Design and load–displacement behaviour of anchorages in weathered sandstone and shale

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Technical Note 533

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An anchorage contract is currently being carried out in the UK where anchorages are generally founded in weak sandstone. It is apparent that the original designed anchor lengths for the UK contract had been established from optimistic values of working bond stresses obtained from limited published data rather than from in-situ tests. For this reason, I have read the Paper in detail and I find it difficult to justify several statements. In particular, the conclusion in § 13 that: 'The grout/rock bond stress of anchorages founded in slightly to highly weathered shale/sandstone in Singapore can be taken as 640 kPa for a load safety factor of 2'. (I interpret this as a working bond of 640 kPa with an ultimate bond of 1280 kPa.)

15. Firstly, it is necessary to establish the grout/rock bond stresses from the data given

$$T_{max} = 2560 \text{ kN} \qquad \text{from Fig. 1}$$

$$\text{Proven } \tau_{max} = \frac{2560}{\pi \times 0{\cdot}115 \times 8 + \pi \times 0{\cdot}150 \times 5{\cdot}5}$$

$$= 467 \text{ kPa}$$

T_{ult} based on projected value = 7050 kN

(Established from Fig. 5

$$m = \frac{18{\cdot}56 - 10{\cdot}05}{6} \times 10^{-4}$$

$$m = 1{\cdot}418 \times 10^{-4}$$

Therefore

$$1/m = 7050 \text{ kN})$$

Projected

$$T_{ult} = \frac{7050}{\pi \times 0{\cdot}115 \times 8 + \pi \times 0{\cdot}150 \times 5{\cdot}5}$$

$$= 1286 \text{ kPa}$$

Utilized values of T_w from Fig. 6

$$T_w = 1000 \text{ kN} \qquad \tau_w = 426 \text{ kPa}$$

$$T_w = 500 \text{ kN} \qquad \tau_w = 395 \text{ kPa}$$

$$T_w = 400 \text{ kN} \qquad \tau_w = 316 \text{ kPa}$$

16. Secondly, the statement in § 9 of the Paper: 'For the purpose of the design of permanent rock anchorages founded in this weathered shale/sandstone, the grout/rock bond stress can be taken as 640 kPa for a load safety factor of 2, as recommended by the Fédération Internationale de la Précontrainte.[6] This value is within the range recommended by design values for shale and sandstone reported in Littlejohn and Bruce'.[1] Inspection of Littlejohn and Bruce, table III, records *working* bond stresses of 100–140 kPa, and *ultimate* bond stresses of 690–850 kPa and 370 kPa in the shale/sandstone strata. Therefore, it appears that the test data provided and the published recommendation do not provide evidence to justify a conclusion that a working bond of 640 kPa is either proven or recommended.

17. Other statements which appear to be in question relate to the extent of debonding at 1·5 T_w (cycle 5, Fig. 2). The gradient of the deload line of cycle 5 as interpreted from Fig. 2 appears to be considerably different from the gradient of the 'theoretical elastic' line. This would indicate that progressive debonding had, in fact, taken place and is considerably greater than the 4% debonding into the fixed length stated in § 5 and shown in Fig. 3. Moreover, the ground condition detailed in Fig. 1 (which indicates that a mixed stratum including 'silty clay' extended to 5·5 m into the unusually long 13·5 m fixed length) would support the probability that progressive debonding had occurred to a considerable distance into the fixed length. Had such debonding through the mixed strata occurred and the load been resisted by the 8 m effective fixed length in the stronger stratum which had been open-hole drilled, then the following bond stresses could be calculated

$$\text{Proven } \tau_{max} = \frac{2560}{\pi \times 0{\cdot}115 \times 8} = 885 \text{ kPa}$$

18. In conclusion, I would suggest that, in the circumstances, the effective fixed length of working anchorages should not be considered as beginning above the end of the drill casing, and that, although projected anchorage capacities (based on Southwell and Chin) are of interest, only proven capacities should be considered for design. In this situation, working bond stresses (factor of safety of 2) of not greater than 230 kPa (or possibly 400 kPa in the case of the latter interpretation) are justifiable. This would bring recommendations in line with the range of working bond stresses in weak shales/sandstone summarized in refer-

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ence 11 (i.e. 400 kPa in weak sandstones, 200–300 kPa in weak fine grained sandstones/siltstones/shales). This may prevent over-optimistic assumptions when choosing bond stresses for estimation of fixed anchor lengths in weak rock. The installation and testing of preliminary proving anchorages to failure is the only approach that can substantiate fully the ultimate capacity and the factor of safety in hand.

Dr Yue-Choong Kog

I disagree with Mr Barley that the rock/grout bond stress of anchorage founded in slightly to highly weathered shale/sandstone in Singapore of 640 kPa recommended is optimistic.

20. The aim of the method proposed by me, based on Southwell's plot of load and immediate permanent displacement of the test anchorage, is to provide a useful tool for the estimation of ultimate rock/grout bond stress of anchorage from test anchorage not loaded to failure. It is obvious that the rock/grout bond stress obtained from test anchorage not loaded to failure will be conservative, since the rock/grout bond stress may not be mobilized fully. Therefore, it is natural for the rock/grout bond stress obtained by the proposed method to be higher than those values obtained from the test anchorage. As to the utilized values of permanent anchorages, they are less than the 640 kPa recommended for various reasons. For the anchorages with nominal working loads of 400 kN and 500 kN, it is the practice in Singapore to provide a minimum fixed anchorage length of 3–3·5 m, even though a shorter length will suffice. For the anchorage with nominal working load of 1000 kN (not 100 kN as given in the Introduction to the Paper), other considerations such as laminar failure, differential settlement and failure cone, etc., also influence the design.

21. The validity of the proposed method can best be verified by loading a sufficient number of test anchorages to failure. A comparison can then be made between the ultimate rock/grout bond stress obtained by the proposed method and the values obtained by loading test anchorages to failure. I hope that this can be the subject of another paper by those who have the relevant information.

22. The statement in § 9, quoted by Mr Barley in § 16, indicates that my recommendation is valid only for slightly to highly weathered shale/sandstone in Singapore, as reported in Fig. 1, and not for weathered shale/sandstone elsewhere. A detailed study of the soil profile in Fig. 1 will reveal that the shale/sandstone is slightly to highly weathered and not entirely weak and completely weathered. Therefore, when a comparison is made with design values for shale and sandstone reported in Littlejohn and Bruce,[1] it has not been confined, like Mr Barley's, to weathered sandstone and soft sandstone/shale only.

23. It is of interest here to summarize the rock/grout bond stress values reported in Littlejohn and Bruce to show that the recommended value of 640 kPa is indeed within the range recommended by them. The ranges of working and ultimate rock/grout bond stress values recommended for design are 400–2450 kPa and 690–2240 kPa respectively for sandstone, and 100–140 kPa and 170–830 kPa respectively for shale. The range of working rock/grout bond stress values employed in practice is 310–1440 kPa for sandstone, 130–620 kPa for shale, and 70–1820 kPa for shale/sandstone. Information on the ultimate rock/grout bond stress employed in practice for shale and sandstone is limited. It is apparent that the value of 640 kPa is well within the range of working rock/grout bond stress values for shale and sandstone reported by Littlejohn and Bruce.

24. It is also of interest to note the range of working rock/grout bond stress of 63–1000 kPa for sandstone and 100–447 kPa for shale reported by Barley. Once again, the value of 640 kPa is well within the range reported by Barley.

25. The debonded length as shown in Fig. 3 is calculated from data on the extension of the strand of the test anchorage, and discussion on debonding based on Fig. 3 should be more appropriate than that inferred from the load–displacement curve of the test anchorage.

26. I agree with Mr Barley that the installation and testing of preliminary proving anchorages to failure is the best approach for substantiating fully the ultimate capacity and the factor of safety in hand. However, when it is not possible to test the anchorage to failure, the proposed method is useful for estimating the ultimate rock/grout bond stress of the anchorage so that the design of the anchorages will be economical and not unduly conservative.

Reference

11. Barley A. D. Ten thousand anchorages in rock. *Ground Engineering*, 1988, Sept.–Nov.