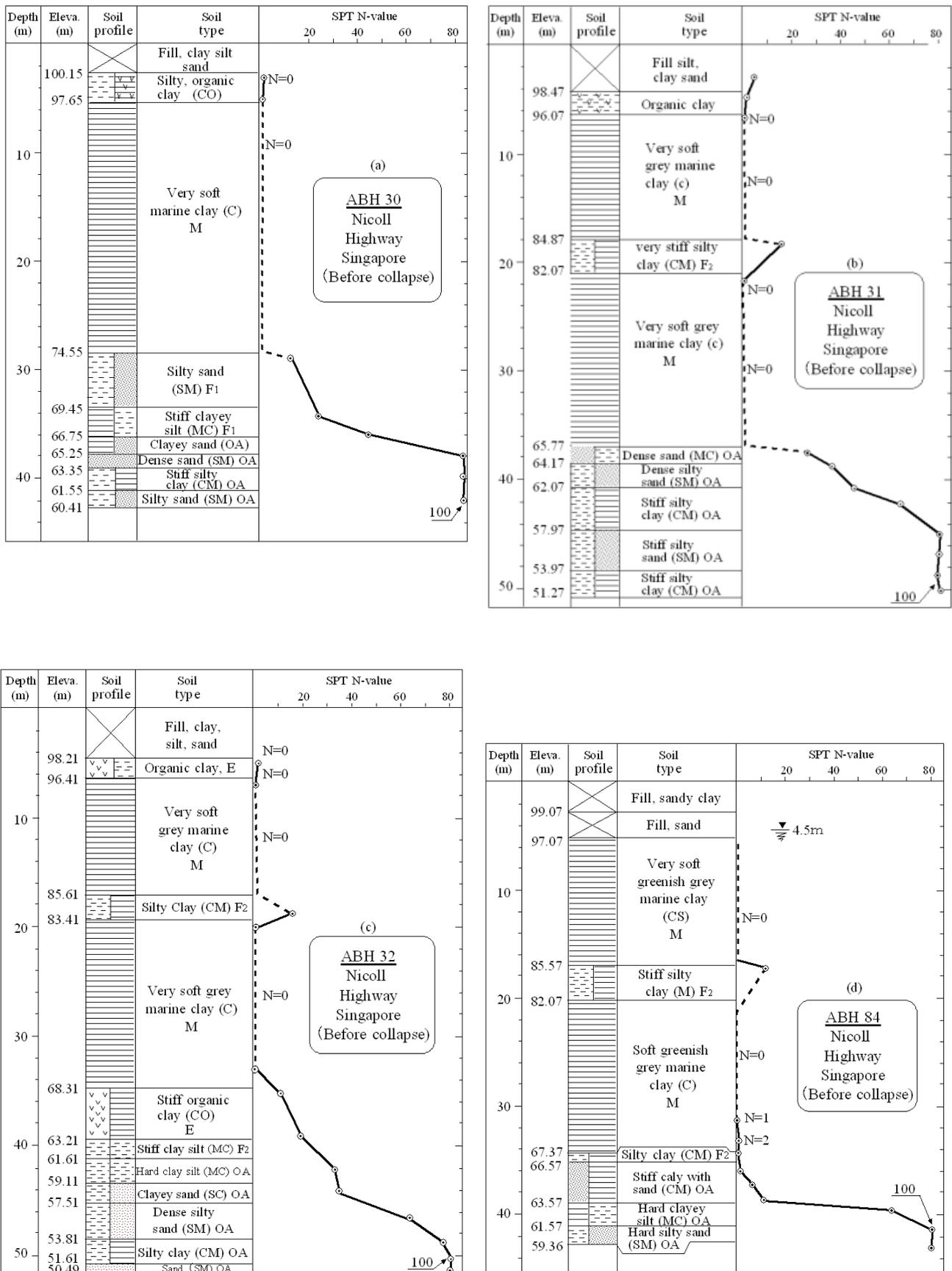


Figure 9. Soil profiles at the sites near the excavation obtained before the collapse

The results of the boring and SPT N-values at the site ABH30, 31, 32 and 84 are shown in Figure 9. It may be seen that the recent fills exist to a depth of 3 to 5m underlaid by a thick deposit of soft marine clay, M, having a SPT N-value of zero. In the middle of this clay deposit, there exists a stiff silty clay layer of fluvial origin about 3m thick at the depth of 18 to 20m. This layer has a SPT N-value of 10 to 15 and denoted by F₂.

The deposits M and F₂ are called Kallang Formation. Under the Kallang Formation, there exists a stiff silty sand layer having variable SPT N-value from 10 to greater than 100. This layer is an old alluvium and denoted by OA. Between the marine clay M and the OA layer, there exists occasionally the estuarine deposit consisting of organic soil and also the fluvial layer F₂.

As a result of physical and mechanical tests, the marine clay was found to possess the plasticity index of 30 to 50 and compression index of C_c 0.9. The undrained shear strength was shown to be in the range of S_u=20~60kPa at depths of 40m and the clay is still slightly under-consolidated state with OCR=0.8~1.0. It was considered important to determine the elevation of the Old Alluvium (OA), as it was defined as the base of the bottom deposit into which the diaphragm wall was to be embedded. In the design, the toe of the diaphragm was set to be 3m below the top of the OA layer.

In view of its importance, the contour lines indicating equal elevations of the top of the OA layer were also drawn in Figure 8, on the basis of the information available at the design stage. It is noted in Figure 8 that the distance between two adjacent boring logs was no shorter than about 20m and consequently the buried valley near the southern wall close to the access shaft was not identified. This fact turned out to be vital for creating the worst situations to occur afterwards.

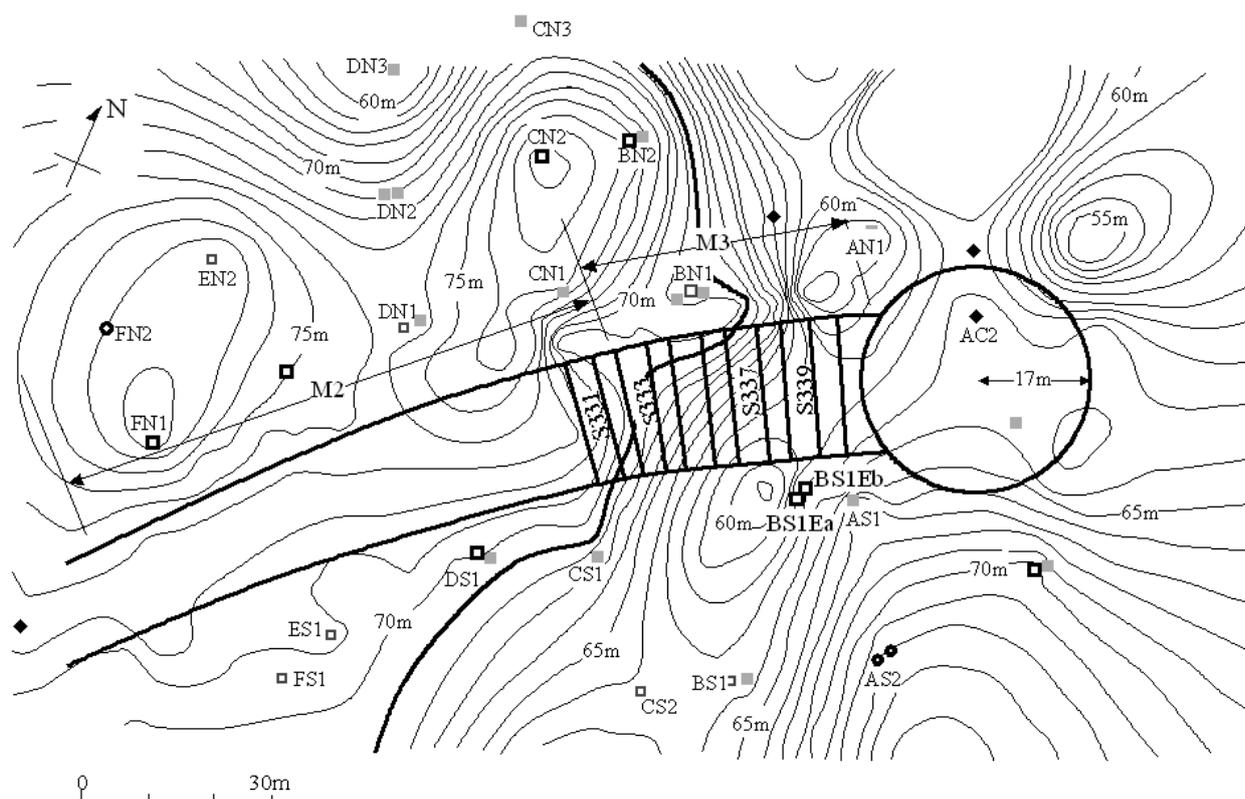


Figure 10. Counter of elevation at the top of the old alluvium (OA-layer) based on boring data after the collapse

2.7 Post-collapse site investigations for forensic diagnosis

After the incident, no attempt was made to dig out the debris buried within the excavated space, because a new route for the subway construction was laid down at the southern side. Thus, the excavated space was buried instead by dumping soils and then in-situ boring and soundings were carried out. These included also the investigation of the post-collapse conditions of the broken pieces of the diaphragm wall panels, construction machines buried within the excavated space.

(a) Borings

Outside the zone of the excavation several series of boring and soundings were performed to depths of about 40 to 50m corresponding to an elevation of about 50m RL (Reduced Level). Nearest the tunnel access shaft, a pair of boring, AN1 and AS1 was performed at sites north and south, respectively, of the excavation. Further westward, another pair of boring was conducted at the locations BN1 and BS1. When the drilling hit an obstacle before reaching a targeted depth, another hole was drilled. Likewise, still other pairs were conducted westwards as indicated by the heading letter C, D, E and F in Figure 10.

Some of the results of the borings conducted at points BS1Ea and BS1Eb about 7m south of the collapsed wall is displayed in Figure 11(a) and (b) where it can be seen that remains of the grouted cement were encountered at an elevation of 73m and 67m RL. In another boring at CS1 shown in Figure 11(c), the soil stratification was found to be about the same as that before the failure event.

(b) Magnetic logging test

This test was intended to detect ferrous metal obstructions such as broken pieces of walers, struts or construction equipments buried in the debris within the sliding zone. The method consists of incremental lowering or rising of a magnetic probe through a borehole and to detect the magnetic reaction at varying depths. During the boring, the kind of soils was identified permitting depthwise pictures of soil pro-files to be established.

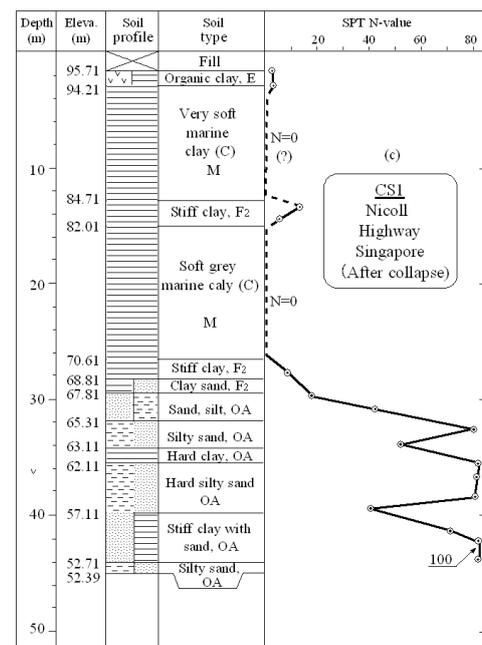
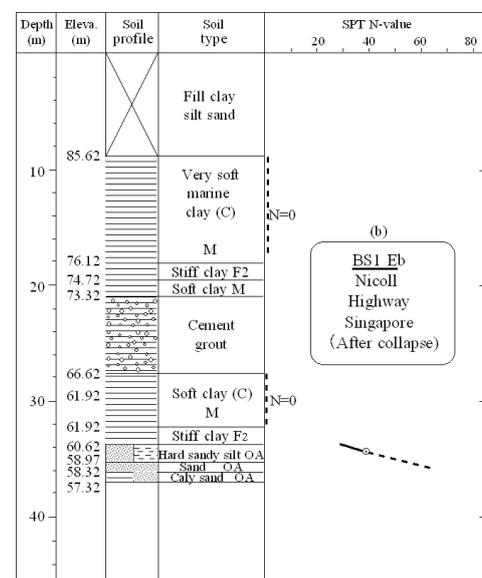
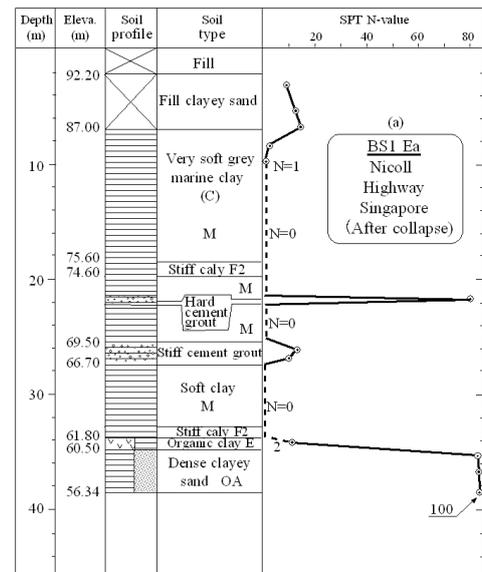


Figure 11. Soil profiles just south of the collapsed diaphragm wall panels

The magnetic logging was conducted at a spacing of about 2m along the alignment 3m behind the northern diaphragm wall. These are denoted by WN1 southward to WN34 as shown in Figure 12. Similarly, along the alignment 3 to 5m behind the south diaphragm wall, the magnetic logging was performed. These are indicated by WS1 to WS33 in Figure 12. When the boring hits some obstructions, the drilling was made once again nearby. In the area between WS10 and WS14, it was not possible for the boring to reach the targeted depths because of obstructions encountered midway. Thus, several more borings were carried out until they reached the OA base deposit.

2.8 Damage features under the ground

After investigating the location of the exposed top of the diaphragm wall panels, it became possible to approximately figure out the scenario of the failure. Exact locations of the panel top in plan view are shown in Figure 12. Based on the features shown in this figure it may be mentioned that the collapse mechanism at panels M213, M212 and M306 were different from that elsewhere and almost certainly it involved the toe of the diaphragm wall panels kick-ing in, and flow of the soil underneath the diaphragm wall panels. This form of the failure will have allowed the maximum southward movement of the opposite north wall panels, namely M211 and M301.

As a consequence of inward collapse of the retaining diaphragm walls, it was speculated that a huge amount of soil mass moved into the excavated space. In fact, a subsidence of the ground surface as much as 10m was observed on the south side in proximity to the wall and the settlement extended about 50m southward, as seen in Figures 4 and 12. The area of the ground settlement extended also north towards the Nicoll Highway but to a lesser extent as can be seen in Figure 4. Thus, it was considered certain that the failure was initially triggered by the breakage of the diaphragm walls on the southern side, thereby involving a landslide towards the excavated space. As a result of losing the horizontal support, the northern walls moved also towards the excavation, inducing sliding of soil mass towards the south but to a lesser extent.

This scenario of the event was substantiated by the borings conducted after the failure in which the elevation of the intermediate layer F_2 was found to be located almost 10m lower than that found by the borings before the failure. Note that prior to the collapse, the elevation of the base of intermediate F_2 layer was remarkably consistent across the collapse area, and so provides a good marker for identifying the vertical displacement of the soil deposit. The location of the F_2 layers after the collapse is shown in the cross-section in Figure 11. It was then possible to depict the mode of soil movement involved in the slides on the south and north sides. The approximate locations of the sliding surfaces are displayed in Figure 13 for the cross section 6-6 of Figure 12.

As a result of the post-incident in-situ investigations as above, it became possible to figure out approximately the states of collapse to each of the cross sections indicated in Figure 14. Shown in Figure 15 are probable features of the collapsed waler-strut system buried within the ground which were speculated for the cross section 1, 5 and 7. It is to be noticed that in the cross section 1 and 5, the inter-mediate layer F_2 is inclined towards the excavation as verified by the boring data at ABH31 and 32 and also at AS1 and AN1 conducted after the failure. This is indicative of the settlement of the even deep-seated soil layer involved in the slide.

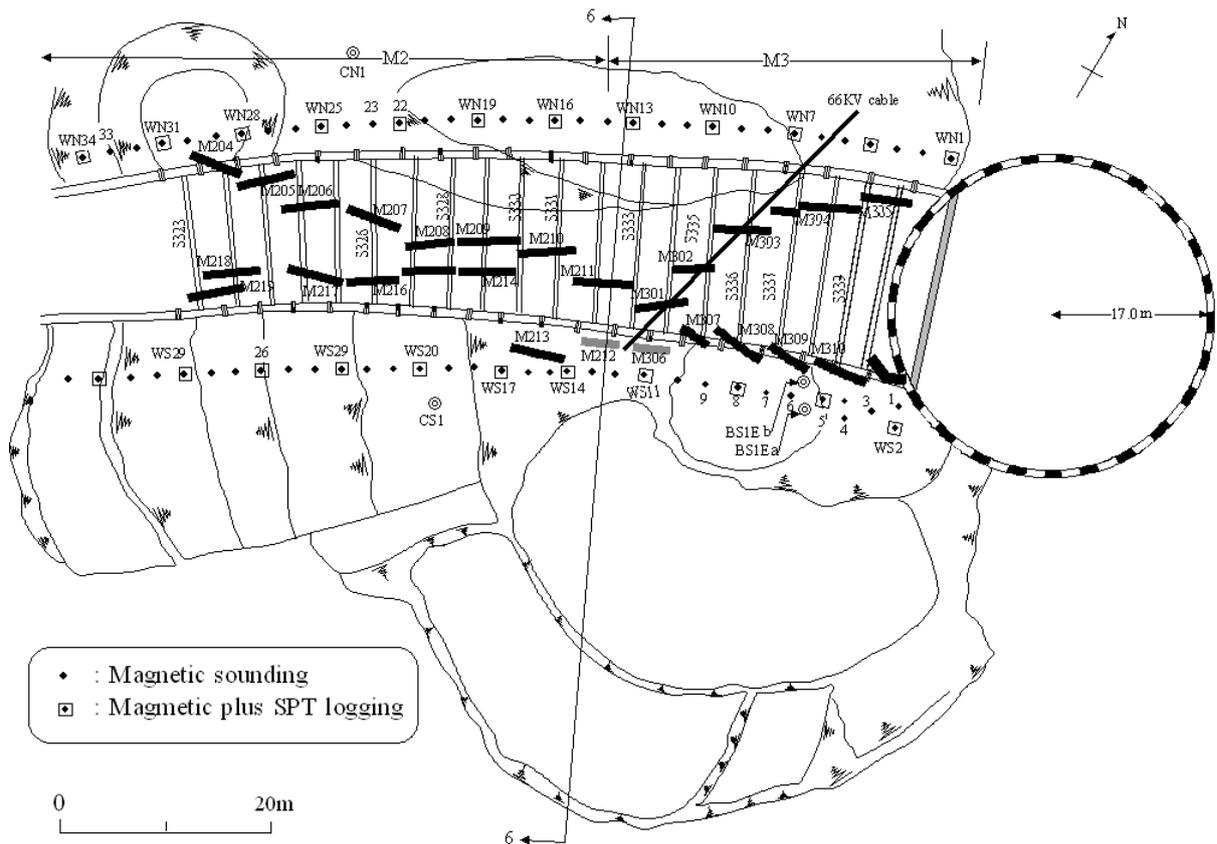
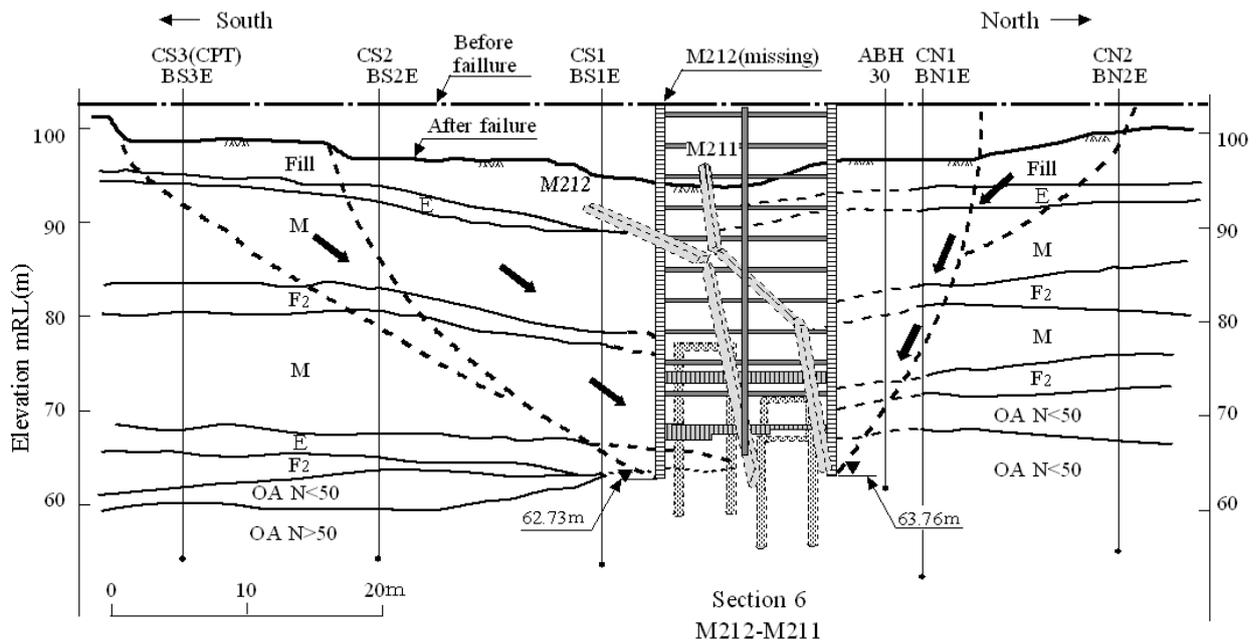


Figure 12. Locations of the magnetic sounding and boring conducted after the collapse



Note: All shapes of diaphragm walls underground are unverified assumption

Figure 13. Features of the ground movements towards the excavated space accompanies by the subsidence

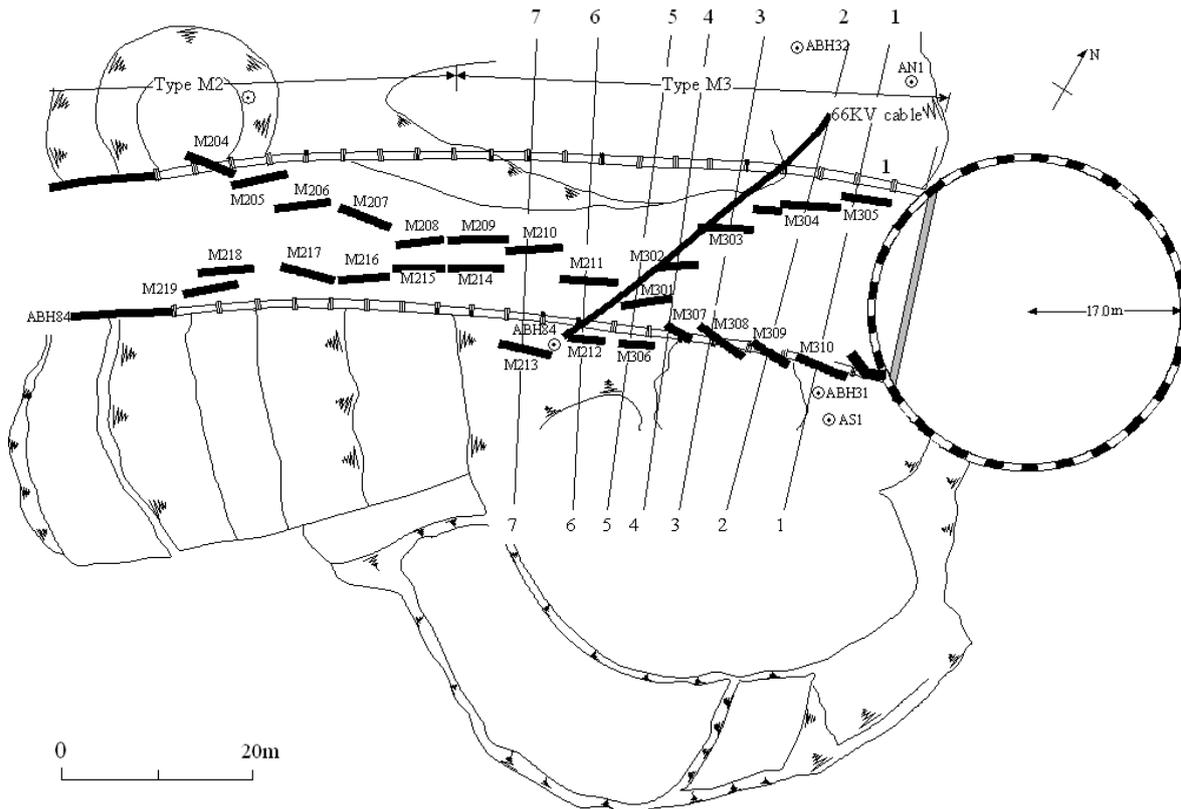
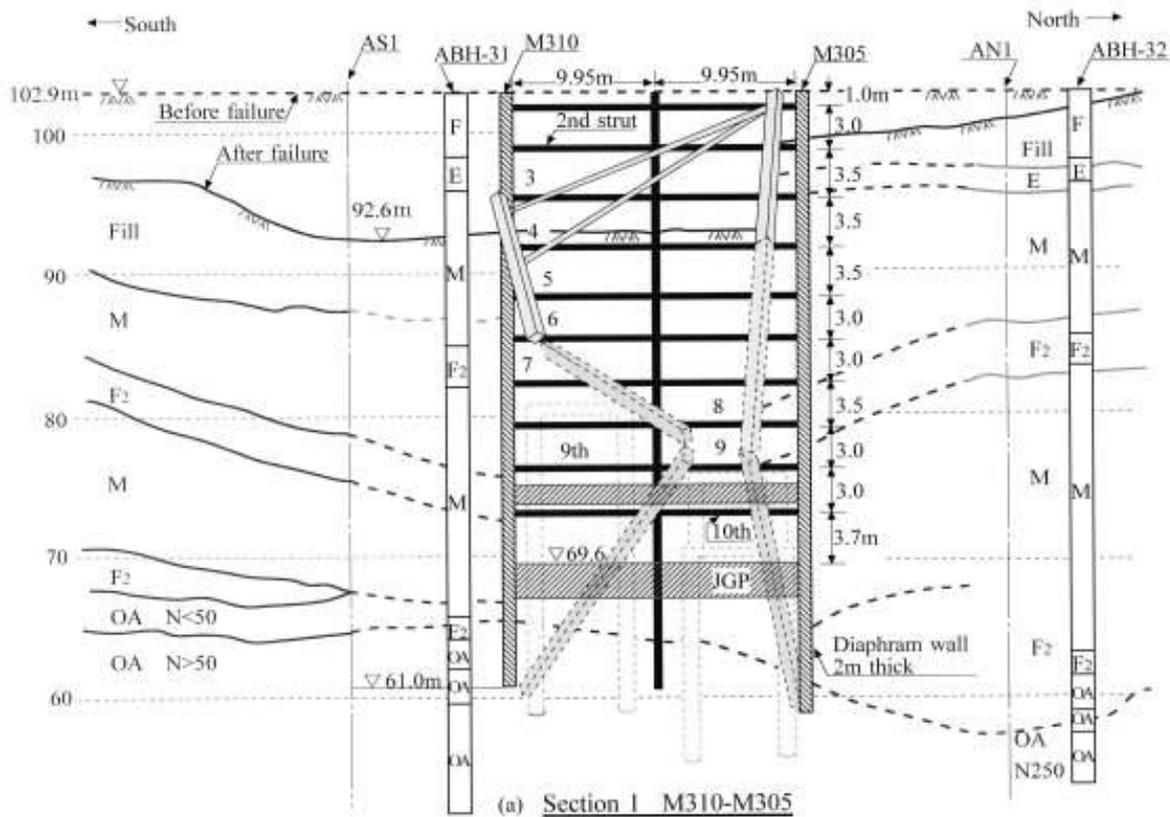
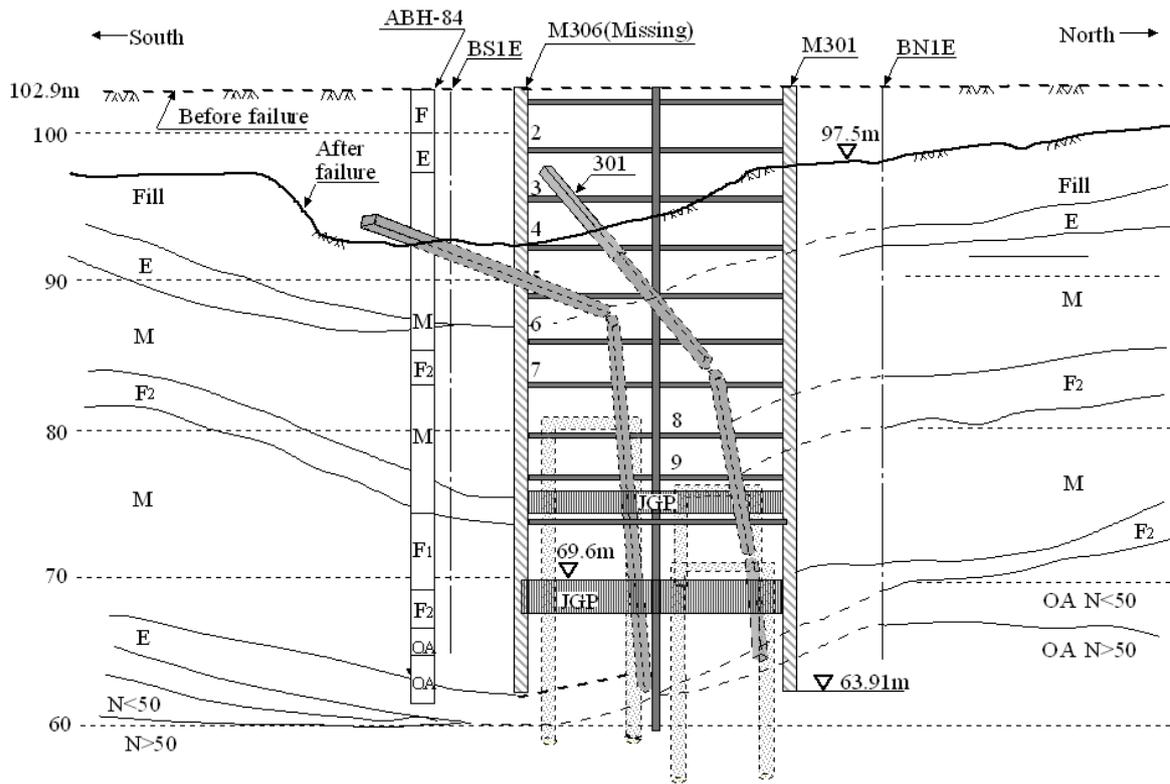


Figure 14. Cross sections for which the damage features were depicted

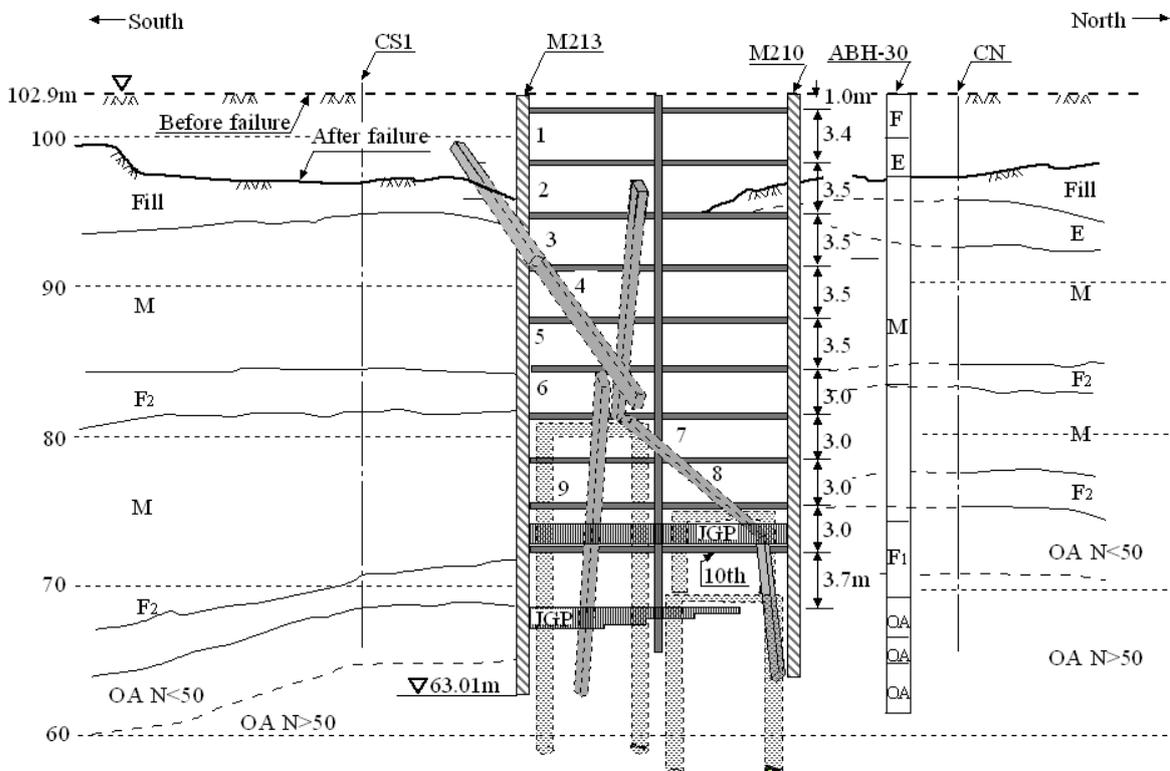


Note: All shapes of diaphragm walls underground are unverified assumption



(b) Section 5 M306-M301

Note: All shapes of diaphragm walls underground are unverified assumption



(c) Section 7 M213-M210

Note: All shapes of diaphragm walls underground are unverified assumption

Figure 15. Cross sections showing the mode of collapse in the zone of failure

2.9 Existence of the buried valley

One of the key issues associated with generic cause of the collapse is the depth of embedment of the diaphragm wall toe into the stiff base layer OA which had been considered competent enough at the time the design was made. Thus, it is of crucial importance to identify exact configuration of the top of the OA layer, and its mechanical properties as well.

By compiling all the data made available after the collapse, the contour lines of the top of the OA layer were established as demonstrated in Figure 10 with a higher level of accuracy as compared to that shown in Figure 8 which was made up with the boring data at the time of the feasibility study and the design. The contour lines in Figure 10 unveil some important features as follows which had not been identified in the previous map in Figure 8.

- (1) There exists a steeply depressed buried valley at the location on the south side a few meters south of the strut S336 and S337. The bottom of the buried valley has an elevation of about 59m RL. The buried valley dips steeply towards the east from the south end of struts S331.
- (2) The buried valley extends towards the north-east running on the west side of the tunnel access shaft (TSA).
- (3) As typically observed in the soil profiles at BS1Ea and BS1Eb in Figure 11, the SPT N-value tends to increase at the elevation of 61m RL upon hitting the base layer OA. The SPT N-values corresponding to the transition zone below the OA layer were collected from other borings and shown together in Figure 16 by choosing the nominally identified top of the OA layer as zero point. It may be seen that the property of soils is not so competent with a SPT N-value less than about 30 to a depth of about 3m from the top of the OA layer.

2.10 Features of the toe-in of the diaphragm wall

As a result of the detection of the buried valley, questions cropped up as to whether the toe of the diaphragm wall had been embedded sufficiently deep into the base layer of the Old Alluvium (OA), and also as to whether the stiffness of the infill materials near the top of the OA layer was strong enough to mobilize the resistance.

With regard to the latter suspect, closely-spaced soil investigations revealed that the upper part of the OA layer in the buried valley is comprised of sands, silts and clays sometimes containing organic materials with the SPT N-value not greater than 30 as demonstrated in Figure 16. This fact indicates that the infill materials were not necessarily as competent as they had been expected at the design stage.

To obtain a clear picture of the toe-in depth, the results of the borings at the time of the magnetic logging tests plus some others nearby were compiled and arranged side-by-side so that soil profiles along the alignment south of the diaphragm walls can be visualized. The picture arranged in this way is demonstrated in Figure 17 where the top of the OA layer considered at the design stage is indicated, together with the similar top line established after the collapse based on the detailed investigation by the magnetic logging. Note that the symbol such as M306 to M310 and M212 indicate the location of the diaphragm wall panel. It is also to be noted that for the section between M212 and M309, the estuarine deposit E and the upper part of the OA layer might have been displaced laterally being involved in the slide. At the same time, settlements of the order of 5-10m are envisioned to have occurred in the upper

part of the OA layer, as inferred from the settlements of the upper lying layer F₂, as shown in Figures 13 and 15, which was confirmed by the post collapse boring data. From the records of the diaphragm wall construction, the bottom line of the wall before the collapse is known and it is shown together in Figure 17.

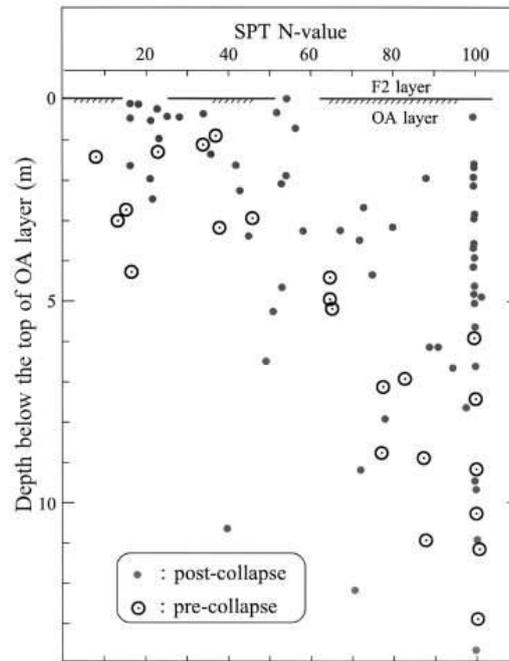


Figure 16. SPT N-values from the top of the OA layer

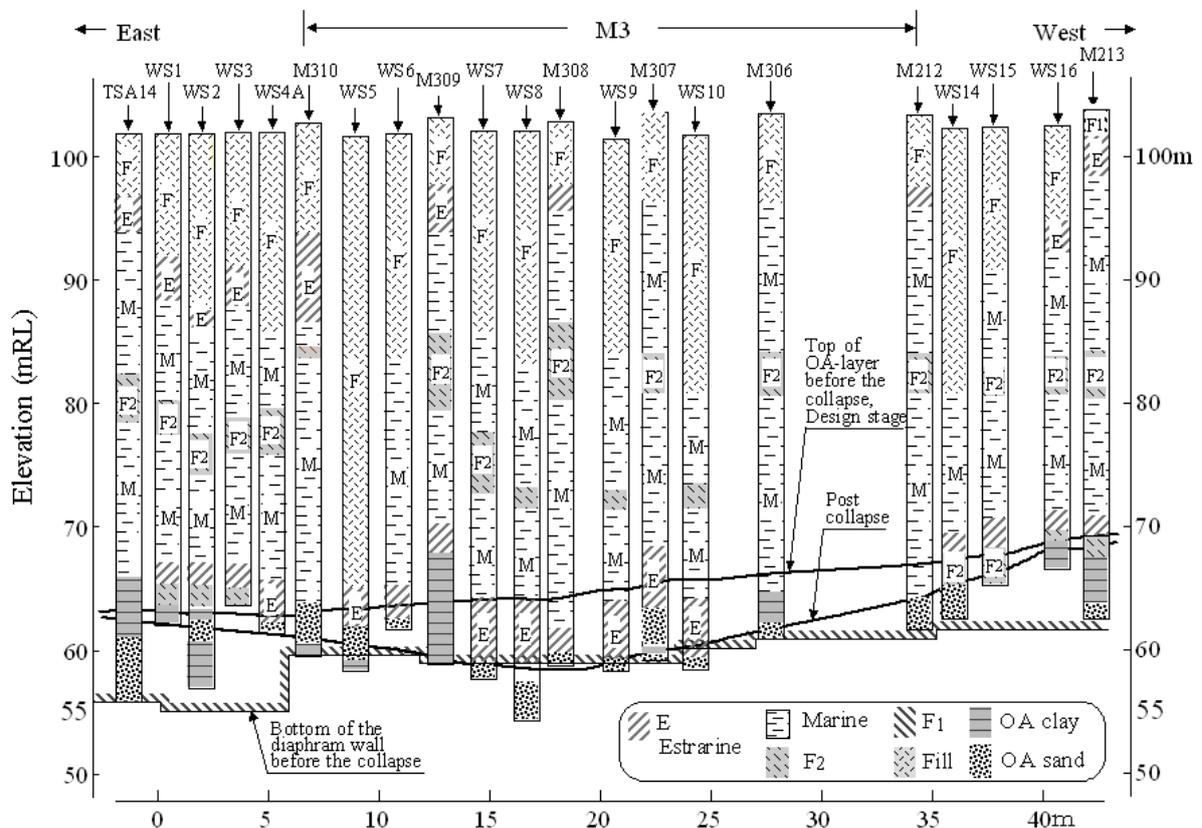


Figure 17. Soil profile along the south wall and bottom of the diaphragm wall

By comparing the elevations at the top of the OA layer before the collapse with those of the bottom of the diaphragm wall, it can be seen from Figure 17 that in the eastern zone between WS1 and WS4A, the toe-in of the wall was more than 3m. In the zone of collapse between WS5 and WS16, the toe of the diaphragm wall seems not to be embedded sufficiently deep. However, even if allowance is made for the settlement of the OA layer of the order of 3~5m, the depth of embedment into the OA layer may be considered reasonably deep almost in consistence with that considered in the design.

It is to be pointed out, however, that the quality of the soils at the upper part of the OA layer was less competent in stiffness or strength as compared to that assumed at the time of the design. This point would have a more important bearing from the forensic aspect.

In the area which failed there were inclinometers at or just behind the wall at the north and south sides of the excavation. Shown in Figure 19 is the time change of depthwise distributions of the wall deflection estimated from the inclinometer, I-104, which had been installed in the soils 1.0m apart from the south wall near M212 (Fig. 18). It is known that the wall deflection started to increase significantly from April 17, 2004 as indicated by a dash line in the figure. The maximum displacement measured at 13:12 on April 2004 is known to have amounted to 43.5cm at the elevation of 72m. It is to be noticed that actual collapse took place at 15:33. i.e. 2 hours 21 minutes (141 minutes) afterwards. Thus, it is likely that just at the time of collapse, the maximum value of the wall deflection might have been larger than 43.5cm. It should be cautioned that the waler-strut system had sustained the deflection of at least 43.5cm for the period of 141 minutes without causing collapse.

To be noticed, moreover, is the fact that the distribution of the wall deflection shown in Figure 19 was derived by integration of tilts at several depths measured by the inclinometer on the assumption that the tilt at the toe was equal to zero. This assumption may not be correct in the present case, because the toe of the diaphragm wall had not been firmly embedded in a stiff base stratum. Thus, the distribution of the wall deflection shown in Figure 19 should be taken a simply as indicating the relative displacement counted from the toe of the diaphragm wall.

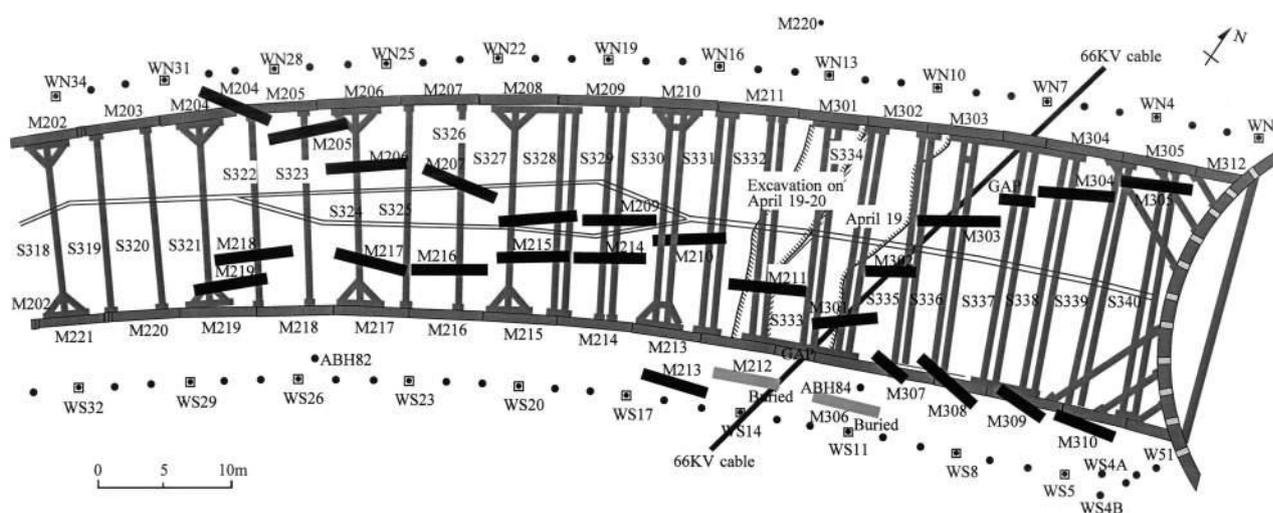


Figure 18. Detailed arrangements of the struts, diaphragm wall panel and locations of the magnetic logging

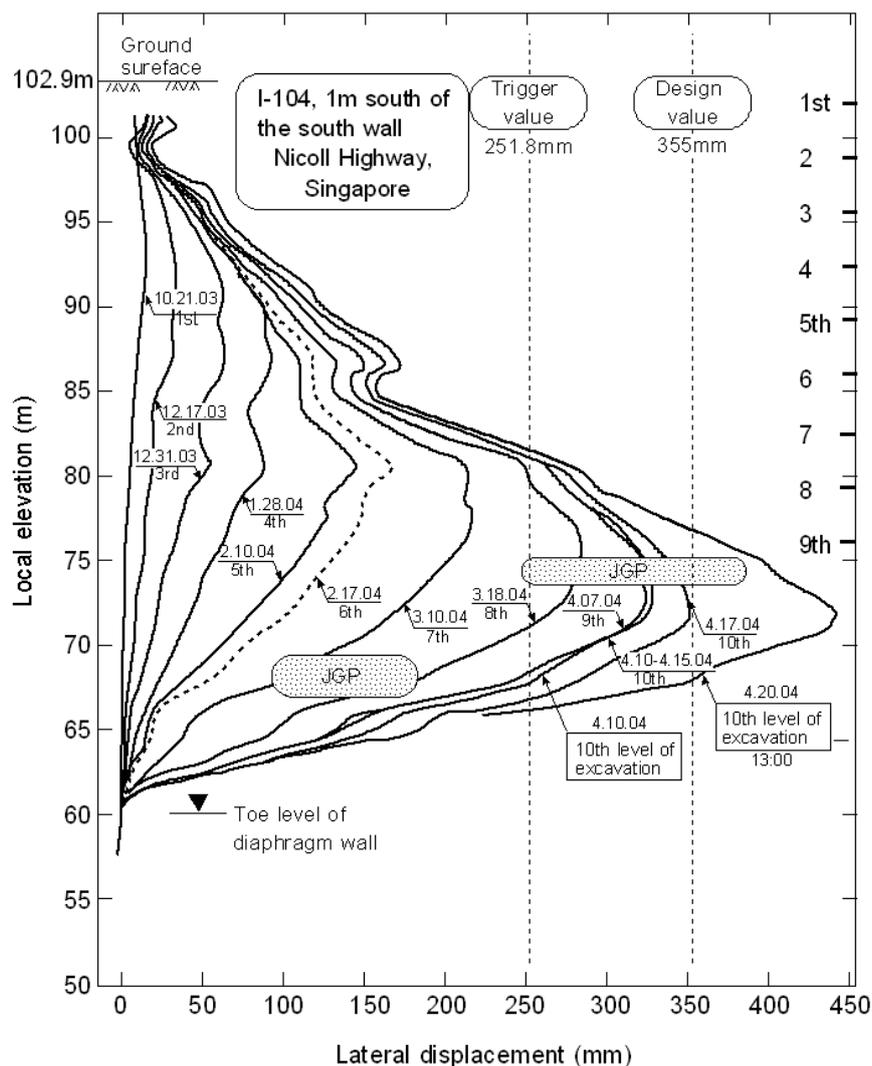


Figure 19 Time change of depthwise distribution of the wall measured by the inclinometer in the soil deposit 1m apart from the south wall

2.11 Likely scenario leading to the collapse

In the light of all pieces of available information as described above, the scenario leading to the collapse may be envisioned as follows.

- (1) When the accident occurred at 3:33p.m. on April 20, 2004, the front of westward excavation was located at the place of strut S332 as shown in Figure 18. At that time, the 1.5m thick jet-grouted stiff soil (sacrificial strut) was being re-moved. The underlying soil layers of the OA having a SPT N-value less than about 30 at an elevation 61-63m must have pushed the wall toe towards excavation. The second Jet-grouted stiff layer (JGP) at an elevation of 69.6 to 66.6m was unable to resist against that earth pressure and thus the diaphragm wall panels M212 and M306 were forced to deform inwards. It is considered likely that the second JGP layer might have been possessed of horizontal stiffness lower than that expected before or it might have had brittle characteristics. It is to be noticed here that, be-cause of the presence of the 66KV electrical cable buried at shallow depth through the

two panels denoted as GAP in Figure 18, it was not possible to install the bored piles from the ground surface in advance underneath the elevation of 71m. This is clearly indicated in the plan of Figure 5. Because of the absence of the bored piles, it was considered easier for the toe of the M212 to M306 to move towards the excavation.

- (2) The inward movement of the wall panel toe must have caused some uplift of the soil deposits within the excavation, which induced in turn a lift-up of the kingpost in the middle of the struts. In addition, excavation of the upper sacrificial jet-grouted layer is envisioned to have suddenly reduced friction between the jet-grouted lower layer and the diaphragm wall, and may have accelerated the soil heave and the uplift of king-posts. At the same time, sag is considered to have occurred in the diaphragm wall.
- (3) This uplift, combined with the sag of the wall must have caused the local buckling or breakage of the flange of the H-beams at connection between the walers and struts, creating as much as 20cm vertical offset as illustrated in Figure 20. As a result, the net length of the struts was shortened and diaphragm wall must have deformed further inwards.
- (4) There were inclinometers installed behind the walls on the north and south sides as mentioned above. The lateral displacement measured on the south increased to at least 43.5cm at the level of 10th strut at 13:12 on April 20. This appears to have been induced by the earth pressure from behind which was mobilized by the inward movement of the wall.
- (5) With the loss of functioning of the waler-strut system and increased inward movement of the wall panel toe, the integrity of the structural support as a whole was completely lost, leading eventually to the total collapse. This is illustrated in Figure 20.
- (6) As a consequence of the collapse to the structural system, the diaphragm wall panels were twisted, displaced and rotated, accompanied by the slides of the surrounding ground as described in Figure 13.

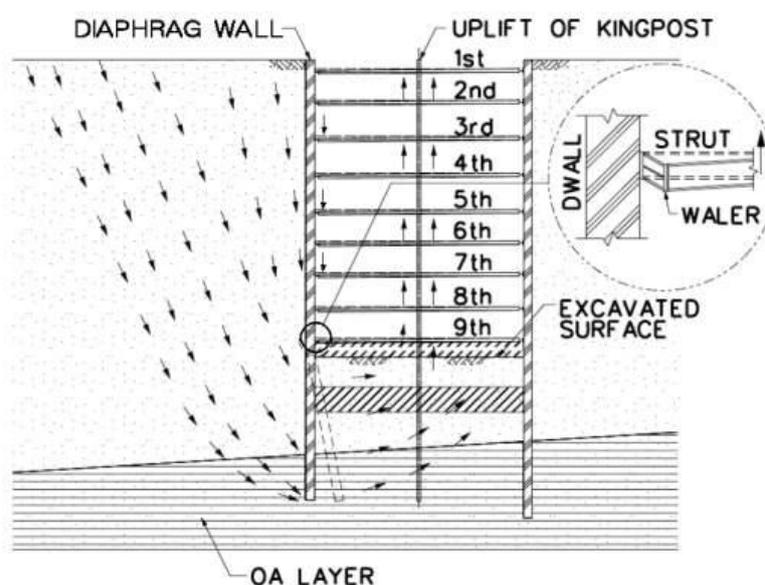


Figure 20. Probable movements of soils near the toe of the diaphragm wall, and uplifting of the kingpost and sag of the wall leading to buckling and offset between strut and waler

The scenario of the collapse as described above is based on the point of view that the generic cause of the failure was the excessive inward movement of the toe of the diaphragm. Another scenario could be established alternatively on the basis of the assumption that the earth pressure acting on the diaphragm wall had been underestimated at the time of the design. A greater earth pressure than that considered in the design must have induced the large deflection of the diaphragm wall leading to the breakage of the water-strut system. This collapse scenario is addressed thoroughly in the papers by Tang (2006), Yong et al. (2006) and Yong-Lee (2007). The authors, however, prefer to take the views described in this paper.

2.12 Lessons learned from the forensic diagnosis

One of the most important lessons learned from the incident in Singapore seems to be the fact that, although the spatial distance of borings or sounding is reasonably short in view of the commonly adopted practice, there always are chances to miss irregularity in the deposits such as buried valleys. Therefore, special precaution should be paid to the soil conditions particularly in deep-seated deposits. In the area of complicated geological setting, existence of buried valleys should be suspected at the stage when projects are put forward. In such a case soil investigations need to be carried out at spacing closer than that normally adopted. In addition, properties of soil materials within the buried valley should be investigated in details, if it is discovered. The outcome of these studies should be meticulously reflected on the grand planning as well as on detailed design of the soil-structure system considering each phase of construction processes.

3. COLLAPSE OF SUBWAY EXCAVATION IN KAOHSIUNG, TAIWAN

3.1 Outline of the incidence

Subway construction had been underway in east-west direction in the central part of Kaohsiung. Upon finishing the tunnel construction by the method of earth-balanced shield, the corridor connecting the two tunnels (up-line and down-line) was constructed by means of what is called the NATM method involving open excavation with the help of steel -framed support and injection. Then, a vertical shaft 3.3m in diameter was excavated to provide a sump for water collection in the middle of the corridor in open dry conditions with the support of the H-shaped circular beams. When the shaft excavation reached a level 4.95m from the floor of the corridor, a chunk of wet soil tumbled out from the southern wall of the shaft at the bottom.

The small collapse was followed by steadily increased outflow of mud water. The amount of water increased minute after minute. This breakage led to undermining the already built-up tunnels, accompanied by an inflow of soil and water from the rips opened at the junction on the ceiling between the corridor and tunnel. This breakage culminated eventually in a large scale collapse of the tunnel structure involving formation of two cave-ins on the ground surface.

3.2 Description of the site conditions

The location of the collapse is shown in Figure 21 and the feature of the cave-in on the ground surface is displayed in Figure 22. A bird-eye view over the cave-in is displayed in Figure 23. The plan

and side view of the tunnel are shown in Figure 24. It is to be noted that there was an underground roadway called Chung Cheng under path just above the subway tunnel. The dual tunnels were constructed by the tunnel boring machine (TBM) which can advance by rotating a large steel disk equipped with cutting blade, while the cutting face is balanced by the mud pressure.

The soil profiles at the locations BO29 and BO30 shown in Figure 22 are indicated in Figure 25, where it can be seen that the deposits are comprised predominantly of silty sand with occasional layers of low-plasticity clay (CL) to a depth of 40m. The blow count, N-value, in the standard penetration test is shown to increase with depth and to have a value of 20 to 30 at the depth where the sump for water collection was excavated.

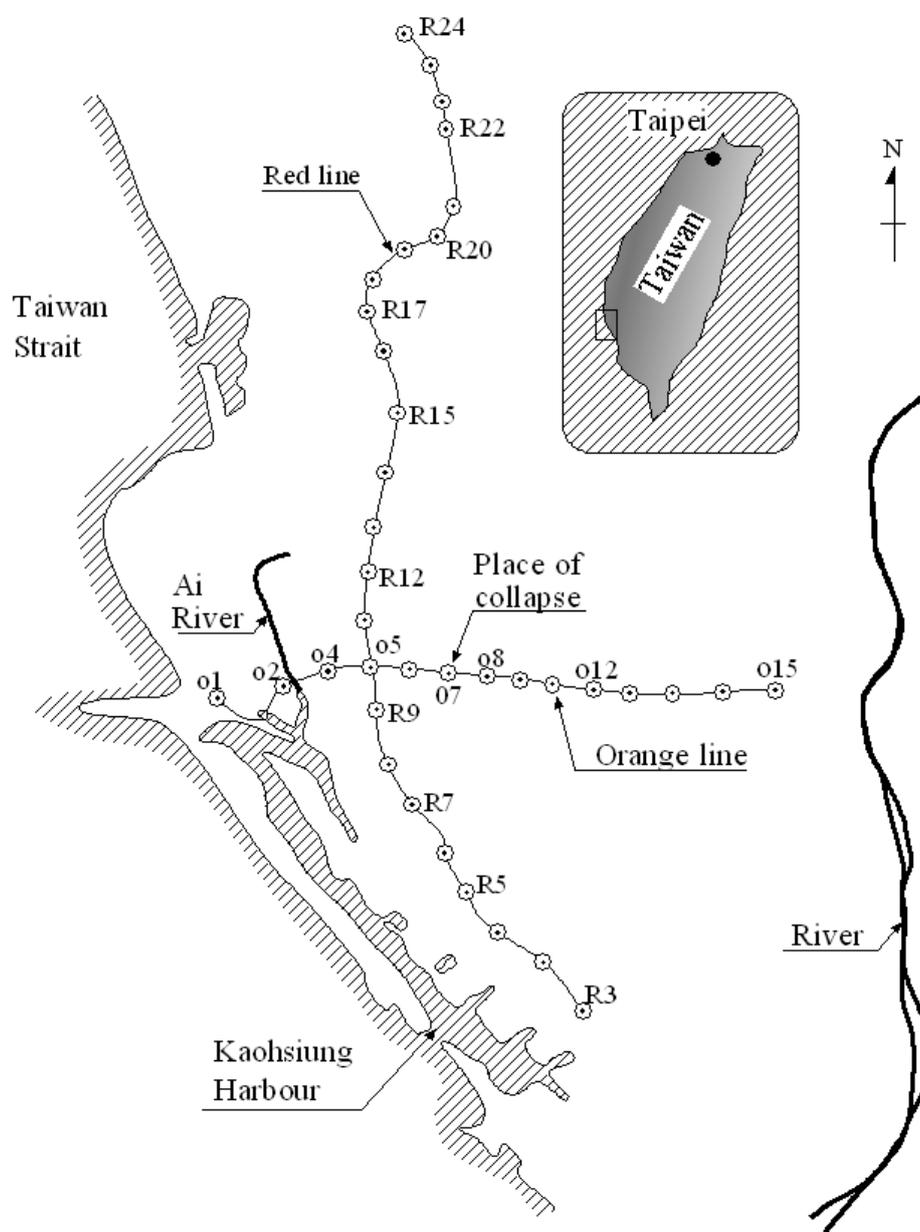


Figure 21. The location of the collapse

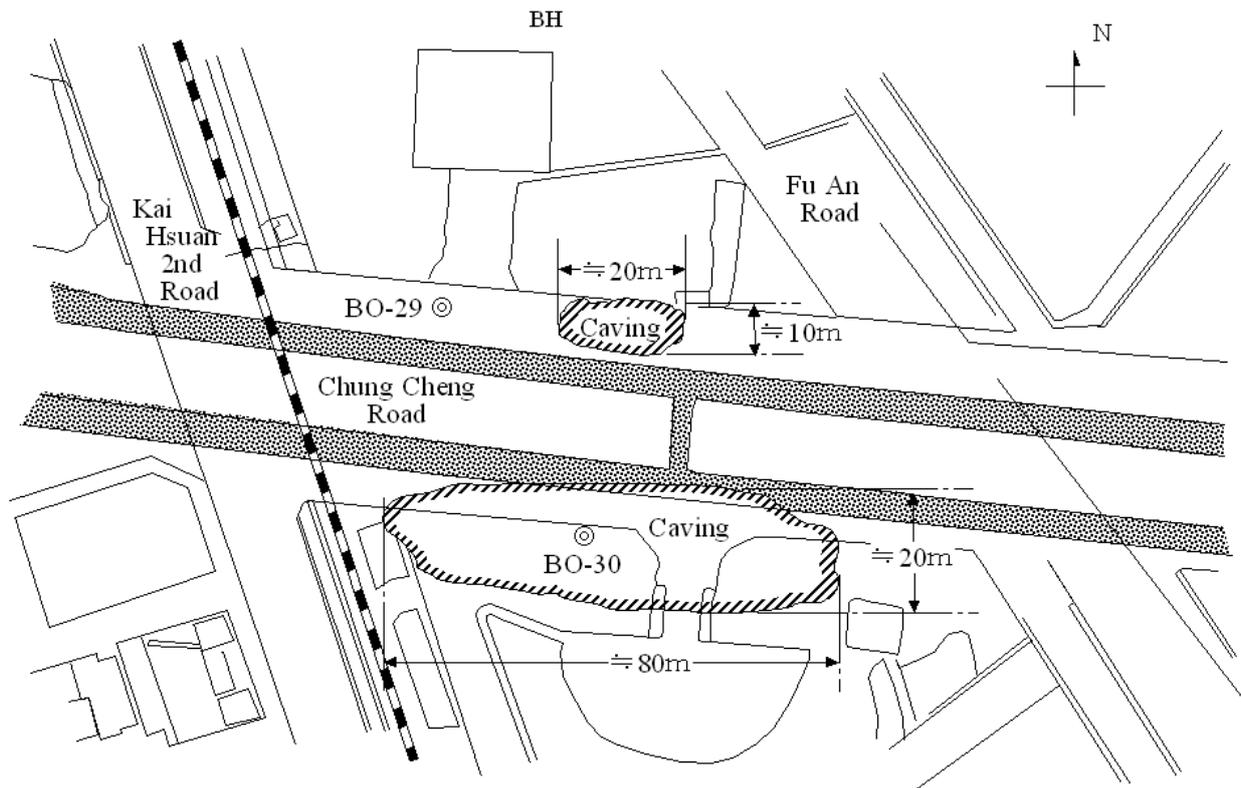


Figure 22. Features of the cave-in in plan view



Figure 23. A bird-eye views over the cave-in in Kaohsiung

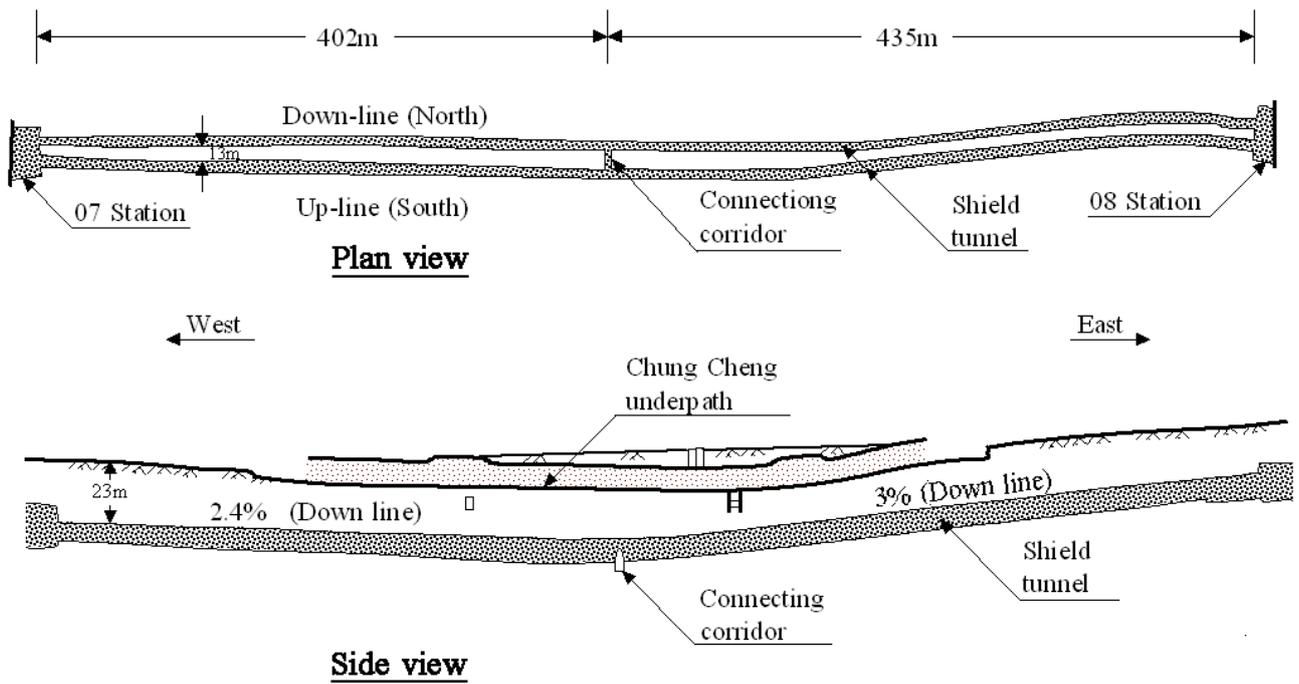


Figure 24. Plan and side view of the tunnel

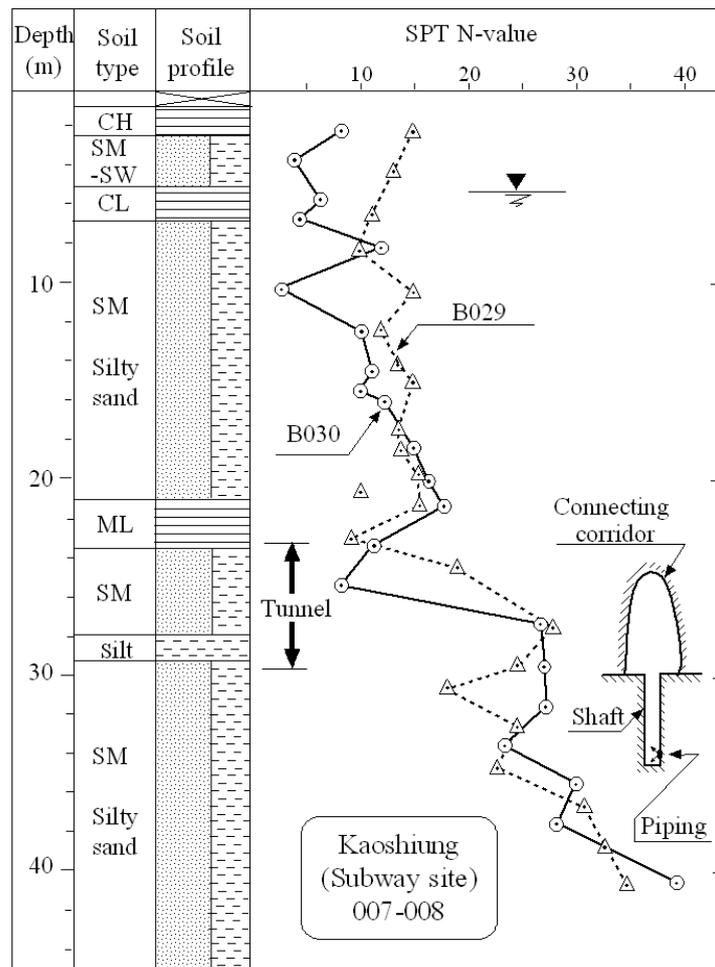


Figure 25. Soil profile at the site of collapse

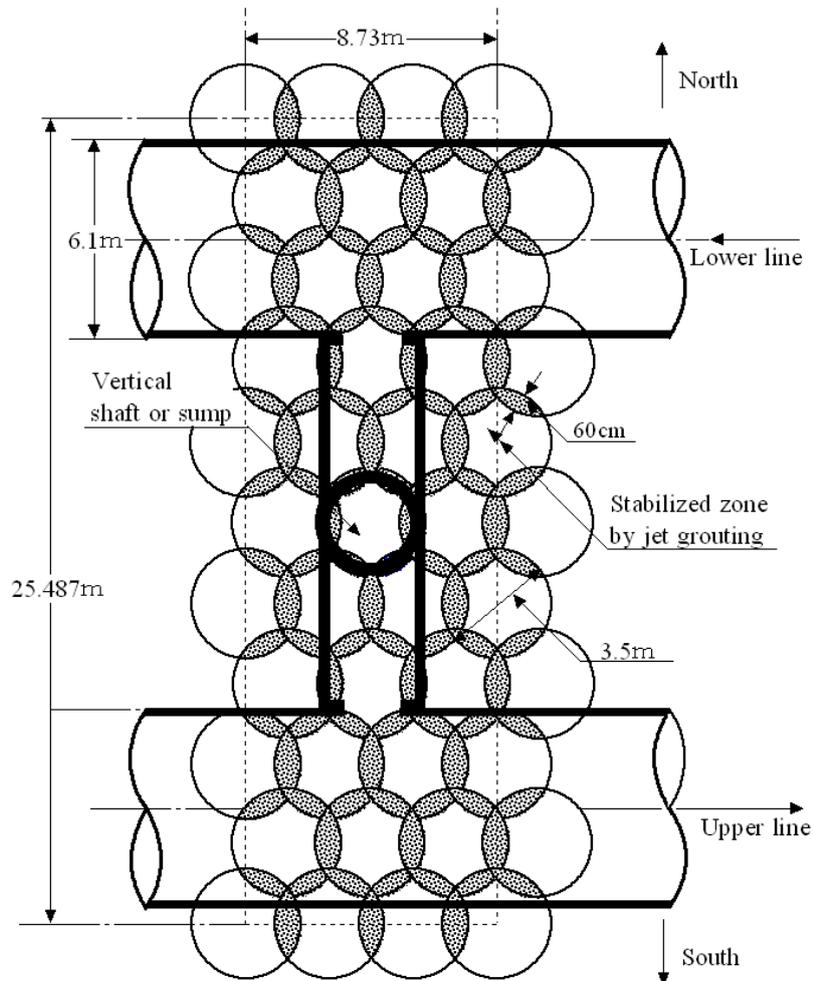


Figure 26. Plan view at the top of the stabilized zone (El. 85.83m)

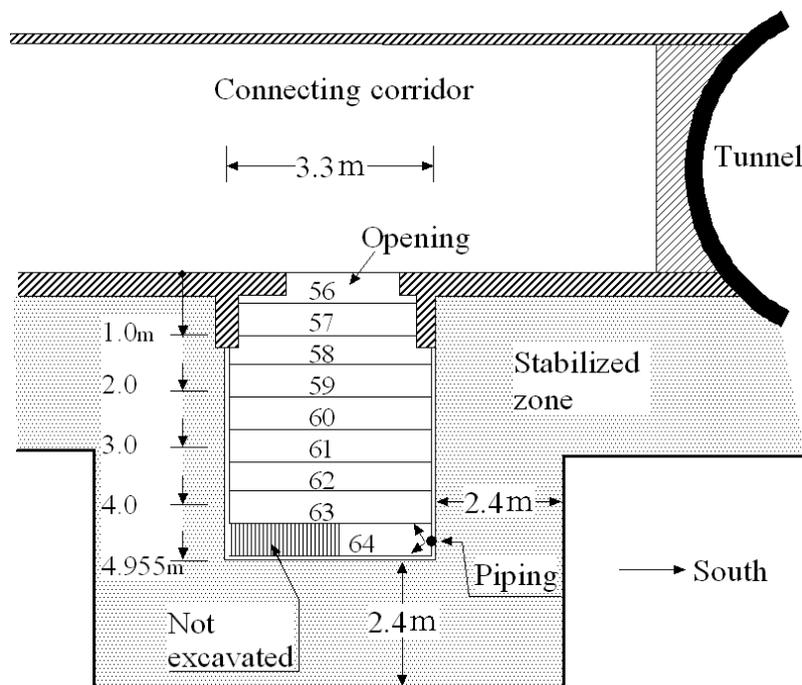


Figure 27. Details of the excavation for the sump and initiation of piping

3.3 Soil stabilization

The section of the ground where the corridor was to be installed had been stabilized by means of the jet grouting, where milky cement mortar is jetted out with compressed air horizontally into the ground from two nozzles attached 180 degree apart to the steel cylinder. The cylinder is lowered into a bored hole to a targeted depth and lifted stepwise while jet-ting the cement mortar with a pressure of 30Mpa. During 2 to 4 rotations per minute, the cylinder was lifted 8.3cm, thereby creating a solidified vertical column of soil-cement mixture with a diameter of 3.5m.

The arrangement of the stabilized soil columns in plan view at an elevation of 85.83m (zero point is taken at a depth 100m below the median sea level) or at the depth of 31m is shown in Figure 26. It may be seen that each column was installed so as to have mutual overlapping of 60cm and to produce a huge stabilized massive zone 8.73m by 25.487m in plan which was considered strong and competent enough to permit excavation of the connecting corridor and the sump to be carried out in dry open conditions without any distress.

3.4 Piping failure of the tunnel

On December 4, 2005, the piping occurred at 3:30p.m. at the very last stage of excavation of the sump. A block of silty sand was detached from the south wall at the bottom of the excavation which is located at a depth of 35m from the ground surface. The circumstance at this time at the bottom of the sump excavation is illustrated in details in Figure 27.

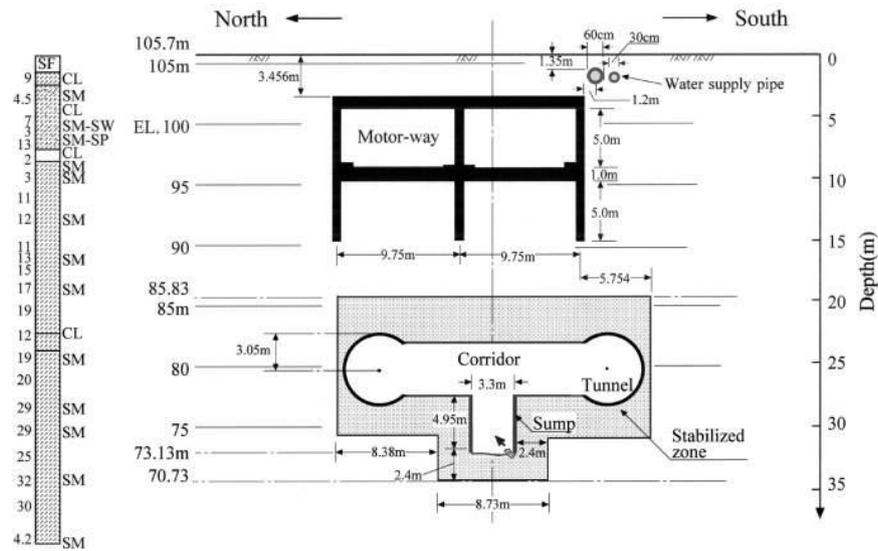
A sequence of the events after triggering the piping which are likely to have occurred is illustrated in the cross sections in Figure 28. The blackish mud water continued to come out in the sump with increasing volume. Two men at work at the bottom strove in vein to clog the hole by dumping sand bags from the corridor. About one hour later, the sand bags were seen moving around in the sump as illustrated in Figure 28 (b). At this stage, the workmen closed the entrance to the sump by placing a steel lid and several bars for its support as illustrated in Figure 28 (c).

It is believed that the mud water flowed into the sump from the bottom through a single hole probably with a diameter of about 30cm. This assumption would hold true in the light of the fact that the zone stabilized by the jet grouting could not so easily broken down in the horizontal direction, and a weak zone might have existed in the form of vertical pipe.

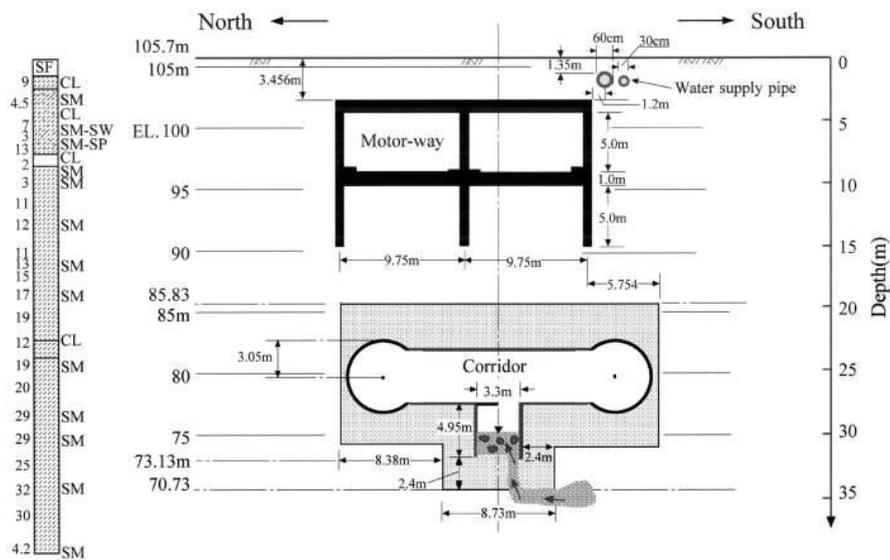
About two hours later, engineers at spot heard squeaking sounds of breaks at segment joints of the tunnel on the south, accompanied by mud water cascading from the ripped joints of segments at the ceiling. It appears likely that because of the underscoring of the bottom portion, the stabilized zone subsided together with the tunnel body, resulting in the stepwise settlement in the longitudinal direction. The probable feature of the breakage at this stage is illustrated in Figure 28 (c).

Around 10:20p.m., the fall-off of the soil above the ceiling had spread upwards and, assisted by the slip-ping along the vertical wall of the motorway structure, soil mass fell down into the tunnel. This resulted in creation of a large cave-in on the ground surface on the south side. At this time, two mains 60m and 30m in diameter for water supply were broken releasing a large amount of water for a pro-longed time. This breakage of buried pipes appears to have transferred the collapsed soil deposits into more flowable debris.

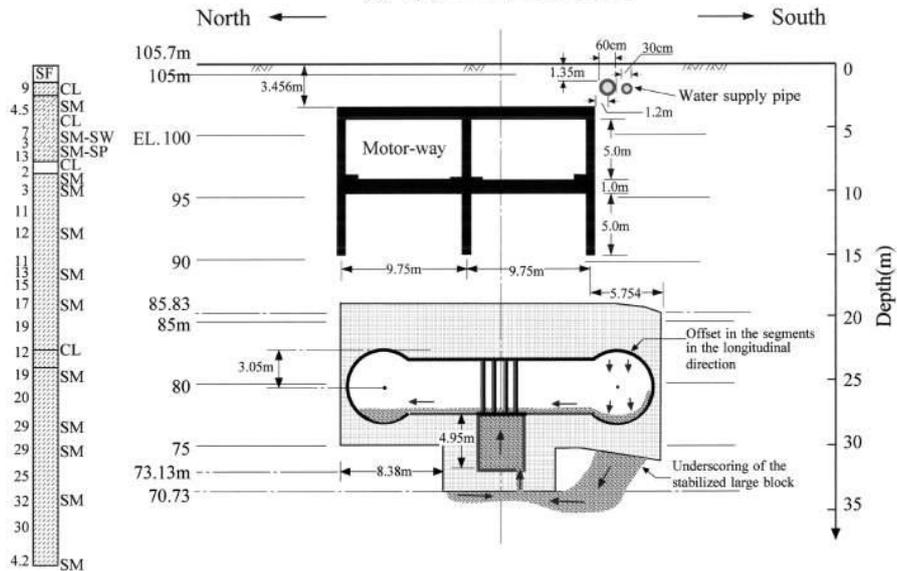
(a) (1) Dec. 4, 15:30, 2005



(b) (2) Dec. 4, 16:15, 2005



(c) (3) Dec. 4, 17:30, 2005



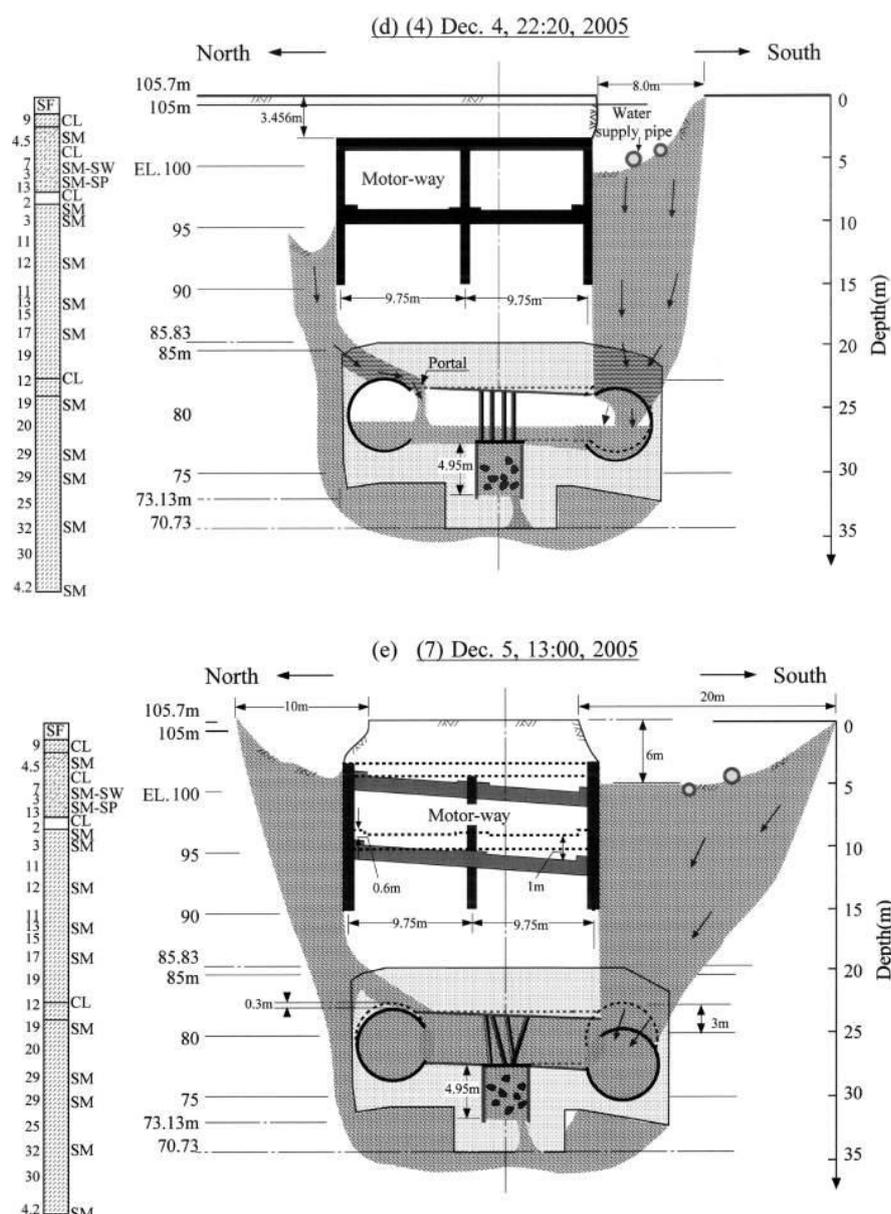


Figure 28. A sequence of likely scenario for progressing the piping failure

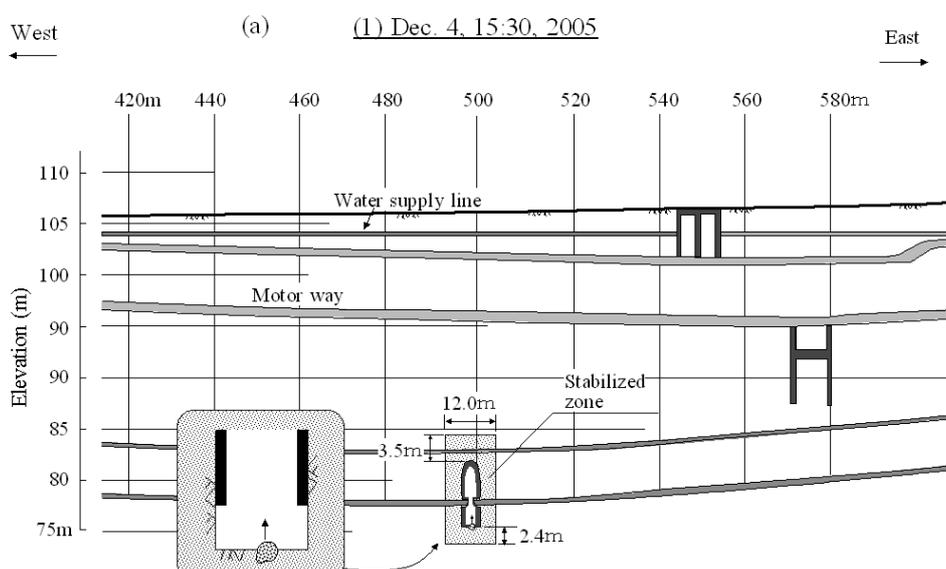
The feature of the collapse propagation as above would be understood more vividly by visualizing the sequence of events that might have occurred in the longitudinal direction. These features are described in Figure 29(a) through 29(d). Figure 29(c) and 29(b) shows the progress of the collapse from 3:30p.m. to 5:30p.m. on December 4, 2005. Figure 29(c) indicates the breakage of the tunnel, segment by segment, resulting in the fall-off the debris from the offsets of the segments in the ceiling on the east side. On the north side undermining of the tunnel body did occur as well as shown in Figure 28(d), resulting in the sinking of the tunnel together with the large block of the stabilized zone. This led to the breakage of the segment joints on the north side and allows the mud water cascading mainly through the opening at the junction between the portal and the adjacent segment. The length of the tunnel in which the debris packed in the entire cross section was as long as 130m on the west side and 80m on the east side in the up-line on the south, as shown in Figure 29(d). Thus, another cave-in occurred on the north

side, as seen in Figure 22. The amount of soils and concrete dumped to fill the caves was $12,000\text{m}^3$ and the volume of water inflow from the 60cm diameter water main lasting about 18 hours until 11:50 p.m. next day was approximately $2,000\text{m}^3$. Thus the total amount of debris was as much as $14,000\text{m}^3$.

3.5 Investigations into causes of failure

After the collapse, the Committee of Investigation was organized by the Kaohsiung Mass Transit Authority and its work commissioned to National Tai-wan Construction Institute in Taipei. Even after comprehensive studies and discussions, it appeared difficult to single out definitive causes with proven evidences. However there were several points agreed upon by the Committee which could be summed up as follows.

- (1) It was obvious from the testimony of the two men at work that the phenomenon of piping was responsible for triggering the collapse.
- (2) The reasons why the piping was initiated were conceived variously. One of them was existence of seepage-prone weak zones resulting from imperfect overlapping in the arrangement of the soil-cement columns created by the jet-grouting method. In fact, because of the obstructions in the ground, it was difficult to install some of the jet-grouted columns exactly in plumb and the overlapping of adjacent columns at the elevation of the piping is estimated less than 60cm.
- (3) The hydraulic pressure at the depth of 34m where the piping was initiated was estimated as being about 300kPa and the length of conceivable shortest path for seepage was 2.4m as counted from the bottom level of the stabilized zone. Thus, the hydraulic gradient at the time of the piping is estimated as having been of the order of $i=30\text{m}/2.4\text{m}=12.5$ which was fairly high.
- (4) Thus, it is expected that the critical hydraulic gradient for unstable seepage was about 12.5, although the reason for this to occur remained unidentified. It was also suspected rather strongly that the silty sand in the area of Kaohsiung might be possessed of a characteristic which is vastly prone to internal erosion as compared to soil deposits existing in other parts of the world. Thus, once the piping develops, the deposit is least self-healing and tends to become easily un-stable.



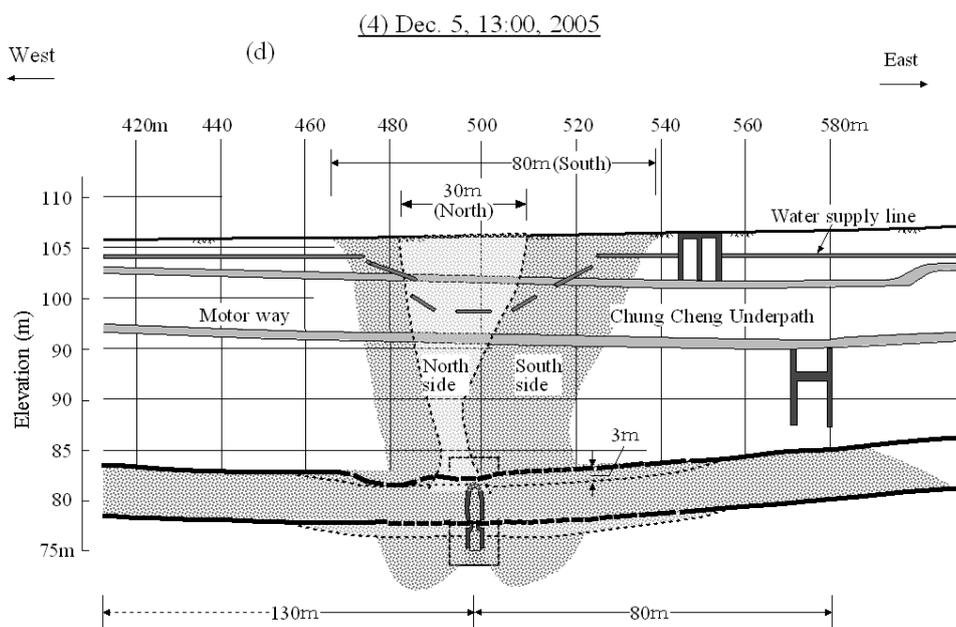
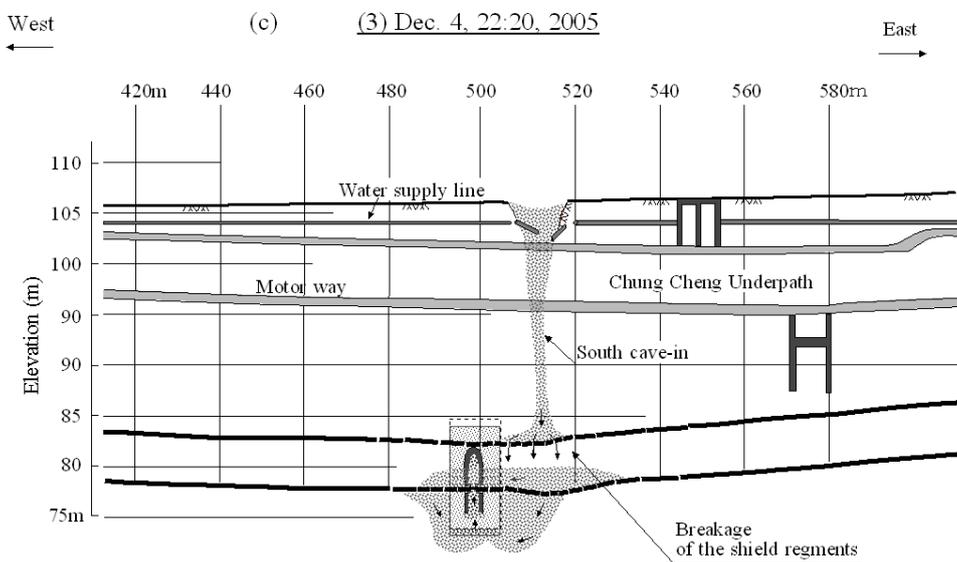
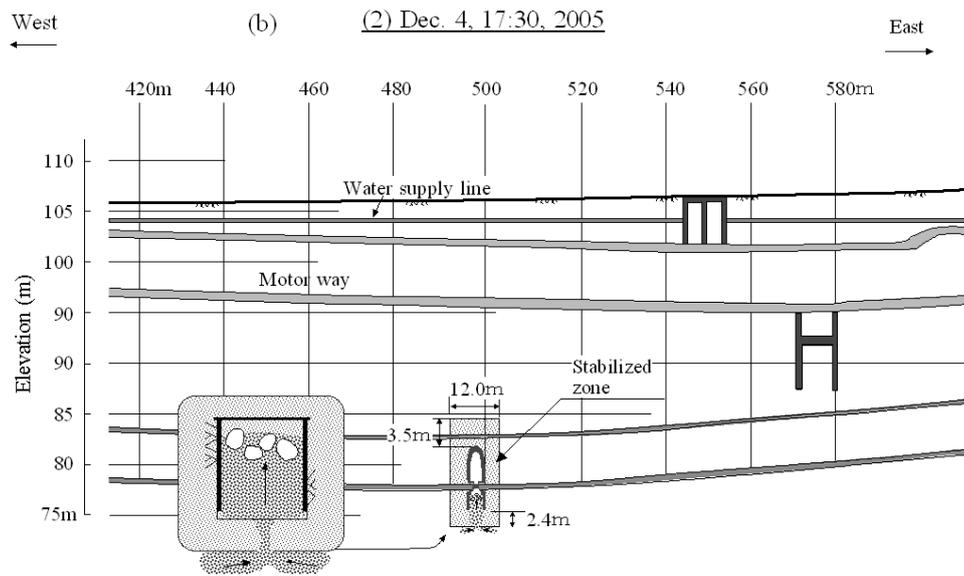


Figure 29. A sequence of likely scenario in the longitudinal direction

3.6 Proneness of local soils to internal erosion

There have been several small collapses reported in other sections of subway construction in Kaohsiung. In fact, it was a concern among geotechnical engineers working there that the fines in sand deposits in Kaohsiung is highly non-plastic and easy to flow. Thus, one of the key issues for forensic diagnosis for piping of the local soil was considered to focus on the investigation into the extremely flowable nature of silts in Kaohsiung area which has not ever been addressed elsewhere.

As is well-known, the susceptibility of a given silty sand to internal erosion can be understood from two points of views, that is, generic reason and durability or resistively.

- (1) If an aggregate of a soil is comprised of two major groups each having significantly different particle sizes, the grains with smaller size can move easily through pores of the matrix formed by larger particles. Thus, if such a soil with a gap grading is subjected to seepage flow, the smaller particles can be easily detached and washed away. This is the generic concept under-lying the criterion for the design of filters in rockfill dams. However, it is normally applied to the type of soils in a range of particle sizes which are larger than those of silt sands encountered in Kaohsiung area.
- (2) The condition as to whether or not a given soil actually suffers the piping collapse is expressed in terms of the critical hydraulic gradient. In fact, the experiments by Skempton and Brogan (1994) showed that the critical hydraulic gradient, i_c , for the occurrence of piping in gap-graded sand-gravel mixture could be as low as 0.2-0.3 as against $i=0.9-1.0$ for clean sands. This would hold true for soils with the range of particle size coarser than the silty sand in Kaohsiung. To the author's knowledge, there appears to be no study ever performed to clarify the vulnerability to piping or internal erosion for silty sands or sandy silts subjected to the seepage flow. Thus, this issue was taken up as a new problem area deserving further scrutiny to identify causes of the collapse in deep-seated excavation.

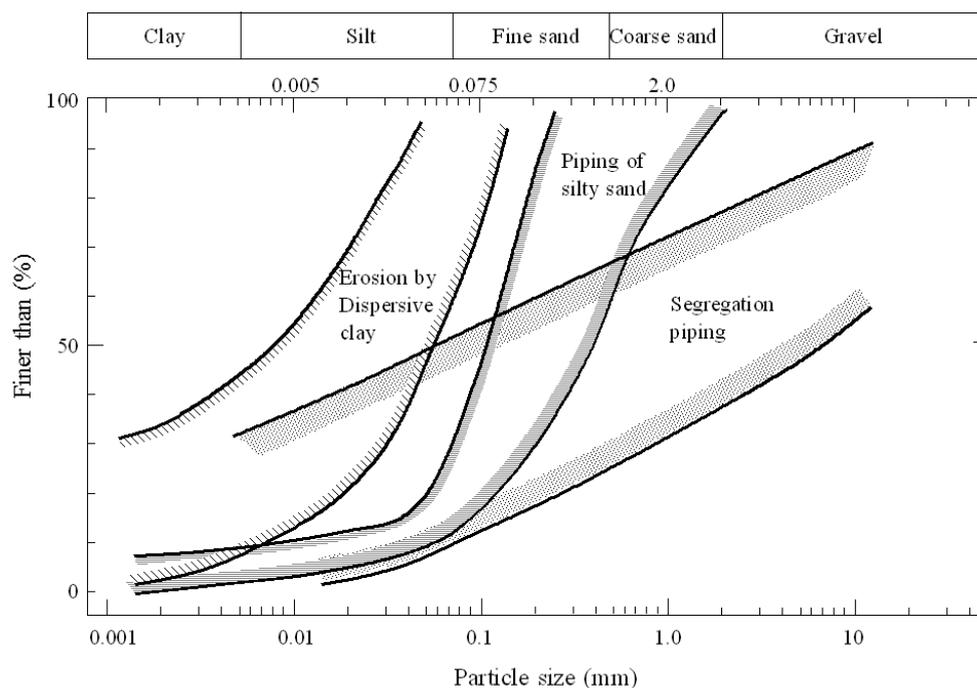


Figure 30. Three groups of soils in terms of grain size range associated with erosion and seepage instability

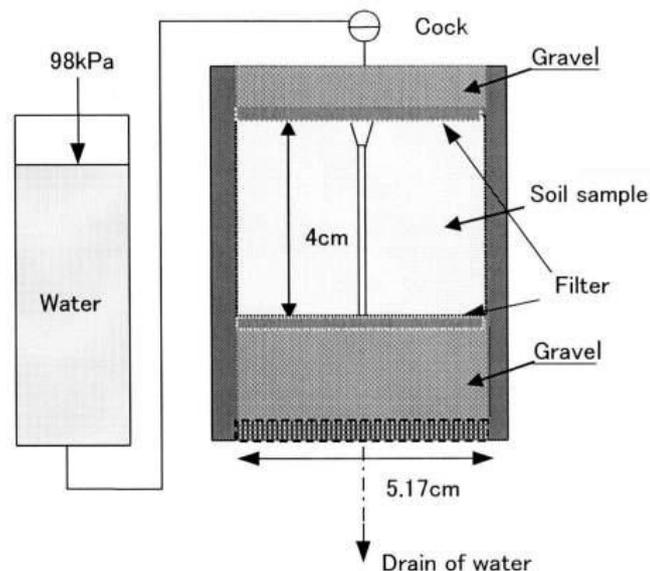


Figure 31. Layout the pinhole test

3.7 Laboratory tests on internal erosion

With respect to the piping or erosion, there would be three types of problems to be distinguished depending upon the range of grain size of soil materials in question. These are shown in a graphical form in Figure 30. The segregation piping has been the target of extensive studies in the past in association with the filter criteria for the design of rock fill dams. There are many studies reported in this context such as those by Terzaghi (1939), Kenny-Lau (1985, 1980), Skempton-Brogan (1994) and Sherad (1979).

Erosion in clay-silt materials has been investigated by Sherad (1979) in response to the problem emerging from occasional failure of low-height earth dams which occurred upon water filling in Australia and U.S. The grain sizes associated with erosion or gully formation has been known to lie in the range indicated in Figure 30. It is known that dispersive nature of clayey soils is a dominant factor for inducing gully or erosion tunnels in the clay fills. Tests for chemical analysis and pinhole tests were suggested as means to identify the dispersive nature of the fine-grained soils.

The outcome of the grain-size analyses for the silty sand at Kaohsiung has shown that the soil there belongs neither to the broadly graded gravelly sands for the segregation piping nor to the dispersive clay related to the erosion or gully formation. In fact, the grain size in Kaohsiung soil lies midway in the range of silt and sand, as shown in Figure 30, which is coarser than the dispersive clay and finer than the soils related with segregation piping.

Thus, the collapse of the silty sand in the subway construction in Kaohsiung was considered to have posed a new challenge and addressed a novel problem area. The solution for this is not yet settled, but performing some tests was regarded as a useful attempt to shed some light on this problem. In this context, a set of pinhole tests and measurements of specific surface test were performed as described below.

(a) Pinhole Tests

In order to examine the vulnerability of the silty sands at Kaohsiung to the internal erosion or piping, what might be called “Pinhole test” was carried out. The layout of the test system is displayed in Figure 31. The sample 4cm thick and 5.17cm in diameter was sandwiched by highly pervious gravel layers at the top and bottom with filter meshes placed between the sample and gravel. A vertical hole 3mm in diameter was drilled as shown in Figure 31. Water was then circulated through the sample. In this type of test, water is supposed to flow mainly through the pinhole. If the silty portion is erodible, the water coming out is expected to be muddy, but otherwise the water transparent.

The silty sand from Kaohsiung tested had a grain size distribution as shown in Figure 32. For comparison sake, another material from a site in Chiba, Japan was secured and used for the same testing. The grading of this Japanese soil was almost the same as indicated in Figure 32. It is to be noticed that both silty sands had an average grain size of about $D_{50}=0.075\text{mm}$ and fines content was about 50%. The physical characteristics of these two materials are shown in Table 1. The two samples were compacted so as to have the same wet density of 1.902 g/cm^3 . A sample from Kaohsiung as pre-prepared for the pinhole test is displayed in the middle of Figure 33 and gravel layers to be placed on top and at the bottom are shown on both sides of Figure 33. The procedures for testing were as follows.

- (1) Water was circulated first slowly with a low pressure through the sample to ensure saturation.
- (2) Water was then circulated under a pressure of 98kPa and colour of water coming out of the sample was observed. Pictures were taken every one-minute. Water percolation was continued for 5 minutes.

In the first series, the tests were conducted on disturbed samples, one from Kaohsiung and another from Japan. The result of the pinhole tests on a disturbed sample is presented in Figure 34. For the disturbed silt-containing sand sample from Kaohsiung, the pinhole is seen enlarging to a diameter of 5mm from 3mm as displayed in Figure 34(a). Correspondingly, the drained water from the bottom was muddy for the first one-minute period of water percolation as seen in the water colour in the left-side cup of Figure 34(b). The result of the pinhole test on the Japanese silty sand is shown in Figure 35. In contrast to the Kaohsiung soil, the circulation of water through the Japanese soil did not exhibit any appreciable change in colour in the first one minute of percolation as seen in Figure 35(b). There was no change in the pinhole diameter for the Japanese soil before and after water permeation except for a local collapse at the top. Thus, it may be conclusively mentioned that the silty sand from Kaohsiung is potentially susceptible to internal erosion as compared to the soil from Chiba, Japan, although they had al-most the same grading. This is believed to stem from the highly non-plastic nature of the Kaohsiung silt contained in the sand.

In the subsequent series, similar pinhole tests were conducted on undisturbed samples. The undisturbed sample from the depth of 14m in Kaohsiung also demonstrated dirty blown colour in the first one-minute of percolation accompanied by the enlargement of the pinhole from 3 to 4mm, as displayed in Figure 36. Two fine sand samples from a deposit at depth 22m in Kaohsiung were tested likewise with the result that it is also vulnerable to internal erosion. From the series of the pinhole tests as described above, it may be concluded that, no matter whether the soil is disturbed or undisturbed, the silt ingredient contained in the Kaohsiung sand is not only non-plastic but also highly erodible being susceptible to piping as compared to other silts with the same gradation such as those in Japan.

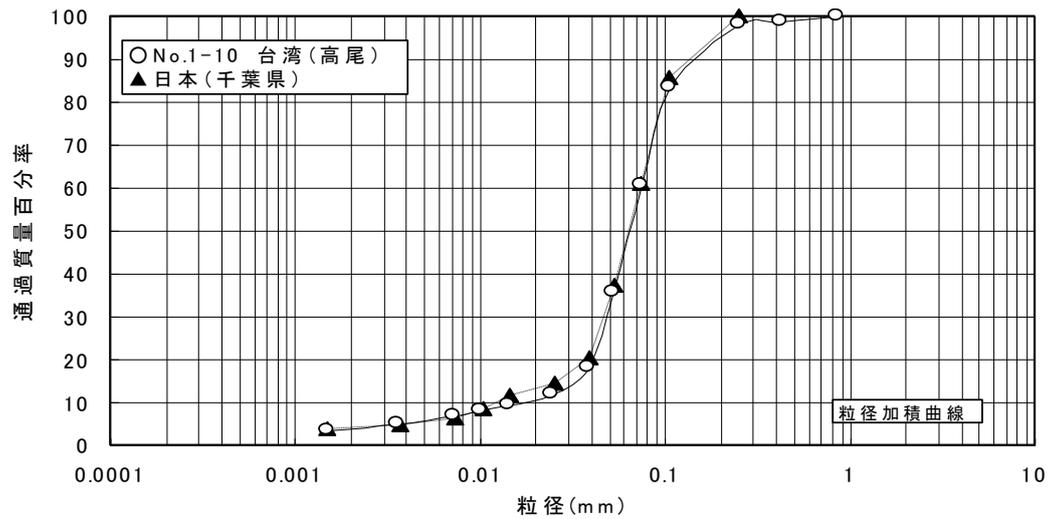


Figure 32. Grain size distribution curves of silty sands used for the pinhole tests

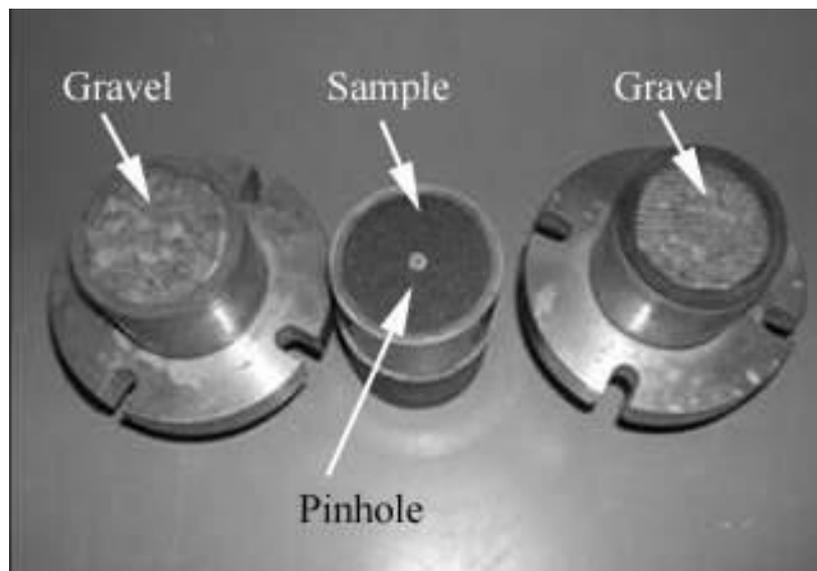


Figure 33. A samples from Kaohsiung for the pinhole test (the middle)

Table 1 Grading of two materials used for the pinhole tests

Material	Kaohsiung sand	Japanese sand from Chiba
Dry density ρ_d (g/cm ³)	1.902	1.902
D ₆₀ (mm)	0.0651	0.0654
D ₅₀ (mm)	0.0746	0.0743
D ₁₀ (mm)	0.0168	0.0124
U _c	4.443	2.51
Specific gravity G _s	2.733	2.708

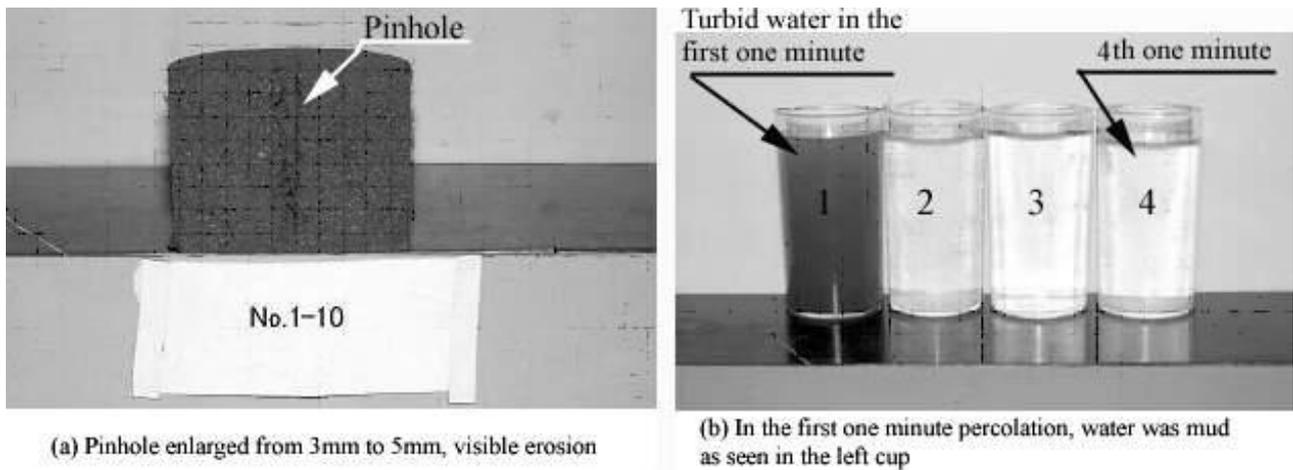


Figure 34. Silt-contained sample from Kaohsiung

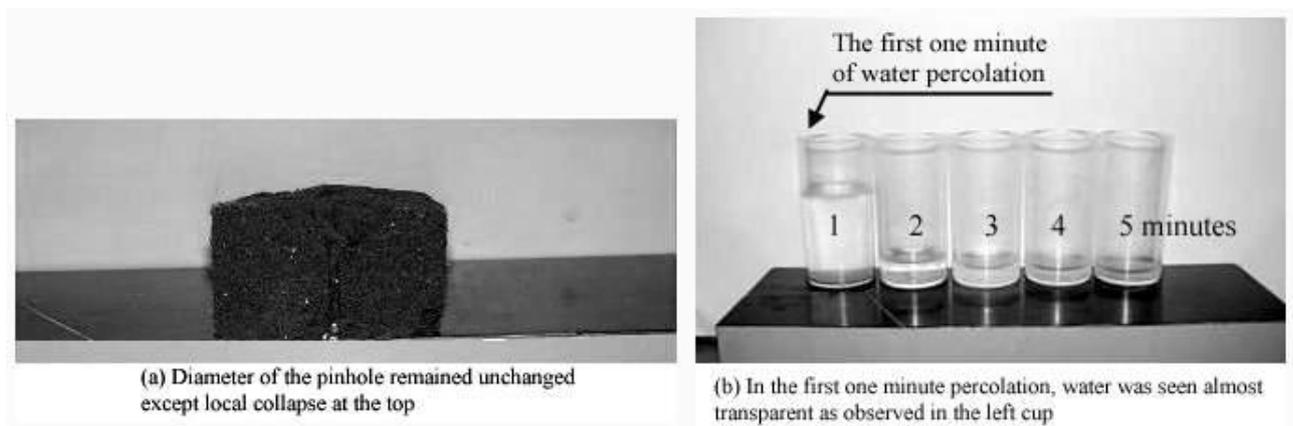


Figure 35. The results of the pinhole test on the Japanese silty sand

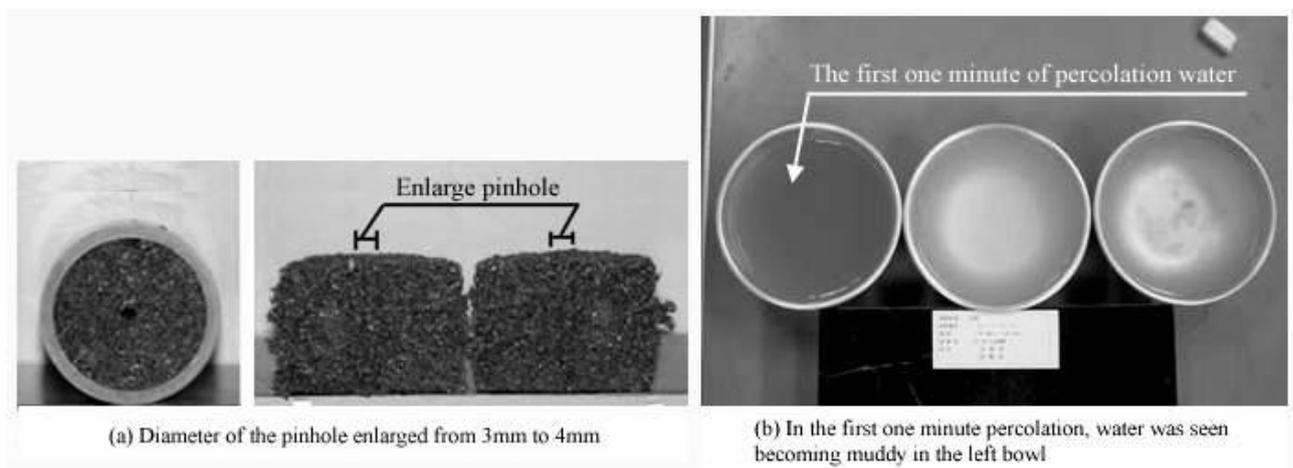


Figure 36. Pinhole tests on undisturbed samples from Kaohsiung

(b) Specific surface test

To obtain another kind of index property for identifying the highly erodible nature of silty sand at Kaohsiung, what is called specific surface tests were conducted. In the Brunauer-Emmett-Teller (BET) method, the surface area is measured per unit weight of silt-size particle and expressed in terms of square

meter divided by the weight in gram. If the specific surface is large, the particle is more of angular shape, and thus envisaged to become more difficult to move through the pores of the skeleton formed by coarser grains. In contrast, the particle of round shape with smaller value of the specific surface would be easier to move in the pores and therefore more erodible. Two tests were performed for the same batch of No. 1-10 silty sand from Kaohsiung having the grading shown in Figure 32. For comparison sake, the tests were conducted also for the Chiba sand in Japan of which the grading is shown in Figure 32. The results of the tests are shown in Table 2, where it may be seen that the specific surface area for Kaohsiung soil is about half of the value for Chiba soil from Japan. In unison with the results of the pinhole tests as mentioned above, this observation of Kaohsiung soil shows that it is considered more susceptible to internal erosion due to seepage flow as compared to other soils.

3.8 Lessons learned from the incident in Kaohsiung

In the practice of subway construction in Kaohsiung, it has been known that the silty sand in that area is of peculiar nature being highly susceptible to erosion due to seepage. Although the silt portion is known to be non-plastic, there has been no way further on to scrutinize the nature of the silt. In an effort to grope for some gauge, what is called pinhole test and specific surface test were conducted for the Kaohsiung silt and also for silt from Japan. The results of these tests have shown that the silt from Kaohsiung is more erodible and has a smaller value of specific surface area than the silt in Japan. This fact is indicative of the tendency of the Kaohsiung silt to be more liable to erosion as compared to other silts. The piping failure in the subway construction in Kaohsiung as described above is considered as a consequence of such a peculiar nature of the local soil which had not explored until now. This incident is to be regarded as a typical example addressing an issue of new challenge in the area of geotechnical engineering.

4. INCIDENTS IN SUBWAY CONSTRUCTION IN SHANGHAI

4.1 Outline of the incidence

A large-scale collapse took place on July 1, 2003 in the south part of Shanghai at a site where a new No.4 line of subway was under construction, leading to a large cave-in and subsidence of the ground. The location of the city near the mouth of Yangzu River is shown in Figure 35. As indicated in a more de-tailed map in Figure 38, the incident occurred at the location on the west bank of the Huangpu River. As shown in Figure 39(a), the curved sections across the river had been constructed by means of the mud-type shield tunneling machine. The river channel is 340m wide and 17m deep at this location as indicated in Figure 39(b). As a result of the cave-in, the city area on the west bank, about 70m wide and 150m long, suffered the ground subsidence of the order of 1-2m, as indicated in Figure 39. The settlement extended into the riverbed, thereby involving the sinking of the river dike accompanied by inundation of the river water. Figure 40(a) shows sinking of two buildings in the forward and tilting of 7-10 storey buildings in the back ground. A high-rising building 30 storey high is also seen tilting in the back ground of Figure 40(b). Figure 40(c) shows the flooding in the city by inundation of water from the river.

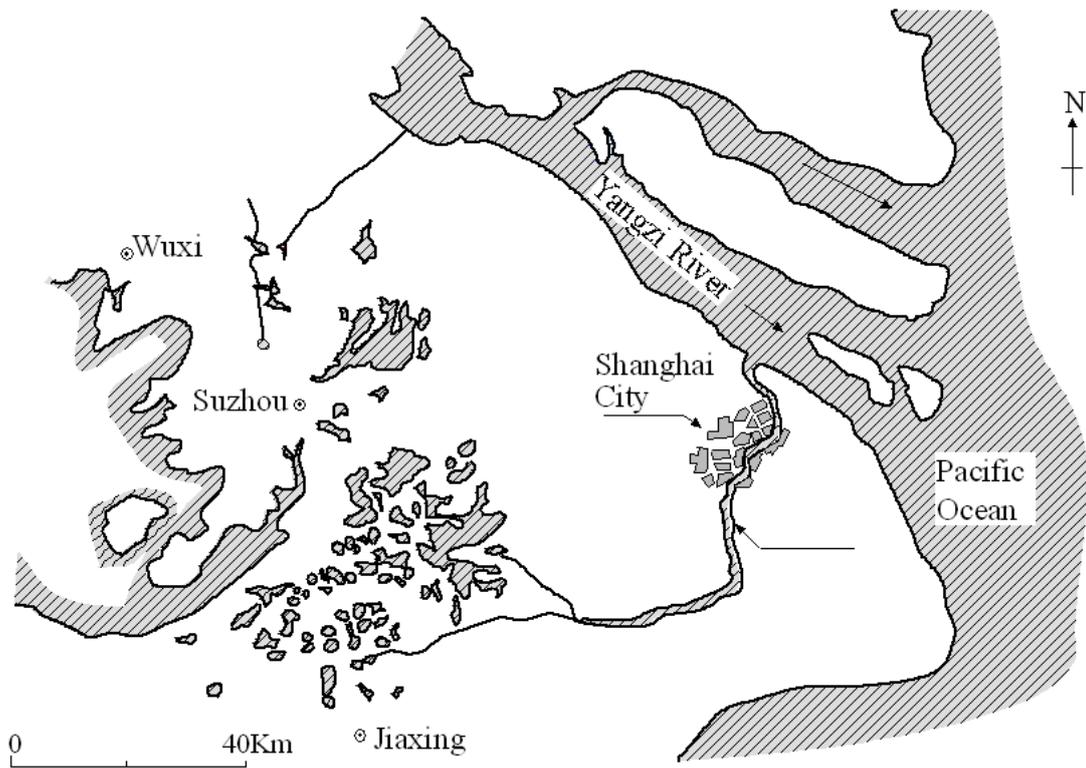


Figure 37. Location of Shanghai

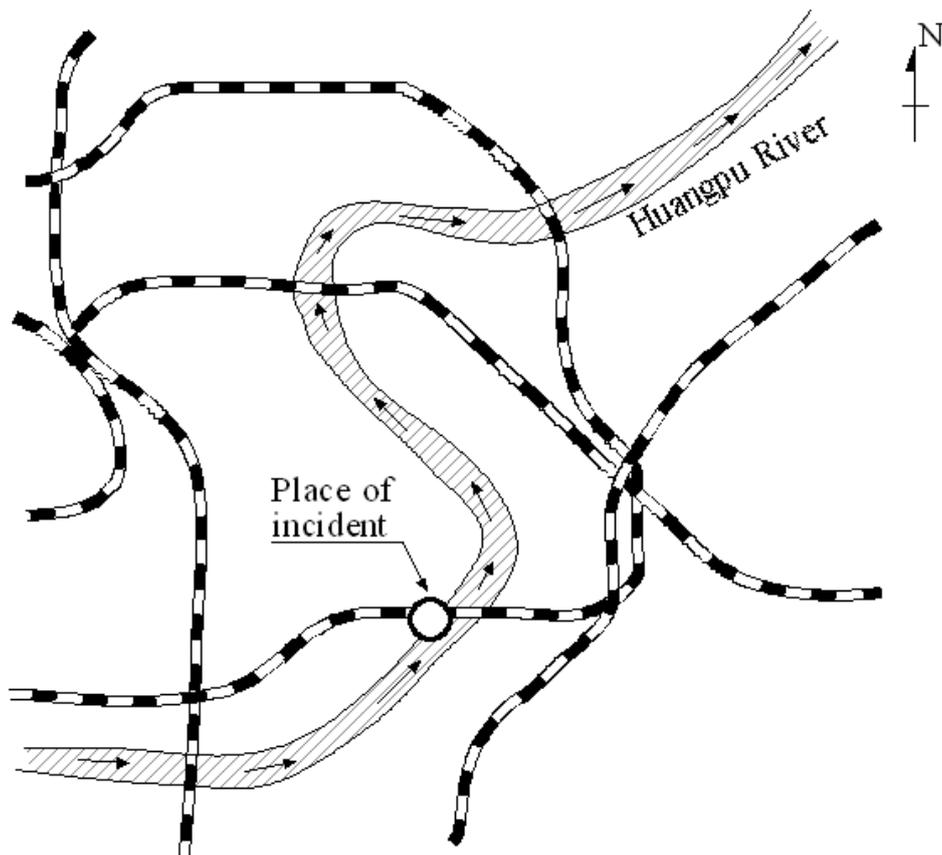


Figure 38. Subway lines in Shanghai and location of the incident

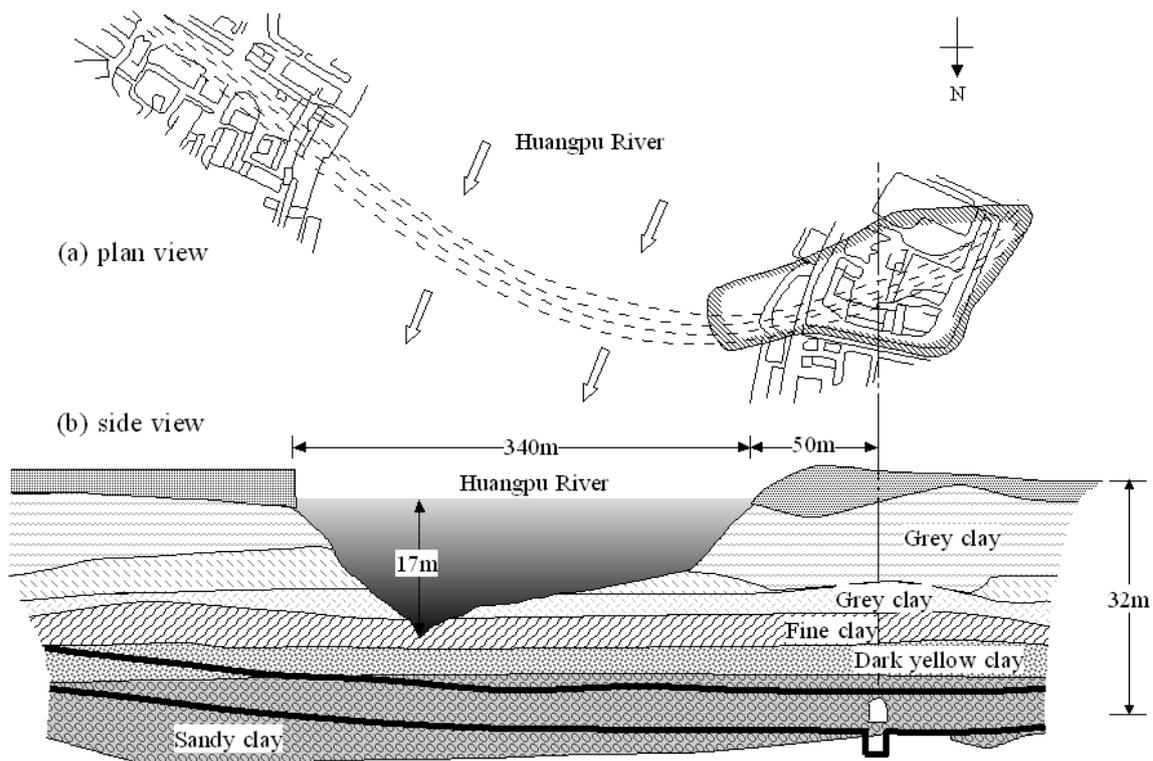


Figure 39. Plan and side views of the tunnel across the Huangpu River

4.2 Features of the incident

Somewhat detailed cross section along the tunnel line is displayed in Figure 41 where it can be seen that the vertical shaft for ventilation purpose had been installed above the corridor. There were two lines of diaphragm walls near the ventilation shaft which had been installed about 50m away to protect the shaft from probable movement of the surrounding ground. There was the connecting corridor and water collecting sump at the depth of about 32m. It is reported, although unofficially, that the excavation was going on for the horizontal corridor through the artificially frozen soil deposits. Because of mal-functioning of machines or cutoff of electric power supply, it is said that the frozen soils lost its strength and breakage took place near the junction between the tunnel and the corridor. As a consequence, a huge amount of water-saturated silty soils flowed down into the tunnel. The collapse propagated up to the ground surface, accompanied by a large cave-in. The area of the subsidence in the city part near the west bank of the Huangpu River is shown in Figure 42. As a result of the ground subsidence, many buildings and infra-facilities for shopping and offices suffered tilting, differential settlements. A high-rise building as tall as 30 stories is said to have experienced a slight tilting.

4.3 Site conditions

The city of Shanghai is located in the deltaic flood plain near the mouth of the Yangzi River. The whole area is covered by a thick fluvial deposit of alluvial or Holocene origin as deep as about 40m predominantly comprised of silt, clay and occasional sand. This alluvium is underlaid by the Pleistocene or diluvial deposit to a depth of about 70m. The whole soft soil deposit consists of multi-layers of dark grey silty clay and yellowish fine sand. The plasticity index of the silty clay is known to be about 15 and its cohesion is 15 to 25kPa.



(a) Sinking and tilting of buildings in the area of the ground subsidence



(b) Tilting of nearby building



(c) Flooding due to breach of the river dike

Figure 40. Damage by ground settlement due to underground collapse in underground excavation, Shanghai

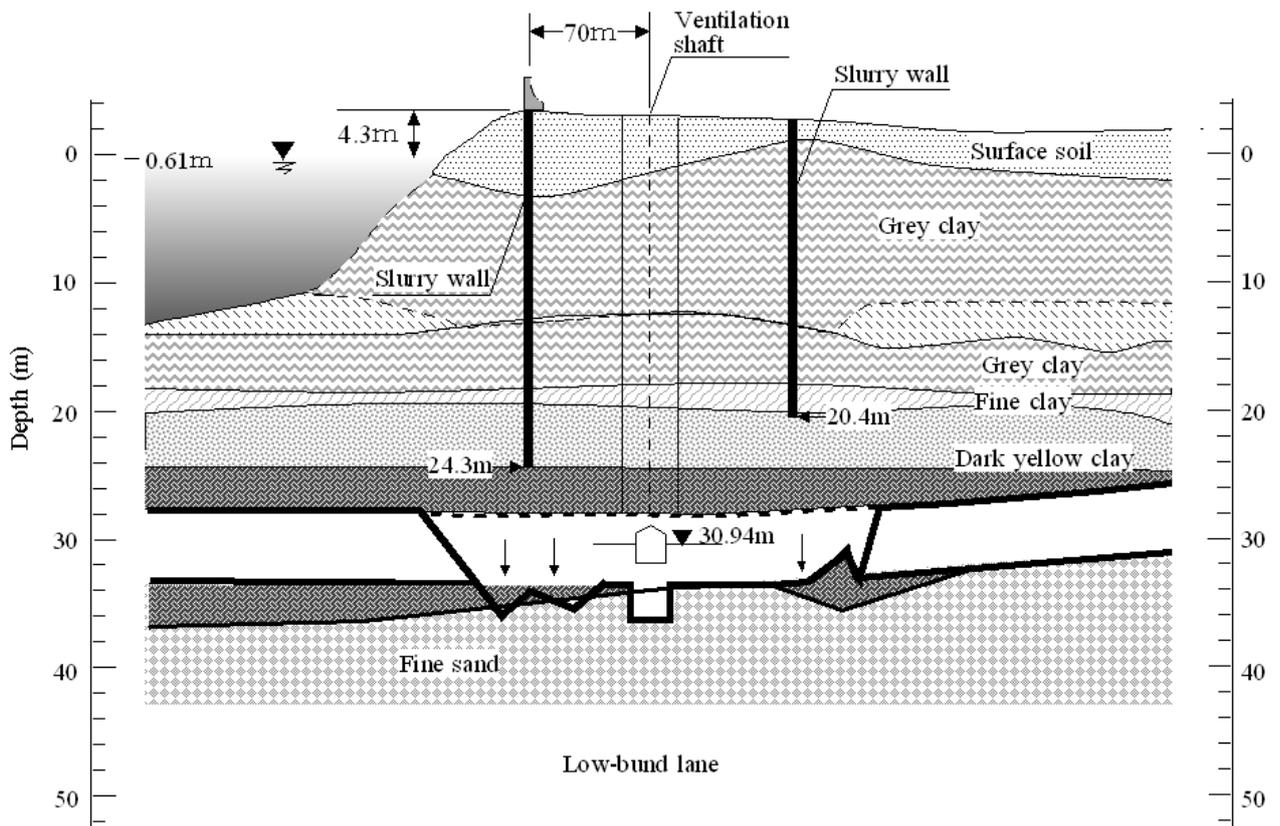


Figure 40. Cross section along the subway line showing the feature of the collapse

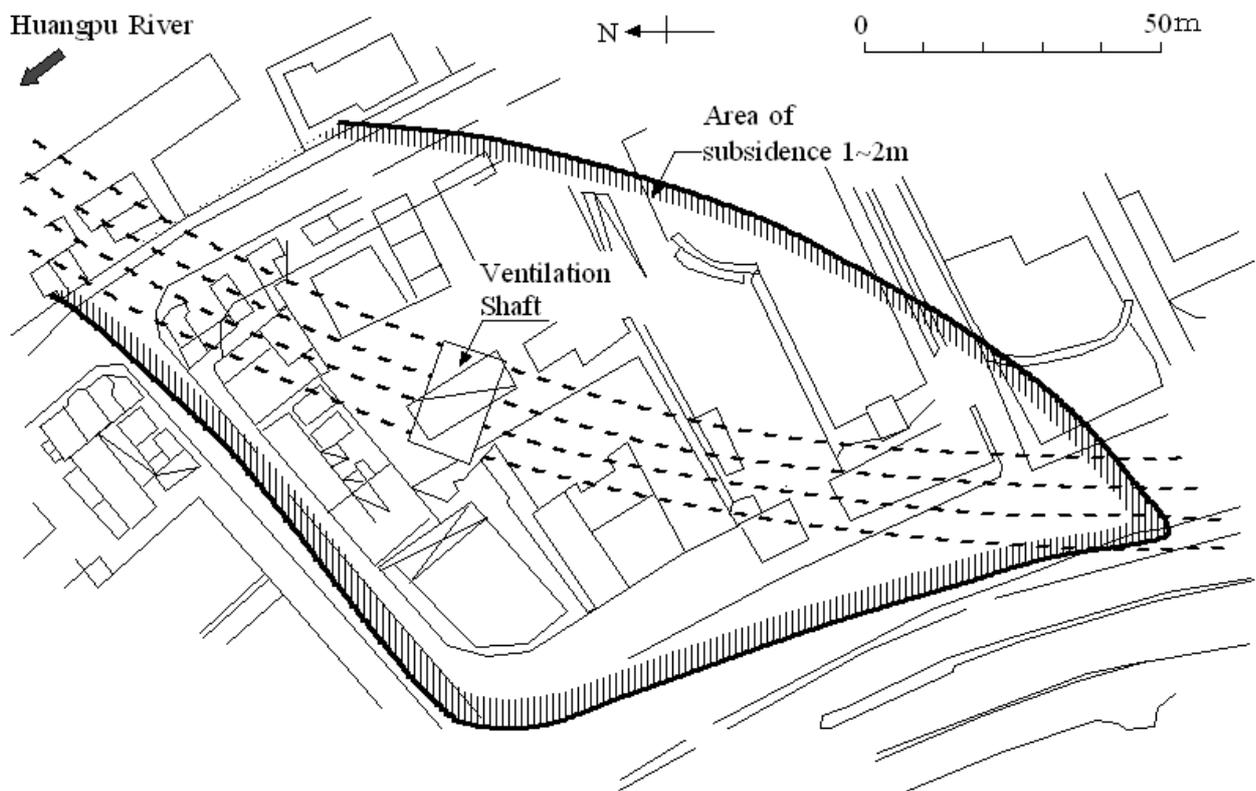


Figure 42. City area of the ground subsidence on the west bank of the Huangpu River

4.4 Occurrence of collapse

It is purported that the construction of the dual tunnels and the installation of the ventilation shaft had been finished and the excavation was underway at the depth of 30m to construct the corridor connecting the up-and down-lines of the tunnels. The silty sand deposits around the corridor tunnel had been frozen, in advance, by circulating coolant by a pipe system to permit the cutting to be made in open and dry conditions. When the excavation proceeded to the place of junction between the tunnel and the corridor, it is reported that malfunctioning or electricity cut-off occurred, resulting in thawing of the frozen soils. It seems likely that the saturated soils fell down and a kind of piping might have taken place where a small hole became gradually enlarged, leading to the large-scale fall-off and flow-in of the saturated soils.

There are no data available for the authors concerning measured values of SPT or CPT indicative of stiffness of silty sands at the location of the incident. It is, however, generally reported that the sandy silts in Shanghai are of low plasticity stiffness with the SPT N-value of 5-15 at shallow depth.

For the reference sake, the specific surface test was conducted on disturbed samples of silt from Shanghai, together with other soils as described in the foregoing section 3.7. The results of the test are listed in Table 2 where it can be seen that the specific surface in the BET-test takes a value of about 4.3 which lies between the two silts, that is, one from Japan and another from Kaohsiung. It may be mentioned that the silt from Shanghai could be somewhat more prone to internal erosion than the silt from Chiba, Japan.

Judging overall from the information as above, it may be mentioned that the silty sands or sandy silts prevailing over the Shanghai area are more or less susceptible to a piping type failure if they are subjected to the seepage field, and this was one of the factors accounting for in the enlarged scale of the catastrophe which occurred at the subway construction in Shanghai.

Table 2 Results of specific surface tests

	Sample 1 (m ² /g)	Sample 2 (m ² /g)	Average (m ² /g)
Kaohsiung silty sand , No1-10	3.2669	2.9540	3.1100
Silty sand from Chiba, Japan	6.2228	6.1469	6.1849
Silt from, Shanghai	4.4584	4.1095	4.2840

5. CONCLUSIVE REMARKS

The three examples of collapse as described above have some features in common, that is, (1) they occurred in the area of soft soil deposits of alluvial origin and (2) they are all associated with deep excavations at depths 30 to 35m.

The case of the collapse in Singapore appears to come from the presence of an unforeseen buried valley which is generally difficult to detect in advance at the spacing interval of boring currently in use.

This incident gives us a lesson for needs for thorough investigations of site-specific soil conditions, if found necessary, in view of complexity of local geology and its history of formation.

The collapse in Kaohsiung was induced by unpredictable occurrence of piping in the sandy silt de-deposit subjected to a seepage field with a high hydraulic gradient. Although difficult to foresee from many past experiences in other sites, the local sandy silt in Kaohsiung was found to possess a specific characteristics in that it is more prone to the piping due to internal erosion than other silts with similar grading. Thus, the incident in Kaohsiung left a message that a local soil could possess specific properties which are difficult to identify from existing knowledge generally incorporated in the design and practice in deep excavations in deposits of low-lying area. It is necessary to investigate characteristics of local soils if they are found to exhibit peculiar behavior judging from the common sense in soil mechanics.

There is no exact information available on soil characteristics in Shanghai, but it seems likely that the low-plasticity nature of the local soil was prone to failure such as piping at deep depths under high hydraulic pressure and flowable nature of the local soil might have perhaps been responsible particularly for enlarging the scale of the incident. Once again, it must be emphasized that nature of local soils be investigated in details and its outcome be incorporated meticulously in the design and operation of construction works.

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