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ABSTRACT

This paper presents a case history of Lean Concrete Column (LCC) design, prediction, installation, and monitoring for a 34-story high-rise condominium tower over a five-level underground parking substructure supported on a mat foundation in San Diego, California. The site constraints and the building configuration imposed unusual design and construction challenges, which resulted in high foundation pressures and eccentric loading for the planned building mat foundation. A geotechnical investigation consisting of deep test borings, Cone Penetrometer Tests (CPT), and laboratory tests indicated that formational soils underlying the site did not provide the necessary bearing capacity to directly support the structure on a conventional mat foundation system within acceptable settlement and structural limitations. Design constraints, economics, and constructability issues dictated solutions requiring an integral waterproofed substructural system supported on a mat foundation. A foundation system incorporating conventional piles structurally tied into the mat was not feasible because of waterproofing constraints. A determined practical solution was to incorporate a ground improvement technique that would be separate from the mat foundation, provide improved mat support, and reduce differential settlement to within tolerable limits. LCCs were found to be a viable method of ground improvement for reducing differential settlement of the mat to acceptable limits.

INTRODUCTION

The proposed condominium project is located in downtown San Diego, California, approximately 1,150 feet from the San Diego Bay waterfront within a highly urbanized area immediately adjacent to a multi-level parking structure and major commuter/freight railway corridor. The 34-story high-rise condominium tower over a five-level underground parking structure is designed to be supported on a waterproofed reinforced concrete substructure established on a mat foundation.

The mat foundation has a plan dimension of about 110 feet by 149 feet and occupies the full footprint area of the site. The bottom of mat extends to depths ranging from approximately 52 to 69 feet below grade and from 38 to 51 feet below the groundwater level (at approximately sea level) at the site. Because of required building setback from the adjacent public streets on the northerly and easterly perimeter of the site, it was necessary to place the tower structure against the westerly and southerly edges of the mat, resulting in an eccentrically loaded foundation. Initial estimated static foundation pressure imposed on the soils beneath the mat ranged from approximately 2,000 psf along the northerly and easterly perimeter to 11,500 psf at the southwest corner of the mat. The mat design consisted of an approximately 5- to 7-foot thick heavily reinforced structural section. However, even with a relatively stiff mat section, computed total settlements ranged from less than 1 inch to 5 inches across the mat, with maximum differential settlement (deflection ratio) up to 0.9 inch in 20 feet. A deflection ratio of 0.25 inch or less in 20 feet was required by the project structural engineer. Accordingly, as an alternative to a conventional pile foundation system, ground improvement was selected for reducing the compressibility and stiffening the supporting soils beneath the building mat foundation to reduce settlement to within tolerable limits. This paper describes the design, prediction, installation, and monitoring of the Lean Concrete Column (LCC) system to control settlement for an eccentrically-loaded major high-rise building structure.

SITE DESCRIPTION

The site is bounded by paved streets on the north and east, and by an existing multi-level parking structure immediately to the south (Fig. 1). The westerly edge of the site is bounded by a busy railroad corridor consisting of six sets of tracks for light rail trolley and heavy rail commuter and freight traffic. The excavation for the building foundation and substructure extended to depths ranging from about 56 to 71 feet below grade. The site excavation and adjacent areas were shored using conventional soldier piles and wood lagging restrained by five rows of tied-back earth anchors. Site dewatering was primarily accomplished by a system of deep wells.

GEOLOGIC SETTING

The site is underlain by two relevant geologic formations that were formed by accumulated sediments eroded from surrounding highlands in the late Tertiary and Pleistocene time. During late Pleistocene time (approximately 125,000 years before present) the Bay Point Formation, the unit underlying the site, was deposited on the San Diego Formation. The Bay Point Formation represents a brackish water estuarine and near shore terrestrial environment (Kennedy, 1975) in which a variety of sediments consisting primarily of clayey sands, fine to medium-grained well-sorted sand, and cobble conglomerates were deposited (Hart, 2005).

SUBSURFACE INVESTIGATION

The site lies in a highly urbanized area of downtown San Diego, California, with a natural harbor to the west and south. Because of the close proximity of site to San Diego Bay and localized fault zones, it was anticipated that the site would possess an elevated level of seismic and geologic hazard risk. Preliminary geotechnical studies suggested the presence of saturated cohesionless soils, a shallow groundwater table, fine-grained soils of low to moderate strength and compressibility, and moderately varying geologic strata, in addition to the potential seismic hazards. The goal of the site subsurface investigation was to characterize the physical engineering properties of the subsurface soils, address potential geologic hazards and their mitigation for foundation analysis and design, and for the construction of the building substructure system.

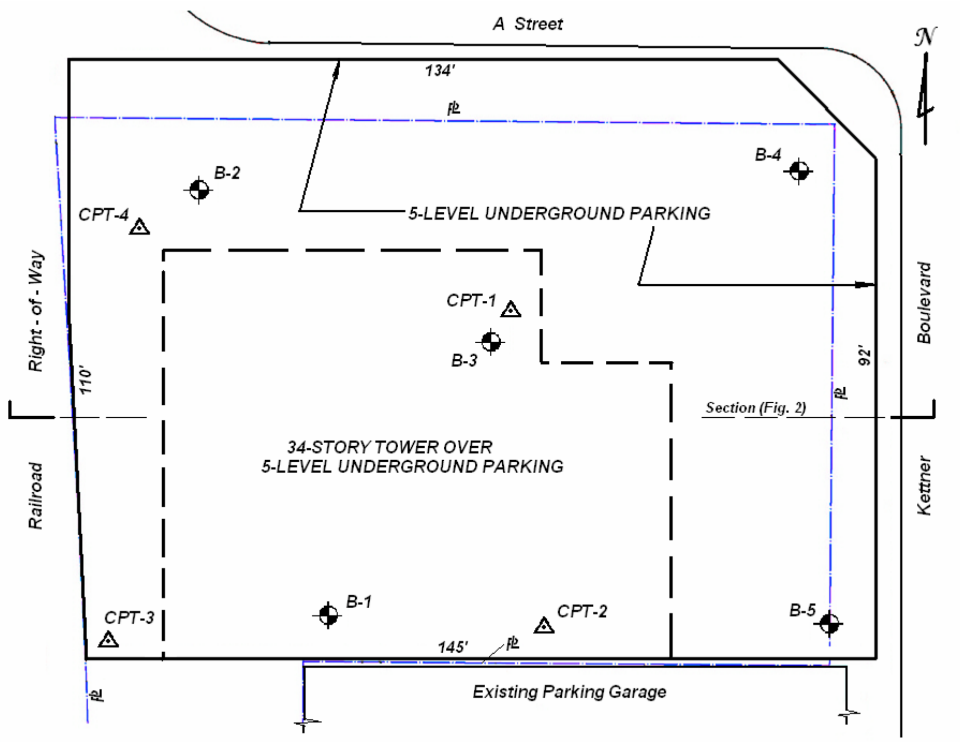

Fig. 1. Exploration site plan

The site is located in a special seismic study zone identified as the "Downtown Special Fault Zone." Specifically, the site is located in the eastern part of a broad structural trough, or basin, formed by downwarping and normal faulting along the Rose Canyon fault system (Hart, 2005). A north-south trending potentially active fault, believed to be a trace of the Rose Canyon fault, was identified approximately 200 feet west of the site. Additionally, the site is located approximately 1,400 feet northwest of an Alquist-Priolo Earthquake Fault Zone, as indicated on the City of San Diego Geologic hazards Maps.

A review of potential investigation techniques was performed to determine the most suitable method of investigation. The selected methods would have to allow for in-situ testing and sample recovery in an area underlain with moderately stratified layers, cohesionless soils prone to caving, and a relatively shallow groundwater table. The subsurface investigation techniques selected included a combination of conventional test borings (4½-inch to 5-inch diameter rotary wash) to obtain both in-situ test data and undisturbed samples for laboratory testing, and Cone Penetrometer Test (CPT) probings (Fig. 1). Six rotary-wash borings were advanced to depths ranging from 56.5 feet to 170.5 feet below existing ground surface. Standard Penetration Tests (ASTM D1586) were conducted using a 2.0-inch O.D. split-barrel sampler at prescribed depths throughout the depth of the borings. Relatively undisturbed soil samples were recovered using a 3.0-inch O.D. split barrel ring-lined sampler driven into the bottom of the borehole at 5-foot intervals, and at 10-foot intervals below 100 feet. Four Cone Penetrometer Test (CPT) probings (ASTM D5778) were advanced to depths of 101 to 128 feet, where refusal to probing was encountered in dense formational material.

sand layers were encountered below 74 to 96 feet, interbedded with stiff to hard silt and clay layers to about 105 feet bgs. Below 105 feet and extending to about 125 feet bgs was encountered hard sandy clay, underlain by a 10-foot layer of dense to very dense sand and silty sand. Below 135 feet, hard sandy clay was encountered extending to about 148 feet bgs. At 148 feet, the Bay Point formation gives way to the San

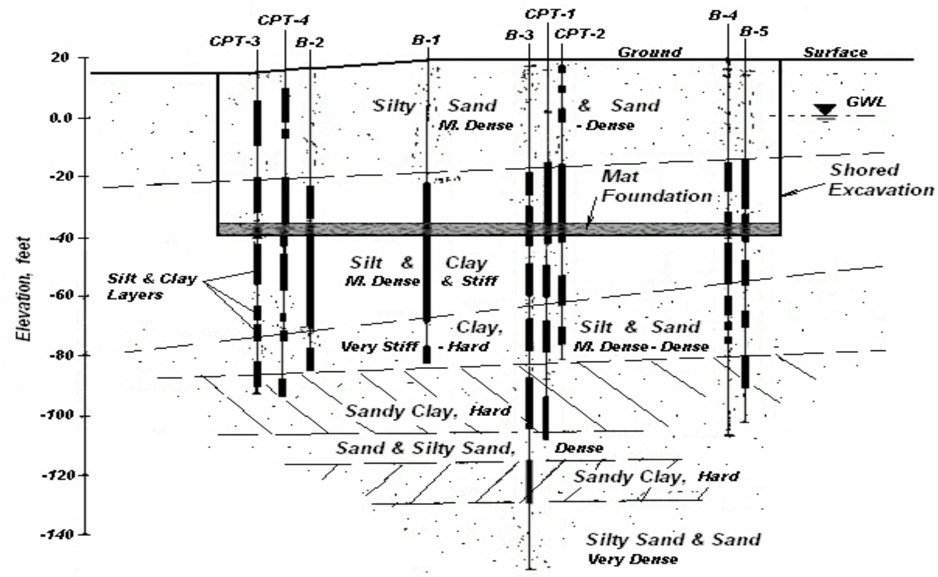

Fig. 2. Site soil profile

The laboratory testing program consisted of in-situ moisture content and dry density testing (ASTM D2216/ASTM D2937), undisturbed single-point direct shear testing (ASTM D3080), unconfined compression tests (ASTM D2166), one-dimensional consolidation testing (ASTM D2435), particle size analysis (ASTM D422), and Atterberg limits determinations (ASTM D4318).

A fault trench was excavated east-west across the site to determine the presence or absence of active or potentially active faulting within the site. The fault trench investigation concluded that there were no active or potentially active faults within the bounds of the site (Hart, 2005). Further discussion relating to seismic issues at the site is beyond the scope of this paper.

SUBSURFACE CONDITIONS

The site is generally overlain by minor localized fill ranging from 2- to 11-feet thick and averaging about 6 feet. The underlying native soils are identified as Bay Point formation, and generally consist of medium dense to dense silty sand, sand, sand with silt, and clayey sand extending to 32 to 43 feet below ground surface (bgs). Underlying this predominantly silty sand layer, and extending to a depth of about 74 to 96 feet bgs, is medium dense sandy silt and stiff clayey silt, with subordinate layers of very stiff clays. Medium dense to dense

Diego formation, which consists of very dense sand and silty sand. This unit extended to the maximum depth of exploration at 170.5 feet bgs.

Based on the exploratory borings conducted prior to construction, as well as measurements in nearby monitoring wells, groundwater was anticipated to be at approximately 18 to 21 feet below ground surface (at or about mean sea level).

Figure 2 shows a composite cross-section of the subsurface soil units.

SITE CHARACTERIZATION AND SOIL PROPERTIES

From a foundation engineering standpoint, the proposed mat foundation extends to depths ranging from 52 to 69 feet below grade and bottoming in generally medium dense sandy silt and stiff clayey silt, which extend to depths from about 74 to 96 feet below grade (Fig. 2). Based on the field and laboratory testing, the silt and clay deposits immediately beneath the proposed mat are of moderate strength and compressibility, with SPT N-values ranging from 11 to 28 and CPT tip stress ranging from 20 to 50 tsf. The soils typically have Liquid Limits between 30 and 42, and Plasticity Indices of between 12 and 23. The silt and clay varied in depth and thickness across the site, generally dipping downward and thickening to the west and southwest. The silt and clay deposits overlie interbedded layers of clay, silt and sand, which were considered low to moderate compressibility and good strength.

SETTLEMENT ANALYSIS

Settlement analyses were performed using one-dimensional laboratory consolidation tests and field SPT and Cone Penetrometer Test data to evaluate settlement of the mat. Consolidation tests were performed on samples of the silt and clay deposits from depths ranging from 50 feet to 141 feet below grade. The silt deposits are generally medium dense to dense, and the clays generally stiff to very stiff and hard. The soils below the planned foundation level are overconsolidated with estimated preconsolidation pressures ranging from 15,000 psf to over 25,000 psf, and over-consolidation ratios (OCR) estimated between 2 and 5, based on the consolidation tests, unconfined compressive strength and plasticity index of the soils. Consolidation tests were performed on the soils to maximum pressures of 16,000 psf and 31,000 psf, which encompassed the range of loading imposed on the soils from the soil overburden and building foundation. Recompression indices, C_r, on the silt and clay deposits typically ranged from 0.0140 to 0.0179. Settlement caused by compression of the interbedded sand layers was determined following the procedure by Schmertmann (1970) using SPT and Cone Penetrometer Test data.

The settlement of the eccentrically loaded mat was determined based on the subgrade reaction (contact pressure) as provided by the project structural engineer for the mat supported on a subgrade with a subgrade modulus of 15 pci. The foundation contact pressure or subgrade reaction beneath the mat is shown on Fig. 3. The effect of the overburden soil at the perimeter of the excavation for the mat was included in the settlement computed for the sides and corners of the mat. The settlement analyses indicate maximum total settlement of approximately 5 inches occurs within the central portion of the mat. At the southwest corner of the mat where foundation pressure up to 11,500 psf is expected, a total settlement of about 2 inches is computed. Total settlement of about 1 inch was computed at the northwest corner and about 1.4 inches at the north side of the mat where foundation pressure on the order of approximately 2,000 psf is expected. Deflection ratios up to 0.9 inch in 20 feet were computed across the mat. As a result, additional engineering studies were performed to determine suitable design measures to reduce mat foundation settlement to tolerable limits.

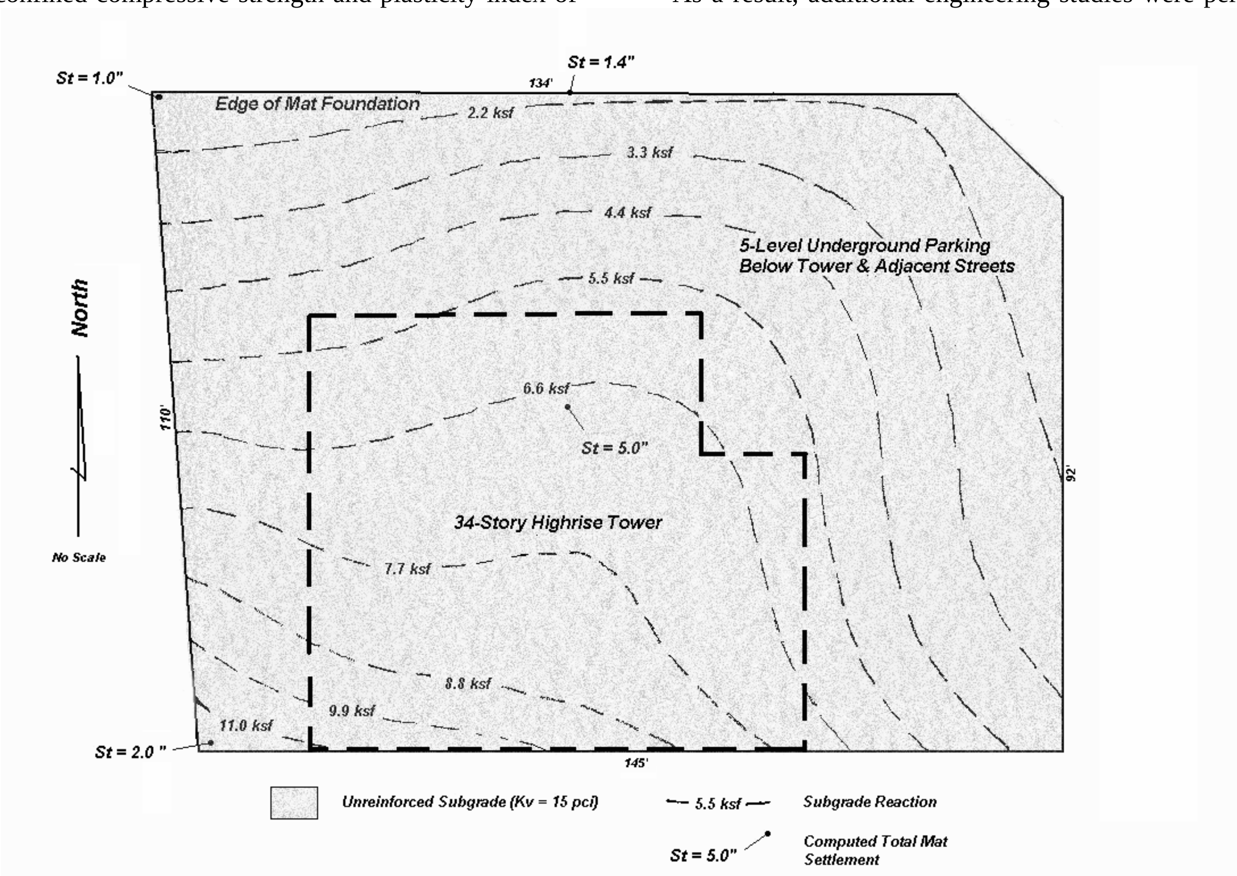

Fig. 3. Mat settlement and subgrade reaction over unimproved ground

FOUNDATION DESIGN/GROUND IMPROVEMENT ALTERNATIVES

Engineering evaluation of several foundation design and ground improvement alternatives was performed to determine their feasibility to reduce differential settlement to within tolerable limits. These alternatives and their practical application for the project are summarized in Table 1. Based on reviews by the Owner and the design and construction team, Lean Concrete Columns were selected to provide the most cost-effective solution for controlling differential settlement predicted for the project.

the waterproofing system at the junctions around the top of piles and building substructure. Driven piles were also not acceptable because the vibrations generated from pile driving may adversely affect the proposed shoring and adjacent existing structures. The presence of the substantially stiff and cohesive silt and clay deposits ruled out the use of vibro-compaction, stone columns, lime columns, and soil cement columns, which cannot be effectively and efficiently accomplished in such soils. Because of the relatively low elastic modulus and strength of stone columns, lime columns, and soil cement columns, their effectiveness for controlling foundation settlement caused by high foundation stresses would be limited.

Table 1 - Foundation Design and Ground Improvement Alternatives

Foundation Design/ Ground Improvement Alternative	Method of Improvement	Practicality of Application at Site	Reference
Driven Piles	Support mat on conventional driven friction piles	Costly; pile reinforcements interfere with foundation waterproofing; vibrations from pile driving unacceptable	
Auger-Cast Piles	Support mat on auger-cast friction piles	Costly; pile reinforcements interfere with foundation waterproofing	
Vibro-Compaction	Soil densification	Not suitable for stiff, fine-grained silt and clay	
Stone Columns	Vibro-replacement	Not suitable for stiff, fine-grained silt and clay; rigidity (stiffness) of stone columns inadequate for heavy foundation loads	Bergado, D.T., et al. [1996]
Jet Grouting	Replacement of soil with cement grout	Costly for broad, large scale application	Schaefer, V.R. [1997]
Compensation Grouting	Ground/foundation displacement with pressurized cement grout to offset observed foundation settlement	Costly; grout pipes would interfere with foundation waterproofing; close vigilant monitoring of foundation settlement during construction required	
Lime Columns	Deep soil-mixing with lime to reinforce/stiffen soil	Not efficient and effective for stiff, fine-grained silt and clay; rigidity (stiffness) of lime column inadequate for heavy foundation loads	Bergado, D.T., et al. [1996]; Schaefer, V.R. [1997]
Soil Cement Columns	Deep soil-mixing with Portland cement to reinforce/stiffen soil	Not efficient and effective for stiff, fine-grained silt and clay; rigidity (stiffness) of soil cement column inadequate for heavy foundation loads	Bergado, D.T., et al. [1996]; Schaefer, V.R. [1997]
Lean Concrete Columns (LCC)	Concrete placed under pressure in drilled hole using hollow-stem auger to reinforce/stiffen soil	Can be effectively and efficiently installed in dense, stiff cohesive soils; rigidity of LCCs suitable for heavy foundation loads	Bergado, D.T., et al. [1996]

In general, constructability, rather than cost, ruled out most of the foundation design and ground improvement alternatives. The requirement for an effective, integral waterproof system for the foundation and around the building substructure under high groundwater seepage pressures essentially eliminated conventional pile foundations as an alternative for mat support, since installation of waterproofing around the piles was considered to be difficult and could compromise

It was the general conclusion of the construction team that jet grouting and deep soil mixing for lime columns and soil cement columns would also be cost prohibitive due to plant set-up requirements. Compensation grouting was considered feasible; however, it was desired that adequate foundation support be provided at the start of construction rather than allowing settlement to occur and addressing such settlement as it occurs during construction. Therefore, except for Lean Concrete Columns, the other alternatives were regarded by the project design and construction teams as impractical or otherwise not feasible.

For the lean concrete columns, it is believed that settlement may be reduced to within tolerable limits by structurally stiffening the compressible silt/clay deposits below the proposed mat foundation by installing a group of large diameter, vertical unreinforced concrete columns in the soil. The LCCs spaced in a grid pattern would form a composite block of soil and concrete columns that has the combined properties (shear strength and compressibility) of the concrete columns and surrounding soil within the block. The quality of the reinforced block may be easily controlled by controlling the quality of the concrete mix to be used in the columns. No structural connection would be required between the lean concrete columns and mat foundation, and a buffer consisting of a cushioning layer of compacted well-graded crushed rock is to be provided for uniform support of the mat over the lean concrete columns. Optimum column spacing and depth were to be determined to accommodate the mat design, based on a targeted reduced total settlement of 2 inches or less and a differential settlement (deflection ratio) of 0.25 inch or less in 20 feet as required by the structural engineer. On the basis of design, constructability and economic feasibility, the use of lean concrete columns was found to be the most suitable method for ground improvement for the support of the proposed building mat foundation imposing high stresses on the soils. The design and installation of the LCCs are described in the following sections of this paper.

LEAN CONCRETE COLUMNS

Design

The design of the lean concrete columns for ground improvement support beneath the planned mat foundation involved determining the optimum column spacing and depth for controlling mat settlement to within the tolerable limit required by the structural engineer. The LCC ground improvement design was governed by the subgrade stiffness required of the reinforced subgrade beneath the mat, the redistributed mat foundation (subgrade reaction) pressures, the mat/subgrade deformation, and settlement of the LCC group. The LCCs were to be designed and installed at and extending below the bottom of the foundation excavation by drilling the column shafts and placing the concrete within the drilled shafts below groundwater. Because of the presence of sand layers and groundwater under seepage pressure from dewatering, hollow-stem auger equipment was specified for drilling the columns and placing the concrete for the LCCs to minimize any caving and assure continuity in the column shafts. Using the largest diameter hollow-stem auger and equipment available in the San Diego area, and from the standpoint of construction and cost efficiency, a column shaft diameter of 30 inches was selected.

The design of the LCC ground improvement was performed in collaboration with the Project's geotechnical engineer and structural engineer. The reduction of the mat deflection ratio to within the desired limits was achieved by increasing the subgrade soil stiffness and redistributing the soil/foundation contact pressures (or subgrade reaction) beneath the mat. The analyses for redistributing the mat foundation pressures and determining the resulting deformation of the mat were accomplished using the finite element based computer software program, SAFE (Version 8.0.6), for which the structural interaction of the mat and superstructure may be accounted for in the analyses. The soil stiffness (soil spring constants), upon which the foundation contact pressures and mat deflection are determined is computed by the program as a function of the specified subgrade modulus and tributary mat area. The modulus of subgrade reaction, K_v , specified in the design analyses is expressed as:

$$K_{\boldsymbol{v}} = p \,/\, \delta$$

Where: K_v = modulus of subgrade reaction for mat foundation of given width (F/L^3)

p = foundation contact pressure or subgrade reaction (F/L^2) and

δ = soil deformation or settlement of mat (L)

The redistribution of the foundation contact pressure beneath the mat was achieved by following an iterative trial process using select subgrade modulus values, K_v , for select areas under the mat (Fig. 4). Subsequent modifications of the subgrade values and mat structural stiffness, as appropriate, were made until the resulting mat deflection ratios across the mat were satisfactory (Fig. 5). Once the mat deflection ratios were satisfactory, the corresponding foundation contact pressures, p, were used in the LCC ground improvement design to determine the preliminary column lengths and to check foundation settlement of the mat over the treated ground. If the computed foundation settlement remains excessive, the design process is repeated with further modification of the subgrade modulus values and new determination of the redistributed foundation pressures, p, and mat structural deformations, δ. The design process is completed when the computed foundation settlement of the mat supported on the improved subgrade is approximately equal to the mat structural deformation, δ, or to the ratio of the foundation contact pressure, p, to the corresponding subgrade modulus, $K_{\boldsymbol{v}}$, for select areas beneath the mat, and that the deflection ratios across the mat are satisfactory. The resulting

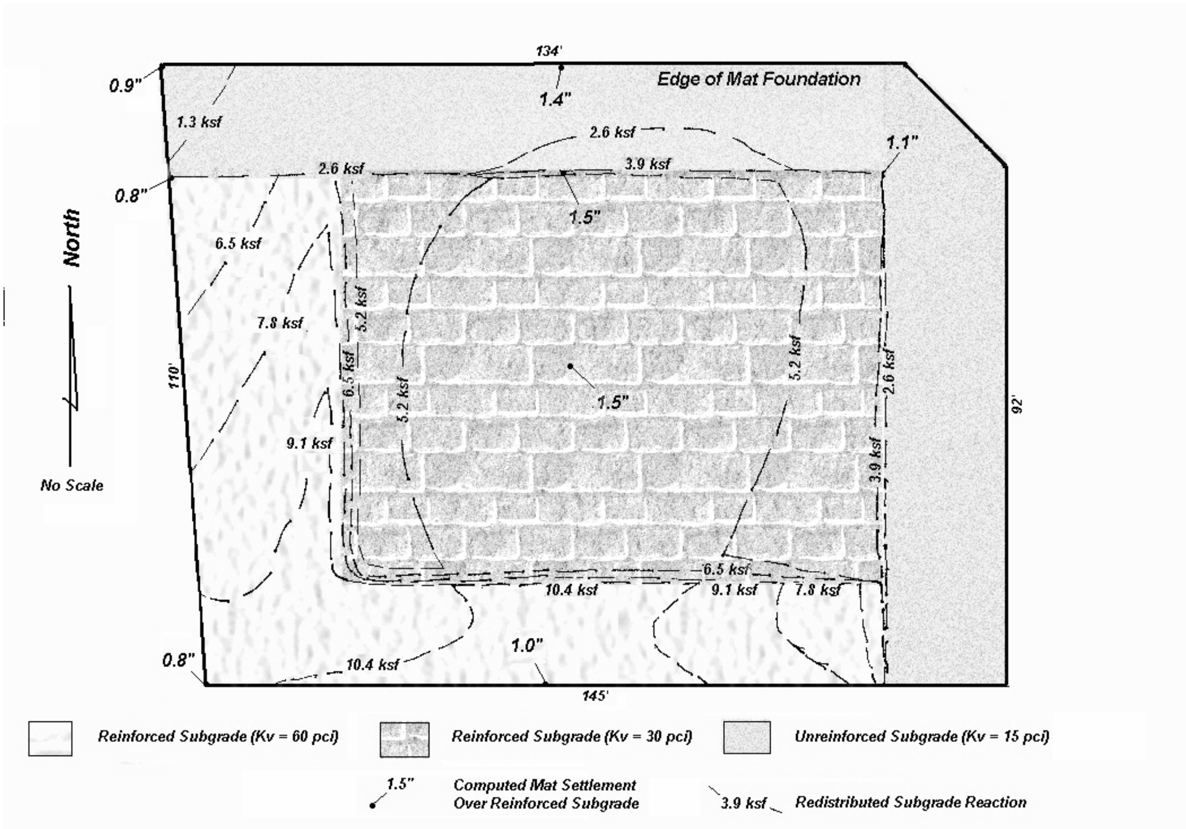

Fig. 4. Modified subgrade modulus values and redistributed subgrade reaction beneath mat used in LCC design

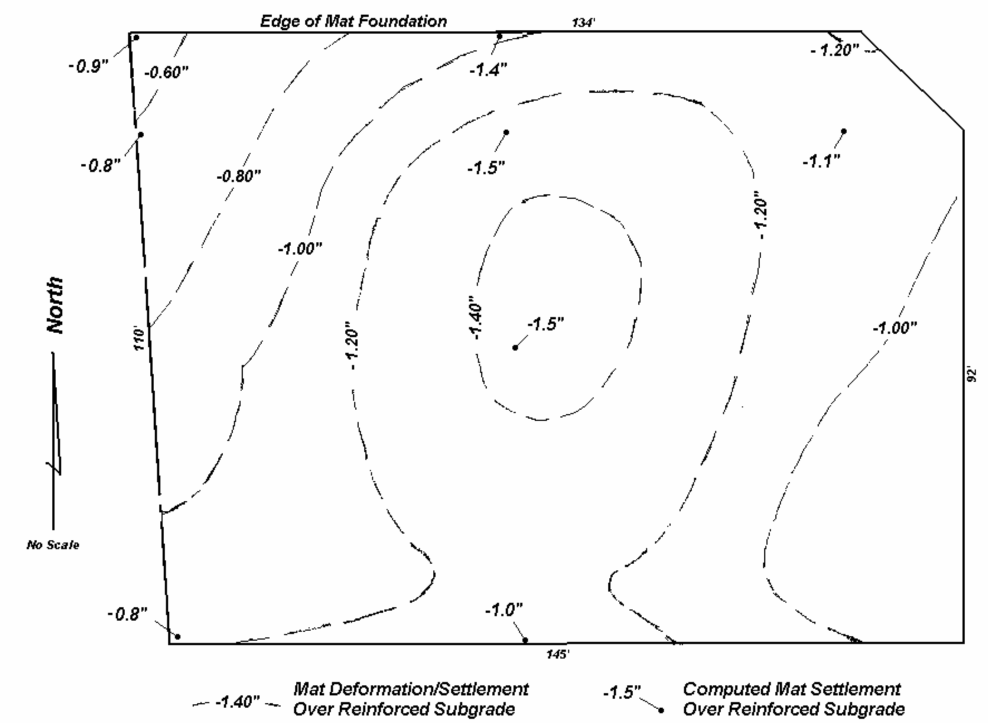

Fig. 5. Computed mat deformation and settlement for mat supported on LCC reinforced subgrade

redistributed subgrade reaction or foundation contact pressures are then used for the LCC design.

The final LCC ground improvement design for the project was based on subgrade modulus values of 15 psi/inch (pci) computed for the unreinforced subgrade areas, and 30 pci and 60 pci for the reinforced areas, which generally occurred beneath the highly stressed footprint of the tower. The soil pressure distribution beneath the proposed mat foundation, as influenced by the modified subgrade modulus values and treated soil under the mat, is shown on Fig. 4. Analyses were performed using one-dimensional laboratory consolidation tests and field SPT and Cone Penetrometer Test data to evaluate settlement of the mat supported over the LCC treated ground. Settlement of the reinforced ground (or block) beneath the mat was taken as the sum of the settlement (compressive strain) of the reinforced block and the settlement of the untreated soils below the block. The total settlement of the mat at specific locations is estimated to range from approximately 0.8 inch to 1.5 inches as shown on Figs. 4 and 5. The lengths of the LCCs and depth of the reinforced block were increased as necessary to reduce the total settlement of the mat/reinforced block. The resulting computed mat deformation and settlement for the mat supported on the proposed LCC ground improvement is shown on Fig. 5. A factor-of-safety greater than 1.0 is provided against creep, which may result from mobilization of the shearing strength of the soil at the soil-column interface.

The LCC ground improvement design consists of 130 LCCs within an approximate 90- by 120-foot area (approximately 60% of total mat area) adjacent to the southwest sector of the proposed mat as shown on Fig. 6. Each LCC consists of 30-inch diameter auger-cast, unreinforced vertical elements spaced 4 diameters or less on centers in a rectangular grid pattern of approximately 9.5 feet by 10 feet. A structural base section consisting of crushed aggregate base material of approximately 2-feet thick and compacted to at least 95 percent of the maximum dry density (as determined by ASTM Test Method D1557) was placed between the bottom of mat and top of the lean concrete columns and subgrade soil to provide load distribution from the mat to the LCCs and

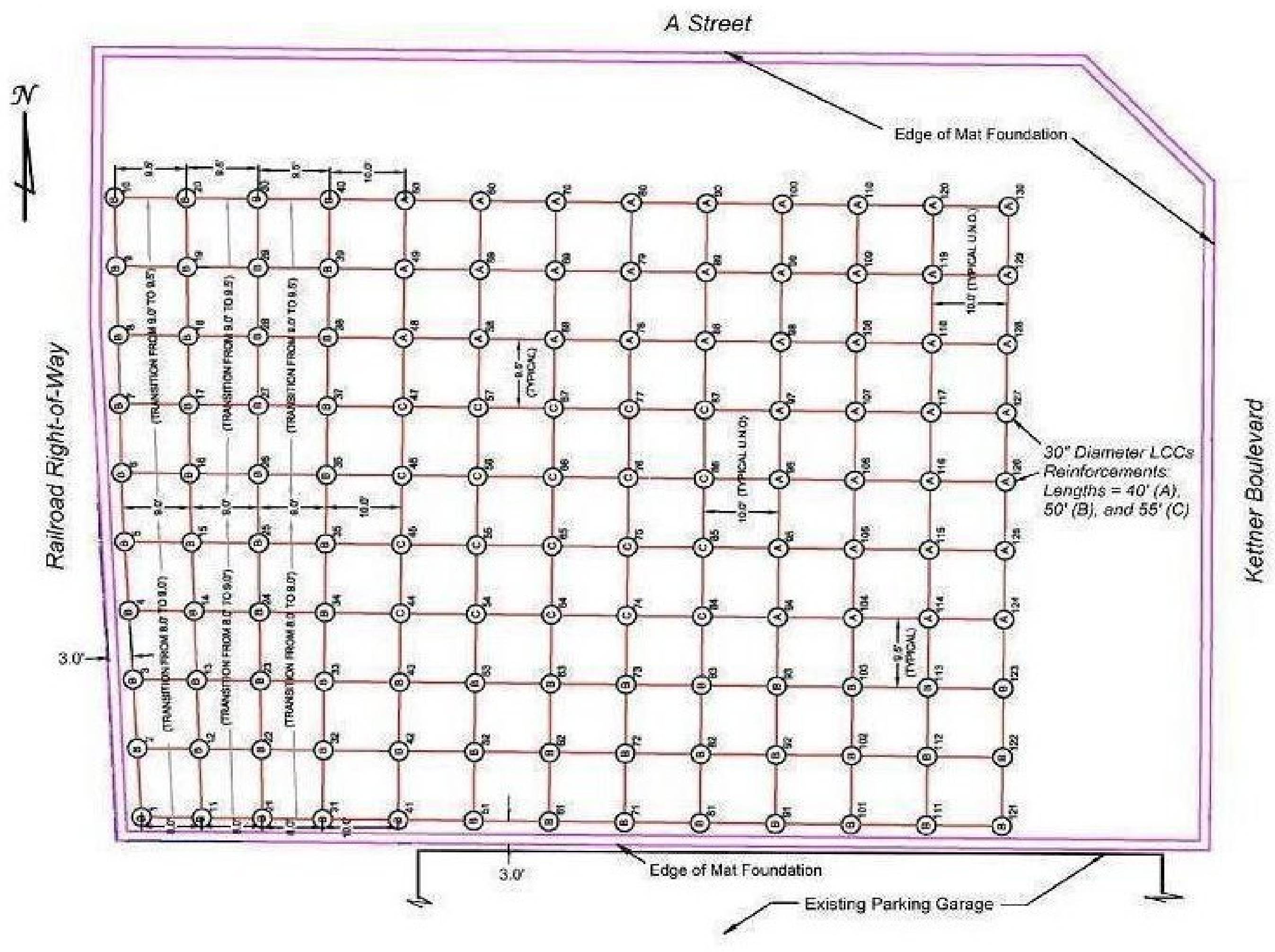

Fig. 6. LCC design layout

subgrade, and to provide proper mat support without extreme stress concentration over the LCCs. For the planned LCC spacing and redistributed foundation pressures of up to 10,500 psf, estimated maximum compressive stress within the LCCs are on the order of 1,400 psi or maximum compressive load of approximately 990 kips per LCC. The stress imposed by the mat on the subgrade soil surrounding the LCCs is estimated to range from less than 2,000 psf to 4,500 psf. Nominal LCC lengths of 40, 50 and 55 feet below the mat subgrade level and crushed rock base section were planned. A suitable concrete mixture with a compressive strength of at least 2,000 psi was specified for the LCCs. Figure 6 shows the design layout and dimensions, as well as the lengths of the LCCs by the designations of "A" (40 ft.), "B" (50 ft.) or "C" (55 ft.).

Installation

The LCCs were installed below the bottom of the foundation excavation to the specified minimum tip elevations using a 30-inch diameter hollow-stem auger. Although the original intention of the design-build team was to install the LCCs near the bottom of the planned excavation (at about 60 feet below the original ground surface), the contractor opted to excavate to within about 3 to 5 feet above the planned excavation level to provide a suitable working platform and minimize disturbing the foundation bearing subgrade during the LCC installation. However, due to delays in site dewatering, the design-build team ultimately elected to install the LCCs much earlier at higher grade than anticipated, at about 13 to 16 feet above the planned excavation level.

After drilling to the specified depth, concrete for the LCC was placed within the drilled shaft as the auger was gradually withdrawn while the concrete was simultaneously pumped through the auger shaft. During concrete placement, the auger was withdrawn at a constant rate while the concrete was pumped under a positive pressure to ensure continuity throughout the length of the LCC shaft. Installation of each LCC was to be performed continuously without interruption. Because the working platform was significantly higher than the planned excavation bottom, the upper 8 to 10 feet of each shaft did not receive pumped concrete. Instead, the auger was unscrewed, leaving about 8 to 10 feet of disturbed, relatively loose soil at the top of each LCC shaft.

Upon completion of the LCC installation, the design-build construction team completed the foundation excavation to the planned subgrade level. At that point, the tops of LCCs extended about 3 to 6 feet above the subgrade level, and were cut off to the design subgrade level by saw-cutting around the shaft perimeter and breaking the excess top portion off using the excavating equipment. The tops of LCCs before and after cutoff are shown on Fig. 7. The exposed LCCs and the reinforced foundation subgrade are shown on Fig. 8.

A two-foot thick structural base section consisting of crushed aggregated base was placed over the completed LCCs and prepared subgrade. The structural base section was constructed by compacting 6- to 8-inch thick lifts of materials, at or near optimum moisture content, to 95 percent of the maximum dry density as determined by ASTM Test Method D1557.

Fig. 7. Tops of LCCs before and after trimming

Fig. 8. Trimmed LCCs and reinforced foundation subgrade

As stated previously, the aggregate base layer served to provide load distribution from the mat to the LCCs and subgrade, and to provide proper mat support without concentrated stress over the LCCs. The base layer also provided protection for the prepared subgrade and a suitable working platform for the installation of waterproofing and for the mat construction.

A comprehensive inspection and testing program was implemented during the installation of the LCCs. Continuous observation was performed during the LCC installation to confirm the soils as encountered during drilling, verify installed column lengths to the design plans, note any deviations from the specified installation procedures, and to ensure material quality and workmanship in accordance with the approved plans. Most importantly, the volume of concrete placed for each LCC was checked against the theoretical volume to assure proper shaft depth and continuity.

Cylinders of the concrete were taken during the duration of the LCC installation for compressive strength tests to assure that the concrete meet the minimum specified 28-day strength of 2,000 psi.

CONCLUSION

The 34-story high-rise condominium tower over a five-level underground parking structure is supported on a mat foundation below the parking structure at over 50 feet below grade and over 35 feet below the groundwater level. Because of limiting site constraints and eccentrically loaded foundation, static foundation pressure imposed on the soils beneath the mat ranged from 2,000 psf to 11,500 psf. Coupled with a variable subsurface soil condition with high rebound and recompression characteristics in the silt/clay deposits underlying the site, mat foundation settlement is computed to range from less than 1 inch to 5 inches across the mat, with excessive differential settlement of up to nearly an inch in 20 feet.

As an alternative to a conventional pile foundation system, ground improvement consisting of Lean Concrete Columns was selected for reducing the compressibility and stiffening of the supporting soils beneath the building mat foundation to reduce settlement to within tolerable limits. Lean Concrete Column construction consisted of the installation of a total of 130, 30-inch diameter columns with lengths ranging between 40 and 55 feet below the mat subgrade elevation. The LCCs were efficiently installed using hollow-stem auger equipment with concrete pumped into the shaft under positive pressure. Lean Concrete Column construction started and finished in 2006. The mat foundation was placed in March 2007, and construction of the tower is expected to be completed in early 2009.

Survey points were installed at multiple locations on the mat for settlement monitoring. Bi-weekly to monthly readings are being taken during construction. At the writing of this paper (December 2007), the settlement data is insufficient to draw conclusions regarding mat performance, settlement and deflection. It is the authors' intention to present a follow-up paper regarding performance of the LCC system during and after construction.

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