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Shoring Wall and Subsurface Structural Systems: IDX Tower, Seattle, Washington

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The 40-story IDX Tower is located in the Seattle financial district adjacent to the historic Downtown Seattle YMCA. Excavations extending as deep as 97 feet below the adjacent streets were modeled and monitored. Underpinning and interfingered tieback anchors supported the YMCA and the associated re-entrant corner. This paper reports on the performance of the excavation shoring. The new building is supported on a mat footing that varies in thickness from 6 to 14 feet. This paper also discusses the design of the mat, and the building core wall support.

The IDX Tower Development at Fourth and Madison

Groundbreaking for the 512-foot IDX Tower at Fourth and Madison took place in October 2000. The Tower is a 40-story, 1,053,000-ft^2 (846,000-ft^2 leasable) office development in the center of downtown Seattle's Financial District. The office tower, the

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eighth largest in Washington and the first to top the 500-foot height mark in Seattle in over a decade, sits atop a retail podium highlighted by a dramatic 5-story atrium entrance. The structure consists of steel framing with a concrete core. Clad in a combination of light gray, figured granite, metal and glass curtain wall, the tower has views encompassing Mount Rainier, Puget Sound and the Olympic Mountains. At 24,400-24,900 square feet, the highly efficient floor plates of tower feature core-to-wall dimensions exceeding 42 feet.

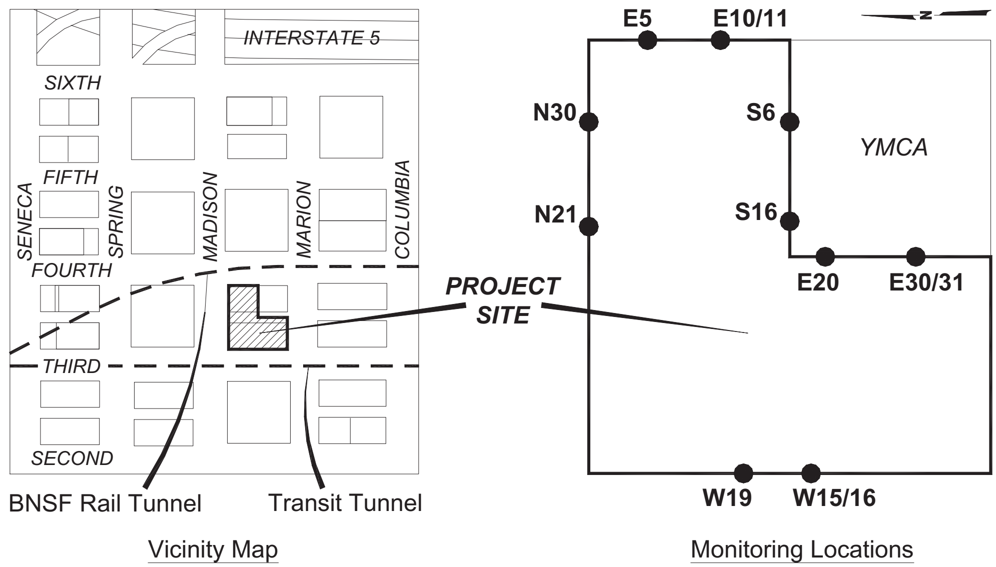
The $95.8 million project is being built for National Office Partners, a Hines Limited Partnership. Hines has developed more than 125 million square feet of commercial space around the world. Zimmer Gunsul Frasca Partnership of Seattle served as Design Architect with Kendall/Heaton Associates as Production & Construction Administration Architect. PCL Construction Services, Inc. of Bellevue Washington is the general contractor. The building is named for IDX Systems Corporation, a Burlington, Vermont company that develops health-care information systems, that will occupy approximately 300,000 square feet.

Built on one of Seattle's hills, this building deals efficiently with the steeply sloped site. There is an elevation difference of 41 feet between the main entries off Fourth and Third Avenues; tenants enter on the fourth floor on Fourth Avenue and the first floor on Third Avenue. Two levels of office space, the loading dock, the parking garage entries, and the building management spaces are located entirely on floor levels between the two main entries.

IDX Tower at Fourth and Madison

Subsurface Conditions Disclosed Overconsolidated Soils and Potential Obstructions

Eight new explorations were drilled to explore the site. Together with sixteen others from nearby projects, they formed the basis for geotechnical design parameters. Ground surface elevations range from 145 feet along Fourth Avenue to 105 feet along Third Avenue. Subsurface soils generally consist of about 5 to 10 feet of surficial sand fill overlying interbedded silt and sand down to about elevation 80 feet. Below this interbedded material is a more consistent unit of clay and silt, varying in thickness from 10 to 40 feet. Some of the clay soils were blocky and had slickensided surfaces. A cemented silt and sand underlies the clay to below the bottom



Vicinity Plan and Soldier Pile Monitoring Plan

of the excavation. Except for the fill, all soils are overconsolidated and very dense or hard.

Isolated wet zones exist in the upper soils, and more significant groundwater was encountered above the silt and clay units between about elevation 80 and 100 feet.

During pre-excavation remodeling of the YMCA, the soils beneath the existing basement floor slab had been revealed as loose or poorly compacted fill. Voids had also been encountered. The reconstruction of the floor slab and installation of new shear wall footings had improved the conditions beneath the YMCA, but the uncertain conditions remained as a design consideration for the support of the building during excavation.

On the east side, below Fourth Avenue is the Burlington Northern Santa Fe railroad tunnel located at a depth of approximately 100 feet and below Third Avenue on the west side is the Seattle Metro bus/transit tunnel at depth of 56 feet. These tunnels are both quite sensitive to disruption of their geotechnical environment. The railroad tunnel was built largely by hand methods in the late 19th century and allowed considerable disturbance of the soil above the bore. The transit tunnel is built of segmental construction and relies for stability on soil pressures to maintain its shape. Consideration of the performance of the tunnels and their locations was a major factor in establishing the configuration of the subgrade levels for the project.

The office tower cantilevers 12 feet over the adjacent historic Downtown YMCA building, to create a gracefully arcing glass facade. Covering 3/4 of a block in the form of a 200 by 200-foot "L," the project also includes shoring for three streets with a maximum excavation depth of 97 feet, a 4-story basement, and complicated underpinning of the brick and terra-cotta YMCA building. The historically registered YMCA built in 1930 occupies the southeastern quarter block of the site.

Design and Construction Criteria and Challenges

General Shoring Wall

Conventional soldier piles and tieback anchors were the obvious and appropriate choice for excavation support. This approach has been used extensively in Seattle, with good and documented results. The method consists of a W-section steel beam installed in a predrilled hole that is then filled with lean concrete. Soldier piles are spaced at 6 to 10 feet laterally.

Tieback anchors are drilled into the soil adjacent to the soldier piles at angles of about 20 degrees below horizontal, affixed with locking pins, and prestressed. They are installed at vertical spacings of about 8 to 10 feet. This method of installation is intended to limit post-installation deflections. Installation difficulties can arise from groundwater or unstable soil, requiring that the soldier pile holes be cased, from obstructions (such as boulders or remnant concrete foundations) that may require coring, or from previously installed (though destressed) tieback anchors from adjacent excavations.

Estimates of the lateral soil pressure envelope on deep excavation shoring walls are usually made using a combination of theoretical relationships, such as those from Terzaghi and Peck (1), Schnabel (2), and others (3, 4), and local experience (5, 6, 7, 8). Since relatively few deep excavations are instrumented to allow measurement of actual loads, local experience and the observations of past successfully completed excavations becomes increasingly important. Typically, the design envelope is accompanied by an expectation of about 1 inch of lateral deflection at the ground surface, a value repeatedly shown to be protective of adjacent streets, utilities, and buildings.

Expressed in terms of the excavation height, H (in feet), the design wall pressure envelope for the IDX Tower was set at 22H psf adjacent to the city streets, and 27H psf adjacent to the YMCA. The additional 5H psf adjacent to the walls adjacent to the YMCA represented an attempt to reduce the deflections of the wall and thus settlement of the YMCA. Surcharge loads from the YMCA footings were added to the soil pressure envelope. The pressure diagram is shown below. The diagram is intended not to predict the actual pressures on the wall, but rather to envelop the maximum pressure at any individual tieback location.

The shoring system selected for the IDX Tower included 137 steel soldier piles ranging in size up to W14 X 159 pounds per foot and a maximum length of 112 feet. Eighteen of the piles are used for underpinning for the northern elevation of the YMCA over a width of approximately 80 feet. A total of 677 tieback anchors up to 96 feet in length restrain the soldier piles.

The project required a complex shoring system, as the site is located in a crowded urban environment. The project had to contend with many existing underground utilities including the railroad and transit tunnels in the adjacent rights-of-way. The project also required demolition and staging coordination with existing buildings on site. A temporary, intermediate shoring wall approximately 15 feet high was designed to allow demolition of existing structures to proceed on the west half of the site.

Support of the YMCA

The west side of the YMCA was set back about 7 feet from the face of the shoring wall, although the YMCA perimeter footings were within a few feet of the wall. Accordingly, all of the loads from the YMCA were applied to the shoring wall as lateral loads on this side. On the north side of the YMCA the renovation had included construction of a new continuous perimeter shear wall along and flush with the face of the exterior wall face. On this side, the new wall was underpinned, transferring the footing loads to the soldier piles as vertical loads. Only the lateral pressures from the interior footings were applied to the shoring as lateral loads.

Tieback anchors passed within about two feet vertically of the bottom of the perimeter footings on the west side of the building, and as close as 4 to 8 feet below interior footings on the west and north sides. In addition, for approximately one-third of the wall the tiebacks were interfingered – with tiebacks from the west wall crossing those from the north wall. The design separation between the ties was two feet.

Shoring of a re-entrant corner such as this is relatively uncommon, and the performance is made more critical by the size and construction of the retained building. Some of the designs of these projects have modified the no load zone behind the wall, or reduced the available frictional resistance of the tiebacks, or increased the design lateral pressures in the area of the tieback interfingering. This design incorporated none of these extra measures because the designers could find no compelling theoretical or performance-based evidence that these modifications better modeled the actual conditions.

"No-load" Zone Modification

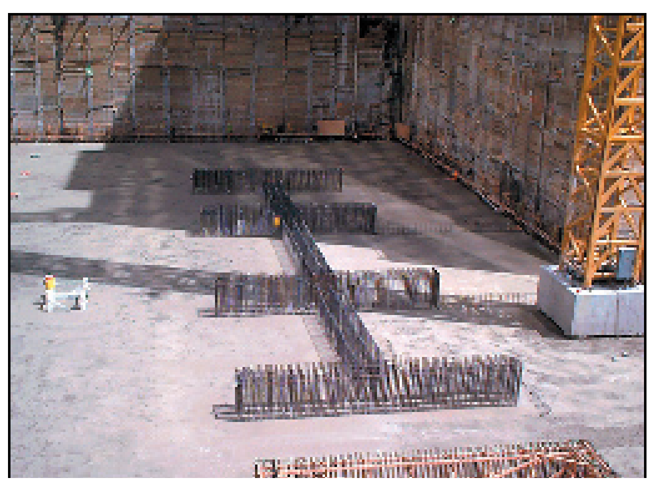
Along Fourth Avenue and the eastern half of Madison Street the street widths were not sufficient to allow construction of upper tieback anchors of traditional length. The no-load zone is an area of soil behind the shoring wall that is approximated by the active wedge. Since the shoring walls are generally designed for some deflection, and thus active pressure conditions, the anchor zones of tiebacks must be behind this yielding soil. The no-load zone is typically defined by a line extending upward from the base of the excavation to the ground surface at a 60-degree angle, and set back by a value of one-third to one-fourth of the height of the excavation. This can result in very long upper row tiebacks. In fact, this traditional construction resulted in a no load zone that intersected the ground surface about 75 feet behind the face of the excavation. On Fourth Avenue the property line to property line measurement (and thus maximum tieback length) was only 83 feet. On Madison Street the available tieback length was only 66 feet. Shorter tiebacks supporting the deepest section of the excavation was a cause for some concern, but truncation of the no-load zone is not unprecedented. Applying a more accurate estimate of the active pressure failure surface results in a log spiral shape that is near vertical at the top. In addition, other excavations in Seattle, most notably for the Columbia Tower (6) faced similar length restrictions and demonstrated that the no-load zone truncation does not necessarily lead to greater deflections of the wall.

Foundation Support

The core mat bearing pressure and settlement design considered not just the competence and consistency of the supporting soil, but also the nature of the applied loads (dead, live, and seismic or wind), the timing of the likely settlement (during construction or long term), and the resulting rebound and recompression caused by the removal of an average of 80 feet of overburden. The resulting allowable soil bearing pressure of 12 ksf beneath the mat was one of the largest values ever used in Seattle. One-dimensional and two-dimensional FLAC analyses gave similar estimates of settlement response – a value of 2-1/2 inches.

Earthquake Design

The project advances the state of the art in earthquake engineering using a performance-based design concept for the concrete shear-wall core. The core walls rise 553 feet above the 14-foot thick, concrete mat foundation. In order to meet the Seattle Building Code's (UBC) height limits, a building of this size would traditionally require dual lateral systems. A similar design, not performance-based, could, for example, employ very large columns and diagonal steel bracing combined with a costly, intrusive moment frame. The height limit mandated by the building code for a traditional concrete shear-wall core for a high-rise building is 240 feet. The design of the IDX Tower insures that the concrete shear-wall lateral system meets the ductility performance objectives of the Building Code by more accurately analyzing the performance of the building under a seismic loading in excess of the requirements of the Building Code. The resulting design produces a highly economical structure. The lateral system maximizes value for the project by taking less space in plan and elevation. The concrete core is very stiff and minimizes damage to interior, skin and building contents during earthquakes. Additionally, more stringent steel reinforcement detailing requirements are employed which significantly enhance the performance of the structure in the event of large earthquake ground motions. The core also enhances occupant comfort under the effects of wind-induced sway.



Shear-wall rising from the foundation

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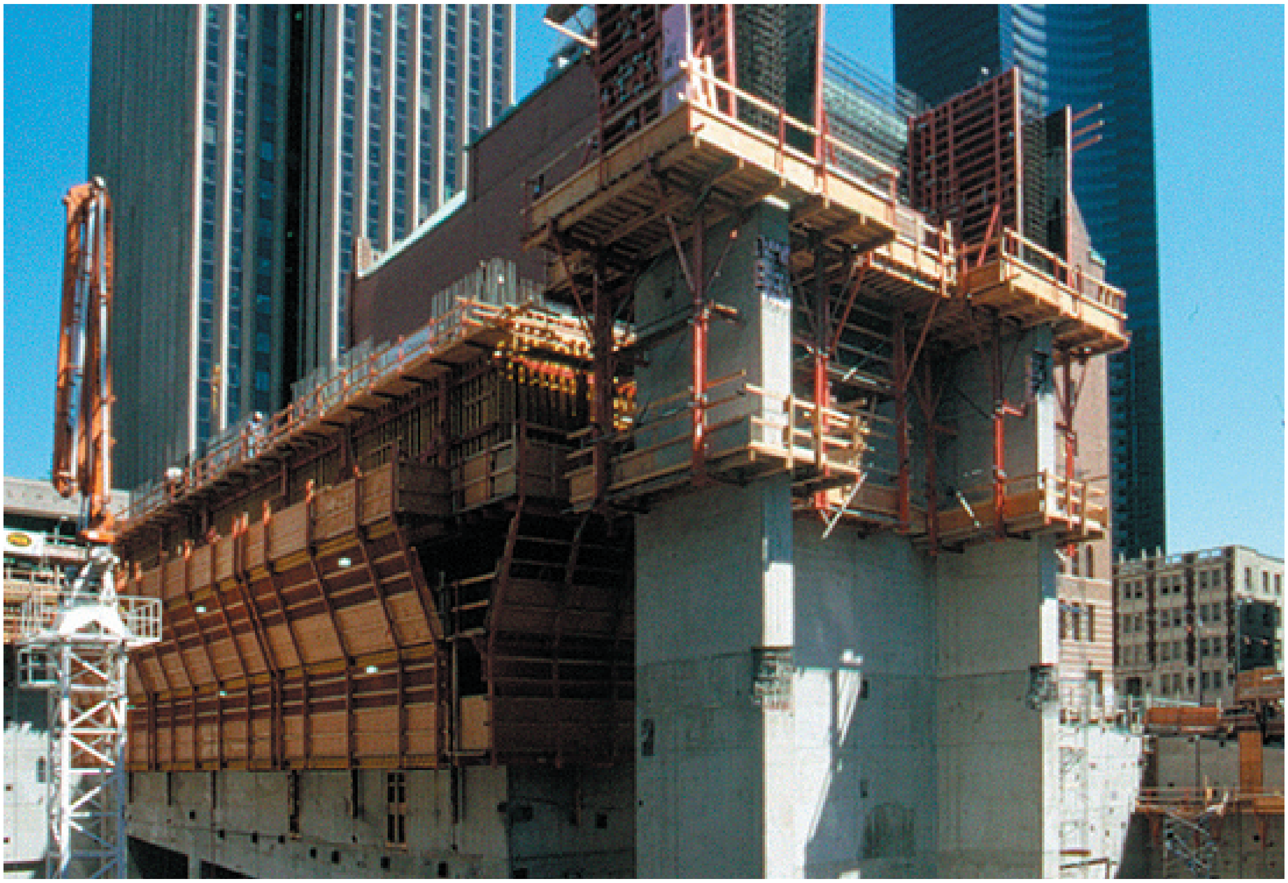


Concrete core

The geometry of the shear-wall core is switched over a height of 42 feet from a rectangular box section to a cruciform section, with the same outside dimensions, 101 by 32 feet at the base. The concrete core walls range in thickness from 1 foot at the top to four feet thick in the parking garage. The walls are much less intrusive to the parking layout and leasable space than large built-up or composite columns or an exterior braced frame system, while creating a building stiff enough to minimize damage to non-structural elements (partition walls, windows, etc.) in the event of a large ground motions.

Subsurface Structural Elements

This building faces severe structural challenges from the slope of the site. The elevation difference of 41 feet between the high and low sides on Fourth and Third Avenues, respectively, imposes a net unbalanced soil loading on the basement structure in excess of 6,000 kips plus an additional allowance for seismically induced pressure in the soil. This large lateral earth pressure is resisted primarily by the shear-wall core. In the lower levels the earth pressure combines with the maximum magnitudes of seismic lateral force. The central wall of the cruciform shape core section is 4 feet thick by 101 feet long in the direction of the principal earth pressure.

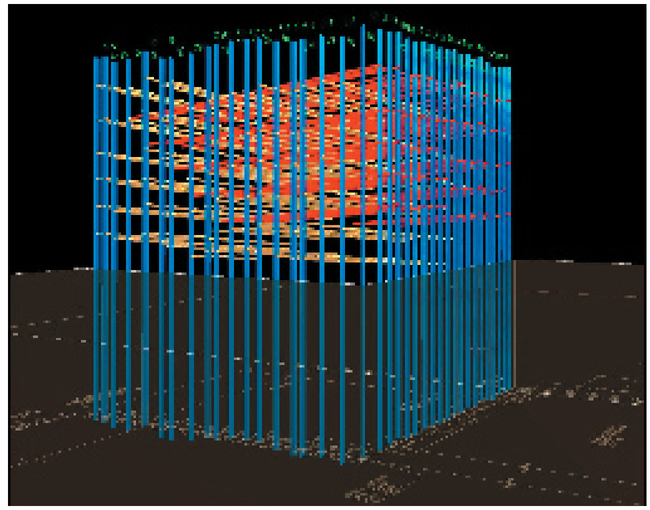
The large interacting set of lateral forces result in a very high overturning effect on the concrete core. The foundation is a mat is 237 by 119 feet overall, with a 6 feet thick perimeter zone that receives all the tower columns. The interior section of the mat is thickened to 14 feet in the region of the core walls; a zone 87 by 157 feet. For design of the mat, the bearing strength of the extremely competent native soil was taken as 12 ksf. This cross-section of the mat was selected as structurally optimum. The mat employed vertical shear reinforcement in limited areas where detailed finite element computer modeling indicated the punching effects at the corners of the core exceeded the strength of the 14-foot solid concrete section. A conventional concrete mixture with a strength of 5000 psi was employed for the mat, which was placed in a continuous pour of 10,952 cubic yards. The mat pour, second only to AT&T Gateway Tower in Seattle history, took place flawlessly over a period of 12 uninterrupted hours. It was determined to take no special measures for the massive nature of the concrete placement. Conditions were cool and moist and the placement on grade allowed for ideal curing conditions. No cracking of the mat has been noted.

As a side note, the 14-foot height of the upper reinforcing layers in the mat required extraordinary consideration of the design of the support system for the top bars. Collapse of the reinforcement during placement would essentially destroy the entire mat. Normal rebar “standees” just weren’t going to make it. Trusses and similar solutions were considered but these had long lead times and were too costly. Skilling designed a system of supports using reinforcing bar in a three-dimensional matrix. The rebar matrix was, in effect, a very large space-truss. The bars were interconnected using twisted, soft iron wire connections of known strength plus limited welding. The resulting assembly was designed for live loads from the placement equipment and workers, and reviewed for hydrodynamic forces resulting from concrete placement. The system proved very cost-effective and functioned perfectly.

Computer Modeling to Predict Performance

Interfingered Tieback Anchors Beneath the YMCA

Skilling developed a complete three-dimensional computer model of the shoring system around the YMCA. The model was intended to review the complex geometry of the 38 soldier piles along this corner of the excavation. The piles were supported laterally by up to 6 rows of tiebacks per soldier pile. With so many tiebacks crossing each other at varying angles, there was a potential for tieback conflicts. The model allowed us to adjusted the soldier pile and tieback locations so that there were no conflicts.



Interfingered tieback computer model

Underpinning the YMCA North Wall

Another complex area studied by three-dimensional computer modeling was the corner excavation. The tower plan made it necessary to excavate below and immediately against the North wall of the YMCA building. This excavation was accomplished using 18 slant-drilled underpinning soldier piles. These underpinning piles supported the existing building’s north wall footing during the excavation. These piles were installed by angle drilling a hole beneath the existing footing, then placing the soldier pile beneath the existing footing, and encasing it in concrete. Jacks

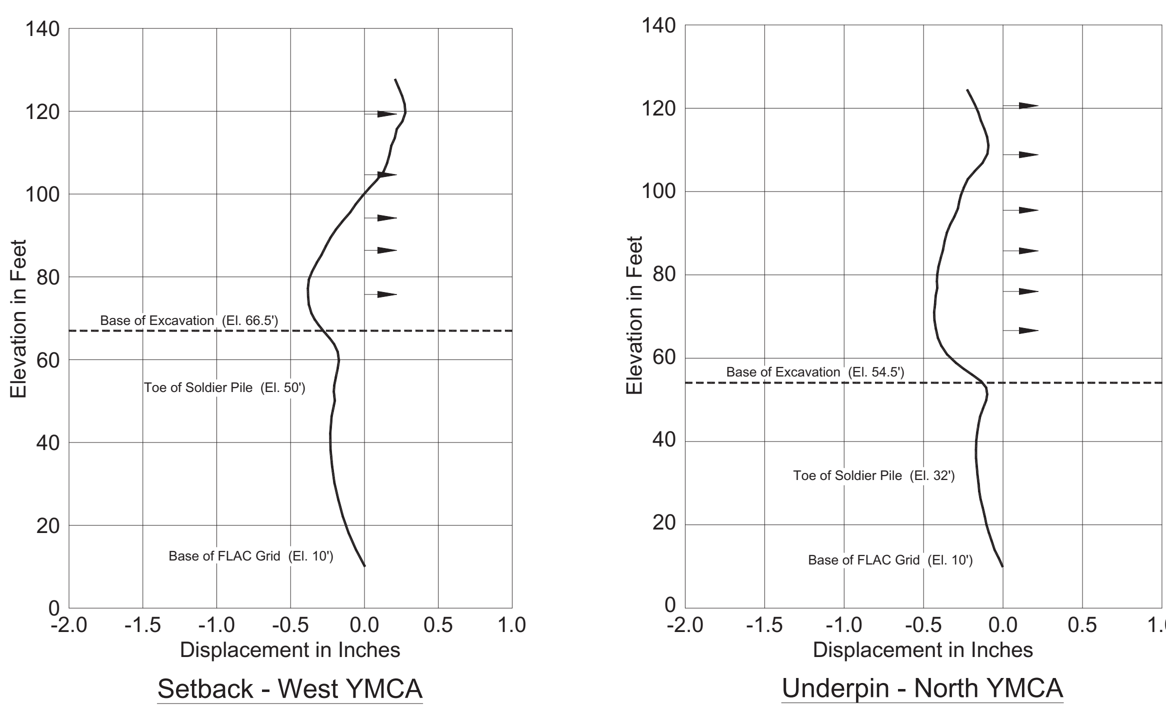
YMCA shear-wall underpinning

were then installed on the soldier pile. These jacks were loaded to provide the vertical force necessary to support the existing footing. This underpinning design allowed the IDX tower basement to extend approximately 70-feet below the existing YMCA basement.

Displacement-Based Analyses Using FLAC

FLAC is a two-dimensional finite difference program used to model soil-structure interaction problems. Instead of a factor of safety result, such as from limit-equilibrium analyses, FLAC calculates forces and displacements directly based on the input structural elements and the soil properties. The two key output values from the FLAC model were displacements along the soldier pile and loads on the tieback anchors. Carefully designed soldier pile/tieback walls rarely fail catastrophically. Rather, they deflect more than anticipated, resulting in ground surface settlement behind the wall, and potential distress to adjacent streets, utilities, and buildings. Overstressed tiebacks deflect much like friction piles can settle. When they deflect, they shed load to other nearby tiebacks causing them to deflect. The result can be a section of wall that rotates toward the excavation before measures (such as additional new tiebacks) can be installed to pick up the excess loads. Thus the relationship between tieback loads and soldier pile deflections predicted by FLAC is a critical check of the traditional design process.

To confirm the key input soil parameters from the geotechnical engineering design report additional field studies were completed using three pressuremeter tests in two new explorations near the YMCA. The pressuremeter test provides a good estimate of the modulus of the soil, which is critical for deformation-based analyses. An outside consultant completed the tests, and recommended the resulting soil modulus values. This same consultant had recently completed pressuremeter testing at another downtown Seattle site with similar soils, and a FLAC analysis had followed. This allowed the analysis for the IDX Tower to be calibrated more accurately to the soil conditions based on actual predicted and observed results at a nearby site. The analysis was further checked by predicting the deflections and loads from a typical cross section of soils combined with a typical cross section of shoring. Under such conditions the actual



FLAC analyses

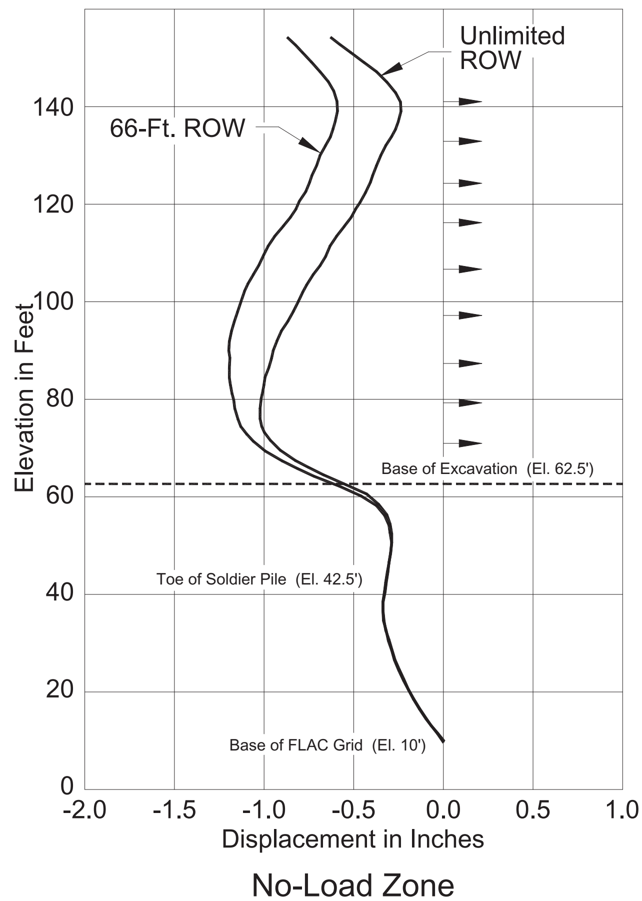
deflections had been well documented from other sites, so the "accuracy" of the FLAC model and the appropriateness of the many input parameters could be assessed before the IDX Tower combination of soil and shoring was analyzed.

The FLAC model was used to predict deflections for three actual/typical conditions: a soldier pile underpinning the north wall of the YMCA; a soldier pile supporting the lateral loads from the west wall of the YMCA; and a soldier pile along the east wall of the excavation representing the maximum required truncation of the no-load zone.

The plots below show the predicted deflections of the two soldier piles supporting the YMCA. Note that the magnitudes of the maximum expected deflections are similar – about 3/8 inch occurring in the lower quadrant of the soldier pile. The maximum expected deflections are similar even though the underpinning soldier pile on the north YMCA wall is about 30 feet longer, and the excavation about 15 feet deeper than at the soldier pile location on the west wall. This indicates that the relationships between soldier pile deflection, pressure envelope, and tieback loads are consistent, regardless of the depth of the excavation and magnitude of surcharge loads. The deflection of the west soldier pile into the soil near the top suggests that the pressure envelope or surcharge loads are overestimated and that the upper tiebacks are overdesigned.

The displacement plot below demonstrates the potential effect of the modified no-load zone. Note that the FLAC model predicted less than ½ inch of additional deflection as a result of the modified no-load zone. Maximum predicted deflections of about 1-1/4 inches again verified both the pressure proposed pressure envelope and the acceptability of a modified no-load zone.

The modified no load zone is a truncated wedge. The wedge is truncated at a distance of H/2 feet behind the excavation for the tiebacks along Fourth Avenue, and as close as H/3 feet for a portion of the excavation along Madison Street.



FLAC analyses

Excavation Monitoring and Instrumentation

Installation of the 137 soldier piles and 677 tieback anchors, and excavation of the 105,000 cubic yards of soil took five months. During excavation the performance of the shoring system was monitored in several ways:

- Five inclinometers installed in borings adjacent to and just behind the shoring wall at soldier pile locations E5, E10/11, E20/21, E30/31, and W15/16;
- Hydraulic load cells attached to tiebacks on seven soldier piles: N21, N30, E5, E20, S6, S16, and W19; and
- Optical survey monitoring of the soldier piles, YMCA, and adjacent streets by both the contractor and an independent surveyor.

Instrumentation Results – Inclinometers Compared to FLAC

Inclinometer data for E5 and E20/21 are presented below. Both deflection shapes are comparable to each other, and generally consistent with the predicted deflection shape of the FLAC analysis. They both indicate greater deflections near the base of the excavation than at the top, and a bulge in the 20 feet or so above the base of the excavation. At E5, the soldier pile is in the deepest part of the excavation, and should be compared to the FLAC analysis for the no-load zone truncation. These two compare reasonably well, although the actual deflections were slightly greater than the FLAC predicted deflections, suggesting that the soil modulus values were slightly too stiff, and that the design soil pressures may have been too low for the planned deflection. It may also indicate that the effects of the reduced no-load zone were greater than predicted.

At E20/21 (located adjacent to the west wall of the YMCA) the surcharge loads from the YMCA are applied to this soldier pile/tieback system, and the FLAC analysis. The actual deflected shape suggests that either the soil modulus in the FLAC analysis is too stiff, or that the surcharge components from the YMCA were underestimated, or that the tieback anchors were underdesigned. These tiebacks were interfingered with those from the north wall of the YMCA. Note also that the load cell on the lowest tieback registered zero load on that tieback despite being locked off at 100% of the design load. This tieback may actually have failed after installation, resulting in load shedding to the other nearby tiebacks, and increased deflection of the soldier pile near the bottom of the pile.

Inclinometer W15/16 and E30/31 (not shown) demonstrated a consistent pattern of deflection. Above the first tieback level, a spike of deflection

Inclinometer monitoring results

occurred into the excavation that was pushed back by each successive anchor installation, gradual increases in lateral deflection as the excavation deepened, and then a bulge in the 10 to 15 feet above the bottom of the base of the excavation. This shape suggests that the pressures are higher than at the top, although the design values were not inappropriate, since the magnitude of the deflections was less than ½ inch.

Instrumentation Results – Load Cells

Twenty-two of the forty-three installed load cells on seven soldier piles were still functioning and accessible at the end of the excavation. All instrumented anchors were locked off at between 80% and 100% of the design values. So any measurements on the load cells that were below the lockoff load represents a relaxation of pressure. Only two of the load cells showed post-lockoff tieback loads exceeding 90% of the design values. Twelve of the load cells showed post-lockoff loads of between 70% and 90% of the design values, and eight showed loads below 70% of design. There was no apparent correlation between the load cell pressure and the observed deflection.

One conclusion to draw from the load cell data is that the wall pressures were overestimated by an average of about 20%. If the wall pressure envelope had been defined as18H for the shoring walls adjacent to the streets and 22H for the walls adjacent to the YMCA, only four of the load cells would have shown loads greater than 105% of design, and only one would have exceeded the 130% proof test value applied to each anchor installation. Since the tiebacks are designed with a safety factor of at least 2.0, plenty of capacity would remain with the lower design value.

Instrumentation Results – Optical Survey

Lateral deflections into the excavation were greater than ½ inch only along Fourth Avenue, the deepest part of the excavation and one of the areas with a truncated no-load zone. Independent surveyors measured lateral deflections of about 1-1/2 inches in the middle of the block, and ¾ inch to 1 inch near the corners. By comparison, lateral deflections on the west wall (Third Avenue) and the north wall (Madison Street) were ¼ inch to ½ inch. Vertical settlements were typically ¼ inch around the site except along Fourth Avenue, where the settlements were generally 1-3/4 inches to 2-1/2 inches. The optical survey data clearly showed the three-dimensional effects of corner support. Within about 15 feet of a corner (two soldier piles) the lateral deflections and the vertical settlements are smaller than elsewhere. This additional restraint from the adjacent wall should allow lateral soil design pressures to be reduced in this zone.

Instrumentation Results – YMCA

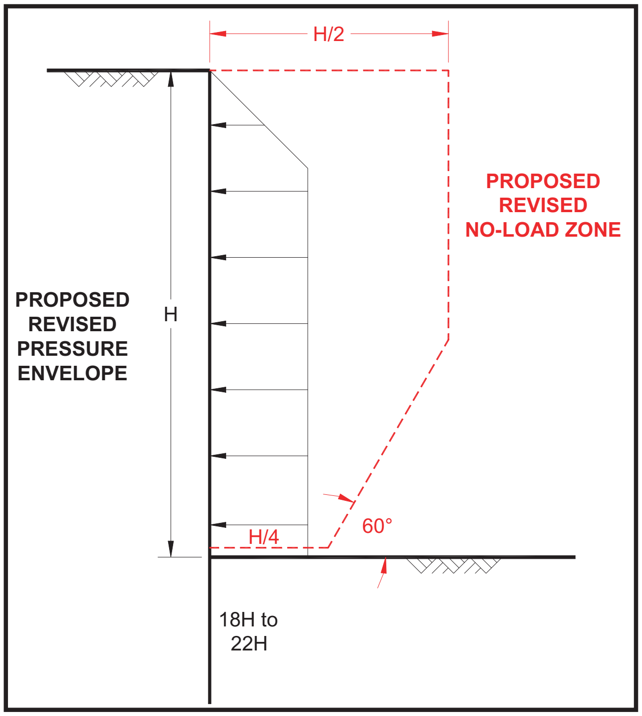
None of the shoring supporting or adjacent to the YMCA performed outside of the expectations of the design. Lateral deflections from the optical survey were generally less than ½ inch to ¾ inches at the top, and that deflection did not result in distress to the YMCA. If the deeper deflections shown on inclinometer E20/21 were typical, this further demonstrates the point that these walls are flexible, and that small lateral deflections do not affect conditions behind the wall.

This positive result further demonstrates another important point with regards to the interfingered tiebacks and the reentrant corner. The presence of tieback anchors parallel to and just behind the wall (and within the no-load zone) does not apparently affect the performance of the wall, nor reduce the mobilization of soil friction along the tieback.

Proposed New Design Criteria

The data support four modifications in the traditional lateral soil pressure design for soldier piles and tiebacks. Modifications of this type have appeared in the literature in the past, and build on the continually increasing availability of research and instrumentation results from deep excavations in overconsolidated soils.

1. Design future shoring walls in these types of soils using a lateral pressure envelope that is 80% of the conventional value. Lower pressures will result in lighter soldier pile sections, shorter or smaller or fewer tieback anchors, thinner lagging, and a less costly shoring wall.



Proposed no-load zone and pressure design changes

2. Unless the lower one-fourth of the excavation is clearly and consistently in hard silt/clay soil, do not truncate the bottom of the pressure envelope. Truncation leads to lower pressures, underdesigned tiebacks, and increased soldier pile deflection.

3. Reduce the soil pressures near the corner to account for the three-dimensional support of the wall. Based on observed reduced deflections, a one-third reduction in the design pressures for soldier piles within 15 feet of a corner is appropriate.

4. Interfingered tieback anchors beneath reentrant corners can be completed successfully, with no changes in design, nor reduction in expected performance.

5. No-load zone truncation is appropriate, and will not result in significant increases in wall deflection. The no-load zone need not extend any farther behind the face of the wall than H/2.

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