*Discussion on Paper No. 6915**

Prestressed concrete penstocks at Benmore

by

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Mr W. M. Slater (Concrete Engineering Consultant, Toronto, Canada) wished to make some comments and to ask several questions of the Authors, with whom he had worked during the initial design stage of the penstocks.

77. No information on water hammer theory in prestressed concrete pipes was available to the designers at the preliminary design stage. A value of 5×10^6 lb/sq. in. was assumed for the modulus of elasticity of the pipe wall from which a pressure rise of 45 lb/sq. in. was calculated. Did the Authors have any opportunity to measure the actual pressure rises in the penstocks due to water hammer, and to relate them to the assumed theory?

78. In deciding between precast and cast-in-situ designs, it was important to consider (*a*) the difficulties anticipated in the placing of high quality concrete on the steep penstock slope in the regions of cable concentrations; (*b*) the problem of effecting any 'repairs' to poor concrete in these areas, and (*c*) the high estimated cost of the rigid exterior and interior formwork required for cast-in-situ construction, although the last might have been offset by the high cost of jointing between the precast units. These points had not been mentioned by the Authors.

79. In their analysis of the pipe section the Authors did not make it clear that one of the significant results of the full-scale testing was to show that the calculations of the behaviour of the pipe under pressure load were surprisingly accurate. Although there were sudden changes of cross section and indeterminate values of stress in the anchorage region, the calculations were based on the elastic theory with simple modifications, using only St Venant's principle, and the Lamé theory for thick cylinders. The cracking (and leaking) pressure was calculated to be about 260 lb/sq. in., and the first leak during the test actually occurred at 300 lb/sq. in., although the position of this leak was not predicted accurately, as the Authors had explained. The difference could possibly be explained by the strength of the actual concrete being higher than in that used to calculate the cracking stress. The important fact was that designers could possibly use this design method to develop penstocks with much greater pressures with some confidence, because of the close agreement between theory and practice.

80. With regard to the anchorage zone stresses, this project, when first proposed, was to be the first major application of a 'large' cable used at close centres. Careful consideration was given to the method of analysis and the detailing of the reinforcement of the anchorage zones. It was considered better to use a large number of small diameter bars than a lesser number of large diameter bars in these zones. Would the Authors please state if there was any visible cracking in the anchorage zones, and if so what was the frequency?

81. Mr Slater disagreed with the Authors' general statement in § 72 to the effect

* *Proc. Instn civ. Engrs*, 1966, **33** (April) 677–696.

that: 'as there is a practical limit to the size of tendons, attention would need to be given to reducing the pull-in loss'. In this particular design where the cables crossed over to double the effective prestressing force over a finite length, and in a number of other examples, the pull-in of the tendon or set of the anchorage could often be advantageous in lowering the stresses near the anchorages, and of benefit to the structure as a whole. In continuous structures, for example, where the prestress at an interior support might be required to be higher or equal to that near the anchorages at the exterior supports, tendons were often fully tensioned to about 80% of the ultimate, then released to 70% of the ultimate at the anchorage. A tendon pull-in was thus effected to create a more desirable distribution of force and an economical solution which could be precisely calculated by considering the tendon friction losses. In the penstock anchorage zone, the stresses would have been very high if the pull-in had been much smaller.

82. In some respects the problems faced in the design of large diameter pressure penstocks were similar to those encountered in the design of low temperature prestressed concrete pressure vessels and the detailed test results might be of significant value to designers of these vessels, since full-scale tests of civil engineering structures of this type were rare. The Authors were to be congratulated on their Paper.

The Authors, in reply, took Mr Slater's points in order.

84. With regard to water hammer, in the course of drop load tests on the power plant the gate closure time was progressively reduced, and the designed pressure rise of 45 lb/sq. in. was obtained with an effective closure time of 4·2 s. As this pressure rise was calculated for an effective closure time of 4 s, the assumptions made in the design were apparently reasonably correct. It was considered that in the most unlikely case of mechanical failure the gates might close in 2 s. It had been calculated that the maximum pressure rise in this case would be about 100 lb/sq. in., and could be sustained.

85. The Paper had had to be greatly condensed from its original form and the points raised by Mr Slater as to the disadvantages of in-situ construction had had to be covered very briefly, and inadequately, in the first line of § 4.

86. When the Paper was shortened it had been decided that the recording of the novel construction would probably be of greater interest than a description of fairly common design procedures. The satisfactory agreement between calculations and test results had not, therefore, been dealt with at length, being mentioned only briefly in § 28. The Authors agreed, however, that engineers would be justified in using the theories outlined by Mr Slater for the design of penstocks for higher pressures, and had had this in mind when drawing up their conclusions, in particular § 72.

87. With regard to the anchorage zone stresses, in some of the earlier construction there was cracking along the lines of the 'A' cable anchorages. Generally it did not occur until some hours after the cables were tensioned and was not obvious. In the upper anchorage blocks, as positioned, it appeared on the top face, and in the lower anchorage blocks on the bottom face (Fig. 5). The addition of four $\frac{3}{8}$ in. dia. reinforcing bars placed near each of these faces at right angles to the cables prevented visible cracking.

88. Mr Slater's point regarding the distribution of forces by pull-in was one deserving careful consideration, but in their conclusions the Authors were concerned with the difficulties of assembling steel and securing high quality concrete if the congestion in the formwork were much greater.

ERRATA

The Authors wished to make the following corrections:

Mr Brathwaite had not been admitted to the degree of B.Sc.

p. 691, Fig. 12: the prestressing cables on the left-hand side should have been shown on the outside of the penstock wall.

p. 693, § 52: *for* 7 ft $10\frac{5}{8}$ in. *read* 7 ft $11\frac{5}{8}$ in.

p. 696, § 72: *for* tensions *read* tendons.