A Case Study for Up-Down Design and Construction Methodology for a High-Rise Development in Los Angeles, California

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Abstract

Over the past few years, numerous high-rise projects have been initiated in downtown Los Angeles. In most cases, at least one and often two or more subterranean levels are constructed. Conventional construction sequencing would consist of shoring installation, mass excavation, and construction of the below-grade structure all prior to construction of the building superstructure.

For a high-rise project currently under construction in downtown Los Angeles, an alternative sequence, referred to as 'up-down' (or 'top-down') was used to reduce the duration of the overall construction schedule. Up-down sequencing typically allows the construction of the building superstructure in tandem with mass excavation and construction of the below-grade levels. The subject project includes a 23-story tower over three subterranean levels and has a building footprint of approximately 60,000 square feet. A modified up-down sequence was implemented in which the installation of basement walls and below-grade foundations and the initiation of superstructure preceded mass excavation and basement floor construction. The project utilized steel sheet pile as both the temporary shoring and permanent basement wall construction – a first in California.

The benefits of up-down construction, the sequence employed by this project, and the innovative structural and foundation design and construction methods devised to facilitate its adoption, are discussed herein.

Introduction

The subject project, coined Evo South by the project developer, includes a 23-story residential condominium tower over three subterranean levels, with a building footprint of approximately 60,000 square feet. It is anticipated for completion in 2008. The building footprint occupies the entire site and is rectangular in shape. It is bounded on the south and east sides by 12th Street and Grand Avenue, respectively; by an alleyway on the west side; and by a recently completed 11-story residential tower with one subterranean level, immediately to the north. See Figure 1 below.

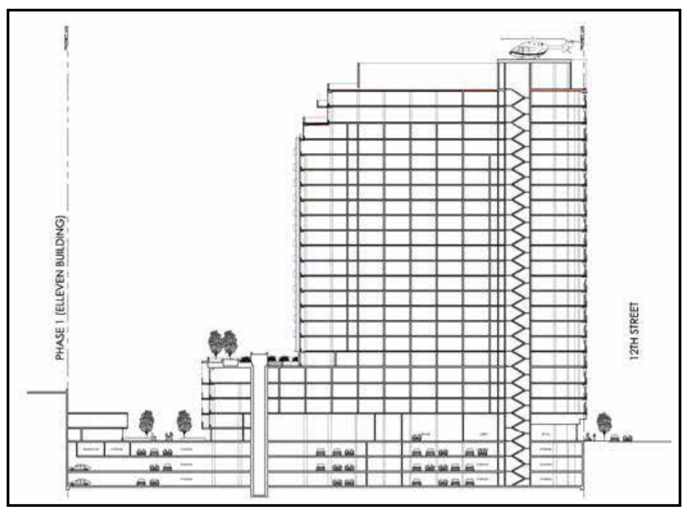

Figure 1: Section thru building looking east

The tower was designed in accordance with the 2002 Los Angeles City Building Code, essentially equivalent to the 2001 California Building Code and the 1997 Uniform Building Code, using the seismic design parameters presented therein. A site-specific response spectra was developed for application and use in determining design ground motions in accordance with 2002 LABC provisions.

Lateral forces due to wind or seismic events are resisted by three shear wall cores that extend from the P3 level around the stairs and elevators. Two of these core walls continue up to the tower roof (within the up-down zone of construction), while a third northernmost core continues up to the underside of the 6th floor terrace. The shear walls comply with the code prescriptive requirements of a Building Frame Shear Wall system (R=5.5) and incorporate diagonally-reinforced coupling beams at stacked door openings where geometry and calculated shear demands warranted their use (CBC, 2002).

To design the lateral force resisting system, a linear response spectrum analysis was performed using a three-dimensional ETABS computer model. The effective properties of the shear wall coupling beams were set to those suggested by Paulay and Priestley (1992). Using a simple non-linear push analysis, as justified by Hindi and Hassan (2004), these properties were subsequently verified and plastic rotation demands at ΔM were compared to the allowable limit of 0.03 radians as suggested by FEMA 356 for Life Safety performance.

Maximum dead-plus-live loading for interior columns was 2,100 kips. Maximum short-term seismic loading on core drilled shafts was 3,350 kips. The average applied foundation pressure over the entire footprint of the tower was 12,000 psf, with peak pressures beneath the cores of 20,000 psf.

The excavation extended two levels below the adjacent mid-rise tower foundation system and a utility vault, requiring support of that existing construction. Low-rise buildings west of the site also imposed surcharge loading on the temporary shoring and permanent walls below grade.

At the direction of the Los Angeles Department of Building and Safety, the structural design was peer reviewed.

Up-down Construction

The up-down construction method involves the construction of above-grade levels prior to (or in tandem with) the construction of below-grade levels within the same footprint. It can be used to mitigate some of the issues associated with temporary shoring of below-grade construction in congested urban environments, as well as to shorten the construction schedule. In this project, an up-down methodology was chosen for the southern half of the project site beneath the tower footprint. This shortened the overall construction schedule, thereby allowing reduced carrying costs on the construction loan and earlier tenant occupation. The northern half of the site—encompassing half of the below-grade parking garage, the ground level plaza, and the 6th floor terrace—was completed using conventional methods following prior excavation down to the P3 basement level.

Sheet piles extending 15 feet below the underside of the eventual P3 basement level was first placed on the east, west and south sides of the site by utilizing a push press driver with an integrated auger bit to help clear cobbles. This sheet piling, with an 18-inch deep profile and nearly ¾-inch flange thickness, would eventually become the finished basement walls.

Figure 2: Installation of sheet piles

The north end of the site is bordered by a large underground Los Angeles Department of Water and Power (DWP) vault and a parking garage for an adjacent property, which extends a single level below grade. With the sheet pile walls now in place, excavation was performed to bring the grade of the entire site (up-down and conventional zones) down one story to the underside of the P1 basement level (see Figure 3). Temporary shoring was not needed for this condition since the sheet pile was capable of cantilevering the single story. Within the conventionally constructed north zone, tiebacks were installed through the sheet pile in order to allow further excavation in a subsequent stage. A bermed ramp on the northern side of the site allowed access into and out of the dig.

Within the up-down zone, drilled shafts were then placed as the foundation elements for the tower columns and shear wall cores. Following drilling and temporary casing, rebar cages were lowered into the shafts and concrete tremied up to an elevation consistent with the underside of the P3 basement level. Two-story-tall precast concrete columns extending from the P3 to P1 levels were lowered into the shafts and held in place. Pea gravel was placed in the interstitial space between the precast column and sides of the hole prior to removing the temporary casing.

Figure 3: Excavation to P1 level

Within the up-down zone of the site, a temporary low-strength concrete waste slab was then poured on grade to allow a smooth forming and working surface, followed by the placement of the P1 basement slab. With the sheet pile walls, drilled shafts, columns and P1 slab now in place, tower construction could proceed upward prior to further excavation downward. Columns, shear wall cores, and slabs up through the 4th floor were placed in conventional fashion, with the P1 slab used as the base of re-shoring. See Figure 4.

Figure 4: P1 slab in place, construction proceeds upward in up-down zone

Excavation beneath the P1 slab then proceeded, with particular care taken when the underside of the P1 slab (which was somewhat protected by the waste slab) and the pea gravel surrounding the precast columns were exposed to view. This "mining operation" continued to the lowest excavation limits below the eventual P3 basement level slab. Since the P1 slab now effectively braced the sheet pile basement walls, and these walls are capable of spanning between the bottom of excavation and the P1 level, there was no need for tiebacks within the up-down zone. Placement of the remainder of the below-grade structure within the up-down zone – including the grade beams, P3 slab, and P2 slab – followed thereafter, all while tower construction continued upward. See Figure 4.

Precast concrete was chosen for the two-story gravity columns extending from the P3 to P1 basement levels within the up-down zone due to its compatibility with the remainder of the concrete construction. Vertical reinforcing bars were extended below the bottom ends of the columns to provide a compression lap splice into the drilled shaft below, and extended above the top end of the columns to be lap-spliced or mechanically coupled with the reinforcing bar for the cast-in-place columns above.

Figure 5: Mining excavation beneath P1 slab

To facilitate placement of the precast columns within the drilled shaft holes, special guide templates were designed and fabricated by Malcolm Drilling for each combination of column size and drilled shaft diameter (see Figure 6). These templates consisted of a slightly smaller diameter steel casing than that of the drilled shaft and incorporated adjustable jacks that reacted against the inner surface of the temporary drilled shaft casing to provide any adjustments needed to achieve correct final column location.

With the aid of this custom-guiding mechanism, the precast columns were lowered into the cased holes and the bottom rebar extensions wet-set into the drilled shaft concrete below (see Figure 7). A retarder was used to prevent the concrete within the drilled shaft to set up prior to column placement. The columns were then locked off at the proper location and elevation until the concrete within the drilled shaft set up sufficiently to support the weight.

Figure 6: Column installation template

Figure 7: Lowering precast gravity column into place

Given the inherent uncertainties associated with completing a blind wet-set rebar connection 25-feet down a hole, high-strength non-shrink grout was subsequently pumped thru a 2-inch-diameter port at the center of each precast column into the column-to-shaft joint to ensure complete and competent contact bearing between the underside of column and top of drilled shaft below.

Threaded dowel inserts and ¾-inch depressed keys were strategically cast within the precast columns at the P2 and P3 slab elevations for eventual connection with grade beams, the elevated P2 slab, and elevated beams.

Construction of the shear walls below the P1 basement level began in much the same fashion as the columns, with L-shaped corners of the shear wall cores plant-cast and set atop large diameter drilled shafts. These corner elements were cast with threaded dowel inserts and horizontal keys along the entire length of those vertical faces in eventual contact with the remainder of the walls (see Figure 8). Instead of rebar dowels at their base, an embedded W12 steel wide flange section with headed shear studs was cast within and extended below each of the L-shaped columns. This W12 shape is wet-set into the drilled shaft below such that it will then be capable of transferring uplift forces due to a seismic event from the shear wall into the drilled shaft. The remaining wall panels between the precast corners were reinforced and cast in place after the completion of excavation within the up-down zone and during subsequent basement construction.

Figure 8: Precast shear wall core corner prior to installation

The Sheet Pile Innovation

The use of steel sheet pile as a permanent basement wall, while successfully employed in Europe and in a few locations in the United States, was a new and unproven technology to the City of Los Angeles. It was chosen on this project mainly for its speed of installation and its advantages in facilitating up-down construction. Since the same standard sections of sheet pile used more typically for marine and other foundation applications were employed here, it also was readily available. Furthermore, sheet pile installation could be made with the same tolerances as conventional concrete construction.

Sheet pile is traditionally driven in sections with vibratory or impact hammers, with sections joined at vertical interlocks. Due to negative implications of loud hammering or vibration on neighboring buildings in the Los Angeles southern downtown urban environment, as well as the presence of large cobbles at the site, a "silent" press piler was employed to hydraulically jack or press the sheets of pile into place. The press piler was fitted with a pre-auger bit that mitigated the effects of the large cobbles on the installation of sheet pile. The press piler was laser guided, used previously placed sections as reaction piles, and was self-advancing. The noise level during pile installation was roughly the equivalent of that made by a semi-truck and vibrations were virtually non-existent. Following excavation, the interlock seams joining individual sections of sheet pile were welded with non-structural seal welds, which were then subjected to periodic visual inspection and a magnetic particle testing program. These welds completed a basement wall construction type that is an impervious water and methane barrier. See Figure 9.

Figure 9: Magnetic particle testing following welding of sheet pile

Advancement of the sheet piles through the upper gravel and cobbles in the upper 20 to 30 feet of the subsurface soil was facilitated by the pre-drilling at the leading edge of each half sheet. This pre-drilling loosened the soils and effectively reduced the lateral confining stresses. A total of approximately 400 sheet pile half-sheet sections were installed to a depth of roughly 46 feet, which provided approximately 800 lineal feet of temporary shoring support for the subsequent excavation, as well as for the permanent building walls below grade.

Where sheet pile walls terminate, a direct positive connection along the vertical joint between the sheet pile and adjacent concrete wall construction was required. At the northwest corner of the site, a continuous steel angle was used to connect the sheet pile wall with the existing concrete DWP vault wall. This angle was secured to the sheet pile with a continuous fillet weld and to the vault wall with adhesive anchors. At the other few locations where sheet pile was not used, such as at the northeast corner of the site, deformed dowel bar anchors were welded to the last section of sheet pile and lapped with horizontal reinforcing within the adjacent shotcrete basement wall.

At the connection between ground floor slab and perimeter sheet pile walls, welded deformed bar anchors facilitate the transfer of wind or seismic diaphragm forces from the interior core walls to the sheet piling. In-plane forces are then transferred from the sheet piling into the adjacent soil using friction capacities resulting from the lateral soil pressures acting normal to the exterior face of wall. A soil-to-steel friction coefficient of 0.35 was used at outer pile flanges and a soil-to-soil friction coefficient of 0.75 was used between these outer flanges. For conservatism, lateral pressures within the upper 20% of retained soil height were ignored.

The P1 and P2 basement level slabs are connected to the exterior sheet pile walls with only a steel ledger angle. This ledger angle is a redundant feature, since a row of building columns are located just a few feet within the sheet pile perimeter, thereby eliminating the need for the sheet pile walls to support gravity loads. In the project's design phase, the sheet pile wall had not yet been approved as a four-hour fire rated assembly, which is required for gravity load-carrying basement walls in the City of Los Angeles (LABC, 2002). (It is important to note that testing sponsored by the sheet pile manufacturer, Nucor Skyline, has since been successfully completed by an approved testing agency, thereby allowing sheet pile walls to carry gravity loads in future projects).

Approval for the use of sheet pile on this project involved a comprehensive effort by the design team, general contractor, sheet pile manufacturer and sheet pile installer to address and answer all questions put forth by the City's permitting agencies.

Observing the Bottom of a Drilled Shaft

As previously mentioned, drilling for foundation shafts was performed from the P-1 level. At the time of the shaft construction, the upper 20 feet of the soil was within the limits of excavation for the P-2 and P-3 levels. Thus, the foundation shaft begins approximately 20 feet below the ground surface level at the time of drilling.

Perched groundwater present at the site impacted the construction of the drilled shafts, and drilling mud and/or casing were used in most of the drilled holes to minimize caving of the shaft sidewalls. The shafts ranged from 3 to 6 feet in diameter and were designed to develop their resistance in end bearing. Thus, careful observation of the bottom of the shaft excavation was required to assure loose materials were properly removed.

Capacities of the drilled shafts were on the order of 2,100 kips, resulting in an applied bearing pressure of over 35 tons per square foot at the shaft tip.

To perform the necessary bottom observation, a customized drilled shaft inspection camera (DSID) was fabricated by the geotechnical engineer (see Figure 10). The device consists of a 24-inch bell that houses a video camera, water, and compressed air lines. The bell is equipped with notched triangular blades labeled in half-inch increments. When viewed remotely via the fiber optic camera, the depth of penetration of the bell into the bottom of the shaft excavation can be observed.

Figure 10: Drilled shaft inspection device

The sequence for the observation of the shaft bottom includes at least one pass with a clean-out bucket by the drilling contractor, lowering the device to the bottom of the shaft, activating the compressed air line to force the drilling mud, and viewing the amount of penetration of the bell. Several observations were made in each drilled shaft to provide coverage over the large-diameter shaft bottoms.

The video is fed to a remote television and viewed and narrated by the geotechnical engineer. A complete video and audio recording of each observation is saved electronically and later transferred to appropriate media.

Discussion of Geotechnical Conditions

Geotechnical exploration borings were drilled using a combination of hollow-stem auger drilling equipment and large diameter bucket auger drilling equipment. The boring depths ranged to 150 feet below the existing ground surface. The large diameter borings were drilled to provide the foundation drilling contractor with an estimate of the stand-up capabilities of the shaft side-walls; to indicate the presence of any perched groundwater; and to help evaluate the need for special provisions to minimize or mitigate caving of drilled shaft sidewalls.

The site is underlain by unconsolidated river sediments consisting of alternating layers of clays, silts, sands, and gravels that extend to depths of approximately 200 feet below ground surface (bgs). The primary groundwater aquifer at the site is deeper than 150 feet bgs, although shallower perched groundwater zones are typically present above cohesive deposits. Perched water was encountered in the drilled shaft excavations.

The soils at the site included up to 5 feet of fill materials overlying medium stiff to very stiff silt and clay to depths of approximately 15 feet bgs. Very dense, well-graded to poorly graded sand with gravel and cobbles is present below the upper silt and clay, typically extending to approximately 45 feet BGS. The overall dense sand layers include occasional stiff to very stiff layers and lenses of silt, and dense layers of silty sand. Stiff to very stiff, silty clay was encountered below the dense sand and gravel layer to a depth of approximately 70 feet, and that layer is generally underlain by dense sand to the depths explored. A geologic cross section is shown on Figure 11.

The project site is located within a City of Los Angeles designated methane zone, and protection from methane as seepage into the building was required.

Interestingly, perched groundwater was not encountered at the time of the borings drilled in the summer months; however, groundwater was encountered during the construction of the sheet pile and drilled shafts, which was performed in late spring. One explanation for the groundwater encountered during the drilled shaft installation is that the sheet pile installation may have trapped perched water within the site boundaries that would have otherwise migrated downstream.

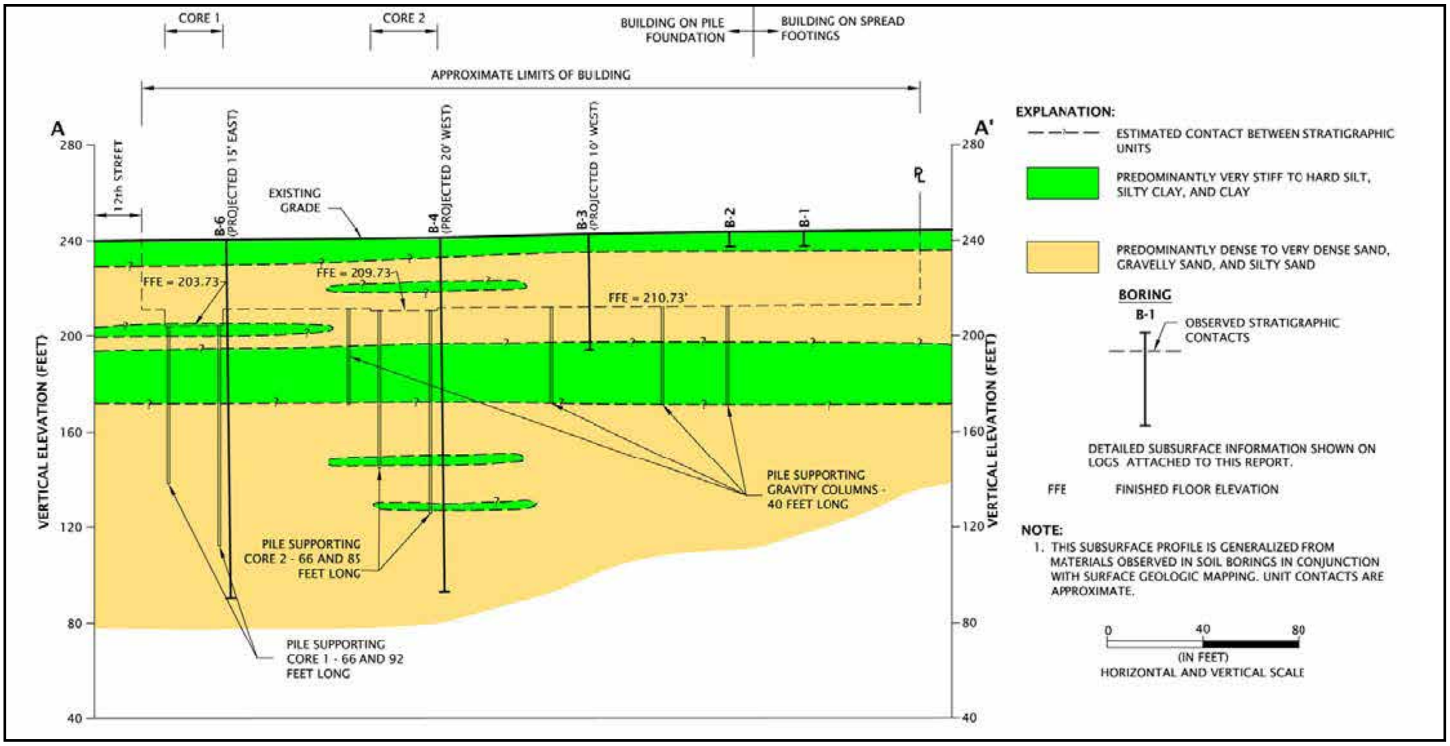

Figure 11: Geologic cross section

Drilled shaft foundations supporting gravity columns were designed to tip in the dense sand layer approximately 70 feet below the original ground surface and 40 feet below the P3 level. Deeper shafts were required to support the tower core walls and associated seismic loading. These shafts extended upwards of 120 feet below the original ground surface (85 feet below the P-3 level) and tipped in the dense sand layer present at that depth.

For the temporary shoring condition within the conventional zone, sheet piles were designed as part of a braced system that included tie-back anchors (see Figure 12). For the permanent wall below grade condition and for the temporary shoring condition within the up-down zone, internal bracing is provided by the P3, P2, P1 and ground level floor slabs. Due to the potential for periodic perched groundwater, the lower section of the sheet piles was designed to resist hydrostatic pressure in addition to the active soil pressure loading.

Since the excavation extended below the subterranean level of the adjacent DWP vault, the adjacent vault walls were supported during the construction by conventional underpinning techniques. These included installing soldier piles, located immediately in front of the existing walls and slant drilled at a very slight angle, which were then tilted into place and aligned beneath the adjacent foundation. Bearing plates were attached and shimmed to provide positive support for the adjacent tower foundation. See Figure 13.

Figure 12: Installation and tensioning of tiebacks at sheet pile within conventional zone

Figure 13: Underpinning at existing DWP vault

Conclusion

The use of the up-down construction method on this project resulted in a savings of approximately two months in the construction schedule due to the inherent ability of the up-down methodology to remove the below-grade construction component from the critical path. Above-grade tower construction was able to proceed at the same time as the below-grade construction within the same footprint. The use of plant precast concrete vertical elements from the P3 to P1 basement levels allowed for prior fabrication, increased quality, and tighter tolerances of these elements.

Further advantages were gained with the use of sheet pile as basement wall construction along three sides of the site. The sheet pile could be placed entirely before the commencement of excavation, needed no footings or piles to support it, and served as both temporary shoring and the finished basement wall. As an impervious barrier to both water and methane transmission, it eliminated the need for a separate membrane on its backside.

Acknowledgements

The authors of this paper wish to thank Mr. Roger Zimmerman, formerly of Malcolm Drilling, Inc. and Mr. Jaime Albornoz, formerly of GeoDesign, Inc. Mr. Zimmerman was responsible for many of the innovations in the design and construction, including the fabrication of the drilled shaft installation guide template. His energy, design and construction ingenuity, and collaborative approach to this work provided inspiration to the design and construction team. Mr. Zimmerman passed away in January 2008 after a three-year battle with cancer.

Mr. Albornoz performed key components of the geotechnical investigation and nearly all of the geotechnical design calculations. He also led the geotechnical field observation and testing, which included securing the permit from the City to use the remote shaft installation device, as well as implementation of this device in the field during construction. Mr. Albornoz brought a positive and collaborative approach to his work that was reflected by other design and construction team members in their interaction with him. Mr. Albornoz passed away in March 2008 after a tragic car accident on Easter Sunday. The authors of this paper owe a great debt to Mr. Zimmerman and Mr. Albornoz, and wish that their contributions to this project be recognized.

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