NUMERICAL MODELING AND USE OF SETTLEMENT REDUCING AUGER CAST-IN-PLACE PILES BELOW A MAT FOUNDATION

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ABSTRACT

The 45-story Great American Tower, the tallest building in Cincinnati, is not only changing the downtown skyline, but is an excellent case study of innovation in geotechnical engineering. This project is an excellent example of the use of technological advances in site characterization and soil-structure methods. When an opportunity arises to combine state-of-the-art concepts with advanced modeling tools, engineers need to combine knowledge and forward thinking to geotechnical solutions to promote the state-of-practice. The Great American Tower at Queen City Square in Cincinnati, Ohio combines a mat foundation with a limited number of auger cast-in-place (ACIP) piles, with the piles primarily acting as settlement reducers. Promoting the unique soil-structure interaction based foundation system required the use of sophisticated numerical modeling tools and seamless communication with the designers, contractor, and owner. Traditional standard penetration test boring data (SPT) was initially used to develop a numerical analysis of the soil-structure interaction using FLAC 3D software. The model was further modified with cone penetration (CPT) and pressuremeter testing (PMT), load test results on several ACIP pile elements of varying lengths, but all tipping above bedrock, and on-going monitoring. Integrating industry knowledge, with sophisticated modeling techniques, has provided a successful real-world case study.

INTRODUCTION

New or state-of-the-art concepts and methodologies within the traditionally theoretical field of geotechnical engineering are often met with skepticism and resistance. As a result, the industry struggles to advance the state-of-practice. When a schedule-driven contractor and cooperative owner teamed with a progressive team of designers, the result was the use of advanced sophisticated numerical modeling tools to design a mat foundation system combined with a limited number of auger cast-in-place (ACIP) piles, with the piles primarily acting as settlement reducers. Terracon's engineering team suggested a design approach that uses ACIP piles to stiffen the mat foundation and act as settlement reducers where the piles bear in the traditional zone above bedrock. The number of piles needed using this approach is significantly less than if the mat foundation was structurally supported on piles.

Early in design it was thought that the building could be supported on a mat foundation due to several constructability advantages. Predicted settlements of a traditional mat foundation for support exceeded tolerable movements due to a variably thick zone of moderately to highly compressible lakebed soils across the structure footprint. Common practice in the local area would suggest the use of ACIP piles to bedrock; however, the costs and construction time, and

estimated 1,200+ ACIP piles extending to bedrock, were not favorable to the project completion timeline, construction issues with large pile caps in a constrained deep excavation, and the owner's budget.

The unique soil-structure interaction based foundation system needed a more thorough understanding of the properties of the subsurface and a reliable way of modeling the stresses from the structure and their interaction with the soils. Three-dimensional numerical analysis software was used to model the soil-structure interaction. Such tools allow the geotechnical designer to model different types of soil and groundwater conditions, consider interaction between structures and surrounding geo-materials, simulate construction, and predict structure deformation and soil movement during excavation and under specific loading conditions.

Through numerous discussions and consulting between the projects structural engineers, architects, and owner, and based on the results of detailed numerical modeling analysis, a total of 281 ACIP piles located at specific locations below the mat foundation were designed to reduce mat foundation total and differential settlement and mat stresses to within tolerable limits. The design experience proved to be challenging, as it used advanced geotechnical analysis and design to mesh

traditional mat and ACIP pile foundations into a unique soil-structure interaction based foundation system.

This paper focuses on the design steps, analyses, and methodology that developed into the ACIP pile settlement reducer/mat foundation system for the now completed Great American Tower. Pertinent project information, along with an overview of the general subsurface and geologic conditions at the project site is also provided. The mat foundation and ACIP piles were monitored throughout construction using sophisticated instrumentation to assist with confirmation of the soil-interaction model used during design.

PROJECT INFORMATION

The Great American Tower consists of an office tower and a parking garage northeast of the intersection of 3rd and Sycamore Streets in downtown Cincinnati, Ohio. The 203-meter (665-foot) tall Great American Tower will provide 92,903 square meters (1,000,000 square ft) of office space. The office tower measures about 46 by 67 meters (150 by 220 ft) in plan measured north-to-south and east-to-west, respectively). The tower occupies the approximate southern one-third of the site. North of the tower, the parking garage area, along with a promenade with retail and restaurant space, measures about 76 by 37 meters (250 by 120 ft) in plan measured north-to-south and east-to-west, respectively. The project includes approximately 1,858 square meters (20,000 square ft) of retail space and a 2,350 car parking garage. Construction of the project began in 2008 was fully open for occupancy in early 2011. The completed tower along with the Cincinnati skyline is shown in Fig. 1.
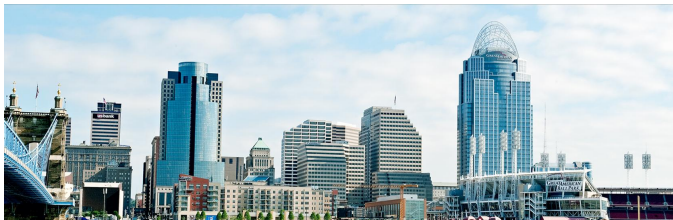
Fig. 1. Cincinnati, Ohio skyline with Great American Tower to the far right.

BACKGROUND AND PREVIOUS SITE USE

Establishing the three-dimensional model for analysis of the foundation system for the Great American Tower considered previous and existing development at the site. The downtown setting for the project site just north of the Ohio River resulted in several cycles of construction and demolition. The most recent construction was a seven to nine story above-grade parking garage across the majority of the site. The garage had three to four stories below grade. The previous brick and concrete masonry garage was constructed in 1968 and was founded on spread footing foundations. The structure, including foundations and below grade walls, were completely

demolished and removed prior to construction of the new tower and garage (Fig. 2).

Prior to the 1968 construction of the parking garage, there were existing structures at the north end (a multi-level garage and an 8-story brick building). There were also two narrow brick buildings along 3rd Street. Other buildings existed along 3rd Street, but have since been razed. The basement depths of the previous structures were variable.

Fig. 2. Demolition of 7 to 9-story Parking Garage

The Great American Tower is the second phase of the overall development. Construction of the first phase – adjacently located to the east – was completed in 2002. Design requirements for the overall complex included "linking" the two structures at two floors. The first phase construction is supported on xx-mm (18-inch) diameter, approximately XX m (50 to 60 ft) long ACIP piles bearing on/within the interbedded shale and limestone bedrock over XX m (100 ft) below existing site grades. The lowest level in the first phase construction is about xx m (6.5 ft) below the mat subgrade elevation of the Great American Tower.

Prior to the first phase construction, a parking garage structure supported on spread footings occupied the site. The garage was razed during construction of the first phase.

GEOLOGIC CONDITIONS

In addition to previous site use, the variable geology and depositional history at the site was considered in the foundation model. The project site maps within the Cincinnati Basin, which generally consists of granular glacial outwash and earlier water-deposited lakebed material in the subsurface. The deeper lakebed soils were deposited within lakes created by advancing ice sheets, which dammed the northward flowing deep stage river. These lower deposits were later covered by granular outwash materials consisting mainly of sands and gravels, with varying amounts of silt. The outwash was eroded during various glacial periods to varying degrees. Glacial terraces exist along portions of the river such as downtown Cincinnati where substantial depths of outwash exist. The glacial outwash typically reduces in thickness as the Ohio River is approached and is nearly absent across the project site.

Lakebed deposits (i.e., lakebed bottom sediments) consist essentially of clays and silts, but with occasional sand layers. Lakebed soils are characterized by a gray to dark gray color, but sometimes brown in the upper oxidized zone. The soils typically occur as silty clay, lean clay, and occasionally plastic to fat clays. The lakebed soils have a varved appearance due to the presence of thin layers, lenses, or partings of silt and sand. Lakebed soils are usually low to moderate in strength, but can be very stiff in the lower profile due to overconsolidation effects. These soils exhibit a moderate to high compressibility, with moderate to high moisture contents and occasional organics.

The underlying bedrock at the project site maps as Ordovician Age interbedded shale and limestone of the Kope Member. Based on the bedrock elevation at the site, the bedrock falls within the Economy Member of the Latonia Formation. The bedrock is "shale-rich", with shale making up about 75% of the formation and limestone making up the remaining 25%.

SUBSURFACE CONDITIONS

Terracon Consultants, Inc. has performed numerous geotechnical studies in downtown Cincinnati and multiple studies within the limits of the Great American Tower complex. The earliest study was performed in 1968 for the now demolished parking garage. During the preliminary planning for the project in the 1980's and 1990's, geotechnical studies were performed in 1985, 1989, and 1991. Additional studies were performed in 2001, 2002 and 2008 as the plans for the project evolved. In total, nearly fifty test borings were performed at the site as well as Cone Penetrometer Testing (CPT) and Pressuremeter Testing (PMT).

The subsurface profile was subdivided into seven major strata based on material type, geology, and engineering characteristics. Existing fill (Layer 1) was encountered below the ground surface at the majority of the test borings in the project area. The existing fill was underlain by a relatively thick granular zone, which was divided into "upper sands" (Layer 2), "fine sands" (Layer 3), and "lower sands" (Layer 4) based on consistency and gravel content. The predominant material underlying the site consists of Lakebed soils (Layer 5). Compressible Lakebed soils were encountered below the granular zone. A relatively thin and variable transitional zone of cohesive and/or granular soils (Layer 6) was encountered below the lakebed and prior to encountering bedrock (Layer 7). Bedrock was encountered at depths of approximately 27.5 to 43 m (90 to 140 ft) below the existing grades. A cross-section of the subsurface profile across the Great American Tower footprint is shown in Fig. 3.

Existing fill (Layer 1) was encountered at the majority of the test borings drilled across the project area. The fill encountered was variable in thickness and content, and was encountered up to about 6.1 m (20 ft) below existing grades in the test borings. Upper Sands (Layer 2) generally contained glacial outwash soils consisting of fine to coarse sands, with varying amounts of gravel. This medium dense to dense layer had a maximum thickness of about 10.7 m (35 ft) in the northern and mid-portions of the site (parking garage area); however, was minimal or even absent in the southern portion of the site (office tower area). An approximate 1.5 to 6.1 m (5 to 20-ft) zone of predominantly fine sandy soils (Layer 3) were encountered below the upper coarser zone in the proposed parking garage area and below the existing fill in the tower area. Similar to Layer 2, the Lower Sand zone (Layer 4) generally consisted of dense to very dense, fine to coarse sands, but with larger gravel content (i.e., sands with gravel, and sand and gravel). This layer was encountered relatively uniformly across the project area. The thickness of this layer was up to 6.1 m (20 ft) and extended to depths of about 13.7 to 22.9 m (45 to 75 ft) below existing grades at the site.

Underlying the granular soils, glacial lakebed material (Layer 5) was encountered in all of the test borings and generally extended to a maximum depth of about 36.6 m (120 ft) below existing grade. This layer was typically 9.1 to 15.2 m (30 to 50 ft) thick and consisted of lean clays, fat clays, clayey silts, and sandy lean clays. A "transitional layer" (Layer 6), just above the bedrock below the site varied from sandy lean clays to sands, and sands and gravels, with gravel and rock fragments. The thickness of this zone between the lakebed soils and underlying bedrock had a variable thickness, which ranged between about 1.5 and 7.6 m (5 and 25 ft). Shale (75 to 80% of matrix) and limestone (20 to 25% of matrix) bedrock (Layer 7) within the project area was generally encountered at depths of about 27.4 to 42.7 m (90 to 140 ft) below existing grade, or between about elevations 129.8 and 123.7 m (426 and 406 ft).
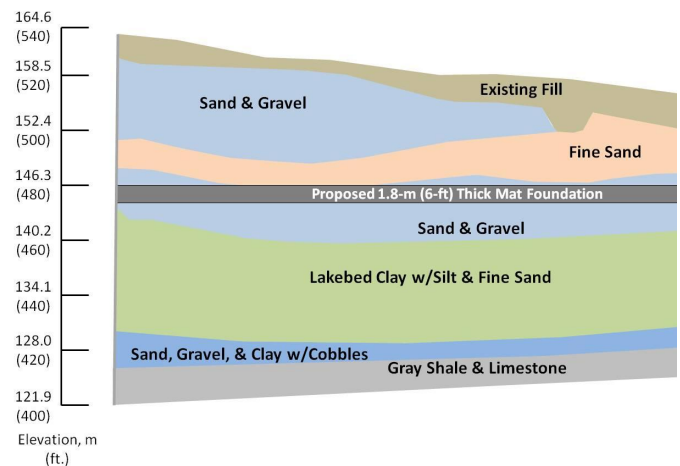

Fig. 3. Generalized Subsurface Profile

Based on observations in the boreholes and piezometer data, the groundwater table is at about elevation 143 m (470 ft) near the south end of the site (area of the office tower) and rises to about elevation 145 m (475 ft) to the north (area of parking garage). Based on review of the groundwater data, there does not appear to be a significant change in the groundwater table below the project site when the river stage changes. However,

this reaction to river stage was not ignored in during our evaluation, since water levels at the site likely depend on the height of river stage and duration of elevated stage.

Based on the USGS topographic quadrangle for Covington Kentucky-Ohio, the normal pool elevation of the Ohio River in the project area is elevation 139 m (455 ft) at Mile Marker 470). The 10-year and 100-year flood levels are presently at elevations 148 and 152 m, respectively (elevations 487 and 498 ft, respectively). The recorded maximum flooding occurred in 1937 where the river reached elevation 155 m (elevation 510 ft) or slightly over 9.5 m (31.5 ft) above the mat foundation subgrade elevation.

FOUNDATION SYSTEM EVALUATION

The type of foundation support for the Queen City Tower was thoroughly considered and multiple foundation options were evaluated. The anticipated high structural loads, load distribution, and total/differential settlement limitations, along with the relatively thick compressible lakebed soils as shallow as 3 m (10 ft) below the proposed lowest level elevation, each impacted the evaluation of feasible foundation alternatives. Shallow spread footings were not considered feasible for support of the office tower. Large footing sizes [over 6.1 m (20 ft wide] would be needed and settlements would be well in excess of tolerable limits.

Driven steel H-piles or pipe piles were considered; both have been successfully used in downtown Cincinnati construction. Piles would need to be driven to a practical refusal on bedrock, which would require pile lengths ranging between about 12.2 and 24.4 m (40 and 80 ft). Due to the urban setting, the risks associated with vibrations that occur during driving piles include damage to adjacent structures and/or utilities, which may or may not be immediately evident made driven piles an unattractive option.

Both straight shaft drilled piers and ACIP piles socketed into bedrock were considered as a viable foundation alternative for support of the proposed construction. Pier/pile element lengths could be as much as 24.4 m (80 ft) and would require penetration of granular zones below the groundwater table. Caving of the drilled pier during construction and tremie methods for concrete placement were anticipated. The drilled piers were cost prohibitive and would take much longer construction time than the project schedule allowed.

A mat foundation was thoroughly analyzed using FLAC 3D as a cost-effective approach to eliminate or minimize the number of elements required in a deep foundation system. Initial analyses showed that a traditional mat foundation would still result in unacceptable settlements. A mat supported structurally on piles would result in significant cost and long construction time. Therefore, a "piled mat foundation system" consisting of a mat supported by an optimal (limited) number of deep foundation elements was selected. Deep foundation elements (ACIP piles) primarily serve as settlement reducers. A mat foundation, with an optimal number of ACIP piles (i.e., augercast piles), is considered a cost-effective alternative. This is not a structurally pile-supported mat, but rather the primary purpose of the piles is to reduce total and differential settlement. A mat foundation system was not considered in the parking garage area due to the potential for varying lowest level grades across the garage footprint and the resulting constructability challenges. Tradition ACIP piles were evaluated for support of the parking garage portion of the construction.

DESIGN METHODOLOGY

In the office tower area, the design methodology for use of the ACIP piles to "stiffen" the mat foundation and act as settlement reducers considers terminating the piles in the Layer 6 transitional zone above the bedrock. The pile elements are not extended to top of bedrock. The number of piles needed using this approach is significantly less than if the mat foundation was structurally supported on piles. The pile locations have been selected based on detailed analyses to maximize the benefits of settlement reduction while reducing the total number of piles needed. The mat foundation and ACIP pile elements were modeled using the numerical modeling program – FLAC 3D. Several iterations of the numerical model were performed and the results were provided to the project team following each iteration.

The primary concern with the soil subgrade supported mat foundation is the magnitude and distribution of total and differential settlements. Settlement across the mat due to the building loads will be non-uniform and must be accurately estimated in order to ensure that the tolerable limits are not exceeded. The specified maximum differential settlement, between the center core and the edge of the mat is 25 mm (1 in). Differential settlement between adjacent columns was limited to 12 mm (1/2 in).

Settlement is influenced by several factors. The complex and heterogeneous subsurface profile, comprising of a variety of soil types, starting with existing fill, then fine sand, sand and gravel, silty clay (lakebed), dense sand and gravel, and shale bedrock, responds to the imposed loads differently. In addition, the thickness of each layer of soil varies across the site. Therefore, even if load conditions are symmetric, settlements will not be symmetric. A second factor is the construction sequence. A specific soil layer will respond differently to an imposed load if it experiences different stress paths. For example, ground settlement will be different if a soil stratum is experiencing a direct loading, compared to when the soil stratum is experiencing an unloading (removal of existing structure) and reloading (construction of piles, mat, and superstructure) condition. The combined weight of the piles, mat and superstructure is much higher than the weight of the excavated soil; therefore, the soil below the mat foundation will experience large settlements. The loading

sequence, variability of subsoil conditions, and non-symmetrical loading, results in a complex stress condition change that needs to be accounted in the design analyses. The loads from the superstructure are not symmetrical and their actual distribution on the mat must be accurately modeled. A third factor is soil-structure interaction. The pattern of foundation settlement depends not only on the soil and external load conditions, but also on the rigidity of the building structure. That is, the rigidity of the structure can even out differential settlement through the interaction between structure and foundation soil. The loads from the superstructure are not symmetrical and their actual distribution on the mat must be accurately modeled.

A conventional settlement analysis cannot include all the above factors. Several simplifying assumptions need to be made in conventional settlement analyses, which may result in inadequate modeling and oversimplification of the design. A conventional settlement analysis will also not be able to account for the rigidity of the central core. A 3D numerical modeling program was able to model the mat in adequate detail, including modeling behavior of the different soils with appropriate soil constitutive models, tracing the construction sequence, accounting for the actual load distribution, and simulating the interaction between the foundation soil, mat foundation and superstructure.

MODELING SOFTWARE

FLAC 3D is an advanced three dimensional continuum modeling for geotechnical analysis of rock, soil, and structural support, and is one of the most widely used three dimensional numerical modeling tools for geotechnical analysis of soil-structure interaction problems. Three-dimensional numerical modeling overcomes many of the assumptions and shortcomings of conventional analysis.

FLAC 3D has been available for over 15 years and is used by engineers, consultants, and in university teaching and research. It is currently licensed by over 900 users in over 54 countries – making it one of the most widely used three-dimensional numerical modeling tools for geotechnical analysis in the world. Three-dimensional numerical modeling overcomes many of the assumptions and shortcomings of conventional analysis and allowed designers to model many subsurface and loading variables of the project.

GEOMETRY AND SOIL CONDITIONS

The initial numerical model established for the analyses included plan dimensions of 152.4 m (500 ft) in the north-south direction and 97.5 m (320 ft) in the east-west direction [the model includes a 15.2-m (50-ft) wide soil area beyond the perimeter of the building area on each side). The top elevation of the model varies from elevation 157.3 to 164.6 m (elevation 516 to 540 ft), following grade elevations. The bottom

elevation of the model is elevation 121.5 m (elevation 398.5 ft), which is approximately 5.2 m (17 ft) into the shale bedrock. The model also includes the different soil strata. The initial model described here is shown below as Fig. 4. The name designations of each soil stratum and corresponding colors are shown in the legend on the left side of the diagram. Boundary conditions consisting of 1) no movement is allowed across the bottom of the model and 2) only vertical movement is allowed along the sides of the model were assigned.
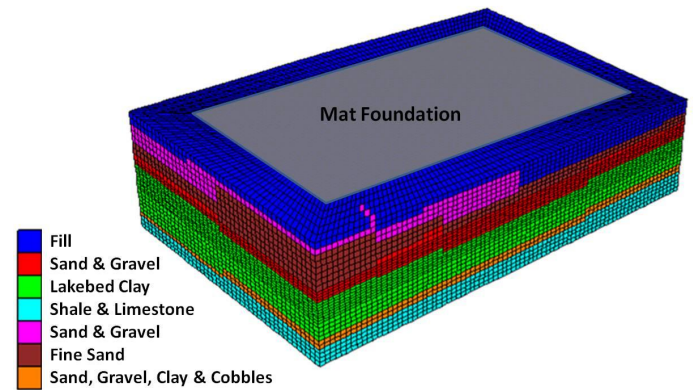

Fig. 4. FLAC 3D Subsurface Model

ANALYSIS

With the development of numerical methods, it has become feasible to analyze and predict the behavior of complex soil structures and solve soil/structure interaction problems. Such analyses depend considerably on the proper representation of the relations between stress and strain for each material and soils represented in the project. In numerical computations the relation between stress and strain in a given material is represented by a so-called constitutive model, which consists of a mathematical function of several parameters.

The Mohr-Coulomb model, one of the most widely used elasto-plastic models, was used in our calculation for the granular soil and existing fill. The Cam-Clay model was used in our analysis for the lakebed clay and silt. The parameters for these models were obtained from field measurements, laboratory tests, and/or published data. The parameters are tabulated in the following table. Some judgment was exercised in selection of the various soil model parameters.

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The analysis consists of three major steps, or conditions, which influence what is/will be felt by the soils. The three steps include establishing the initial stresses in the soil strata under the weight of overburden soil, simulating the excavation

for the mat foundation, and calculating the settlement of the mat under the building load. The three steps are illustrated in Fig. 5.
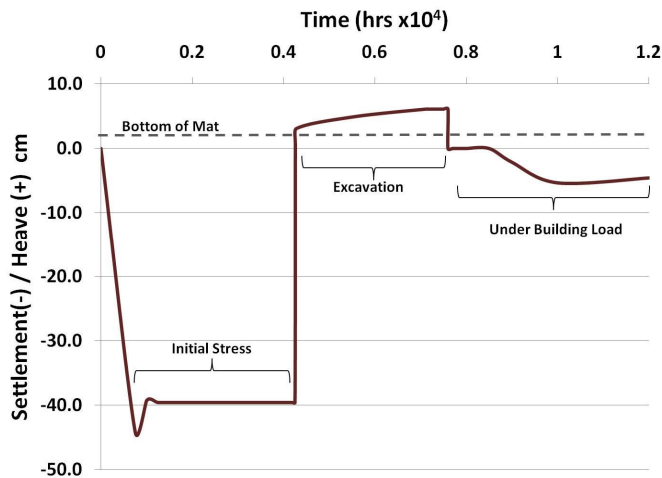

Fig. 5. Foundation Settlement during Construction Sequence

The first step established the initial stresses in the soil strata under the weight of overburden soil. The model illustrates how the vertical stress increases with depth. Near the bottom of the mat foundation, the vertical stress is approximately 143.6 kPa (3,000 psf).

The second step of the analysis simulated excavation for the mat foundation. In this process, the soil in the building area is removed from the surface to the bottom of the mat foundation at elevation of 145.8 m (478.5 ft). During excavation, a shoring system is installed from top to bottom. Therefore, no horizontal movement is allowed in the wall of the excavation. Excavation releases the vertical stress at the bottom of the excavation, therefore the bottom of the excavation heaves. The largest heave was calculated to be about 10.2 cm (4 in) and occurred near the northeast corner of the excavation where the largest amount of soil is removed and where lakebed clay is the thickest. In the field, the bottom heave may not be noticed, because heaved soil will be cut down by the excavator to the design elevation. Also, the excavation process will occur over a period of time causing stress relief over time. Therefore, in the calculation, the heave-related deformation was set back to zero before the next step of the calculation. Excavation will also change the stress conditions in the soil profile.

The third step of the analysis was to calculate the settlement of the mat under the building load. The load distribution and magnitude was provided by the structural engineer. Three foundation options were analyzed, including: the building supported only by a mat foundation; mat foundation with building core affect; mat with limited number of piles for settlement reduction and building core effect.

RESULTS OF ANALYSES

Mat Foundation on Soil Subgrade – Full dead load and fifty percent of live load was used in the analyses for estimating the mat foundation settlement. The calculated maximum settlement was about 25.9 cm (10.2 in) near the center of the mat. The maximum differential settlement was about 17.0 cm (6.7 in) over a distance of about 22.9 m (75 ft) in the north-south direction. The variation of the settlement across the mat foundation is shown in Fig. 6.

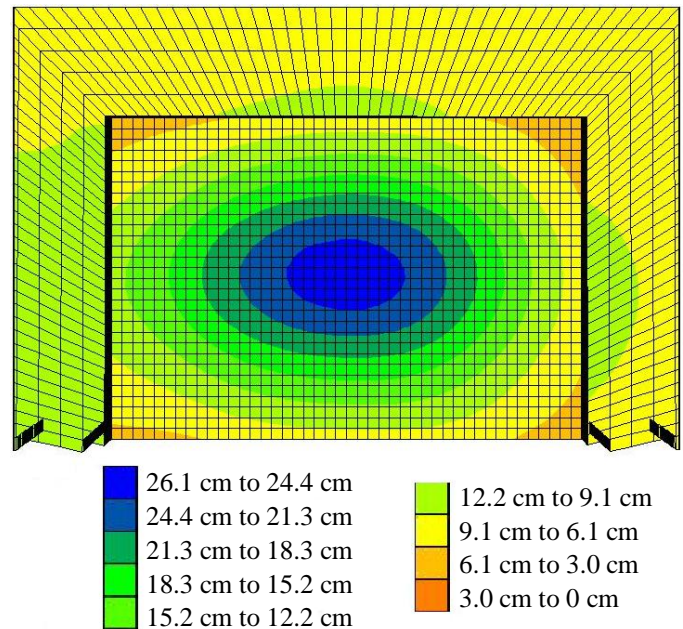|	
26.1 cm to 24.4 cm	12.2 cm to 9.1 cm
24.4 cm to 21.3 cm	9.1 cm to 6.1 cm
21.3 cm to 18.3 cm	6.1 cm to 3.0 cm
18.3 cm to 15.2 cm	3.0 cm to 0 cm
15.2 cm to 12.2 cm	

Fig. 6. Settlement Contours – Mat Foundation Only

Mat Foundation with Building Core – The building core is a very rigid concrete structure located at the center of the tower and is connected to the mat, which enhances the mat rigidity when modeled as a composite structure. The enhanced rigidity of the mat foundation can serve to reduce differential settlement across the mat. In order to accommodate the effect of the core, the core was simulated into the model. The maximum settlement of the mat with the core added is reduced to about 12.2 cm (4.8 in) from about 25.9 cm (10.2 in) without the core. The maximum differential settlement is about 10.7 cm (4.2 in). Even with the core attached to the mat, the settlements are considered too large. The variation of the settlement across the mat foundation modeled with building core is shown in Fig. 7.
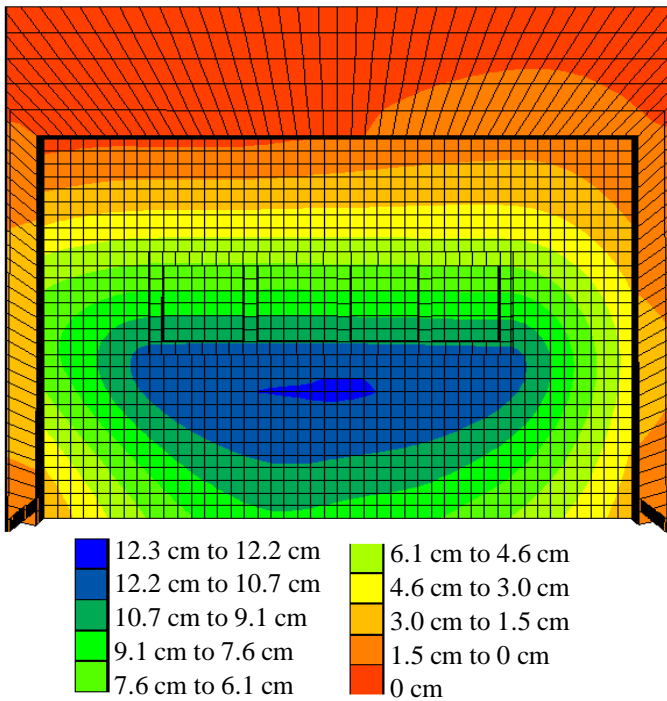
Fig. 7. Settlement Contours – Mat Foundation with Building Core

Legend:
- 12.3 cm to 12.2 cm
- 12.2 cm to 10.7 cm
- 10.7 cm to 9.1 cm
- 9.1 cm to 7.6 cm
- 7.6 cm to 6.1 cm
- 6.1 cm to 4.6 cm
- 4.6 cm to 3.0 cm
- 3.0 cm to 1.5 cm
- 1.5 cm to 0 cm
- 0 cm

Mat Foundation with Limited Number of Piles and Building Core – In a conventional pile foundation design the entire structural load is carried by the piles. The mat serves as a large pile cap to distribute and transfer the loads to the piles. However, in this project, the pile supported mat is a combined foundation, where piles act as settlement reducers and only carry a portion of the load, allowing the mat to settle so that the soil subgrade can carry a portion of the total load. The soil-structure interaction between the mat and soil subgrade along with the piles serve to reduce foundation settlement. The locations of the piles (light hash marks) and the computed mat settlements, and the mat contact pressure distribution with the piles modeled are shown in Fig. 8 and 9, respectively.
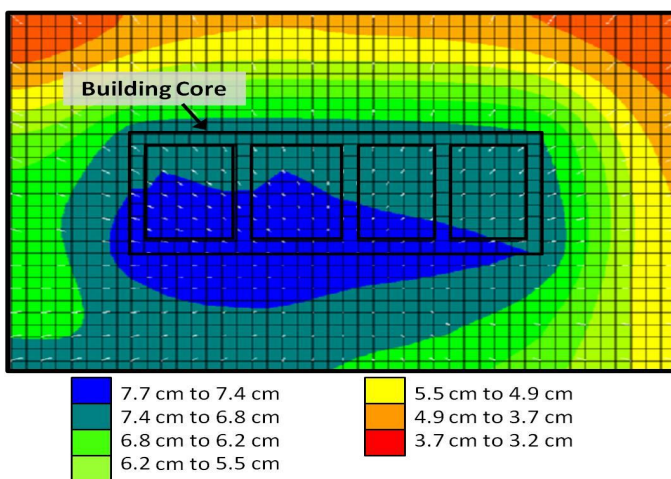

Fig. 8. Settlement Contours- Mat Foundation with Settlement Reducing Piles

Legend:
- 7.7 cm to 7.4 cm
- 7.4 cm to 6.8 cm
- 6.8 cm to 6.2 cm
- 6.2 cm to 5.5 cm
- 5.5 cm to 4.9 cm
- 4.9 cm to 3.7 cm
- 3.7 cm to 3.2 cm

The design of the mat foundation with piles was carried out as a two-stage process involving a preliminary design phase to obtain an approximate assessment of the required number of piles and a detailed phase (using 3D FLAC numerical modeling) to refine piling requirements and locations and provide spring constant information for the structural design of the foundation. The piles are designed to operate at a working load of which significant creep starts to occur, typically 70 to 80% of the ultimate load capacity. Sufficient piles have been included to reduce the contact pressure between the mat and soil. The pile locations have also been selected to reduce the differential settlement, rather than to substantially reduce the overall average total settlement.
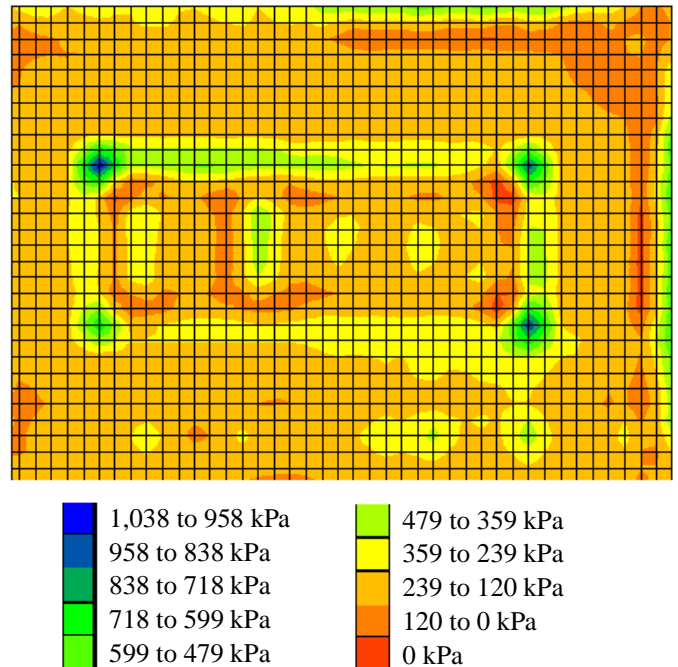

Legend:
- 1,038 to 958 kPa
- 958 to 838 kPa
- 838 to 718 kPa
- 718 to 599 kPa
- 599 to 479 kPa
- 479 to 359 kPa
- 359 to 239 kPa
- 239 to 120 kPa
- 120 to 0 kPa
- 0 kPa

Fig. 9. Mat Contact Pressure Contours – Mat Foundation with Settlement Reducing Piles

This philosophy of designing piles as settlement reducers has lead to fewer piles than in a conventional design, but which still satisfies the specified design criteria with respect to ultimate load capacity and settlement. If the load level on a pile is too low (<70%), the pile will not settle adequately and will become a hard spot under the mat. On the other hand, if the load level is too high, the pile may fail due to excessive settlement. Therefore, this design required several iterations to determine the optimal number of piles, pile spacing, and length. In addition, the load level on each pile, mat settlement, the number of piles, the length of piles, and the location of the piles are all interrelated (i.e., altering one component will affect all). Without numerical analysis, it is very difficult to perform this kind of design. In the analysis, full dead load, full live load, and 25% wind load was used. Based on numerous trials, it was recommended that 281 piles be used, where the piles extend to the lower sand, gravel, and clay with cobble layer (Layer 6). To assist the structural engineer in the

structural design of the mat foundation, spring constants of soil and piles were generated from the results of the numerical analysis.

FIELD TESTING AND MODEL REFINEMENT

Traditional ACIP pile load testing was performed prior to installation of production piles and to further refine the three-dimensional model. The load testing program consisted of tipping a pile within the lakebed soils for evaluation of shorter piles. The pile "plunged" before the design load was achieved. The model showed that the geotechnical capacity and corresponding settlement of the piles tipping in this zone exceeded our design assumptions. As a result, the predicted mat settlements exceeded the design requirements. Therefore, piles tipping into the weaker lakebed soil strata were not further evaluated.

In contrast, a load test was performed on an ACIP pile tipping into the underlying bedrock. As suspected, the load test results confirmed that the high capacity coupled with the approximate 2.5 cm (1 in.) settlement, when placed in the three-dimensional model will structurally "fail" due to the high building loads.
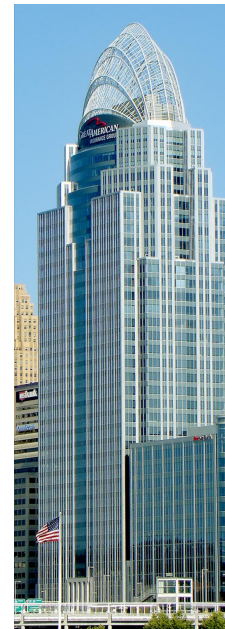
A load test was performed with a pile tipping within the stiffer transitional zone between the lakebed materials and bedrock. The 14 m (46 ft.) long pile deflected about 9.4 cm (3.7 in.) under a load of 127 metric tons (140 tons). The resulting load-deflection information from this load test was used to calibrate the soil-structure model, and resulting predicted settlements and contact pressures. The model was further modified with cone penetration (CPT) and pressuremeter testing (PMT), load test results on several ACIP pile elements of varying lengths, but all tipping above bedrock. Strain gage and settlement monitoring continues with initial results indicating relatively close to model predictions.

CLOSING

The Great American Tower at Queen City Square in Cincinnati, Ohio combined traditional geotechnical subsurface exploration methods, with advanced soil-structure interaction modeling software. Using state-of-the-art concepts and advanced modeling tools, which also resulted in significant construction cost savings, resulted in a geotechnical solution promoting the state-of-practice. Traditional standard penetration test boring data (SPT), along with cone penetration (CPT) and pressuremeter testing (PMT), and load test results on several ACIP pile elements of varying lengths were combined to develop the three-dimensional model.

The result was a total of 281 "settlement-reducing" ACIP piles strategically located below a mat foundation being recommended as the foundation system. Monitoring of the stress/strain development along several piles has been

reasonably consistent with model predictions. Combining traditional mat and ACIP pile foundations into a unique soil-structure interaction based foundation system, resulted in successfully advancing geotechnical analysis and design. The completed tower is shown in Fig. 10.

Fig. 10. Completed Great American Tower