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Failure of Embankment in Kinkai Bay

Rupture d'une digue de protection dans la baie de Kinkai

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Summary

The authors describe soil investigations and detailed analysis of the sliding action after the failure of the Kinkai reclamation embankment occured in 1958 during construction.

This failure was of the typical circular arc form. The influence of the length of the sliding surface on the factor of safety against the sliding computed by the current method is discussed. By considering the results of soil studies and circular arc analysis on the probable causes of failure, the repair work was planned and carried out efficiently.

Sommaire

Après la grande rupture en fondation de la digue de Kinkai survenue pendant la construction, en 1958, les auteurs ont procédé à l'étude du sol et à des analyses détaillées du glissement. Leurs études ont montré que la rupture était du type circulaire. Les auteurs discutent l'influence de la longueur de la surface de glissement sur le coefficient de sécurité déterminé par la méthode courante. Le projet des réparations et leur exécution tiennent compte des études du sol et de l'analyse en rupture circulaire des causes probables de la rupture.

General Description of Embankment Construction

A reclamation embankment was constructed in January 1957 at the mouth of Kinkai Bay, Okayama Prefecture, Japan. A very soft alluvial clay layer with a thickness of about 22 m underlies the major part of the embankment. Profile of the underlying soil strata along the embankment is shown in Fig. 1. Mean void ratio of the clay soil is about 3·0, and unconfined compressive strength is 0·08 kg/sq. cm for the upper portion, and 0·5 kg/sq. cm for the lower stratum.

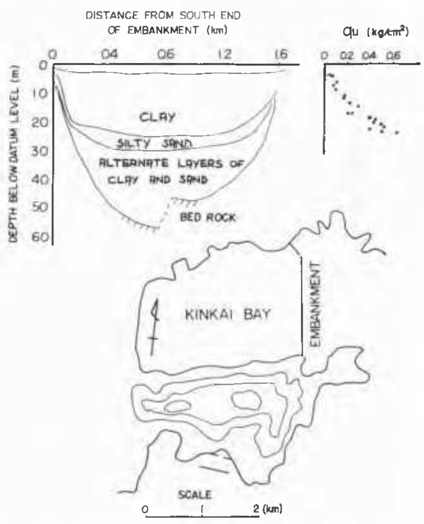


Fig. 1 Location of embankment and soil profile.
Emplacement de la digue et coupe du terrain.

According to soil investigations, the clay is considered to be in normally consolidated condition.

In Fig. 2 the typical schematic cross section of the embankment before failure is shown. Vertical sand drains were driven into the soft clay layer at the interval of 1·7 m and the soil was consolidated by the weight of embankment materials, which were transported over the embankment by dump trucks from a borrow pit near the site, and were placed in five layers every four months in order to ensure adequate consolidation.

Before each stage of filling, counterbalances were placed on both sides of the main embankment so that the computed safety factor of stability of the embankment against sliding was at least 1·2 at each phase of construction. After the third layer was placed, the cofferdam for the embankment was closed by gates constructed on the bed rock at the north end of the embankment in May 1958, and then the water in the cofferdam was gradually pumped out.

Failure Conditions and Soil Investigations after the Failure and the Rate of Settlement of the Embankment during Construction

On the August 12 th 1958 large scale failure of the embankment, of which the crest length was 380 m, suddenly took place at the centre. When the embankment failed, the inside water level had been drawn down to 0·5 m below the datum level, and the outside sea level had been 0·6 m above the datum level. The fifth layer and parapet concrete blocks were under construction and these works had proceeded from the north end of the embankment to the south end of the part which failed.

Transverse and longitudinal cross sections of the embankment before and after failure are shown in Fig. 3. The maximum settlement of parapet concrete blocks was about 6 m, and the blocks were slightly inclined outwards the central portion of the embankment which failed having been completely submerged. The inside counterbalance and the inside sea bottom were heaved upward, and the maximum heaving

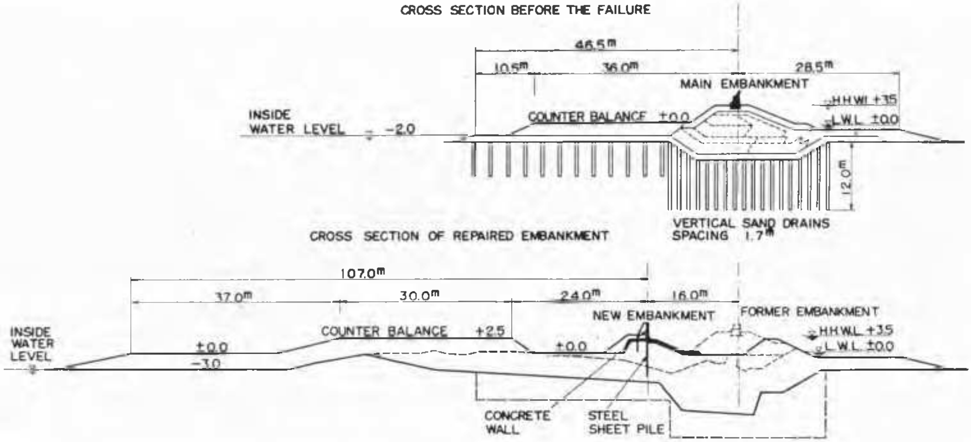


Fig. 2 Cross section of the embankment.
Sections transversales de la digue.

amounted to 3 m or more. Any movement in the outside of the embankment was not observed. According to these observations the failure seemed to be of the circular sliding type.

Before, the failure, it had been assured by soil tests that unconfined compressive strength of clay soils increased by sand drain procedure were not less than the expected values. In an unconfined compression test on clay, deformation corresponding to maximum strength becomes larger and strength decreases if the soil is remoulded.

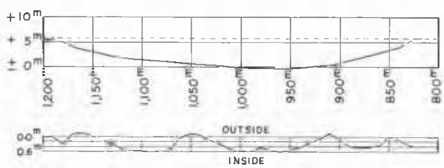
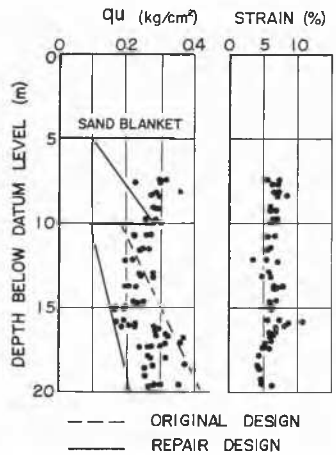


Fig. 3 Displacement of embankment crest.
Déplacements du couronnement de la digue.

Undisturbed sampling and unconfined compression tests were performed shortly after the failure, in order to find the sliding surface by the decrease of soil strength in the failure zone resulting from remoulding caused by the large sliding deformation. In Fig. 4 a typical result of these surveys is shown. Unconfined compressive strength of soil at a depth of 14 m to 16 m is comparatively small, but corresponding compressive strain is large, which may be caused by remoulding of the clay along the plane of failure. In Fig. 5 these are indicated by thick lines. The maximum decrease of unconfined compressive strength found by soil tests was about 40 per cent of the original value.

In the original design, unit weight of soil materials in the main embankment was assumed to be 1·8 tons per cub. m. Nevertheless, measurements after failure gave a value of from 2·02 to 2·33 tons per cub. m. in the upper layer of the embankment. This increase of unit weight was considered to be due to compaction by heavy traffic of dump trucks. However, this effect might not reach to soils in the lower layers of the embankment which had been transported by barges and placed in sea water. Therefore, the average value of the unit weight of the main embankment materials may be somewhat smaller than the average value obtained by measurement. This increase of unit weight of the soil is sure to have been an important cause of failure.



Fig' 4 Unconfined compressive strength after failure.
Résistance à la compression sans contrainte latérale après la rupture.

Immediately after the fifth layer and parapet concrete blocks of the failed portion had been placed, namely 7 to 14 days before the failure, the rate of settlement of this embankment suddenly increased and reached to 3 cm per day, in spite of the fact that under this load condition the theoretical value of the rate of settlement was calculated to be 0·9 cm per day. By these records it could be assumed that the actual

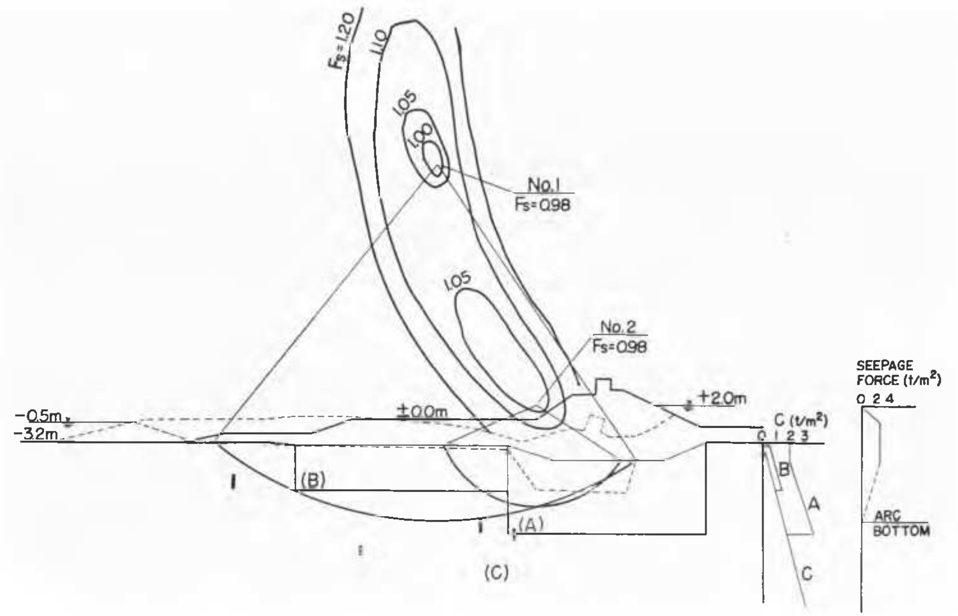


Fig. 5 Results of analysis by the circular arc method.
Résultats des analyses par la méthode de rupture circulaire.

sliding action started 7 to 14 days before failure and gradually increased until the accident occurred.

Failure Analysis and Causes of Failure

By considering the measured movements of the embankment and the neighboring sea bottom, and the above mentioned results of soil investigations, this failure was probably of the circular arc type. A detailed analysis of the failure by the circular arc method was therefore performed. The following assumptions were made :

1. Outside sea water level was 2·0 m above the datum level which was the highest level since the rate of settlement of the embankment had become noticeable, and this value gives the lowest sliding stability to the embankment;
2. Inside water level was 0·5 m below the datum level;
3. Unit weight of the main embankment was 2·1 tons per cub. m;
4. Seepage force and shear strength of the underlying clay soil were estimated as indicated in Fig. 5;
5. Buoyancy was effective only for the embankment materials below inside water level;
6. Angle of internal friction of the embankment materials was 45 degrees.

In Fig. 5, contour lines of the computed safety factors and also circular arcs of the lowest safety factor are shown. Two possible centres of circular sliding arc of the same minimum safety factor 0·98 were obtained. One of them, namely centre No. 1, is located in a higher position with a large circular arc, which coincided with the sliding surface found by soil investigations. Another centre, centre No. 2, is located in the lower position near the main embankment with a small circular arc. In these calculations seepage forces acting as sliding forces against soil masses within the circular arcs were assumed to correspond to items (4) and (5) for simplicity of calculation, and therefore in cases of small circular arc sliding, seepage forces were considerably overestimated in the calculations. According to the stability computations about centre Nos. 1 and 2, considering seepage forces estimated by flow net, safety factors were 1·13 and 1·17 respectively. Therefore, it is conceivable that the sliding occured along the large sliding surface.

By the above analysis it may be concluded that the failure is one of typical circular sliding caused by the lowering of the safety factor due to the increase of unit weight of the embankment materials by compaction from heavy traffic.

The circular sliding surface method is quite conventional, and the shear strength of clay is also determined by the conventional compression test, so that the strain of the clay which might become the cause of progressive failure is completely ignored. Therefore the real safety factor cannot be found by the current circular arc method.

During construction and before failure, although there were several cases in which the computed safety factors against sliding became about 1·0, there did not appear any indication of failure. In these cases the lengths of circular arcs, corresponding to the minimum safety factors against sliding, are considerably smaller than those in the case of this failure. These facts seem to relate to the difference in amount of the progressively mobilized shear strengths along the sliding surfaces which have different lengths.

Repair Works

The cross section of the reconstructed embankment is shown in Fig. 2. The light weight embankment and reinforced concrete wall have been constructed on a line moved to a distance of 16 m from the crest line of the embankment which failed, in order to avoid sliding along the same plane of failure. The application of sand drain procedure was difficult, and therefore the counterbalance method was adopted. The design items were as follows. Considering the decrease of soil strength due to remoulding, shear strength of the clay was assumed to be half the value before failure. Inside water level was 3 m below the datum level. Counterbalances were placed so that the safety factor became not less than 1·2 which corresponded to about 1·2 times the value at failure and also values in the original design. Repair work was completed in December 1958.

Conclusions

1. According to the analysis of the failure by the circular arc method the critical safety factor against large sliding is about 1·0, and the estimated arc of failure is very similar to the plane along which sliding actually occured. Therefore, this method can be applied in practice if soil strength is adequately determined.

2. For circular arcs of small length, the circular arc method gives rather lower values for the factor of safety so that it errs on the side of safety.

3. If the circular arc method of calculation is used, the estimation of seepage force by flow net may become complicated. If the seepage force is assumed to be the difference between the outside and the inside hydrostatic pressure as explained above, it gives nearly similar values for the factor of safety to those obtained by flow net for large arcs, and lower values for small arcs.