Settlements Under Changed Structural Loadings

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Proceedings: Fifth International Conference on Case Histories in Geotechnical Engineering
New York, NY, April 13-17, 2004

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ABSTRACT

The paper deals with settlement analysis of the foundation systems which currently support the Tower City Center located in Downtown Cleveland, Ohio. The project features the complete renovation of an old retail arcade and conversion of abandoned space in the Old Union Railroad Terminal.

The major foundation system consists of a number of closely spaced spread footings bearing at varying elevations. A very small section of the development is supported by deep-seated belled caisson units. These foundations were originally installed during the late 1920's to support the construction of a proposed twenty-five story U-shaped building with a lower structure in the open space. However, due to reduced economic activity after 1929, the project was scaled back, and only one to three story buildings were constructed on these foundations.

It was determined that, for the proposed development, the soils at the foundation bearing elevations should be capable of withstanding the expected maximum column loads. Therefore, the primary concern was not the soil's bearing capacity, but the total and differential settlements under the new design structural loading conditions. Theoretical settlements calculated for several key locations were compared with the actual field data developed over a period of eleven months.

INTRODUCTION

Upperlying subsurface formations within the downtown Cleveland area are extremely variable in their physical and structural characteristics. In addition, perched water conditions within the upperlying granular strata, which vary in thickness and encounter elevation, complicate prediction of foundation response to building loads. The presence of Lake Erie to the north and the Cuyahoga River meandering along the westerly and southerly proximity of downtown Cleveland adds further uncertainties to the groundwater depth determination, and consequential influence on the foundation response.

This paper deals with the comparison of theoretical and actual settlements resulting from the redevelopment of a section of the historic Terminal Tower Complex, located in downtown Cleveland, Ohio. The project required using primarily the existing large shallow bearing spread footings and a limited number of existing belled caissons, end-bearing within the upperlying soils. Installation of a new deep foundation system to keep settlements within structurally feasible limits, though acceptable, would have been difficult and costly to install due to overhead considerations. Consequently, limitation of the proposed structural loads by limiting the height of the structure was deemed the most practical and cost effective method to utilize the existing footings while controlling settlements (Cannon, 1986). Still, this approach posed a major challenge for the design team to accommodate anticipated large total and differential settlements expected under the new construction loads.

Settlement discussions in this paper will be limited to only a few of the critical structural components of the development. Discussions and conclusions in this paper are based on the data acquired by PSI (PSI, 1988) and those reported in the previous studies for the subject site (Lewin, 1974; Lewin, 1980).

PROJECT DESCRIPTION

The project, known as Tower City Center, is located in the city of Cleveland, Cuyahoga County, Ohio, and was developed by Forest City Enterprises, Inc./Tower City Development, Inc. The Project Planner/Architect was RTKL Associates, Inc. of Dallas, Texas. The general location of the project site in relation to the nearby streets and structures is shown in Figs. 1 and 2.

Fig. 1. Aerial View of Terminal Tower Complex: #1 West Office/Hotel Tower, #2 Steam Concourse, #3 East Skylight Office Tower, #4 Midland Building, #5 MK-Ferguson Plaza, #6 Terminal Tower, #7 West 3rd Street, #8 West 2nd Street, #9 Superior Avenue, #10 Ontario Street

Fig. 2. P2 Block: (L to R) West Office/Hotel Tower, Steam Concourse (with Terminal Tower in Background) and East Skylight Office Tower

The discussion in this paper will be limited to the area located between Prospect Avenue and Huron Road, designated as P-2 block. P-2 block is bounded on the east and west sides by, respectively, West 2nd Street and West 3rd Street. The historic Terminal Tower sits on the north side of Prospect Avenue, across from the P-2 Block area. P-2 Block, at its lowest elevation, is a relatively flat plateau with finished uniform top-of-slab elevation of about 190.8± m MSL whereas Canal Road exists at an elevation of 186± m MSL. Huron Road and Prospect Avenue are viaducts with their decks at an elevation of approximately 203.0 ± m M52.

Primary structures within the P-2 block include the thirteen stories high East Skylight Office Tower on West Second Street, the fourteen stories high West Office/Ritz Carlton Hotel Tower on West Third Street, and the Steam Concourse (Atrium), between the East and West Towers. Each tower measures approximately 39.6 m by 73.1 m with the major dimension being in the north-south direction. The height of the Steam Concourse varies from two stories at its connection with the towers to four to five stories at the crown of the arch.

The existing foundations in the P-2 Block area consist primarily of a number of closely-spaced spread footings which support concrete encased steel columns extending up to the Huron Road and Prospect Avenue levels. These foundations bear at varying elevations ranging between about 187.8 and 190± m MSL. The general arrangement of the footings is shown on Fig. 3. In addition to the spread footings, there are four belled caissons, three located east of columns 6, 24 and 41 and the fourth immediately south of column 26. These caissons bear at an elevation of approximately 159± m MSL i.e., about 31.8 m below the existing P-2 Block's finished floor level.

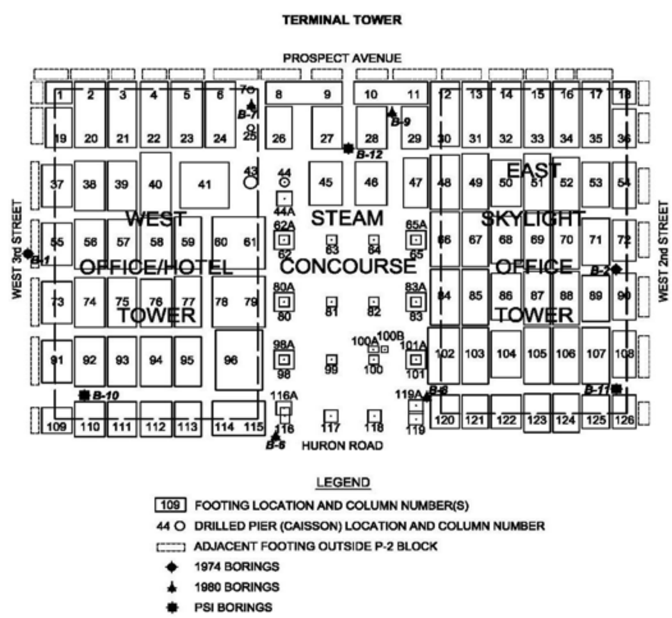

Fig. 3. General arrangement of footings in P-2 Block

Based on the project structural engineer's estimate, prior to the current development, the pressures at the foundation/soil interface due to dead and live loads were as follows:

East Tower Footings	20.6-53.6 kN/m²
West Tower Footings	17.2-62.7 kN/m²
Caissons	84.3-130.7 kN/m²
Steam Concourse Footings	38.8-186.7 kN/m²

From the available information (Cannon, 1986; Goldberg-Zoino, 1986), it appears that the existing spread footings were proportioned for soils' load carrying capacity of between about 215.5 and 253.3 kN/m² while a bearing pressure of about 598.5 kN/m² in end-bearing is indicated for the four caissons.

For the two towers, the new loads were supported by the existing footings with no modifications in their dimensions. However, for the steam concourse area, the sizes of some of the existing footings had to be increased to accommodate new columns and increased loads. In addition, a new footing was included in the southerly vicinity of caisson 44. The design pressures at the foundation/soil interface resulting from the new construction, including dead and live loads, were as follows:

East Tower Footings	69.4-165.7 kN/m²
West Tower Footings	91.4-206.8 kN/m²
Caissons	184.3-351.9 kN/m²
Steam Concourse Footings	53.1-302.1 kN/m²

The project started in July 1988 and was completed in different phases. The Steam Concourse was the first to be finished in March 1990. The West and East Towers were completed in December 1990, and June 1991, respectively.

PROJECT AREA GEOLOGY

The project area is located where the Lake Plain Section of the Central Lowland Province meets the northern end of the glaciated portion of the Appalachian Plateau Province. The Lake Plain Section of the Central Lowland Province rises from Lake Erie (where the north-flowing Cuyahoga River empties into Lake Erie) at an elevation of approximately 580 feet (Geodetic Vertical Datum). Underlying the Lake Plain Section within Cuyahoga County is the pre-glacial Cuyahoga River valley, through which the present Cuyahoga River has cut its channel. The valley is filled with approximately 61 to 91.5 m of fine sand, silt, clay and well-graded till deposits. The surficial deposits of sand vary in thickness and extent, and thin to the south of the downtown area. Beneath the sand deposits, lies a considerable thickness of interbedded silts and clays, and well-graded till deposits.

The overburden soils are underlain by Devonian and Mississippian-aged bedrock, including the Chagrin Shale Member of the Ohio Shale, Berea Sandstone, and Bedford Shale. In general, the Chagrin Shale Member is greenish-gray in color, medium to thick-bedded, containing thin interbedded layers of siltstone and sandstone and dark gray concretions. Overlying the Bedford Shale is the Berea Sandstone which is light gray to yellowish brown, medium to fine-grained quartz sandstone.

SUBSURFACE PROFILE

Up to the bearing elevations of the spread footings, in relation to the elevations within the overall P-2 block area, fill materials consisting of sand and slag containing brick and wood were encountered and represent materials used to fill excess excavations at the footing locations. The Standard Penetrations Resistance (SPT) measurements indicate that the fill was generally placed in an uncontrolled manner. Underlying the fill materials and extending to the terminal depths of the borings, the area's predominant subsurface formations consist of layers of varying thickness of silts and clayey silts containing varying degrees of sand, clay and occasional trace of organics. Silts were found to be intersticed with layers of varying thicknesses of silty clay containing trace sand and some rock fragments. Based on the SPT results, silts and clayey silts appeared to exhibit medium to dense relative density states while the silty clays evidenced stiff to very stiff structural states. Consistencies of the subsurface formations were found to vary between moist and wet.

Due to the fact that water was utilized for coring through the surface concrete and asphalt concrete, encountered water elevations could not be established accurately. However, based on the information gathered from the previous geotechnical data within the P-2 block area, it appears that water table depths could well range between about 3.65 and 5.49 m relative to the surface grade within the P-2 block.

For the cohesive soils, the results of the laboratory plasticity tests conducted for this study and those reported in previous studies (Lewin, 1974; Lewin, 1980; PSI, 1988) are shown in Fig. 4 and indicate liquid limit of between 28 and 48 and plasticity index of 10 to 21. Based on the results of the plasticity tests, these soils are indicated to be of low to medium plasticity. The percentage finer than the No. 200 (0.074 mm) sieve for these soils was tested to be between 78 and 97 percent. The unconfined compressive strength of the cohesive soil strata, to depths of 17.4 m below the P-2 block surface, is plotted in Fig. 5 and was indicated to range between 93.8 and 143.6 kN/m². These soils exhibited moisture content of between 17.7 and 31.5 percent and total density of 19.5 to 22.4 kN/m³.

BEARING CAPACITY

Maximum loads, combination of both dead and live loads, of up to 302 kN/m² were expected at the foundation/soil interface within the Steam Concourse area. At the locations of the maximum column loads, the existing footings have plan dimensions of 2.74 m by 2.74 m and are bearing within the area's upperlying clayey silt/silty clay formations at depths of approximately 1.07 m relative to the surface grades within the P-2 block. Within the effective zone of influence of the foundation loads, the average unconfined compressive strength of the cohesive formations was indicated to be 214.5 kN/m². This corresponds to ultimate load bearing capacity for these materials, under underdrained conditions, of 783 kN/m², i.e., a factor of safety of 2.59 with respect to the anticipated maximum loads. Considering that this factor of safety is for the combined maximum dead and live loads, bearing capacity was not considered as the governing factor in the evaluation of the structural performance of the proposed development. The primary concern was, therefore, settlements under the expected maximum column loads.

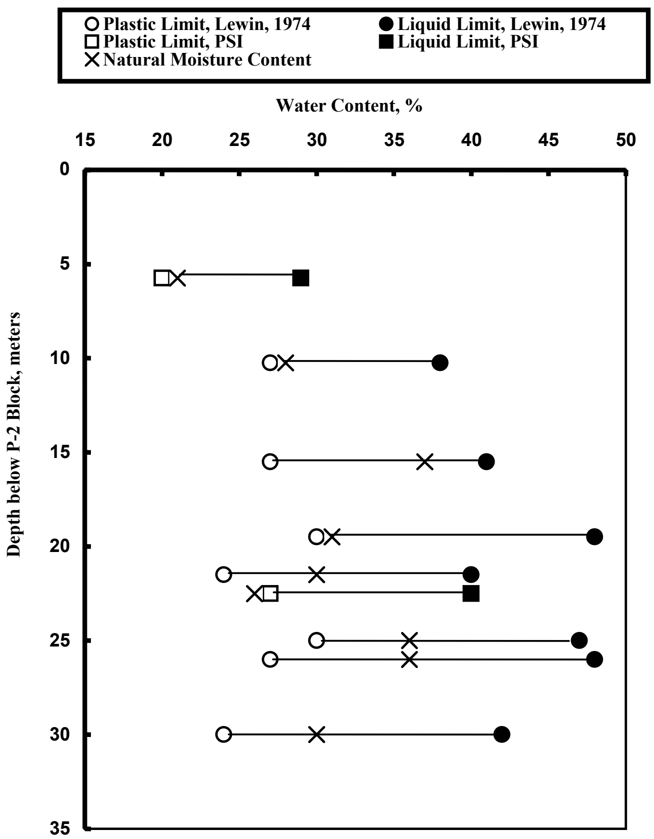

Fig. 4. Atterberg Limits Verses Depth

SETTLEMENT ANALYSIS

In addition to the previously discussed laboratory tests, several consolidation tests were conducted by PSI (PSI, 1988) and others (Lewin, 1974; Lewin, 1980). From these tests, preconsolidation pressure, compression index and recompression index values for various soils at differing depths were evaluated, and are shown in Figs. 6, 7, and 8.

The test data indicate considerable variability in the compressability characteristics of the various soil strata with depth. To account for variability and gain some feeling for the degree and gravity of uncertainty associated with the various parameters derived from the consolidation tests and to be utilized in the settlement analysis, a statistical approach was adopted. Upper and lower bound limits for the preconsolidation pressure, compression ratio and precompression ratio were established by taking standard deviation with 90 percent confidence around the mean value for a given parameter. These values are shown in Table 1.

Table 1. Laboratory Consolidation Tests Data

Parameter	Mean	Standard Deviation ±	Upper Limit*	Lower Limit *
Preconsolidation Pressure, Pr, kN/m²	186.7	62.2	138.8	234.6
Compression Ratio, C'c	0.127	0.031	0.155	0.099
Recompression Ratio, C'r	0.031	0.015	0.048	0.017

*Upper and lower limits from standard deviation with 90% confidence.

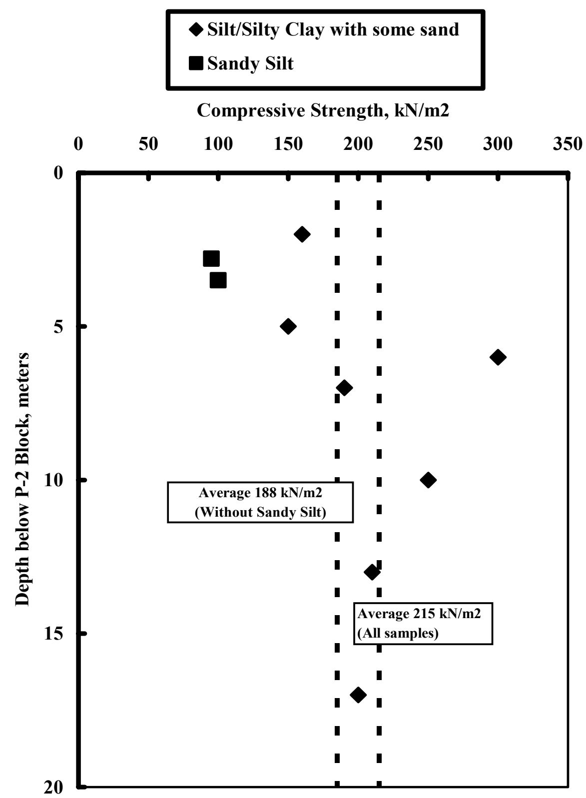

Fig. 5. Unconfined Compressive Strength Versus Depth

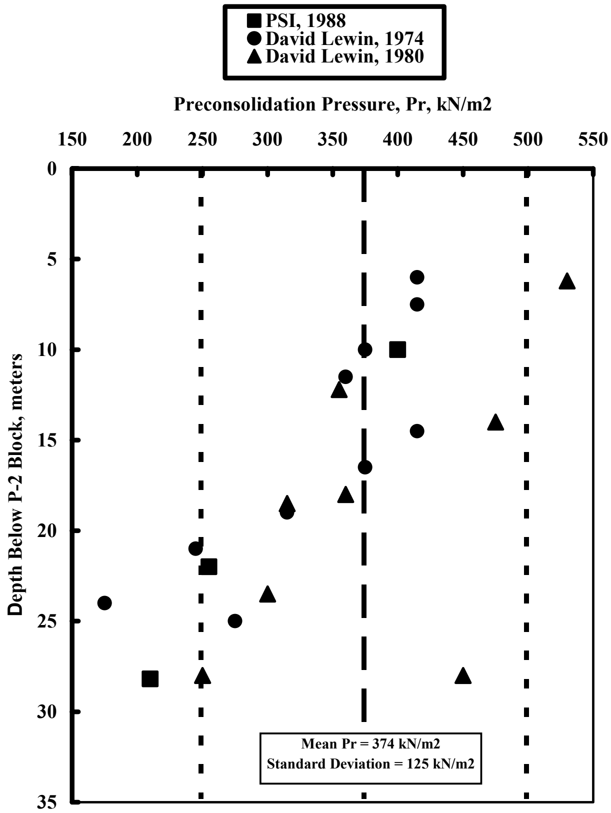

Fig. 6. Preconsolidation Pressure, Pr, Versus Depth

The vertical stress distribution at each selected column location, for settlement determination, was computed according to Boussinesq theory of elasticity using the computer program SETTL/G titled "Settlement and Stress Distribution Analysis" and developed by Geosoft of Orange, California. For a given location, the computer program evaluates variation of the applied vertical stress with depth while taking into consideration the effects of the surrounding loaded areas within the zone of influence of the selected column.

Five soil layers ranging in thickness of between about 1.52 m and 5.1 m were used. The water table was assumed at a depth of 3.05 m below the presently existing surface grades within the P-2 Block area.

A total of fifty-seven locations including fifteen within the East Skylight Tower, eight for the West Office/Hotel Tower, twenty-five for the Steam Concourse and nine outside the P-2 Block area were chosen to estimate settlements. The settlements were calculated using one-dimensional consolidation theory and the above-referenced computer program. Stress distribution calculations indicated the vertical stresses within the selected soil layers to be within the upper and lower 90% confidence limits for the preconsolidation pressure shown in Fig. 6; therefore, the recompression index was used for the settlement analysis.

The settlement analysis showed maximum settlements of about 13.2 cm within the central portions of both the East and West Towers. The settlements were generally anticipated to decrease toward the periphery, though not at the same rate at all locations. Within the Steam Concourse area, maximum settlement of 7.9 cm was indicated. Actual variations in settlements, however, were expected to be governed by the foundation size, imposed loads, interaction of the surrounding loaded areas and variability of the subsurface conditions at a given location.

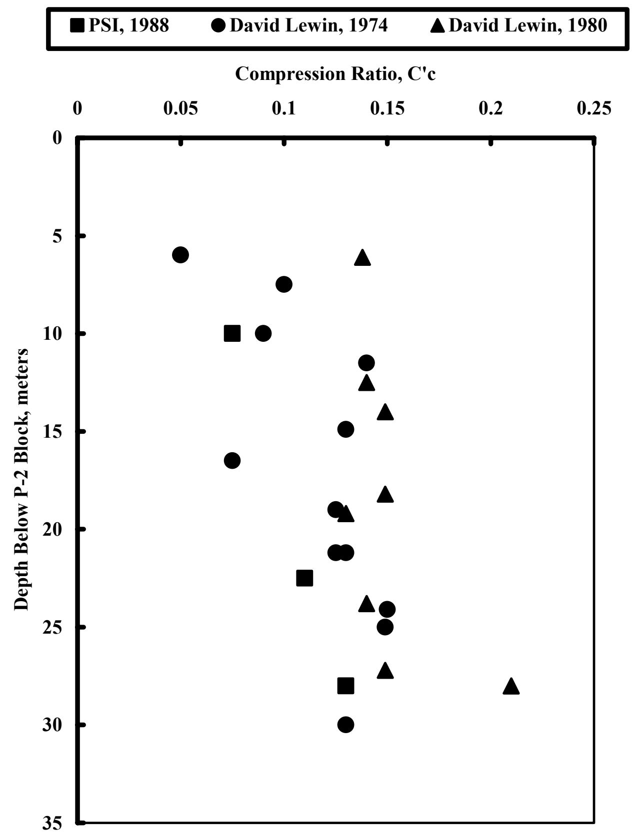

Fig. 7. Compression Ratio C'c Versus Depth

Maximum differential settlements were anticipated where the footing sizes and pressures at the soil/foundation interface change suddenly. From the calculated settlements, it appeared that the maximum differential settlements of 8.4 cm with a probable range of 4.3 to 8.4 cm would occur between adjacent columns where the East Tower meets Steam Concourse. Between the West Tower and Steam Concourse, the analysis indicated maximum differential settlements of 4.6 cm with a probable range of 2.3 to 4.6 cm.

Within the East and West Towers, maximum differential settlements of, respectively, 3.96 cm with a probable range of 1.78 to 3.96 cm and 5.08 cm with a probable range of 2.54 to 5.08 cm were possible.

Within the steam concourse area, maximum differential settlements were expected not only at its connection with the bounding towers but also between some of the adjacent footings such as the heavily loaded new footing 44A and the lightly loaded existing caisson 44. Caisson 44 was not expected to settle appreciably due to interaction resulting from settlement of the surrounding loaded areas. A differential settlement of 7.6 cm with a probable range of 4.1 to 7.6 cm was considered possible between caisson 44 and the new footing 44A.

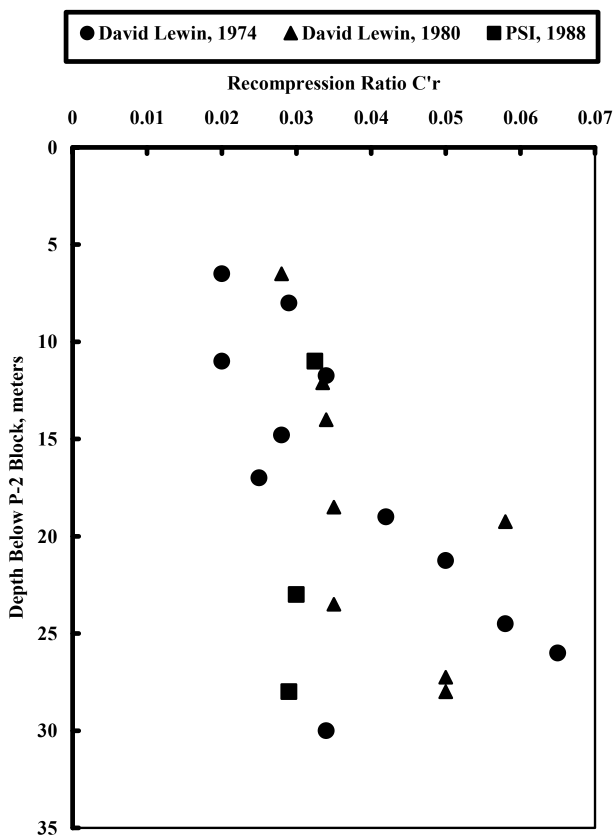

Fig. 8. Recompression Ratio, C'r Versus Depth

Total settlements would be expected to decrease as the horizontal distance from the proposed tower increased. For example, maximum settlements at Terminal Tower, Post Office Building, Midland Building and Huron Road were estimated to vary between 0.2 and 0.4 cm as a result of the new construction.

Table 2 provides a summary of the estimated maximum total and differential settlements, due to the contemplated new construction, for the various structures evaluated.

These settlements are very large and posed serious problems for the structural engineers in the design of structural connections within a given structure itself as well as its juncture with the adjoining structures. The maximum theoretical differential settlements within the East and West Towers were reduced from the maximum of, respectively, 4.0 and 5.1 cm to 2.5 and 4.0 cm by relieving the critical corner columns of the building façade load.

Table 2. Estimated Maximum Total and Differential Settlements Due to New Construction

Structure	Total Settlement cm	Differential Settlement cm
East Tower	6.9-13.3	1.8-4.0
West Tower	6.9-13.3	2.5-5.1
Steam Concourse	4.1-7.6	7.6
Steam Concourse/East Tower	--	8.4
Steam Concourse/West Tower	--	4.6
East Tower/West 2nd St. Bridge	--	1.5 or >
Terminal Tower	0.2	--
Midland Building	0.2	--
Post Office	0.4	--
Huron Road	0.2	--

A part of the total and differential settlements was anticipated to be realized during the course of the proposed construction. Estimates of the magnitude as well as duration over which these settlements will occur are generally extremely difficult to predict since construction sequence and loading conditions at the individual footings, as the construction progresses, can never be estimated with any reasonable degree of confidence. Some relief in reducing detrimental effects of total and differential settlements was provided by the construction team by agreeing to alter the construction sequence of some of the structural elements to permit sufficient time for the foundations to adjust to the applied building loads.

THEORETICAL VS. ACTUAL SETTLEMENTS

Prior to the initiation of the new construction, the presently existing elevations of columns 48, 115 and 120 were measured and compared with the elevations established at these locations at the time of the original construction in the 1920s. At a given point, the difference in the two elevational readings is assumed to be the actual settlement that has taken place since the original construction. The measured settlements are tabulated in Table 3. This table also includes the predicted settlements which would have been expected due to the existing loads (prior to new construction) at these locations, utilizing the previously outlined parameters and analysis. The tabulated data indicate that the actual settlements are almost half to one-quarter of the predicted settlements. Based on this data, one may conclude that the actual settlements in the field due to new loads will be far less than the predicted values.

Table 3. Summary of Measured and Estimated Settlements Due to 1920's Construction

Column Location	Measured Settlement cm	Estimated Settlement cm
48	1.4-1.7	3.2-6.3
115	1.4-2.5	2.7-5.3
120	1.4-3.0	3.2-6.2

Generally, the discrepancy between the predicted and actual settlements can be attributed to several factors, including, but not limited to, ideal laboratory conditions for the consolidation tests, variable soil and water conditions in the field, evaluation of construction sequencing on the settlements, and uncertainty in the prediction of load, particularly, true live loads.

Actual monitoring of the settlements was initiated on June 30, 1989 and continued until at least May 29, 1990. Although the construction activity started in July 1988, with the exception of minimal steel erection for only the Steam Concourse only, the existing foundations had not been subjected to any load. Therefore, June 30, 1989 was considered a datum for the field settlement response.

As of May 29, 1990, i.e. eleven months after the initiation of the settlement monitoring, the maximum total and differential settlements shown in Table 4 were recorded.

Table 4. Field Settlement Data Summary

Structure	Settlements, cm Total	Differential
West Tower		
Overall	3.1	1.0
North Exterior	3.1	0.1
South Exterior	2.4	0.1
West Exterior	2.8	1.0
East Tower		
Overall	3.5	1.1
North Exterior	2.8	1.1
South Exterior	2.7	0.9
East Exterior	3.1	1.1
Steam Concourse		
Overall	2.7	0.6
North Exterior	N/A	N/A
South Exterior	2.7	0.6

The corresponding foundation loads at the time of the last settlement reading are included in Table 5.

Table 5. Field Structural Loads

Structure	Present Loads, Percentage of Design Load: Dead Load	Live Loads
West Tower		
Interior Columns	95	15
Exterior Columns	90	20
East Tower		
Interior Columns	85	5
Exterior Columns	80	4

Many structures can tolerate substantial downward movement without cracking; however, it is general engineering practice to limit total settlements to less than 5.0 cm for most facilities (ASCE, 1994). Differential settlement, which causes distortion and damages in structures, is a function of the uniformity of the soil, stiffness of the structure, stiffness of the soil, and distribution of loads within the structure. Differential settlements should not usually exceed 1.25 cm in buildings, or cracking and structural damage may occur (ASCE, 1994).

Comparison of the theoretical predictions with the field settlement data indicated that the measured total and differential settlements were approximately 40 percent of the settlements expected under the full design loads, and that the ultimate settlements were not expected to exceed 70 percent of the total theoretical settlements shown in Table 2. Table 6 shows the expected long-term settlement values.

Table 6. Projected Long-Term Settlements

Structure	Total Settlement cm	Differential Settlement cm
West Tower	9.1	2.5
East Tower	9.1	3.6
Steam Concourse	5.6	5.0
Steam Concourse/East Tower	--	5.8
Steam Concourse/West Tower	--	3.3

The above-tabulated settlements at the project site, though still high, are considerably lower than those originally anticipated and reported in Table 2. The structural engineers were able to adequately accommodate these settlements in the design of the critical structural elements and their connections. Structural connections were designed to allow maximum flexibility of new construction. Butt joint spacing between the adjacent building façade panels was adjusted for the settlements while control joints were introduced at frequent intervals to minimize visual impact of any settlement cracking. Also, the individual structures were structurally divorced from each other to account for thermal and construction variations, and thus alleviate problems in adjacent elements due to anticipated large settlements.

CONCLUDING COMMENTS

1. Many studies had been conducted by others since the early 1970's to explore the viability of using the existing foundations for the redevelopment of the site, but had concluded that this project was not feasible in view of the anticipated large settlements. However, by working together with the developers and the design team, the planned development became a reality, by the developers agreeing to alter the scope of the development while the designers used innovative techniques for load reduction on the critical structural elements, in view of the expected large settlements.

2. From the onset of the project, everyone was cognizant of the fact that to alleviate the damaging effects of the anticipated large total and differential settlements, proper sequencing of the construction activities would be extremely critical to the project. This meant delaying installation of those structural elements which were expected to be affected by foundation movements until the field movement monitoring data indicated that a major portion of the settlements had occurred. To achieve this, cooperation of the construction managers and other trade contractors was deemed essential, and successfully accomplished, by including them with the developers and designers in the decision making process.

3. Continually monitoring of the settlements greatly assisted the project structural engineers to continually evaluate and modify, wherever possible, the load-transfer mechanisms and structural connections for the various structures to limit settlements to levels that could be practically accommodated in the design.

4. Close partnership between the geotechnical engineer and the design team continued until evaluation of the settlement data indicated that the long-term structural integrity of the development would not be expected to be compromised because of foundation settlements under the new construction loads.

5. The development, which was once thought to be impractical at the subject site, in view of the expected large total and differential settlements, was completed within budget and on time.

ACKNOWLEDGEMENTS

Geotechnical Engineering Evaluation and Construction Materials Testing and Observation for the project were conducted by PSI under contract with Tower City Properties/Forest City Enterprises, Inc. RTKL Associates, Inc. of Dallas, Texas and Cannon Design, Inc. of Grand Island, New York were, respectively, the Project Planner/Architect and Structural Engineers.

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