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Foundations for the roof support arch for Durban's 2010 World Cup stadium

AN ARCH THREE-QUARTERS of the length and height of the main span of the iconic Sydney Harbour Bridge will be the most visible feature of Durban's new Moses Mabhida stadium. At time of writing, the foundations had been constructed and erection of the arch was imminent.

The 380 m long arch will provide a frame for the harp-like array of cable-stays supporting the grandstand roof. The wishbone-shaped arch has three springings, one at the northern end and two at the southern end of the major axis of the stadium.

Because of the tension forces in the cables, the design loads are exceptionally large for such a slender structure. To resist horizontal loads of up to 100 000 kN, with minimal deflections, the foundations are 20 m deep shear-walls socketed into bedrock.

SITE GEOLOGY

The site in King's Park is quite flat at a level of about 4 m above mean sea level, except where raised by filling. Groundwater is a metre deep, or less after rain. Slightly clayey silty fine sands known locally as the Harbour Beds extend down to a relatively flat erosion surface on Cretaceous siltstone at 18 m to 20 m depth.

Hard rock boulders scattered towards the base of the Harbour Beds did not influence the design but were a nuisance to Esor, who constructed the diaphragm walls. The siltstone, beneath a thin (less than 0,5 m thick) mantle of stiff residual clay, comprises inter-bedded thin, or very thin, layers of very-soft (UCS = 1 MPa to 3 MPa), soft (3 MPa to 10 MPa) and harder (>10 MPa) rock. Exploratory boreholes drilled a few metres into the siltstone did not reach the underlying Dwyka tillite.

THE DIAPHRAGM WALLS

The foundations are elongated rectangular boxes with 0,8 m thick reinforced-concrete walls. The northern foundation is 44 m by 7 m in plan and the two southern foundations are each 30 m by 4 m. The walls were excavated and cast under bentonite slurry (drilling mud) in panel lengths of up to 7 m. Alternate (primary) panels were cast as independent barrettes with tubular stop-ends to form concave half-round shear keys. In excavating the infilling (secondary) panels, after removal of the stop-ends, the revealed concrete surfaces were scraped by the bladed shoulders of the excavator grab-tool. Some bonding of the concrete is expected to occur between panels but there is no reinforcing across the (vertical) construction joints.

In addition to vertical and horizontal reinforcement each wall panel contains four or more vertical multi-strand post-tensioning cables.

The vertical reinforcing is fully bonded into, and the strands extend through, the deep (3 m on the south foundations, 5 m on the north) capping beams, which are also post-tensioned.

Each wall-panel is embedded at least 0,5 m into siltstone with a compressive strength of 2 MPa or stronger. These rock sockets have been ignored in assessing resistance of the foundations to sliding and over-turning, but were considered necessary to ensure adequate bearing capacity.

STABILITY ANALYSIS

The critical loads imposed on the foundations were determined through structural analysis of the roof for a large number of load cases including wind, cable prestress, self weight, maintenance loads, temperature effects and imposed loads.Many of the design load cases (approaching one hundred in total) related to the staged construction process for the slender arch. Because the dominant loads will be cable forces, controlled by the erector, a load factor of 35 was adopted. However, the resultant loads were multiplied by an 'analysis uncertainty factor' of 1,1 so that the 'global' load factor was slightly less than 1,5.

As shown graphically in figure 4, for most of the load cases the in-plane horizontal component is proportional to the vertical component. This is because the

1 The Durban Stadium arch and foundations silhouetted on a view of Sydney Harbour Bridge
2 Twenty tonne reinforcing cage
3 Key plan showing topology and foundation layout of the stadium arch and roof
4 Comparison of in-plane horizontal and vertical components of load
5 Summary of heaviest load components

arch has been designed to be predominantly in compression at all times.

The load case with the largest horizontal (destabilising) component is summarised on the load diagram of figure 5.

In-depth consideration was given to finding an appropriate method for proving the stability of the foundations with regard to rotational (over-turning) and translational (sliding) stability. Consensus was eventually reached within the design team on lateral earth pressure coefficients and reduction factors appropriate to disturbing (active) forces and restraining (passive) forces.

- Because lateral deflections of a few centimetres will influence the structural behaviour of the arch, passive pressure coefficients were clearly irrelevant and were replaced by 'at-rest' pressure coefficients
- Active pressures were determined in accordance with conventional earth pressure theory
- In assessing the potential for sliding, the cohesive components of strength of both the Harbour Beds and the Cretaceous siltstone were ignored
- The weight of soil contained within the diaphragm-wall box was taken into account with regard to sliding, but excluded from the assessment of overturning

Guidance by Peter Day, of Jones and Wagener, on how to present the analysis in a manner compliant with Eurocode EN7 and SANS 10160 (now in draft) is gratefully acknowledged.

CALCULATION OF DEFLECTIONS

Analytical method

The load-deflection behaviour of the foundations was analysed using PLAXIS 3D FOUNDATIONS[1] version 1.5 (current at January 2007) which applies finite element elasto-plastic techniques for calculation of soil–structure interactions.

Strength parameters

PLAXIS is a geotechnical (not a structural) design package and has limitations with regard to the modelling of reinforced concrete. The post-tensioned pile-cap was modelled as a linear-elastic (no crack) material.

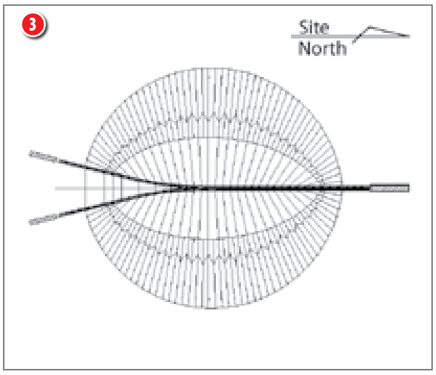
The reinforced concrete of the diaphragm wall panels was defined to comply with Mohr Coulomb failure criteria expressed as a shear strength of 20 MPa (zero friction angle) and a tensile strength of 5 MPa. The tension cut-off was introduced to limit unrealistically high stresses predicted to occur in the bottom corner of each panel, as can be seen towards the bottom of figure 4.

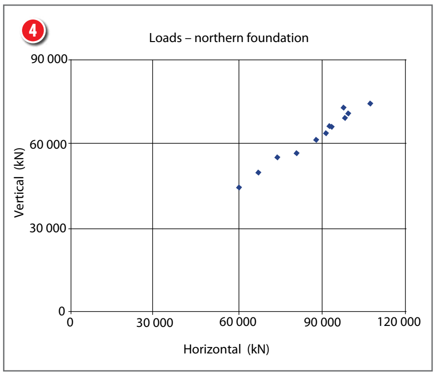
The construction joints between diaphragm wall panels were modelled as thin inter-face elements with a fraction (in this case 1 %) of the strength of the parent material.

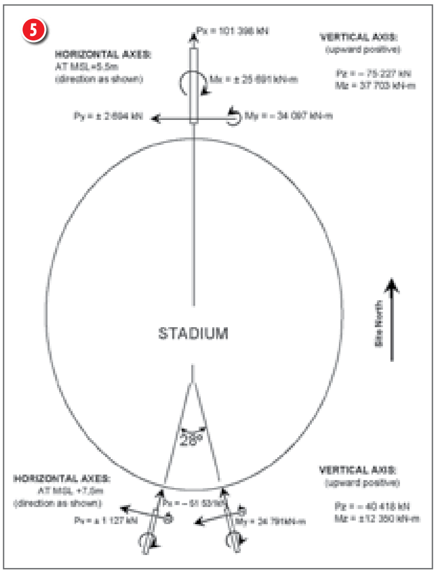
Elastic parameters

The elastic modulus of the Harbour Beds was assumed, as proposed by Stroud (1989), to be the characteristic standard penetration test 'N' value multiplied by 0,8 MPa.

No published data could be found on the elastic properties of the local Cretaceous siltstone. Guidance was sought from a Ciria report by Gannon et al (1999) and Tony Brink's *Engineering geology of Southern Africa* (Brink 1983). It was concluded that the value of Young's modulus is likely to be at least 160 times the UCS.







STRUCTURAL ANALYSIS

PLAXIS does not have element types to model the reinforcing in concrete, but the forces induced in the reinforcing were assessed by multiplying the area under consideration by the stresses calculated to act on that area. Zones of high stress were reviewed to confirm that plastic yield (crushing or shearing) of the concrete would not occur except, as allowed for, in the construction joints between adjacent panels.

Further analysis was done using PROKON[2] (both frame analysis and plate analysis) and STRAP[3] (structural analysis package). STRAP was used for the composite design of the reinforcing and post-tensioning in accordance with TMH 7 Part 3 (1989), 'Code of Practice for the Design of Highway Bridges and Culverts in South Africa'.

PROKON provided a useful link between STRAP and PLAXIS to demonstrate strain compatibility between the structural and geotechnical analyses.

CONCLUSIONS

The deflection tolerances of a few centimetres under enormous lateral loads presented an interesting design challenge. There remain areas of uncertainty, such as the degree of interlock between adjacent diaphragm wall panels and creep deflections of the foundations under sustained load. To deal with these uncertainties hydraulic jacks will be built into the arch springings and deflections will be carefully monitored during and after construction of the arch. It is hoped that these measurements and their influence on the construction and maintenance of the arch will be discussed in a sequel to this article.

Notes

1 See http://www.plaxis.com/.
2 See http://www.prokon.com/.
3 See http://www.gtscad.com/strap.htm.

References

Brink, A B A 1983. *The engineering geology of South Africa*, Volume 3.
Gannon et al 1999. *Piled foundations in weak rock*. London: Ciria (Report 181) (reprinted in 2004).
Stroud, M A 1989. The standard penetration test, its application and interpretation. *Proceedings* of ICE Conference, Birmingham.

6 PROKON output showing deflected shape of diaphragm wall

7 PLAXIS output showing vertical stress distributions in diaphragm-wall

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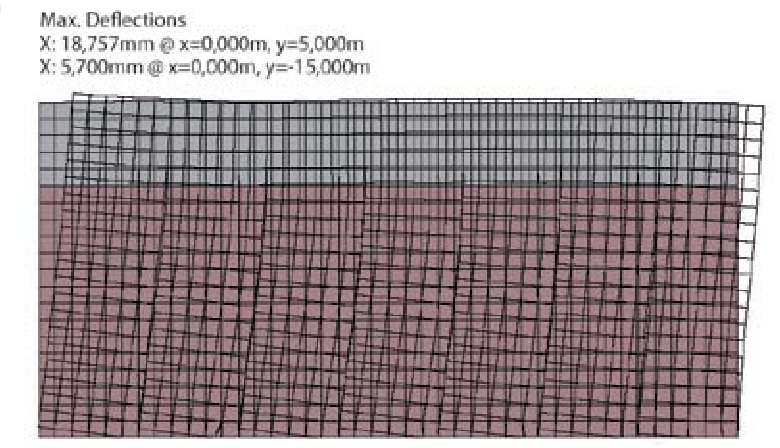

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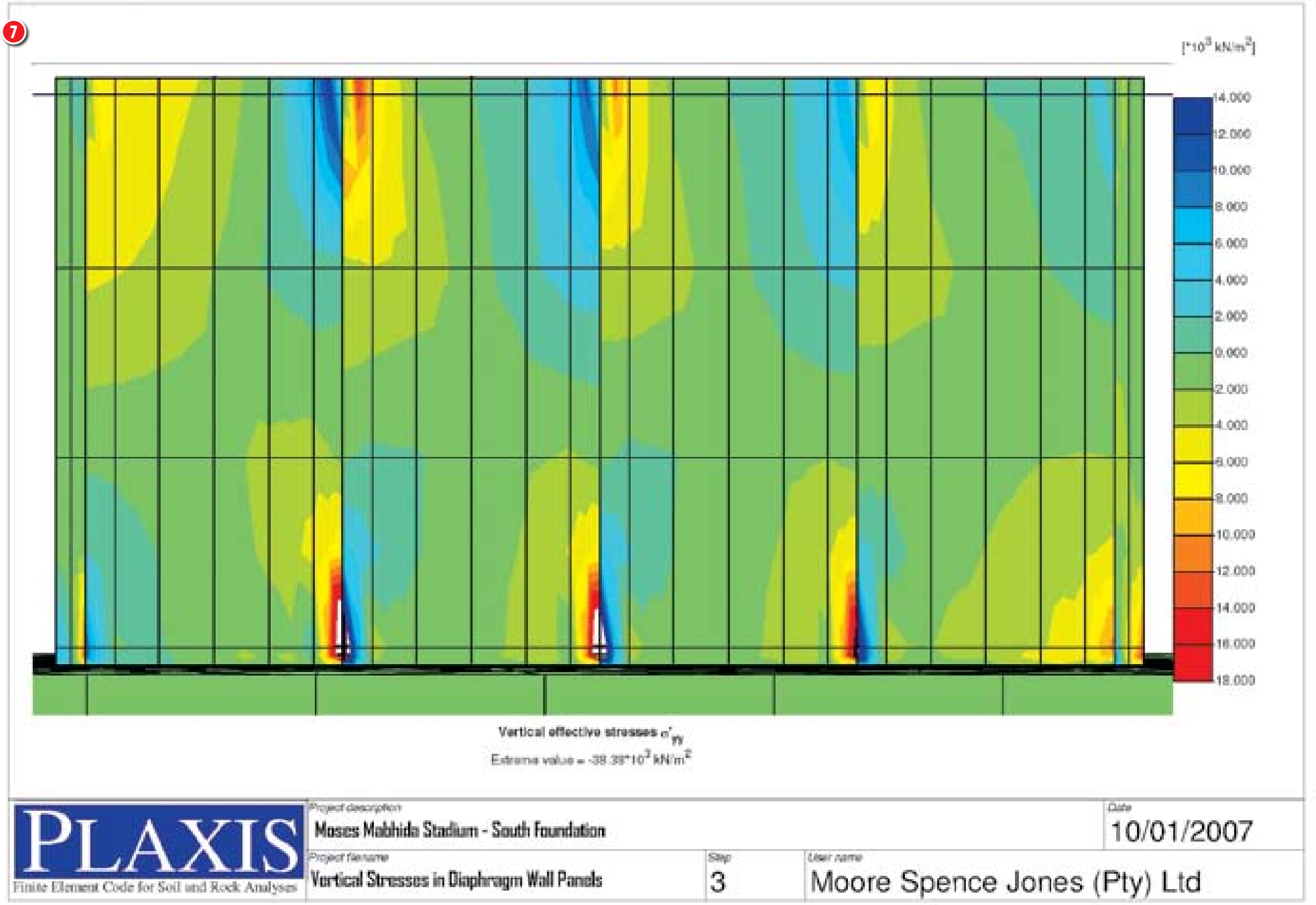

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