INTERIM TECHNICAL REPORT #68

VERIFICATION OF HLA SOILS WORK

SUPPLEMENTARY INFORMATION

Contents - Pages 26, 26a, 27, 28, 30, 59, 98 and 100

The addenda and minor errata are marked with side bars. In addition, these changes have been discussed with and approved by Dr. R. McNeill, Mr. R. Wray and Prof. M. Holley.

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Approved P105-4-839-086

A portion of the sliding resistance is due to simple friction between the concrete and the rock; but, because the bottom of the building is at different elevations, a portion of the sliding resistance is due to shear strength in the rock. The friction portion of the calculation is direct, using a concrete/rock friction coefficient of 0.6 with the minimum vertical normal force.

The required strength of the rock to supply the remaining sliding resistance was then computed by assuming a uniform rock shear strength to act over the remaining area. The result shows that a rock shear strength of 8.3 ksf would be required, as shown in Table 5.

The required shear strength of 8.3 ksf is considered to be conservative by the IDVP for the following reasons:

Senior HLA geologists and engineers visited the excavation at final grade, and prepared memos to file of those visits, recording their observations and recommendations, and presenting data from tests in the excavation (References 36, 37, and 38). Reference 37 records that the exposed rocks at the bottom of the excavation were blocky and massive, and that they were hard and strong. Those terms have strictly defined meanings in HLA technical standards. Hard means that the rock can be scratched with a knife or pick with difficulty and the resulting scratch produces little powder and is often faintly visible; and strong means that a specimen will withstand a few heavy hammer blows before breaking into large fragments.

The reports of Reference 37 clearly stated that blasting was necessary in order to excavate.

Finally Reference 38 indicated that the rock in the Intake excavation was more competent than in the other excavation site, based upon observations and professional opinion of the senior geologist.

In addition, Reference 36 reports that four holes were drilled within the periphery of the intake structure foundation after excavation was complete. These drillings were made to a depth of 30' below final grade to ensure the rock was solid. Based upon observation of the drilling process and the cuttings, rock below grade is uniform and consistent, and no evidence of clay seams was observed. Weathering was slight.

HLA also ran two seismic lines in the excavation, determining that the compressional velocities along those lines ranged from 5,000 to 14,000 fps, and HLA characterized the average compressional velocity as 7,000 fps.

These values agree well with the observation that blasting was required in the excavation. According to data published by the Caterpillar Company (Reference 20), rock which has to be blasted rather than being ripped by a large tractor (D-8), would have a P-wave velocity of 6,000 fps or greater. According to work by Deere (Reference 21), a rock with that velocity would have uniaxial compression strength of at least 220 ksf. Shear strength is half that value, if internal friction is neglected. Introduction to Rock Mechanics (Reference 22) quotes a lower value of 167 ksf for shear strength of a sandstone. These values are significantly above the required 8.3 ksf shear strength.

Thus, from a number of viewpoints, it is clear that the rock is capable of supplying the necessary strength to resist the sliding forces with an adequate safety factor. Therefore, the IDVP considers the intake structure to be qualified against sliding.

The NRC has mentioned the possibility of a westerly dip of the bedrock under the structure. The bathymetry offshore is not steep, so for a bedding plane to be adverse, it would have to be nearly horizontal. Therefore, for simplicity, a horizontal plane is assumed. The strength properties on a plane located at such depth would be no less than the strength of weathered materials. That is, it should be very conservative to assume c = 3 ksf and phi = 30 degrees. It is also conservative to assume that the bedding plane would fail just under the structure footprint. Considering the conservatisms of the DCP analysis, the IDVP accepts the DCP factor of safety value of 1.3, and believes it to be a conservative one for sliding on a postulated adverse bedding plane.

4.6 CONCLUSIONS

The IDVP has reviewed the HLA/DCP calculations for computation of the stability factors of safety and supplemented its review with alternate analyses.

The IDVP found all sliding forces computed by HLA/DCP to be acceptable, with the exception of the dynamic soil pressure. The IDVP determined the sliding forces due to dynamic soil pressure to be 14,500 kips versus 27,666 kips computed by the DCP using the Mononobe-Okabe method. The IDVP then added its value of sliding force due to dynamic earth pressure to the sliding force due to the structural response on an absolute sum basis instead of the SRSS basis used by the DCP. The net result was a total sliding force of 93,809 kips (see Table 5), as compared to the DCP value of 75,515 kips.

The IDVP considered the DCP assumption that a shear failure plane would occur only throughout the bedrock to be inappropriate. The IDVP considered the total resistance to consist of:

(1) Shear fracture in a specific area of the bedrock due to the presence of shear keys in that area (see ITR #40, Reference 8)

(2) Friction at the concrete/rock interface over the remaining contact area

(3) Friction between the north and south concrete walls and soil.

The IDVP computed the rock shear strength required to attain a factor of safety of 1.1 against sliding. The NRC Standard Review Plan, Section 3.8.5, specifies a minimum factor of safety of 1.1 against sliding and overturning. This is for a load combination consisting of dead load, lateral earth pressure, and safe shutdown earthquake (Hosgri). The shear strength value required was 8.3 ksf, which is below published values for the types of rock below the intake structure. The IDVP also found the factor of safety to be greater than 1.3 for a shear failure plane entirely within the bedrock. Thus, the IDVP found the intake structure to have an acceptable factor of safety against sliding.

. The IDVP determined the factor of safety against overturning and bearing pressure. For overturning, the factor of safety was computed to be 1.4. The IDVP found the maximum bearing presure to be 11.5 ksf. A shear strength of 8.3 ksf is equivalent to an ultimate bearing strength of 42 ksf; thus the factor of safety is 3.5, which is conservative. The IDVP therefore concluded that the intake structure meets licensing criteria with respect to structural stability.

5.0 DIESEL FUEL OIL TANKS

5.1 DESCRIPTION

Two diesel fuel oil storage tanks (DFOs) are located about 40 feet west of the turbine building. Each tank is approximately 10.5 feet in diameter and 63 feet long. The tank is a 3/8 inch shell with 3x3x3/8 intermittent angle stiffeners welded to the shell. The tanks have ASME 9/16 inch thick ellipsoidal heads. Each tank has a 40,000 gallon capacity. Figure 10 shows details for the diesel fuel storage tanks.

5.2 LITHOLOGY

5.2.1 DCP Evaluation

Extensive excavation at the site removed most of the rock to below the full buried depth of the tanks. Then the area was brought back up to grade by mass filling. Thus the trenches were excavated into both rock and mass fill. A concrete slab was placed at the bottom of the trench and bedding sand was laid to accommodate the shape of the tank. The tank was then placed on the bedding sand, and backfill was added and compacted. Figures 11 and 12 show sections illustrating the material and profiles.

5.2.2 IDVP Assessment

HLA performed static and dynamic tests to determine the backfill properties and relied upon dynamic tests by others for the dynamic properties of the rock. HLA assumed conventional properties for the bedding sand and the concrete pad. The IDVP agrees with the HLA approach in choosing properties and evaluated the HLA determination of soil property values.

The rock properties selected by HLA are expressed as compressional wave velocity, which is a function of elevation above MSL for this area, as shown in Figure 15. The values are in the range of those expected for rocks of these types with these overburdens. The IDVP concurs with the HLA values.

5.3 REVIEW OF DCP METHODOLOGY

The HLA analysis of the DFO tanks was composed of static and dynamic analyses. The static analyses consisted of the formulation of a model to represent the tank and surrounding soil backfill inside the trench. Stresses were evaluated for each step of the construction sequence, and the final states of stress for the completed backfill operation were later added to the dynamic results. Further details of the HLA procedure are given in Section 5.4.1. The IDVP finds the DCP methodology for the static analyses to be acceptable.

The dynamic analyses consisted of examining the tank/soil system for both horizontal and vertical seismic excitation. The DCP used the FLUSH computer program to analyze a transverse section taken through the two DFO tanks. Dynamic soil/rock properties were determined by HLA and assigned to the model elements. HLA analyzed four models used to represent the transverse section of the DFOs. The fluid was modeled using solid elements. Maximum responses from the horizontal and vertical analyses were combined on an absolute sum basis. The total dynamic forces and moments in the tank wall elements were then added absolutely to the static stresses.

The IDVP finds the DCP procedure for separating the dynamic analyses of the DFO tanks into horizontal and vertical excitation acceptable. Though the behavior of the soil is non-linear and results are superimposed as if a linear analyses had been performed, the conservatism of adding nonconcurrent maximum responses compensates for this. The IDVP noted several parameters which required further study within the dynamic analyses, and these are discussed in Section 5.5.

Sliding Forces	Force (Kips)
Structure Response[a]	23,988
Pedestal + Base @ ZPA[b]	35,502
Dynamic Soil Pressure	14,500
Surcharge	867
Dynamic Water Pressure	9,254
Static Soil Pressure	9,698
	93,809
Resisting Forces	
Friction Force	9,129
Shear Fracture	89,070
Skin Friction of Soil along North and South Walls	4,991
	103,190

F.S. = 1.10 Minimum. This F.S. was computed using a shear strength for rock of 8.3 ksf.

Notes:

1) Values for items a and b were determined by the DCP and accepted by the IDVP.

2) IDVP considered shear fracture to occur in the eas adjacent to the shear keys. Friction resista at the concrete/rock interface was considered r the remainder of the contact area.

3) The shear strength of 8.3 ksf consists of the total of the cohesion and shear friction components. As mentioned in Section 4.5, the IDVP considers the value of 8.3 ksf for shear strength and resultant factors of safety of 1.10 to be conservative.

Table 5

IDVP Factor of Safety Against Sliding
Intake Structure

Reference No.	Title	RLCA File No.
16	IDVP Design Review of DCP Calcultion #OT-1.0, Revision 0, 1, 2.	P105-4-506-176
17	DCP Calculation #8324-04-CA-03, Intake Structure Member Evaluation, Revision 0.	P105-4-431-108
18	DCP Response to RLCA RFI #837 May 20, 1983.	P105-4-441-121
19	Water Pressures on Dams During Earthquakes, Westergaard, H. M., Transactions of the American Society of Civil Engineers, Vol. 98, 1933	
20	Caterpillar Handbook of Ripping, Caterpillar Company, Peoria, Illinois, December 1966	
21	Engineering Classification and Index Properties for Intact Rock, D. U. Deere et. al., University of Illinois - Urbana, Illinois, December 1966.	
22	Introduction to Rock Mechanics, Goodman, Wiley and Sons, 1980.	
23	Soil Mechanics In Engineering Practice, Terzaghi and Peck, Wiley and Sons, 1967, Figure 33.4.	
24	IDVP Design Review of HLA 1978 and 1982 Analyses of Diesel Oil Tanks, Revision O.	P105-4-506-186
25	IDVP Design Review of Diesel Fuel Oil Tank Ellipsoidal Heads, Revision 0.	P105-4-506-177

Reference No.	Title	RLCA File No.
34	Geotechnical Studies, DFO Storage Tanks - DCNPP, Harding Lawson Associates, August 19, 1983,	P105-4-499-127
35	IDVP Design Review of DCP Correction Action for DFO Tanks (1983 Analysis) Revision 0.	P105-4-506-219
36	HLA Memo by Steve Korbay, Intake Structure Excavation Inspection, Feburary, 17, 1972	P105-4-449-128
37	HLA, Memo to file by Steve Korbay, Site Inspection. Intake Structure Diablo Canyon, March 2, 1972	P105-4-449-129
38	HLA, Memo to File by Henry Taylor, PG&E, Diablo Canyon Site. Visit on March 3, 1972, Memo Dated March 16, 1972	P105-4-449-130