SEISMIC DESIGN CONSIDERATIONS FOR A SURFACE SUPPORTED SURGE TANK

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SUMMARY

This work summarizes the most important considerations regarding the seismic analysis and design of an 88m high, 18m diameter, surface supported steel surge tank for the Corobici Hydroelectric Project in Costa Rica. The tank is located in an active seismic zone and it is the most critical structure of the project. The paper presents the alternative structural forms considered and the criteria for selecting the design parameters, mainly seismic and hydraulic, and their joint probability of occurrence. The analysis procedure and some special structural details are commented as well.

INTRODUCTION

The Corobici Hydroelectric Project was constructed by the Instituto Costarricense de Electricidad (I.C.E.), as the second stage of the Arenal-Corobici Hydroelectric generation system, using the waters from Lake Arenal. The intake water level, located 8170m away from the tank in the Arenal Power House, is at 328 meters above sea level (masl) whereas the Power House is at 90masl. With a 238m of water height and a maximum flow of 97,5m^3/s, its three Francis turbines have a capacity of 175MW.

Due to hydraulic and topographic reasons, the penstock is not located underground, therefore the tank is not partially embedded--the usual situation --but surface supported. It is located on a mesa at level 265masl. Hydraulic considerations defined the maximum water level of the tank and its diameter above minimum water level as 355masl and 18m, respectively. The foundation level is at 258masl.

The tank was designed and constructed by Kawasaki Heavy Industries Inc. (K.H.I.), of Japan. The author participated as a consultant for I.C.E. in the preliminary design, definition of the design parameters, specification of the analysis and design requirements and review and approval of the design process.

STRUCTURAL FORM

As mentioned, tank height and diameter above the minimum water level (301masl), were predefined conditions. Besides, use of concrete--a local product--was encouraged by I.C.E. officials in order to save some steel, which had to be imported.

Several alternatives were considered in the preliminary design process. The first (Fig. 1.a), was a steel cylindrical tank with a hemisphere at the bottom, at level 287masl, supported by a reinforced concrete conical shell, with a 4.4m diameter pipe connecting to the penstock. This solution presented problems with differential displacements between the bottom of the tank and

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the penstock, construction of the hemisphere and anchoring of the tank to the concrete cone. The second alternative, substituted the hemisphere by a cone, of easier construction (Fig. 1.b).

To eliminate the anchoring problems, the concrete conical shell was substituted by a steel cylindrical shell with reinforcing nerves (Fig. 1.c). This solution, as well as the previous ones, separated the hydraulic stresses, carried by the cone, from the gravity and overturning stresses, carried by the cylinder. However, the problem of the connection between the cone and the penstock still persisted, as well as some constructive difficulties at the intersection of the cone with the cylinder.

Cone and connecting pipe were eliminated in the fourth alternative (Fig. 1.d), where the weight of the water was directly transmitted to the foundation, producing a simpler structure, easier to design and construct. The main problem was that the penstock had to go through the tank in a perfectly watertight joint. Besides, the region where the penstock opens inside the tank presented vibration problems during water level variations. The final solution (Fig. 1.3), avoids these problems by raising the foundation level to embed the penstock in concrete, eliminating any openings in the tank and reducing the penstock vibrations. The concrete octogonal foundation, has dimensions controlled by overturning effects and allowable stresses for capacity and settlement. The depth of the foundation was very suitable for anchoring the tank.

DESIGN PARAMETERS

The importance and size of this structure makes the definition of the design parameters a very critical step. It is necessary to be extremely careful in defining the limit states that can occur during the economic life of the structure, avoiding any possibility of failure, and expensive overdesigns as well.

Besides the weight of the tank and its foundation, the main loads were hydraulic, seismic and wind. This section discusses the criteria for defining the main loads, the different load combinations considered as limited states and their associated allowable stresses.

Hydraulic Conditions

In normal operating conditions, the water level varies between level 301masl, corresponding to the three turbines operating at full capacity, and level 328masl, corresponding to the hydrostatic level at the intake. Besides, occasional water hammer will occur which, in the most critical condition, may raise the water level to 353.5masl.

Table 1 summarizes the amount of hours per year associated with different water flow and their corresponding water level in the tank. Based on this information, the probability of exceeding a particular water height may be easily estimated. These results are also presented in Table 1 and on Fig. 2.

The probability of exceeding the level 328masl is drastically reduced because it is associated with water hammer conditions, which may occur only a few times during the year and last for a few minutes (Fig. 2).

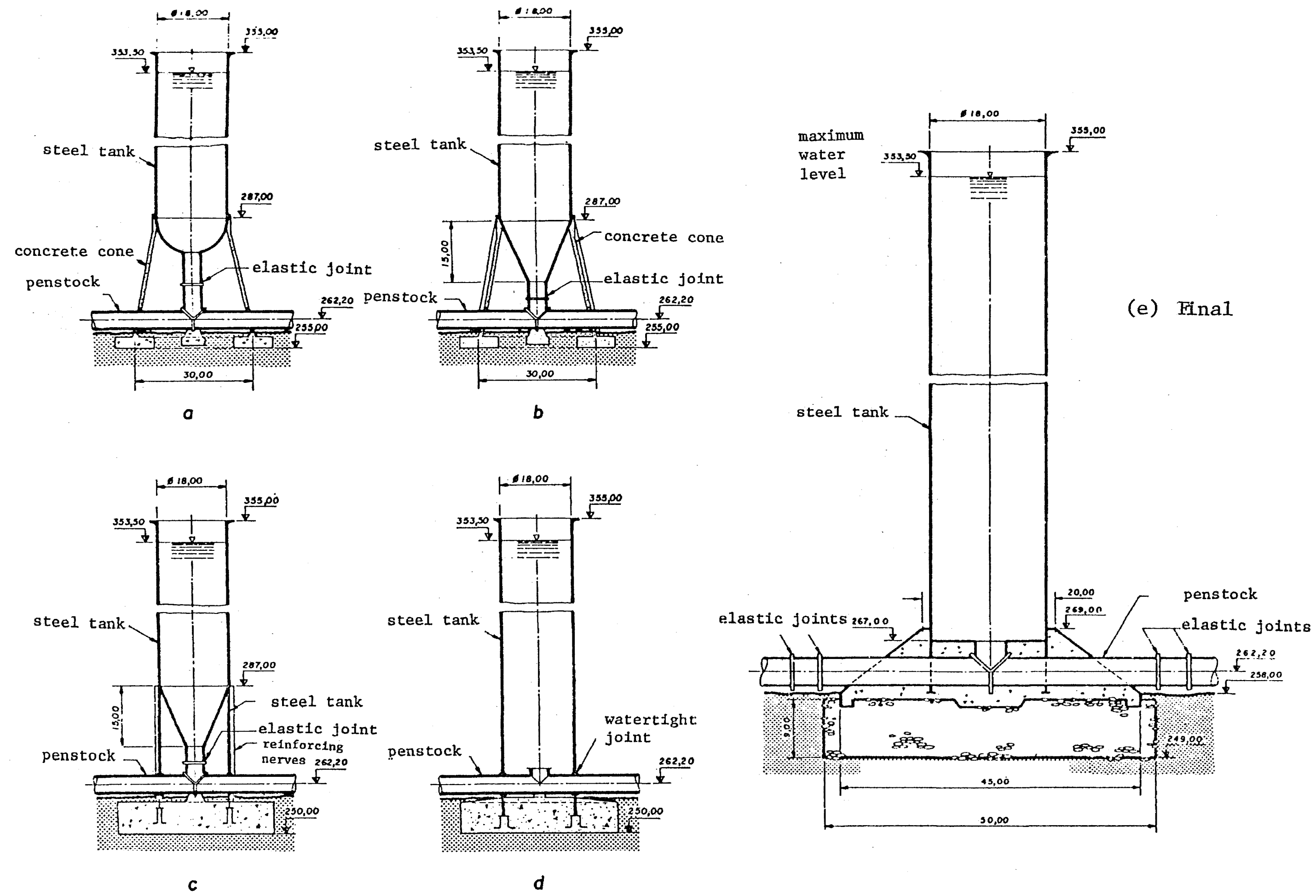

Fig. 1.- Evolution of Structural Form. (Dimensions in meters)

TABLE 1. ANNUAL DISTRIBUTION OF WATER HEIGHTS

Flow (m^3/s)	Water Height H (masl)	Annual Time (hours)	Probability of $h \geq H$
97.5(3 turbines)	301.0	1958	1.00
84.0	307.5	1032	.78
68.3(2 turbines)	313.8	926	.66
55.9	317.8	499	.55
34.5(1 turbine)	322.9	1508	.50
28.0	323.9	118	.32
17.4	325.2	1042	.31
0. (0 turbines)	328.0	1677	.19

Seismic Conditions

Given the importance and the characteristics of this structure, standard seismic code regulations were obviously not applicable. The Costa Rican Seismic Code (CSCR-74) states that such cases require a specific study to define the probable ground excitation. It also requires that the analytical model must be congruent with the expected deformation levels. This particular structure should remain within the elastic limit, even in extreme seismic conditions, because it must be operational after the event. Hence, a mode superposition analysis can be performed with the ground excitation represented by a Response Spectrum.

Information regarding probable ground accelerations and earthquake duration were obtained from Ref. 1. Table 2 presents these values for selected return periods. Intermediate values can be logarithmically interpolated.

TABLE 2. PROBABLE MAXIMUM GROUND ACCELERATION AND DURATION

Return Period (Years)	Maximum Ground Acceleration (Fraction of g)	Duration (Sec)
50	.15	12
100	.18	15
500	.27	24
1000	.31	30

The Dynamic Amplification Factor (DAF) for the Response Sepctrum was derived with the methodology from Ref. 2. The values correspond to the mean plus one standard deviation, unit ductility and 5% critical damping. This value accounts for the energy losses in the tank, in the water, in the foundation anchorage and for radiation damping. Fig. 3 presents the DAF and compares it with values for structures Type 5 of the CSCR-74, which also corresponds to 5% of critical damping and unit ductility. A comparison with values derived later from dynamic amplification studies of the soil site, using the SHAKE program (Ref. 3), is also presented.

Simultaneous Seismic and Hydraulic Conditions

Tank water heights above level 328masl occur only during water hammer conditions, a few times during the year and lasting a few minutes. Hence, it is highly improbable that another extreme condition, such as an earthquake, may occur simultaneously. Furthermore, even for normal operating conditions, the water level will be variable, affecting the seismic response. Therefore, it seems necessary to rationally define the water level associated with a particular earthquake excitation in order to achieve a similar joint probability of occurence for different pair combinations.

It was assumed that water levels and maximum ground accelerations were independent events. Obviously, a strong earthquake will cause water hammering. However, this effect will require about 220 seconds to reach the maximum water height, whereas the earthquake duration will hardly exceed 30 seconds, justifying the independence hypothesis.

In order to evaluate the return period of the joint event, an economic life of 50 years was defined for the structure with a probability of excedence of .10 for that period, which corresponds to a return period of 475 years. Hence, as an example, for a water level of 301masl, with a probability of excedence of 1.0 (Table 1), the corresponding return period for seismic excitation is 475 years. Similarly, for a water level of 328 masl, with a probability of excedence of .19 (Table 1), the corresponding seismic return period is 90.25 years, for a return period of 475 years for the joint event.

Considering the fact that a severe earthquake will probably produce a water hammer, the water level was increased with values based on the actual rejection responses and corresponding to half the earthquake duration. Table 3 presents four combinations of probable maximum ground acceleration and water levels, corresponding to a joint probability of 1/475 years. Case 4, for the maximum water level, was calculated based on possible hydrostatic tests with a duration of eight days in a particular year.

TABLE 3. WATER LEVELS AND MAXIMUM GROUND ACCELERATIONS

Case	Return Period of Joint Event (years)	Maximum Ground Acceleration (Fraction of g)	Water Level (masl)
1	475	.27	305.5
2	475	.23	324.5
3	475	.18	328.0
4	475	.08	355.0

Wind Conditions

For this particular structure, wind was not a problem. The extreme design velocity was defined as 120 Km/hr at 10 meters above the ground.

Yield Stresses, Load Combinations and Safety Factors

The steel used in the tank and penstock is JIS-G3106-SM58 Q.T. with a yield stress of 47 Kg/mm^2 and an ultimate stress of 58-73 Kg/mm^2.

The gravitational, hydraulic, seismic and wind loads were combined into seven limit design conditions. For each particular one, a specific safety factor (SF) was defined. These conditions are as follows:

CC1 = CP + CH(328)	SF = 2.0
CC2 = CP + CH(328) + CR(328)	SF = 1.78
CC3 = CP + CH(355)	SF = 1.4
CC4 = CP + CH(355) + CR(355)	SF = 1.24
CC5 = 1.05[CP +CH(h)] ± CS(a)	SF = 1.25
CC6 = .95[CP + CH(h)] ± CS(a)	SF = 1.25
CC7 = CP + CH(h) + CV	SF = 1.5

where CCi - Load combination for limit state i; CP - Permanent load (dead weight); CH(h) - Hydrostatic load for water level h (masl); CR(h) - Secondary stresses due to restrictions in the membrane behavior for a water level h (masl); CS(a) - Seismic load associated with a given maximum ground acceleration a. a and h combinations are the four cases of Table 3; CS - Wind load; SF - Safety factor.

The radial, longitudinal and shear stresses were combined with the Von Mises yield criteria and compared with the yield stress affected by the safety factors.

Besides, buckling longitudinal stresses were checked with the equation

$$fl \leq fb = 4tE/D\ SFb$$

where fl - flexural stress; fb - critical buckling stress; t - wall thickness; E - modulus of Elasticity; D - Diameter of Cylinder; SFb - Safety factor for buckling = 1.5. These values correspond to Japanese standards for cylindrical tanks and are quite conservative. Unfortunatelly, the empirical evidence regarding buckling of this type of structures is still very limited to justify less conservative values.

ANALYSIS PROCEDURE AND CRITICAL CONDITIONS

For seismic analysis, the tank was considered as a cantiliver vertical beam on a rigid base. Both flexural and shear deformations were included. Water displacements relative to the tank were neglected, which in general leads to conservative results (Ref. 4). The mode superposition analysis considered the first four modes of vibration for each one of the four cases presented in Table 3. Total response for each case was calculated by the rooth mean square procedure. Soil-structure interaction was studied in a separate analysis. As expected, these effects were neglegible given the foundation size and the stiff soil conditions. For hydrostatic and wind analyses, the structure was modeled by axisymmetric finite elements.

Longitudinal buckling, rather than yielding, governs the design. Case 2 (Table 3), with a water level of 324.5 masl, controlled the design of the first 44m of the tank. Case 4, with 355masl, controlled the remaining upper part. Final shell thickness varied from 56mm at the base to 10mm at the top. This later value was a predefined condition for a minimum annualar stiffness.

SPECIAL DETAILS

Anchorage to the Foundation

Bearing stresses are transmitted to the concrete foundation through a steel annular plate at the bottom of the tank. Tensile stresses are carried out by 52 pairs of steel bars, 13cm in diameter and 5.50m long, with a bearing plate at the end, embedded in the concrete.

Of particular interest is the region where the 4.6m diameter penstock underlies the tank shell. Instead of bars, which obviously could not develop their anchoring length, two steel cylindrical shields, 9.00m long, 5.50m high, 50mm thick and with the same curvature of the tank, were provided. The shields are able to transmit the tensile stresses around the penstock without interfering with it. Their dimensions were selected to achieve a longitudinal stiffness equivalent to that of the substituted bars in order to avoid perturbations in the distribution of stresses around the tank perimeter.

Penstock Flexible Joints

Two Dresser Type flexible joints were provided at each side of the penstock, near the points of emergence from the concrete foundation, in order to accomodate any differential displacements between these two structures.

Stiffening Rings

Secondary stresses at the base of the tank due to boundary restrictions were reduced by three stiffening plate rings located at the bottom of the tank interior. A plate around the top of the tank stiffens the particularly thin shell to avoid wind vibrations and provides space for inspection crews and load lifting. A helicoidal exterior ramp allows for ascension and provides further stiffening throughout the tank.

FINAL REMARKS

Construction of the concrete foundation was done by I.C.E. in two stages, in order to allow for space for the penstock, anchoring bars and shields. K.H.I. constructed the tank 8 weeks before schedule, in 92 weeks. Further details regarding the foundation characteristics and the constructive process can be found elsewhere (Refs. 3, 5).

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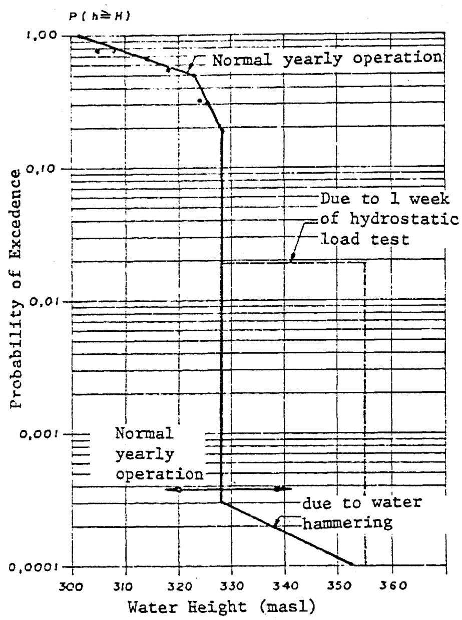

Fig. 2
Probability of Excedence
of Water Level

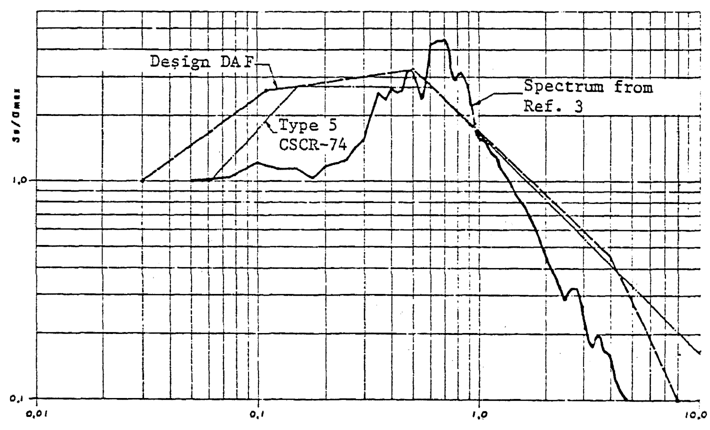

Fig. 3
DAF for Design
Response Spectrum