RESEARCH REPORT

VTT-R-01167-13

Principles and practice of exploiting beyond elastic-response reserves in structures

Authors: Vilho Jussila, Ludovic Fülöp

Confidentiality: Public

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Principles and practice of exploiting beyond elastic-response reserves in structures	
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Summary

This report is deliverable for 2012 of Subproject 2 in the project SESA "Seismic Safety of Nuclear Power Plants – Targets for Research and Education", part of the SAFIR 2014 program on nuclear safety.

The aim of the task was to conduct a survey on the structural typologies interested for the utilities from seismic design point of view and to start assessment of some typologies for their beyond elastic response reserves. With a reactor configurations, we carried out finite element modelling (FEM) runs in order to (1) understand the level of complexities required to assed beyond elastic response (2) to bound the expected response depending on the expected loading in Finland.

The work is concluded by a set of proposals for the continuation of the parametric FEM investigation in 2013. Since this is reporting the 2012 part of a continuing investigation, no definitive conclusions are presented concerning the performance envelopes in beyond design basis scenarios.

Confidentiality	Public

Espoo 13.2.2013

Written by	Reviewed by	Accepted by
Ludovic Fülöp Senior Research Scientist	Ilkka Hakola, Senior Research Scientist	Eila Lehmus, Technology Manager

VTT's contact address

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Preface

The decisions to increase the number of nuclear power plants (NPP) in Finland, and especially the selecting the location of one NPP in the northern part of the country, called for reassessing the potential effect of earthquakes on plant safety requirements.

As a response to this need, the project SESA - ***Seismic Safety of Nuclear Power Plants – Targets for Research and Education*** was included in the Finnish Research Program on Nuclear Power Plant Safety, SAFIR 2014, under the umbrella of Reference Group 7 - Construction safety. SESA is in its first year of financing in 2011, and it has 3 Subprojects:

- Subproject 1. Earthquake hazard assessment,
- Subproject 2. Structural assessment,
- Subproject 3. Equipment qualification procedures,

This report is a deliverable of Subproject 2 for the 2012.

The work in SESA has been supervised by the Reference Group 7 and the Ad-Hoc group specifically named for the SESA project.

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Jouni Saari	ÅF-Consult

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Authors

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1 Introduction

The design for earthquake loads is a challenging task both for ordinary buildings and Nuclear Power Plants. While in some regions, ordinary buildings are exempted from seismic design however NPP's are designed for earthquake loads due to higher demands on safety, even in regions with very low seismicity.

The situation in Finland is so that the country is located on a stable continental plateau, with very limited seismic activity and virtually no experience in damaging earthquakes. While this is a very fortunate situation, the scarcity of seismic observation data is also the source of epistemic uncertainty leaving many un-answered (and probably un-answerable) questions concerning the strength and precise characteristics of earthquake loads expected on NPP structures.

This uncertainty, together with recent experiences in the world when engineering predictions proved to be inaccurate, was the driver of opening a study on what can be expected in NPP structures in case of beyond design basis loading scenarios. Given the generally low seismicity of Finland, the question of widespread damage to building structures is not the focus point of this study; but we try to estimate if exceedence of loads may influence floor spectra to the degree of influencing component qualification.

2 Goal

The aim of this document is to:
- .Present a short review of building typologies in NPP interesting for earthquake evaluation, and introduce a, yet unsuccessful, survey with Finnish utilities to map their interest concerning building configurations;
- To propose bounds for the characteristics of earthquake loads we consider for the assessment;
- To describe the modelling methodologies of the generic reactor building used as test case and present selected outcomes of the modelling carried out in 2012. With this the specific scope is to help guide the parameters of the modelling effort in 2013 (decide only a set of techniques to be used further).

3 Methods

The first part of the report is focused on introducing design targets in NPP applications.

In the second segment we introduce the building typology review and the survey SESA initiated - with the hope that the contacted utilities and companies will contribute data on buildings interested for their application areas.

The third part describes the modelling techniques and assumptions used so far. To review the intention in to submit this information in order to have feedback before the parameterization of the modelling effort in 2013. Summary FEM results are also presented in this segment.

4 Design code objectives for seismic performance in NPP's

Nuclear seismic design rules (e.g. YVL 2.6 [2]) focus on functionality of structures, systems and components (SSC's) important for safety. The designed structure must maintain its function during and/or after the seismic event. By default, or stated explicitly, functionality also means integrity. Naturally, it is expected that loss of integrity of a structure or equipment also means loss of functionality. Nuclear design codes do not usually define multiple levels of performance objectives to structures, systems or components (SSC's) (i.e. they remain functional or they become non-functional) – however, levels of objectives are achieved when the functionality requirements are detailed for each SSC.

Nuclear code objectives (YVL 2.6 [2])
All structures, components and systems important to safety must be designed to withstand seismic loads in order to assure the safety of the nuclear power plant. The NPPS's design shall be such that a design basis earthquake will not compromise reactor shutdown, decay heat removal and the containment function, or the confinement of radioactive materials.

Different safety objectives defined by the codes are sometimes translated as "prohibited to cross the elastic limit" in practice [3]. This view is too simplistic, but it results from the desire to transform code objectives into practical engineering terms. NPP design objectives are to stay functional/operational – and damage is not acceptable since it implies loss of functionality. However, to directly associate damage with elastic limit is more troublesome since local plasticization is unavoidable (e.g. in connections) even in conventional steel structures.

The next question is, what the load levels are at which the performance objectives are to be reached? Nuclear codes are usually operating with two basic levels of loads – one corresponding to little or no damage (a serviceability limit state load), and one corresponding to server loading (an ultimate limit state load). The names of these two levels differ in design codes; and the performance requirements demanded by the code may also be formulated slightly differently. Examples of definition are given in the table below.

Nuclear (Not Specific)
The Safe Shutdown Earthquake Ground Motion (SSE) is the vibratory ground motion for which certain structures, systems, and components important to nuclear safety must be designed to remain functional.
The Operating Basis Earthquake Ground Motion (OBE) is the vibratory ground motion for which those features of the nuclear power plant necessary for continued operation will remain functional.

The annual probability of exceedance, or return period is the usual way to define the different levels of design seismic action. Nuclear codes – due to the potentially high consequences of any damage to NPP's – tend to push return period for OBE and SSE very high.

Nuclear (YVL 2.6[2], JEAG, IAEA[4])
SSE: ➢ YVL 2.6 : operates with the design basis earthquake in every 100 000 years (not clearly state if OBE or SSE, but behaves more like an SSE – because requires review of only S1 class components critical for safety) ➢ JEAG : 50 000 years (S2), 10 000 years (S1) ➢ IAEA : defines SL-2 associated with most stringent safety requirements
OBE: ➢ YVL 2.6: Does not define it. ➢ IAEA : SL-1 for less severe, more probable earthquake – usually a portion of the SSE ➢ In moderate seismicity regions OBE is close to return period 475 years [3]

In Figure 1 the two dimensions - performance objectives and seismic hazard - are represented in a matrix form, for NPP related SSC's and tradition buildings. The generic group "very rare earthquakes" has been split in two in order to emphasize the significant difference in return periods.

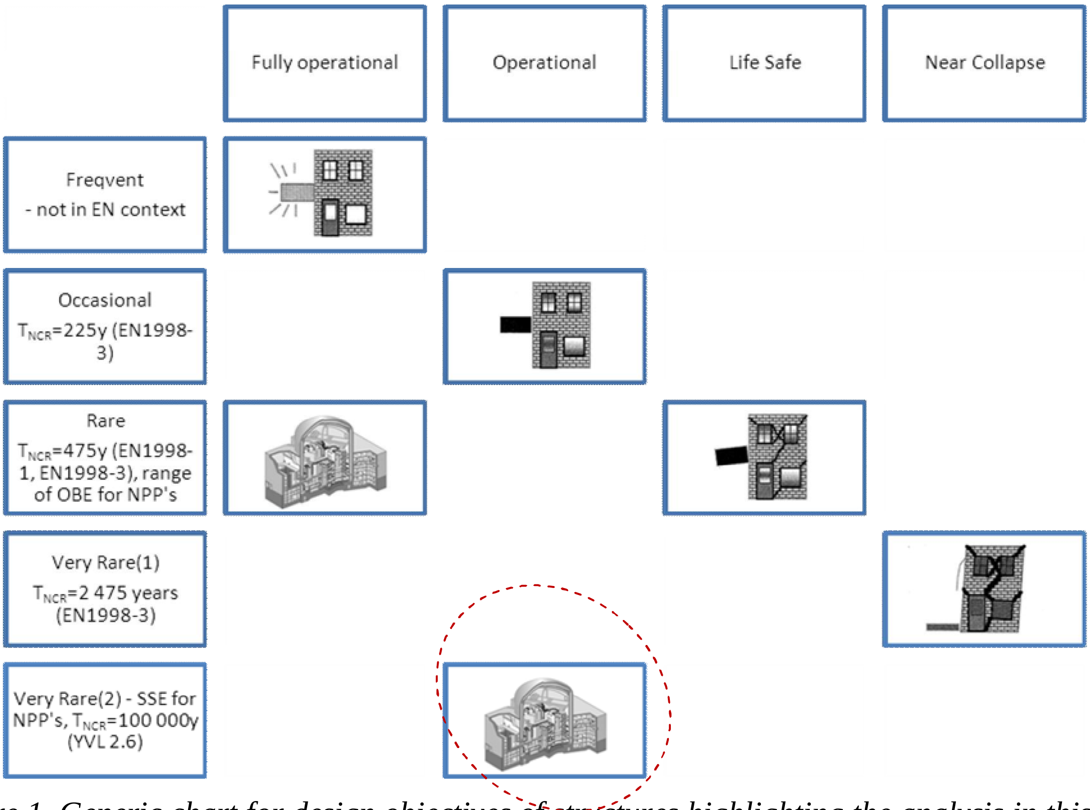
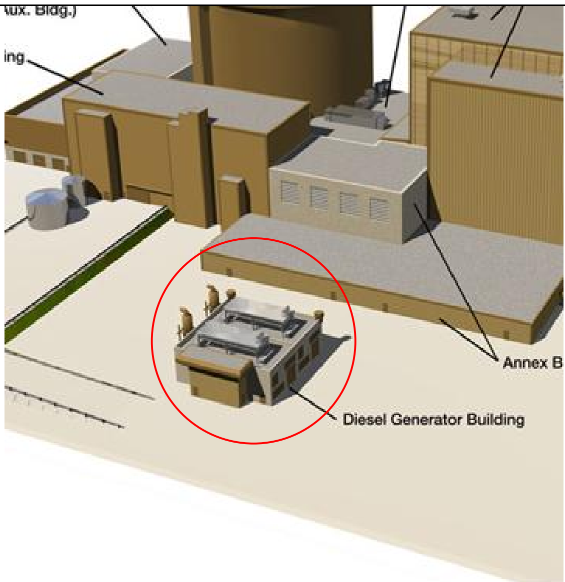


Figure 1. Generic chart for design objectives of structures highlighting the analysis in this document

5 Building typology review and the survey

The following questioner was sent to utilities in Finland in order to collect information on building configurations interesting for earthquake analysis.

Contact for questions:	ludovic.fulop@vtt.fi
Name/Function of building:	Diesel Generator Building for AP1000 NPP (Westinghouse)
Pictures/Sketches:	 https://www.ukap1000application.com/modularization_and_construction.aspx
Reason for interest in seismic behavior:	In AP1000 the diesel Generator Buildings are NOT Seismic Category I structure. Seismic design is according to the Uniform Building Code. (Source: www.ukap1000application.com/modularization_and_construction.aspx) Seismic review of the building is interesting from protection of investment point of view only.
Approximate dimensions (m):	20x20m
Number of floors:	2-3 floors.
Describe structural typology:	Vertical elements are in situ cast concrete frame + shear wall construction. Floors are in-situ cast concrete floors. Small steel frame penthouse structure on the roof.

As the reactor building chosen for this study represents a very stiff building typology, it maybe that conclusions are limited to this typology. Hence, we are still requesting companies to share building configurations with the project in order to keep in focus other typologies than typical reactor buildings. Basically studied configurations are safety related (e.g. diesel generator building, control room complex, turbo-generator pedestal, etc.), but targets of investment protection may also bring significant interest in jet unstudied building types.

6 Exploratory modeling of a reactor building

The calculation in this segment has been carried out guided by provisions of design codes YVL 2.6 [8], and ASCE 4-98 [1].

The aim of the modelling was to explore the range of loads experienced by the studied reactor building, in loading scenarios thought to be typical/expected in Finland.

6.1 Loading

The seismic load is represented as set of artificially generated accelerograms – these representations are acceptable in ASCE 4-98 [1]. The accelerograms were generated using prescriptions of ASCE 4-98 [1], from 2 sets of response spectra – the YVL 2.6 spectra and the spectra proposed for the NPP location in the north of Finland by Fennovoima (Figure 2).

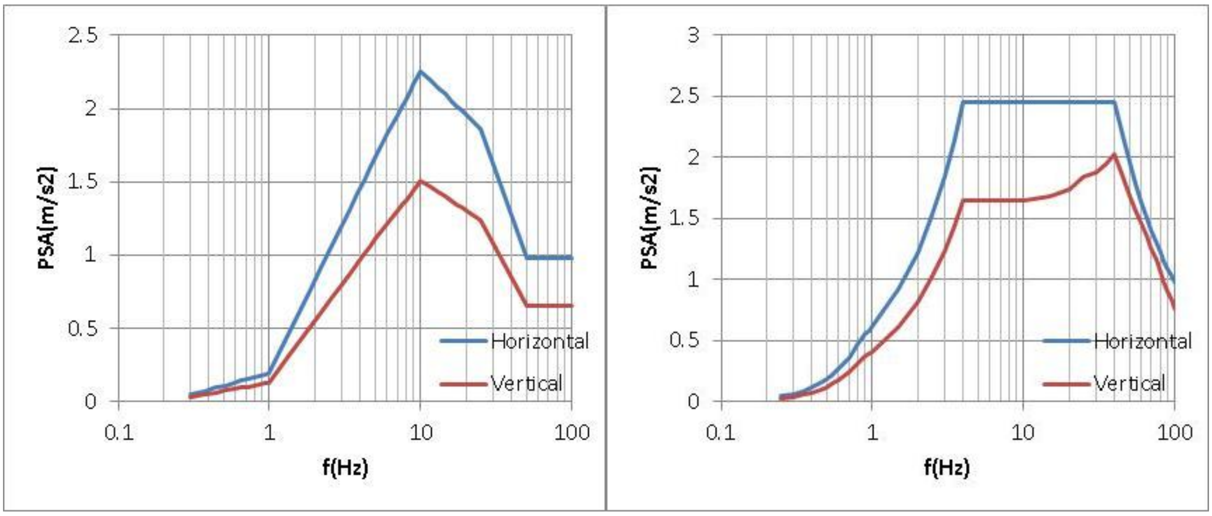


Figure 2. Target response spectra for accelerogram generation (a) YVL 2.6 guide, (b) Fennovoima

Three sets of 3D component accelerograms were generated for the 2 spectra, respecting duration requirements for a 6-5-7 magnitude earthquake [1]. Hence total duration of records was 18.5s, with 1.5s rise time, 7s decay time and 10s duration of strong motion. The cross-correlation of the records was checked and was never exceeding 0.2 (better than code requirement 0.3 [1]). One of the sets created for YVL 2.6 [2] was used for the analysis presented here with a PGA scaled to 0.1g.

Magnitude	Rise Time (t_r)	Duration of Strong Motion (t_{sc})	Decay Time (t_d)
7.0–7.5	2	13	9
6.5–7.0	1.5	10	7
6.0–6.5	1	7	5
5.5–6.0	1	6	4
5.0–5.5	1	5	4

Figure 3. Required duration of strong motion records ASCE 4-98 [1]

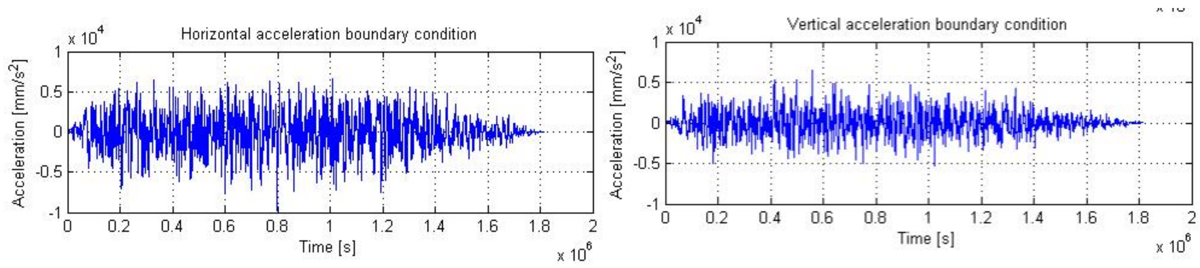


Figure 4. Example pair of horizontal and vertical acceleration components used for loads

NPP design methods place attention on the combination of the 3D effects of earthquakes. We applied the 3 shaking components, of the confirmed independent accelerograms, simultaneously to the base of the reactor model.

The following extreme environmental load combination was used (DIN 1055-100, KTA-GS 78 [9]):

- extreme design load situations
  E_{dA} : $G_k + P_k + A_d + \psi_{1,1} \cdot Q_{k1} + \Sigma(\psi_{2,i} \cdot Q_{ki})$
- design load situation due to earthquakes
  E_{dAE} : $G_k + P_k + A_{Ed} + \Sigma(\psi_{2,i} \cdot Q_{ki})$

Based on DIN 1055-100 and KTA-GS78, the following physical effects are defined:

- independent permanent physical effects, G_k
- independent physical effects due to prestressing, P_k (=0 in this case)
- dominant independent temporary physical effects, Q_{k1}
- other independent varying physical effects, Q_{ki} (i>1)
- extreme physical effects, A_d
- physical effects due to earthquakes, A_{Ed}, based on recommendation of YVL 2.6 [2]

Regarding the design-basis earthquake in accordance with safety standard KTA 2201.1, a value of 1.0 shall be assumed for, both, the weighting factor γ_1 in and the importance factor γ_I in accordance such that the design value A_{Ed} is considered as nominal value.

6.2 Modell description

6.2.1 Basic FEM of the structure

The FE modeling of a complex model of a “generic” reactor building started in 2012. The mesh of the model has been received from FENNOVOIMA and it is a generic version of one of the configurations considered by FENNOVOIMA [10].

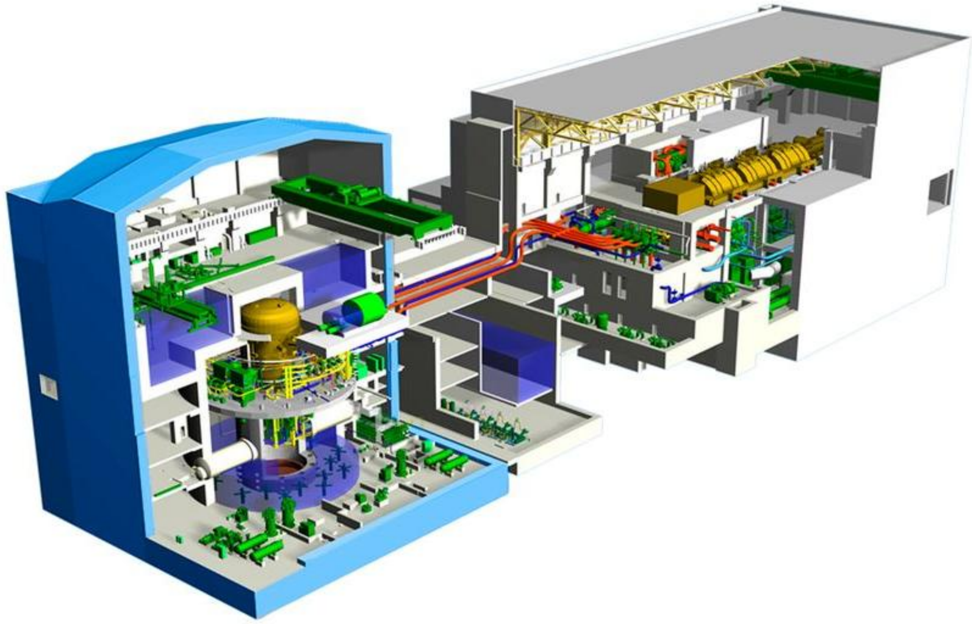


Figure 5. Reactor building (light-blue) analyzed in this study [10]

The basic FE model had the following properties:

- Plan dimensions 61x58m;
- Elevation 55.76m to edge of the dome;
- Outer walls 2m thick;
- Structure supported at base in vertical direction, but horizontal supports are provided by lateral soil;
- Total mass 215998.5 tons, pressure vessel and 2 spent fuel storage tanks not modeled;
- Prevailing mesh size ~0.67m;
- The model was realized in mm, tons and mm/s^2;
- The model had 151158 elements and 143144 nodes with six degree of freedom. 6870 were linear line elements, 144272 were quadrilateral elements and 16 were linear triangular elements.

The full description of the model techniques used was not available to us, only the orphan mesh of the FE model.

Most mesh type elements were modeled with the “Shell / Continuum Shell Homogeneous” element type in ABAQUS, using 5 integration points on the thickness. Some shells, in total 10 sections, modeled using “General shell stiffness” definition. It was impossible for us to back-calculate the equivalent construction elements resulting in these stiffness.

6.2.2 Supports

As stated before, in the initial model received from FENNOVOIMA, vertical movement was suppressed at the base nodes of the model, while lateral/horizontal support was provided to the depth corresponding to embedment in the soil.

In later stages of modeling we concentrated both vertical and horizontal supports to only the base nodes of the model, starting from the hypothesis that the re-fill soil exercises little restraint to the walls structure during earthquake shaking.

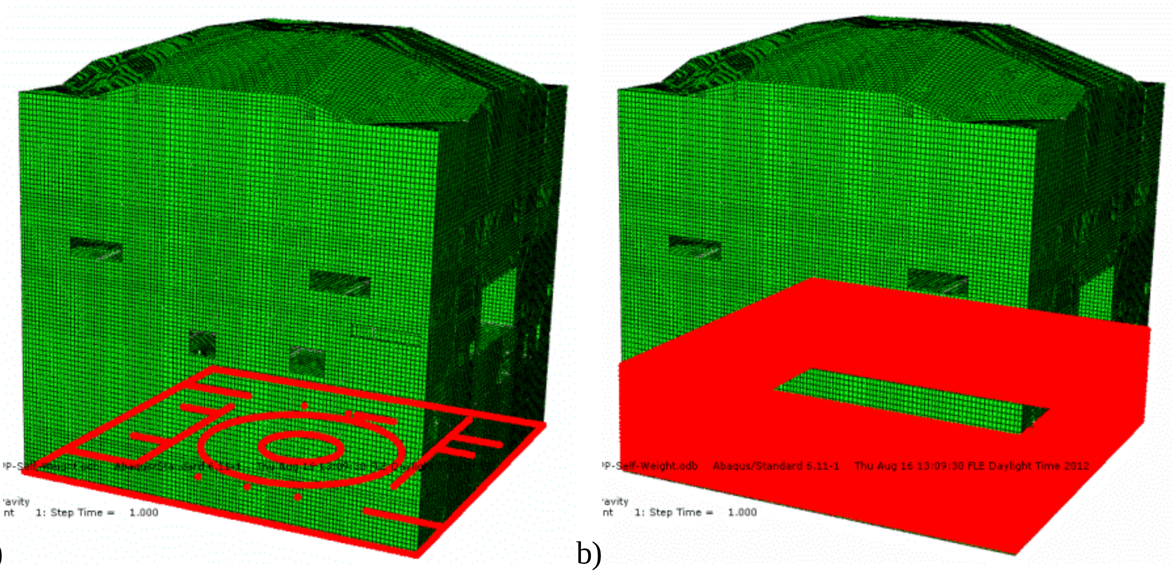

a) b)

Figure 6. Vertical/Z direction (a) and horizontal/X and Y direction (b) support at base nodes

6.2.3 Configuration of the building

The general configuration of the building, as resulting from the FE model is presented in the following drawings. Conventional names of the axes in Figure 7.

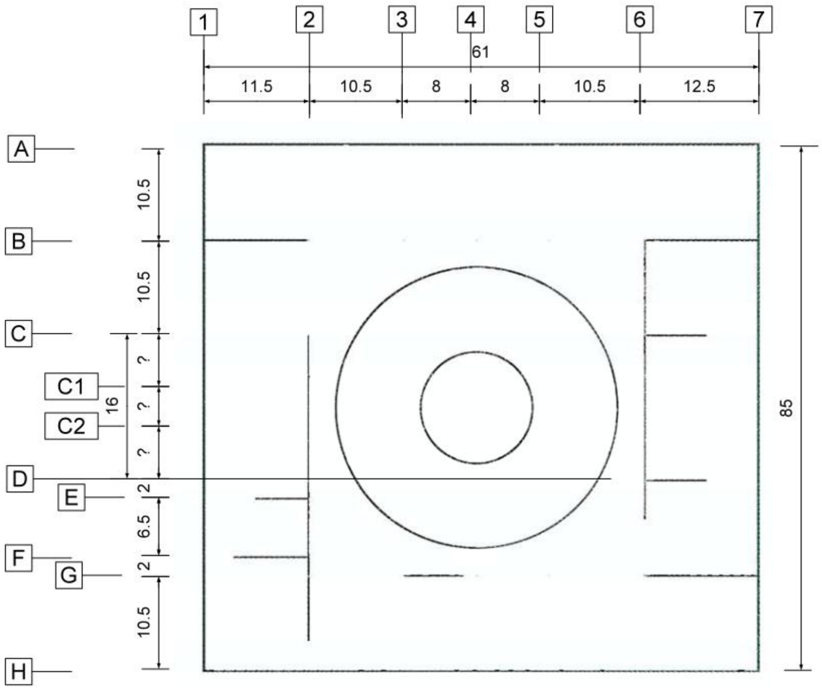


Figure 7. Axes where vertical load bearing elements (walls or columns) are located (meters)

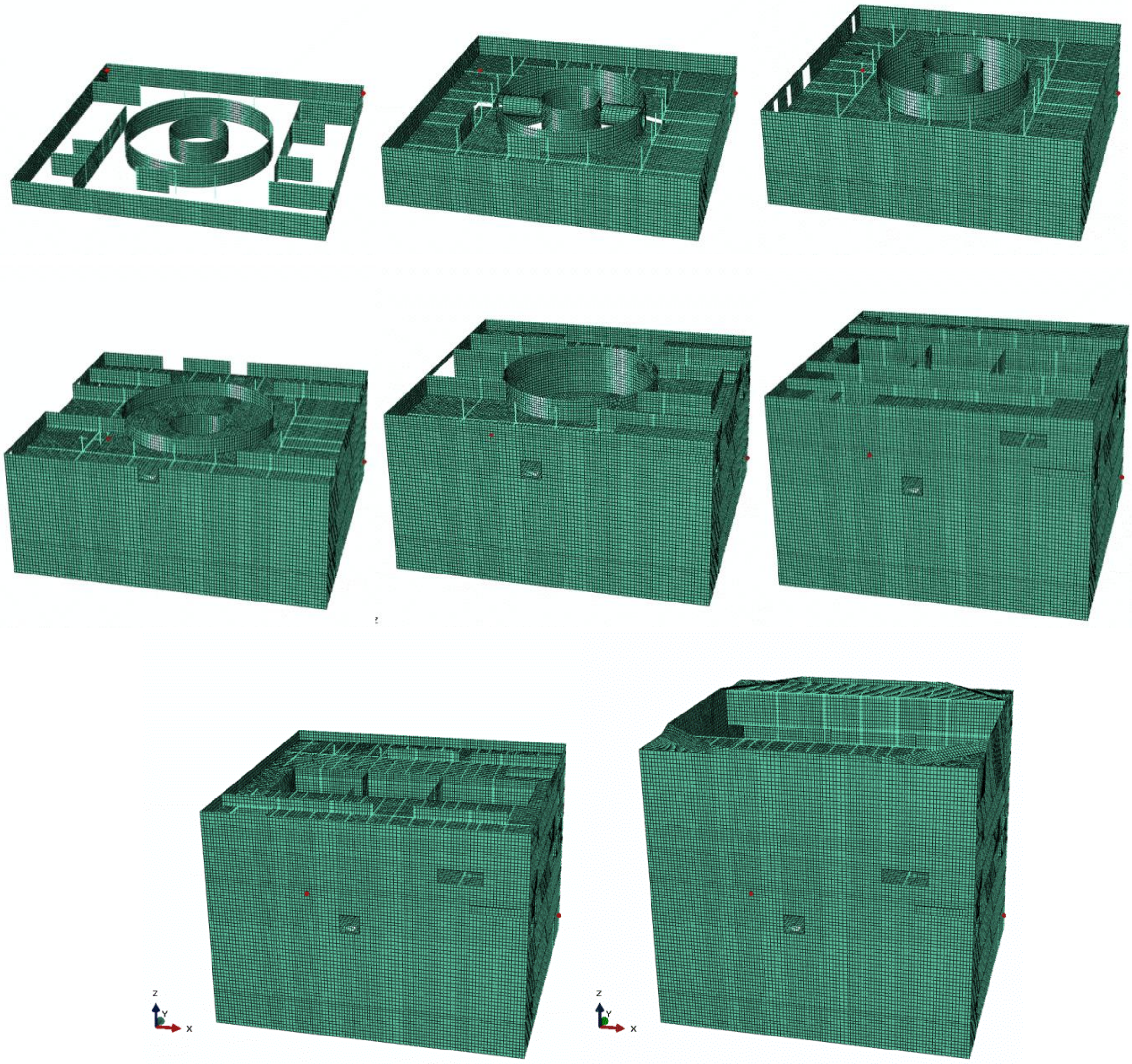
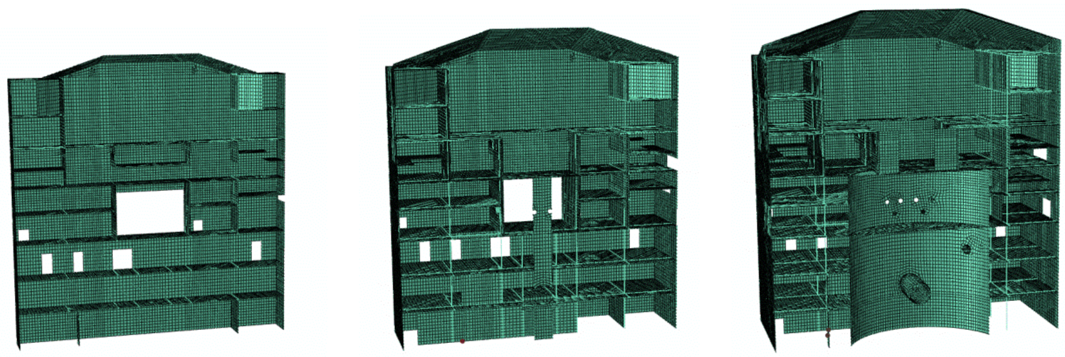


Figure 8. Horizontal diaphragms/complete or partial



Axis 1 Axis 2 Axis 3

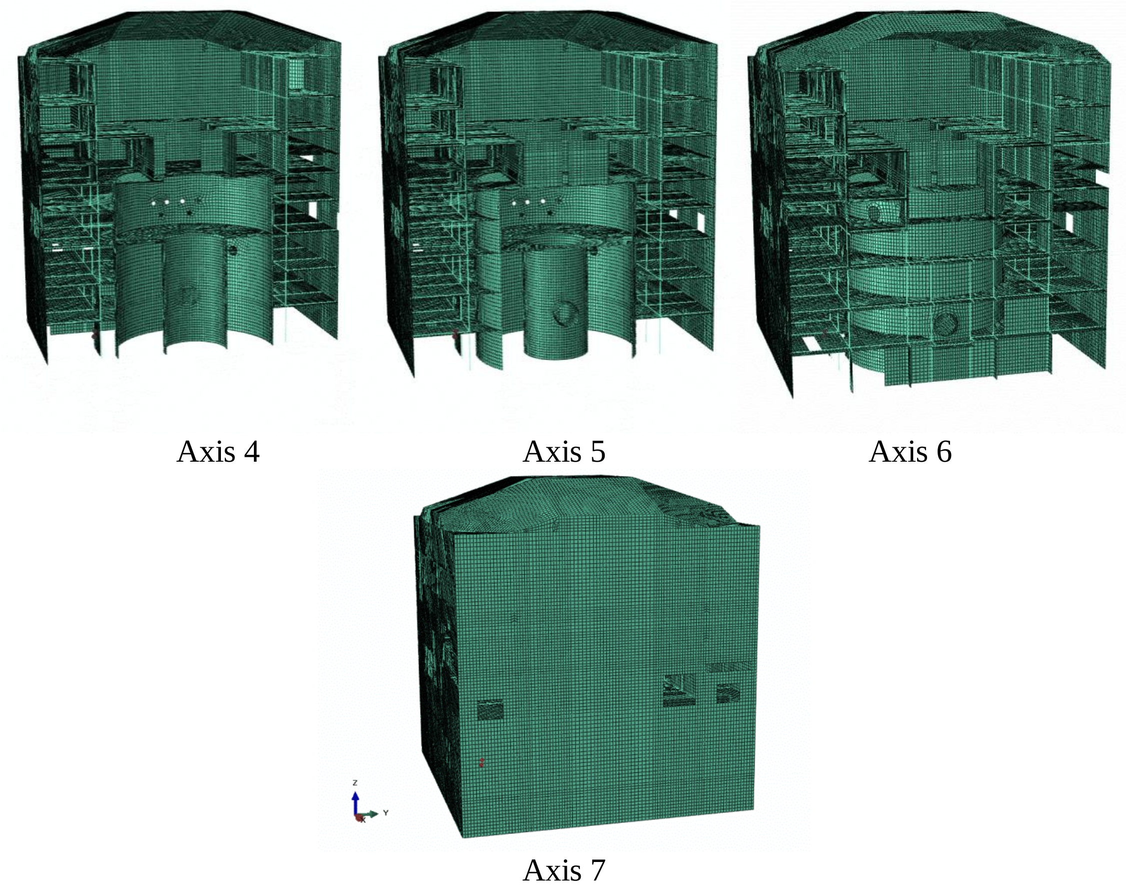
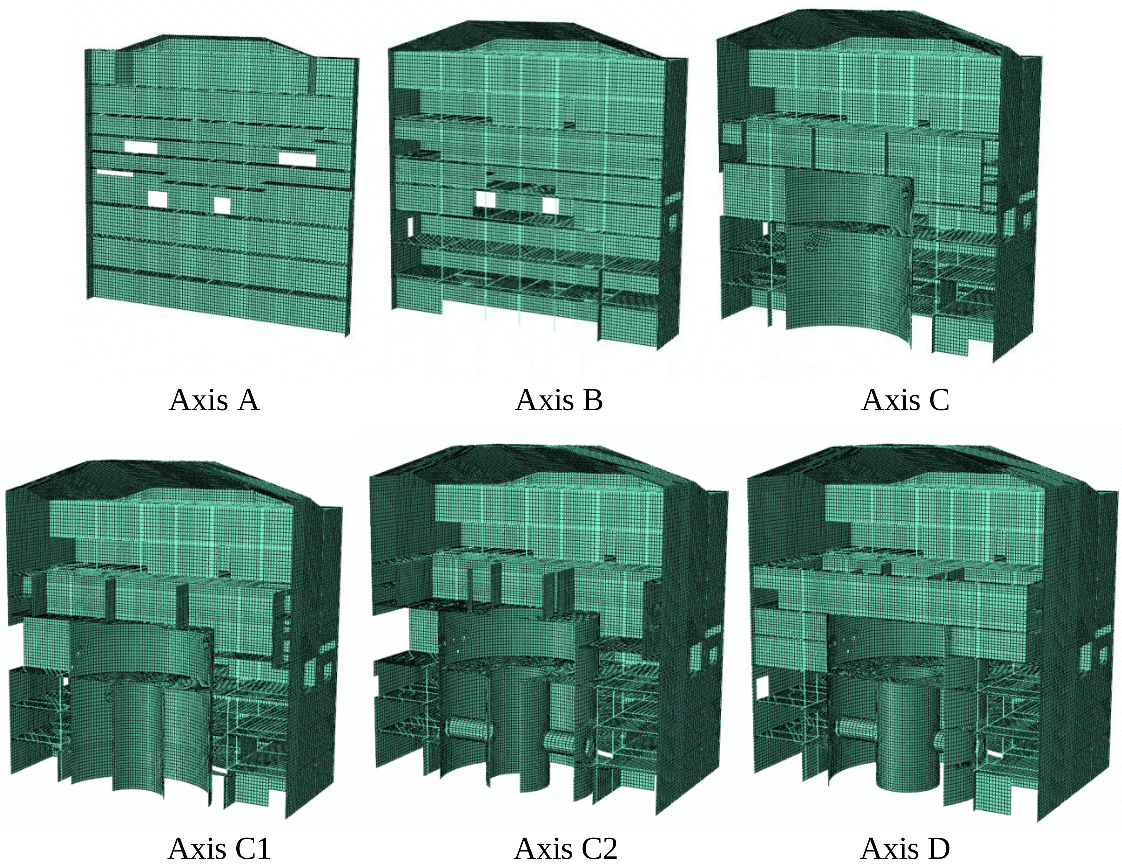
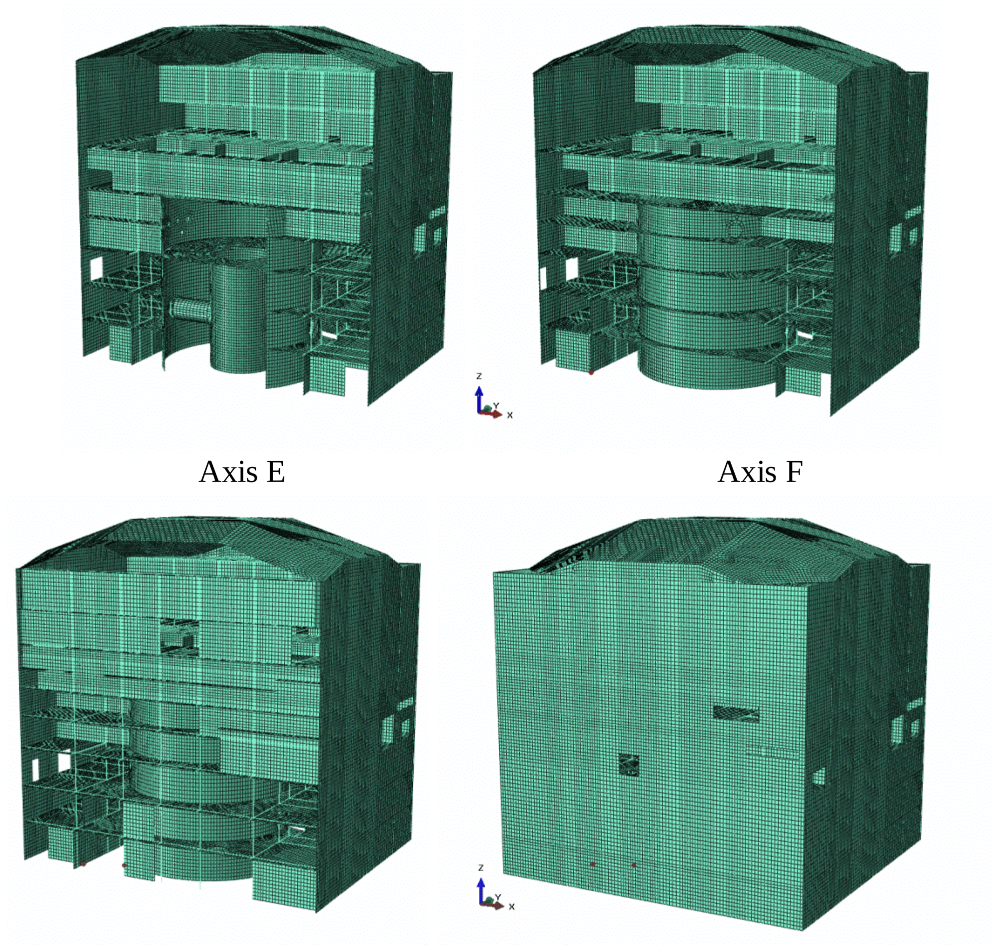


Figure 9. Shear walls in Y global direction





Axis E Axis F

Axis G Axis H

Figure 10. Shear walls in X direction

6.2.4 Modeling of spent fuel storage tanks

The extra masses from the 2 spent fuel tanks and the pressure vessel have been estimated and added to the initial model. Hence the total mass of became 222425 tons.

Table 1. Mass of the initial FE model, and additional masses introduced in the model

	Mass (ton)	
Mass of structure from FEM	215999	100%
+ the spent fuel storage tank - small	2132	0.99%
+ the spent fuel storage tank - large	3494	1.62%
+pressure vessel	800	0.37%
TOTAL	222425	

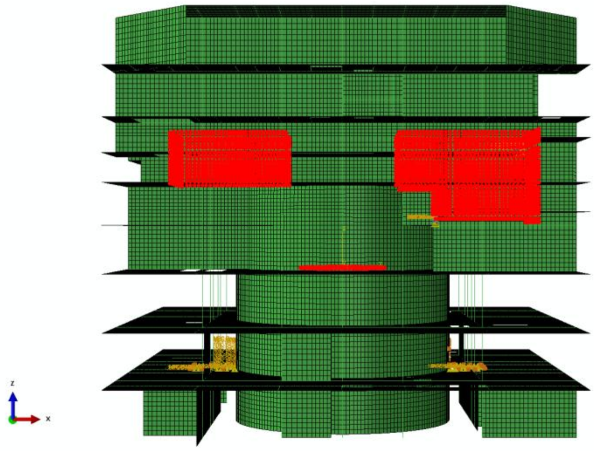


Figure 11. Location of additional masses; the two spent fuel tanks and the support points of the pressure vessel

As stated in Section 3.1.7.1 [1], if mass of equipment is less than 1% of the primary structure (building), no analysis of the interaction between the two is required. Hence, we modeled the pressure as concentrated masses attached to its support points.
The liquid in the pools have been modeled using three techniques:

(1) The water mass has been added to the element nodes in the walls and floor of the pools as concentrated masses. In order to take into account more realistically the behavior of the liquid, the un-isotropic mass option of ABAQUS was used. Hence, mass components acting perpendicularly to the walls of the pools and in the plane of the pool were activated. <u>*Unfortunately this modeling option proved to generate convergence problems.*</u>
(2) The water mass has been added to the element nodes as concentrated masses, using the isotropic mass option. This way one can account to the inertial effect of the water, but spurious components of inertia in the plane of the reservoir are also created (i.e. as if water would be able to transmit shear forces to reservoir walls). <u>*These models proved to be numerically stable.*</u>
(3) Previous models estimate the inertial forces from the water more or less correctly; however there is no possibility to evaluate vibrations transmitted by the liquid and sloshing in the pools. As sloshing is expected to damp vibrations. A more sophisticated way of modeling water was used as described below. In this case both vibrations transmitted by the liquid and effects of sloshing can be estimated.

When considering water sloshing in a reservoir simulations with a traditional Lagrangian FEM domain, the huge node distortion creates difficulties solve the problem. For such simulations, Abaqus 6.12-3 have a procedure where water sloshing simulation can be carried out in a coupled Euler-Lagrangian domain. The water modeled in the Eulerian domain. The reservoir walls and bottom were modelled in the Lagrangian domain. The principal concept in modeling with Eulerian domain is the idea of a material flow through element mesh. At the beginning of the simulation prescribed condition of occupancy of the material in the void is given i.e. water in the void. Due to external excitation on water, it begins to slosh and occupies void volume. Additional Eulerian boundary conditions were defined. On the boundaries of the void, material outflow was allowed but inflow was prevented. Due to spatial movement of Lagrangian domain reservoir, a mesh motion boundary condition was given to Eulerian domain water. That is the Eulerian domain follows by short delay the movement of Lagrangian domain. By doing this the Lagrangian domain will be always inside of the Eulerian domain. This holds true assumption that Eulerian domain is larger than Lagrangian domain.

The water material model was defined with Mie-Grunesein equation of state (EOS), density, viscosity and specific heat. Interactions between wall, bottom and water can be modeled with general contact, where tangential friction was assumed rough i.e. slip won't occur in contact nodes. The normal contact was assumed hard contact and nodal separation was allowed. *These models proved to be numerically stable.*

6.2.5 Damping

The classical Rayleigh damping has been used for modeling (Figure 12). Two sets of damping factors α and β were calibrated to a targeted 4% uniform damping level (Table 3.1-1 in ASCE 4-98 [2]). The main objective of the second model was to see if reduced damping on higher vibration modes has significant effect on the frequency content of vibration transmitted by the structure to components:

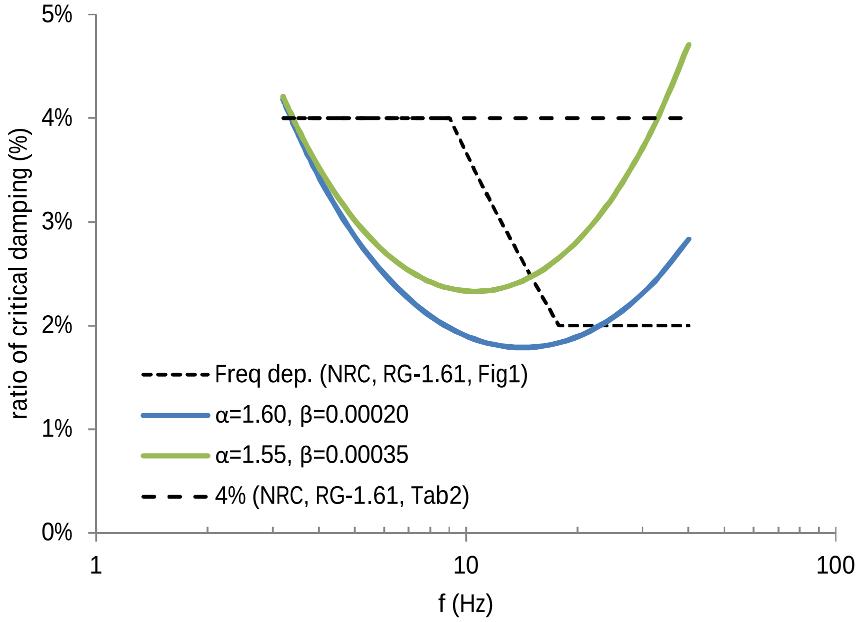
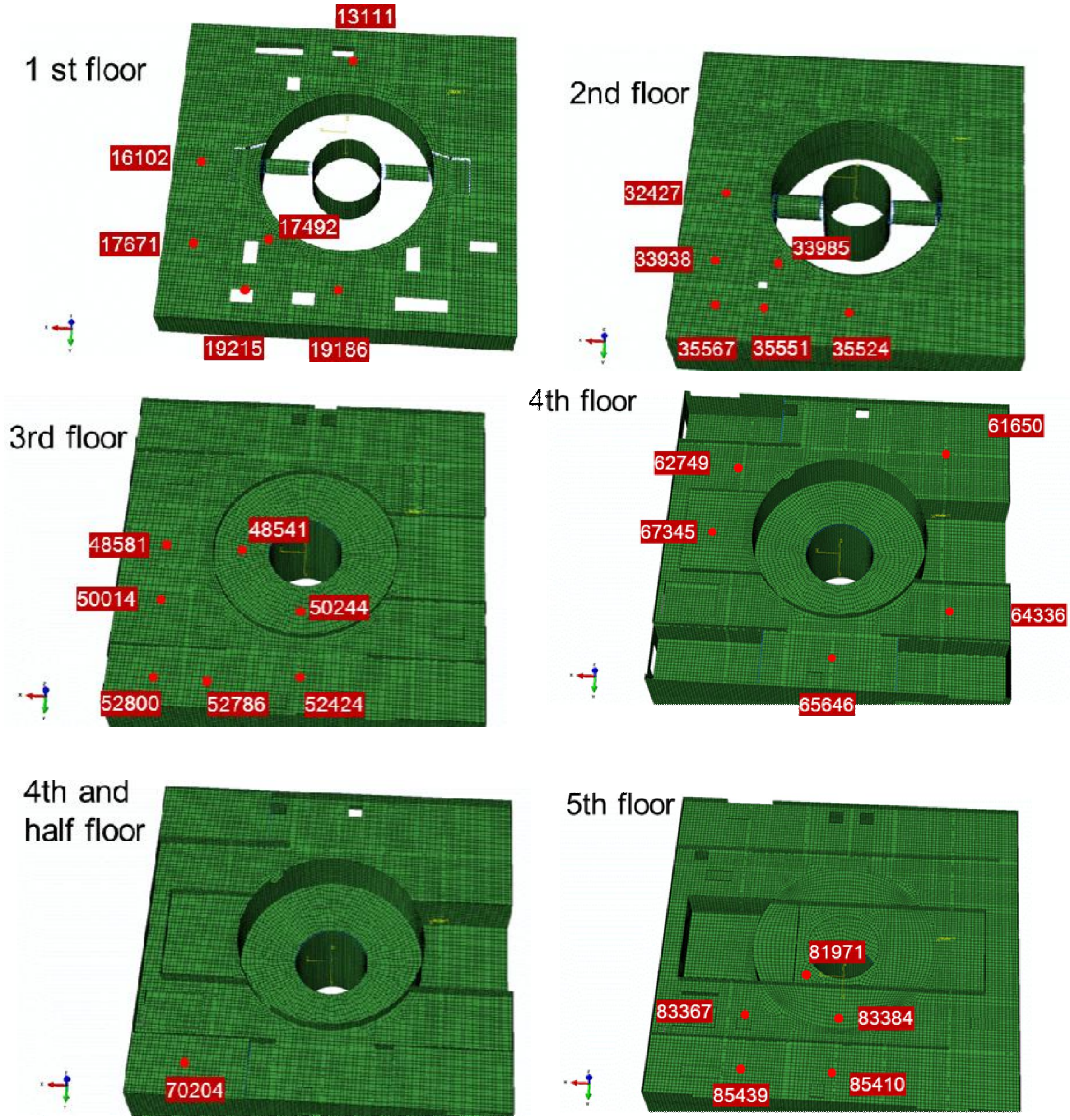


Figure 12. Rayleigh damping definitions used in the models

6.2.6 Points of measurement for floor spectra

For the evaluation of the floor spectra the following nodes in the FE mesh were monitored:


1 st floor
13111
16102
17492
17671
19215
19186
2nd floor
32427
33938
33985
35567
35551
35524
3rd floor
48581
48541
50014
50244
52800
52786
52424
4th floor
62749
61650
67345
64336
65646
4th and
half floor
70204
5th floor
81971
83367
83384
85439
85410

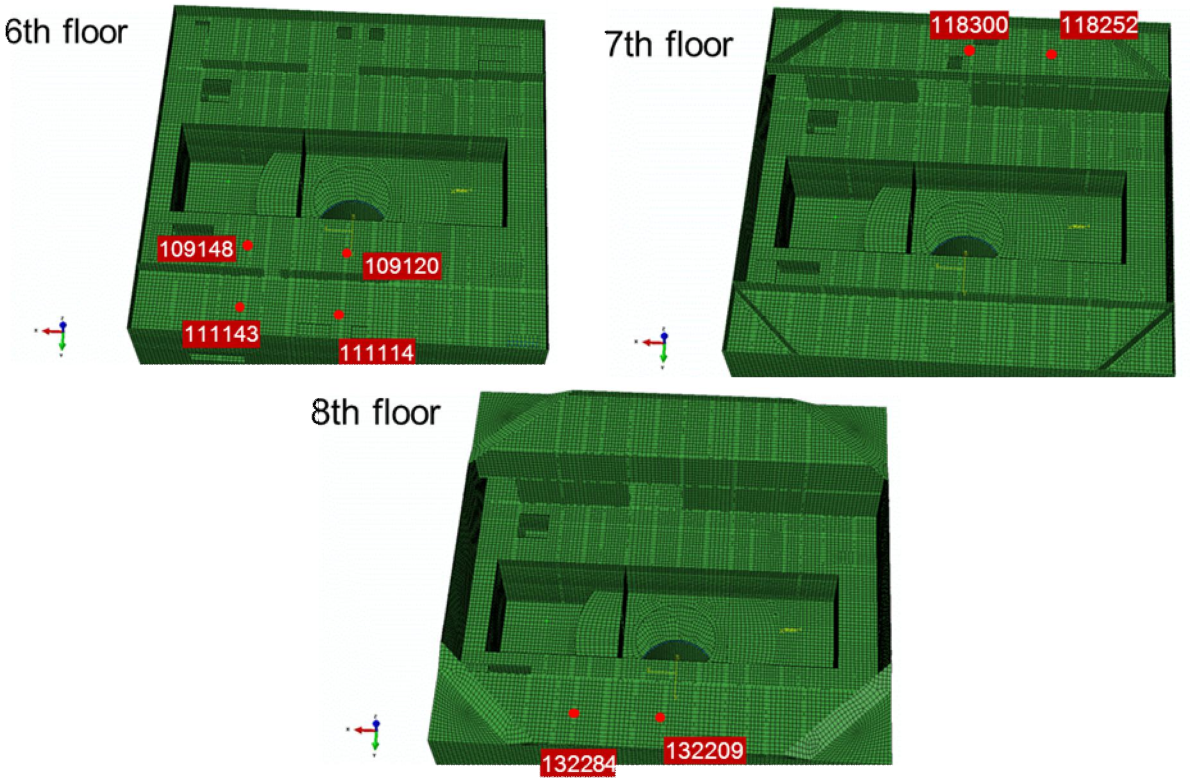


Figure 13. Monitoring points for calculating floor spectra

6.2.7 Summary of model configurations

Table 2. Configurations of the FE model analyzed in SP2

Model	Description	Eigenvalue analysis	Time domain – Explicit	Run time of Explicit model
ORIG	- Model as received	Yes	-	-
M1	- Base support concentrated to base nodes only. See section 6.2.2; - Additional masses Table 1 modeled as <u>un-isotropic</u> concentrated mass elements;	Yes	No convergence	-
M2	- Base support concentrated to base nodes only. See section 6.2.2; - Additional masses Table 1 modeled as <u>isotropic</u> concentrated mass elements;	Yes	Yes	2 days, 4 CPU's
L1	- Base support concentrated to base nodes only; - Water modeled in Eulerian domain	NA	Yes	13 days, 16 CPU's
NL	- Base support concentrated to base nodes only; - non-linear properties of concrete;	-	Next step	-

6.2.8 Expected effect of mesh size on frequency content

As in this case one of the focus areas of the modeling is to evaluate high frequency vibration, the adequacy of the mesh size for transporting vibrations is checked. The external wall thickness is 2000mm, and the prevailing mesh size of

670mm. The basic concrete material properties used throughout the model are: E=27800 N/mm^2, Poison's ratio ν=0.17 and density of the material ρ=2.4*10^{-9} t/mm^3. Deviations from density are observed in floors, where they go up to ρ=5.23*10^{-9}t/mm^3. (probably life loads are included in the model this way – but we cannot check this with developer of the original mesh).

Several sources define limits for the wave-length (λ) of vibration transported by a mesh with node distance d. ASCE 4-98 [1] is suggesting a less strict criteria of d<λ/5 for soil modeling. In other contexts the more strict limits of d<λ/8 up to d<λ/12 have also been used for vibration transported in solid medium [11], [12].

The slowest wave to propagate being the shear wave, its velocity can be calculated as.

$$v = \sqrt{\frac{G}{\rho}} = \sqrt{\frac{E}{2(1+\upsilon)\rho}}$$

And the highest frequency calculated with:

$$f_{\max} = \frac{v}{\lambda}$$

We can estimate the upper limit of frequencies as 277Hz (Table 3). The conclusion is that mesh size is not limiting the validity of the analysis – in fact we could increase size of the elements 2..3 times if needed for computational efficiency.

Table 3. Estimate of highest frequencies transported by FE mesh

E	ν	G	ρ	V_s	d	λ=12*d	f_{max}
N/mm^2		N/mm^2	t/mm^3	m/s		12	Hz
27800	0.17	11880.34	2.4E-09	2225	0.67	8.04	277

6.3 Results

6.3.1 Selected eigenvalue analysis results

ORIG configuration

Table 4. Summary of mass participation of first 200 modes (f_{max}=19.5Hz)

Quantity	$Mass_X$	$Mass_Y$	$Mass_Z$	$Part_X$	$Part_Y$	$Part_Z$
	(tons)	(tons)	(tons)	(%)	(%)	(%)
200 modes (f_{max}=19.5Hz):	1.71E+05	1.75E+05	1.79E+05	82%	83%	86%

Table 5. Individual participation factors of first 20 modes

			Effective mass - trans (tons)			Effective modal mass percentage		
Mode	Freq(Hz)	Period(s)	X	Y	Z	X	Y	Z
1	3.89	0.26	0.139532	118253	7.9525	0%	56%	0%
2	4.53	0.22	117575	1.58552	23.9849	56%	0%	0%
3	5.94	0.17	0.499657	12.7913	23621.8	0%	0%	11%
4	6.10	0.16	0.465904	25.9869	3.76507	0%	0%	0%
5	6.65	0.15	100.85	103.881	486.788	0%	0%	0%
6	6.66	0.15	0.448609	184.131	1088.94	0%	0%	1%
7	6.67	0.15	173.224	0.058234	8.991	0%	0%	0%
8	7.00	0.14	59.7156	54.8587	1.86394	0%	0%	0%
9	7.23	0.14	17.0316	9725.95	2.46461	0%	5%	0%
10	7.67	0.13	0.546829	1079.79	2.17312	0%	1%	0%
11	7.79	0.13	0.595793	4.75001	1196.62	0%	0%	1%
12	7.92	0.13	138.948	8053.43	2792.2	0%	4%	1%
13	8.01	0.12	48.4151	2110.83	13599.4	0%	1%	6%
14	8.10	0.12	2542.19	774.011	62.5578	1%	0%	0%
15	8.74	0.11	2755.1	286.173	67549.4	1%	0%	32%
16	8.79	0.11	24623.4	600.784	18450	12%	0%	9%
17	8.97	0.11	133.643	15.9828	806.608	0%	0%	0%
18	9.07	0.11	16.9277	1.50177	163.175	0%	0%	0%
19	9.08	0.11	2.06253	6.16582	60.2662	0%	0%	0%
20	9.52	0.11	2911.95	70.3786	5405.72	1%	0%	3%

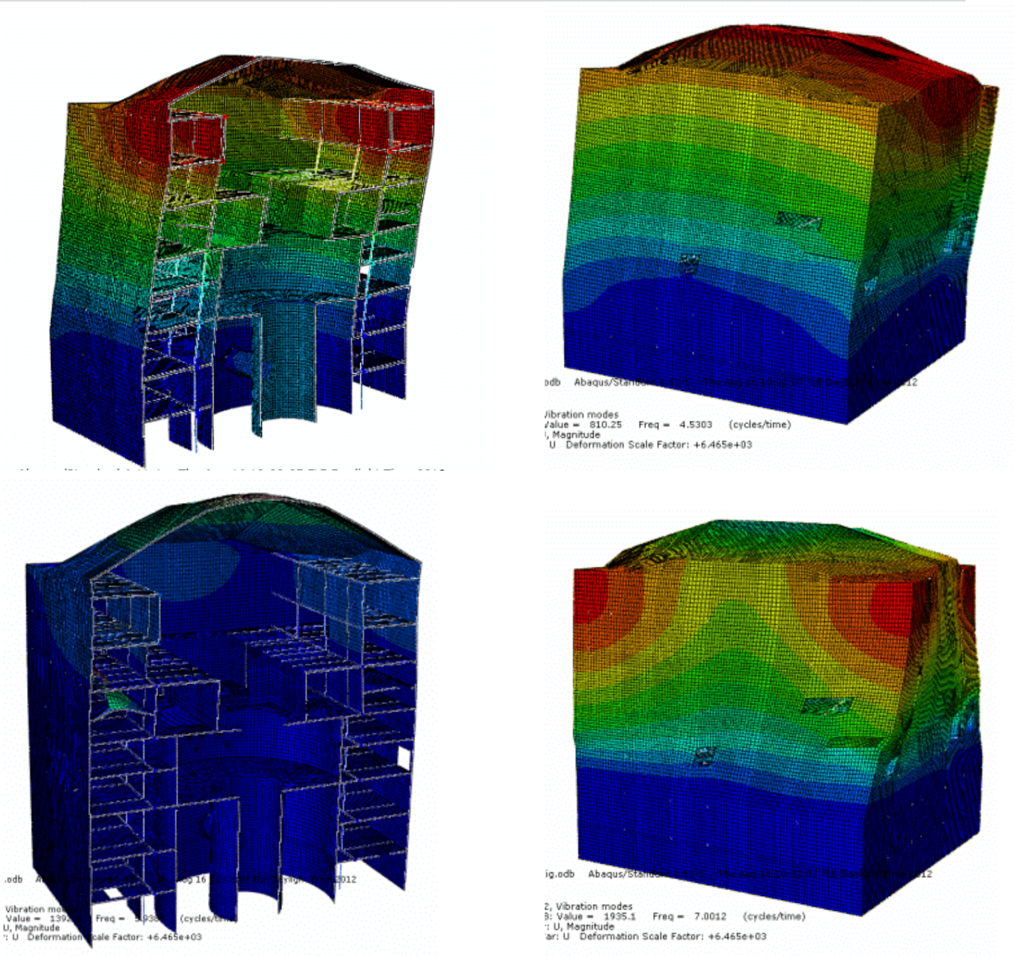


Figure 14. Mode shapes of the modes 1, 2, 3 and 8 of model ORIG

M1 configuration
Compared to the ORIG configuration, the modal analysis of M1 yielded results that are fully expectable considering (1) the increase of masses, due to presence of the liquid in the pools and (2) the decrease of stiffness because of the change in the support conditions. Hence the following fundamental vibration modes and mass participating factors were recorded:

Table 6. Summary of mass participation

Quantity	$Mass_X$	$Mass_Y$	$Mass_Z$	$Part_X$	$Part_Y$	$Part_Z$
	(tons)	(tons)	(tons)	(%)	(%)	(%)
Modes up to 40Hz/778 modes:	2.16E+05	2.16E+05	2.02E+05	97%	97%	91%
First 20 modes				85%	83%	65%
First 100 modes				91%	92%	80%

Table 7. Individual participation factors of first 20 modes

	f	T	$Mass_X$	$Mass_Y$	$Mass_Z$	$Part_X$	$Part_Y$	$Part_Z$
	(Hz)	(s)	(ton)	(ton)	(ton)	(%)	(%)	(%)
1	3.20	0.31	34	160629	6	-	72%	-
2	3.59	0.28	165910	36	24	75%	-	-
3	5.50	0.18	28	106	70	-	-	-
4	5.93	0.17	0	10	23243	-	-	10%
5	6.10	0.16	0	30	7	-	-	-
6	6.64	0.15	134	475	261	-	-	-
7	6.66	0.15	10	374	1544	-	-	1%
8	6.67	0.15	170	3	39	-	-	-
9	7.05	0.14	66	20870	1	-	9%	-
10	7.62	0.13	10650	152	13750	5%	-	6%
11	7.68178	0.13	2045	22	6278	1%	-	3%
12	7.73914	0.13	7660	77	12533	3%	-	6%
13	7.79232	0.13	140	1	586	-	-	-
14	8.08249	0.12	248	268	1337	-	-	1%
15	8.36629	0.12	344	56	39702	-	-	18%
16	8.96588	0.11	27	53	580	-	-	-
17	9.06668	0.11	14	0	1060	-	-	-
18	9.0775	0.11	6	1	461	-	-	-
19	9.17165	0.11	857	453	42461	-	-	19%
20	9.52323	0.11	21	717	422	-	-	-

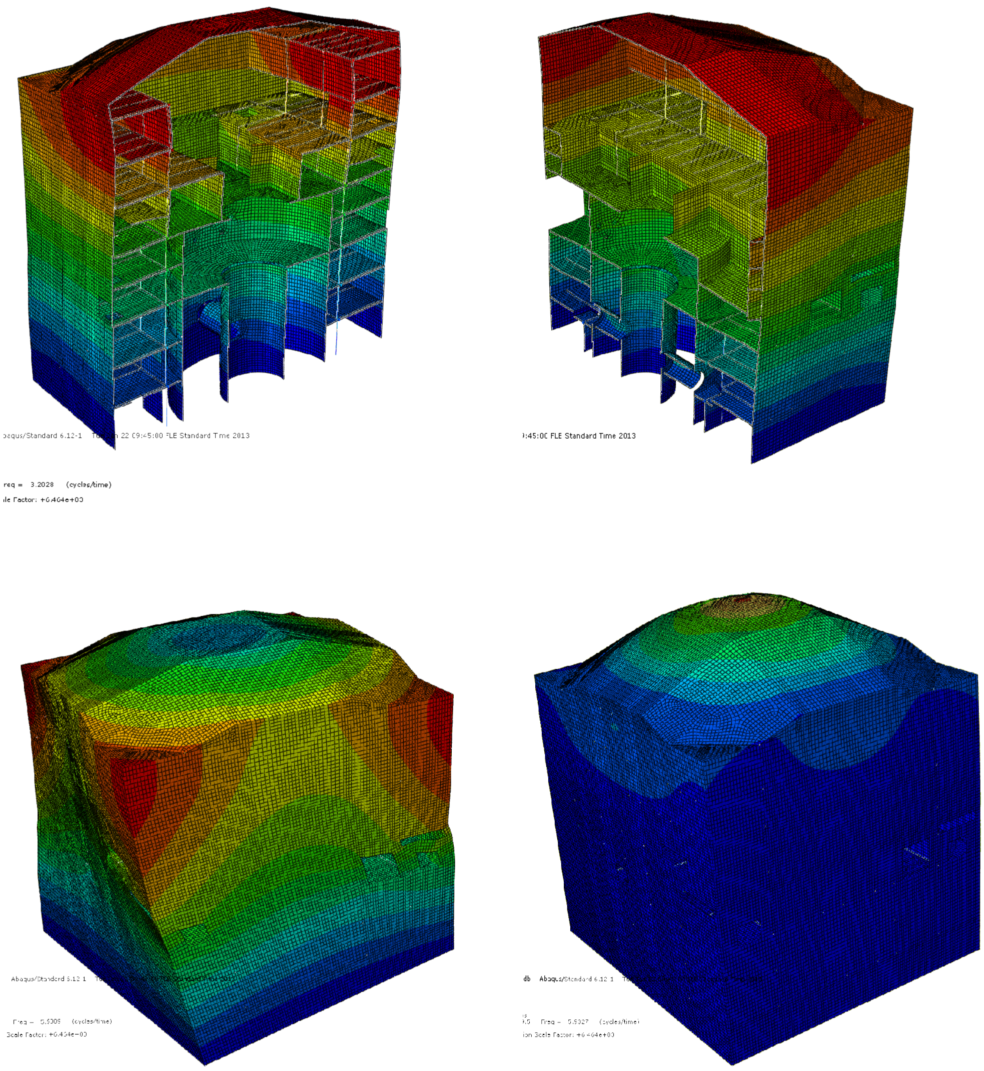


Figure 15. Mode shapes of the first 4 modes of configuration M1

6.3.2 Selected results of explicit analysis

Concentrated mass model (M2 configuration)

The summary of horizontal and vertical floor spectra, calculated with 5% damping, is presented for the M2 model in Figure 16 and Figure 17. It can be observed that shape of Floor 1 horizontal spectra is very close to the spectra of the excitation. Already in Floor 4 response, but more prominently in Floor 8 response, the fundamental frequencies of the model in the 2 horizontal directions (between 3 and 4Hz) are showing up as peaks.

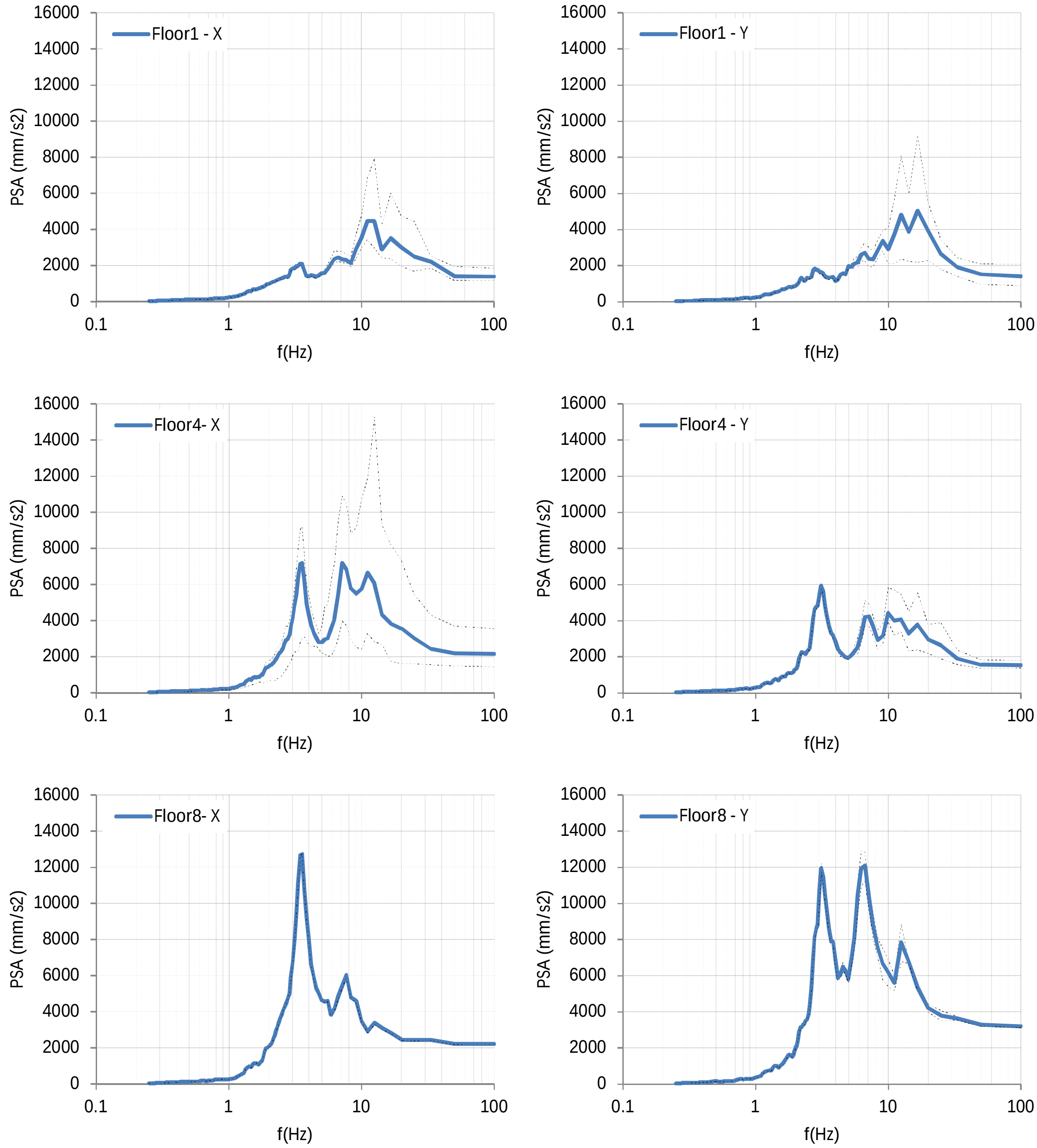


Figure 16. Floor spectra in horizontal directions (X and Y) – average, minimum and maximum values for all measuring points on for Floors 1, 4 & 8 of model M2

In the vertical direction more amplification can be noticed already in Floor 1 response. The predominant frequency of vertical signals is higher, with peak developing in the range of frequencies 8-11Hz.

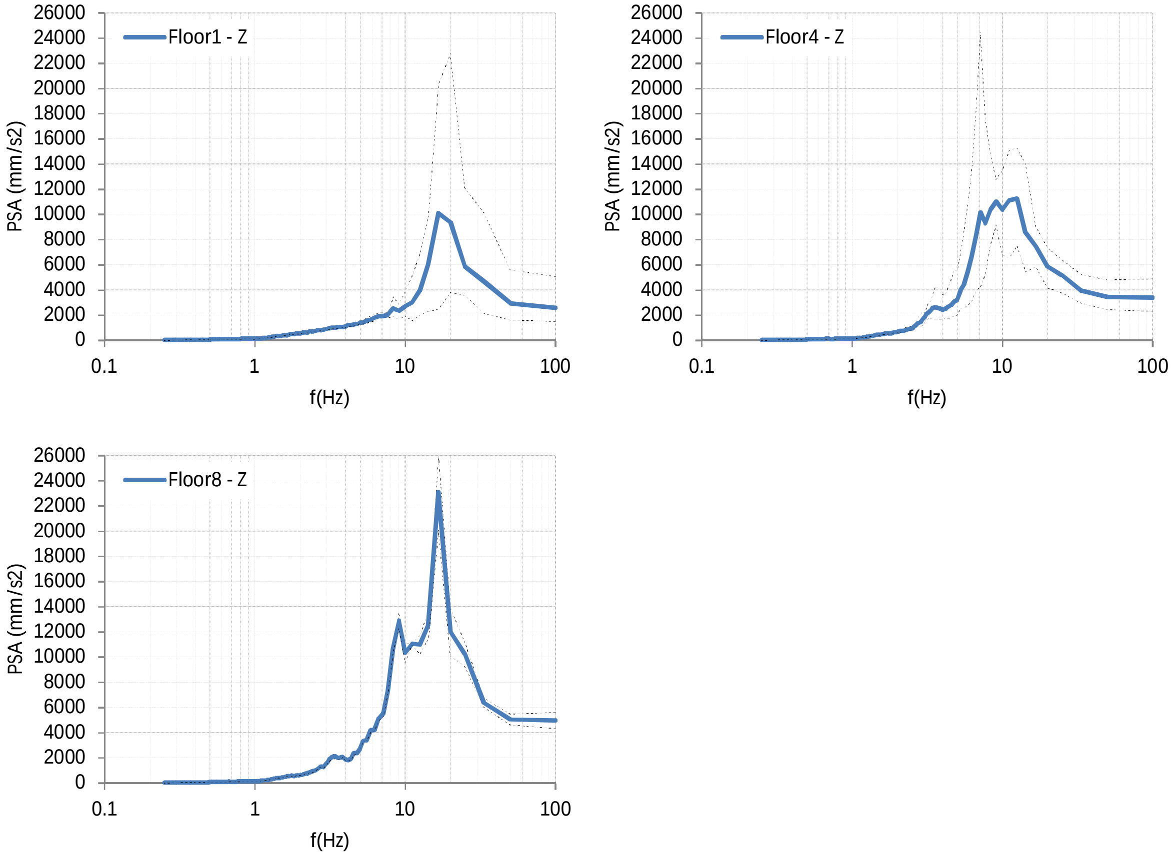


Figure 17. Floor spectra in vertical direction (Z) – average, minimum and maximum values for all measuring points on for Floors 1, 4 & 8 of model M2

Water modeled in Eulerian domain (L1 configuration)

The water sloshing model is fully compatible with the modeling of water using concentrated masses, as it can be observed from Figure 18 and Figure 19. The main advantage of this modeling technique for reactor building is the ability of the model to predict sloshing height in the pools (Figure 20). However, the computational effort is also quite significant, practically doubling analysis time for the Fem used in this analysis.

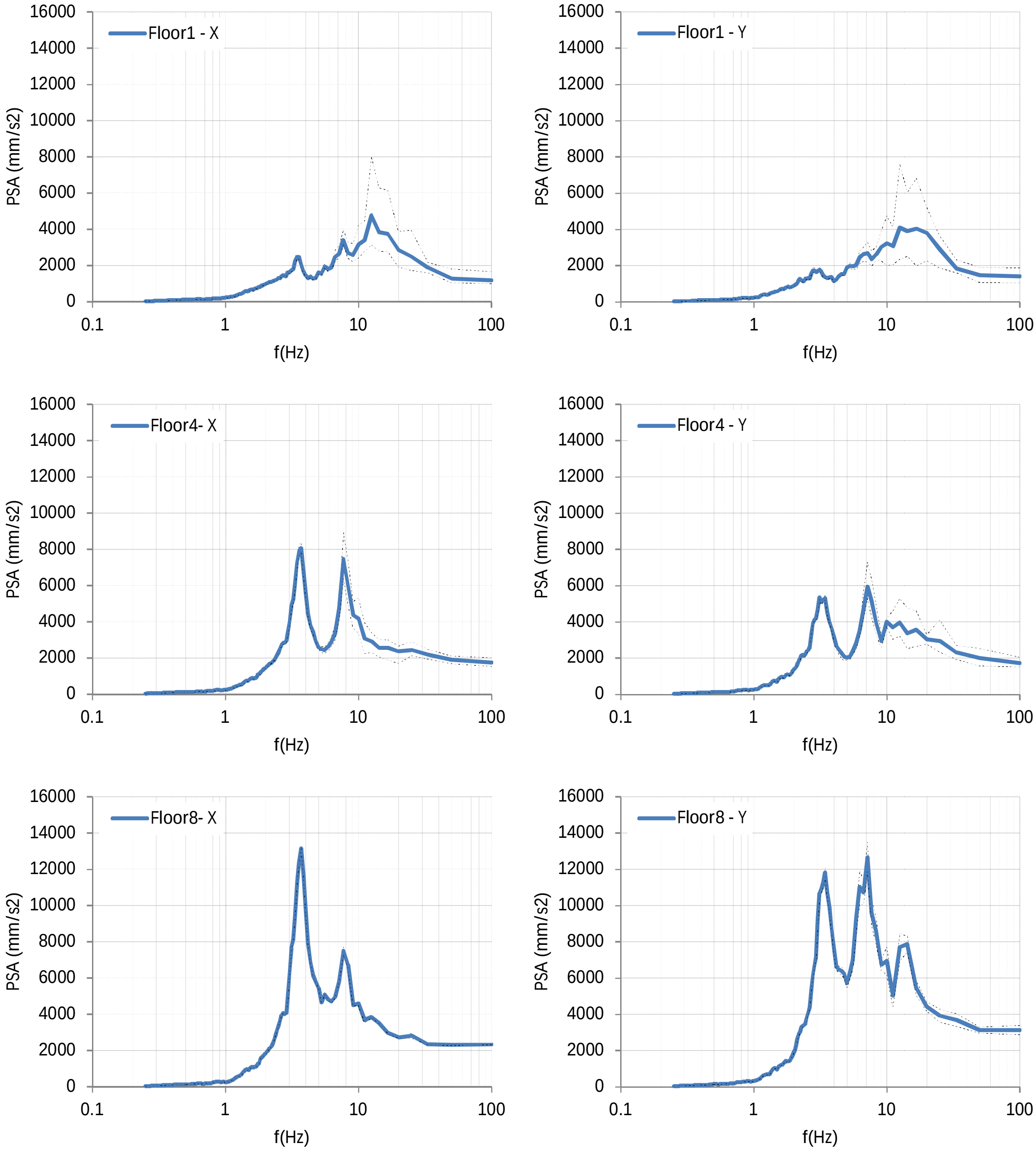


Figure 18. Floor spectra in horizontal directions (X and Y) – average, minimum and maximum values for all measuring points on for Floors 1, 4 & 8 of model L1 ***Note:*** *Node 67345 (Floor 4) was removed from this interpretation because it is in direct contact with the liquid, and resulting signals are disturbed by local effect.*

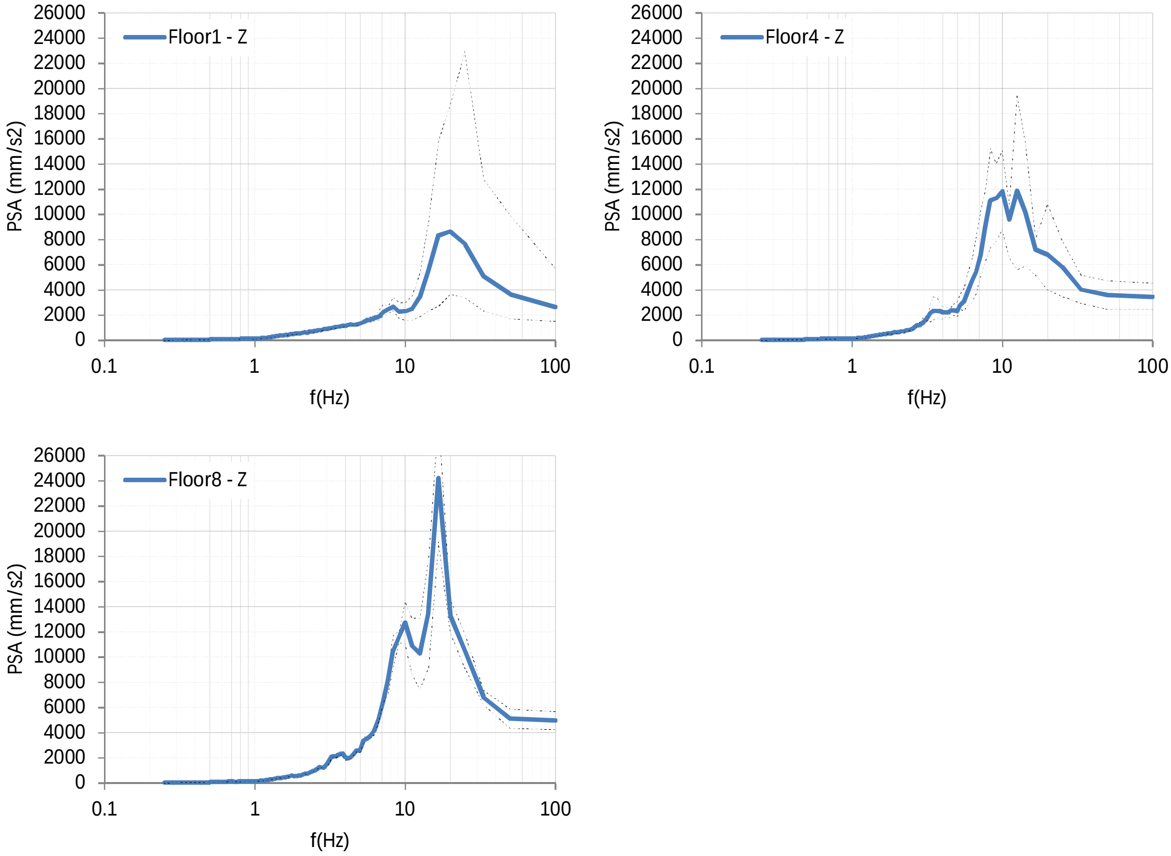


Figure 19. Floor spectra in vertical direction (Z) – average, minimum and maximum values for all measuring points on for Floors 1, 4 & 8 of model L1. ***Note:*** *Node 67345 (Floor 4) was removed from this interpretation because it is in direct contact with the liquid, and resulting signals are disturbed by local effect.*

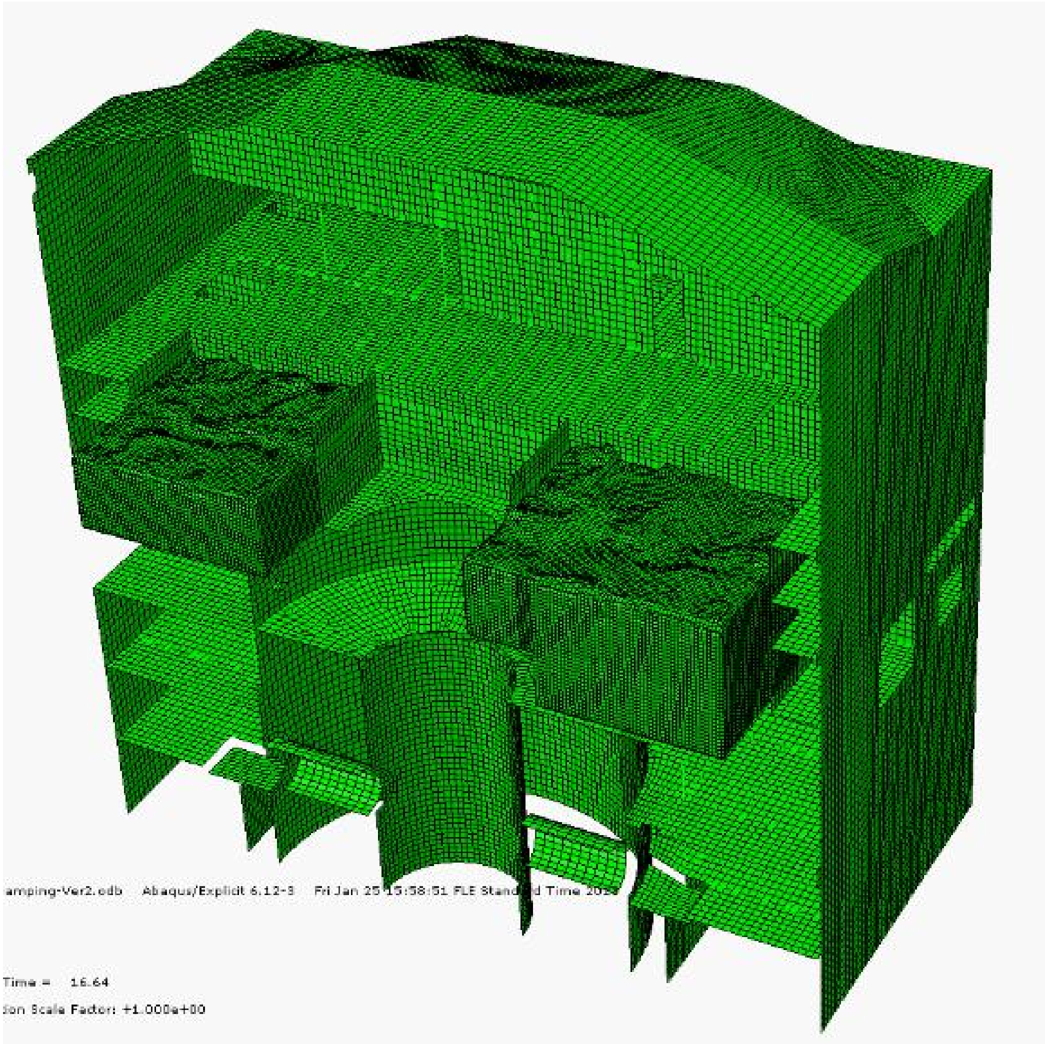


Figure 20. Sloshing of liquid in the pools due to earthquake shaking

7 Conclusions

The following preliminary conclusions and recommendations for can be presented:

The modeling of water masses as concentrated mass elements seems adequate for the purpose of assessing globally the performance of the building;

Both modeling techniques are feasible, but on the upper limit of affordability in terms of analysis time. Future parametric study should consider the required effort to simplify the models. On the other hand CPU time is only one factor when it comes to considering the cost of modeling. The costs are strongly driven also by the work time of modeler.

Two main directions of modeling are planned for exploration in 2013:

1) To explore in detail the effect of the Fennovoma type load spectra (Figure 2, b) on the shape of floor spectra.

2) To estimate stress levels in the different elements of the structure, and to extend the model to include concrete plasticity as source of beyond elastic response. Obviously this will also require the deviation of loads from the current levels used, as the 0.1g YVL 2.6 is not expected to generate significant stresses in the concrete structure. It is a question for 2013, if we can at all formulate credible beyond design basis loading scenario, which would force this structure to reach its elastic limit. It may be that choosing such a robust and stiff building configuration is a disadvantage from this point of view.

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