SESSION 5: PILES

Papers:

THE TESTING OF LARGE DIAMETER PILE ROCK SOCKETS WITH A RETRIEVABLE TEST RIG
R.W. Johnston, I.B. Donald, A.G. Bennet, and J.W. Edwards; vol 1, 105-108

THE UPLIFT CAPACITY OF STEEL PILES DRIVEN INTO HAWKESBURY SANDSTONE
B.L. Rodway and R.K. Rowe, vol 1, 109-114

THE DESIGN AND PERFORMANCE OF CAST IN SITU PILES IN EXTENSIVELY JOINTED SILURIAN MUDSTONE
A.F. Williams and M.L. Ervin, vol 1, 115-121

Paper by R.W. Johnston, R.B. Donald, A.G. Bennet and J.W. Edwards

Prof P.W. Taylor said that it was unfortunate that since the side shear and base resistance were tested simultaneously, the base of the socket was not loaded until failure. He said that theoretically the ultimate bearing capacity of the base of the socket would be approximately nine times the undrained cohesion value (2800 kPa), ie, approximately 20 MPa. The author considered that it was more important to find the side shear resistance, and, since displacements occurred in side shear at lower loads, it was not possible to test the base of the socket to failure. However, the average maximum normal stress on the base during tests 1 and 2 were 2.73 and 2.34 times the design stress respectively, giving an average factor of safety of at least 2.5.

A.F. Williams noted that base resistance tests carried out at embedment ratios (L/d) less than two might result in catastrophic collapse and heave around the pile but that ratios greater than 2 always resulted in work strengthening. Further to Prof Taylor's comment on the ultimate bearing capacity of the socket base, he suggested that, where the rock behaved nonplastically the assumption that the ultimate bearing capacity would be nine times the undrained cohesion was not valid. Tests on intact Melbourne mudstone had shown base resistance at p/D values of 25 percent to be equal to between 5 and 30 times the unconfined strength, depending on the embedment. In the present paper, the unconfined compressive strength was approximately 3 MPa.

Paper by B.L. Rodway and R.K. Rowe

Prof P.W. Taylor made the point that the uplift resistance of piles driven to practical resistance in the Hawkesbury sandstone was probably higher than the conservative value of 1000kN. The author agreed with the comment and stressed the difficulty in calculating the forces exerted by heavy vibrators and explosive charges. The ultimate skin friction value of 230 kPa was conservative and it was more likely to be in the region of 1000 kPa.

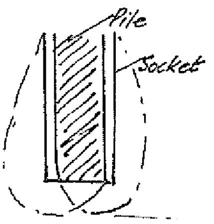
Mr W. Hartley noted that when driving piles into shale bedrock there was a need to be aware that slaking could occur in the rock at the base of the pile. With time, this could reduce the bearing capacity of the material. In his experience, driven piles had been found to be very effective in overcoming uplift forces.

During his presentation, Mr Rodway mentioned that a modified rig had been assembled. The new rig had a 2m concrete bucket so a 2m concrete annulus could be built enabling higher side shear and base loads to be applied.

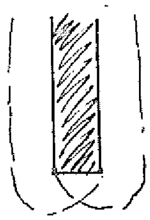
Paper by A.F. Williams and M.C. Ervin

Prof P.W. Taylor commented that in the preceding papers the test piles differed from construction piles in that they did not fill the socket. This would alter the stress distribution (as shown in the sketches below) giving a conservative value.

Test pile



Construction pile



Classically, the settlement ratio for side shear resistance was 1% and for base resistance was 10%. However, in this paper both values were greater than 15% and still increasing.

In tertiary clays and silts in New Zealand there was a problem with smearing along the sides of the socket. This had to be removed by either high air or water pressure.

Finally, as an alternative to preloading by jacking, a small cavity left below the pile could be filled with grout at high pressure.

This applied a permanent load which would take up much of the pile settlement. This method had already been used on a structure at Lake Maracaibo in Venezuela and on the Southern Cross Building in Auckland.

Mr Ervin suggested that, in the case of the side shear test pile, the small void below the test pile might have allowed additional relaxation of the rock, thereby giving a more conservative result. Similar considerations applied to the gap between the pile and socket wall in the base resistance test. However, with build up of normal stresses associated with the shearing of the extremely jointed but relatively compacted rock, up to 20 percent strain hardening should not be considered unusual.

With regard to Prof Taylor's point on side wall smear, the action of the underwater bucket served to clean the socket wall.

Dr R.H.G. Parry noted that work on the Kings Bridge in Melbourne had given rise to much of the present work on rock socket investigation. This bridge was originally designed as a concrete structure and substantial caisson piles were to be used. Later, designers decided to build a steel bridge which, because of its less rigid structure, could be supported by driven piles. Finally, a steel bridge was decided upon, supported by five feet diameter Benoto caissons. Caissons were used instead of driven piles because of the presence of large concrete beams in the structure. The caissons were driven deep enough so that settlements were less than 6 mm and side shear resistance was not taken into account in design. This was a case of over-design of the bridge foundations.

In defence of the use of socketed piles as opposed to driven piles, Mr Williams discussed the case of the West Coast freeway in Melbourne. In this case there had been a cost advantage in the use of cast-in-place piles rather than driven piles. There were also problems of noise and vibrations with driven pile installations. Mr Williams agreed with Dr Parry that present design restrictions were very severe. A case in point was the common restriction of less than 15 mm differential settlement between two adjacent piles, between 4m and 5m apart.

Prof I.B. Donald noted that, in the rail overpass project, $1 million was spent on sockets and piles. Sockets were relatively cheap at approximately $4000 for a 10m socket at the end of a 40m shaft, which might cost about $16,000. Each pile supported a 30 tonne span plus axle loads of up to 22 tonnes. There would also have been a problem with noise if driven piles had been used at this site.