In Situ testing, Soil-Structure Interaction, and Cost Effective Foundation Design

The Fourteenth Spencer J. Buchanan Lecture

By *Cly*de N. BAKER Jr.

And

Evolution of Risk Management as a Project Management Tool in Underground Engineering

Special Lecture

by William HANSMIRE

Friday November 17 2006
College Station, Texas, USA

http://ceprofs.tamu.edu/briaud/buchanan.htm

SPENCER J. BUCHANAN, SR.

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.

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2005	Thomas D. O'Rourke	"Soil-Structure Interaction Under Extreme Loading Conditions"

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

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You may also visit the website http://ceprofs.tamu.edu/briaud/buchanan.htm

AGENDA

The Fourteenth Spencer J. Buchanan Lecture
Friday November 17, 2006
College Station Hilton

2:00 p.m.	Welcome by Jean-Louis Briaud
2:05 p.m.	Introduction by John Niedzwecki
2:10 p.m.	Introduction of William Hansmire by Jean-Louis Briaud
2:15 p.m.	“Evolution of Risk Management as a Project Management Tool in Underground Engineering” Special Lecture by William Hansmire
3:15 p.m.	Discussion
3:25 p.m.	Introduction of Clyde Baker by Jean-Louis Briaud
3:30 p.m.	“In Situ testing, Soil-Structure Interaction, and Cost Effective Foundation Design” The 2006 Buchanan Lecture by Clyde Baker
4:30 p.m.	Discussion
4:40 p.m.	Closure with Philip Buchanan and Spencer Buchanan Jr.
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 inTaipei, 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 SEAOI 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 SEAOI 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.

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.

William H. Hansmire

Dr. William Hansmire received his BSc from the University of Nebraska in 1968, his Master from the University of Illinois in 1970, and his PhD from the University of Illinois in 1975. He is a registered Professional Engineer is 10 States and a Member of many national tunneling and other civil engineering organizations including ASCE, ARMA, AEG, and ITA. He was elected to the National Academy of Engineering in 2002.

A senior member of Parsons Brinckerhoff's geotechnical and tunneling staff, Bill Hansmire remains at the forefront of tunneling industry practices in the United States. During his more than 30 years experience in the field of underground engineering he has had a wide range of responsibilities and positions ranging from project engineer, project manager, to principal-in-charge. His project experience includes U.S. and international roadway, heavy rail, transit, water, and wastewater projects. Projects have been by traditional and design-build contract delivery methods.

He first started with Parsons Brinckerhoff (PB) in the New York office in 1977. Major projects included the Strategic Petroleum Reserve (SPR); Harvard Square Station, Cambridge, Massachusetts; and the 14-km-long Rogers Pass Rail Tunnel in British Columbia, Canada. He moved in 1984 to Hawaii as Project Manger for the Interstate Route H-3 Trans-Koolau Tunnel and then moved in 1993 to Las Vegas as the Engineering Manager for the Kiewit/PB contractor/engineer team that constructed the 8-km-long Exploratory Studies Facility tunnel for the Yucca Mountain Nuclear Waste Storage Project (YMP). From 1996 until 2003, he was with Jacobs Associates in San Francisco, a firm specializing in tunneling and construction engineering. He was Principal-In-Charge and led all tunnel designs for Tren Urbano new transit system in San Juan, Puerto Rico, the first major tunnel project in the United States to be completed under a design-build contract delivery. In 2003 he returned to PB in the San Francisco office as the Engineering Manager for preliminary design of the San Francisco Muni Central Subway.

With the strategic purpose of developing the firm's water and wastewater business in the Midwest United States, he relocated to Detroit, Michigan in 2005. As Project Manager, he will lead design and construction management services for the Detroit River Outfall No. 2, a difficult 1.3-km-long, 7-m-diameter rock tunnel that will encounter 6 bar ground water pressure and gassy geologic conditions. This is the second attempt to design and construct this tunnel, which suffered a catastrophic failure and flooding in 2003.

Dr. William Hansmire lives in Detroit where he is the Senior Vice President of Parsons Brinckerhoff Quade & Douglas, Inc.

In-Situ Testing, Soil Structure Interaction and Cost Effective Foundation Design

By: Clyde N. Baker, Jr., P.E., S.E.
Senior Principal Engineer

Abstract

Cost effective foundation design may entail use of mixed foundation systems which can require more accurate and reliable load deformation prediction capability. In the writer's experience, the Menard in-situ pressuremeter test has been very useful in enabling better load deformation prediction with different foundation support systems. Case histories are presented to illustrate five different design concepts:

1) Complex existing foundation conditions required special dewatering efforts to permit belled caisson construction below the old belled caissons.
2) Use of piles or drilled shafts as settlement reducers rather than as required structural elements for building support.
3) Use of deeper basement excavation stress release effects in ways to maximize site building capacity.
4) Use of variable length piles under a mat to minimize differential settlement.
5) Re-use of existing deep foundations on one bearing stratum in combination with new foundations on a deeper stratum.

Introduction

The objective of this paper is to illustrate how, in the writer's experience, in-situ testing with the pressuremeter has been helpful in better predicting how the ground responds to building loads, particularly in preconsolidated soils, and how, with cooperative structural engineer and geotechnical engineer interaction, this can facilitate cost effective and innovative foundation design.

Historically, structural engineers have been reluctant to mix foundation type or foundation levels on the same structure for fear of potential differential settlement and their lack of confidence in settlement predictions.

This paper describes several highlights of early pressuremeter use in Chicago and then presents five selected case histories to further illustrate the paper objective. Since the selected case histories have been published elsewhere (see reference list) only the portions of the papers necessary to illustrate the objective are available here.

The Menard pressuremeter was first introduced in the Chicago area in 1969, and along with Menard Empirical Rules for using and interpreting the data obtained from the test, was immediately found to be helpful in developing more economical foundation designs particularly for belled caissons on hard clay or hardpan. The typical downtown Chicago soil profile is shown in figure 1 with the typical potential foundation types indicated on the profile.

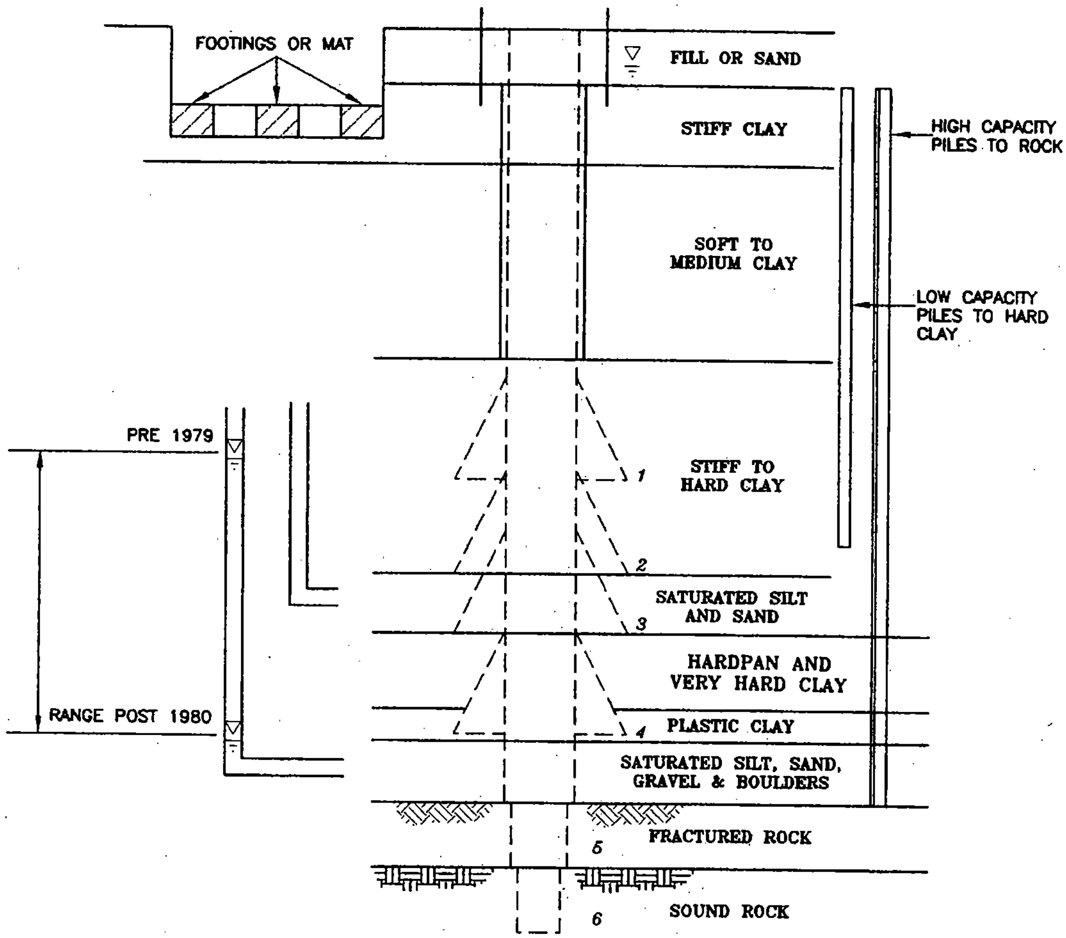


FIG. 1 TYPICAL SOIL PROFILE FOR DOWNTOWN CHICAGO – (3)

Prior to 1969, foundation design bearing pressures were typically based upon unconfined compression tests performed on samples obtained either by 2" (50.8 mm) or 3" (76.2 mm) Shelby tubes and 2" (50.8 mm) OD split barrel samples obtained following ASTM specifications

D 1587 and D 1586, respectively. The maximum allowable bearing pressure on good Chicago hardpan had increased gradually from 12 ksf (574.6 kPa) (the typical design value prior to the Depression and World War II) to a maximum of 30 ksf (1436 kPa) at the 65 story Lake Point Tower project built in 1965. This was based upon the Skempton theory (1951) that the ultimate tip capacity for a deep foundation in clay was 9 times the cohesion requiring a cohesion of 10 ksf (479 kPa) for a factor of safety of 3. Since unconfined compression tests sometimes failed to yield the necessary 20 ksf (958 kPa) unconfined compressive strength (required for a factor of safety of 3) due to silt sand and gravel content in the hardpan, triaxial compression tests were necessary to confirm the design bearing pressure. While triaxial testing could be performed to demonstrate significant friction angles in the hardpan, theoretical bearing capacities at great depths became unrealistically high. In addition, the prediction of settlement appeared even less reliable.

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. Allowable bearing pressures on good hardpan increased from 30 ksf (1436 kPa) in the early seventies to 50 ksf (2390 kPa) in the late eighties.

Determination of Pre-Consolidation Pressure

Early research by Lukas and Debussy and others (1976) indicated that the creep pressure determined during the performance of the in-situ pressuremeter test compared favorably to the preconsolidaton pressure determined from well run consolidation tests. One of the difficulties of determining preconsolidation pressure from consolidation tests in glacial till is the difficulty of testing a sufficiently undisturbed sample to provide a sharp break on the void ratio versus pressure curve, thereby leaving considerable room for interpretation. The creep pressure from the pressuremeter tests appeared to be simpler and more reliably determined with consistency.

Settlement Theories Using Pressuremeter Test Data

The two most common approaches for predicting settlement using pressuremeter data in our experience are the Menard semi-empirical procedures described by Menard (1975) and Briaud (1992), and the 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 somewhat with the stress and strain leve (as well as Poisson's ratio), a theoretically correct approach would involve special tests at the stress/strain level anticipated in each soil strata below the bearing level.

In the case of the approach using the Menard rules, the settlement formula is:

$$S = (1.33/3E_b)\ p\ R_O\ (\lambda_2 R/R_O)^{\alpha} + (\alpha/4.5E_1)\ p\ \lambda_3 R$$

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

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_o= Reference radius (11 feet or 30 cm)

α = Ed/E+

Menard recommends α factors to use in the formula, as noted in Table 1. However, in our practice, we modify these factors based upon the actual test results, assuming α = the initial pressuremeter modulus E_d divided by the reload modulus E+, except we do not use an α less than 0.5 for clay nor less than 0.4 for silt, nor less than 0.33 for sand or sand and gravel. When running the pressuremeter test, we typically run 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.

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

	Peat		Clay		Silt		Sand		Sand and Gravel	
Soil Type	E/P_l	α	E/P_l	α	E/P_l	α	E/P_l	α	E/P_l	α
Overconsolidated			>16	1	>14	⅔	>12	⅓	>10	⅓
Normally Consolidated	For All Values	1	9-16	⅔	6-14	½	7-12	⅓	6-10	¼
Weathered and/or remoulded			7-9	½		½		⅓		¼
Rock	Extremely Fractured α ≈ ⅓		Other α ≈ ½		Slightly Fractured or Extremely Weathered α ≈ ⅔					

$E =$ The pressurementer modulus of the soil assumed to be homogenous

$P_l =$ The limit pressure determined from the pressuremeter test

In the case of the elastic theory approach, the formula used is:

$$\text{Settlement } s = \frac{\mu_o \mu_1 Bq}{E}$$

where:

q = bearing pressure

B = foundation width

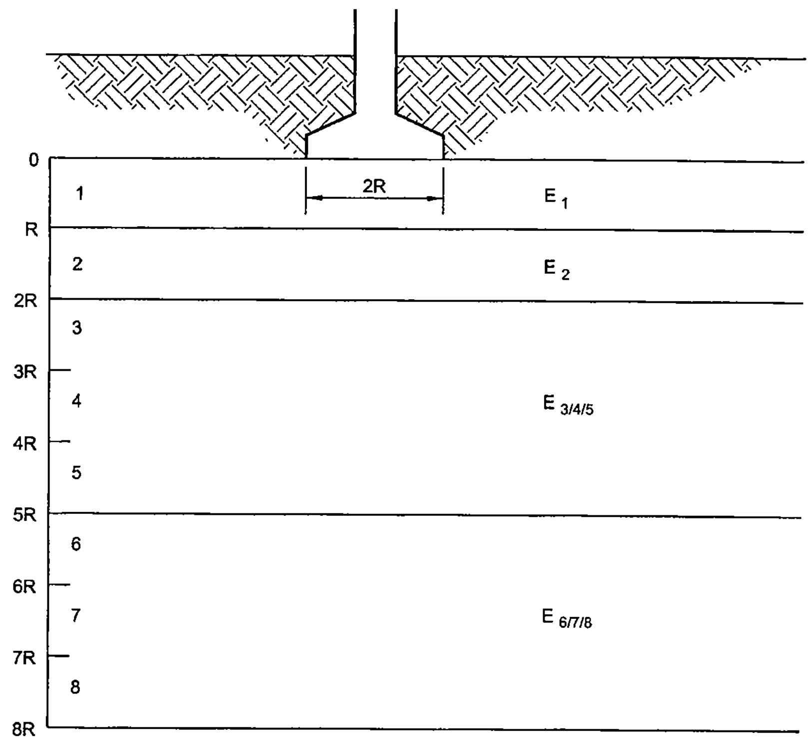
E = modulus of elasticity

$\mu_o\mu_1$ = geometry influence factors as indicated in Figure 3.

In both settlement prediction theories, it is assumed that the stress level is within the pseudo elastic range which in pressuremeter terminology means the total stresses must be below the creep pressure.

Caisson Load Tests and Correlations With Prediction From Pressuremeter Test Results

Performance of limited historic caisson load tests in Chicago compared with what might have been predicted using pressuremeter tests is presented in Reference 5 with a tabular summary shown in Table 2 below.



EQUAL OR INCREASING E

$$E = \frac{3.6}{\frac{1}{E_1} + \frac{1}{0.85E_2} + \frac{1}{E_{3/4/5}} + \frac{1}{2.5 \times E_6 / E_7 / E_8}}$$

LAYERS OF SOIL UNDER A FOOTING TAKEN INTO CONSIDERATION FOR THE COMPUTATION OF EQUIVALENT MODULI
(MENARD, 1975)

FIGURE 2

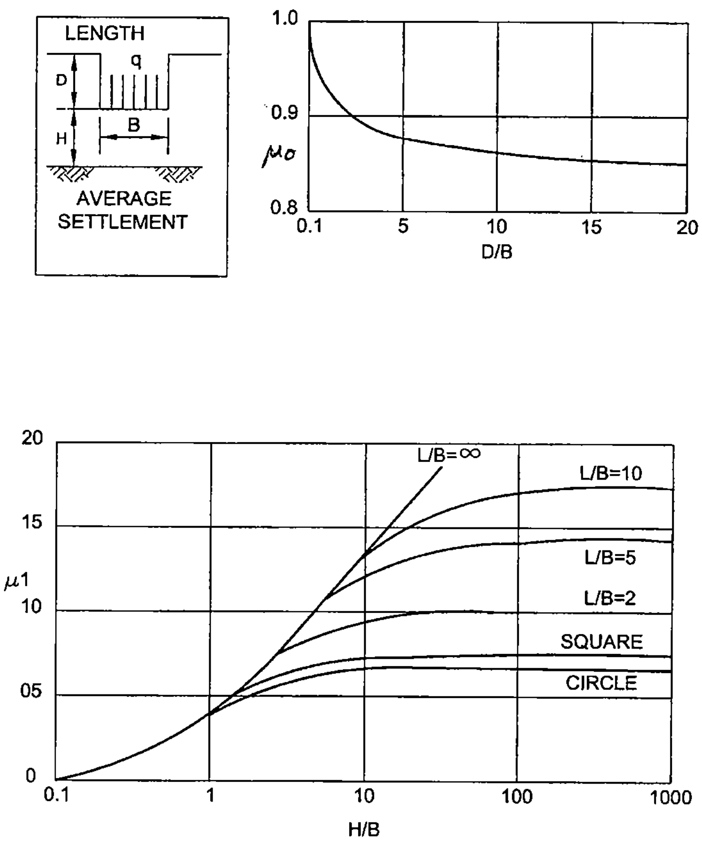


CHART FOR ELASTICITY INFLUENCE FACTOR
(CHRISTIAN, CARRIER 1978)

FIGURE 3

TABLE 2

Test Location	Caisson Diameter	Caisson Elevation	Maximum Test Load Bearing Pressure	Observed* Settlement of Base	Observed* Settlement @ ½ Max. Bearing Pressure	Average Pressure-meter Modulus in TSF E_a	Average Pressure-meter Modulus in TSF E_b	Pressure-meter** Settlement at ½ Max. Load Bearing Pressure	Ultimate Capacity On: Pressure meter	Ultimate Capacity On: 9 x C
Union Station 1	8.2'	-60.0±	18.4 tsf	0.75"	0.3"	335	335	0.33"		
Union Station 2	4.2'	-60.0±	61.0 tsf	2.0"	0.9"	335	335	0.88"	85.0 tsf	36 tsf
One Park Place	6.3'	-67.4±	24.0 tsf	1.4"	0.4"	247	320	0.55"	54.4 tsf	27 tsf
Univ. of Chicago	2.5'	-38.0±	50.0 tsf	2.2"	0.45"	460	460	0.41"	48.6 tsf	52 tsf

Conversion Key: 1 Ton Per Square Foot (tsf) = 95.8 kilopascals (kPa)
1 inch (IN) = 25.4 Millimeters (mm)

*First Load Only
**Based on Menard Rules and using $\alpha = +0.5$

From this we can conclude that the settlement magnitude under a given load within the normal working load range can be reliably predicted on highly preconsolidated glacial till (Chicago hardpan) using appropriate in-situ pressuremeter test results and current pressuremeter theory.

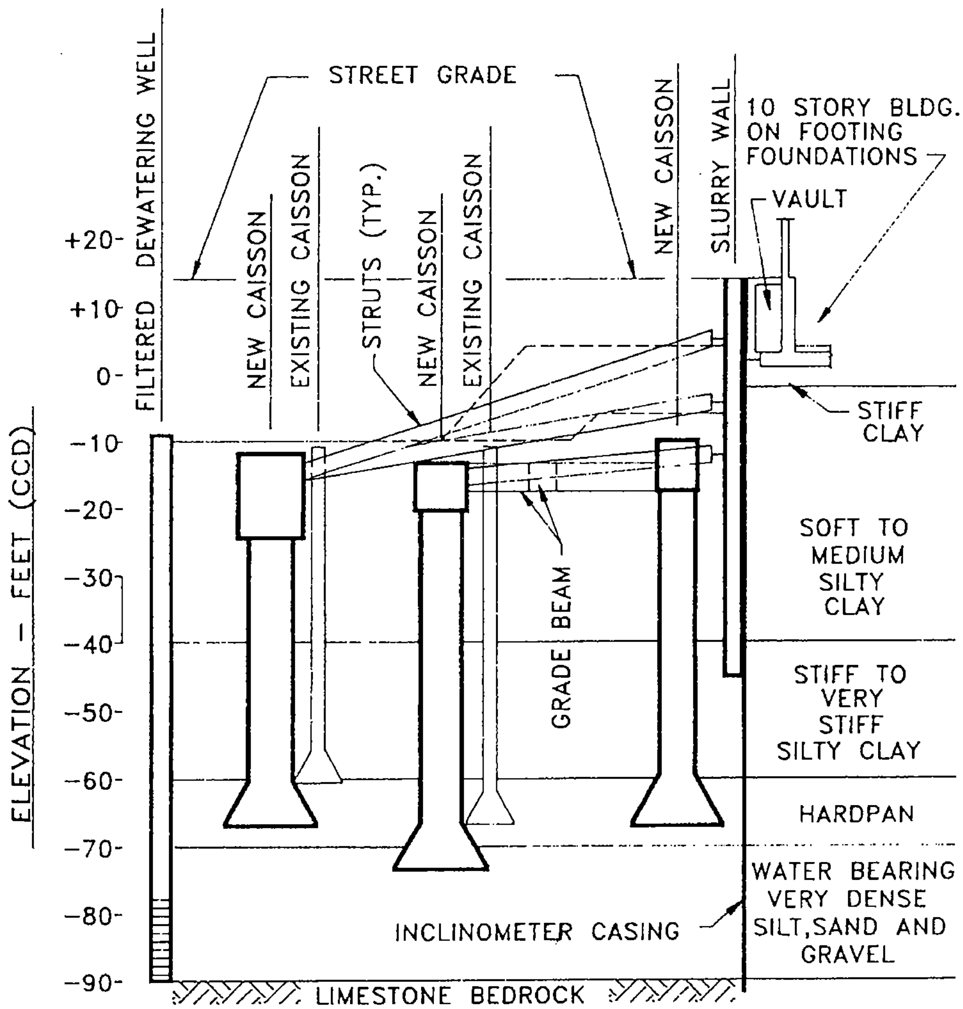
Correlation With Building Performance

In the early use of the pressuremeter much confidence was gained when predicted settlement of the then tallest reinforced concrete building in the world (75 story Water Tower Place) matched closely the measured settlement after construction (2.0 inches vs. average of 1.94 inches with a range of 1.69-2.19 inches). In the following five Case Histories where prediction has also been compared with observed performance, the more complete details have been presented in earlier papers listed in the references. In the summarized presentation here we have elected to reprint figures and tables as labeled in the papers and to present them with each Case History. The intent is to illustrate how increasing confidence in our ability with the pressuremeter to better predict ground deformation under load has facilitated innovative foundation design involving soil structure interaction.

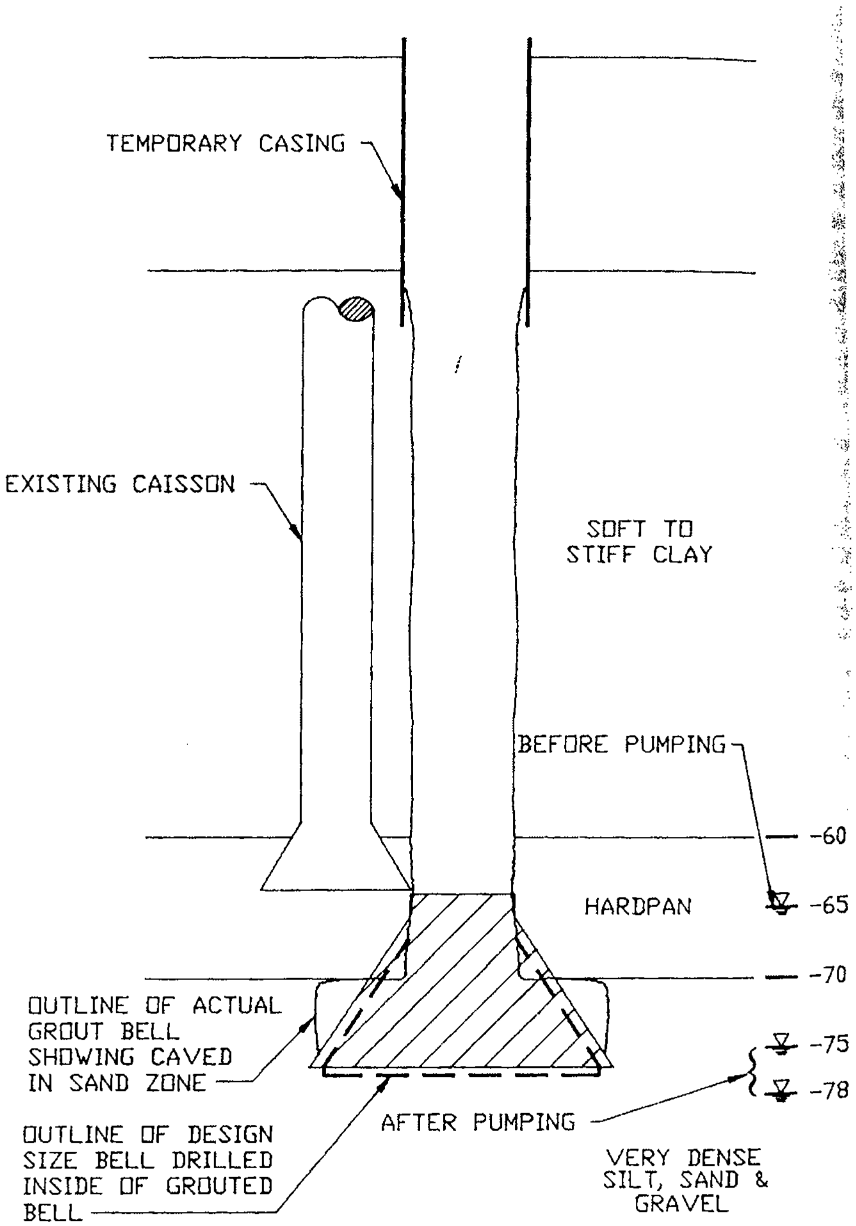
1. AT& T Case History

The next three figures summarize the AT&T case history (Reference 17). AT&T was the first building in Chicago to utilize a bearing pressure of more than 40 ksf on the underlying Chicago Hardpan. 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. However, rock socketed caissons are typically three times the cost of belled hardpan caissons so every effort was used to come up with a belled caisson solution. Because of the presence of the existing caissons and the large bells required by the very heavily loaded new caissons (around 14,000 kips), it was necessary to extend the biggest diameter belled caissons in the core through the hardpan and into a very dense waterbearing silt, sand and gravel. Pressuremeter testing indicated that 45 ksf bearing could be used in the hardpan and as much as 50 ksf bearing could be used in the underlying very dense silt, sand and gravel if it was possible to construct bells there. This required the use of filtered dewatering wells which lowered the water table so that the bells could be constructed. However, it was not known whether the bell walls would remain stable in the silty sand and gravel even though dewatered so a contingency plan was developed as shown in the second figure which involved constructing an oversized bell that would be filled with grout of such strength that the next day a design sized bell could be constructed within the grout bell. This approach had been used successfully on other projects, when necessary. The owner's representative was made aware of this contingency plan and that it would involve additional costs if it occurred. However, in this project it proved not to be necessary as the surface tension in the water provided enough apparent cohesion to the very dense silt, sand and gravel to keep it stable after dewatering, even when excavated for the bell.

AT&T Corporate Center Substructure Profile



AT&T "Grouted Bell" Method of caisson construction in caving soil



The building was successfully completed and settlement measurements taken during construction to compare with the predicted measurements based on pressuremeter testing. The results are shown below and indicate that the observed settlement was even less than the predicted settlement based on pressuremeter testing lending further support to not only the utilization of the pressuremeter test but of using high bearing pressures in the 45 to 50 ksf range in the best hardpan.

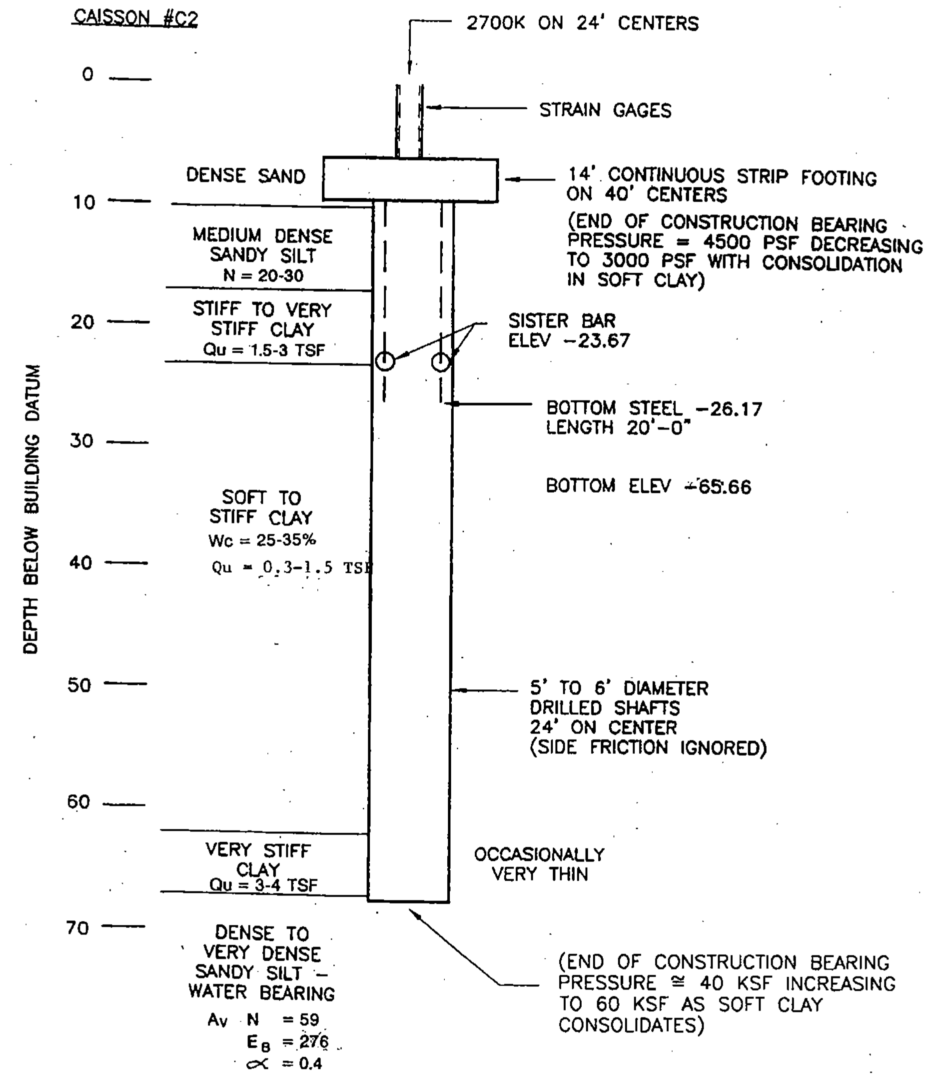
- Settlement – Predicted and Observed
  - Average measured core settlement at 80% DL = 0.55 inch
  - Average measured perimeter settlement at 70% DL = 0.56 inch
  - Core extrapolated to Design Load = 0.7 inch
  - Perimeter extrapolated to Design Load = 0.79 inch
  - Core predicted settlement based on pressuremeter testing = 0.95 inch
  - Perimeter predicted settlement = 1.0 inch

2. Chicago Southside Office Building

An interesting combination of shallow and deep foundations where the practicality of the solution depended on the reliability of predictions on ground deformation under load is represented by the following case history of a unique foundation system for a Chicago southside office building (Reference 15).

Construction of a 10-story (with provision for two more stories) combination parking structure and office building was completed in 1996 at 1911 South Indiana Avenue in Chicago, Illinois. The new structure is of reinforced concrete design with 24 feet x 40 feet (7.3 – 12.2 m) bays. The lower floor levels are parking with upper floor levels office. The lowest floor over half of the structure is at grade with the other half depressed approximately four feet. Initial construction is 10 stories with two additional floors to be added at a later date as the need arises. Maximum design column loads are 2,700 kips (12,000 kN).

The soil profile at the site consists of medium dense to dense sand and sandy silt to a depth of approximately 16 feet (4.9 m) followed by a stiff clayey crust underlain by soft compressible clay gradually increasing in strength to stiff extending to a depth of approximately 65 feet (19.8 m) where a thin sometimes non-existent very stiff to hard silty clay layer exists underlain by layers of dense to very dense water-bearing sandy silt to limestone bedrock at 90 feet (27.4 m). Because of the potential for squeezing of soft clays and the relative thinness of an adequate bearing layer at depth, a preliminary geotechnical report prepared for the site recommended against the use of conventional belled caissons for this project as too risky and expensive. STS Consultants, Ltd. 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.



12 STORY PARKING GARAGE AND OFFICE COMBINED STRIP FOOTING DRILLED SHAFT INTERACTION INSTRUMENTATION FIG. 1

Shallow Foundation Analysis

Because of the presence of the upper dense sand layer and stiff clay crust, strip footings were a possibility for support of the structure as they act in effect like a mat when combined with the dense sand layer. Because of the stress spreading effect of the dense sand and stiff clay layer, the actual bearing pressure design of the footings has less influence on the ultimate settlement since it is primarily the average stress increase in the underlying soft clay resulting from the total weight of the building that causes the settlement. Even allowing for a small amount of preconsolidation in the underlying soft clay due to past partial post glacial desiccation, calculated maximum settlement for this equivalent mat case was eight inches with 2-3 inches (51-76 mm) occurring during construction and 5-6 inches (127-152 mm) thereafter. This was considered excessive and ruled out shallow foundation only solutions.

Deep Foundation Analysis

Various deep foundation solutions were considered including rock caissons, piles, and straight-shaft caissons, but cost estimates on all solutions were outside of the project budget.

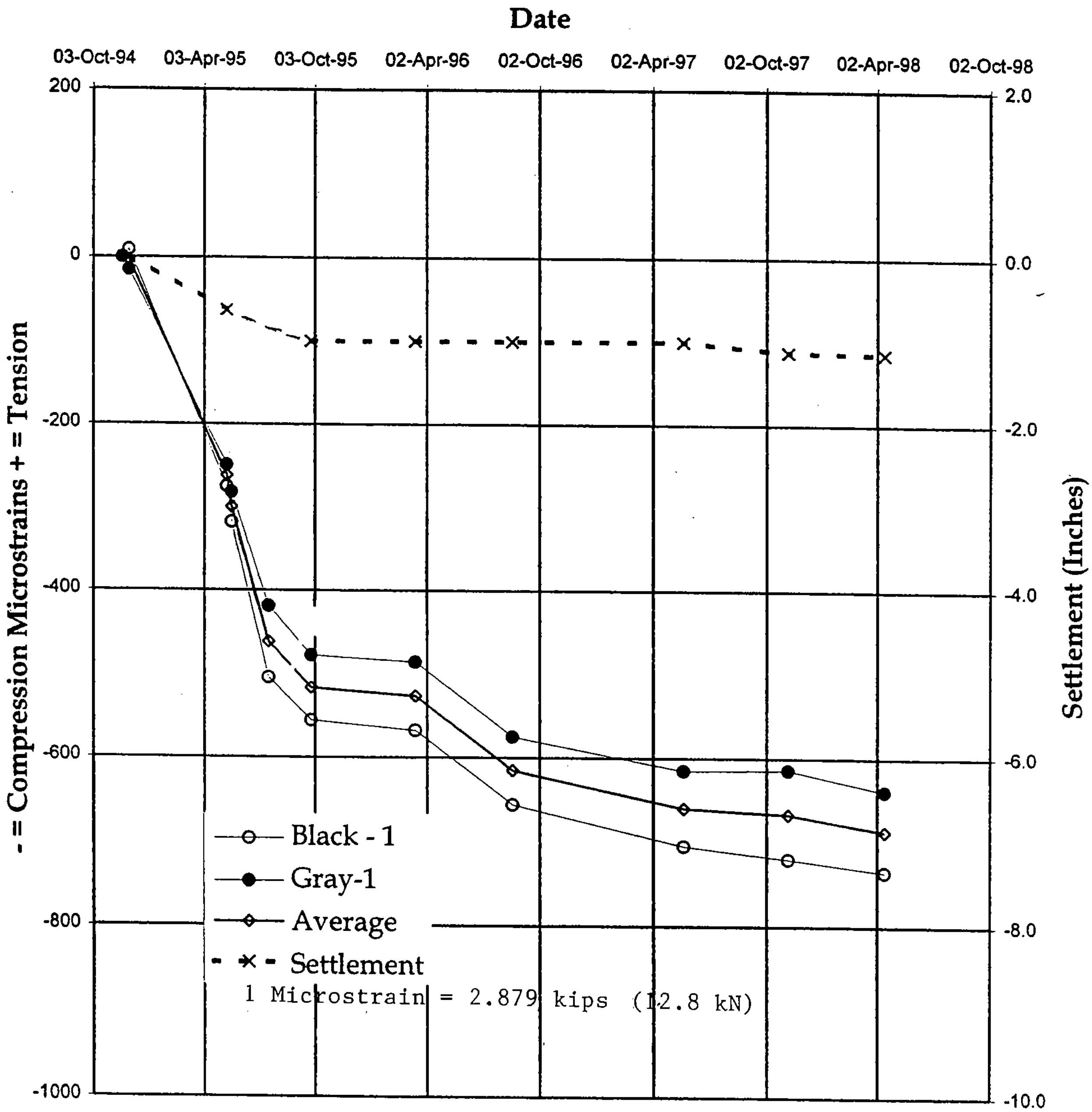
Combination System Analysis

To take advantage of the lower cost of the strip footing solution, while trying to reduce the settlement to an acceptable range, a combination system was designed. The combination consists of 14 foot (4.3 m) wide continuous strip footings on a 40 foot (14.2 m) spacing supported on the surface dense sand layer and five to six foot (1.5-1.8 m)diameter straight-shaft caissons extended down to the dense water-bearing sand and silt layer. It was anticipated that the straight-shafts could be excavated and filled with concrete before water seepage became a problem (not possible for belled caissons).

Column Strain Gage Data

1911 South Indiana

Column Line B-6



STS Job Number 1187

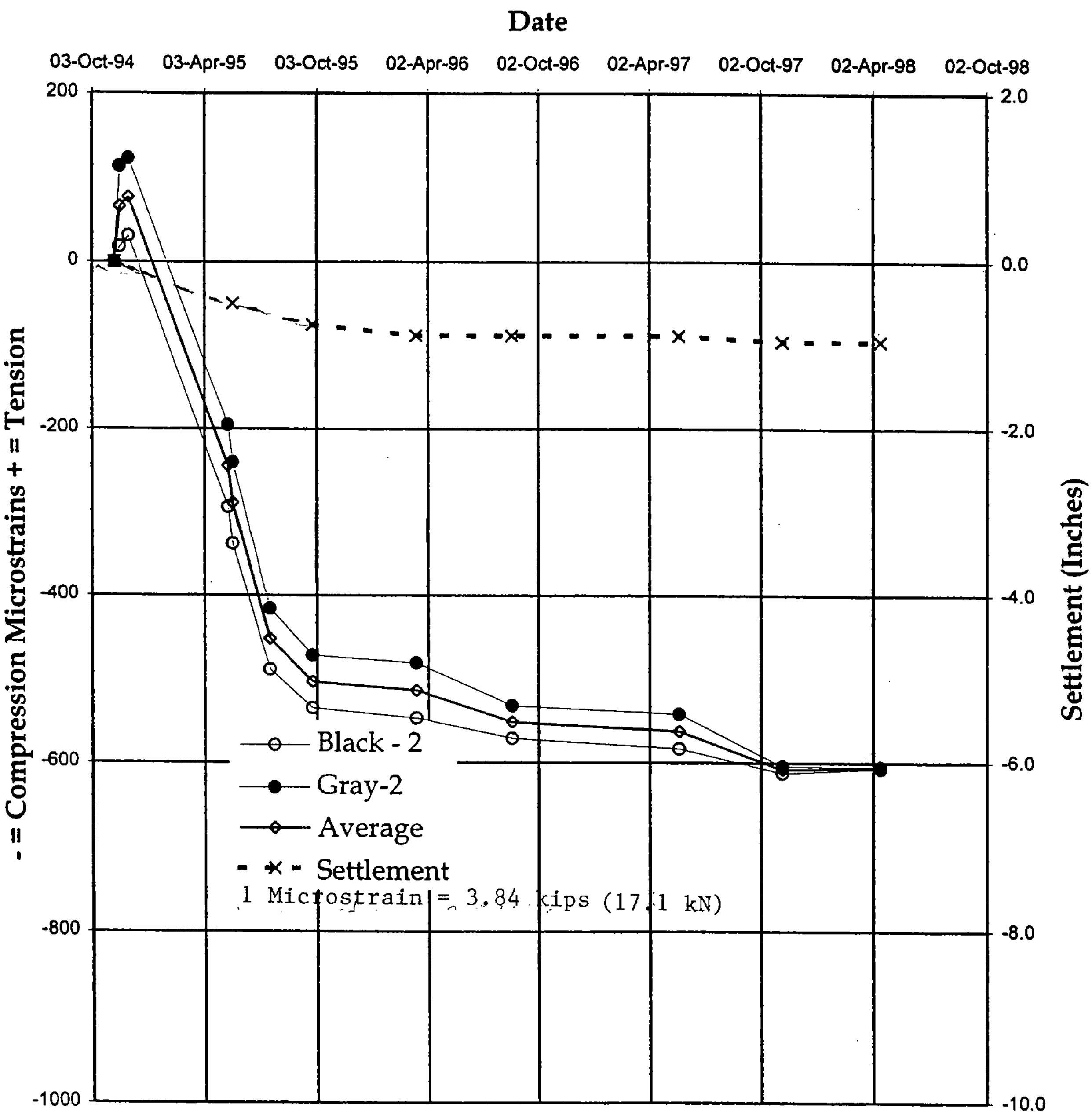
Date: 04-14-98

FIG. 3

Column Strain Gage Data

1911 South Indiana

Column Line C-2



STS Job Number 1187

Date: 04-14-98

FIG. 7

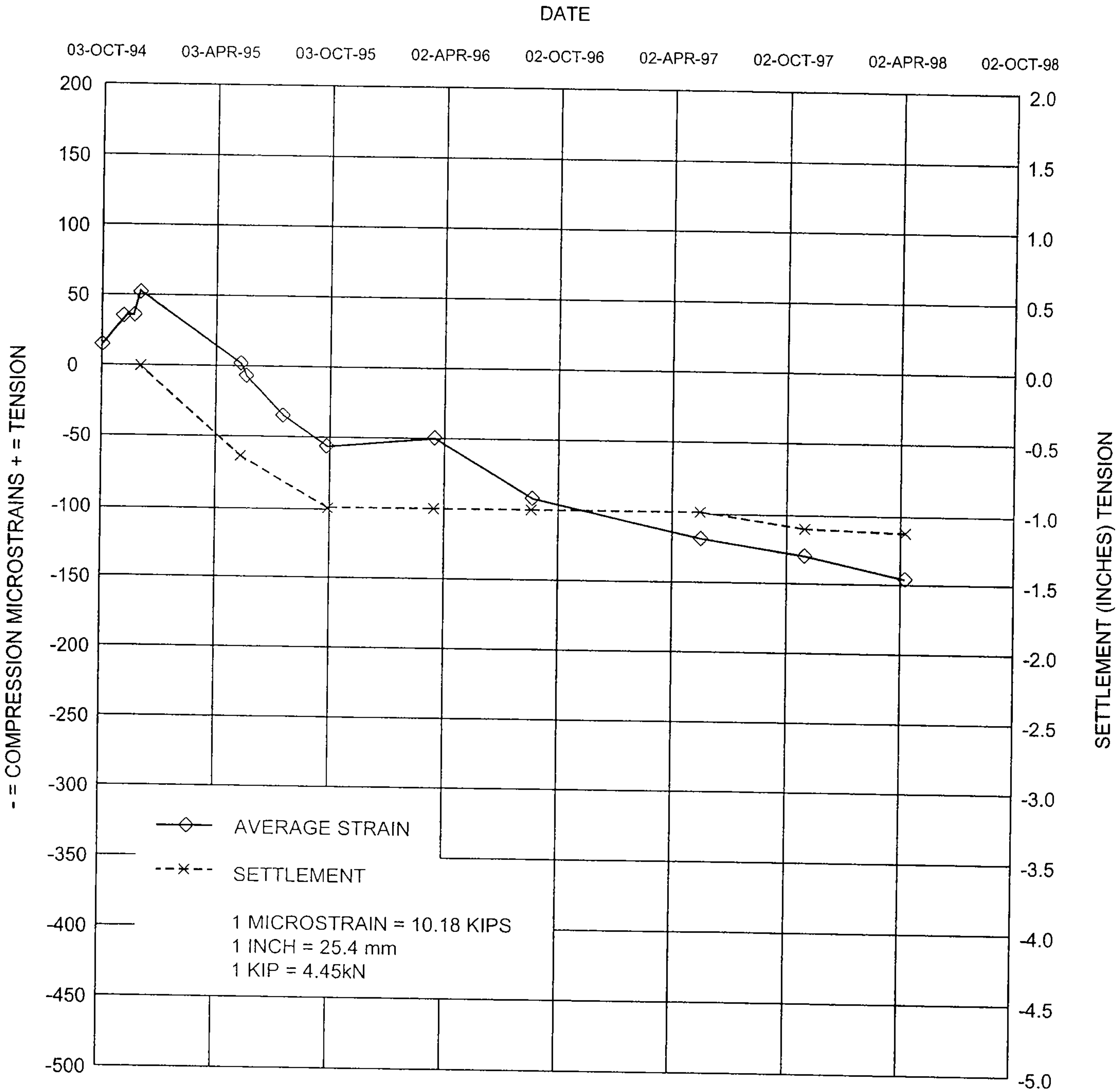


FIGURE 4: AVERAGE STRAIN GAGE READING MEASURED SETTLEMENT OF CAISSON B-6

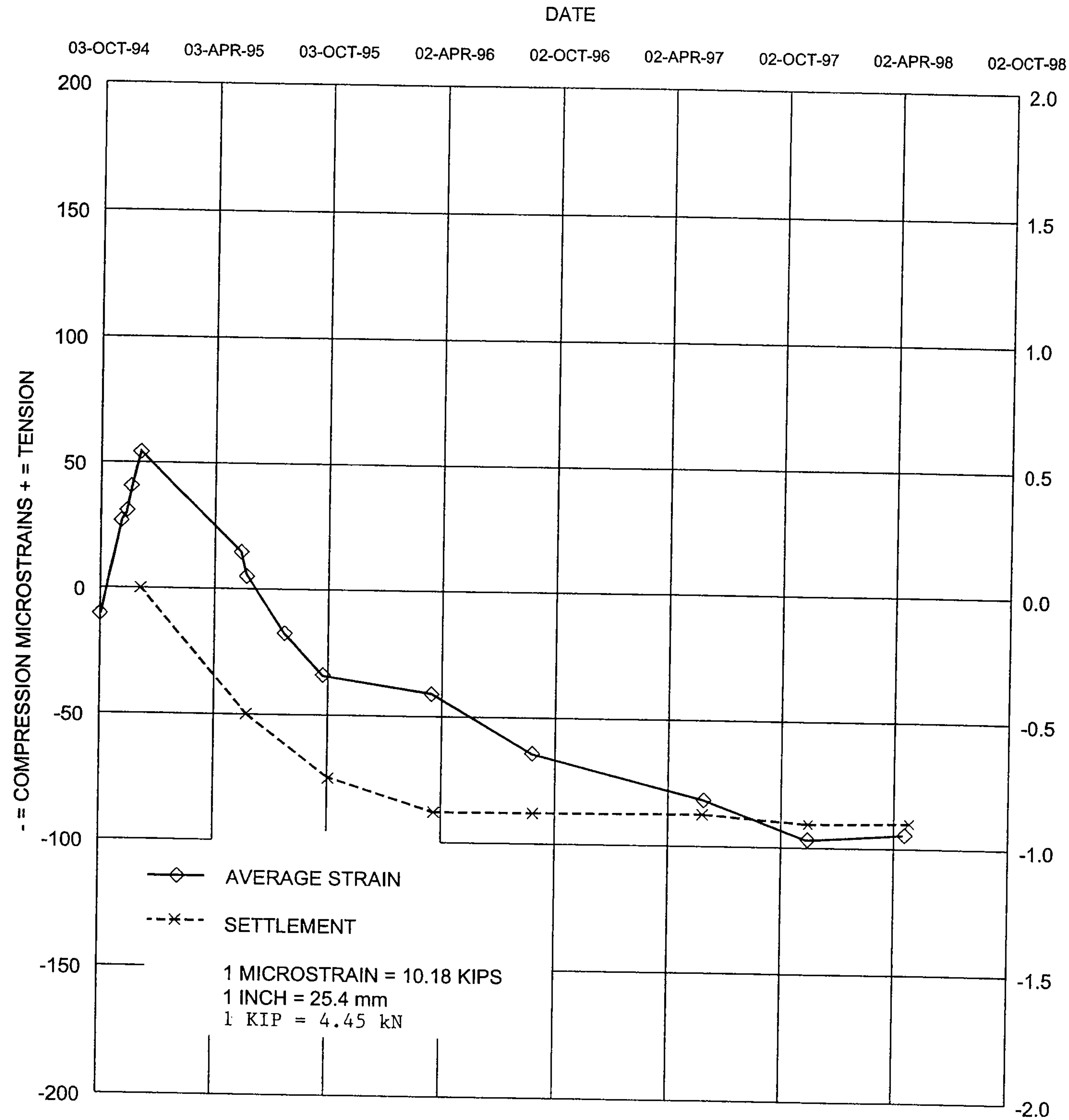


FIGURE 6: AVERAGE STRAIN GAGE READING MEASURED SETTLEMENT OF CAISSON C-2

Table 2
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

The design contemplated approximately 60% of the building load being initially supported by the strip footings with 40% carried by the straight shafts with this ratio reversing with long-term consolidation of the soft clay reacting to the strip footing pressures. The combination of strip footing and straight shaft reduces the projected settlement to less than one-third that predicted for the strip footing or mat foundation solution alone. Since the straight shafts are considered primarily as settlement reducers, a higher than normal bearing pressure can be accepted consistent with the desired settlement limitation. The design approach is 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 on settlement reducing piles is normally utilized. This approach is only practical if end bearing is in strain hardening material offering increased resistance to increasing penetration without a sudden plunging failure point such as a dense frictional material at depth (no plunging failure) as opposed to a hard cohesive material (possible plunging failure).

The Structural Engineer designed the strip footing to withstand a range of 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 is on the order of two inches compared to seven to eight inches for the strip footings only.

In order to determine how load actually gets distributed into the ground, strain gauges were placed in two representative shafts and first floor columns to monitor the load to see how much gets into the shafts compared to the strip footing. The soil profile, foundation schematic and instrumentation are shown in Figure 1.

Strain gauge data for both the columns and the caisson shafts taken over a 3-1/2 year period are shown in Figures 3, 4, 6 and 7. The strain gauge data on the columns is relatively consistent and similar whereas the strain gauge data in the caisson shafts differs drastically from one side of the shaft to the other indicating possible bending. However, the average values appear reasonably consistent and reasonable and are shown. The initial tension readings could be due to shrinkage in the concrete in the shaft being restrained by the large overlying strip footing to which the shafts were connected shortly after construction of the shafts and while cement hydration was undoubtedly still occurring.

It is also interesting to note that there has been very little load increase since the building was completed in early 1996. The small load increase as noted may be due to live load changes or possibly due to small concrete creep. Measured settlements have also been very small since completion of building with a total measured settlement ranging from 0.9 inch (23.9 mm) at column C2 to 1-1/4 inches (32 mm) at column B6. Column B6 also has the greatest percentage of the load carried by the caisson shaft as compared to the strip footing. This is probably due to the fact that the column is at the end of the footing and does not get the same stress spreading influence that the massive footing provides for interior columns. The B6 caisson appears 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 current loading is only approximately 83% of the ultimate design loading. A summary is shown in Table 2.

From the data obtained to date, it appears that the caissons are behaving slightly stiffer than anticipated and the ultimate settlement will 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 actual test pressuremeter data obtained in the bearing stratum below the shafts. 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 values adjacent to the pressuremeter tests and to the overall average N value at the test level. A proportional downward adjustment was made to reflect the fact that the overall average N value was less than the average N value at the pressuremeter test boring. In addition, to be more conservative the modulus values were further 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 predicted settlement (calculated) then agrees very well. At 60 ksf (2874 kPa) bearing the calculated settlement is 0.95 inches (24 mm). The observed settlement under shaft C-2 was approximately 0.9 inches (24 mm) at 59 ksf (2825 kPa).

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

3. Dearborn Center

Dearborn Center (Reference 16) is a case history that illustrates a mixed foundation system in which existing caissons which previously supported an 11-story building (and had been demolished down to street level) share the load with a mat constructed in the lowest basement level on top of the existing caissons to support a new 38-story office building.

The geotechnical program for this project consisted of performing seven new soil borings denoted B-101 through B-107. These borings supplemented ten earlier borings (performed for an earlier planned 85 story tower on rock socketed caissons which was never built), nine of which were performed outside of the existing building perimeter. Five of the seven new borings were performed from the existing lowest basement elevation at -23 Chicago City Datum (CCD) with two borings performed at the first basement level at elevation -4 CCD. A location plan showing all borings, as well as the existing caissons, is included as Figure 1. Borings B-101, B-103 and B-106 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 blank drilled to the top of the caisson bell at which point the concrete caisson bell was cored with a diamond bit core barrel. These three borings were then extended below the bottom of the caisson bell to elevations ranging from -79 CCD to -85 CCD. Pressuremeter tests were performed below the caisson bell in all three of these borings. Borings B-102, B-104, B-105 and B-107 were extended through the lowest level basement slab to elevations ranging from -57 CCD to -60 CCD. Pressuremeter tests were also performed in these borings through the floor slab.

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

A summary soil profile, along with a graphical plotting of the key pressuremeter test results is shown on Figure 2. A complete tabulation of pressuremeter test results is included in Reference 16.

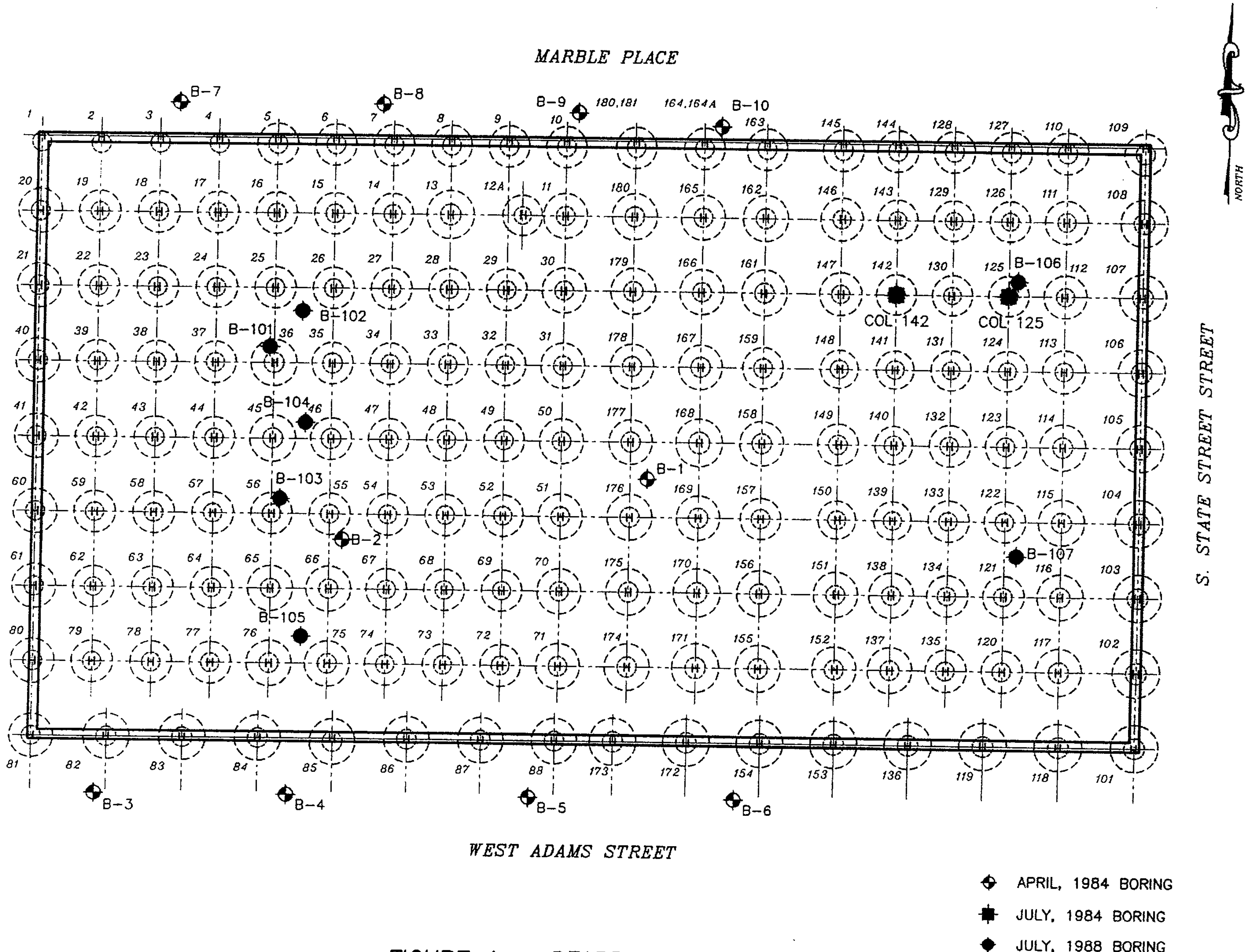


FIGURE 1 - DEARBORN CENTER
CAISSON FOUNDATION PLAN

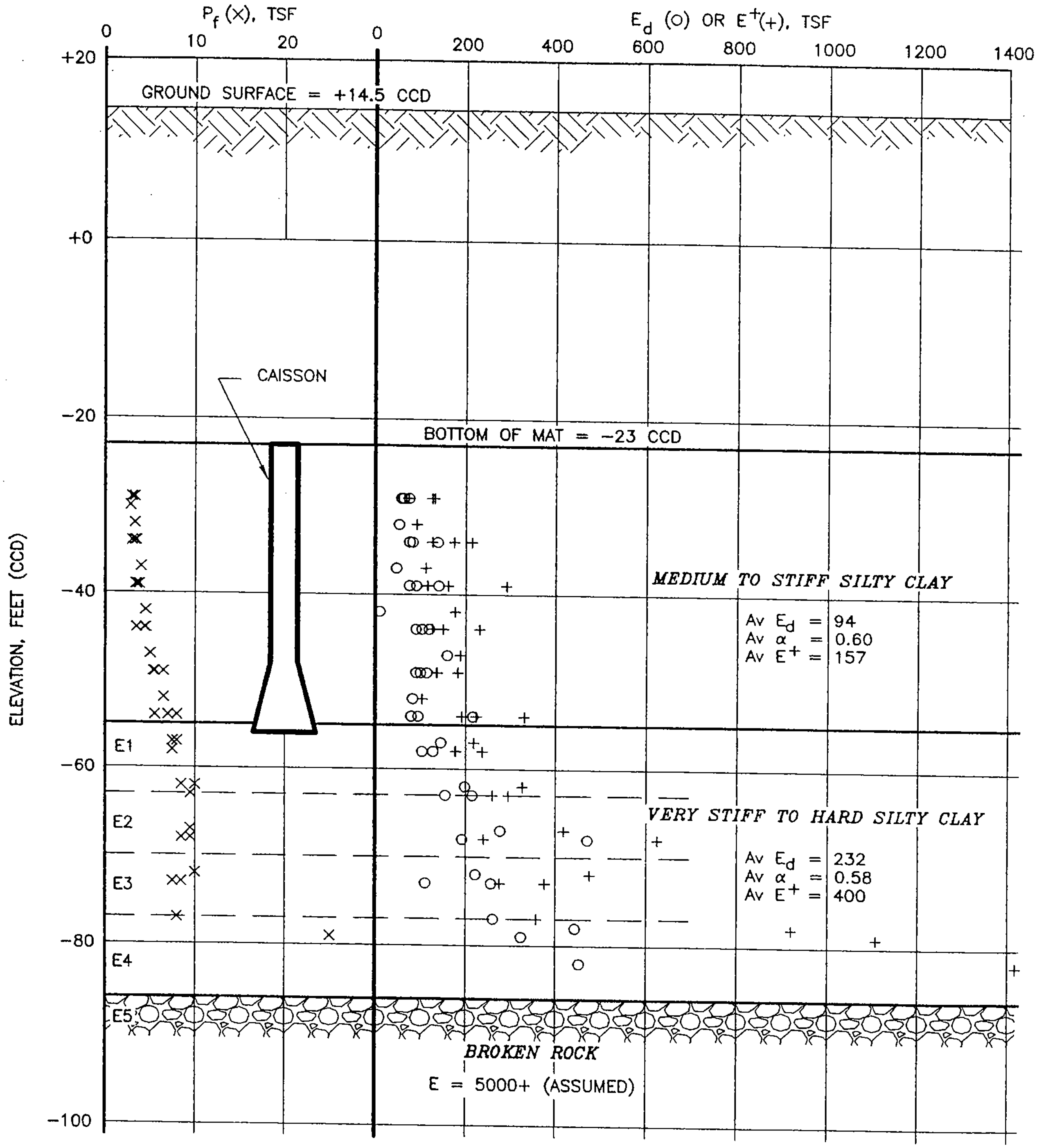


FIGURE 2–DEARBORN CENTER
PRESSUREMETER PROFILE

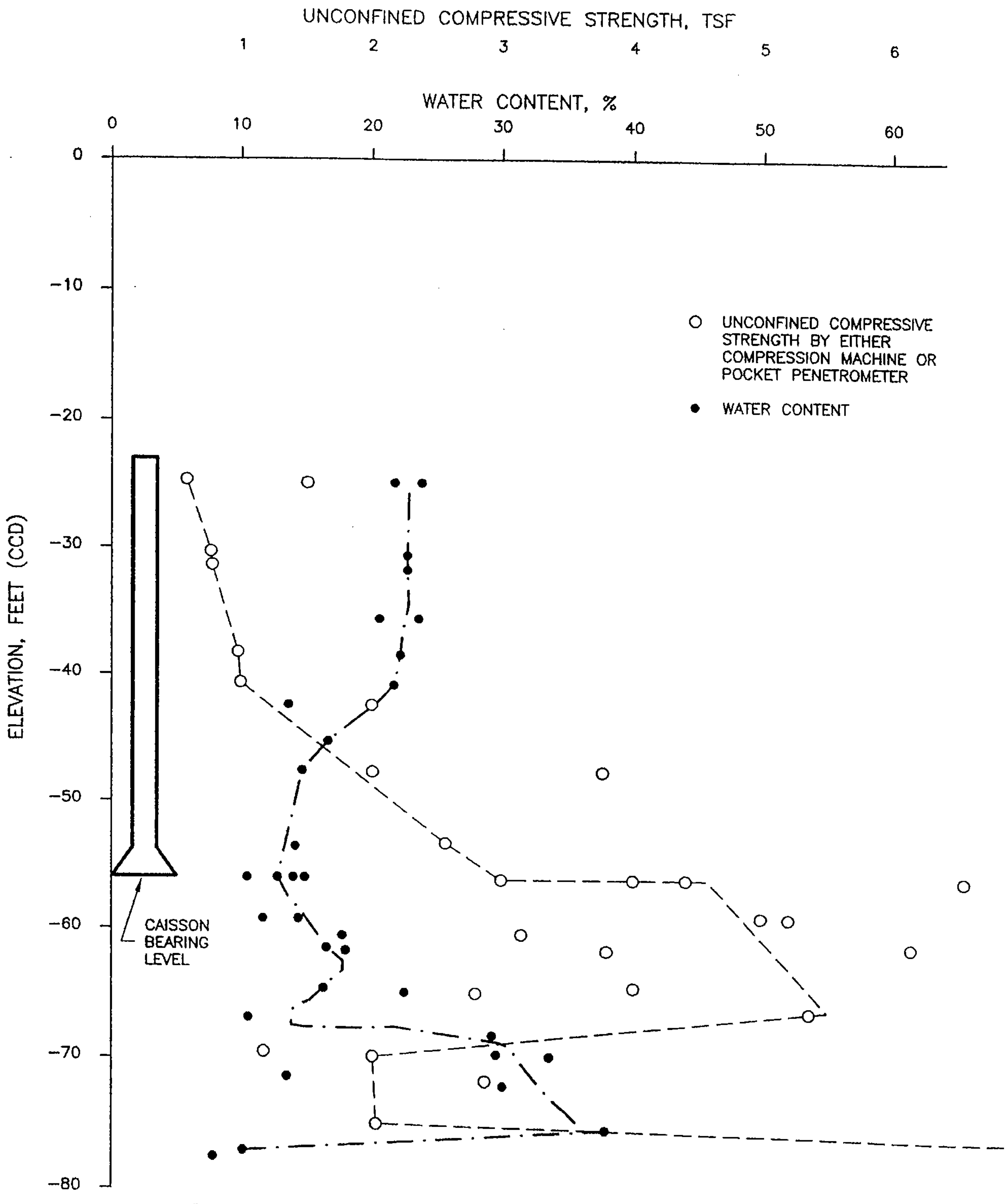


FIGURE 3 – DEARBORN CENTER
UNCONFINED COMPRESSIVE STRENGTH
AND WATER CONTENT VS ELEVATION

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 (including two levels of retail at ground level). Substructure levels would be utilized primarily for car parking. To accomplish this, the design concept involved re-using the existing belled caisson foundations which are supported on the hard clay stratum approximately 33 feet below basement level, or approximate elevation -56 CCD, and then developing additional load carrying capacity by using a mat placed on top of the bottom basement slab connecting to all of the existing columns and caissons. The new building load would be carried by the combination of the caisson foundations and mat foundation with the load distribution between the two foundation types based upon the compressibility of the subsoils. Because of the approximately 40 feet of basement excavation resulting in stress unloading of the subsoils below mat level, it was anticipated that significant loads (up to the weight of the soil removed) could be applied at the mat level with only a modest settlement for a subsoil deformation based on the elastic or pseudo-elastic properties of the subsoil.

The pressuremeter test results which measure the pseudo-elastic properties of the soil up to the creep pressure indicate an average creep pressure of approximately 9 tons per square foot (tsf) (861.8 kPa) 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 CCD (Figure 3) did not result in significantly reduced modulus or creep pressures 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 predictions to be reliable using pressuremeter data, the dead load bearing stress plus the overburden pressure should not exceed the average creep pressure. Thus, allowing for an existing overburden pressure in the hard clay just below caisson bearing level of approximately 2 tsf (191.5 kPa) relative to top of mat level, the maximum dead load pressure should not exceed 7 tsf (670.3 kPa) to keep the combined total less than the average creep pressure of 9 tsf (861.8 kPa). 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 1 inch (25.4 mm) deflection under a pressure of 18 ksf (861.8 kPa) on a representative 14 foot (4.27 m) diameter belled caisson. Illustrative calculations are shown in Reference 16.

With regard to the mat, utilizing the pressuremeter data obtained in the subsoils beneath the mat, the average mat pressure required to produce a 1 inch (25.4 mm) settlement comparable to the caisson settlement is approximately 2000 pounds per square foot (psf) (95.76 kPa). 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 support mat described in Reference 11, the Petronas Towers, which is also included in abbreviated form in this paper.

Foundation Structural Analysis and Design

The foundation design for the Dearborn Center project was driven by two major project requirements. First, the new structure would be maximized in terms of height and size while being founded on the existing foundations. Second, the existing basement walls and lower level 3 slab-on-grade must both be maintained, but the 3 basement levels must be replaced with 3 new basement levels. Figure 5 contains a foundation plan illustrating various elements of the structure.

The existing caissons were regularly spaced throughout the site on approximately an 18 X 22 foot (5.49 x 6.7m) grid. With the exception of the caissons along the north property line that extended to rock, all of the caissons were belled and supported on hardpan clay. The new building columns were somewhat irregularly placed, with bays ranging from 20 to 38 feet (6 to 11.6 m). Obviously, the new columns did not align with the existing caissons. Furthermore, the caissons located around the perimeter of the site were positioned directly beneath the 4-foot thick basement walls and inaccessible from the basement.

In order to maximize the new building's size, all of the caissons must be loaded to their capacity. A thick concrete mat foundation would be the logical choice for distributing the new column loads to the existing caissons and the soil, but two project requirements prevented this. First, a thick, heavy concrete mat would use foundation capacity, thus decreasing the allowable

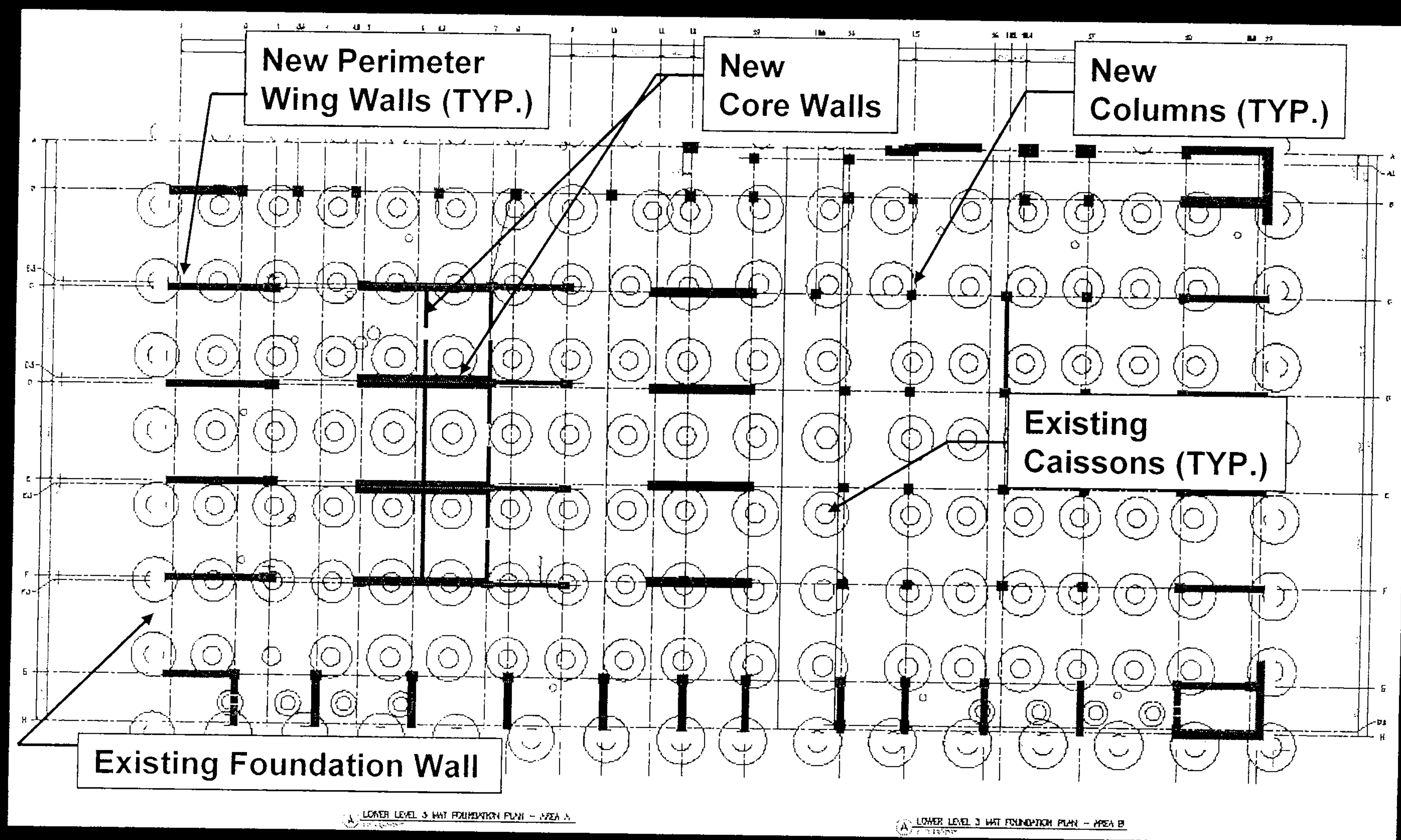


FIGURE 5 - DEARBORN CENTER FOUNDATION PLAN

building size. Second, fitting three basements in the existing excavation would leave very little depth for structure.

A relatively thin, heavily reinforced, 10,000 psi (69.9 mPa) concrete mat that varied from 54 inches to 42 inches (1066.8 to 1371.6 mm) was chosen. Preliminary analysis of the mat proved that a mat of this thickness would not be stiff enough to adequately distribute the high column loads to the existing caissons. To stiffen the mat, a series of concrete walls were introduced. The wall locations were coordinated with the architectural requirements for parking and mechanical space so that no parking spaces were sacrificed.

Two computer analyses were used in designing the concrete mat. A 3-dimensuional SAP model was built to determine the overall building behavior. Soil spring values generated by the geotechnical engineer were utilized as supports. Each caisson was assigned a spring value based on its bell size, while the caisson shaft was input as a concrete column. The soil springs directly beneath the slab-on-grade were arranged in a 2-foot (0.61 m) grid. The caissons that extended to rock were given an extremely stiff spring, allowing no more that 1/16 inch (1.6 mm) settlement. (Caisson shaft side friction was ignored because the soil under the mat was being considered for bearing.) The caissons, soil, mat, existing basement walls, new walls, new columns and the entire buildings lateral support system were included in this model.

At the time of the last available settlement data, Dearborn Center was nearing completion. The entire superstructure was erected, as well as the majority of the superimposed dead loads such as the exterior wall, raised floor system and mechanical systems. Tenants had not yet begun to move in, so live loads, partitions, etc. were not in place. It wais estimated that that approximately 70% of the full design load was being supported by the foundations. Given the estimated load at that time, and the 1 inch (25.4 mm) anticipated settlement under full load, the anticipated present settlement would be approximately 5/8 inch to 3/4 inch (15.9 to 19.0 mm).

Settlement reference marks set on the building walls and mat at the start of construction and used during construction were checked (those that could be found and were not covered). The readings indicate reported settlement that varied from 0 on the north wall (reported to be on rock caissons) to 1/2 inch (12.7 mm) on the west wall, 5/8 inch (15.9 mm) settlement on the south wall and 5/8 inch (15.9 mm) settlement on the interior mat. Allowing for survey accuracy of 1/8 inch (3,2 mm), we estimate settlements ranging from 1/8 inch (3.2 mm) at the rock supported caissons to 3/4 inch (19.0 mm) 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.

4. **Petronas Towers, Kuala Lumpur in Malaysia**

The fourth case history for this paper is the Petronas Towers, Kuala Lumpur, Malaysia, which until recently, was the world's tallest building, 10.9 meters taller than the 110 story Sears Tower in Chicago, Illinois (references 10 and 11).

The Petronas Towers are also believed to have the world's deepest building foundations. The Petronas Towers barrette foundations extend to a maximum depth of 130 meters below grade in soil and weathered rock, plus ground improvement cement grouting to depths up to 162 meters. Thus, measured from the bottom of the deepest foundations to the top of the building, Petronas Towers would measure either 582 meters (1909 feet) or 614 meters (2014 feet) 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 1. The geologic profile consists of 12 to 20 meters (39 to 66 feet) 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 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 meters (260 to 295 feet) below street grade. The towers themselves are situated with rock at 100 to 180+ meters (330 to 590+ feet) below street grade. As shown in Figure 1, there is also a valley feature in the bedrock surface between the towers extending deeper than 200 meters. (658')

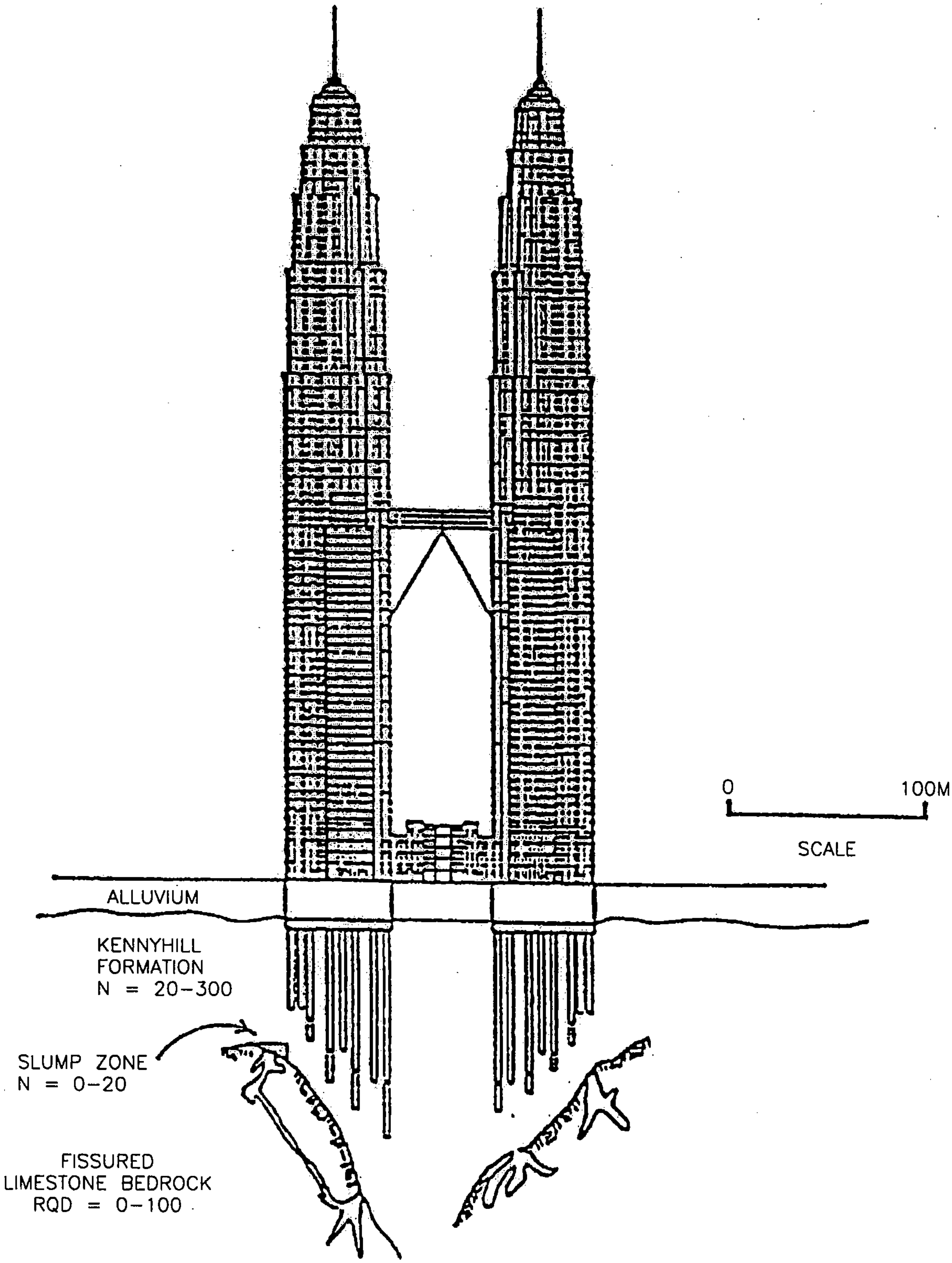


FIG. 1 TOWER FOUNDATION PROFILE

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 (less than 1/2 inch, or 13 millimeters across the base of the towers).

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 well above the limestone (grouting cavities and slump zones as necessary), 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 meters (196.9 m) 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.

Exploration Program

The exploration program consisted of more than 200 boring and 200 probes on 8 meter centers in the mat areas to check for major cavities. In addition, 260 in-situ pressuremeter tests and 2 fully instrumented 3500 ton (31,000 kilonewton) pile load tests were performed to define the modulus properties of the supporting Kenny Hill formation. The pressuremeter test summary is shown in Table 3 from Reference 10. (This is the only referenced case history where the pressuremeter used was other than a Menard 3 cell unit.)

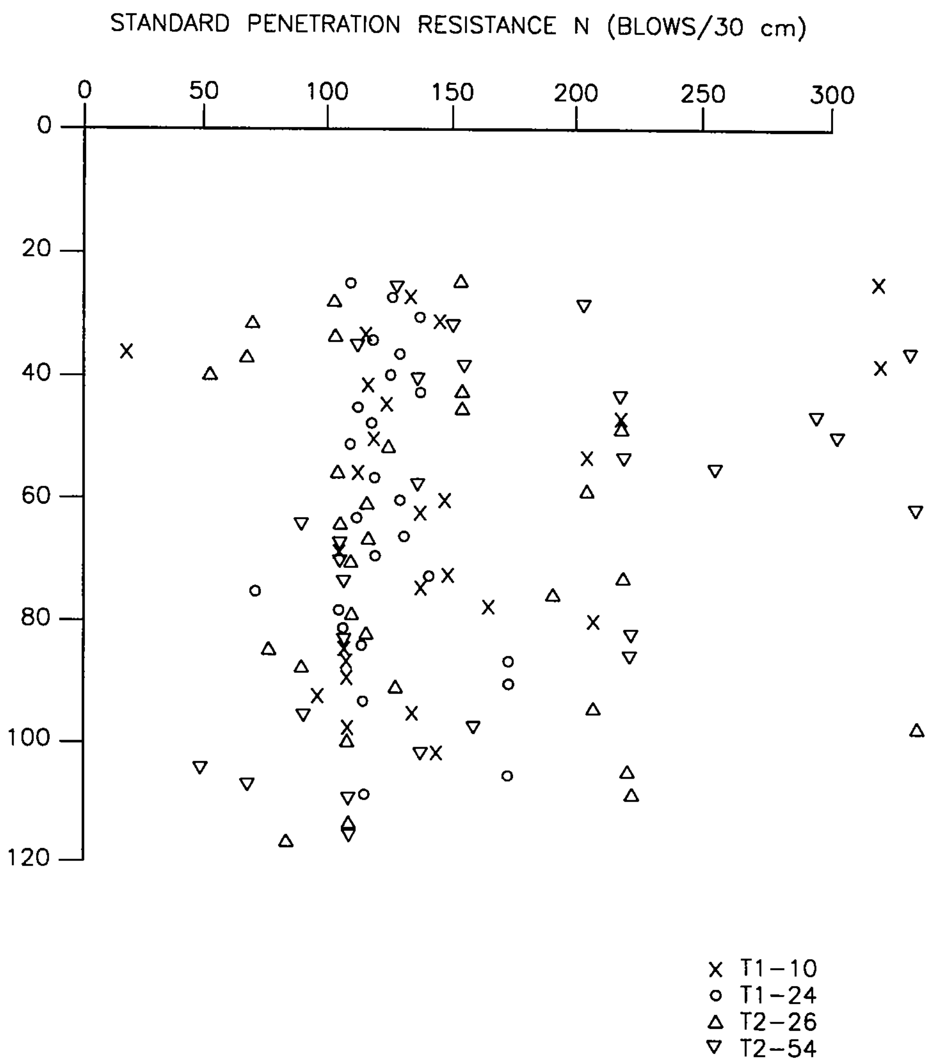


FIG. 3 STANDARD PENETRATION RESISTANCE PROFILE

TABLE 3. Pressuremeter Test Results

Boring	B14	B23	T1-10	T1-24	T1-54	T2-26	T2-54
E_d Min.	9.3 Mp_a	10 Mp_a	32 Mp_a	17.8 Mp_a	38.5 Mp_a	18.3 Mp_a	11.7 Mp_a
Max.	99	309	683	222	199.4	157	470
# of tests	18	15	27	26	26	31	27
Avg.	37.6 Mp_a	133.9 Mp_a	67.9 Mp_a	109.8 Mp_a	101.8 Mp_a	64.1 Mp_a	149 Mp_a
E_r Min.	27.5	22.3	55	32	57.7	47.8	68.3
Max.	479	931	851	496	590.3	495	383.3
# of tests	17	15	27	25	25	31	27
Avg.	186.9 Mp_a	391.8 Mp_a	176 Mp_a	226 Mp_a	223 Mp_a	190 Mp_a	535 Mp_a

Overall weighed
E_d Avg. = 94.3
E_r Avg. = 267

A representative Standard Penetration Resistance profile is shown in Figure 3. The load tests were of the Kentledge dead load reaction type with house high blocks of concrete providing the reaction.

Settlement Analysis and Assumptions

Settlement analyses were performed using the equivalent footing method and simple hand calculations as shown on Figure 12. 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 meters would have been sufficient to support the design loads, but final pile lengths under the main towers varied from 40 meters to 105 meters based on settlement considerations. Figure 4 shows the predicted settlement and ground deformation for the final design case using Figures 15 and 16 from Reference 10. Calculated average settlement from the equivalent footing method and average uniform conditions, ranged from 41 mm using the Menard rules to 73 mm based on elastic theory. This brackets the computer generated values using actual pile length and rock slope geometry;

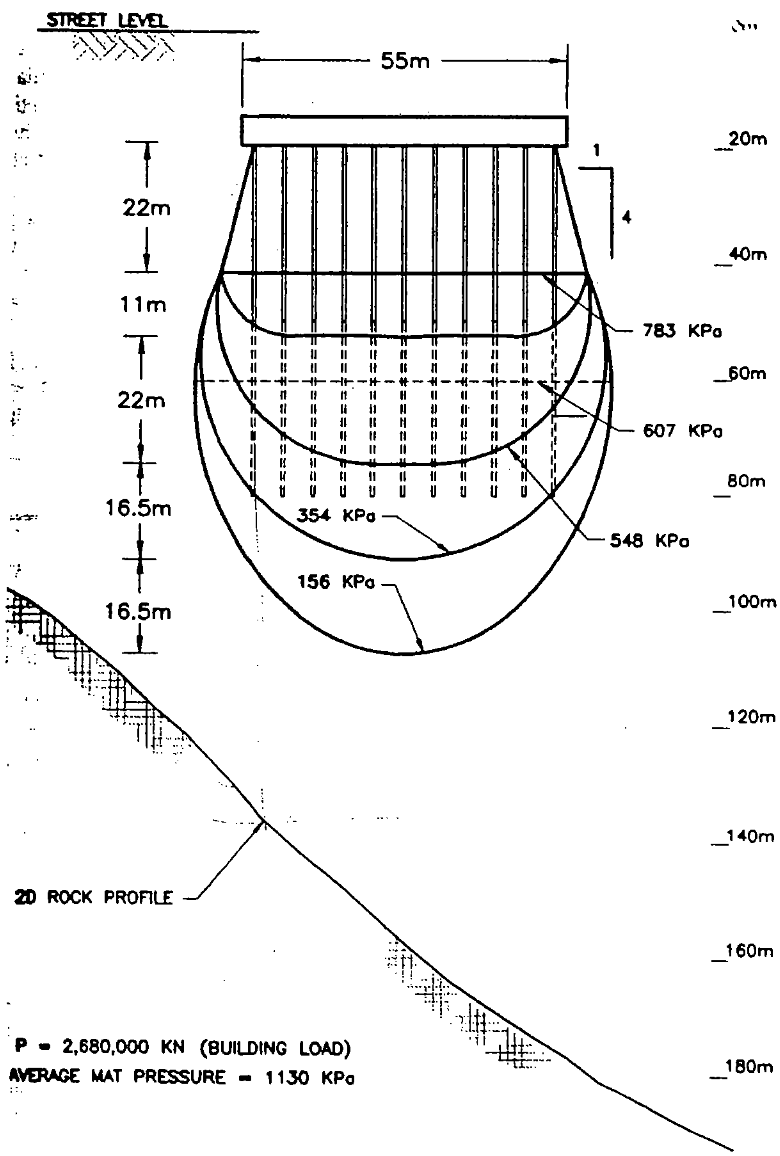
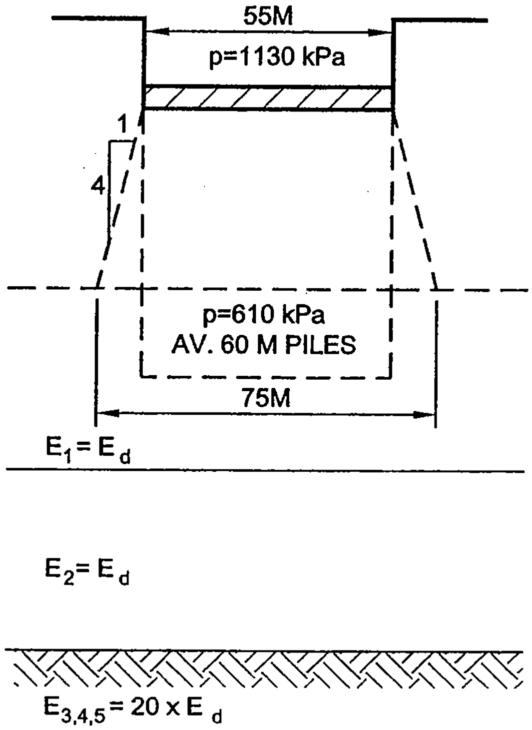


FIG. 12. Foundation Schematic for Simple Settlement Analysis

Settlement Analysis using Equivalent Footing Method



Pressuremeter Data:

$E_{d,av}$ = 94.3 MPa

E^{+}_{av} = 267 MPa

$\alpha = \dfrac{E_d}{E^+}$ = 0.35, use 0.4

p = bearing pressure added to soil by equivalent footing

$$\text{Settlement} = \frac{1.33}{3 \times E_B} \times p \times R_0\left(\lambda_2 \frac{R}{R_0}\right)^{\alpha} + \frac{\alpha p \lambda_3 R}{4.5 E_1}$$

λ_2, λ_3 = 1.0 for Circle
R_0 = 30cm

$$\text{Settlement} = \frac{1.33}{3 \times 135} \times .610 \times 30\left(\frac{7500}{30}\right)^{.4} + \frac{.4 \times .61 \times \left(\frac{7500}{2}\right)}{4.5 \times 94}$$

Settlement = 0.55cm + 2.16cm = 27.1mm

$$E_B = \frac{3.2}{\dfrac{1}{E_1} + \dfrac{1}{.85 x E_2} + \dfrac{1}{E_{3,4,5}}}$$

$$= \frac{3.2}{\dfrac{1}{94} + \dfrac{1}{.85 x 94} + \dfrac{1}{20 x 94}}$$

= 135 Mp_a

Using Elastic Theory $\quad s = \dfrac{\mu_0 \mu_1 p B}{E}$

$$s = \frac{.35 \times .92 \times 610 \times 75000}{250{,}000} = 59\text{mm.}$$

Elastic Compression in shafts down to the equivalent footing level

$$\text{Stress in barrette} = \frac{2{,}680{,}000 kN}{82 \times 1.2 \times 2.8} = 9727 \text{kN/m}^2$$

$$\Delta \ell = \frac{SL}{E} = \frac{9727 \times 40{,}000}{E}$$

conc. E $\cong$ 27,000,000 kpa $\quad \Delta \ell$ = 14.4mm.

Total Predicted Settlement
by Menard Emperical Method $\quad$ s = 27.1 + 14.4 = 41.5 mm.
Elastic Theory $\quad$ s = 59 + 14.4 = 73.4 mm.

Details of both the soil property information obtained, design parameters developed and settlement analyses performed are given in Reference 10.

Required Ground Improvement, Foundation Installation and Instrumentation

Since 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 for potential 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. Details of the grouting program, foundation installation and instrumentation program are described in Reference 11.

Foundation Installation and Instrumentation Program is also described in Reference 10.

Performance Evaluation

Predicted maximum settlement for the completed towers was 70-73 mm, (2.8 inches) with maximum differential across the mat of 11 mm (0.5 inches). Based on settlement measurements taken during construction, it appears that both measured total and differential settlements of the towers are 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 16. The maximum reported average settlement for the core is about 35 millimeters with maximum reported differential settlement of 7.0 millimeters. This is approximately ½ of that predicted in Reference 1 where a maximum settlement of 72 millimeters and differential settlement of 12 millimeters was predicted based upon an assumed modulus for the Kenny Hill formation of 250 MPa. As depicted in Figure 29A, the predicted settlement following the Menard rules and equivalent footing method is only slightly more than that experienced to date (41 mm vs. 35 mm).

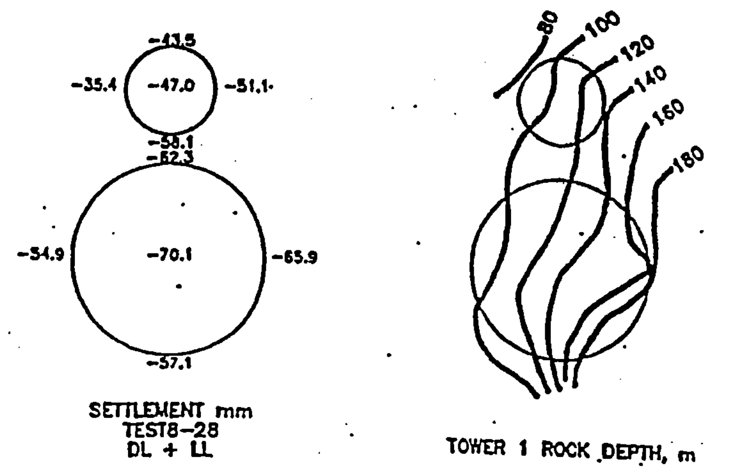
It should be noted that part of the reported differential settlement is suspect since the major portion (about two-thirds) was reported immediately after pouring the concrete mat before significant additional load had been applied. Thus, the level of reading reliability may be only on the order of 2 to 3 millimeters.

From the less than anticipated differential settlement it appears as if 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 that predicted when extensive in-situ testing was performed including two full scale instrumented load tests and 260 in-situ pressuremeter tests. As mentioned earlier in this paper based on these observations we arbitrarily doubled the reload modulus when using an elastic modulus approach to mat settlement on the Dearborn Center project. However, there are possible unique explanations for the observed performance at Petronas Towers.

In this connection it should be noted that correlation of prediction and performance would be improved if the prestressing effect of the barrette installation from the 4 meter level (with basement level at –20 meters) had been considered in making the prediction. Sixteen meters of soil excavation represents approximately 25% of the weight of the building. If this weight had been omitted, the predicted settlement would have been proportionately less.

Also, as a final observation, settlement predicted using the empirically determined Menard rules, as they are used by STS Consultants, Ltd. in Chicago, and the simple equivalent footing method, comes very close to the observed settlement, particularly if allowance is made for some prestressing effect of the pre-excavation barrette installation.



—FIG. 15. Tower 1 Settlement Map and Rock Contour Plan

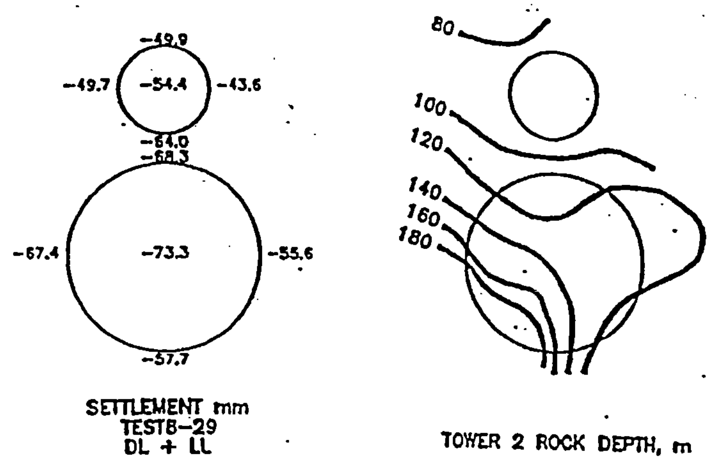
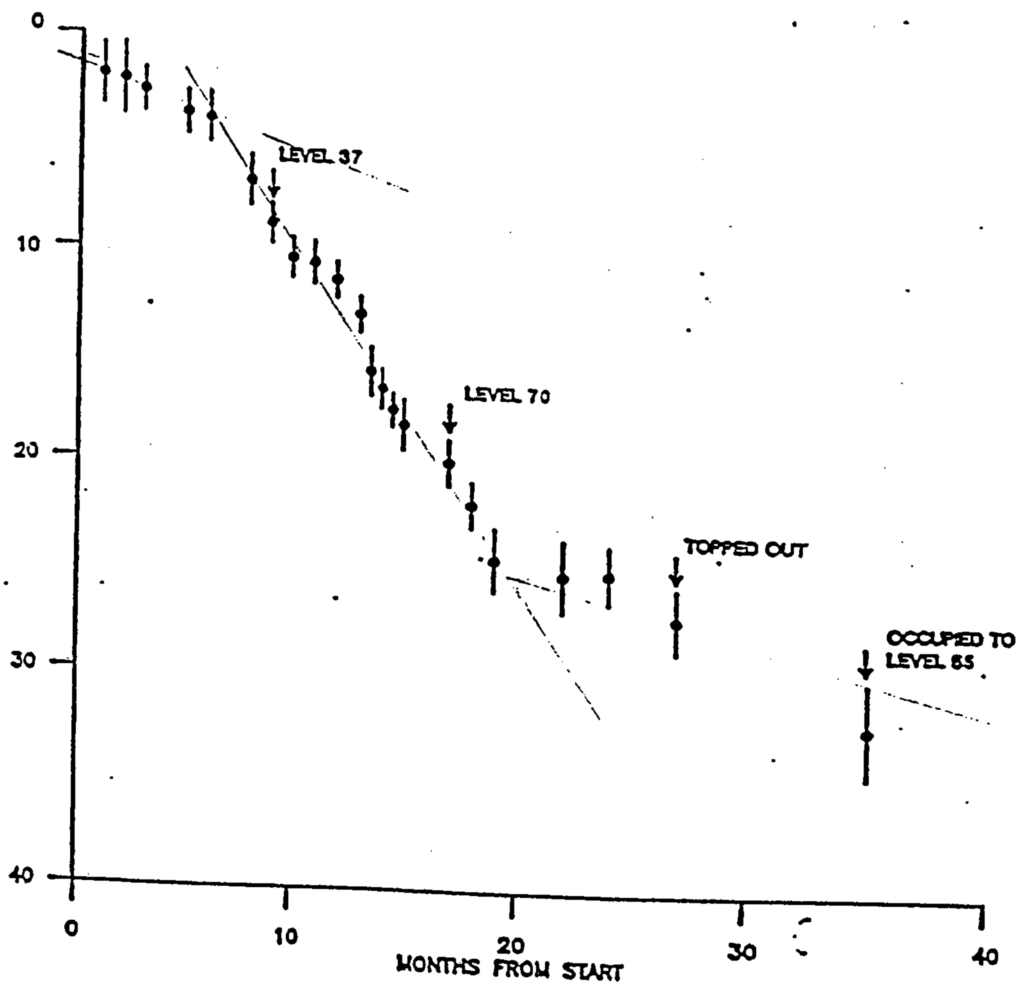


FIG. 16. Tower 2 Settlement Map and Rock Contour Plan



SETTLEMENT OF TOWER 1 COLUMNS

FIG. 16

5. PRACTICAL REUSE OF CAISSON FOUNDATIONS IN HIGH-RISE CONSTRUCTION

The fifth Case History (reference 20) illustrates the mixing of old and new foundations in a high rise office building and the predicted and observed performance. 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 at elevation -56 Chicago City Datum (CCD). For reference, the ground surface at Upper Wacker Drive is approximately elevation +20 CCD. The bearing material was a hard silty clay, locally described as Chicago Hardpan. The foundation caissons were designed using an allowable bearing pressure of 16 ksf (766 kPa) based on unconfined compression tests. In 1996, the USG superstructure was razed, and the three-level basement was converted to an underground parking ramp.

111 South Wacker Development

The new structure was a 51-story high-rise building completed in 2005 with a structural height of 681 feet (208 m).

To reduce the cost of foundations, the new building was supported on 26 new belled caissons, supplemented by the reuse of 25 caissons from the USG foundation. The new caissons were designed to bear at a depth approximately 10 feet (3.0 m) below the existing caissons. The foundation plan for 111 South Wacker is depicted in Figure 3. Reused caissons are shaded and abandoned caissons are hatched.

Construction cost savings were also realized through reuse of the existing concrete basement walls for the new underground parking garage and loading dock which service the development.

Geotechnical Evaluations

STS had provided construction monitoring and caisson inspection services for the USG foundations in 1961. Our historic records included documentation of the as-built caisson dimensions and bearing soil information, as well as the results of concrete compressive strength tests. We were 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 of the investigation (and consequently, the cost) and accelerate the design and foundation permitting process for the new structure.

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

The site is generally representative of the typical downtown Chicago subsurface profile: beneath approximately 20 feet (6.1 m) of urban fill and a thin desiccated clay crust, nearly horizontal strata of silty clay and glacially consolidated hardpan extended to dolomite bedrock at about 101 foot depth.. A site plan view and cross-sections depicting subsurface stratigraphy are provided in Figures 4 through 6.

The PMT tests indicated that the bearing pressure under the existing USG caissons could safely be increased from 16 to 24 ksf (766 to 1,150 kPa), and that the new caissons, founded in the Chicago Hardpan (at elevation -65 CCD), could be designed for a net allowable bearing pressure of 45 ksf (2,155 kPa). The reduced pressure and interpolated modulus data are plotted (the units are shown in tsf which is roughly equivalent to kg/cm^2) versus elevation in Figure 7. The bearing elevation for the new caissons is indicated by the dashed line. The typical design envelope for settlement calculations is depicted on the center deformation modulus plot.

A thin layer of low modulus clay was revealed during the PMT testing at a few locations beneath the hardpan. Conventional pocket penetrometer testing indicated this material had an estimated unconfined compressive strength between 3 and 4 ksf (144 and 192 kPa). Although it was located 10 to 15 feet (3 to 4.5 m) below the new caisson bearing elevation, this clay had the potential to adversely affect the long-term settlement of the structure at the anticipated bearing pressures.

Overlap of Caisson Bearing Pressures

The reinforced concrete building core is supported on an approximately 100-foot (30.5 m) 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 north (Monroe Street), south (public alley) and west (South Wacker Drive) sides. These non-redundant columns are supported by either a single new caisson, or a combination of reused and new caissons connected with grade beams.

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. Classic elastic theory and Menard's method were applied at representative locations. The core mat and perimeter caisson foundation settlements were predicted to be less than 1 inch (2.5 cm).

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. The spring constants for the new caissons ranged from 10,500 to 14,000 kip/inch (1,875 to 2,500 Mg/cm). The spring constants for the reused caissons ranged from 4,000 to 8,000 kip/inch (715 to 1,430 Mg/cm).

Unanticipated water problems encountered at 2 caissons (caisson D.5-5A and B.5-5A) and the corrective measures used are described in detail in the reference paper. Caisson D.5-5A was supplemented with 14 rock socketed micropiles. Caisson B.5-5A had to be redrilled as a steel cased rock socketed caisson.

Performance Monitoring

Because the micropiles installed at Caisson D.5-5.9 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 1 for various key building elements.

Conclusions

The reuse of existing foundation caissons to supplement new caissons can be a practical solution for redevelopment of project sites that might otherwise be restricted by extensive obstructions. Settlement performance can be reasonably predicted by proper interpretation of in-situ pressuremeter testing even when complicated by different caisson support systems.

TABLE 1: SUMMARY OF PREDICTED AND SURVEYED SETTLEMENT

Location	Element Geometry	Dead & Live Load DL + LL	Est. Settlement DL + 0.5LL	Measured Settlement
Elastic Theory				
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)
Menard's Method				
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)
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)
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)

As demonstrated by this project, micropiles can effectively and predictably reduce the settlement of highly loaded foundation caissons which may be undersized, or may not be constructible to the design geometry. With the support of owners, design professionals, and local building officials, performance monitoring can provide an alternative to costly and time-consuming load tests. In addition, when material properties are relatively well known, the risk assumed by foregoing load tests can be reduced to acceptable levels.

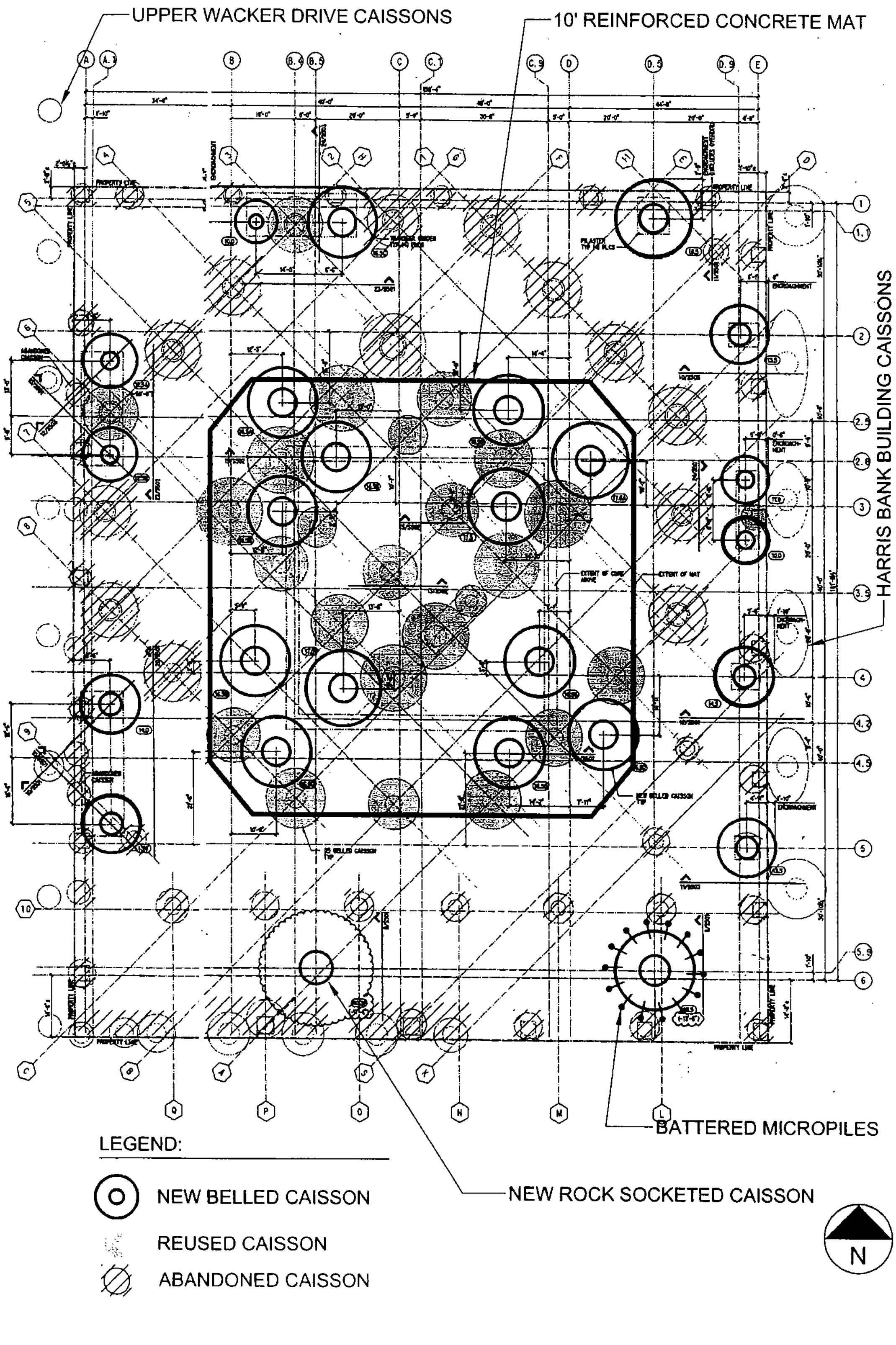


FIGURE 3: Foundation plan for 111 South Wacker.

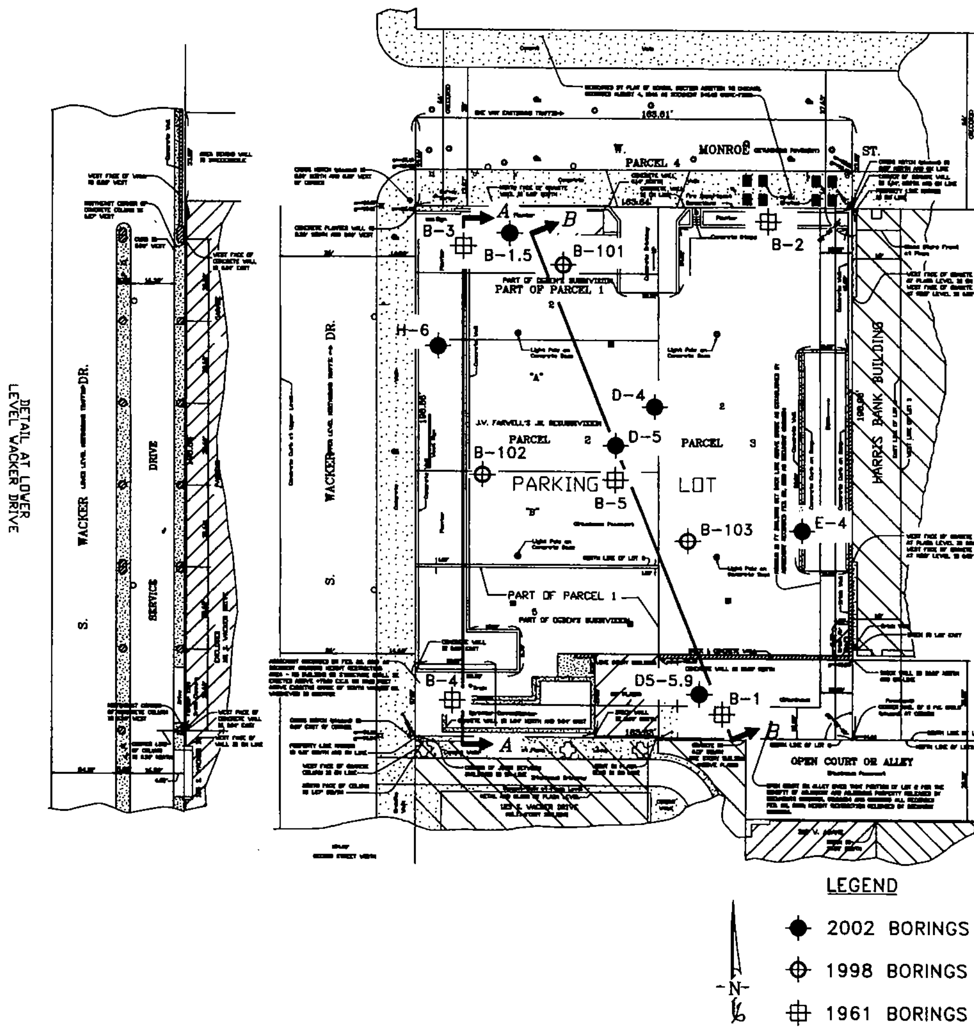
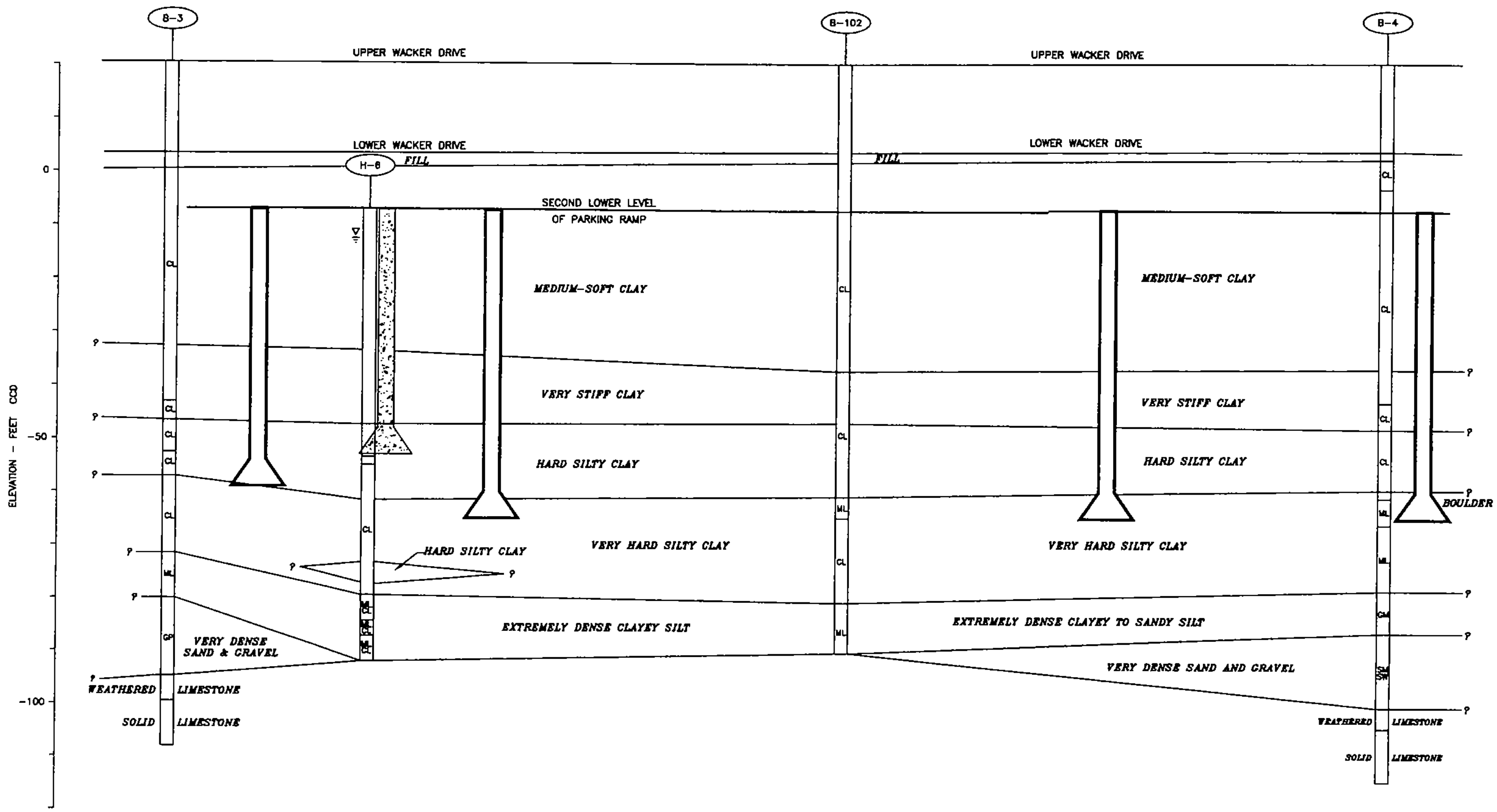


FIGURE 4: Plat of survey for 111 South Wacker.



PROFILE A-A

FIGURE 5: N-S Subsurface profile (parallel to South Wacker Drive).

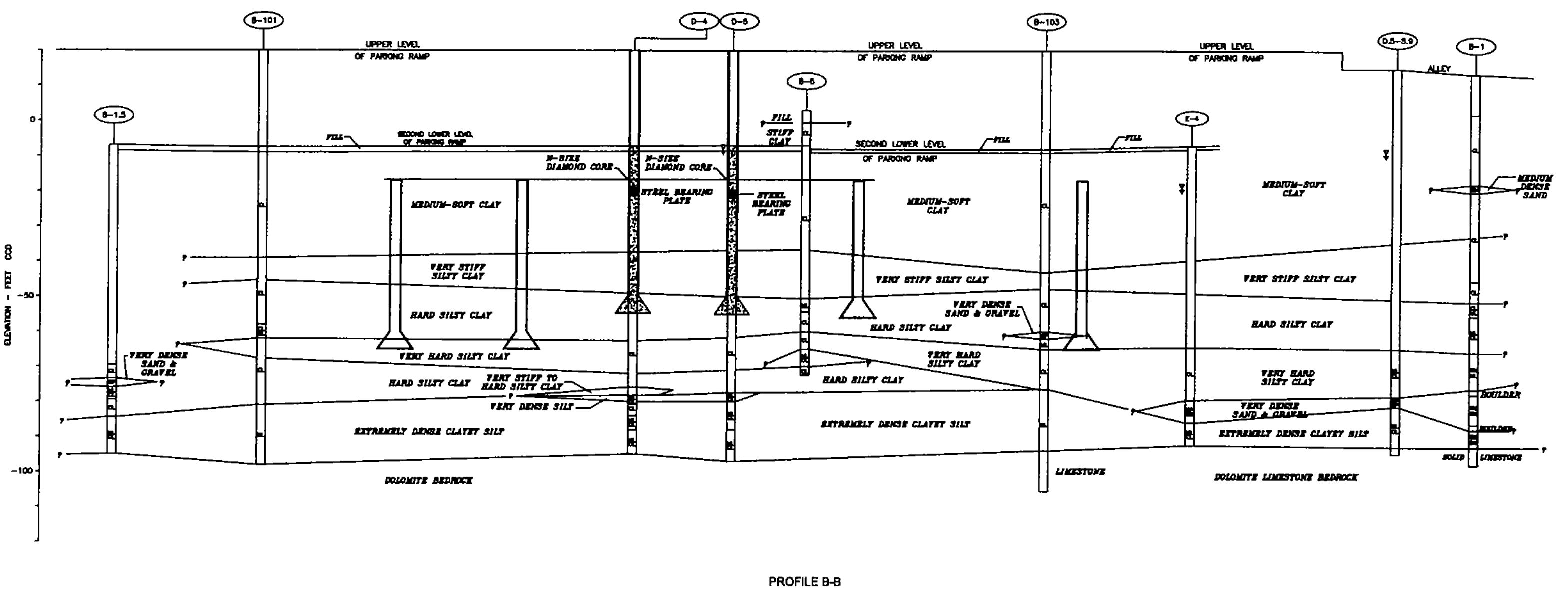


FIGURE 6: NW-SE Subsurface profile (abandoned caissons not shown for clarity).

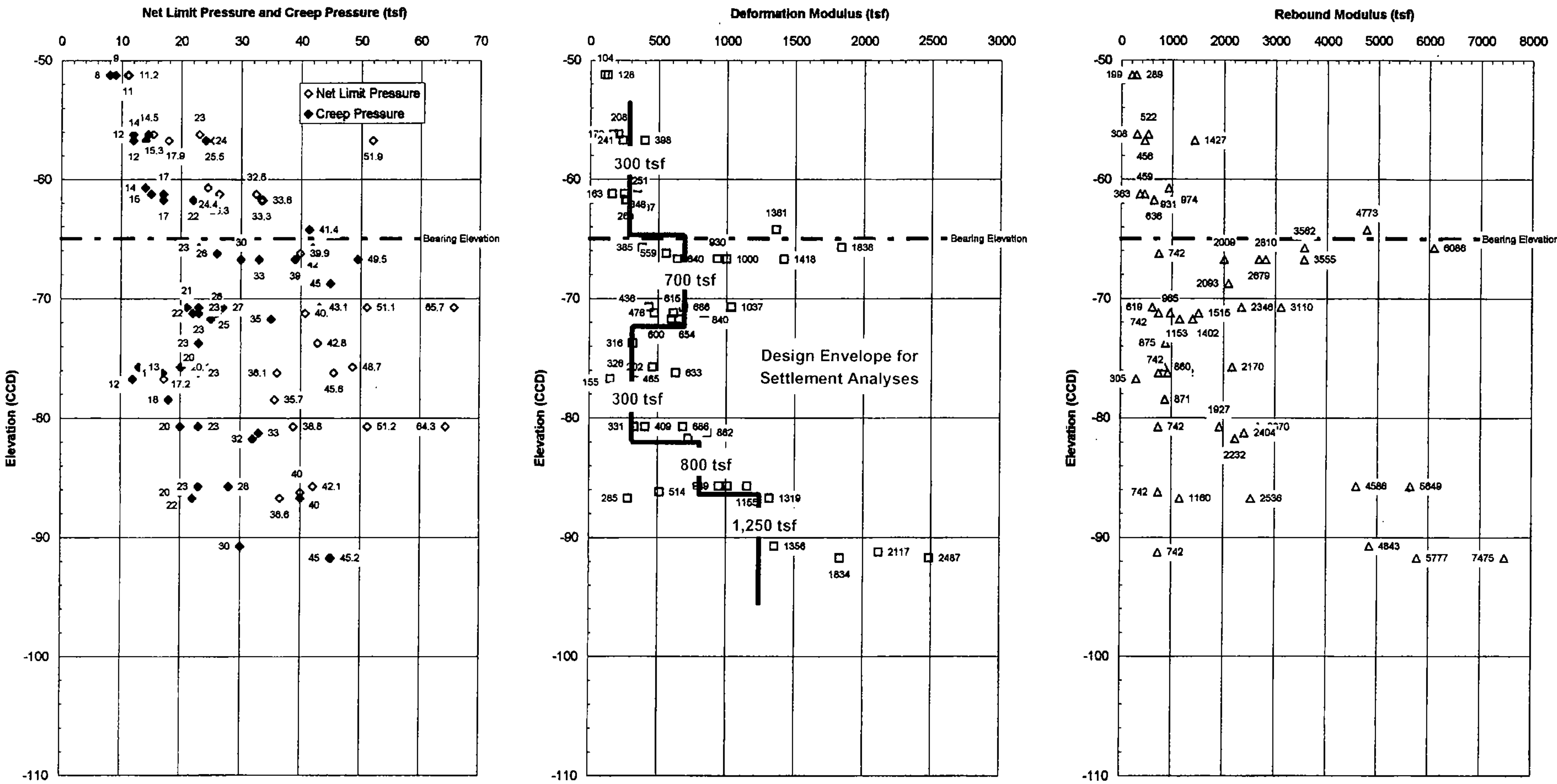


FIGURE 7: Graphical pressuremeter test results.

Conclusions and Lessons Learned From The Case Histories

1. In-situ testing with empirical correlations works well enough for engineering purposes in predicting ground deformation under load in preconsolidated soils. .
2. Menard empirical procedures yield better settlement predictions compared to elastic theory using test pressuremeter modulus values as the Young's modulus for the soil and geologic conditions reported herein.
3. Simple hand calculations for settlement and bearing capacity can be as reliable as sophisticated computer solutions considering the current level of our site exploration and property determination practice.
4. Maximum reasonable bearing values are now being used on Chicago hardpan and dense silt.
5. Innovative cost effective foundation solutions are often possible with close interaction of geotechnical and structural engineer and cooperation of experienced contractor.

Difficult Soil Profiles for Routine Pressuremeter Testing and Analysis

A precautionary comment may be in order.

The illustrative case histories selected for this paper all show excellent correlation between prediction and performance, with the possible exception of the Petronas Towers, where some explanation is required to obtain a favorable comparison. In the writer's experience, good correlation can be expected as long as there is sufficient pressurementer testing to fairly represent the deposit and total stresses are maintained less than the creep pressure. However, in the writer's experience there are soil profiles wherein this criteria appears to be met and yet the observed correlation is poor. One such profile is a finely layered profile where the layering varies from hard to soft but the soft layers are thin (2 or 3 inches in thickness or less). In this case the harder layers mask the softer layers and their effect does not get truly measured. This type of layering is observed in the Las Vegas Nevada area. A second type of profile involves weakly cemented sandstone and siltstones that may have a low density and high porosity structure that tends to collapse under pressure. In this type of formation (which is often found in the United Arab Emirates area) there can be as much as a factor of 10 difference between the initial load modulus and the reload modulus or second cycle modulus. This can be due to breakdown of the cementation bonds during the initial load cycle resulting in the compression of the sample to a denser state from which it does not rebound on unloading. Selecting the proper modulus values and the proper alpha values becomes questionable and will require local experience and local correlation with building performance for proper utilization.

Conclusions

The Menard pressuremeter and the empirical rules developed by Louis Menard have increased our ability to predict ground deformation under load in medium dense to dense soil deposits and in preconsolidated soil deposits and thereby increased our ability to mix foundation types and better accomplish more economical innovative foundation design.

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Evolution of Risk Management as a Project Management Tool in Underground Engineering

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ABSTRACT

The design and construction business has evolved over time to deal with the inherent geologic risks associated with underground engineering, tunneling in particular. Tunneling is a risky business. Population growth and society's commitment to cleaning up the environment have driven the demand ever higher for tunnel projects. Accordingly, engineers, geologists, contractors, and lawyers work continuously to design and construct bigger, longer, better, and more complex underground projects. For projects with tunnels, risk management tools have been customized to manage geotechnical risks. The authors present the history, the changes in practices that are in progress, and the status of the many contractual aspects relevant to tunnel projects. A report card is given on progress in the last 30 years to manage risks of tunnel projects. Examples are given of how "risk management" can be a powerful tool not only for dealing with geologic risk, but an effective tool for project management. Special attention is given to presenting the role of "risk management" as a technique in tunnel engineering within the broader context of how tunnel contracts are, or should be, prepared to address the many risks associated with tunneling.

1. INTRODUCTION

To illustrate evolving tunnel contracting issues, consider the following statement:

> "For several years it has been recognized that contracting practices in the United States are inadequate even for past methods and constitute a serious barrier to the application of new technology and to the most economical development of underground space. In Europe, Japan, Australia and Canada, underground works have been and are being constructed that equal United States projects in size and complexity, but employ contracting practices that vary significantly from those used in the United States. Inflationary pressures, material-shortage problems, and energy deficiencies, all of which radically affect construction costs over relatively short construction periods, have also signaled the need for review and analysis of current contracting practices."

The forgoing statement is not current, but over 30 years old! From a seminal document (USNCTT, 1974), this quotation refers to the condition in the United States where major tunnel projects were

* Note: This paper is based on the prior collaborative work of Bill Hansmire and Jim Monsees and their work "Evolving Practices for Tunnel Contracts in the United States," which was presented as a Keynote Address at the International Tunnelling Association World Tunnelling Congress 2007, Seoul, Korea, 24 April 2006."

experiencing great construction difficulties and uncontrolled costs. As a benchmark, this document has been used by the authors as the base from which change is measured for good, or not.

2. HISTORY

2.1 Tunnels Until 1974

For the two decades prior to the 1970's, tunnels were becoming increasingly more prominent in major infrastructure projects in the United States: large vehicular tunnels for the Interstate Highway System and rail tunnels for new transit systems in San Francisco, Washington, D. C, and Boston. Later, new transit systems with tunnels were built in Pittsburgh, Atlanta, Los Angeles, Atlanta, and Portland, Oregon. Need for clean water spurred tunnel construction for water supply and waste water conveyance tunnels in New York, Chicago, Milwaukee, San Francisco, and many others. Tunnelling was often the only sensible construction method that made highway, transit, water and wastewater projects possible.

Several major problem tunnelling projects (notably Water Tunnel No. 3 in New York, and the Eisenhower highway tunnel in the State of Colorado) motivated the USNCTT study of contracting practices. Before that date, looking back, the projects and people were implementing tunnel contracts in they way they had been done by the generation before them, without much change. Change was certainly needed. The evolutionary changes after 1974 came in several steps over years of time. As the authors see the situation, change came in three aspects: tunnelling methods and materials, equipment, and contracting practices.

For *tunnelling methods and materials*, competitive markets drove importation to the United States of sequential excavation methods (SEM, or "the New Austrian Tunnelling Method") and shotcrete as an engineered material for ground support. For *equipment*, the world has seen the enormous change from 40 years ago, when the equipment was either soft-ground shield tunnelling in soil or drill and blast excavation in rock, to today with tunnel boring machines (TBMs) that can excavate both soil and rock below the groundwater table with a closed face. The contractors' demand for bigger, better, faster construction equipment, coupled with entrepreneurial equipment manufacturers drove change in ways that were unimaginable in 1974

On the other hand the *tunnel construction contracts* in the United States have not seen nearly the change. Tunnel contracts have become better, but slowly and inconsistently in the environment where tunnel projects come from a multitude of owners, often with little or no tunnel experience.

2.2 Turning Point in United States - 1974

Better Contracting for Underground Construction (USNCTT 1974) was the product of the most experienced tunnelling people at the time and broad industry representation. Being the product of a National Research Council/National Academy of Sciences project, the 17 specific recommendations had unprecedented credibility. Many recommendations were subsequently adopted in varying form. In 2004, a serious assessment of the recommendations was started under the auspices of the American Underground Construction Association (AUA). See later in this paper for status.

Of particular importance are three recommendations that are best considered as a suite of linked and complementary contract terms:

- Disclosure of All Subsurface Information
- Make No Disclaimers
- Changed Conditions – Differing Site Conditions.

Adoption of these three contractual conditions has made possible all other improvements to follow. In particular, the "changed conditions" clause, which provides relief to a tunnel contractor if project conditions are different than understood at the time of bid and if those conditions had a negative impact on their work. United States government construction procurement required this clause for construction contracts well before 1974, but for non-federal tunnel contracts, it was not typically used. To be sensibly implemented, the changed conditions clause works best when all subsurface information is disclosed and disclaimers are not made regarding factual information.

2.3 Major Developments Since 1974

Steady professional activity over the years on the part of contractors, engineers, owners, and lawyers have worked to improve the tunnel contracting situation. Implementation of the 1974 recommendation has progressed with the issuance of three subsequent documents summarized below. In the first two, the vehicle for disclosure of subsurface data was called the Geotechnical Design Summary Report (GDSR) and in the third it became the Geotechnical Baseline Report (GBR).

Avoiding and Resolving Disputes in Underground Construct: Successful Practices and Guidelines (ASCE 1989) and ASCE (1991) which was an update extending the concepts to all types of construction. This professional effort became the implementing document in the United States that set guidelines for three new contracting practices: Geotechnical Design Summary Reports (GDSR), Escrow Bid Documents, and Disputes Review Boards. For each item, they evolved first from the need to rectify, or avoid next time, a tunnel project problem. After some years of trial use and enough positive experience to demonstrate the merit, a guideline document was finally published. As the dates indicate, it took 15 years from 1974 to get to this state of evolution.

Geotechnical Baseline Reports for Underground Construction: Guidelines and Practices (Essex, 1997). This document is a guideline representing a consensus (ASCE, 1997) opinion within the industry. For the most part, underground projects in the United States now use Geotechnical Baseline Reports (GBR), but after several years of use, GBRs were recognized as not fully addressing the critical issue – the specific geotechnical conditions upon which the tunnel contract was based. There is a new committee currently at work to review and update the 1997 guidelines. It is anticipated that this committee's efforts will be reflected in a new guideline publication to be issued. See below.

2.3 Risk Sharing Motivated Risk Management

Throughout the years, tunnelling and tunnel contracts have struggled with the concept of "risk sharing." Almost any contractual dispute on a tunnel construction contract can be put into a framework of "who's risk was it". Underground projects, including any construction involving subsurface excavation, present many risks, all of which must be assumed by either the owner or the contractor. The greatest risks are associated with the materials encountered and their behavior during excavation and installation of support. Definition and allocation of these risks was the focus of the GBR, but had limits.

Historically at the start of a project, the tunnelling risks, in particular geologic risk, were not well understood. The evolutionary step necessary was that, if the goal is to eliminate, mitigate, or allocate (share) tunnelling risk, one must understand just what constitutes the risks. Despite having geotechnical baselines, which could be set on the basis of engineering geology, a good process for evaluating risk from the point of view of the tunnel contractor was not in place. It was not clear how to "share" risk. As this was being sorted out in the profession, more attention was given to eliminating or mitigating the risk, and the residual risk clearly allocated to one or more parties.

Today risk sharing is often gotten to by a "risk management process" where the consequences of sharing/not sharing "risk" have been more formalized. The remaining risk has to be allocated among the parties to the tunnel contract. This has been a huge, positive step in evolving tunnel contracts. The authors see this as achieving a solution to part of the problem, but there remain risk issues to be resolved in all tunnel contracts.

3. ON-GOING PROFESSIONAL ACTIVITIES AND SPECIFIC DEVELOPMENTS

3.1 Update of 1974 Guidelines

The 17 recommendations and current status are shown in the following Table 1. Over the years some of these recommendations have been implemented with success, and some not implemented at all. The authors count 7 of the 17 (or 40% and shown by a "*") are not working, or still require improvements. Under the leadership of W. W. Edgerton, a re-assessment is in progress of the recommendations from the 1974 report (Edgerton, 2004). Although all 17 recommendations are being considered, the goal is to re-energize the industry with update of practices for specific items and promote the value of adoption in tunnel contracts.

Table 1. Comparison of 1974 and Present Recommendations for Tunnel Contracting Practices

	1974 Recommendations	2006 Status
1	Owner to Provide Rights-of-Way and Some Materials, Plant and Equipment.	Generally implemented but RWO sometimes slow. Plant and equipment usually (rightfully) by contractor.
2	Disclosure of all Subsurface Information, Professional Interpretations ….	Some holdouts but generally implemented. Led to GBR.
3	Disclaimers	Generally eliminated. Attempts still to use.
4	Include Differing Site Conditions Clause	Accepted generally. Administration evolving concerning "baselines".
5*	Handling Extraordinary Water Problems	Still having problems in effectively achieving this recommendation.
6*	Types of Contracts	Considerable attention being given. Design/build interest continues. Some successes, e.g., "Portland." Contracts still predominantly design/bid/build.
7*	Bidder Qualifications	Seldom used to truly "qualify." Often only a formality. Interest remains in workable approach.
8	Bid Pricing	No major changes
9*	Alternative Bids	Used infrequently
10	Escalation	Under used. Not as important in times of low inflation.
11	Wrap-up Insurance	Generally used on large projects
12*	Tunnel Support …	Improved, still a source of problems.
13*	Change Negotiations	Improved but still needs attention
14	Value Engineering	Not included in all construction contracts.
15	Publication of Engineers Estimate	Not typically practiced
16*	Contractor Financing Costs	Not typically practiced
17	Arbitration	Not often practiced. Better is Disputes Resolution Boards (DRB)

3.2 Update of Geotechnical Baseline

With a strong consensus in the profession that an update is needed, an update is in progress and is being led again by the original leader, R. J. Essex. Scope of the update has not been finalized but the general intent is to incorporate other types of underground but non-tunnel types of construction (such as for foundations), applications to design/build contract delivery, and lessons learned. Publication in 2007 is projected.

The authors have re-visited their prior opinions on this matter. Our recommendations in 2004 (Hansmire and Monsees, 2004) regarding geotechnical baselines remain valid and are repeated below:

> "Return to the core contractual purpose of a 'geotechnical baseline.' We need to implement this concept as a fully integrated element of the tunnel contract. We suggest the shorted terminology of Geotechnical Baseline (GB) to more correctly reflect the intent. This means getting rid of the "R," and adjusting the content of the many supporting documents (GDR, GIR) in a tunnel design accordingly. Finally, make these baselines short. As a guideline, not a rule, we suggest the GB should be no more than 10 pages consisting of succinct text, summaries, tables, and bullet items.'

The authors of this paper are participating in the update, and the intent is to help shape the standard of practices ever toward better tunnel contracts.

3.3 Disclaimers

As a rule, the GBRs being used now do a reasonable good job of following the 1974 and 1997 recommendations. However, there is still some reluctance on the part of owners to accept the fact that "the owner owns the ground". Similarly, on occasion, GBR writers with conventional geologic backgrounds still have difficulty with writing definitive baseline statements devoid of the usual geologic disclaimers. The community as a whole, however, is alert to this possibility and the result is that a concerted effort is being made to eliminate disclaimers.

3.4 Types Of Contracts

There has been perhaps no greater "buzz" in the tunnelling business in the United States than a design/build contract delivery. In 1974, reference to the " …contracting practices that vary significantly from those used in the United States" was at least in part referring to design/build. Design/build is common in many industries, such as for buildings and power generation projects. But for civil works construction in the US, design/bid/build could be considered standard. In particular, it was the standard of all 50 states and major cities that for decades has used federal funding for highway construction. With the need for faster, more efficient project delivery, major design/build projects were initiated in the 1990s. Examples are major highway upgrades in Salt Lake City, Utah with a fixed deadline to be completed before the 2002 Winter Olympics, and the transit project in San Juan, Puerto Rico which the US Department of Transportation designated as a "demonstration project" to be by design/build. The highway project was successful and made the deadline. The Tren Urbano transit project (Río Piedras tunnels) was completed, but not without some controversy.

Regardless, wholesale adoption of design/build has not replaced design/bid/build. It is still the general practice in the U.S. to use conventional design/bid/build contracts for underground construction projects. The use of design/build contracts has been gaining popularity since the mid 1990's. As a result, we estimate that one-quarter to one-third of new tunnel projects are considering design/build. Although owners are interested in saving money and time, they rarely are aware of the institutional changes required in their organizations to implement a major tunnel project using a design/build contract.

Because of the relatively short period of use for design/build, we would argue that the jury is still out on the efficiency of this contract type. At this time the consensus seems to be that they get projects completed and in service faster but there is a great debate as to whether they reduce costs or not. For example, on the Tren Urbano project, one of the first major U.S. design build projects, a major conclusion at the end of the project was as follows:

> "Contractor's don't buy design any cheaper than owners do. Money saved on doing lots of alternative designs (typical for design/bid/build) is spent re-doing designs because of different construction methods." (Gay, et al, 1999)

Another new type of contract is that used in Portland, Oregon on a major sewer project (Gribbon et al, 2003). In this case the design was advanced to a preliminary stage. Tenders were then taken, and then with the selected contractor, the contractor, designer, and owner worked together as a team to complete of design and construction. The first of two tunnels has been completed quite successfully on this model and the second is well underway.

3.5 Bidder Qualifications

The use of bidder qualifications in the U.S. is still seldom used but, we believe, gaining in popularity. In some states this requires legislative action because the existing system of low bid, hard dollar contracts has been adopted and a contractor is deemed qualified if he can provide bonding for the work. It is fairly common for such contracts to contain minimum requirements for contractor's experience and for the experience and qualifications of major contractor personnel (project manager, superintendents and the like). It is our observation, however, that these requirements are often taken more as a formality and, hence, this recommendation certainly requires more study.

4. PROJECT MANAGEMENT FOR TUNNELS

The authors have experience over the past 40 years with tunnel projects designed and constructed by many firms for many owners. Changes in contracting practices for tunnels have been driven largely by the experience of problem tunnel projects. As the several sections of this paper above elaborate, the changes were evolutionary. What emerged were "tools" to manage risk, e.g. GDRs, GBRs, and the like. Fitting these contractual risk-management tools into a construction contract has not been easy. The typical boilerplate terms and conditions of a construction contract for a major city in the United States has been difficult to change.

From our viewpoint what has emerged is risk management embedded into the management of a tunnel project. For a single tunnel, risk management is a key element of the Project Management Plan for that specific project. The great value is that the tools, such as a Geotechnical Baseline, can be efficiently empowered and implemented by the management process that has used risk management systematically to deal with project hazards ("risks") as one of several project management tools. Viewed another way, the contractual tools like the GBR are not tacked on late in the design process, but are assumed to be there in the design, just like calculations will be checked and design drawings will be sealed and signed by professional engineers in responsible charge.

4.1 Traditional Practices

The practice of engineering has in one way or another always had to deal with "risk." Traditionally "risk management" was not particularly visible, and certainly was not a driving element of the typical civil works project. It was the insurance and surety industries that dealt with risk, and was fundamental to their business. The engineers purchased insurance to cover their risk of "errors and omissions" and contractors purchased insurance for many things, including "builders risk." An owner typically worked to shift risk to the construction contractor.

A common form of risk management in engineering practice came through the process of alternatives analysis. During design, alternatives were evaluated with respect to several factors: function, sometimes constructability (with varying success depending on the experience of the designer), and cost. For a comprehensive example see the ASCE Manual 78 *Quality in the Constructed Project* (ASCE, 2000), Chapter 8 “Alternative Studies and Project Impacts”. As another example of the complex world of civil works projects and risk management practices see Hatem (1998).

During bidding and when they were awarded the contract, the construction contractor would consider alternative construction methods and sequence for one or more elements of a project in terms of both cost and schedule. A construction contractor will typically have their own views of project hazards and how to avoid or mitigate risks, as a business necessity, in order to ensure successful project completion and profitability.

Risky elements of a project that were known in advance were usually dealt with by some design mitigation. The extreme way of dealing with risky site conditions, such as a large bored tunnel in sand below the ground water table, was to avoid it all together – and not build the tunnel. In some situations a bridge was built if it were sensible for the specific project. If a tunnel was truly needed, alternative construction methods could be considered such as construction as an immersed tube tunnel (sunken tube). In the past, risks were dealt with in a broad way, but the process was usually not connected over the life of the project when compared to a project management philosophy today with risk management built in.

4.2 Risk Management Process

Essential to understanding, key definitions and the basic process are presented in below. Special note is made that this is a simple framework that can be implemented by staff on any project without special software or special skill in probability and statistics. The authors acknowledge the many variations in practices today that are used successfully. We are also aware that in many situations, risk management has become more analytical, but the fundamentals remain.

Key definitions are as follows:

Hazard:	An event having a consequence to cost, schedule, operations, environment, quality, public …
Risk:	Combination of severity of impact and probability of occurrence
Risk Register:	Management tool to track mitigation actions and manage risk throughout the project.

Confusion and misunderstanding on a specific project often comes when only the word *risk* is thought of first. Fundamental thinking must focus on the *hazard*, as defined above. In tunneling, the geologic hazards have certainly been established as posing hazards in countless ways. For instance groundwater is a given as a hazard to almost any tunnel.

For the overall management of tunnel design and construction of the project, dealing with just geologic hazards will not be all the hazards the project will face. In some cases, mitigating or avoiding one specific hazard results in creating another hazard. When the project management philosophy encompasses all project hazards, a better design fit to the project specific conditions will be achieved with some balance among sometimes competing project conditions requiring mitigation.

The basic process for each hazard requires the following steps::

- Identify the hazard
- Quantify the hazard as a risk
- Eliminate/Mitigate risk and develop action plans
- Allocate residual risk

As a compact signature statement, the process is:

Identify, Quantify, Mitigate, Allocate

Identifying a hazard is typically done in a brainstorming session with broad representation across all disciplines. In addition to geologic hazards, it is common for hazards to be identified in categories related to operations and maintenance of the facility, community impact, and environmental compliance.

Quantifying a hazard requires a secondary effort to establish the relative probability of occurrence and the relative impact if the hazard was realized. For both severity and probability, there must be a *calibration* to the specific project and owner's risk tolerance and sensitivity to local issues. For instance, the hazard of construction noise usually has a high probability of occurrence. Its impact is relative to project location. In unpopulated areas outside of cities, noise impact is typically low, whereas in a city in a quite neighborhood, impact is high. Often the impact can be quantified as the number of community complaints that reach the mayor's office. One call may be too many, and a subsequent call can result in stopping construction. This defines severity of impact "high" for that project. We advocate using 3 levels, or thresholds, defining Low, Med, and High. Although we have worked with systems using 5 levels quantifying hazards, we feel it does not give a better basis for decision-making. The following are examples.

Severity of Impact

Low	Insignificant – no impacts
Medium	Significant – potential serious delays, costs
High	Fatalities, months delay, litigation

Probability of Occurrence

Low	Improbable; extremely unlikely
Medium	Likely – 1 in 100
High	Very likely – 1 in 10

A hazard that is quantified as a risk is typically plotted on a 3 by 3 chart shown in Fig. 1 below. If low, medium, and high become numbers 1, 2, and 3, a quantified score is obtained. A "score" or risk rating is determined by multiplying the respective probability and severity ratings, e.g. a hazard rated both high probability and high impact has as a score of 9.

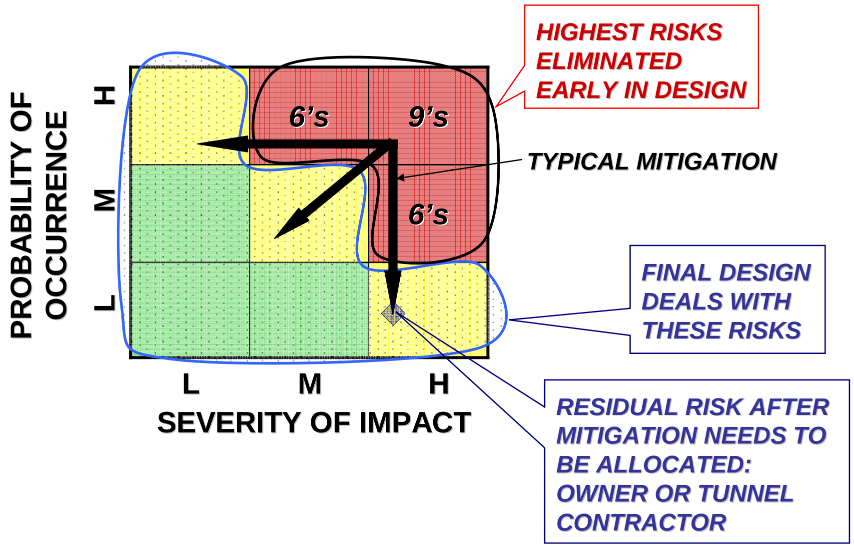

Figure 1 Risk Management Graphic Showing Hazards Quantified as a Risk and Various Risk Management Actions

Eliminating/Mitigating is always a project-specific action. By definition, the highest risks (shown in Fig. 1 as 9's and 6's) require elimination or substantial mitigation in order to make a project feasible. Such risks sometimes are call *fatal flaws*, or *no-go conditions*.

Allocating residual risk is the final step. It is at this stage of a project where a geotechnical baseline can be employed. See Essex (1997).

As a final comment on this brief overview of the risk management process, a *risk register* is a management tool, not an end in itself. It should not be the first action when a project is set up. Starting with the list is to undermine the value of the process. The critical evaluation of hazards must be undertaken and one-by-one the risks identified and captured in the risk register. Throughout the life of the project the successive actions to manage risk would be documented in the risk register.

4.3 Risk Management Throughout life of Project

We are advocating using risk management throughout the life of a project. Our personal experience has been focused on tunnels, tunneling, and geologic risks. However, we have found that in order to implement the risk management concepts in a geotechnical baseline, it is essential to be a part of a bigger plan for risk management. It cannot be effective on its own, but has to have contract provisions, and a management attitude on the part of all parties to make it work. Many past experiences, good and bad, have led us to our position that the project as a whole has to embrace risk management. As shown in Fig. 2, there will be many people, firms, and organizations that require action on their part.

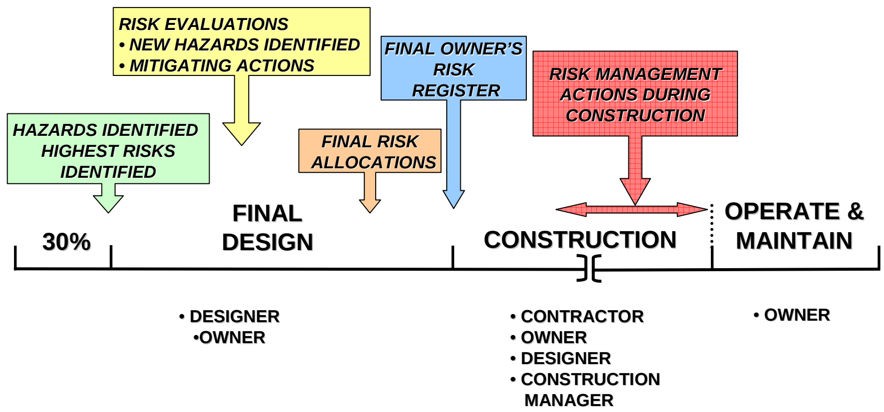

Figure 2 Project Entities Required for Beginning-to-End Risk Management

5. CONCLUSION

To close our evaluation of evolving contracting practices, consider the following:

> "By these [17 recommendations for better contracting] methods the owner would receive the completed construction at lower cost and the contractor would receive a just profit. These benefits would foster a cooperative atmosphere in which there is incentive for both the owner and the contractor to stimulate the use of advanced technologies and innovative construction techniques. The new methods would also include provisions for equitable sharing of the risks, particularly those not identifiable at the bidding stage, which are inherent in underground work." (USNCTT, 1974)

Our prime conclusion is there has been some improvement since 1974, but not nearly enough to realize the benefits projected in the statement above. Our evaluation has also led us to the simple conclusion regarding why the construction industry (contractors, suppliers, and equipment manufacturers) have changed so much: each has the incentive to achieve profit, and only through better, faster tunnelling can that be achieved. On the other hand, the owners set the terms of the tunnel contract and the owners have only the motive to limit cost to their budget. The tunnel engineers have been in the middle, working to get their clients, the owners, to adopt risk-sharing contract terms. Finally even though risk sharing is still evolving, nearly all projects now include some level of risk evaluation in the design process and sharing of risk between owner and contractor in tunnel contracts.

Use of risk management embedded within the project management philosophy is the direction the authors see the tunnel business going. We see the customized risk management tools that have come from the geotechnical engineering side of tunnel engineering maturing to be widely accepted practices. The most effective implementation is possible when the process starts at the beginning of the project. We thus are advocating that the project management approach for tunnel projects should have risk management embedded as management tool, where not already in place.

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