14 Mar 1991, 10:30 am - 12:30 pm

Liquefaction Potential of Sand Layers in Foundations of an Embankment Dam

Irfan Saeed
ACE, Kuala Lumpur, Malaysia

Proceedings: Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, March 11-15, 1991, St. Louis, Missouri, Paper No. 3.11

Liquefaction Potential of Sand Layers in Foundations of an Embankment Dam

Irfan Saeed
Project Manager - ACE, Kuala Lumpur, Malaysia

SYNOPSIS : The liquefaction potential of sand layers present in river bed alluvium underlying a major embankment dam was evaluated under design earthquake conditions. The results of the field and laboratory investigations and dynamic analysis carried out for this purpose are described in the paper. Recommendations made to ensure the post earthquake safety of the embankment are presented.

INTRODUCTION

It is known that during an earthquake there is a build-up of excess hydrostatic pressure in saturated cohesionless material due to applications of shear stresses induced by the ground motions. Soils, especially those consisting of loose saturated cohesionless material and located in seismically active areas, on which major engineering structures are to be founded must therefore be checked for their liquefaction and accompanying deformation potential. This paper describes such a study carried out as part of geotechnical design for a major dam project in Pakistan.

A part of the dam will be founded on the river bed consisting of alluvial sandy gravels in which clean fine sand layers have been found to be present at various depths. Evaluation of data obtained from field investigations and results of static and dynamic analyses carried out for embankment - foundation system are discussed in the paper.

It is concluded, on the basis of the above, that by removing a part of the upper most sand layer from below the upstream and downstream shoulder acceptable factor of safety and deformation levels can be achieved even under the Maximum Credible Earthquake (MCE) postulated for the Project.

DESCRIPTION OF THE MAIN EMBANKMENT DAM

The main embankment dam to be located across the river valley will be constructed mainly of sandstone fill obtained from the excavations required for the Project structures. A central impervious clay core will be provided for seepage control. In the river, the shoulders will be founded directly on the river bed while the clay core will be extended through the alluvium to the bedrock below in order to provide a positive cutoff against seepage. The dam will have a maximum height of 260 feet and total length of about 5000 feet.

The space between the main dam and upstream and downstream river coffer dams will be filled with excess material which will become available in abundant quantities from the "required excavations". In this way additional safety will be realised at very low costs.

FOUNDATION CONDITIONS - RIVER BED

Investigative drilling carried out in the river bed indicated the presence of sand layers interbedded in the coarser alluvial deposits consisting of sandy gravels with cobbles and boulders. These sand layers were considered to be more important for liquefaction potential analysis as compared to the coarser and more denser gravelly material and as such were subjected to additional investigations to determine their type, nature and aerial extent.

The method of investigations and the results obtained are described in detail by IRFAN (1989).

Out of the total of five sand layers identified only three, SL-1, SL-2 and SL-3, were considered sufficiently extensive to be of any consequence for the dam. The location of these layers in plan and section are shown on figure 1. Their gradation and particle size characteristics are shown on figure 2. The SPT 'N' corrected for method of test and equipment used and normalised to an energy ratio of 60 per cent for each of the three sand layer is as follows (IRFAN 1989).

Sand Layer	$(N1)_{60}$
SL - 1	14
SL - 2	21
SL - 3	18

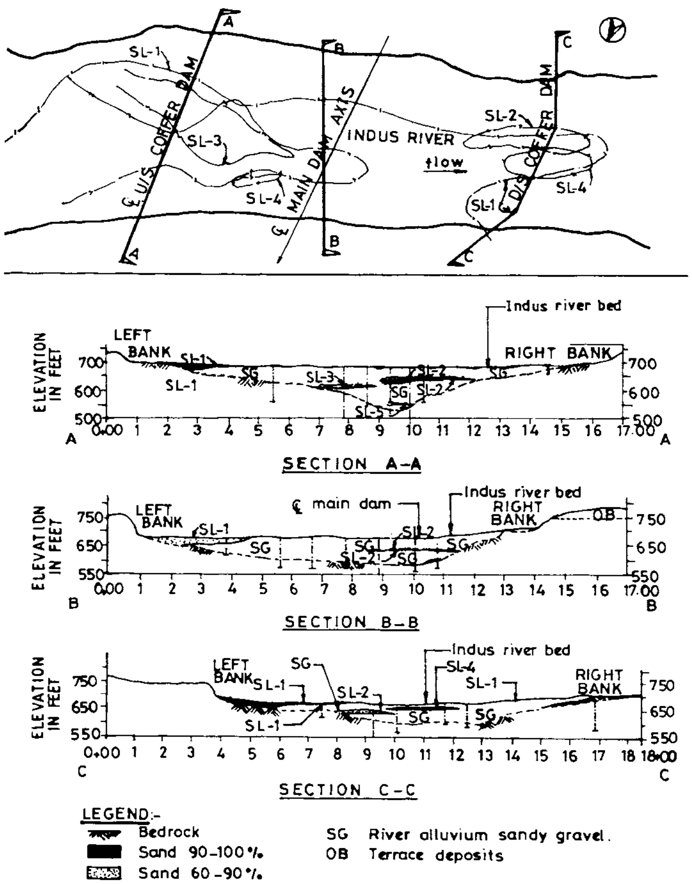

Fig.1. LOCATION OF SAND LAYERS IN RIVER BED.

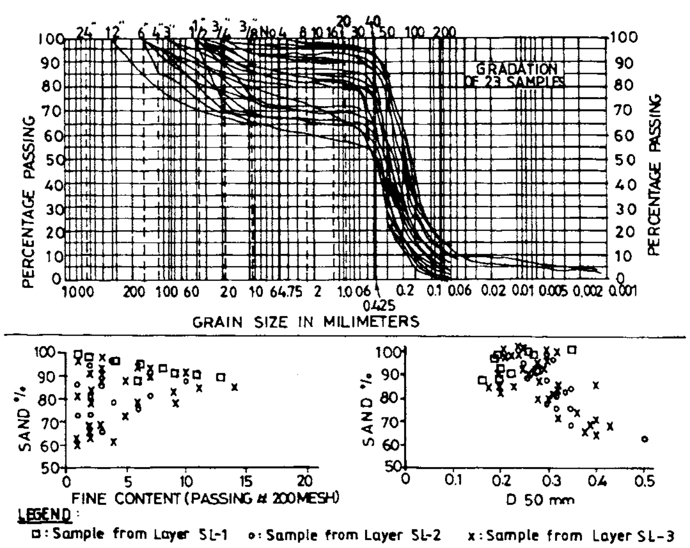

Fig.2. SIZE CHARACTERISTICS OF SAND IN RIVER BED

DESIGN EARTHQUAKE

The design earthquake for the Project was derived from a detailed study of the seismotectonic setting of the Project vis a vis location and characteristics of local and regional faults system and historical and recorded earthquake data. It was concluded that the Maximum Credible Earthquake (MCE) resulting from a Magnitude 7 earthquake occurring along an active fault at site will have the following characteristics.

Peak ground acceleration, Amax = 0.4 g
Duration of significant shaking= 30 sec

The time history for the Tabas Earthquake which occurred in Iran in 1976 under a similar seismotectonic environment was selected and after appropriate modifications, to take into account local characteristics, was used as seismic excitation input in the analysis. The design acceleration time history is shown on figure 3.

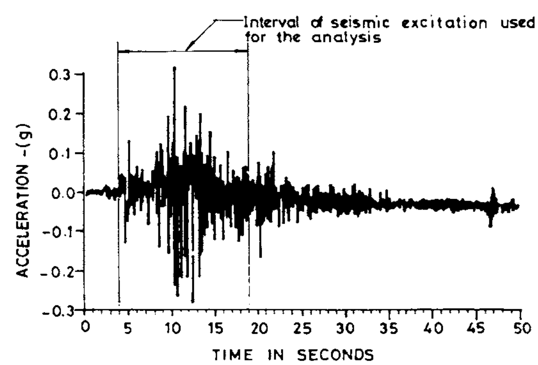

Fig 3. BASE MOTIONS ACCELERATION TIME HISTORY

F.E.M. Model

An idealised section representing the main embankment in the river, the weight berms between the main embankment and the river coffer dams and the foundation materials were modelled as shown on figure 4. It contained 192 four noded iso-parametric elements with a total of 222 nodes. The nodes at the base of the mesh were fixed and the lateral boundaries were supported by vertical rollers. The total width covered by the mesh was 3300 feet and it modelled the foundation down to EL 100. Plain-strain conditions were assumed throughout the mesh.

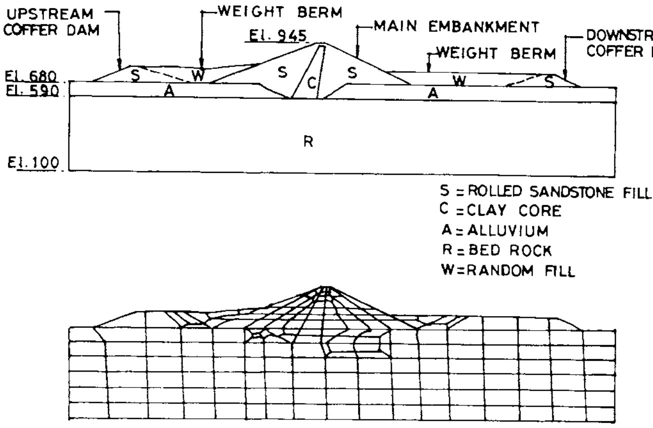

Fig.4. SECTION OF THE MAIN EMBANKMENT WEIGHT BERMS AND COFFER DAMS AND CORRESPONDING FINITE ELEMENT DISCRETIZATION

STATIC ANALYSIS

A static stress analysis simulating as closely as possible the construction sequence likely to be followed under prototype conditions was carried out by F.E.M. The sand layer SL-1 was assumed to lie along a horizontal plane connecting the upper set of gauss points of the main elements representing the river alluvium. Such a plane will be at a depth of 20 feet from the top of the river bed. Similarly, sand layer SL-3 was assumed to lie at the level of the lower set of gauss points of elements representing the alluvium i.e. 70 feet from top of the river bed. Sand layer SL-2 was assumed at an intermediate depth i.e. 45 feet from the top of the river bed.

The material behaviour was modelled by Mohr-Coulomb non-associative laws. Material properties and rules relating stiffness to out of plane effective stress were determined from field and laboratory tests.

The static analysis was carried out in a number of computer runs simulating the construction sequence.

CYCLIC STRESS RATIOS CAUSING LIQUEFACTION

Field performance data available for soil deposits which have or have not liquefied under known conditions of earthquake shaking have been correlated to standardised values of (N1) by Seed (1984). The correlation is however valid for M=7.5 earthquake and an effective vertical stress of $1 T/ft^2$. For a M = 7 event the correlation was adjusted by a factor for 1.1 as shown on figure 5. For adjustment to overburden effective stress other than 1 Ton /ft the following equation was used.

$$\frac{1}{C_a} = 0.85 + 0.15 \; \sigma v' \quad \ldots\ldots\ldots \; 1$$

where C_a = correlation factor
$\sigma v'$ = vertical effective stress

The limiting cyclic stress ratios calculated for the sand layers after making the above two corrections are tabulated below :

Sand layer	$(N1)_{60}$	Limiting stress ratio
SL - 1	14	0.17
SL - 2	21	0.24
SL - 3	18	0.18

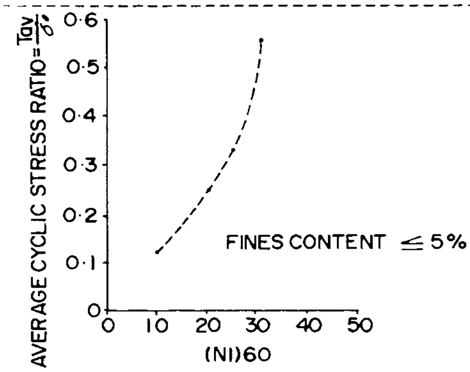

Fig.5. AVERAGE CYCLIC STRESS RATIOS CAUSING LIQUEFACTION Vs (N1) FOR M = 7.0 EARTHQUAKES

It has been found from research and actual observations that existence of initial shear stresses on the horizontal plane to which the dynamic stresses are applied causes a marked reduction in the rate of pore pressure build-up in sands having N1 greater than about 10. In the case under consideration the sand layers will be subjected to initial static shear stresses because of the non-level profile of the overlying fill. To account for this phenomenon the limiting value of shear stress that will cause liquefaction to occur in the sand layer was increased by a factor 'K' which is related to magnitude of initial static stresses by the following equation :

$$\frac{T_h}{\sigma_v'} = 2.514057 K^{0.251857} \quad \ldots\ldots \; 4$$

where T_h = total horizontal shear stress
σ_v' = effective vertical stress

The static shear and effective vertical stresses acting on the sand layers were found from static runs described above. These stresses were determined at the gauss points located at the corners of an element.

The limiting shear stress determined for each sand layer are shown on figure 6.

DYNAMIC ANALYSIS

The horizontal seismic excitation used for the dynamic analysis is shown on figure 3. It consists of a 15 second acceleration time history representing T = 4 to 19 seconds with the Tabas acceleragram scaled so that Amax = 0.3 g. The seismic loading was applied to the base of the FEM model used in the static analysis.

The vertical rollers at the lateral boundaries of the mesh were replaced by dampers together with horizontal concentrated loads equal to static reaction forces.

The reservoir water was represented by concentrated masses. Damping was assumed to be 3% of the critical for bedrock, 15% for sandstone fill and alluvium and 10% for clay core.

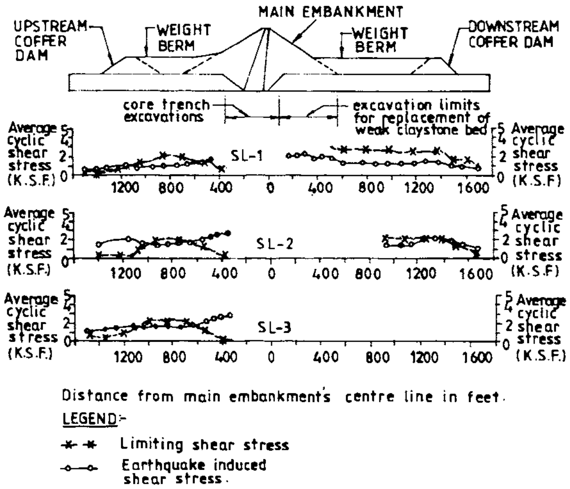

Fig.6. LIMITING AND EARTHQUAKE INDUCED CYCLIC SHEAR STRESSES IN SAND LAYERS

LIQUEFACTION POTENTIAL

Maximum cyclic shear stresses induced in the sand layer by the seismic excitation determined from the dynamic analysis were multiplied by factor of 0.65 to obtain equivalent cyclic shear stresses. These stresses were compared to the limiting shear stress values previously obtained as shown on figure 6. Liquefaction was implied at a point when the applied cyclic shear stress exceeded the limiting shear stress at that point.

It can be seen that :

a) liquefaction is likely to occur in the sand layer SL-1 in an area extending between 500 and 1100 feet from the centre line i.e. below the upstream weight berm and coffer dam and along its entire length below the downstream fill.

b) sand layer SL-2 will in all likelihood liquefy in an area between 700 and 1100 feet upstream of the centre line of the main embankment and along its entire length below the downstream fill.

c) sand layer SL-3 will also liquefy in an area 700 to 1100 feet upstream of centre line of the main embankment.

POST EARTHQUAKE SAFETY OF EMBANKMENT

To ensure post earthquake safety of the main embankment against sliding or deformation due to liquefaction of sand layers in its foundations the following criteria was specified.

Min. static F.O.S. against sliding

* upstream	1.2
* downstream	1.4
* for slips not involving the main dam	1.2

Allowable deformation (shear strain) involving main dam	20% of a 15 feet thick sand layer

For sake of conservatism the entire length of the sand layers were assumed to liquefy under MCE conditions. However sands will still have some residual or steady state strength after developing ru = 100% conditions. This strength will be mobilised after some limiting strain has developed. Such a condition is however transient as the sand on consolidation will rapidly regain static strength which is much higher than this residual value. The residual strength values applicable for the sand layers from available relationships (Saeed, 1984) are given in Table 2 below :

Table 2 - Residual strength of Sand layers Just after an earthquake

Sand layer	$(N1)_{60}$	Residual Strength (Psf)
SL - 1	14	600
SL - 2	21	2000
SL - 3	18	1200

POST-EARTHQUAKE STATIC STABILITY

A multiple wedge analysis based on limit equilibrium principles was performed to check the stability of the upstream and downstream slope of the embankment assuming that the strength of the foundation sand layers is at residual after liquefaction.

The MCE was assumed to occur at a time when the reservoir has been drawn down to the minimum operating level i.e. EL 825. Steady state seepage pore pressure were assumed in the clay core while pore pressure in hydraulic equilibrium with the reservoir level and a tailwater level of 705 were assumed in the upstream and downstream shoulder respectively.

Based on the results of the analysis the following observations can be made :

Upstream side :

1. The critical surface was found to be the one that cuts across the Main embankment, travels along the sand layers and finally daylights just upstream of the cofferdam. Any slip which comes up through the weight berm has a higher factor of safety. Similarly any slip that involves the cofferdam and the weight berm and their foundation have a higher factor to safety.

2. Using the undrained strength given in Table-2 the factors of safety for critical slips along sand layers SL-1, SL-2 and SL-3 and involving the Main embankment are about 1.09, 1.73 and 1.59 respectively.

3. The minimum F.O.S obtained for slips along sand layer SL-1 and SL-2 which will involve the cofferdam (and the weight berm) but not the Main embankment is 1.45 and 2.44 respectively.

4. Slip surfaces along a sand layer which is at a depth greater than about 20 feet from the top of the river bed will still have a factor of safety greater than unity even if zero strength is assumed for the sand. Slips along sand layers at shallower depths, however, will have a F.O.S lower than 1.

Downstream side :

5. Excavations necessary for replacement of weak claystone bed from below the downstream shoulder of the Main embankment at the left bank will also remove the surface sand layer SL-1 presently found deposited in that area.

   Therefore, slip surfaces involving the Main embankment and its foundation were found to have a higher F.O.S. as compared to those which are initiated in the weight berm or the downstream cofferdam and travel along the sand layer SL-1 remaining in the foundations.

   Similarly sand layer SL-2 is present only in a limited area extending about 500 feet from the toe of the downstream cofferdam below the weight berm. Therefore the critical surface along it will also not involve the Main embankment.

6. Using the undrained strength given in Table-2 the minimum calculated F.O.S for slip along sand layer SL-1 and SL-2 involving the Main embankment is 1.93 and 1.70 respectively. While for slips not involving the Main embankment the minimum F.O.S obtained along sand layers SL-1 and SL-2 is 1.22 and 1.93 respectively.

7. It is possible that at the time of construction the location of the Sand layer SL-1 would have shifted such that all or a part of it lies outside the limits of excavation necessary to remove weak claystone beds. In such a case it will be present below the downstream shoulder of the main embankment and the factor of safety for the critical slip along this sand layer will be about 0.95.

8. Factor of Safety greater than unity will still be available even if zero strength is assumed in the sand layers. However, in the case of 7 above it will only be available for a sand layer deeper than about 20 feet from the top of the river bed.

Deformations :

It has been found that loose sands having (N1) less that about 10 undergo very large deformations when the residual pore pressure ratio attains a value of 100% under cyclic loading conditions. Medium dense sand, however, undergo a limited amount of cyclic strains.

The limiting shear strains in sand can be related to their (N1) value as shown in table 3 below :

Table 3 - Limiting Shear Strains in sand layers

Sand layer	$(N1)_{60}$	Limiting shear strains
SL - 1	14	30%
SL - 2	21	15%
SL - 3	18	20%

Because of the great depth of Sand layers SL-2 and SL-3 from the top of the fill surface no large surface manifestations are expected as a result of their liquefaction. However deformations resulting from liquefaction of sand layer SL-1 exceed the permissible limits set out in the criteria for post-earthquake safety.

CONCLUSIONS

1. Below the upstream shoulder of the dam adequate factors of safety are available against sliding along sand layers SL-2 and SL-3. Factors of Safety greater than unity are still available even if there is complete loss strength of the sand in these layers. The deformations expected in these sand layers on liquefaction is also within the limits.

2. In the case of the surface sand layer SL-1 below the upstream shoulder the factor of safety against sliding is low and the expected deformations excessive.

The situation can be improved quite effectively by removing the sand layer from below the upstream shoulder, over a length necessary to achieve the required factor of safety, and replacing it with a well compacted sandstone fill. Such replacements will be more effective at the toe of the Main embankment.

3. The replacements mentioned above will however prevent any significant shearing strains from developing in the part of the sand layer remaining below the shoulder on liquefaction. Without the shearing strains the residual strength can not be mobilised and therefore the amount of replacements have to be calculated using zero strength for the part of the sand layer remaining below the shoulder. Analysis show that about 150 feet of removal of sand layer SL-1 measured from the toe of the Main embankment will be sufficient to achieve a Factor of Safety of 1.2.

Similarly about 200 feet of excavation of Sand layer SL-1 will be sufficient at the downstream toe of the Main embankment to achieve the required factor of safety if it is found to be present below the dam during constructions. No excavations will be necessary (apart from that required for replacement of weak claystone beds) if the present conditions remain unchanged.

4. Material obtained from these excavations can be used in the constructions of the weight berm so haul distances will be short.

5. No excavations will be necessary below the weight berm and the upstream Coffer dams as adequate factors of safety are available against sliding along the sand layers.

REFERENCES

IRFAN SAEED (1989) :

Drilling and penetration testing of Indus river alluvium- paper presented at The Third National geotechnical Conference PNSSMFE Lahore Oct 1989.