Failure of a prestressed concrete reservoir

by

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Mr G. P. Manning (Chartered Civil Engineer) wrote that a thin ring subject to uniform external fluid pressure became unstable when $p = 3EI/R^3$. Assuming $E = 3 \times 10^6$ lb/sq. ft,

$$p = 3 \times 3 \times 10^6 \times 144 \times \frac{(0.833)^3}{12} \times \frac{1}{(65.5)^3} = 222 \text{ lb/sq. ft.}$$

27. This was assuming that the ring was perfectly circular and the concrete perfectly uniform. It was possible that in practice a wall could fail at half this figure.

28. A good backfill which was uniformly placed would immediately develop counter-pressure which would suppress any tendency to instability failure. The hoop steel had the same effect. But it was an uncomfortable thought (particularly in regions subject to prolonged drought followed by torrential rain) to realize that the tendency to instability existed.

29. The values in Figs 7 and 8 could be obtained direct from standard tables. Neither of the two figures included a correction for 'crown-drop' (diameter-stretch) effects.

30. Mr Manning believed that had the design complied with § 1.89 of the Foundations Code C.P. 4 (1954) the collapse could have been averted.

Mr H. S. Bhatia (Principal Research Officer, Building and Road Research Institute, Kumasi, Ghana) wrote that the Author had presented a very interesting Paper on the failure of a structure caused by the earth pressure of cohesive soil, when used as a backfill material. The analysis had clearly indicated the factors responsible for the failure, but the Writer would confine himself to few additional remarks on the analysis.

32. A laboratory study previously conducted by the Writer with a modified triaxial cell (a type of cell similar to that which was used previously by the Soil Mechanics Laboratory, Delft, Holland), to study the coefficients of earth pressure in soils, had given some interesting results. The modified triaxial apparatus had a deviator stress piston of the same diameter as the soil sample used in the test. This arrangement had meant that the deviator stress and the horizontal stress on a sample could be changed independently. This was because cell pressure was acting on two horizontal axes only, whereas the deviator stress was acting on the third axis. The sample in the test had been given the same cell pressure and deviator stress and allowed to stabilize. At this stage the horizontal stress on the sample had been slowly reduced, without changing the deviator stress, until the sample had started to fail. A number of values of deviator stress and the corresponding horizontal stress at failure had been taken, and a linear relationship was found to exist between them. The slope of this line gave the coefficient of earth pressure of a soil at failure. A typical curve for a saturated soft clay as obtained in the test was shown in Fig. 9.

FIG. 9: A GRAPH OF VERTICAL STRESS *VERSUS* LATERAL STRESSES AT FAILURE OF A CLAY

This study gave the coefficients of earth pressure for loose soft clays between 0.30 and 0.33. The Writer therefore considered that the value of the coefficient of earth-pressure k_2 , which was used by the Author for working out the lateral stability of the reservoir due to backfill, was rather low for a very loose, saturated and soft material. However, the Writer agreed that the coefficient of earth pressure (k_2) used by the Author was reasonably close to the figure quoted for clays in many soil mechanics books. Using a k_2 value of 0.32, the thickness for lateral stability of the wall worked out to be 15 in., instead of the value of 10 in. which had been provided for in the design.

33. Referring to § 6, the Author used a value of 110 lb/cu. ft for the wet density of the backfill to estimate the lateral pressure exerted on the base of the reservoir wall. The backfill density of 110 lb/cu. ft was likely to give a lateral thrust at the base of the reservoir of the order of 1400 lb/sq. ft. This lateral thrust could give a compressive stress of 750 lb/sq. in. in a 10 in. thick concrete wall. Adding this stress to the compressive stress of 1340 lb/sq. in., induced by prestressing, the stress in concrete rose to 2100 lb/sq. in. This gave the ratio of compressive stress in concrete to its compressive strength as 0.6. This ratio is slightly higher than 0.55, which was calculated by the Author and which the Author had considered as rather high. The Writer felt that a saturated density of 110 lb/cu. ft (the value used by the Author in his analysis) was rather low and a saturated density of about 120 lb/cu. ft would have been expected for the clay under the conditions discussed by the Author. This tended to suggest that the actual ratio of compressive stress to compressive strength at the time of failure could have been even higher than 0.6. Mr Bhatia felt that this point should have been considered at the time of design and that a minimum thickness of 15 in. for the reservoir wall could have proved satisfactory in safeguarding the concrete wall against compressive stresses.

34. The Author also pointed out that at the time when the ground was being excavated the water table was at 11 ft from the ground surface. If the effect of hydrostatic pressure had been considered the compressive stress in concrete could have been still higher. However, the Author pointed out that pressure release valves were provided to reduce the hydrostatic pressure against the reservoir wall. The Writer would be very interested to have more information on the location of various pressure valves and their effectiveness in the drainage of the soil. This could provide valuable information for the use of pressure valves in clay backfills for effective drainage in the future.

35. The Writer agreed with the Author that although backfilling over half of the circumference of reservoir did create conditions favourable to cracking, the risk of cracking could not have been completely eliminated even if the backfilling was carried out all around the reservoir at the same time. This was due to the fact that the structure was very much under designed for conditions of maximum earth pressure when the reservoir was empty.

36. One important fact revealed by the Paper was that there was a great need for further study on earth pressures in clays at different moisture contents and densities. In fact there was no precise information on the earth pressures in clay soils in the field of soil mechanics and further research was definitely called for. The Writer felt sure that this interesting Paper would act as a great stimulant to other workers in this field.

Dr R. D. Mackey (Assistant Engineer, Highways and Bridges Department, West Riding of Yorkshire County Council) and **Mr D. Pearson-Kirk** (Department of Civil Engineering, University of Leeds) wrote that after reading Professor Neville's most interesting Paper, which stressed the fact that the pressure exerted by a backfill caused the collapse of a prestressed concrete reservoir, they felt that it might be of interest to give details of recent research work carried out at the University of Leeds. This work had shown that at-rest, active, and passive earth pressures all increased as the compacted density of a granular soil increased.

38. Series of tests had been carried out, using three types of sand, with a 12 in. high retaining wall which formed one end of a steel tank. By using a very sensitive pressure-measuring device in the face of the retaining wall (consisting of silicon semiconductor strain gauges fixed to the backs of a series of 1 in. span fixed-end beams) it had been possible to measure accurately the pressures acting on the retaining wall.

39. Since the pressures could be accurately measured, it was essential that no side-wall effects or end-wall effects should mar their accuracy. The side-wall effects had been investigated by measuring the pressure distributions down the retaining wall at various positions across it. The results of these tests had shown that the central 8-10 in. width of the tank was not affected by the friction between the sand mass and the side walls.

40. The end-wall effects had been investigated by carrying out a series of tests using four different depths of sand. These tests had shown that the pressures determined, and the corresponding failure surfaces, had not been affected by the end walls. The pressures found experimentally for the retaining wall were therefore free of side-wall and end-wall effects.

41. The Author had stated that the design of the reservoir had been carried out assuming that the coefficient of active earth pressure, k_2 , was given by the equation:

$$k_2 = \frac{1 - \sin \phi}{1 + \sin \phi} \quad \dots \dots \dots (1)$$

This equation had been found to give safe values for granular soils in the loose density state, but for compacted granular soils equation (1) gave unsafe values which were only from 30 to 50% of the values which had been determined experimentally. Equation (1) suggested that as the density of a sand increased, and thus the angle of shearing resistance increased, so the value of k_2 decreased. However, it had been

found experimentally that the value of the coefficient of active earth pressure of a compacted granular soil, k_{2c} , increased with an increase of compacted density.

42. The results had been used to give the following suggested empirical formula:

$$k_{2c} = 0.9 k_{qL} + 8.2 \frac{(k_{qL} - k_{q1})}{(\gamma - \gamma_L)} \quad \dots \quad (2)$$

where k_{qL} and k_{q1} were values of the coefficient of active earth pressure calculated using Caquot and Kerisel's tables at densities γ_L and γ , respectively; γ_L was loose density, and γ was compacted density.

43. By carrying out shear-box tests at the loose density and at the compacted density which was to be aimed for on site, the angles of shearing resistance at these densities could be determined and, assuming the angles of wall friction given in C.P. 2, the Caquot and Kerisel values could be calculated, and hence the value of k_{2c} could be obtained.

44. The Writers' experimental work had also shown that the active pressure had been mobilized by a wall movement, or yield, in the range of 0.005–0.003 times the wall height, the required wall movement decreasing as the compacted density increased. This showed good agreement with the wall movement of $2\frac{1}{2}$ in. which the Author said had been observed, this movement being equivalent to 0.0075 times the wall height.

45. In the design of the reservoir, k_2 had been assumed to be equal to 0.286, and the corresponding value from Caquot and Kerisel's tables would be almost exactly the same. The value given by equation (2) would then be:

$$k_{2c} = 0.9 \times 0.28 = 0.252. \quad \dots \quad (3)$$

46. However, this would be a minimum value, as the value of k_{2c} would increase as the compacted density increased. This minimum value was still greater than the maximum value (0.222) which the Author had evaluated for lateral stability.

47. The Writers had also investigated the effect of compaction on earth pressures at rest. For granular soils in the loose density state, it had been found that the value of the earth pressure at rest coefficient, k_0 , was equal to 0.45. For a compacted granular soil the coefficient, k_{0c} , was found to correspond to values which could be determined from the following suggested empirical formula:

$$k_{0c} = 0.45 + 0.45 (\gamma - \gamma_L). \quad \dots \quad (4)$$

48. The coefficient of earth pressure at rest for a compacted granular soil had been found in some cases to be as high as 3.4, i.e. a value which lay between the at-rest and passive earth pressure coefficients for the soil in the loose density state. With wall movement, or yield, of 0.005H to 0.003H, the values obtained for k_{0c} from equation (4) had reduced to the values for k_{2c} given by equation (2).

49. Thus, whilst the Writers' work had been confined to granular soils, the work had shown that the earth pressures exerted by a compacted soil were far greater than those which were calculated using present-day theories. Empirical formulae, which might err slightly on the safe side, had therefore been suggested.

50. The Writers agreed with the Author that the confined nature of the area in which the fill had been placed would increase the lateral earth pressure acting on the reservoir wall, the pressure taking a value between that of the active and at-rest values. This pressure would, however, be further increased by compaction of the fill.

Mr P. G. Ramakrishnan (Resident Engineer, Lummes Company, India) wrote that the Author was to be congratulated for the excellent treatment of the problem. The method of analysis was most lucid and interesting.

52. It was hard to judge the validity of one point in the problem. If the cylindrical tank was free to slide, how could the earth pressure induce stresses in the wall, since

once the tank moved, the intensity of active pressure reduced considerably? In spite of this, the sides had failed. So it amounted to saying that the tank had offered resistance to sliding and since the tank walls were not strong, the failure had occurred.

53. The analysis did not include the friction between the wall and the fill. This force would actually reduce the active pressure and it was possible that the force causing sliding would be less than the resisting force. The resisting force would be (i) the frictional resistance of the concrete walls, and (ii) the friction due to the soil and the side walls which would have a tendency to lower the values of the active pressure.

54. Thus the tank would not have slid or there should be some other obstruction to sliding which would expose the weak walls for a compressive stress more than the strength of concrete.

55. Once the wall had failed, the bottom portion should have moved more than the top due to the distribution of pressure.