Dearborn Center: A Unique Soil Structure Interaction Design

Clyde N. Baker Jr.
STS Consultants, Ltd., Vernon Hills, Illinois

Ted D. Bushell
STS Consultants, Ltd., Vernon Hills, Illinois

Rob Diebold
Consulting Engineer, Chicago, Illinois

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Clyde N. Baker, Jr.
STS Consultants, Ltd.
Vernon Hills, Illinois-USA-60061

Ted D. Bushell
STS Consultants, Ltd.
Vernon Hills, Illinois-USA-60061

Rob Diebold
Consulting Engineer
Chicago, Illinois-USA

ABSTRACT

The paper describes a case history that illustrates a mixed foundation system in which existing caissons which previously supported an 11-story building that 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 investigation to determine the properties of the supporting soil strata is described as well as the material investigation to confirm the integrity of the existing foundations. The soil structure interaction concept developed and the analysis performed is presented. The observed settlement is compared with the predicted settlements with reasonable agreement reached.

INTRODUCTION

The history of high rise building foundation design and construction in Chicago prior to World War II is described by Ralph B. Peck (1948) and after that by Peck and Uyanik (1954). Prior to about 1895, most buildings, even the tallest (the Monadnock at 16 stories and Auditorium at 19 stories) were supported on footing foundations on the thin desiccated crust over the soft Chicago clay. However, experience with very large settlement under the heavier buildings which reached a reported 23 inches (584.2 mm) differential settlement under the Auditorium building by 1900 (10 year period) caused a change in design philosophy with increasing numbers of designers requiring deep foundation support for the taller buildings. This trend was accelerated by the shallow foundation settlements observed due to ground squeeze occurring during the construction of Chicago's freight tunnel system beginning in 1904.

The above discussed experience with large unpredictable settlements occurred before the development of modern soil mechanics including the theory of consolidation. The University of Illinois Bulletin by Peck and Uyanik (1954) on the "Observed and Computed Settlements of Structures in Chicago" demonstrates that the settlement of foundations built over normally consolidated clay soils can be reasonably well predicted with modern investigation and testing tools. However, settlement prediction in over-consolidated soils is much less predictable. Settlement in over-consolidated soils can range from as little as 2 percent to as much as 20 percent of the calculated settlement in normally consolidated soils depending upon how close the foundation bearing pressure is to the preconsolidation pressure in the soil (Baker 1993).

To avoid any questions with regard to the possibility of excessive differential settlement, most designers historically have tried to support their structures on the same type of foundation system and not attempt to maximize the cost/performance efficiency of their foundations based on magnitude of loading. Conventional practice until fairly recently has been to support the entire structure on either hardpan or rock caissons (but not both under the same structure), if any portion of the structure was heavy enough to require deep foundations.

IMPROVED SETTLEMENT PREDICTION IN OVER-CONSOLIDATED SOILS

During the past thirty years, there has been some modification in design thinking resulting from our ability to better predict settlement in over-consolidated soils using in-situ pressuremeter testing. These developments have been used in Chicago to facilitate economical use of mixed foundations for a number of high rise buildings constructed in downtown Chicago over the past twenty years. In a number of cases, the structural engineers have found it advantageous to support the core of some of the heavier buildings on rock with the lesser loaded (but still very heavy) non-core caissons on the hardpan or very dense silt immediately under the hardpan, with the primary question being the magnitude of differential settlement expected between the rock caissons and the hardpan caissons. Typical examples would be the 50-story office towers at 35 and 77 West Wacker Drive, as well as 1 North Wacker Drive. The use of the pressuremeter in mixed high rise foundation design in Chicago is described by Baker (1993).

SOME CURRENT INNOVATIVE DESIGN CONCEPTS

Several different innovative design concepts involving mixed foundation systems are currently being used in Chicago. In one approach, a mat foundation has been used to transfer the load from columns to old existing caisson foundations that are not located directly under the new structure columns. In this case, no load is assumed to be taken by the mat itself. Examples of this are the Associates Building at the northwest corner of Randolph and Michigan and the office tower at 181 West Madison. The other design concept involves using a mat over existing caissons in which the mat and supporting soil share the load with existing caissons, i.e., part of the load is transferred to the existing caissons and part of the load is carried by the soil under the mat based on strain compatibility and comparable settlements. Dearborn Center is an example of this latter design concept.

Dearborn Center is a case history that illustrates a mixed foundation system in which existing caissons which previously supported an 11-story building (and has 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.

This project illustrates how different foundation systems can sometimes be cost effectively designed utilizing in-situ pressuremeter testing to help predict ground deformation under load.

FIELD EXPLORATION

Since the design for the Dearborn Center involved combining different foundation systems, it was essential to be able to predict how the different systems would perform. Therefore, a comprehensive geotechnical exploration program was necessary. The geotechnical program for this project consisted of performing seven new soil borings denoted B-101 through B-107. These borings supplement ten earlier borings, 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 assess the soil immediately below the bells. 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.

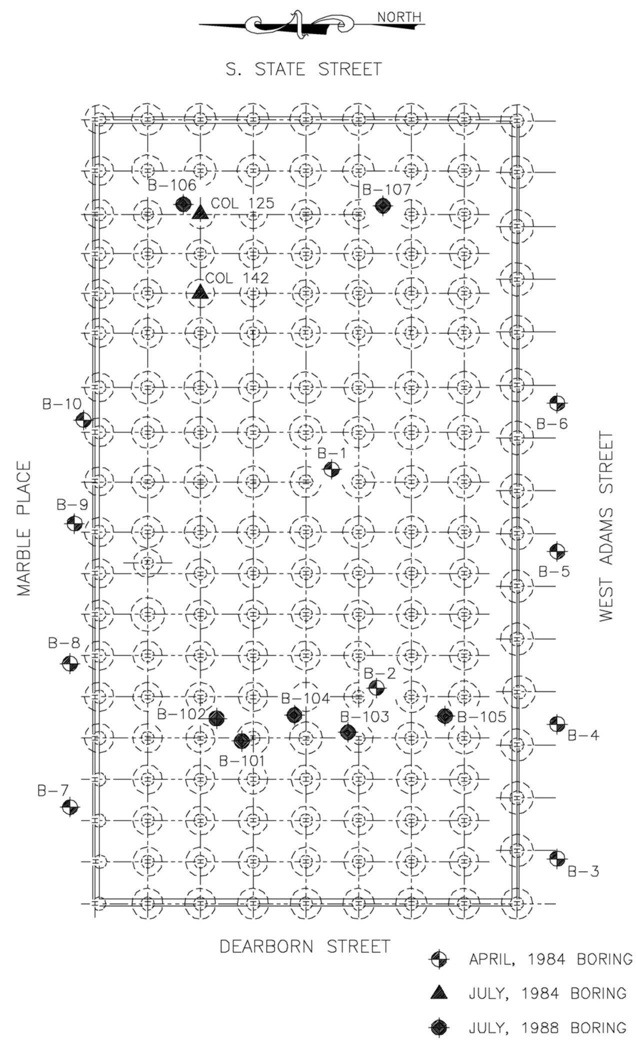

Fig. 1. Dearborn Center Caisson Foundation Plan.

Unconfined compression tests were performed on selected samples of the caisson bell concrete and indicated strengths ranging from 6300 to 7800 psi (43.4 to 53.7 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 in Fig. 2. The water content and unconfined compressive strength data, including penetrometer data, are shown graphically in Fig. 3.

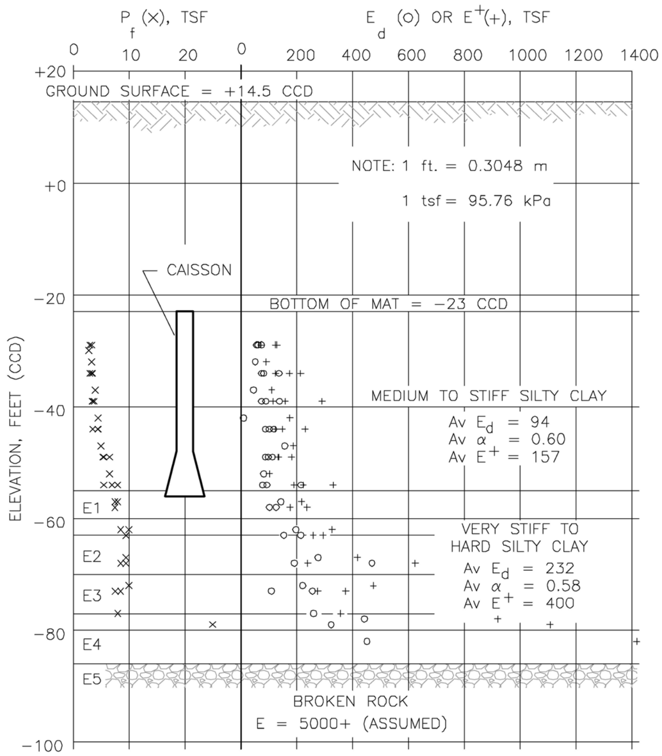

Fig. 2. Dearborn Center Pressuremeter Profile.

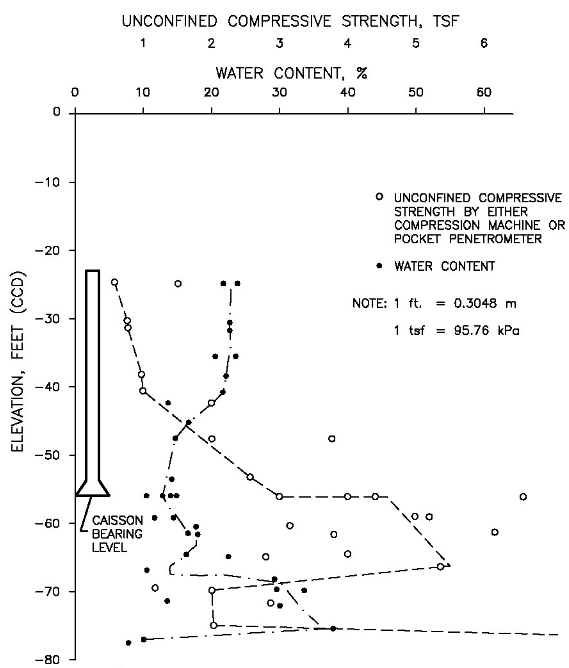

Fig. 3. Dearborn Center Unconfined Compressive Strength and Water Content vs Elevation.

ANALYSIS AND DESIGN

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 (10 m) 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 (12.2 m) 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 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 Figure 4.

Material Properties

Conc. Comp. Strength: $f'_c := 5000\text{psi}$ (34.47 MPa)

Elastic Mod. of Conc.: $E_c := 57000 \cdot \sqrt{f'_c \cdot \text{psi}}$

$E_c = 4.031 \times 10^6 \text{psi}$ (27793 MPa)

Length of Shaft: $L := 30\text{ft}$ (9.14 m)

Shaft Diameter: $d := 6\text{ft}$ (1.83 m)

Reference Length: $R_0 := 1\text{ft}$ (0.3048 m)

Caisson Bell Diameter: $D := 14\text{ft}$ (4.27 m)

The Caisson Bell Area: $A := \pi \cdot \frac{D^2}{4}$ $A = 153.9\text{ft}^2$ (14.3 m^2)

Units

$\text{tsf} := \frac{2000}{144}\text{psi}$ $\text{psf} := \frac{1}{144}\text{psi}$ $\text{ksf} := 1000\text{psf}$

Pressumeter Moduli of the Soil (From Test Data)

$E_1 := 232\text{tsf}$ (22.2 MPa)

$E_2 := 232\text{tsf}$ (22.2 MPa)

$E_{3.4.5} := \frac{16}{21} \cdot E_1 + \frac{5}{21} \cdot 5000 \cdot \text{tsf}$ $E_{3.4.5} = 1367\text{tsf}$ (130.9MPa)

$\alpha := 0.58$

Determine Equivalent Moduli

Spherical: $E_A := E_1$ $E_A = 232\text{tsf}$ (22.2 MPa)

Deviatoric: $E_B := \dfrac{3.2}{\frac{1}{E_1} + \frac{1}{0.85 \cdot E_2} + \frac{1}{E_{3.4.5}}}$ $E_B = 316\text{tsf}$ (30.3 MPa)

Determine Estimated Settlement (Ignores Elastic Shortening)

Axial Load: $p := 9\text{tsf}$ (0.86 MPa)

Shape Coefficients: $\lambda_2 := 1$ $\lambda_3 := 1$

$w_2 := \frac{1.33 \cdot p}{3 \cdot E_B} \cdot R_0 \cdot \left(\dfrac{\lambda_2 \cdot \frac{D}{2}}{R_0} \right)^{\alpha}$ $w_2 = 0.039\text{ft}$ or, $w_2 = 0.468\text{in}$ (1.19 cm)

$w_3 := \frac{\alpha}{(4.5 \cdot E_1)} \cdot p \cdot \lambda_3 \cdot \frac{D}{2}$ $w_3 = 0.035\text{ft}$ or, $w_3 = 0.42\text{in}$ (1.07 cm)

$w := w_2 + w_3$ $w = 0.074\text{ft}$ or, $w = 0.888\text{in}$ (2.26 cm)

Determine Elastic Shortening of Caisson Shaft

Elastic Shortening Due to an Applied Vertical Load is Defined as: $\delta := \frac{P \cdot L}{A \cdot E}$

The Ratio of the Bell (Bearing) to the Caisson Shaft Area is $\frac{A_{Bell}}{A_{Shaft}} := \left(\frac{D_{Bell}}{D_{Shaft}} \right)^2$

$\delta := \frac{p \cdot L}{E_c} \cdot \left(\frac{D}{d} \right)^2$ $\delta = 5.066 \times 10^{-3}\text{ft}$ or, $\delta = 0.061\text{ in}$ (0.15 cm)

The Total Deflection is: $W := w_2 + w_3 + \delta$ $W = 0.949\text{in}$ (2.41 cm)

Determine the Pressure Beneath a Mat to Cause an Equivalent Deflection

Assume Young's Modulus: $E := 2 \cdot \frac{(400 + 157)}{2}\text{tsf}$

$E = 557\text{tsf}$ (53.3 MPa)

Define a Geometry Factor: $i := 0.8$

For: $H := 60\text{ft}$ (18.3 m)

If: $\text{Settlement}, W := \frac{H \cdot p \cdot i}{E}$

Then: $p := \frac{E \cdot W}{H \cdot i}$ $p = 0.917\text{tsf}$ (0.088 MPa)

For a Settlement of 1": $p := \frac{E \cdot 1\text{in}}{H \cdot i}$ $p = 0.967\text{tsf}$ (0.093 MPa)

or,

$p = 1934\text{psf}$ (92.6 kPa)

Fig. 4. Dearborn Center Deflection and Spring Calculations.

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 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 rebound modulus. This is an empirically derived relationship based upon monitoring of large scale projects (Baker et al. 1998)

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. Because of the high cost of installing deep foundations in an existing 3-story basement, no new deep foundation elements could be added to support the new building. 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. Fig. 5 contains a foundation plan illustrating various elements of the structure.

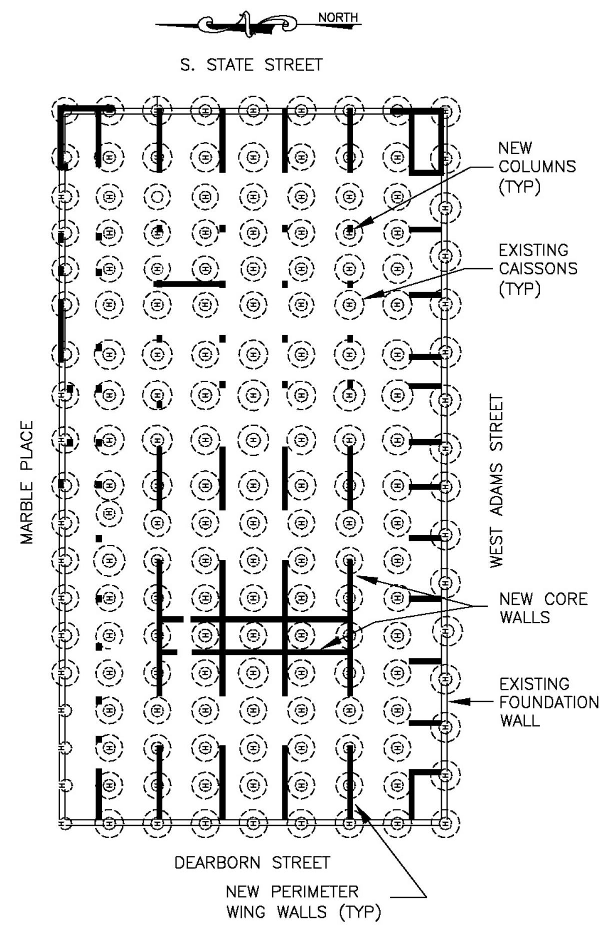

Fig. 5. Dearborn Center Foundation Plan.

The existing caissons were regularly spaced throughout the site on approximately an 18 X 22 foot (5.49 x 6.7 m) 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 (6.1 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 (1.2 m) 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. Additionally, as described in the geotechnical analysis, it was determined that the soil directly below the lower level three was adequate to support building loads. 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 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 42 inches to 54 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-dimensional 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 building's lateral support system were included in this model.

The location, thickness, height, and exact location of the stiffening walls were refined using this SAP computer model. Both the soil and caissons capacities were determined by geotechnical analysis to generate 1 inch (25.4 mm) settlement. Therefore, strategically locating and sizing the stiffening walls achieved a uniform settlement of 1 inch (25.4 mm) maximum under full load. Accurate soil settlement predictions combined with an exact representation of the building loads and an accurate model of the building structure is critical in designing a highly refined and integrated foundation system such as this.

Decreasing the weight of the braced core was key in maximizing the height of the building. Clearly, a full height concrete core was far too heavy, and the glassy exterior of the building eliminated using columns spaced closely enough to create a tube structure. Therefore, a braced steel core was chosen as the lateral force resisting system for the building. The SAP analysis indicated that differential foundation settlements generated enormous forces in the core bracing. To minimize the forces in the steel bracing, and to help distribute the loads from the heavy core columns, shear walls were introduced in the core area. These walls extended from the mat at lower level three up to lower level one. These walls optimized the load distribution while minimizing the building weight.

New shear walls were added at the perimeter of the building, perpendicular to the existing basement walls. These walls performed three functions. First, the SAP analysis indicated that the existing caissons that landed between the core and the exterior columns were not receiving enough load because few new columns landed in this zone. These new shear walls helped to shift loads from the exterior columns to these under-utilized caissons.

The second function of these shear walls was to provide a temporary site retention system. As mentioned, the existing basement walls were to remain, but the basement slabs would be demolished and replaced. The mat placed directly on the lower level three slab-on-grade and the new shear walls were constructed before the existing basement slabs were removed. The shear walls that were perpendicular to the existing basement walls were designed to cantilever up from the mat with sufficient strength to resist the lateral soil pressure. Therefore, the three basements could then be completely cleared, and most of the retention system was also part of the permanent building structure.

The third function of these walls was to connect the new mat to the existing caissons at the perimeter of the site. As mentioned, these perimeter caissons were directly beneath the existing basement walls, and so the new mat did not rest on them. These exterior caissons were engaged by creating a concrete girder at lower level one that rested on the existing columns that were supported on the perimeter caissons. This concrete girder supported the new building columns. To decrease the differential settlements between these perimeter caissons and the mat, the two were connected vertically by these shear walls that were dowelled into the mat at lower level three and framed into the concrete girder at lower level one.

A second computer model of the mat was generated for design purposes as shown in Figure 6. A SAFE model of the complete mat was used for the design of flexural and shear reinforcement. To ensure that the mat had adequate strength under all possible soil conditions, load cases were run that varied the support of the soil directly under the mat. These load cases generated an envelope of shears and moments in the mat that were used for design.

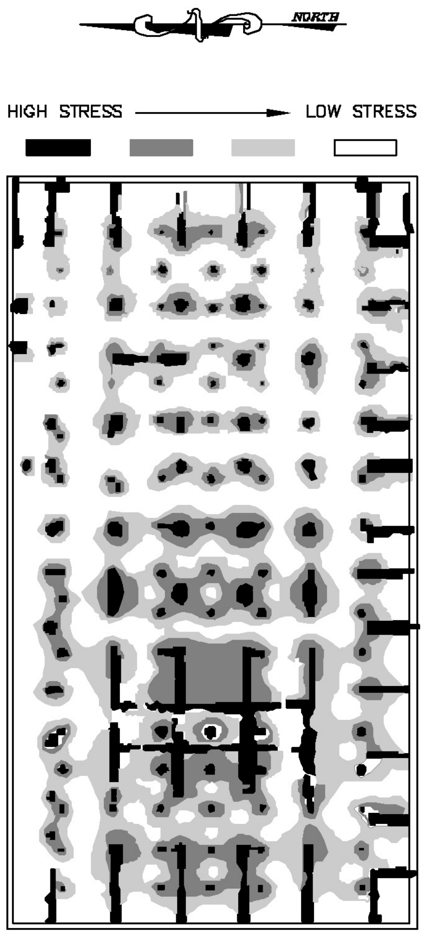

Fig. 6. Concrete Mat Shear Stress Diagram.

Since the soil directly under the existing slab-on-grade was being considered as part of the foundation system, a series of explorations were conducted to determine that no voids were present under the slab-on-grade. A series of trenches through the slab-on-grade were required for the installation of a new sub-soil drainage system. Observations of these trenches provided evidence that there were no significant voids under the slab.

OBSERVED SETTLEMENT

At the time of this writing, Dearborn Center was nearing completion. The entire superstructure had been 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 was estimated that approximately 65% to 70% of the full design load was being supported by the foundations. Given this loading, and the 1 inch (25.4 mm) anticipated settlement under full load, the anticipated settlement at this stage would be approximately 5/8 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 at this time (those that could be found and were not covered). The readings indicated 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.

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