Seismic Foundation Retrofit of West Stands of Michie Stadium

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SEISMIC FOUNDATION RETROFIT OF WEST STANDS OF MICHIE STADIUM

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ABSTRACT

The Michie Stadium is located at the U.S. Military Academy at West Point, New York. The West Stands of the stadium, consisting of upper and lower tier seating and other facilities, are supported on columns which are founded on shallow footings resting mainly on shallow bedrock. A structural seismic assessment of the West Stands indicated that uplift of the foundations would occur during a seismic event thereby causing significant distress to the superstructure. In order to prevent the uplift of the foundations and to provide base fixity, it was proposed to install rock anchors through the foundation. A supplementary subsurface investigation program consisting of borings and test pits was carried out to estimate the bedrock elevations and characterize the bedrock. In addition, full-scale anchor pull-out tests were performed to evaluate the bond-strength between the anchorage and rock interface. This paper will present the design of the anchorage system including the concrete reaction ring installed around the base of the column and the rock anchor system. The paper also includes the results of the full-scale anchor tests and modifications that had to be made to the design during construction due to unforeseen foundation conditions.

INTRODUCTION

The Michie Stadium is located at the U.S. Miltary Academy (USMA) at West Point, New York. The West Stands of the stadium are mainly founded on relatively shallow bedrock. The West Stands consist of upper and lower tier seating and other facilities. The West Stands are supported on a total of 64 columns founded on square spread footings generally 6 to 8 feet wide. A structural seismic assessment indicated that uplift of the foundations will occur during a seismic event thereby causing significant distress to the superstructure. In order to prevent the uplift of the foundations and to provide base fixity, it was proposed to install rock anchors through the foundation and into the bedrock. During the design stage, two other options were considered to limit the lateral movements of the columns. These options consisted of engineered soil fill above the footing and the use of drilled shafts installed around the column and connected to the existing columns through a heavy slab on grade to provide a reaction plane to limit lateral column movement. These options were not pursued due to the uncertainty involved in their effectiveness.

In order to estimate the bedrock elevation, strength and characteristics of bedrock, a supplementary subsurface investigation program was performed with ten geotechnical borings and four test pits. The subsurface information from the original borings performed in 1969 was also used. In addition, full-scale anchor pull-out tests were performed to evaluate the bond-strength between the anchorage and rock interface. The bedrock encountered at the site generally consists of gneiss that is typically fresh, hard, and very competent.

The anchorage system consisted of a concrete reaction ring around the base of the column with cement-grouted rock anchors installed through the foundation and anchored in the underlying rock. The design was performed to prevent failure of the anchor in the following modes: 1) yielding of the steel bolt, 2) shear failure along the interface between the anchorage and the rock, and 3) failure within the rock mass. A total of 402 rock anchors were installed at 64 column footings from February to August of 2001. Generally, 6 anchors were installed in each footing with an approximate spacing of 5 ft. and approximately 17 ft. of embedment into bedrock. In addition, one production anchor per column was tested for pull-out resistance. However, during the construction activities it was found that six columns were supported by footings founded either completely or partially on soil. At these locations, the portion of the anchor bolt between the foundation and the bedrock was encased in a steel casing.

SUBSURFACE CONDITIONS

The bedrock is generally present at depths ranging from 5 ft to 15 feet at the site. The two principal rock types encountered at the site are fine-to-medium-grained biotite/hornblende quartz gneiss and coarse grained quartz feldspathic gneissic granite. The majority of the cored rock was fresh, hard, and very competent. Overlying the bedrock are soils consisting of brown fine silty sands to fine to coarse silty sands, gravel and boulders.

EXISTING FOUNDATIONS

The West Stands are supported on a total of 64 circular columns. The column diameters are 3 ft and 4 ft. Some of the 4 ft diameter columns have a 5 ft diameter pedestal above the footing ranging in height from 5 to 10 ft. The columns are founded on spread footings that bear directly on bedrock. Based on the as-built drawings, the foundation dimensions are primarily 8' x 8' and 6' x 6', with some having dimensions of 7' x 7', 8' x 7', and 9' x 7'. The thickness of footings generally ranged between 3 and 7 feet.

SEISMIC RETROFIT OPTIONS FOR FOUNDATIONS

Three options were considered to restrain lateral movement of the columns or achieve fixity of the foundations. 1. Place compacted fill around the columns to restrain the movement of the columns and foundations under seismic loads to acceptable levels. 2. Install a set of drilled piers around the existing columns and construct a concrete slab at the ground level to connect the piers and the column. 3. Install rock anchors through the footing of the foundation to obtain fixity of the foundation and prevent uplift. The selected design option was the rock anchorage system.

Use of Engineered Fill to Restrain Column Movement

If the foundations are retrofitted such that they can be considered fixed, moments or equivalent lateral forces will be induced at the foundation level during a seismic event. It was estimated that the lateral force on the columns (at ground level) during a seismic event, will range from about 50 to 155 kips for a 10 ft soil cover and 33 to 103 kips for a 15 ft soil cover.

In this option, the effectiveness of using engineered fill to restrain movement of the columns was evaluated by studying the lateral deformation behavior of the columns under lateral loads applied at the top of a soil layer with varying thickness. The analyses were performed for column diameters of 3, 4, 5, 6, and 7 ft, lateral loads of 50 to 155 kips, soil thicknesses of 10, 15, and 20 ft, and 3 soil densities corresponding to dense, medium dense, and loose. The results indicated that the lateral deformation of the columns will be generally greater than 1.0 in. (assumed acceptable lateral displacement) for the cases considered. Considering the anticipated deformations associated with this option this option was not pursued further.

Use of Slab-On-Grade/Pier System

This option consisted of installing drilled piers around the foundation column and connecting the piers and columns through a slab-on-grade. The analyses were performed for 10 ft long piers having diameters of 3 ft and 4 ft, 3 different soil densities corresponding to dense, medium dense, and loose, and lateral loads ranging from 50 to 150 kips. Due to the uncertainty in the direction of seismic waves and the magnitude of the lateral load to be resisted, several piers would be needed around the column. The presence of boulders would also impede the construction of the drilled piers. Due to the constraints of the site and the various uncertainties regarding the response of this system to seismic loading, this option was not pursued further.

Rock Anchors

This option consisted of a concrete reaction ring around the base of each column with rock anchor installed through the ring and footing and anchored in the underlying rock. Because of the established behavior of rock anchor in rock, this option was recommended as a feasible design option. The following section provides specific design information regarding the use of rock anchors.

FULL-SCALE LOAD TEST PROGRAM

During the design stage a cement-grouted anchor and epoxy resin anchor were considered. A full-scale pull-out test program was designed to evaluate the bond strength between rock/cement grout and rock/resin interface. A total of eight anchors were installed and tested. All boreholes for the anchors were drilled using a Gardner Denver Air track rig to a depth of 15 ft below the top of the exposed bedrock. Prior to installing the anchor, the borehole was cleaned by introducing compressed air at the bottom of the borehole.

The spacing between test anchors varied from 4 ft to 7 ft. Four anchors were resin anchors and the remaining four were cement grouted anchors. The anchor rod selected consisted of 1-3/4 in. (nominal) dia., Grade 150 threadbar. The yield load of the bar is 320 kips and the ultimate load is 400 kips.

Since the ultimate bond strength of the anchorage was not known at the beginning of the pull-out test program, the anchorage length was varied from 3.5 to 8 ft with an intention of causing failure along the anchorage/rock interface and also to obtain an average bond strength. A center hole hydraulic jack was used to apply a maximum pull-out force of 360 kips.

Installation of Resin Anchors

The borehole diameter for these anchors was 2-1/4 in. (nominal). The resin cartridges used consisted of FOSROC slow setting resin. The resin cartridges were placed into the borehole and the anchor rod was then spun through the cartridges to the bottom of the borehole at about 100 revolution per minute. The anchorage length was varied by using 3 to 7 cartridges. The anchorage length could not be measured due to the very small annular space (about 1/8-in.) between the anchor rod and the borehole; therefore, the anchorage length was estimated from the fill chart supplied by the manufacturer and a field test.

Installation of Cement-Grouted Anchors

The borehole diameter for these anchors was 3 in. (nominal). The cement grout used had a water/cement ratio of 0.45. Cement grout was first tremied into the borehole and then the threadbar, with centralizers, was lowered into the borehole. The final measured grouted length varied from 5 to 7.4 ft. Samples of the grout were taken to estimate the strength over time. The average 7 day compressive strength (two specimens) was 1,550 psi. After 28 days the compressive strength was 2500 psi.

Pull-Out Tests

The pull-out tests were performed seven days after the cement grouted anchors were installed. A center-hole hydraulic jack was used to test the anchors to a maximum load of 360 kips.

Interpretation of Test Results

Figures 1 and 2 show the load vs displacement plot for an epoxy resin anchor and a cement-grouted anchor, respectively. The theoretical elongation of the threadbar free length shown on the figures was estimated based on the stress-strain curve for the threadbar supplied by the manufacturer. Comparing the measured values with the theoretical values indicates that these anchors did not fail by failure along the resin or cement/rock interface. The failure occurred by the yielding of the threadbar since the applied load was more than the yield load of the bar. None of the resin anchors tested failed along the resin/rock interface. However, it is believed that three of the cement-grouted anchors had failure of the grout. A distinct popping sound was heard for these tests immediately prior to the load dropping. It is believed that this was the grout failing at the grout/rock or grout/bar interface or within the grout itself. Figure 3 shows the load-displacement plot for a cement-grouted anchor with failure of the grout.

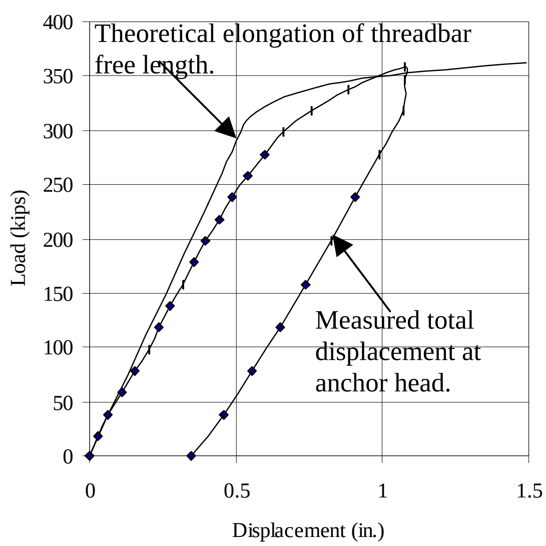

Fig.1. Load Vs Displacement of Resin Anchor B-1.

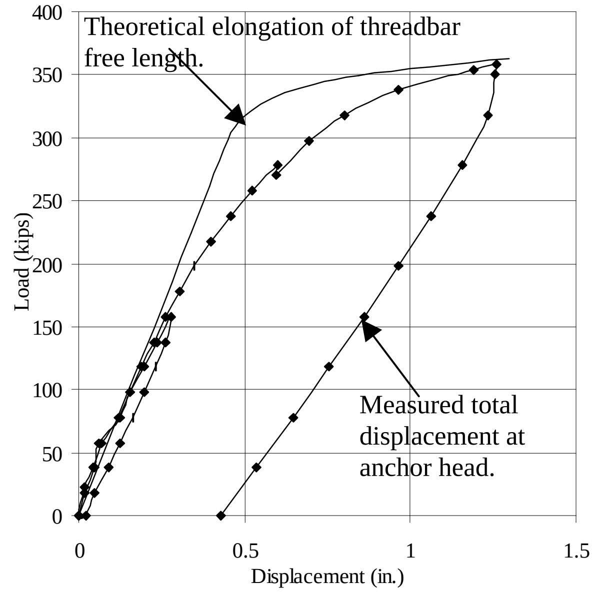

Fig. 2. Load Vs Displacement of Cement-Grout Anchor B-8.

The bond strength at failure or at the maximum load sustained by the anchor was calculated assuming the bond stress is uniform along the anchorage length. A summary of the results of pull-out tests and the bond strength estimated for each anchor is presented below.

Table 1. Summary of Results of Anchor Pull-Out Tests

Bolt No.	Average Borehole Dia. (in.)	Anchor Length (ft)	Free Len. (ft)	Failure Load (kips)	Min. Estimated Bond Strength at Failure Load (psi)
Resin					
B-1	2.32	6	10.8	360	686
B-2	2.32	8	8.7	360	515
B-3	2.32	3.5	13.3	360	1176
B-4	2.32	6	10.6	360	686
Cement Grout					
B-5	3.2	7	9.6	338	400
B-6	3.2	5	11.8	318	527
B-7	3.2	6	10.6	278	384
B-8	3.2	7.4	9.4	358	401

Based on these results, average ultimate bond strengths of 700 and 400 psi for resin and cement-grouted anchors, respectively, were recommended. Considering the increase in the cement grout strength from 7 days to 28 days, it is likely that the actual

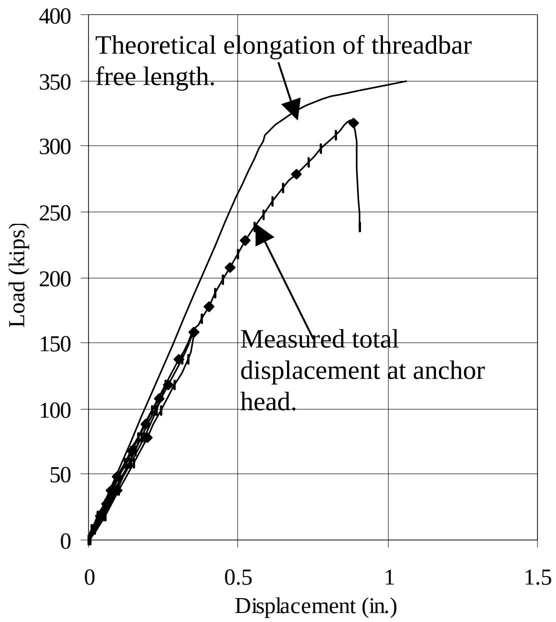

Fig. 3. Load Vs Displacement of Cement-Grout Anchor B-6.

bond strength of the cement grout anchors will be greater than those given in Table 1.

Due to the limited headroom at the foundation locations, some anchors had to be installed in short sections with the use of couplers. These couplers require a borehole diameter that exceeds the maximum allowable borehole diameter for the use of epoxy resin. Considering this issue, the final design was based on the use of cement-grouted rock anchors only.

ROCK ANCHOR DESIGN CONSIDERATIONS

Design Approach

The design was performed to prevent failure of the anchor in the following modes: 1) yielding of the steel bolt, 2) shear failure along the interface between the anchorage and the rock, and 3) failure within the rock mass. Shear failure between the anchorage grout and the steel bolt was not considered likely due to the irregular surface character of the "deformed" steel rock bolts.

An anchor design was developed to optimize the number of anchors required for each column/footing based on reaction uplift force to be resisted (948 to 1422 kips). As per PTI (1996), for permanent anchors, the design load should be not more than 60% of the specified minimum tensile strength of the bar (this corresponds to a factor of safety of 1.67). Considering that the rock anchors will not be under a constant load but will be subjected to very infrequent, short duration loads during the life of the structure, a factor of safety of 1.25 (corresponding to 80% of the ultimate strength of the steel) was considered as acceptable.

The anchorage (or bond) length was estimated from the ultimate bond strength between the grout and rock and the surface area of the grouted anchorage. A factor of safety of 1.25 on the ultimate bond strength was used in development of the anchorage length.

The potential for failure occurring within the rock mass was investigated by assuming a conical failure envelope with its apex located at the bottom of the anchor. The analyses indicate that failure within the rock mass does not control the design (see section on Group Effects).

Anchorage (Bond) Lengths

The bolt diameter is initially selected to ensure that the design load of the anchor is less than or equal to the ultimate capacity of the anchor divided by a desired factor of safety. The minimum length of the cement-grouted anchor was then determined based on the bolt design capacity and the bond strength between the grout and the rock, and the borehole diameter. The borehole diameter was varied depending on the diameter of the bolts used.

Table 2 presents the minimum required anchorage lengths for 1.25 and 1.75 in. diameter bolts. The minimum required anchorage length is based on applying a factor of safety of 1.25 to the recommended ultimate bond strength (400 psi) of the cement grout interface. A design anchor (embedment) length of 15 ft was used based on experience and the practicality of installing anchors. The anchors were grouted along the entire length. For the anchors that were performance tested, a plastic sleeve was installed along the free length (5 ft). It was recommended that a minimum of six anchors be installed for each reaction ring for uniformity of load distribution and to compensate for directional impact of seismic waves.

Table 2. Anchorage (bond) Lengths

Bolt Dia. (in.)	80% Ult. Strength of 150 ksi bolts (kips)	Borehole Dia. (in)	Min. Required Anchorage Length (FOS=1.25) (ft)
1.25	150	2.25	6
1.75	320	3.5	8

Group Effects

The potential for failure occurring within the rock mass was investigated by assuming a conical failure envelope with an apex angle of 90 degrees and a total of up to eight rock bolts. For calculation purposes, the cohesion intercept of intact rock was conservatively assumed to be 400 psi. This value was conservatively based on unconfined compressive strength tests (URSGWC, 2000) performed on intact rock samples, visual observation and typical values (Goodman, 1989) for Schistose Gneiss. The results of the analysis indicate that for the embedment lengths recommended above, the factor of safety against group failure due to shear through the competent rock mass is over 20 (the USACE, 1994 recommended factor of safety is 4.0). Therefore, failure occurring within the rock mass does not control the design of the rock anchors.

UNFORESEEN FOUNDATION CONDITIONS

Based on documentation from the original construction of the West Stands and test pit observations, the rock anchors were designed assuming that the footings are founded on bedrock. During the foundation retrofit construction, it was found that some footings were not resting directly on bedrock. In order to over come this problem, the portion of the anchor bolt between the foundation and the bedrock was encased in 10 ft long steel casing (4.5 in. O.D.) driven into bedrock. The steel casing had pairs of $^5/_8$-inch holes drilled into it at 6-inch intervals to facilitate grouting of any potential loose foundation soils while installing the anchor. After the steel casings were installed into the borings, 1-$^3/_4$ inch diameter anchors were centered in the casing and pressure grouted. If the inflow of water and sand into the casing obstructed the insertion of rock-anchor, then pressurized grout was initially injected through the $^5/_8$-inch diameter holes and out into the surrounding soil. This was performed to stabilize the surrounding soil and eventually seal the openings in the steel casing once the grout hardens. Then hole was reamed and cleaned to install the rock-anchor.

INSTALLATION AND TESTING OF ANCHORS

A total of 402 rock anchors were installed at 64 column footings from February 2001 to August 2001 (URS, 2001). The contractor opted to install all anchors in 4-inch diameter boreholes. In addition, the borehole length in rock was increased to 17 ft to account for any potential loss of bond length at the bottom of the borehole due to deposit of sediments from inflow of groundwater into the borehole. Installation activities included drilling of boreholes, installation of anchors, pressure grouting, performance testing, construction of concrete reaction rings and protective end-caps on rock-anchors.

As per the design requirement performance tests were performed on 10% of production anchors, generally one anchor per column. These tests were performed 7 days after the installation and grouting of anchors. Performance tests involved loading and unloading the anchor in cycles to ¼, ½, ¾ and 1.0 of design load. At the end of the 4th cycle (after reaching design load), creep test portion of the performance test was carried out. During the creep test portion movement of anchor was measured every minute for 10 minutes duration. The conditions of the proof tests were: 1) maximum elongation (creep) of anchor is 0.004 inches for 10 minutes at 1.0 of design load (end of 4th cycl4), 2) elongation of anchor of anchor had to be between 80% and 100% of the theoretical elongation of the free length of the bar. A total of 63 anchors were performance tested, and 10 min. creep elongation was generally less than 0.001 in. and the average elongation was 97% of theoretical elongation.

Since the anchors were not prestressed, proof tests were not performed. After the completion of the performance tests, an additional length of epoxy coated steel anchor was attached before the construction of concrete reaction ring. Finally, the anchors were provided with necessary end hardware to lock them at a load of 12 kips and to protect against the corrosion.

CONCLUSION

The bedrock encountered at the site was generally competent and based on the conservatively assumed cohesion intercept of intact rock, it was found that failure occurring within the rock mass does not control the design of anchors. The results of pull-out tests indicated that the rock-to-anchor bond strength (ultimate) can be 400 psi for cement-grouted anchors and 700 psi for resin anchors. All the performance tests conducted satisfied the design criteria indicating that about 12 ft long bond length was adequate.

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