ON THE DESIGN HISTORY OF A HYDROELECTRIC POWER PLANT IN TECTONIC ACTIVE ENVIRONMENTS

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SUMMARY

The paper deals with a rather unusual concept. Tectonic faults were encountered in the presumed sound rock, to which a powerhouse basement should be closely coupled. To permit eventual movements, a freedom between house and rock was introduced in the design of the foundation. Dynamic analysis, confirmed by on site testing, revealed that the applied change caused a considerable magnification of response from the basement and upwards creating an unforeseen increase in seismic loading for equipment. Critical equipment, designed under original response assumptions and already ordered, might accordingly, suffer severe damages in a design earthquake. Management of the apparent risk situation involved special attention to flexibility of connections and mountings of equipment and, in some cases base isolation techniques were recommended. Operational strategies for the plant should include monitoring and system related interpretation of earthquake effects.

INTRODUCTION

The powerhouse for the aboveground Sultartangi 120 MW hydroelectric power plant was initially planned and designed in a traditional way in respect of defined seismic parameters. In the beginning of the construction phase, however, its support conditions needed to be re-evaluated. The excavation revealed two pronounced faults passing across the presumed sound rock against which the power house basement (see fig. 1) should be anchored. The faults, which effectively divided the excavation into three parts (see fig. 2), were judged to be "geologically active" and the occurrence of relative displacements in future major earthquakes could not be ruled out. The potential displacement of the rock supporting the powerhouse had to be accommodated by re-design to minimise the risk of rupturing of the building. Thus, parts of the building foundation were detached from the base and the gable sides of the excavation, chiefly by means of mineral wool and teflon like sheeting of critical surfaces. Consequently, as revealed by this study, the expected response of the house, affecting contained machinery and equipment, changed dramatically. The design provisions or parameters, listed in the contract documents to the manufacturers of machinery and equipment, became in some cases considerably invalidated.

The prime objectives of this study were to evaluate the described risk situation and to determine how earthquake resistance of critical equipment, already designed and bought, could be maintained at an acceptable level. The study as a whole is an educating example of a construction stage risk management. It follows, that certain risk retention must be compared to cost for post-manufacturing alterations, the effectiveness of which might even be extremely difficult to verify.

THE POWER PROJECT, LOCATION AND CHARACTERISTICS

The power station is located in the northern outskirts of the South Iceland seismic area, just west of the Eastern Volcanic Zone (i.e. 19°37'W, 64°10'N). The Plant utilises a gross head of 51 m, between approximately 299 and 248 m a.s.l. The river Thjórsá is diverted by an approximately 4 km long tunnel from a reservoir to the intake

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stilling basin placed just above the powerhouse and then returned to its old course by an approximately 7.5 km long tailrace canal.

The investments in the Plant amount to approximately 165 million USD whereof the powerhouse structure represents 37,5 million and its installed plant and machinery 35 million USD.

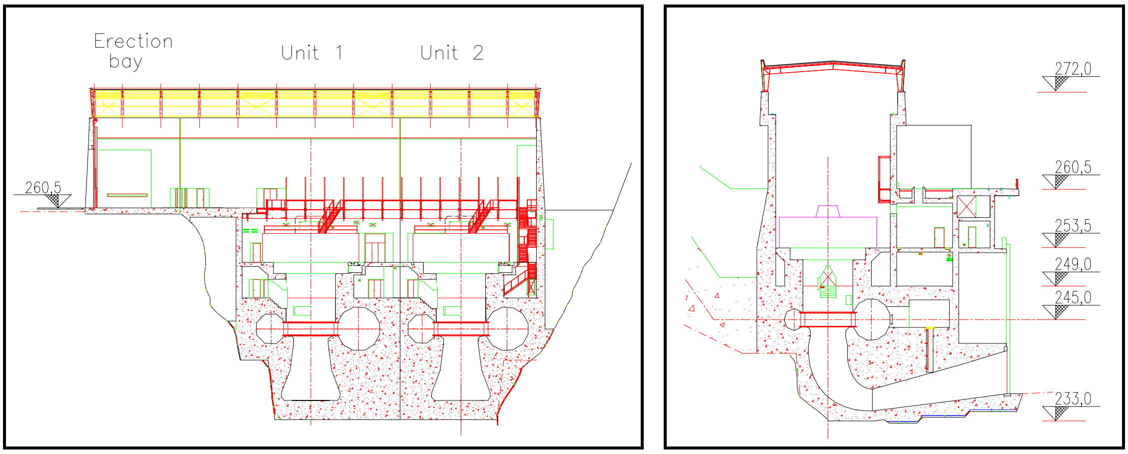


Figure 1: Longitudinal section and cross section of Powerhouse

The Powerhouse

The powerhouse (see fig. 1) is basically a concrete cast in situ structure. Expansion joints divide the structure into three sections (erection bay, Unit1 and Unit2). Shear keys are provided on the joints. The upper part (above 260 m a.s.l.) is approximately 58 m long and 18 m wide. It rises 17 m above the surrounding ground level and forms a “box” with 12 m high and 1 m thick solid concrete walls but the top 5 meters and the roof is a steel frame construction. Below ground the powerhouse has a depth of about 30 m, with its back- and side walls facing the excavated rock, while the downstream front protrudes about 15 m towards the tailrace canal. On the ground level deck (el. 260 m a.s.l.), above the protruding part, the main transformers are placed.

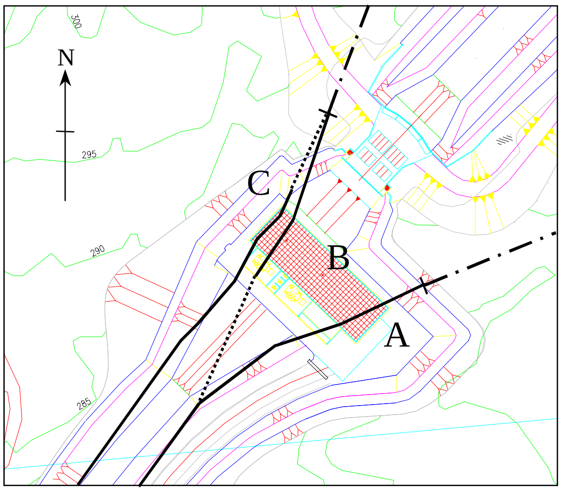


Figure 2: Plan of fault locations vs. Powerhouse

Geological aspects

The Powerhouse is located within bedrock of early quaternary formations on the southern slope of a low mountain, Sandafell. The Sandafell Mountain is covered with surface sediments and topsoil of varying thickness (generally 1-5 m). The bedrock in Sandafell consists of highly jointed, normally magnetised, olivine tholeiite basaltic and acidic lavas inter-bedded with layers of consolidated sediments, mostly of fluvial and glacial origin. This bedrock was piled up during the ice-age approx. 0.7-1.5 million years ago. Down to elevation 260 m a.s.l. a continuous highly fractured and cube jointed basalt was encountered. Pronounced layers of scoria appeared at these levels but the rock below is also fractured.

Some of the tectonic joints and strike slip faults encountered during the excavation (see fig. 2) show clear marks of horizontal slip and are commonly adjoined by up to 0,8 m thick fault breccias with crushed rock fragments mixed in clay. In spite of no evident signs of movement in the overlying sediments, mostly originating from the end of the last glaciation period, relative displacements in future major earthquakes could not be ruled out.

SEISMICITY AND TECTONICS

A seismological study indicates that magnitude seven earthquakes may be expected within a 50 km south of the plant and smaller earthquakes in the neighbouring area. Furthermore, the above mentioned tectonic faults indicate increased risk of locally induced seismic effects.

Table 1, summarises what size of earthquakes, based on historic evidences (Einarsson 1991), can be expected to effect the plant. Based on these data, as well as available geophysical information (Bjarnasson & Einarsson 1991), time series and earthquake response spectra have been derived applying ramified source models (Sigbjörnsson et al. 1995).

Table 1: Basic earthquakes for simulation of seismic effects.

Case No.	Historic event (yr.)	Epicentral location (degrees)		Magnitude (M)	Epicentral distance (km)	Epicentral direction[1] (°)	mean(a_p)[2] (% g)	std(a_p) (% g)
		Latitude	Longitude					
1	1784	63.97	20.37	7.1	43	58	11	1
2	1896	63.97	20.26	6.9	38	54	8	1
3	1912	63.94	19.95	7.0	30	31	19	2
4	1987	63.91	19.78	5.9	30	14	4	0.4
5	(prediction)	63.94	19.95	7.1	30	31	23	3
6	(prediction)	63.97	19.62	6.5	22	-1	19	2

[1] Direction from epicentre to site measured clockwise from north.
[2] Average of peak acceleration in 30 simulated time series.

To facilitate the engineering calculations a basic (elastic) design spectrum is suggested enveloping roughly the simulated elastic response spectra. The spectrum is shown on figure 3 along with some simulated case spectra.

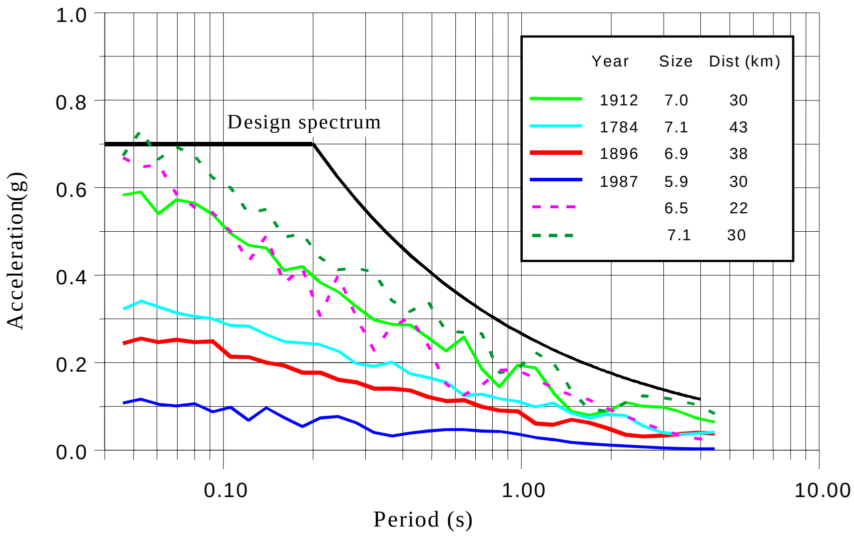


Figure 3: The design spectrum and simulated earthquake response spectra based on the data in Table 1. The simulated spectra are obtained as a mean value plus one standard deviation of 30 simulations for each case assuming 5% critical damping ratio.

EARTHQUAKE RESISTANT DESIGN PROVISONS

The powerhouse of the Sultartangi hydropower project was designed in accordance with European standards. Earthquake resistant design of the powerhouse was evaluated according to Eurocode 8, ENV 1998.

In the earthquake design of the powerhouse, the structural parts of most concern are the upstream and downstream walls, and the steel frame roof structure. The steel frames afford limited bracing, so the upstream walls of Unit 1 and 2 act almost like a cantilever, 18.5 m high, with the largest moments occurring at the generator floor el. 253. The wall facing downstream is supported laterally up to el. 268 and the downstream part of the powerhouse is a relatively stiff structure, containing separating rooms, shafts and sump pit.

2D- and 3D-FE models were made of the building, for static and response spectrum analysis, in the structural design of the concrete walls and steel frames. Behaviour factors ranging from 1.5-2.4 were used, depending on which part of the structure being considered. In addition an importance factor of 1.4 was applied. The earthquake loading influenced especially the reinforcement for the outer faces of the upstream walls and the gable wall of Unit 2 above el. 260.

The faults encountered in the excavation pit, split the powerhouse foundation in three parts, A, B and C (fig. 2). One fault zone crosses the foundation of Unit 2 (located on parts A and B), with the other cutting approximately along the movement joint between Unit 1 (located on part B) and the erection bay (located on part C). Measures had to be taken to minimise effects of relative displacements of the faults, if activated by major earthquakes. Thus, it had to be ensured that Unit 2 would not follow the movement of foundation part A. Additionally, the friction between the wall of Unit 1 and the rock walls of foundation part C had to be minimised. To achieve these goals, a double layer plastic sliding sheet (Gumba Nofri sliding) was placed on lean concrete, under the part of Unit 2, located on foundation part A as well as under the corner of the erection bay on part B. Furthermore, a 100 mm thick layer of mineral wool (density 1 kN/m^3) was placed between the concrete wall and the adjoining rock along the fractures, at both gables. Where excavation was excessive, the gap between the powerhouse walls and rock was filled with free draining material, instead of concrete as originally presumed.

With the powerhouse gable walls detached from the adjoining rock, lateral support was unaccountable. Thus the original design assumptions regarding the response spectrum for attached or contained equipment were potentially invalidated. Consequently a detailed FE-model of the powerhouse was made to study the effects of the revised support conditions on the response of the house and its effects on critical equipment.

DYNAMIC ANALYSIS OF THE POWERHOUSE

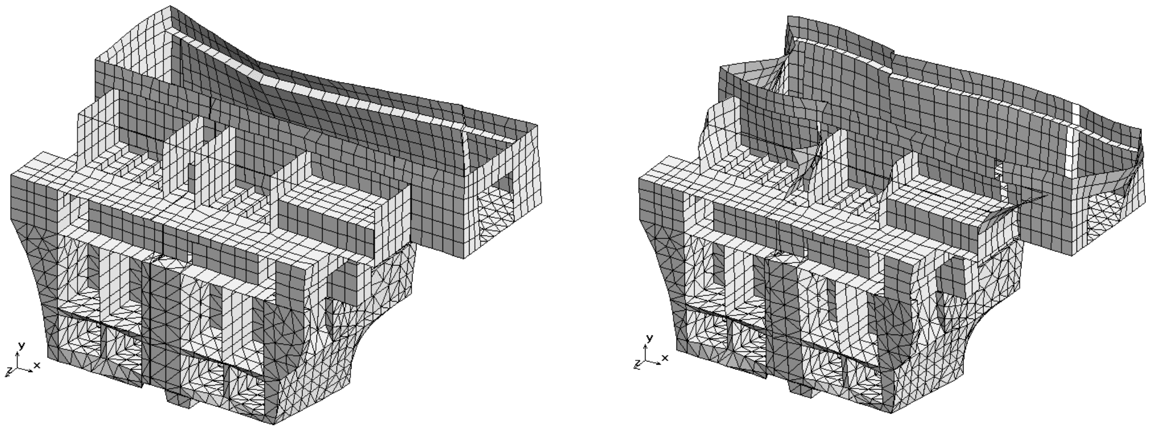
Finite element modelling

The dynamic analysis was based on a fairly detailed three-dimensional finite element modelling of the powerhouse (see fig. 4). The elements used were beam, shell and tetra elements, all containing 6 DOF at each nodal point. The final versions of the model contained about 12070 elements, 5370 nodal points and about 30320 DOF.

Emphasis was put on the inclusion of as much structural detail as feasible. The solid concrete base of the structure was modelled with tetra elements. The effect of the sliding sheet placed on foundation block A (see fig. 2) was accounted for through boundary conditions, defined in such a way that part of Unit 2 is only constrained vertically but free to move laterally. The shear keys of the expansion joints were modelled by special elements to account for the resulting discontinuities. The walls and floors of the building were modelled with shell elements whereas the structural system of the roof is modelled with beam elements. The penstock for machine no. 2 was included in the model, using curved isoparametric shell elements. The dead weight of equipment was accounted for.

Natural frequencies and mode shapes

The natural frequencies, mode shapes and participation factors of the FE-model were evaluated. The first 55 natural frequencies of the structure have a value between 3 and 16 Hz, about 12 of those 55 modes have a considerable participation in the overall response, and six of these are in the range of 9 to 10 Hz. The fact that so many natural frequencies are distributed over such a small frequency range demonstrates the complexity of the structure. The two mode shapes shown in figure 4 demonstrate typically the discontinuity of the three parts of the powerhouse, separated by the expansion joints. Several of the mode shapes demonstrate rotational tendencies, mainly caused by the difference in boundary condition of the three different sections of the powerhouse.



Mode no. 1 - 2.9 Hz

Mode no. 32 - 9.54 Hz

Figure 4: Two of the key mode shapes of the FE-model (the roof structure is not shown).

Response analysis

The FE-model was analysed by applying a response spectrum to obtain peak value estimation and by frequency response analysis to provide time series information. All response estimates were based on an equivalent structural damping ratio of 5%. The vertical ground acceleration was taken as 50% of the horizontal one.

Applying the frequency response method (Snæbjörnsson et al. 1994), using 20 simulated time series of ground motion as excitation, time series of acceleration and displacement were evaluated at 112 selected locations in the powerhouse on 6 different elevations. The number of points and their distribution was chosen to give an overview on the variability of the building response. The frequency response was based on 55 modes of vibrations with natural frequencies in the range of 3 to 16 Hz. An equivalent critical damping ratio of 5% was used for all modes of vibration. Figure 5, shows the average peak values of acceleration as a function of elevation in the powerhouse for simulated earthquake series no. 1, 3, 5 and 6 (see chapter 3).

Figure 5: Average peak value of absolute acceleration as a function of elevation for EQ. no. 1(◊), 3(o), 5(Δ) and 6(☐). Average taken of 20 simulated response time series for each simulated ground motion time series.

The results from the frequency response analysis agreed reasonably well with the results from the response spectral analysis. Displacements were seen to increase exponentially from about 1 mm at el. 245 up to 30 mm on el. 272. In general the elastic displacements are not high but the acceleration is considerable, as expected for a structure with relatively high natural frequencies.

The response varies considerably between different locations at the same elevation, especially the amplitude (see fig. 5) although variability in frequency response is also increasing with height. The response was influenced by the expansion joints, which caused a discontinuity between the different building sections. The different boundary conditions of Unit 1 and 2, created by the sliding sheet placed on the rock (block A) beneath Unit 2, were seen to cause increased rotational effects in the structural response. The ratio between average peak values of floor response and ground motion went from 1,0 (rock base) to 2,7 on el. 260 and up to 4.6 on elevation 272. The vertical magnification ratio was much smaller than for the horizontal components or about 1 to 1.4, except for the roof where the magnification ratio was 3.3 on the average.

Testing and validating the structural model

Acceleration, induced by ground explosion work in the tailrace canal about 1300 m downstream of the building, was recorded in the powerhouse at three elevations (236, 254 and 262). At the time of recording the outer shell of the powerhouse was complete and the structural system supporting the roof in place. However, the roofing was incomplete, the turbines not installed and the free draining material had not been placed between the concrete and the adjoining rock. The data set from tailrace canal was in many respects not unlike earthquake recordings, which is reasonable when considering the large amounts of explosives used and the relatively long distance from the powerhouse.

The FE-model was modified to represent the incomplete building stage of the structure. Fourier analysis and system identification was used in analysing the data set (Snæbjörnsson et al. 1996). Natural frequencies estimated by system identification of the data seemed to correspond fairly well with the natural frequencies from the finite element model. Damping was in the range of 2 to 5% of critical, depending on mode of vibration.

The simulation of response was based on the motion recorded at el. 236, i.e. the base of the powerhouse. It should be noted that no detailed calibration of modal damping was performed, but a fixed average damping value was used in the simulation. Good comparison was achieved between simulation and recordings of horizontal acceleration, especially in the time domain.

On the importance of complete modelling

In simplified modelling of a structure of this type, it is often assumed that the concrete mass is attached to the rock foundation in such a way that the response of the massive lower part is more or less equivalent to the overall ground motion. This simplified modelling approach was primly used in the design and planning of the structure. At later stages, however, when investigating the effects of earthquake response on equipment, such approach was considered unacceptable. In view of the situation at hand it was considered necessary to model the powerhouse in its complete form, from the rock bottom up to the roof.

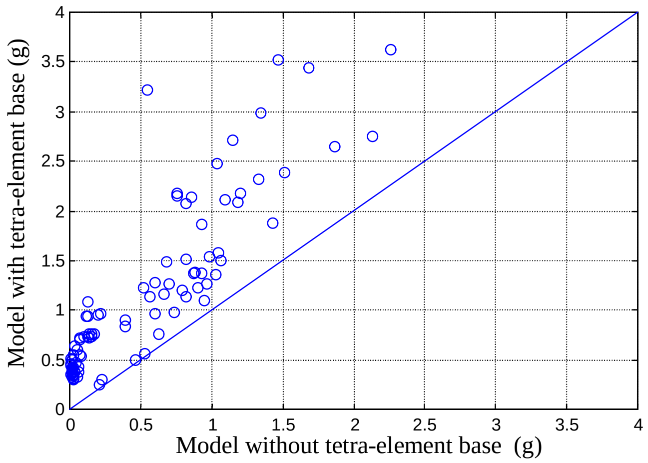


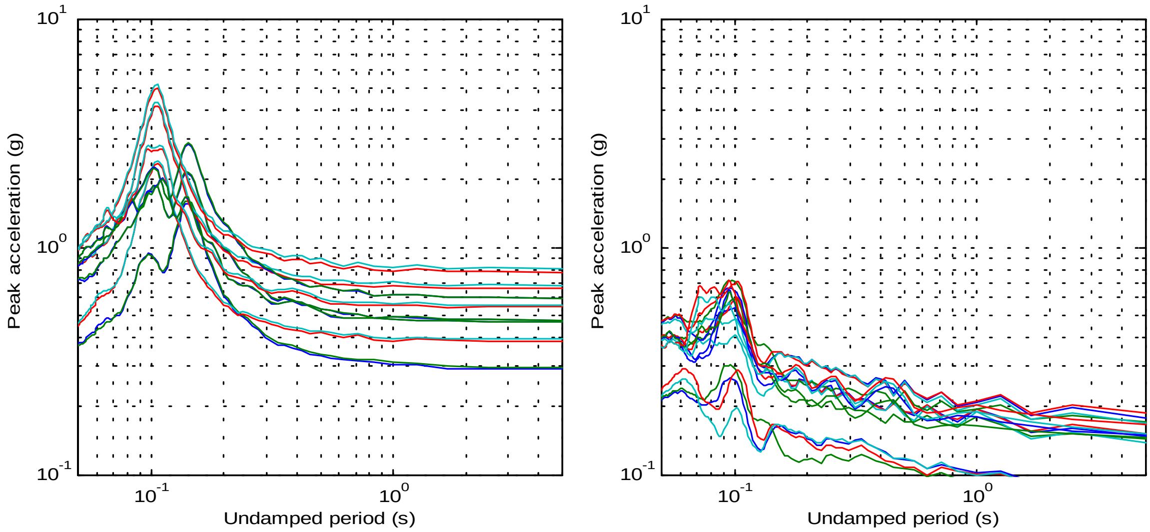
Figure 6: Comparison of model response with and without tetra-element base.

A limited investigation was instigated in order to validate this decision. That was done by comparing results from the complete FE-model with result from a simplified FE-model, where all tetra-elements representing the base of the structure were removed up to el. 249 (halfway between base and surrounding ground surface) and all D.O.F. at el. 249 were fixed in the traditional manner. The excitation was in the form of a ground response spectrum. Comparison of the results shows the importance of making a complete model that incorporates the base of the structure and allows for a better simulation of boundary conditions. The difference in response was between 10% and 100% (see fig. 6), which proved to be especially important for its effect on equipment.

EARTQUAKE EFFECTS ON EQUIPMENT

Frequency response analysis using simulated time series of floor response

Applying the frequency response method, using the simulated floor response series of chapter 5 to excite SDOF systems of different natural frequency and damping characteristics, a response spectrum can be evaluated for the equipment (Henje et al. 1990). This approach relies on the assumption that the dynamics of the equipment can be modelled as equivalent linear SDOF system. A critical damping ratio of 5% was used, assuming that structural material is steel stressed to yield but without significant “plastic” deformation. The equipment response spectra were evaluated for simulated EQ. no. 1, 3, 5 and 6 at each of the 112 calculation points mentioned in chapter 5. Figure 7 shows the response spectra for the transformers on el. 260 m a.s.l. It is seen that considerable magnification of response occurs for equipment with natural frequencies around 10 Hz.



(a) (b)

Figure 7: Average peak acceleration response for SDOF system at the transformer location on el. 260 from EQ no. 1, 3,5 and 6. (a) Horizontal acceleration and (b) vertical acceleration.

Equipment vulnerable to earthquakes

The analyses of the detailed model resulted in higher demands for earthquake resistance of equipment than required in the original bidding documents. It was assumable that some equipment might not sustain the expected excitation levels, i.e. an unaccounted risk situation had come up. When this became apparent, the manufacturing of the equipment was either completed or in its final stage. Hence, it would be difficult to alter the design or placement of equipment and other measures to minimise the risk of failure had to be considered.

When viewing the possibility of equipment damage it should be kept in mind, that equipment can generally sustain fairly high acceleration levels, as they are relatively robust structures. However their resistance is often hampered by lack of freedom to deform especially if they are rigidly connected to other equipment.

The key equipment affected by earthquake excitation was identified as, the service traverse crane located at el. 268, transformers located at el. 260 and breaker/protection units for the transformers located at el. 254.

The equipment response spectrum was evaluated for several locations along the travelling path of the crane. It was clear that the excitation of the crane was very dependent on where it was situated on the supporting beam. Since the crane is in operation less than 1% of the plants operating period it was judged sufficient precaution to require that when not in operation it should be kept at the gable of the powerhouse. There, horizontal response levels were expected to be less than 2g and vertical response levels less than 0.5g.

The most critical equipment is the two transformers (160 ton each) located on el. 260. Response spectra evaluated for the transformers are shown in figure 7. As can be seen the horizontal response levels could reach 5g if the transformer has a natural frequency around 10 Hz. Preliminary analysis of the structural frame and supports of the transformer indicated a natural frequency of around 15 Hz, assuming that other transformer components could be considered rigid elements. Contract requirements for the manufacturers’ design of equipment was based on the assumption that this might be exerted to seismic forces amounting to 0.7g, considerably less than the dynamic analysis predicted. Evidently, in order to ensure the integrity of the most vulnerable equipment itself (transformers and appurtenances), the effective seismic forces must be restrained at a level no higher than the design maximum. An obvious solution was to isolate the equipment from the structure. This can be done without making any changes to the transformers except at foundation level. A special type bridge bearings were selected (Reinforced Elastomer Bearings, type 2b from GUMBA). The aim of the isolation was to achieve a system with a fundamental natural frequency about 2,0 Hz, that should insure acceleration levels below the manufacturer seismic design value of 0.7g, giving peak horizontal displacement of about 30 mm. A control response test of the transformers "as erected" is fundamental for the final risk evaluation.

The breaker and protection units on the floor below the transformers (el. 253 m a.s.l.) were judged to be reasonably safe. Some checking of their supports and fastening to the floors were recommended. However the rigidity of their connection to the transformers on the floor above was a concern. The reason being that the transformers were expected to be heavily excited, creating considerable relative deformation between the two components which might very possibly cause rupture of the busbar or its insulating shell (SF_6 inside), especially if the transformers were to be isolated from their base. Actions to ensure required flexibility between the transformers and breaker/protection units have been strongly advised**.**

FINAL REMARKS

Earthquake engineering methods have been applied in the design and construction of an important utility building. The aim is to predict earthquake-induced response for the building enabling to minimise, through design, the risk of damage in a major earthquake.

It is demonstrated that the earthquake impact on equipment contained in this typical hydroelectric powerhouse is strongly affected by the properties of foundation and equipment/building natural frequency ratios. 3-D dynamic analysis as described herein is recommended both for selection of proper equipment locations and/or to specify ample design criteria for earthquake excitation of equipment.

The detailed model of the powerhouse structure and its equipment did not significantly alter the forces and moments relevant for the structural walls obtained with the more simplified models used in structural design. However, through dynamic analysis of the entire structure, the interaction between "detached" sections could be studied, and the forces, transferred by the particular shear keys on the expansion joints of the upstream and downstream walls, assessed. Furthermore, the analysis clearly demonstrated a significant increase in estimated equipment response. Analysis of this type can be a valuable instrument for improvements, particularly in the early stages of planning and design.

The structural properties of the surrounding free draining material are difficult to model accurately as well as their effect on dynamic response. Such uncertainty might best be studied and resolved by observations enabled by an instrumentation of the powerhouse. The instrumentation will then also be amalgamated with an existing monitoring and emergency management system for South Iceland and important for operational strategy in situations of earthquake emergency.

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