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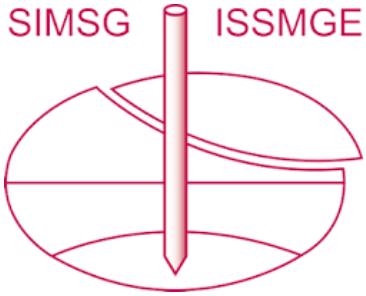


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# Failure of a Reinforced Concrete Reservoir

Rupture d'un réservoir de béton armé

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SUMMARY

This paper gives an account of the failure by overturning of a reinforced concrete reservoir wall and describes the soils and the various contributory causes of the failure.

SOMMAIRE

Cet article rend compte du renversement d'un mur en béton armé de réservoir et décrit les sols et les causes diverses qui y ont contribué.

THE AUTHOR CARRIED OUT an investigation into the causes of failure of a 10-million-gallon reinforced concrete reservoir on Hartman Hill, Salisbury, Southern Rhodesia.

Fig. 1 is a section through the wall. The reservoir is rectangular in plan, the internal dimensions being 287 feet by 247 feet, impounding a 21 foot 3 inch head of water. The

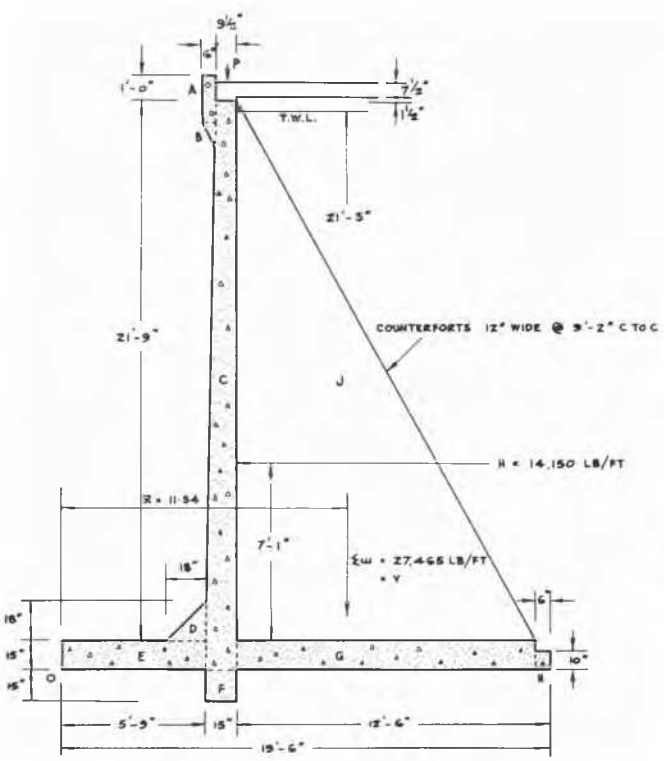


FIG. 1. Cross-section of wall.

original site was levelled to accommodate the reservoir, the east side being in heavy cut. On the west, or failure side, the toe of the wall was generally above original ground level. Because of this, concrete thrust blocks had been cast at intervals of about 14 feet on the "failure" side (see Fig. 2). The reinforced concrete counterfort walls were constructed in bays linked by rubber water-stops. The roof is a flat slab supported on a system of columns at 20 ft 9 in. by 16 ft 0 in.

FIG. 2. Spalling of concrete at makeshift thrust blocks.

FIG. 3. Inside reservoir showing underside of overturned wall.

FIG. 4. View from roof of reservoir looking down on overturned wall.

FIG. 5. View looking down on overturned wall.

FIG. 6. Wall adjacent to overturned section showing uplift.

centres. The floor is a two-layer concrete slab, the floor joints being bitumen-filled without membrane water-stops.

The filling commenced early in December, 1960. Some 17 feet of water were contained for at least two months, after which the full supply level was attained and held for approximately three weeks before the sudden failure occurred—two bays of southwest facing wall overturning outwards (Figs. 3 to 6). During the period of full head, a considerable amount of rain had fallen.

## THE SITE

### *Geological*

The rock formations on Hartman Hill are generally plutonic, with gabbro predominating. The weathered rocks are clearly exposed in the cut on the east side of the site. The soils are evidently all residual, derived from gabbro.

### *Description of the Soils*

Typical particle size distributions are given in Fig. 7.

*Ironstone gravel.* Imported material varying from 1 ft to 1 ft 6 in. but generally 1 ft 6 in. thick under the floor area. Generally non-plastic gravel graded from about 1 in. maximum size, having about 20 per cent silt. The gravel forms a highly permeable but compact layer on which the reservoir was cast.

*Red to red-brown silt-clay.* This leached soil is usually the weakest on site, generally occurring directly under the imported layer, but it may be found as very soft and unconsolidated lenses in highly decomposed gabbro. The silt-clay has a $w_L = 59$ and an $I_P = 38$. A feature of this soil is its high natural void ratio ($e = 1.4$ approx.) which makes it difficult to sample in the field and to prepare samples for testing without undue disturbance. Twelve-inch cube undisturbed samples were excavated.

*Fine-grained grey-brown decomposed gabbro.* In the mottled structure of this massive soil ($w_L = 28$, $I_P = 8$) the dark minerals of the gabbro can be seen. This soil is far denser than the silt-clay although in some instances both it and that described immediately below were soft *in situ*. It normally occurs in a hard, compact state.

*Grey-brown mottled decomposed gabbro.* Similar to fine-grained gabbro described above but coarser and generally fresher, with a slightly lower plasticity index. This soil usually has a high bearing capacity.

### *Soil Profiles*

From 17 boreholes and trial holes, Fig. 8 gives two typical sections, illustrating two distinct types: (*a*) the softer profiles characterized by horizons free from large masses of rock and occurring on the failure side of the reservoir; (*b*) weathered and shattered rock with soil in a compact state either interspersed or underlying the shattered rock and occurring on the "unfailed" side of the reservoir. The soils vary considerably in thickness, density, and position, with apparently random arrangement. It should also be noted that red or red-brown silt-clay soil contains many random spherically weathered boulders—a fact which makes the interpretation of consolidation tests difficult. When sampled, most of the soft, unconsolidated soils were saturated, especially below a depth of ten feet. The high saturated moisture contents revealed uniformly high void ratios. A water table was found at a depth of about 12 feet.

### *Triaxial Tests*

Triaxial compression tests were run on representative soft soils taken from under the reservoir failure area. Average

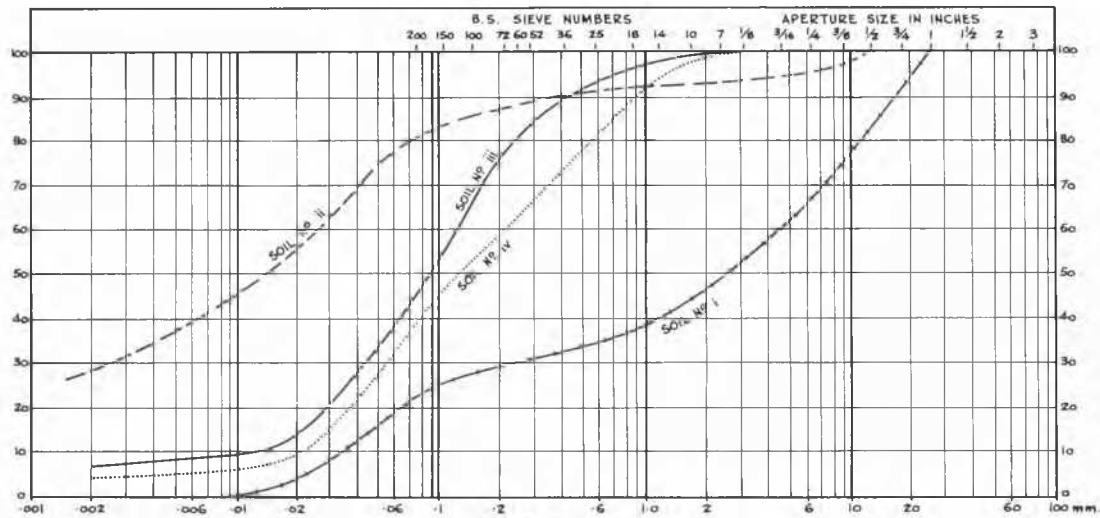


FIG. 7. Soil gradings.

results from three series of saturated drained tests are given in Table I.

TABLE I. TEST RESULTS FROM THREE SERIES OF SATURATED DRAINED TRIAXIAL TESTS

| Description | $\gamma$ (lb/cu.ft.) | $\gamma_d$ (lb/cu.ft.) | $w$ (per cent) | $e$ | $c$ (lb/cu.ft.) | $\phi$ |
|---|---|---|---|---|---|---|
| (*a*) Red silt-clay | 109.8 | 84.7 | 29.7 | 1.00 | 8.0 | 9°30′ |
| (*b*) Red-brown decomposed gabbro with red silt-clay | 119.6 | 96.2 | 24.4 | 0.77 | 2.2 | 7°0′ |
| (*c*) Light grey decomposed gabbro | 112.0 | 96.1 | 16.6 | 0.77 | 12.5 | 14°30′ |

*Consolidation Tests*

Twin consolidometer tests were run simultaneously (at natural moisture content and saturation) on undisturbed samples taken outside the reservoir area. The samples had not been subject to plant compaction as had the soils in the reservoir area itself. The densities and void ratios of these soils are given in Table II. These samples were surprisingly unconsolidated in spite of virtual saturation *in situ*; but the tests showed no tendency to the "collapse" phenomenon under load and inundation. An average compression index, $C_c = 0.0695$, was determined for that portion of the corrected $e - \log p$ curves between ½ and 2 kg/sq.cm.

TABLE II. DENSITY AND VOID RATIO OF LEACHED SOILS

| Description | $\gamma$ (lb/cu.ft.) | $\gamma_d$ (lb/cu.ft.) | $w$ (per cent) | $e$ |
|---|---|---|---|---|
| Red, highly weathered gabbro, heavily leached | 98.6 to 101.7 | 69.4 to 71.4 | 42.1 to 42.5 | 1.45 to 1.29 |

*Percolation Tests*

As an indication of the relative permeability of the ironstone gravel layer and the underlying red silt-clay, percolation tests were performed in excavations 1 ft square by 1 ft deep. In the gravel layer water from a high-pressure hose ran away so fast that it was not possible to fill the excavation. However, the rate of percolation in the red silt-clay was only 5 in. in 18 hours. That is to say, drainage from this layer was very poor. (Percolation rating = 216).

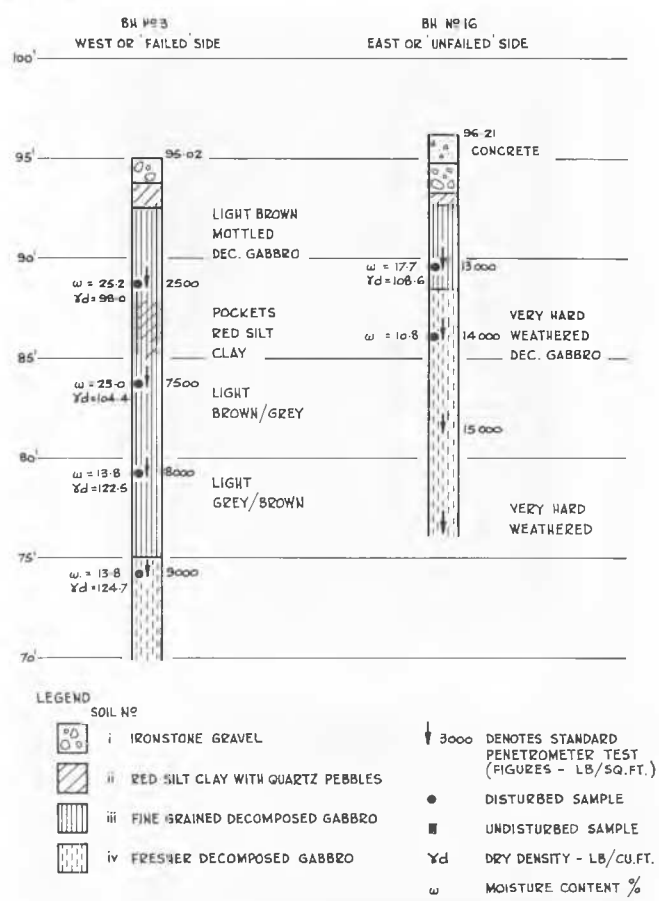


FIG. 8. Typical borehole profiles.

*Leakage Tests*

To determine rates of leakage through the floor joints, a special permeameter was devised which simulated 21 ft head of water. In operation, a 12-in.-diameter closed cylinder was bolted to the concrete. The results represent total leakage measured and are comparative. Typical results are given in Table III. It is certain that considerable leakage took place at various points through the joints.

TABLE III. TYPICAL JOINT LEAKAGE TESTS WITH PERMEAMETER

| Leakage in gallons per minute | Remarks |
|---|---|
| Nil | Centre of concrete panel, control test |
| 1.36 | Bad joint, hole visible completely through joint material |
| 0.25 | Hole visible in joint but not completely through |
| 0.17 | Hole visible in joint but not completely through |

## THEORETICAL TREATMENT

On two sides, the reservoir wall has external support from earth fill. However, this does not apply on all sides and it was assumed that the wall was designed to resist the full water pressure without external surcharge. Various possible modes of failure were considered in detail.

The resultant thrust was calculated by conventional means and found to intersect the base 7.05 ft from the toe, i.e., within the middle third. The factor of safety against overturning was found to be 2.65, which is adequate provided there is no uplift pressure. It was calculated by conventional means that the superimposed pressure was 2,570 lb/sq.ft. at the toe and 239 lb/sq.ft. at the heel under full head. By means of an influence chart and foundation diagrams the imposed vertical stress was calculated for 5 positions between toe and heel, for depths to 15 feet and added to the overburden pressures. The relative settlements of toe and heel were then considered, dividing the soil layers into depth zones and assuming the compacted gravel layer to be incompressible. Allowing for the effects of irregularities of the profile and for random discontinuities, it was considered that the total differential movement between toe and heel was of the order of 1.2 in. The total downward movement of the toe before any shear failure was about 2 in. It should be remembered that this movement would slightly increase the toe pressure and reduce the heel pressure, tending to increase the differential movement.

Attention was next given to the shear stress and bearing capacity. It was estimated that the maximum imposed shear was 807 lb/sq.ft., whereas under the worst conditions (i.e., series (*b*) Table I) the shear strength was only about 650 lb/sq.ft. Working on a rough mean of series (*a*) and (*b*), the shear strength could have been about 1,100 lb/sq.ft. Local shear failure could have been expected and the factor of safety against a general shear failure must have been low. As the soil profiles are irregular, it is unlikely that any horizon was *generally* as weak as the case (*b*) shown in Table I. On the other hand it is likely that even weaker lenses were present. There must have been a redistribution of stress on to harder and stronger soil, in any line parallel to the wall, which in turn was stressed to a much greater extent than assumed.

### *Bearing Capacity*

On the assumption of a uniformly loaded strip, Terzaghi's formula for bearing capacity for local shear failure was tried for $c = 2.2$ lb/sq.in. and $\phi = 7°$ at a depth of 2 ft and the estimated bearing capacity obtained was $p_u = 1950$ lb/sq.ft., which is far less than the calculated vertical stress at a depth of 2 ft. The figures quoted are for drained tests on certain samples, whereas in the field just before failure conditions must have tended to be undrained. Some quick triaxial tests were performed, but on rather stronger samples in the (*c*) group. Again, if the values of $c$ and $\phi$ are slightly increased, which is more in line with average conditions, we have a state of precarious equilibrium. It is felt that in practice the bearing capacity probably had a factor of safety against local shear of 1.1 to 1.5.

### *Sliding Failure*

Depressions in the rough cast–*in-situ* concrete are filled with soil and sliding could only take place through the soil. Assuming $\phi = 7°$ and $c = 317$ lb/sq.ft. (or 2.2 lb/sq.in.), which is low, it was calculated that the theoretical factor of safety against sliding on a plane below the bottom of the keel just below the gravel layer was 0.72. This means that the wall would slide if not prevented by thrust on concrete blocks and/or soil at the toe. In fact, some of these concrete blocks took sufficient thrust to cause severe spalling of concrete (Fig. 6). The further possibility of a circular slip failure was investigated but was rejected.

## CAUSES OF FAILURE

From the investigations it is clear that the walls must have been very near to overturning for some time before actual failure, the factor of safety being only fractionally greater than unity. The following chain of events probably occurred.

Differential consolidation had occurred, causing a tilt of up to 1 inch over the base of the wall, resulting in a tendency to tilt forward. This, as well as the consideration in the following paragraph, did not take place uniformly since the soil profile varies considerably along the length of the wall. It is certain that a proportion of the walls which remain standing have taken a permanent tilt.

At several points along the length of the wall towards the toe and at depths of up to five or six feet below the base, local shear failures took place. There was little or no margin between the total applied load and the ultimate strength of the soil in a saturated condition.

It is certain that considerable leakage was taking place through floor joints at certain points for some months but increasing very greatly during the "full level" period resulting in saturation of the ironstone gravel filling layer and slowly draining downwards through the less permeable layers thereunder. This percolation water was draining free through the comparatively highly permeable ironstone gravel layer towards the outside, but was generally restricted by the cut-off wall. In this respect the effect of the key or cut-off wall was to nullify almost completely the very good underdrainage provided by the ironstone gravel layer. It is also certain that this leakage water could not have built up any head under the heel of the wall, tending to equalize weight on the heel during the filling period, since this would have resulted in *immediate* failure by overturning. The leakage affected the stability of the wall in two ways: by saturating the soft, unconsolidated soils and by hydraulic uplift.

The soft, saturated soils underlying the region between the toe and the cut-off wall consolidated, and some local shear failures took place, causing the wall to tilt, thus increasing the pressure. This movement, possibly accompanied by sliding, very greatly increased the leakage of the joint between wall heel and floor, allowing the groundwater to rise faster than the infiltration rate, eventually exerting an uplift pressure on the heel of the base. At that moment, all stability was lost and the wall overturned. It should be noted that if adequate drainage paths had been provided through the key or cut-off wall to the outside, the uplift head could not have developed.

## CONCLUSIONS

Where the stability of a wall depends entirely upon the effective weight of water superimposed on the base, the following measures are necessary for security.

1. Adequate underdrainage to prevent hydraulic head developing under the base. This underdrainage is most effective when located directly under all joints. In the present case the ironstone gravel layer would have provided adequate underdrainage if it had not been cut by the key wall.

2. A *positive* membrane seal for all joints by means of one of the many types of water-stops in rubber or polyvinyl chloride.

The minimum depth of the foundations should be dictated by: (*a*) the depth to a soil stratum of adequate bearing capacity; (*b*) the depth below which the moisture content is not significantly affected by seasonal moisture variations; (*c*) the depth below any future scour or undermining by local (e.g., water main) excavations.

To sum up, the reservoir failed because of faulty design, in that: (*a*) the wall was founded far too shallow on a soft, unconsolidated soil, which when saturated could not safely carry the imposed loading; and (*b*) inadequate provision was made to guard against hydraulic uplift on the base, which was the final cause of catastrophic failure.