Aspects of Rock Anchors for the Sydney International Athletics Centre

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ABSTRACT: High capacity prestressed rock anchors have been successfully installed in Ashfield Shale to support the grandstand roof of the Sydney International Athletics Centre as part of the NSW Public Works Olympics 2000 Project at Homebush Bay. Testing of a completed anchor proved an anchor capacity with a bond stress of 1000kPa. This paper describes various geotechnical aspects involving the design, installation, inspection, and testing of these anchors.

1 INTRODUCTION

The NSW Public Works Olympics 2000 Project involves the construction of several world class sporting venues at Homebush Bay in Sydney, Australia. The Sydney International Athletics Centre is currently nearing completion, and will be used as a warm-up facility for the main Olympic stadium, which is yet to be constructed.

The western side of the grandstand is covered by a suspended roof. The roof is tensioned by cables attached to catenary anchors outside the stadium. Two tall masts support the roof at several locations and are supported by cables attached to backstay anchors, as shown in Figure 1.

The anchors are pretensioned with their bond length in Class I or II Shale (using the rock classification scheme proposed by Pells et. al., 1978) and have inclinations from the vertical of between 15° and 34°. Average design working bond stresses range are approximately 680kPa. Corrosion protection was provided by a corrugated high density polyethylene (HDPE) sheath. Strands in the free length were individually sheathed in a HDPE tube.

This paper discusses the following aspects of the rock anchors:

- inspection of anchor surface block excavations
- water testing and visual inspection of anchor holes for rock quality and cleanliness
- review of design following discovery of sheared zone in bond length
- suitability testing of one of the anchors to check design assumptions and parameters, and to demonstrate the effectiveness of the construction procedure.

\- Where problems were encountered they are discussed and recommendations for alternative methods provided where appropriate.

Design of the anchors was initially performed by Austress PSC Australia Pty Ltd, with Coffey Partners International Pty Ltd (CPI) producing the original geotechnical investigation, contributing to the specification, providing design parameters and performing design checking (Coffey Partners, 1993). Initial design of the anchors is not considered in this paper.

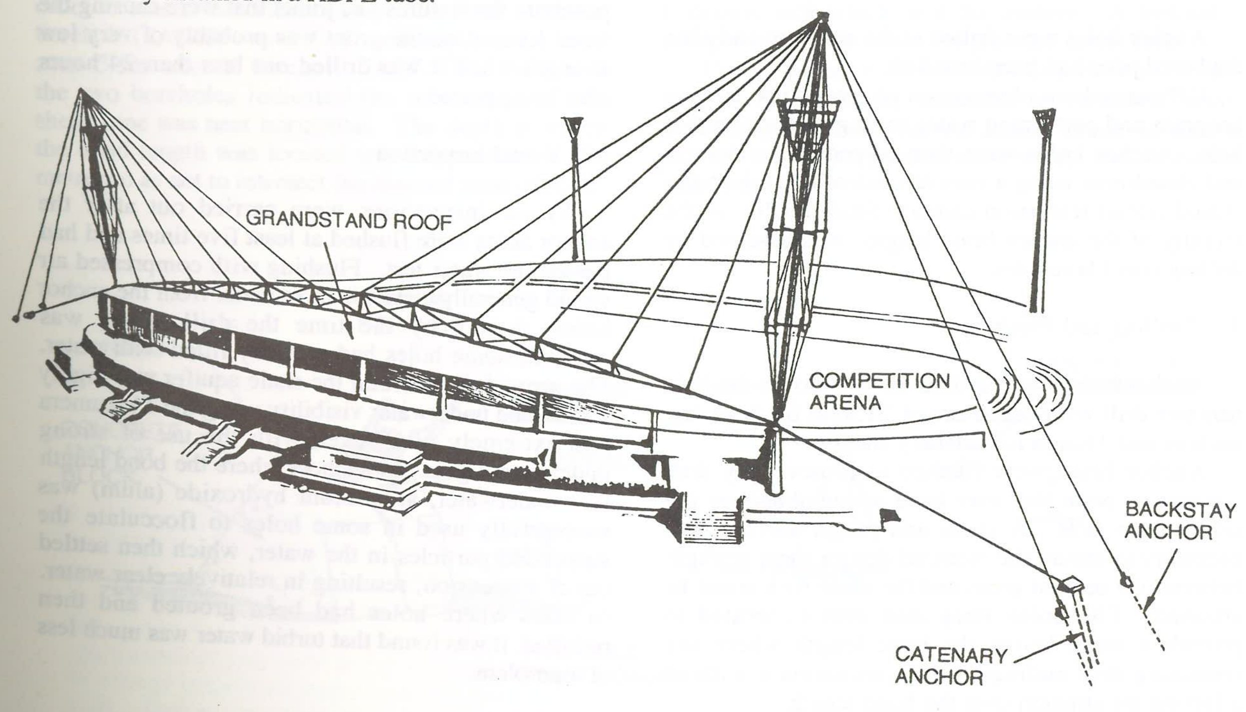

Figure 1. General arrangement of the grandstand roof, superstructure, and rock anchors.

2 INSPECTION OF ANCHOR BLOCK EXCAVATIONS

All the rock anchors were prestressed. Strands from the anchor were tensioned relative to a large concrete block on the surface and locked off.

At the southern end of the stadium Class I or II Shale occurs very close to the surface, requiring only shallow excavations for the concrete block foundations. However at the northern end the shale was much deeper, with an overlying 12m thick layer of controlled fill. To allow prestressing of the anchors without excessive movement of the surface block it was necessary to found the blocks through the fill onto the shale using bored piles.

Holes for the bored piles were 1.2m to 1.5m in diameter, and were excavated into Class II shale. Cored boreholes were drilled at each location to assess Shale quality. Rock strength was confirmed during excavation of the bored pile holes from drill cuttings brought to the surface. Cleanliness was assessed visually prior to placing of the reinforcing cage. The base of the hole was illuminated with the aid of a mirror or electric light lowered to the base of the hole.

The reinforcing cage was constructed around a wide steel tube through which the anchor would later pass. Difficulty was experienced in manoeuvring the 12m long cage down the inclined hole, and some fall-in of the compacted fill was experienced. Wheels had been placed along the reinforcing cage, though these did not roll and became bogged in the sides of the hole.

If the bored pile hole had been cased, allowing access to the base of the hole, it may have resulted in better assessment of rock quality and easier placement of the reinforcing cage.

3 ASSESSMENT OF ANCHOR HOLES

Anchor holes were drilled at the northern end after the bored piles had been installed.

CPI carried out observation of the anchor drilling program and performed water testing of each anchor hole. Anchor holes were then inspected for defects and cleanliness using a remote controlled underwater closed circuit television camera. Shale quality in the vicinity of the anchor bond length was assessed by drilling cored boreholes.

3.1 Drilling and Flushing

Anchor holes were drilled using a down-the-hole hammer drill with diameters of 215mm for backstay anchors and 330mm for catenary anchors.

Anchor holes were flushed to remove any drill cuttings or paste that may have accumulated on the sides of the hole. A clean and rough surface was necessary to ensure the required design shear strength between the cement grout and the shale rock could be attained. The holes were also over-excavated to provide a sump below the bond length where any remaining drill cuttings could accumulate without affecting the strength over the bond length.

Flushing was carried out by filling the anchor holes with water then rapidly blowing compressed air into the base of the hole through the drill string, causing the water to spurt from the top of the hole to greater than twice the height of the drill mast.

Air bubbles were observed at some of the northern anchors around the top of anchor holes when being flushed with compressed air. This is probably due to the high pressure air forcing its way through small openings in the side of the hole, particularly at the shale-concrete interface at the base of the bored pile. The compressed air may have enlarged any such opening, resulting in increased water leakage from the anchor hole.

The only anchors to fail the water test and require the expense and delay of grouting were at the northern end, possibly as a result of opening of defects from compressed air. A less destructive method of cleaning the sides of the holes may prevent this problem. A rotating brush or water jet may prove to be a better cleaning method.

4.2 Water Testing

Water testing was performed at each anchor hole to ensure that grout would not escape from around the bond zone. The water test required that the volume of water that could be lost from each anchor be less than 6×10^{-4} litres per millimetre diameter per metre length per minute. Thus for a typical catenary anchor with a diameter of 330mm and a length (in rock) of 14m, the allowable water loss is 28 litres over a 10 minute period. Only three of the anchors failed the water test and required grouting.

In one instance an anchor failed the water test a second time after grouting. A similar grout mix to the final mix had been used. This mix was probably too stiff, resulting in the grout not being able to fully penetrate the fissures and joints that were causing the water loss. Also the grout was probably of very low strength when it was drilled out less than 24 hours after being placed.

3.3 Visual Inspection

Visual inspections were carried out after the anchor holes were flushed at least five times and had passed the water test. Flushing with compressed air would generally evacuate most water from the anchor holes, though by the time the drill string was removed some holes had partially filled with water. The groundwater within the shale aquifer was highly turbid, and underwater visibility using the TV camera was extremely poor, even with the use of strong underwater lights. In anchors where the bond length was underwater, aluminium hydroxide (alum) was successfully used in some holes to flocculate the suspended particles in the water, which then settled out of suspension, resulting in relatively clear water. In cases where holes had been grouted and then redrilled, it was found that turbid water was much less of a problem.

The TV camera was encased in a cylindrical steel cage from which a cable was attached to raise and lower it manually from the surface. Underwater torches were used to provide illumination. The camera was connected to a video recorder, television screen and a remote control box on the surface. The camera lens was protected by a hemisphere of clear perspex, and the lens could be remotely manoeuvred to point in any direction out of the hemisphere.

The camera was lowered into the anchor holes one metre at a time, with the camera operator inspecting all sides of the hole at each depth by moving the camera lens around. Logging of the holes was carried out by talking into a microphone connected to the video recorder with the video image being recorded simultaneously. In most instances the shale sides of the hole appeared as closely spaced rounded ridges approximately 1 to 3mm high. The sides of the hole were clean on the occasion of each inspection. In one of the anchor holes water was observed to gush from a joint in the rock.

Although the aluminium hydroxide proved very successful in some holes to clear the water, it sometimes did not work at all, and required time to take effect. A better method would be to remove the water from the hole completely using a submersible pump. A suitable pump would be a "screamer" type electric pump, since a relatively high lifting capacity is required.

3.4 Cored Boreholes

Cored boreholes were drilled to assess the rock quality in the vicinity of the bond length. Borehole SAS7 was drilled vertically to 25.5m depth near the bond length at the southern catenary anchor.

This borehole encountered a zone of shearing in the Class I/II shale some 1.5m wide at 22m depth. In order to assess the orientation of this zone a second borehole (SAS10) was drilled vertically into the bond zone 5.5m west of borehole SAS7. The results of the two boreholes indicated the orientation of the shear zone was near horizontal. The depth at which the bond length was located was adjusted by a few metres so as not to intersect the sheared zone of rock.

4 EFFECT OF SHEARED ZONE ON ANCHOR DESIGN

Cored boreholes were drilled to assess the rock quality in the vicinity of the bond length. Borehole SAS7 was drilled vertically to 25.5m depth near the bond length at the southern catenary anchor.

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4 SUITABILITY TEST OF BACKSTAY ANCHOR

4.1 Method

The Suitability Test is a comprehensive incremental load test carried out to check anchor design assumptions and parameters, and to demonstrate the effectiveness of the installation procedure. The main emphasis in this particular test was to check the bond stresses that could be attained in Sydney shale, since no comparable anchors had been previously constructed (Coffey Partners, 1993).

The methodology of the suitability test was very similar to that specified in the Model Specification for Prestressed Ground Anchors (Brian-Boys and Howells, 1984).

The load applied to the tendons was measured using a calibrated pressure gauge connected to the hydraulic jack which held the tendons. A hydraulic pump was used to adjust the pressure in the jack.

The extension of the tendons was measured by dial gauges magnetically attached to threaded bolts protruding from the concrete anchor block, as shown in Figure 2. Two gauges were used, one on either side of the jack.

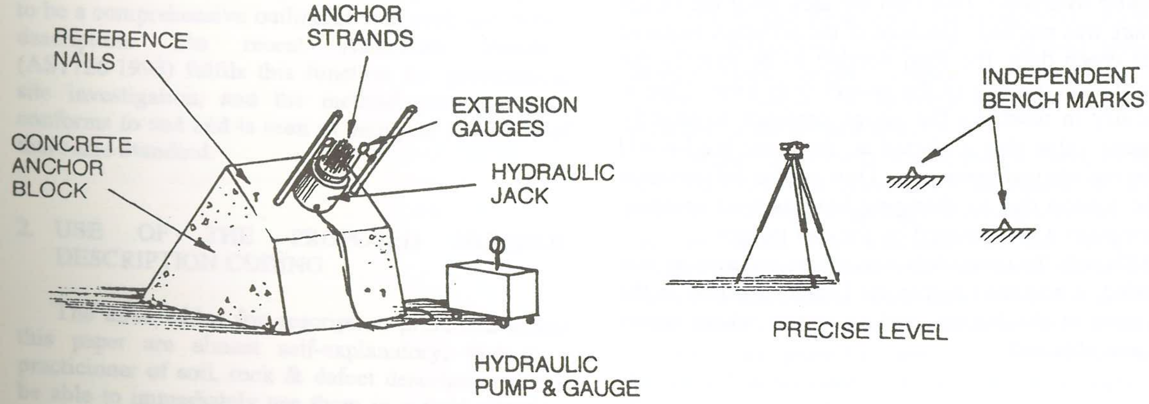

Figure 2. Suitability test apparatus

The deflection of the concrete block under load was measured by a surveyor using a precise level. This level can measure to a nominal accuracy of tenths of a millimetre. Two reference nails were hammered into the concrete on either side of the block. The deflection of the block was measured relative to two other datum points on the grandstand structure away from the anchor site.

The actual extension of the tendons could then be determined by subtracting the inclined component of the concrete block deformation from the tendon extension measured by the dial gauges. Movements of the anchor block reported by the surveyors were less than 1mm vertically, which were considered negligible.

4.2 Results

The anchor was loaded in nine incremental load cycles up to a maximum load of 5400kN, which corresponds to a bond stress of 1000kPa.

All test requirements were met, except for the first loading cycle where the criteria requiring the reload slope to be at least 90% of the unloading slope on the stress-strain plot was not met. This was more than likely due to draw-in of the wedges in the stressing block, since it did not occur later in the test.

There were no indications of the capacity of the anchor having been approached, however the anchor load could not be increased further since the stress in the steel strands was at 83% of its ultimate tensile strength, which is the maximum allowed.

4.3 Discussion

The measurement of creep deformation of the anchor required accurate measurement of deformation over a period of time at a constant load. For the first few loading cycles the deformation occurred in the opposite direction to that anticipated. This was due to slight leakage of hydraulic fluid out of the jack and back through the valve of the hydraulic pump. It occurred quite slowly and resulted in a slight decrease of load over time. Thus the tendons shortened when the load was decreased resulting in "negative" creep measurements.

For subsequent load cycles the jack pressure was "topped up" during the constant load period by pumping hydraulic fluid into the jack until the initial pressure was reached. Because of the accuracy required of the creep data, the load needed to be exactly the same at each reading of the gauges over time. Due to difficulty in resetting the gauge pressure to exactly the same value that it started at, the same load could not be maintained precisely. Thus elastic deformation of the tendon due to changing load as well as creep deformation was measured by the dial gauges.

Although the creep data was not as accurate as was intended, it was well within the limits specified in the test, even at the higher loading stages where better data was obtained.

5 CONCLUSIONS

Logging and inspection of the anchor holes was successfully performed using a remotely controlled closed circuit television camera.

The suitability test proved an anchor capacity of 1000kPa bond stress under the test conditions. The anchor appeared capable of sustaining working bond stresses in excess of 1000kPa.

ACKNOWLEDGMENTS

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