THE INFLUENCE OF A FREE-DRAINING ZONE ON THE SEISMIC DEFORMATION OF THE LA HONDA DAM

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SUMMARY

The La Honda Dam, in Venezuela, is designed for a maximum horizontal ground surface acceleration at the dam base of 0.5 g. The shells of the dam are being constructed of densely compacted sand derived from locally available poorly-cemented sandstones. The cyclic strength of this sand, compacted to densities between 96 and 100% of the Modified Proctor density, is presented and compared to the strength of cohesionless materials used in other embankment dams. Dynamic finite element analysis of the embankment, indicated a limited area with strain potentials of up to 15% under the upstream slope of the dam. The use of a 20 m wide free-draining zone within the upstream shell was investigated and shown to reduce strain potentials and related crest deformations. The free-draining zone was shown to have little effect on post-earthquake upstream slope stability.

THE URIBANTE DORADAS PROJECT

The Uribante Doradas Project is located in western Venezuela in the Andean mountain system near the border with Colombia (Figure 1). The project will develop the water resources of the Uribante, Doradas, Camburito and Caparo Rivers which lie at progressively lower elevations and follow parallel courses coinciding with the alignment of the Andean mountain system. Reservoirs with interconnected tunnels will be created on the three uppermost rivers. Electrical energy will be generated at three locations by the progressive conveyance of water from the uppermost reservoir to the lowest river. The project is owned by the Compania Anonima De Electrificacion Y Fomento Electrico (CADAFE), a Venezuela government-owned company. The designer is CEH Ingenieros Consultores, C.A. with Harza Engineering Company as consultant.

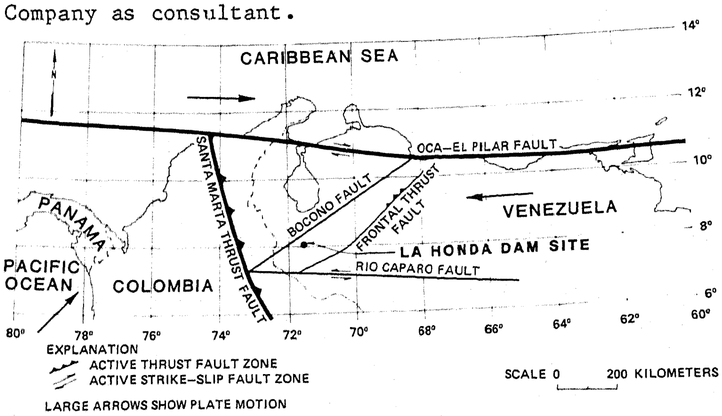

Figure 1 LOCATION PLAN AND NEOTECTONIC MAP

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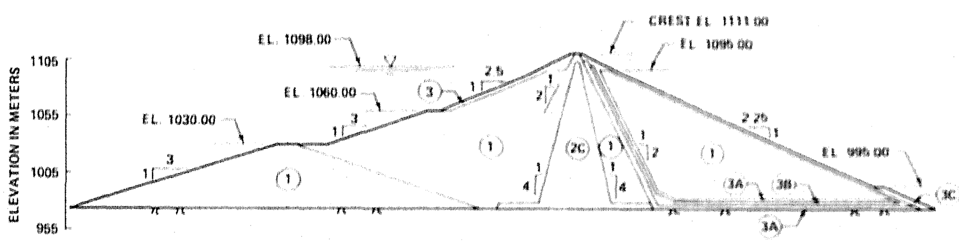
The site of the La Honda Dam on the Uribante River, is approximately at El. 987m. The climatic conditions are cool and wet with an average daily temperature of 19°C and an average annual precipitation of about 4,000mm. Construction of the La Honda Dam began in 1979 and is scheduled for completion early in 1984.

GEOLOGIC SETTING OF THE LA HONDA DAM SITE

The dam is founded in a deep valley formed by a 50m wide streambed, a left abutment with 1.3H on 1.0V average slope, and a right abutment with 1.8H on 1.0V average slope. The left abutment consists of stratified sandstones and shales of the Rio Negro formation of Lower Cretaceous age. The Rio Negro sandstone is generally a homogeneous, poorly-cemented soft rock. Rock fractures are not continuous over long distances but are either open or filled with clay or loose sand. The physical properties of this rock and existing fractures have thus rendered the left abutment pervious and erodible. The right abutment consists of alternating beds of calcareous sandstones, limestones and shales of the Aguardiente Formation of Lower Cretaceous age. The sandstones are considerably harder and less friable than those of the left abutment. During exploration core recovery was poor, and water losses were appreciable. For these reasons an extensive foundation treatment system was developed to prevent piping of foundation materials into the foundation.

DESIGN OF DAM

The La Honda Dam (Figure 2) will be 140 m in height, 600 m long at the crest and will require 9.2 million cubic meters of earth fill materials for construction. The 10.0 m wide crest of the dam at El. 1111.0 provides a maximum camber of 1.5 m and a normal operating freeboard of 13.0 m. The dam will impound a reservoir 11.0 km in length and 775 million cubic meters in volume at the spillway crest elevation, 1098. Reservoir levels will be controlled by a left abutment ungated chute spillway and a low level outlet which will also be used for controlled filling of the reservoir.



ZONE	MATERIAL TYPE
1	RIO NEGRO SAND , SW–SM
2C	CLAYEY SAND , CL–SC
3	RIVER SAND,GRAVEL,COBBLES
3A	FILTER
3B,3C	DRAIN, GP

Figure 2 MAXIMUM HEIGHT CROSS SECTION OF LA HONDA DAM

CONSTRUCTION MATERIALS

Rio Negro sandstone, primarily obtained from required excavation, is the principal construction material. This poorly-cemented sandstone readily breaks down to a sand that has a maximum Modified Proctor dry density of approximately 1.9 ton/m^3 and an optimum moisture content of 9-12%. During construction, field densities have averaged 98% of maximum Modified Proctor dry density. Filter materials are produced from river sand, gravel and cobbles.

The clayey sand, obtained from borrow and placed in the core, is a well graded, low plasticity clayey sand with a PI of 13%. Standard Proctor test results indicate a maximum dry density of about 1.8 ton/m^3 at an optimum moisture content of 14.6%. The core material is compacted to a minimum dry density of 95% of the Standard Proctor maximum dry density. The compaction moisture content has varied between 0.5% below optimum to 2.5% above optimum. Gradation curves for the crushed sandstone (hereafter called Rio Negro sand) and the clayey sand are shown in Figure 3.

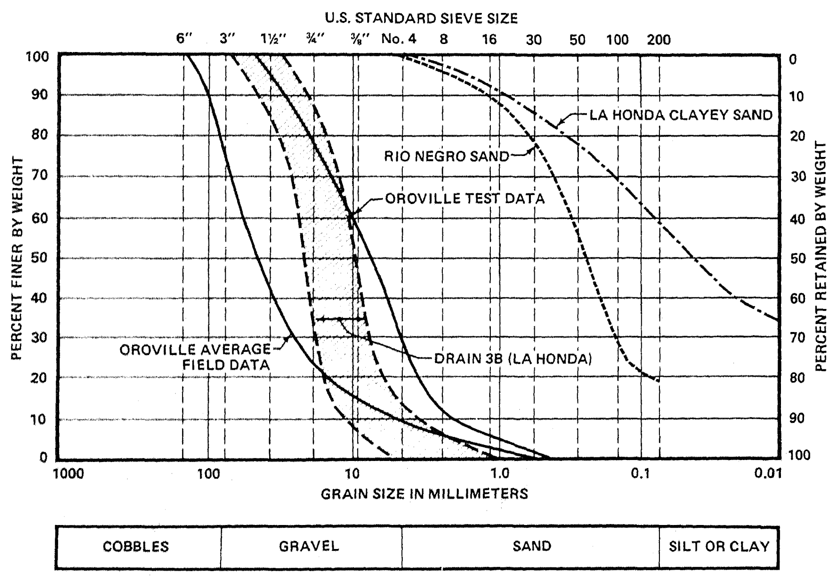


Figure 3 GRADATION CURVES

DESIGN EARTHQUAKE

The postulated earthquake used for the design of the La Honda Dam is a magnitude 8-1/4 event occurring on the Bocono Fault (Figure 1) with the epicenter located about 20 km from the damsite. This earthquake could generate rock motions having a peak horizontal acceleration of about 0.48 g and a duration of about 60 seconds. Dr. H. B. Seed, a member of the Board of Consultants, recommended that the synthetic "Seed-Idriss" record scaled to a peak acceleration of 0.50 g, be used to represent the magnitude 8-1/4 event.

MATERIAL PARAMETERS FOR FINITE ELEMENT ANALYSIS

The finite elemert analysis of the La Honda Dam required determination of soil parameters for non-linear stress-strain, strength, permeability, and energy absorption characteristics of materials in the dam and foundation. These parameters were determined from laboratory static and dynamic triaxial tests, one-dimensional consolidation tests and permeability tests. Field tests were also performed to help determine in-situ properties of embankment and foundation soils.

Of particular interest are the results obtained for the numerous cyclic strength tests conducted on the Rio Negro sand (Ref. 1). The cyclic triaxial strength test program was designed to investigate the effects of specimen density, confining pressure and anisotropic consolidation on the cyclic strength of dense sand. Test results are shown on Figures 4 through 7.

A typical plot of the cyclic stress ratio versus number of cycles to reach 100% pore pressure response, 5% and 10% double amplitude strain, for specimens isotropically consolidated to 2.0 kg/cm^2, are plotted on Figure 4. It may be seen that 100% pore pressure response and 5% double amplitude strain occurred at approximately the same number of cycles and that 10% double amplitude strain values were reached at a slightly higher number of cycles. The relationship between the number of cycles to reach 100% pore pressure response, 5% and 10% double amplitude strain shown in this figure was typical.

Cyclic strength values for specimens compacted using moist tamping procedures at 98% of modified Proctor density and isotropically consolidated at various confining pressure, are summarized in Figure 5. The figure plots peak cyclic shear stress versus the number of cycles to cause 10% double amplitude strain. It may be seen in the figure that large values of peak cyclic shear stress were required to cause deformation in the specimen and that the cyclic shear strength increases considerably with confining pressure.

The effect of anisotropic consolidation on the cyclic strength of dense sands is shown in Figure 6 as the peak cyclic shear stress required to cause 10% double amplitude strain for specimens consolidated to a minor effective principal stress of 5.0 kg/cm^2 for K_c values of 1.0, 1.5 and 2.0. The figure shows that isotropically consolidated specimens show slightly lower strength than anisotropically consolidated specimens, but that anisotropic consolidation ratios of 1.5 and 2.0 give essentially the same cyclic strength.

The effect of specimen density on cyclic strength is shown in Figure 7, which plots the cyclic stress ratio versus number of cycles to cause 10% double amplitude strain for specimens compacted using a moist tamping procedure to densities between 96% and 100% of maximum Modified Proctor density. The large increase in strength with increasing values of density at high percent of Modified Proctor density is clear. This effect may have been magnified because of the uniform grading of the sand which gives a large variation in relative density values for a small percent difference in maximum Modified Proctor density values. Nonetheless, because the Modified

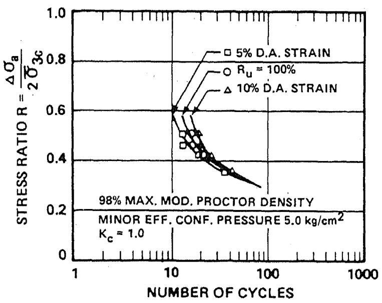


Figure 4 STRESS RATIO VERSUS NUMBER OF CYCLES

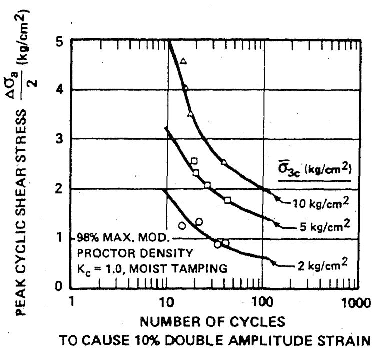


Figure 5 EFFECT OF CONFINING PRESSURE ON CYCLIC STRENGTH

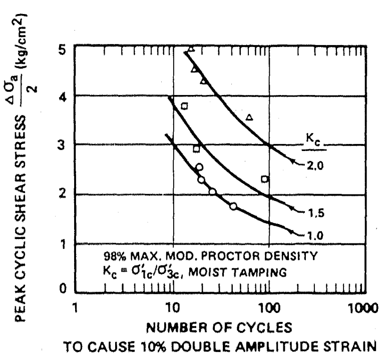


Figure 6 EFFECT OF ANISOTROPIC CONSOLIDATION ON CYCLIC STRENGTH

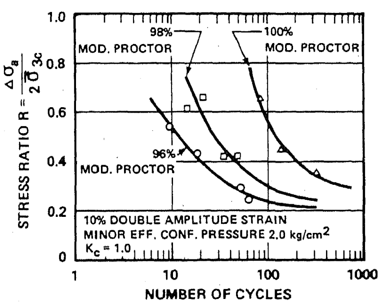


Figure 7 EFFECT OF COMPACTED DENSITY ON CYCLIC STRENGTH

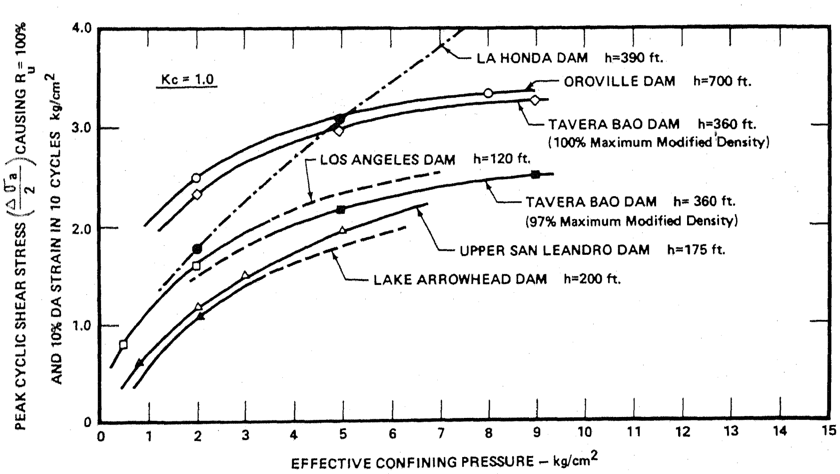


Figure 8 COMPARISON OF CYCLIC STRENGTH OF RIO NEGRO SAND TO OTHER COHESIONLESS EMBANKMENT DAM MATERIALS

Proctor density test is a useful measure of field compaction of sand, it appears that a great increase in cyclic strength can be achieved by a modest increase in density when sands are compacted to a dense state in the field.

The data presented in Figures 4 through 7 provides a comprehensive picture of cyclic strength of Rio Negro sand compacted to densities on the order of 98% of Modified Proctor density. It is interesting to compare the strength of the dense Rio Negro sand with the cyclic strength of other cohesionless materials. This has been done in Figure 8 by plotting the peak cyclic shear stress required to cause 10% double amplitude strain in 10 cycles for a number of embankments constructed of cohesionless materials. It may be seen that the Rio Negro sand compacted to high values of percent maximum Modified Proctor density is extremely strong and has strength values equivalent to or greater than Oroville Dam shell materials. Rigorous analysis of the Oroville Dam has shown it to be safe under strong values of ground shaking. Thus, it is reasonable to conclude that by compacting the Rio Negro sand to high density values, similar acceptable cyclic performance of the dam may be achieved.

UPSTREAM SLOPE STABILITY DURING AN EARTHQUAKE

The dynamic finite element analysis (Ref. 2) indicated the overall dynamic stability of La Honda Dam to be satisfactory for the maximum credible earthquake i.e., a magnitude 8-1/4 event with epicenter 20 km from the dam site. The analysis also indicated a limited area with strain potentials of up to 15% under the upstream slope (Figure 9). Concern that such strain potentials could lead to cracking and excessive crest deformation led to a study of ways by which the strain potentials could be reduced.

Two methods were investigated to reduce the strain potentials and thus improve the seismic stability of the upstream shell (Ref. 3). Both methods involved the replacement of embankment sand fill with a free-draining material that would rapidly dissipate the excess pore water pressures responsible for the high strain potentials. Method 1 would completely replace a portion of the upstream shell sandfill with free-draining material. Method 2 would introduce horizontal layers of free-draining material with vertical spacing sufficiently small to insure that excess pore water pressures are dissipated rapidly.

It was found that Method 2 would require a drain spacing of 2 m or less in order to dissipate 50% excess pore pressure in less than one minute, a maximum time for controlling seismically induced pore water pressure. Installing drains at such close spacing, and the need for a filter between the drain and sandfill, made this option uneconomical and impractical. The studies were therefore directed toward using a free-draining zone (Method 1). The extent of the free-draining zone, to either the 5%, 10%, or 15% strain potential contour, was based on the benefit of the free-draining zone in reducing pore water pressures and strain potential, and in increasing dynamic slope stability of the upstream shell.

The essential requirement of a replacement material for use in a free draining zone is its high drainage capacity. Of the materials used for construction, the 3B drain material with a permeability of 1 cm/sec meets this requirement. The gradation of this material, manufactured from river sand, gravel and cobbles, is similar to that of the Oroville gravel reported by Banerjee, Seed and Chan (Ref. 4 and Figure 3). Since limited test data was available for the La Honda 3B material, the static and dynamic material properties for the Oroville gravel were used in the analysis. In addition, a set of strength curves representing the material resistance at 10 cycles for an earthquake of magnitude 8-1/4 was derived for the 3B drain material from the Oroville data.

For the analysis it was estimated that a 20 m wide replacement zone (i.e. to the 10% strain potential contour) would effectively be drained during the maximum credible earthquake such that the drained strength would be mobilized within this zone. However, outside this zone where the permeability is about 1 x 10^{-4} cm/sec., the embankment was considered to be undrained during the period of earthquake shaking, and undrained strength was used.

Using the above described and referenced material properties and shell drainage assumptions, the La Honda dam cross section with a 20 m wide free-draining zone was analyzed by finite element techniques. Figure 9 shows the strain potential contours of the dam with a 20 m wide free-draining zone. On the same plot, a set of dashed lines represents the contours for the shell without a free-draining zone. The 20 m wide free-draining zone is shown to effectively reduce the strain potential in the shell from 15% to less than 10%. However, the 5% strain potential contour remains at about the same location in the shell with or without a free-draining zone. The strain potential in other parts of the dam was found to be unaffected by the inclusion of the free-draining zone.

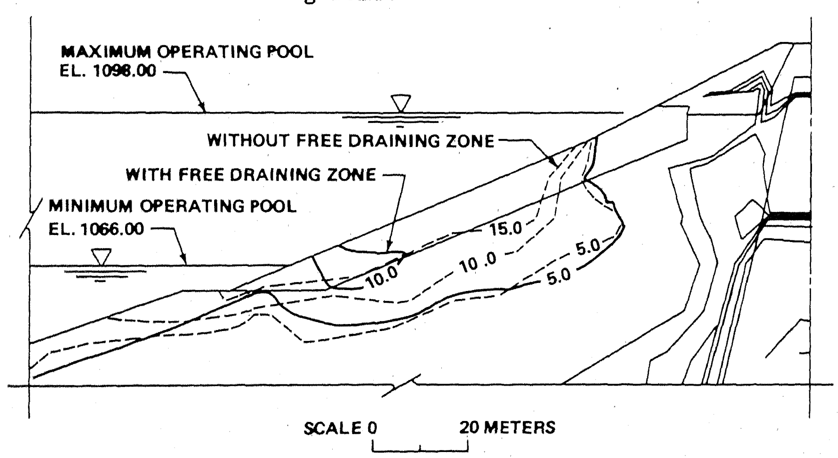


Figure 9 COMPARISON OF STRAIN POTENTIAL IN THE UPSTREAM SHELL

The analysis indicated that the maximum permanent deformation of the dam with the free-draining zone after earthquake shaking could be as much as 2.4 meters horizontally, resulting in crest settlement of about 1.4 meters. After earthquake shaking the factor of safety against sliding in the upstream shell reduced from 1.93 to 1.63. Table 1 presents the above results along with those obtained for the sandfill cross-section for comparative purposes.

Table 1

Deformations and Stability

Case	Estimated Horizontal Deformation m.	Estimated Vertical Deformation of Crest, m.	Factor of Safety Before Earthquake	Factor of Safety After Earthquake
With Free-draining zone	2.4	1.4	1.93	1.63
Without Free-draining zone	3.2	1.9	1.93	1.63

The analysis of the upstream shell with a 20 m wide free-draining zone indicated that the zone would effectively reduce excess pore water pressures and strain potentials within the upstream shell during and after a magnitude of 8-1/4 earthquake with epicenter 20 km from the dam. The analysis also indicated (Table 1) that the surficial (20 m wide) free-draining zone would have little effect on the post-earthquake stability of the upstream slope, but that crest deformation could be reduced from about two meters to 1.4 meters by inclusion of the free-draining zone. With a design free board of 13 m above normal operating pool, adequate freeboard would remain for a crest deformation of these amounts. Comparison of the estimated horizontal deformations for the all sandfill section and the modified section with published deformations for the Oroville Dam (Ref. 4) for a 8-1/4 magnitude earthquake indicated good agreement. The results of the analysis indicated that the benefits gained by the replacement zone as compared to its cost of approximately 2.5 million dollars are marginally attractive. It was therefore decided not to modify the design by incorporation of the free-draining zone because:

1. The permanent crest settlement and horizontal displacements estimated for the existing design are acceptable and within the range of values estimated for other high dams under similar earthquake loadings.

2. The overall dynamic stability of the dam is adequate with or without a free-draining zone.

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