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SYNOPSIS: Due to severe right of way restrictions associated with the relocation and widening of Interstate 85 in Atlanta Georgia, a special post-tensioned caisson retaining wall was constructed within 12 inches of an adjacent parking garage and office building. National Foundation Company's design for the twenty foot high retaining structure was used in lieu of an L-shaped cantilevered concrete retaining wall that required extensive temporary shoring for construction. The caisson wall was instrumented and monitored during and after construction using slope indicators and optical survey.

PROJECT BACKGROUND

For the past several years, the Georgia Department of Transportation has been involved in a rebuilding program requiring major widening of the existing right-of-way for the Atlanta freeway system. In the first extensive use of Permanently Anchored Retaining Walls by a state highway department, Georgia has implemented cost effective alternatives to more traditional methods of retaining wall construction.

In 1981, the Georgia Department of Transportation let a $63,000,000 contract to rebuild the Brookwood Interchange where Interstates 75 and 85 meet on the north side of Atlanta. Part of the work involved relocating portions of Interstate 85. The old highway was to remain in service parallel to the new Interstate, and would serve as a four-lane feeder road. The width and alignment of old I-85 were changed in some locations, including the area of the subject project. Grade at this

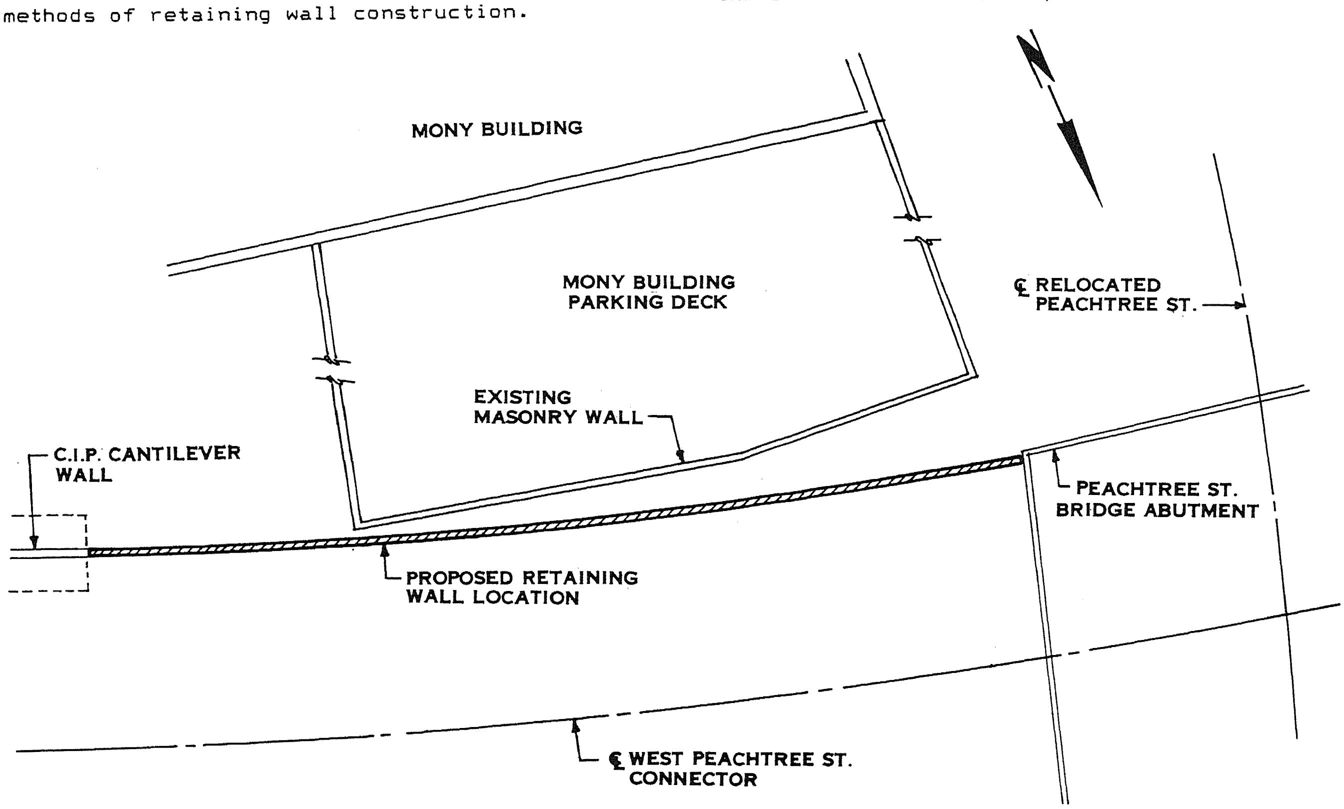

FIGURE 1. PROPOSED RETAINING WALL LOCATION

location was lowered 20 feet but the roadway shoulder was only 5-1/2 feet from an existing masonry wall. Behind this wall was a four story parking deck servicing an attached eleven story office building (see Figure 1.)

The original design drawings issued by the Georgia DOT called for sheet piling to be driven to refusal adjacent to the masonry wall. The contractor was required to design the bracing for the sheet piling to temporarily support the excavation while a concrete cantilever retaining wall was constructed. The cantilever wall was designed as an L-shaped structure due to severe right-of-way limitations which prevented construction of the heel of the footing. To adequately support the wall, the footing was to be founded on bedrock at a depth of approximately 40 feet below grade. Figure 2 illustrates the locations of the proposed temporary and permanent retaining structures. Because the new roadway was only 20 feet below existing grade, 20 feet of additional excavation below design subgrade and then 20 feet of backfill would have been required to construct the L-shaped wall on rock.

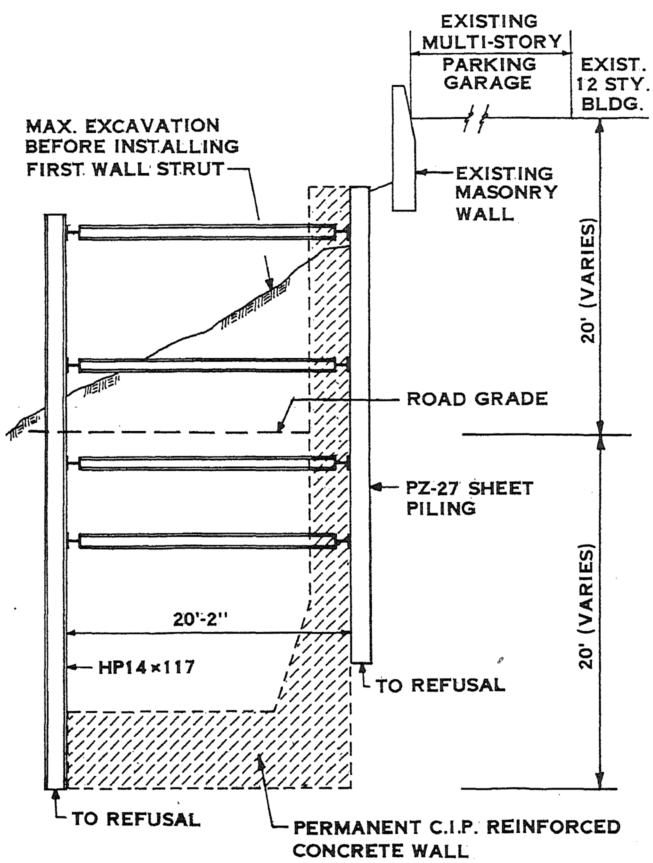

FIGURE 2. ORIGINAL RETAINING WALL SCHEME

The contractor designed a system of struts and reaction piles to support the sheeting during construction of the cantilevered wall. During a review of the shoring design, questions arose concerning the magnitude of possible lateral deflections. The Georgia DOT requested that the contractor investigate the use of temporary tiebacks rather than struts to support the sheeting. It was believed that prestressed tiebacks would greatly reduce lateral movements during excavation. Unfortunately, the owner of the adjacent building would not grant subsurface easements even for temporary tiebacks.

National Foundation Company was originally requested to prepare a cost estimate for installing the temporary tieback wall. When the subsurface easement problem arose, National Foundation proposed an entirely different scheme to limit the anticipated deflections. This scheme was based upon constructing the temporary and permanent walls as one. The proposed design limited the required excavation to 20 feet, or only the amount necessary to reach road grade. This design, which the Georgia DOT eventually accepted, is illustrated on Figure 3 and consisted mainly of the following components:

1. Fifty caissons, 42 inches in diameter, drilled from the ground surface to rock. These reinforced caissons were installed either tangent to each other or spaced 12 to 15 inches apart to form a continuous structural wall.

2. Fifty post-tensioned rock anchor tendons which were installed through a draped conduit cast into the caissons. Below the caissons, the tendons were anchored into the underlying granite gneiss bedrock which was drilled through the conduit from the ground surface. When stressed, the draped tendons imposed an eccentric load on each caisson. This load induced a moment which moved the top of the caisson backward towards the building.

WALL DESIGN

The wall design was performed by Law/Geoconsult International and required extensive coordination between geotechnical and structural personnel due to the complex soil/structure interaction mechanisms involved.

Deflection Calculations

Extensive deflection estimates were calculated to assess the effectiveness of the post-tensioned caisson concept in advance of design. These estimates were based on elastic analyses of caisson deflection using a horizontal modulus of subgrade reaction recommended by Terzaghi (1) and compared with estimates prepared for the sheet pile and strut system. The deflection estimates for the wall were based on theory presented by Kocsis (2).

The designers estimated that the temporary sheet pile and strut system originally designed for the site would deflect about 1.4 inches at the top of the wall. The Georgia DOT felt that this estimate might be optimistic. In any case, they felt that deflections should be less than about 0.75 inches to keep settlement of the adjacent masonry wall within acceptable limits. There was also concern for the adjacent parking garage since visual inspection of the drilled shafts supporting the structure indicated deterioration and cracking.

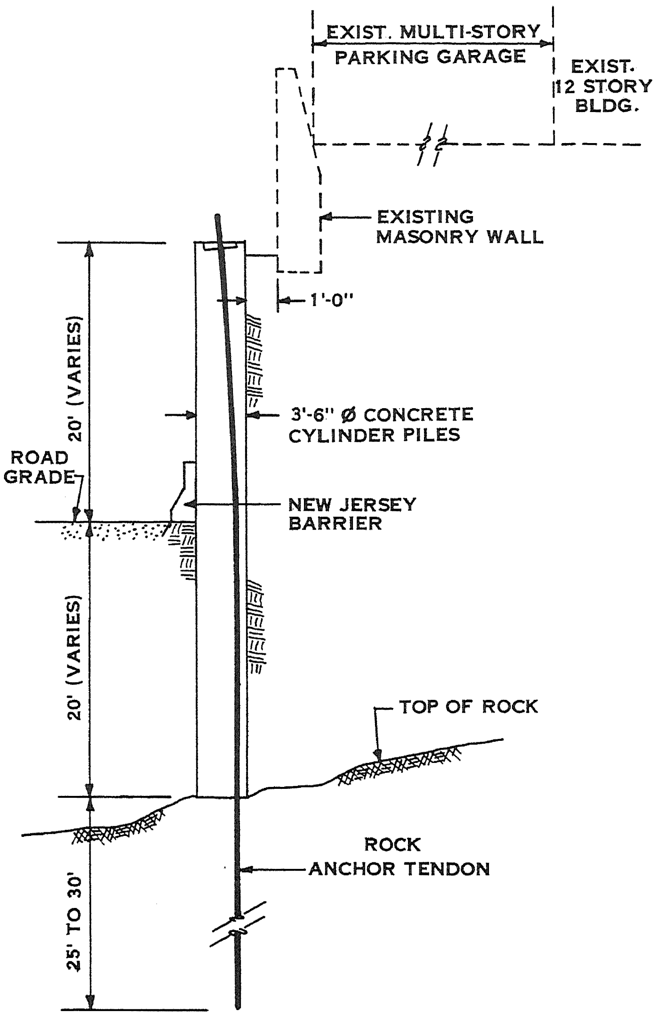

FIGURE 3. POST-TENSIONED CAISSON RETAINING WALL

Deflection calculations for the caisson wall were performed using superposition of elastic theory for the combined effects of surcharge, earth pressure, and post-tensioning. These effects were assumed to generate movements related to cantilevering as well as rotation and deflection expected at the ground line.

Deflection estimates for the caisson wall governed the caisson diameter and spacing with the limited right-of-way as the critical factor. Deflection estimates at the top of the wall were 0.7 inches for the final design caisson diameter of 42 inches.

<u>Earth Pressure Assumptions</u>

Earth pressures were calculated using assumed soil parameters based on standard penetration testing performed in the silty sands and sandy silts located at the site. No shear strength tests were performed on samples from the borings. However, the designers had considerable experience in these soils, so the assumed value for the angle of shearing resistance of 28 degrees had a high degree of confidence associated with it.

The specified at-rest (Ko) earth pressure distribution was added to calculated surcharge loading based on the Boussinesq (3) strip loading solution, doubled to account for stress reflection. Interestingly, the passive resistance against the caissons below grade was estimated based on the elastic horizontal subgrade modulus reactions, with an assumed point of fixity at the bottom of the caissons or at the top of rock. The passive effect was considered a resistance rather than an actual load since the point of fixity was assured by the post-tensioning tendons and/or caisson rock sockets. The resulting passive distribution was triangular shaped with zero points at the excavation line and at the rock line. The designers were comfortable with this conservative assumption since they felt that this distribution better modeled the actual loading conditions. It was felt that the classical triangular distribution which increases from zero to a maximum at the bottom of the structure might overestimate the amount of resistance available.

The groundwater table was located far below final excavation grade near to top of the bedrock, so hydrostatic pressures were not added to the earth pressure diagram. However, positive drainage was provided through the wall to prevent any accumulation of water.

<u>Structural and Geotechnical Design</u>

The structural design relied on theory presented by Kocsis (2) for the determination of shear and moment. The specified at-rest earth pressures and the assumed surcharge loads were used to size the caisson reinforcement and post-tensioning loads. The location of the post-tensioning ducts was governed by the caisson geometry. The ducts were placed as far to the rear of the caissons as possible to maximize the eccentricity and the moment generated by the tendons. However, the ducts were moved toward the center of the caisson near the top of each shaft and reinforcing was added to overcome excessive shear at the back of the caissons. Concrete strengths of 5000 psi were required for the caissons to resist the combined stresses.

The rock anchor tendons consisted of up to 28-0.6 inch diameter steel strands. Maximum working load was about 845 kips. The tendons were up to 88 feet in length with drill holes extending from 25 to 30 feet into sound rock below the bottom of the caissons.

A provision was made for caisson rock sockets when calculations indicated that fixity could not always be achieved with the available soil embedment indicated by the borings. The sockets were designed to be 1 or 2 feet deep, depending on the depth at which sound bedrock was encountered.

Positive drainage was provided along the wall with the use of a prefabricated drainage board placed between the caissons where they were not tangent. At tangent caissons, slotted PVC pipes were covered with filter fabric and placed in drilled holes at the point of tangency at 5 foot vertical intervals and attached to vertical PVC pipes. The drainage board and vertical PVC pipes were connected to weep holes through the New Jersey barrier at the base of the wall.

A cast-in-place, 12 inch thick reinforced concrete facing was designed for the wall. The facing consisted of 19 to 32 foot wide segments. These segments were cast around steel shear studs that were drilled and epoxied into the caissons. The facing was provided with a rusticated finish to architecturally match the adjacent conventional retaining walls.

CONSTRUCTION

Construction began in July of 1984, and took a total of 8 months. Five months were required to construct the caissons and rock anchor tendons. The remaining time was used to excavate in front of the wall and pour the concrete facing. The first phase of work was to drill the caissons, which averaged 42 feet long from the ground surface to the top of rock. Forty-two inch diameter rock sockets in the hard granite gneiss were cored for 25 of the caissons. The sockets were generally 1 or 2 feet in depth, although three of the caissons had sockets of 4, 4.5, and 6 feet in length where seamy rock was encountered. The reinforcing cages were fabricated in advance with the draped metal post-tensioning ducts tied into the proper position. The cages were lowered into the drill holes, and concrete was pumped from the bottom up, maintaining the void space in the post-tensioning ducts. When the caisson concrete had sufficiently cured, a rotary hydraulic drill rig was positioned over the ducts to drill the rock anchor sockets. The drill tools, consisting of a 6 inch diameter down-the-hole hammer and 3 inch diameter drill rods were first lowered through the draped ducts to the bottom of the caissons. The 25 to 30 foot sockets were then drilled into sound bedrock.

FIGURE 4. CAISSON DRILLING

Water pressure tests were performed in the rock anchor sockets to determine the relative permeability of the bedrock. When these tests all indicated water takes beyond the specified limits, the holes were grouted for water tightness and redrilled.

FIGURE 5. ROCK ANCHOR TENDONS

The rock anchor tendons were prefabricated and delivered to the job by truck. The individual strands were "basketed" around centralizers in the bond length to maintain at least 1/2 inch of grout cover over the steel. The tendons were placed with the aid of a crane. Primary grout, consisting of a plain cement-water mix was injected through a plastic tube placed to the bottom of the rock anchor sockets. A predetermined quantity of the mix was injected to fill only the bond length of the anchor. After allowing the primary grout to cure, bearing plates were installed at the top of the

FIGURE 6. EXCAVATION IN FRON T OF WALL

caissons at an angle perpendicular to the drape of the tendons. The rock anchors were tested to 1.5 times the design load (up to 1268 kips) using a large center hole hydraulic jack, which pulled all of the strands simultaneously. Once the anchors were tested and locked off at design load, the strands were cut off and the ducts were filled with secondary grout. The bearing plates and anchor heads were encased in non-shrink grout for corrosion protection.

Excavation was performed in three lifts. As soil was removed from the face of the caissons, two separate operations took place. The drainage board or PVC drainage pipes were placed and the steel shear studs were epoxied into holes drilled into the caissons. When the excavation was complete, the reinforcing steel for the fascia was tied and the facing was cast. The weep holes were cast through a New Jersey barrier which was slip-formed along the base of the wall.

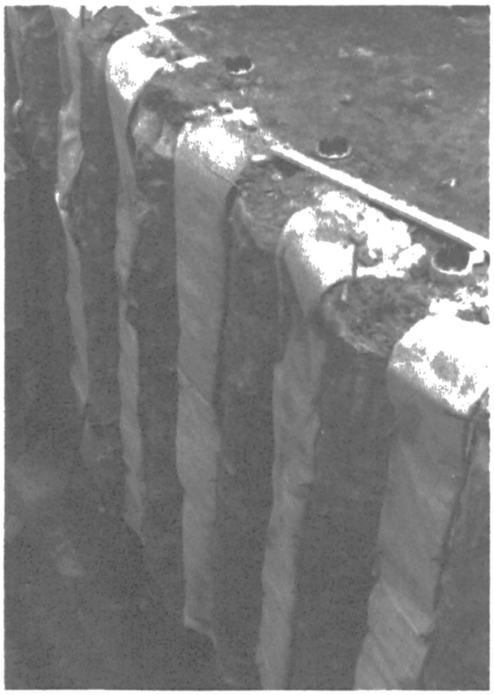

FIGURE 7. CAISSONS AND DRAINAGE BOARD

FIGURE 8. CONSTRUCTION OF CONCRETE FACING

FIGURE 9. COMPLETED RETAINING WALL

INSTRUMENTATION

An instrumentation program was implemented by the Georgia DOT to document the performance of the structure and to provide early warning of unexpected behavior. The instrumentation also provided a means to assess the adequacy of the design including checking design assumptions.

The instrumentation mainly consisted of slope indicator casings placed in four of the caissons. Caisson numbers 16, 28, 37, and 44 had steel pipes tied to the reinforcing steel cages and cast into the shafts. Inclinometer casings were then grouted inside the steel pipes. Also, optical survey measurements were performed at various locations along the wall to check for settlement and tilting.

Several inclinometer measurements were made at each caisson location to establish a baseline for the movements. These measurements were taken several weeks before stressing of the post-tensioning tendons or excavation in front of the wall. Three readings were also made on the day that the instrumented caissons were stressed. These included: 1) before stressing, 2) at 150 percent of design load, and 3) at design load. The instrumentation was read on a weekly basis before, during, and after construction for a total period of 9 months. The instruments were read monthly for a further period of 5 months. The next readings were made at 6 month and 1 year intervals. The instruments were last read in June 1987, more than 2 years after the completion of construction.

The inclinometer data provided the deflected shapes of the instrumented caissons. Plots of the deflected caissons are shown on Figures 10, 11, 12, and 13. The exaggerated horizontal scale of these figures show the evolution of the deflected profile at important milestones throughout construction and up to the most current readings.

The shafts responded to the post-tensioning by bowing backward towards the parking garage at the top and bulging outward at approximately mid-depth. The resulting shape was a smooth curve which looked like an archery bow. When the 20 foot excavation was performed to road grade in front of the wall, the released confinement allowed the tops of the caissons to bow backwards an additional amount towards the parking garage (especially where the caissons were not tangent.) The caissons then deflected and rotated outward in response to earth pressure after excavation. However, the post-tensioning forces were high enough to maintain the caissons in a permanently bowed configuration. Maximum outward deflections of 0.25 to 0.30 inches were measured approximately at roadway grade. The tops of the caissons are currently located either at their original installed positions or slightly more towards the parking garage than when first installed.

Approximately 55 to 65 percent of the outward movement of the caissons occurred within 8 weeks after excavation was complete. In general, these movements occurred uniformly, indicating a gradual application of earth pressure to the wall over an extended period of time.

Caisson 28 experienced a shift in the inclinometer casing at a depth of 37 feet, 12 days after stressing. This caisson contained the deepest rock socket (6 feet) on the project. Very seamy rock was encountered during the drilling of this caisson (a very hard zone was drilled immediately above a soft zone.) The inclinometer casing shift apparently followed the caisson's response to the application of the post-tensioning load in these subsurface conditions.

CONCLUSIONS

1. The design and construction of the wall provided a cost effective alternative to more traditional methods. The scheme used for this wall was advantageous because:

   a. The wall was built from the "top down." No temporary shoring was necessary because the caissons and rock anchor tendons were in place and functioning prior to the start of excavation.

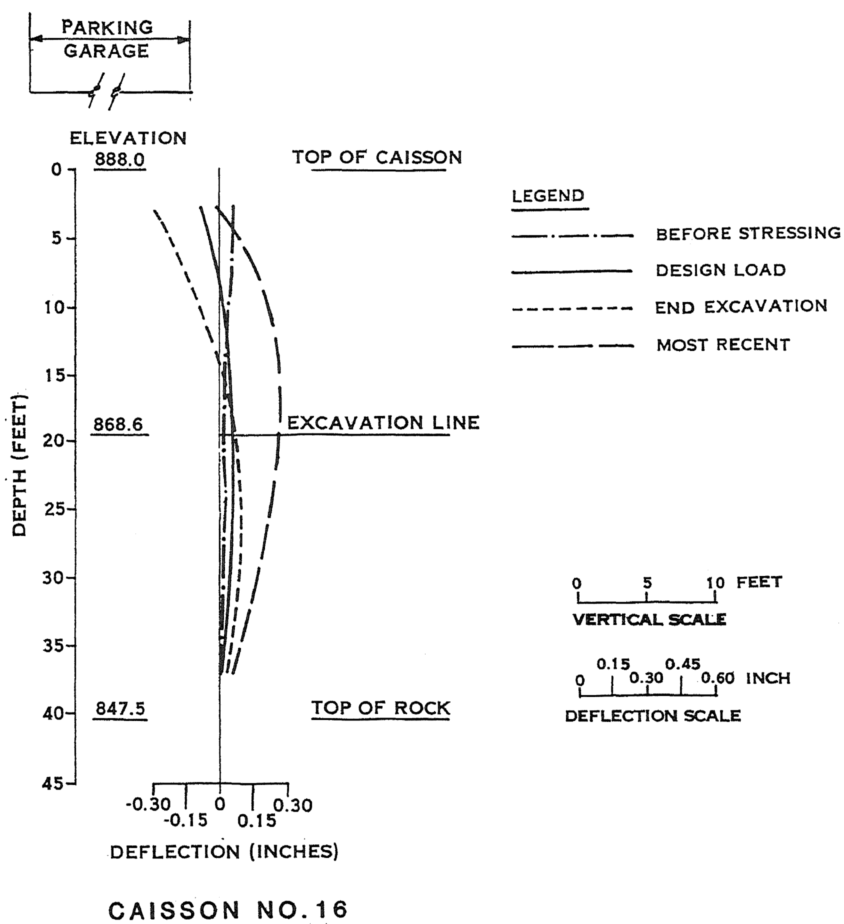

FIGURE 10. SLOPE INDICATOR RESULTS

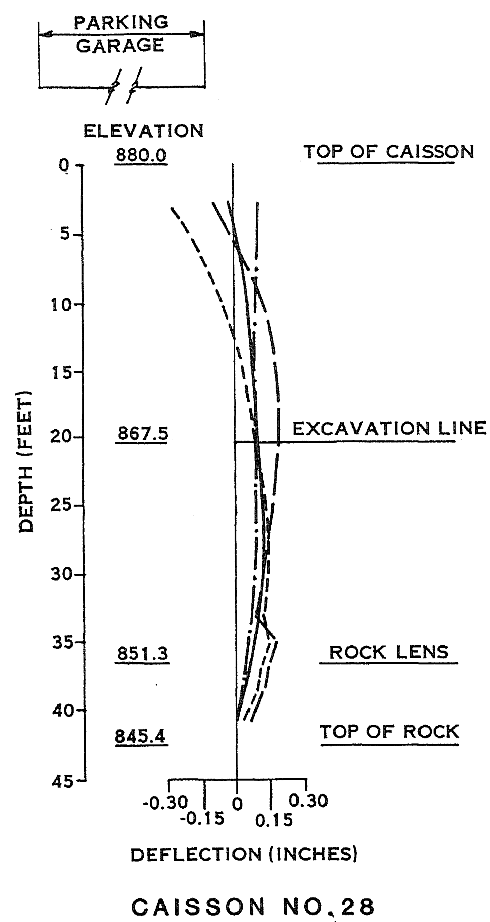

FIGURE 11. SLOPE INDICATOR RESULTS

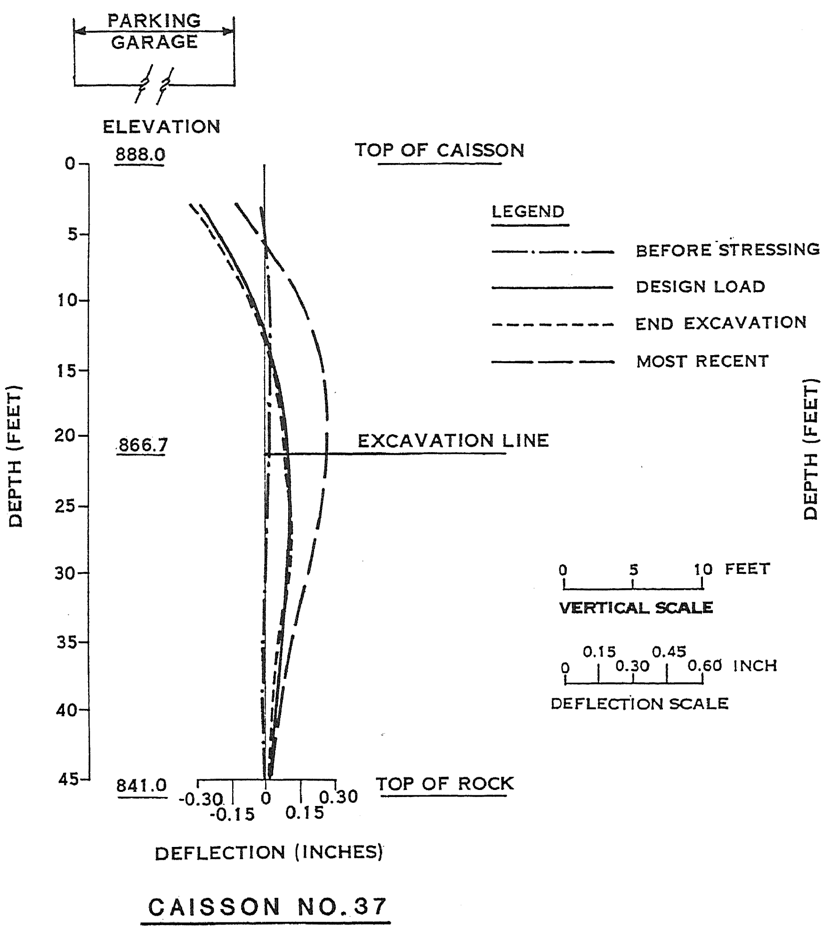

FIGURE 12. SLOPE INDICATOR RESULTS

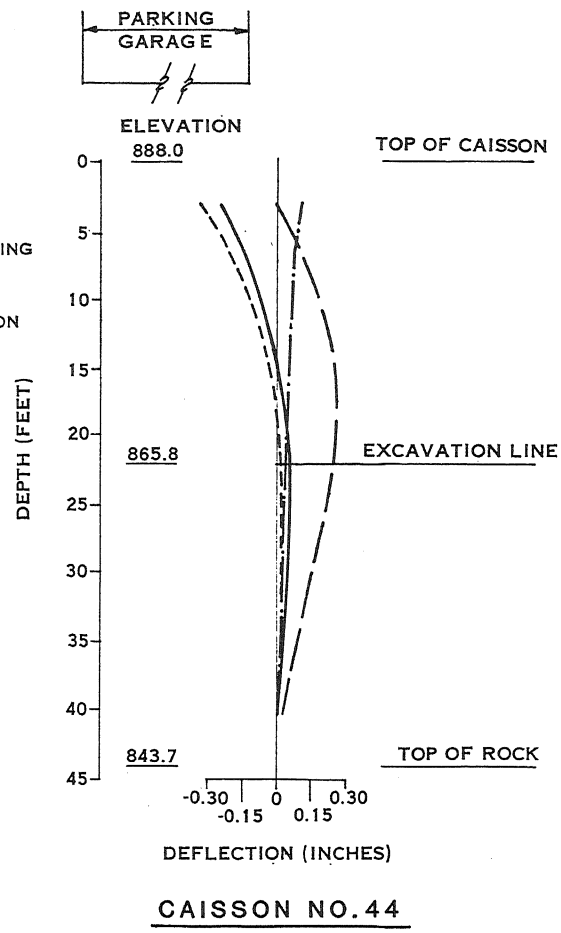

FIGURE 13. SLOPE INDICATOR RESULTS

b. It was not necessary to excavate 20 feet of the 40 foot cut originally planned. This was significant because a sensitive structure was located immediately behind the wall.

c. The rock anchor tendons did not extend behind the rear face of the wall. This feature is noteworthy in urban environments where it is often difficult to obtain easements under adjacent properties.

2. The wall did not deflect outward at the top as much as anticipated, although the pre-excavation response to post-tensioning was predicted accurately during design. The caissons responded to the post-tensioning by deflecting back towards the parking garage at the top; and by bulging outward at approximately mid-depth. The bowed shape became more pronounced after excavation was performed in front of the wall. The post-tensioning force was apparently large enough to retain the bowed shape throughout the project, even after the application of earth pressure. This locked-in shape was responsible for the reduced outward deflections observed at the tops of the caissons.

ACKNOWLEDGMENTS

The writers would like to thank the Georgia Department of Transportation for their foresight in the approval of this project, and for the excellent instrumentation data provided to us. Georgia DOT officials responsible for the project were Mr. Charles Lewis, State Bridge Engineer, and Mr. David Mitchell, Chief of the Geotechnical Branch. Structural design of the retaining wall was performed by Mr. David Shiver for Law/Geoconsult International.

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