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Non Linear Seismic Analysis of the New Icelandic CCC

In this paper, is presented part of a complete thesis version entitled “Seismic Analysis of the New Icelandic Conference and Concert Centre, CCC”. This full version thesis [7] was developed by Rúben Murargi and Jorge Lopes, during the ERASMUS Exchange Program at DTU, Denmark, 2007.

The irregularity of the main structural system of the building, both in plan and in elevation, means that the simplified modal response method is not applicable. Because of this irregularity, a spatial multi modal response spectrum analysis was performed by Rambøll’s design team to assess the seismic response of the structure during an earthquake.

The main objective of the study elaborated in full version [7] is, in a first phase (Linear Range) [9], to run a multi modal response spectrum analysis with the elastic response spectrum and compare results with a linear time history analysis. The results should be similar. However, the irregularity of the building does not permit an immediate conclusion. In a second phase (Non Linear Range), a non linear time history analysis will be run and the results will be compared with those obtained through a multi-modal response spectrum analysis, using the design response spectrum. No non linear dynamic analysis was performed by Rambøll’s design team.

In the linear range, the seismic action was defined through the elastic and the design response spectrum and accelerograms based on the elastic response spectrum.

In the non linear range, the seismic action was defined through the design response spectrum (where the non linear behaviour is given by the behaviour factor q) and through the same accelerograms used for the linear time history analysis. Here, the non linear behaviour is given by plastic hinges.

This paper presents a summary of the thesis entitled “Non Linear Seismic Analysis of the CCC” presented in Portugal by Jorge Lopes [8].

As part of this methodology, synthetic accelerograms are created, based on site-specific response spectrum, and used in the non linear time

history analysis of CCC, in which plastic hinges have been modeled using bi-linear hysteretic models.

The results obtained from the non linear time history analysis and from a multi modal response spectrum analysis, using the design response spectrum, are compared and significant deviations in load effects are found in some key locations along the building. The significance of these deviations is discussed. Additionally, some further work is suggested in order to improve the model and optimize the results.

NOTATION

- a_{gR} Reference peak ground acceleration
- a_g Design ground acceleration
- G_{kj} Characteristic value of permanent action
- ψ_{Ei} Combination coefficients for variable action i
- Q_{ki} Characteristic value of variable action
- γ_{III} Importance factor
- S Soil factor
- ξ Viscous damping ratio (in percent)
- q Behaviour factor
- k_w Factor reflecting the prevailing failure mode in structural systems with wall
- q_o Basic value of the behaviour factor, dependent on the type of the structural system and on its regularity in elevation

1. INTRODUCTION

The government of Iceland and the City of Reykjavik intend to create a major complex in central Reykjavik, the Conference and Concert Hall Centre, CCC.

As is expected, in this kind of complex project, many companies are involved in all the studies before, during and after the construction. Of all of these enterprises, Rambøll Denmark, a Danish company who works within the fields of engineering, environment and management, has made, among other studies, some seismic analysis of this project according to EC8 [3]. Scheduled for completion by the autumn of 2009, the complex, which is being financed by a mix of state, city and private-sector funding, will include a state of the art 1800 seat concert hall, a 450 seat rehearsal and recital hall, and a cluster of conference and meeting halls, the largest of which will accommodate up to 750 delegates.

1.1 Three Halls, Three Buildings under one Shell

The project of the building is composed by three main elements (Figure 1):

- a Concert Hall;
- a Rehearsal and Recital Hall;
- a Conference and Convention Complex;

The three halls side by side will simplify the flow within the building. The assessment of utilization and feasibility unequivocally indicates that this would entail considerable advantage as to initial capital expenditure and operational costs if halls were linked.

Throughout the design process, emphasis is placed on making it both ideal for hosting bigger events and to separate different conference halls from other areas of the building. This makes it possible to host different events simultaneously, without one interfering with the others.

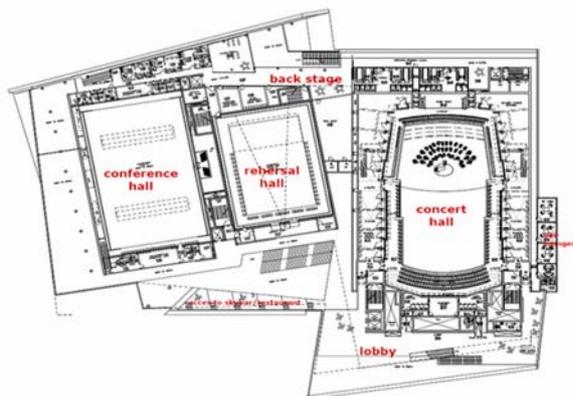


Fig. 1: Plan of the Building [1]

1.2 Location

As stated above, the location of the Iceland National Concert and Conference Centre is in the heart of the city of Reykjavik – East Harbour, adjacent to the commercial centre of the city and linking directly to the principal retail areas. It is situated in a way that emphasizes the visibility as much as possible (Figure 2).



Fig. 2: Location of the CCC inside of the city of Reykjavik [19]

2. EARTHQUAKE HAZARD MAPPING

Iceland is an active seismic area, in fact the most earthquake prone region in the Northwest Europe. Over the centuries, earthquakes have caused quite significant damage to the buildings and structures [13].

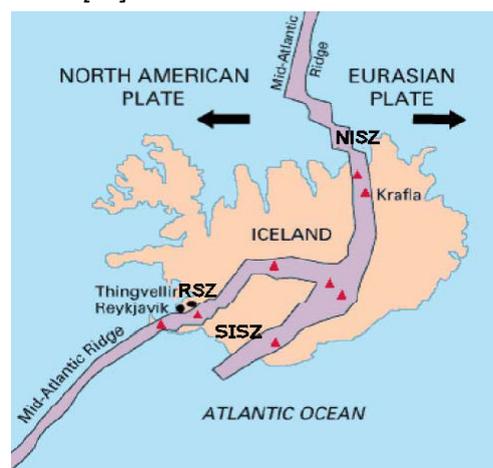


Fig. 3: Iceland lies on the Mid Atlantic Ridge [6]

Iceland is a super structural part of the Mid Atlantic Ridge, and is being split by the divergent plate boundary between the North American and the Euro-Asian Plate, creating a belt of seismic activity

from the Azores in the south, towards Jan Mayen in the north (Figure 3).

Across Iceland, from southwest to the north, the plate boundary is displaced to the east through two major fracture zones, the South Iceland Seismic Zone (SISZ), in the lowlands of the south, and the Tjörnes Fracture Zone (TFZ) in the north.

The City of Reykjavík and surrounding townships are located close to seismic delineation of the Reykjanes Peninsula (RSZ), where the Mid Atlantic ridge enters into the peninsula from the southwest. Moreover, the SISZ is within 30-50 km distance from the Capital.

The Reykjanes earthquake activity is more complex due to the interaction of volcanism and tectonic movements.

As the City of Reykjavik is built on a narrow point of land, protruding towards the northwest from the main Reykjanes peninsula, it is believed that the earthquake activity decreases gradually through the eastern towns and suburbs towards the western tip of the Seltjarnarnes Cape. This indicates that the area should be split into zones with different seismic design requirements.

2.1 A tentative Earthquake Catalogue

The earthquake catalogue of Iceland is not precise. Several historical events may have been missed altogether and the so-called historical gap in the 14th century, when writing of chronicles and annals seems to have waned due to natural catastrophes such as volcanic eruptions, diseases, etc. may have resulted in large events not being reported.

Just from the beginning of the 20th century that uniform retrieval and assessment of teleseismic data has been used as a basis for earthquake catalogues. Instrumental records of Icelandic earthquakes are only available since 1912.

The problem with basing the earthquake hazard on few historical earthquakes available is evident. The historical catalogue is both unreliable and does not take into account future events that may occur in different locations within the zone. Therefore, attempts have been made to simulate or generate artificial catalogues, which are based on the inter-arrival times of past events and known

geophysical characteristics of the seismic zone [16], [17]. Thus, the historical catalogue can be extended into the future in order to produce more consistent hazard maps and test the sensitivity of existing maps to different earthquake scenarios.

The event generation is produced stepwise by a simple Monte Carlo technique. Some information can be provided in [14], [15].

2.2 Attenuation of Earthquake Motion

With the earthquake catalogue for the Reykjavík area, the epicentral peak accelerations can be calculated. Also a suitable attenuation model for earthquake motions is required to assess the expected peak ground accelerations in and around Reykjavík.

2.3 Seismic Zoning of the Reykjavik Area

The Reykjavik area is defined as the geographical region within [64.05°N-64.25°N; 21.67°W 22.150°W], covering the most densely populated part of the capital region. The study area is subdivided into an 11x17 mesh with 187 grid points. At each grid point, the peak ground acceleration, a_{gR} , is calculated for a predefined subset of the earthquake catalogue that is the historical earthquake catalogue, extended by simulated future earthquake events [14], [15].

The peak ground acceleration series at each grid point are reordered in numerical order and the corresponding probabilities of occurrence calculated. Through this procedure, the extreme value distribution of peak ground accelerations, for each grid point within the study area, is obtained. From the distributions, the peak ground acceleration, corresponding to any chosen average return period, can be obtained by interpolation. Based on these values, iso-acceleration curves can be drawn. In Figure 4, the 500 year iso-acceleration curves for the study area are shown.

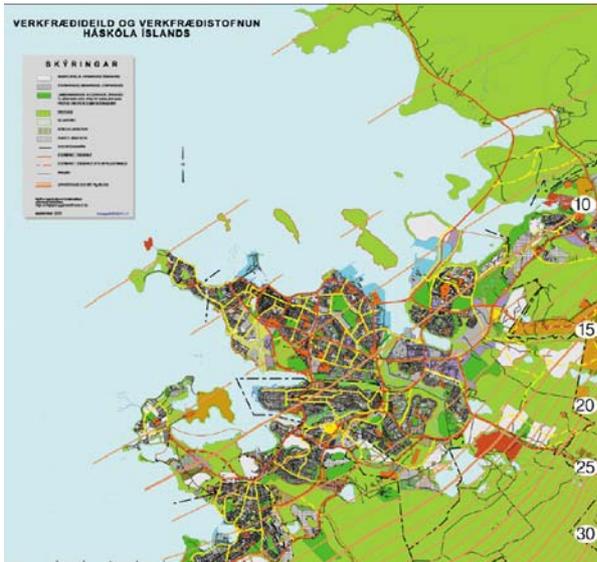


Fig. 4: 500 year iso-acceleration curves

According to Figure 4, the area where the CCC is located has less than 0.10g of peak ground acceleration. However, as identifiable in the Figure 4, the peak ground acceleration changes in a short distance. This fact can raise some doubts about the value that should be used for the seismic analyses. To prevent possible errors in the seismic analysis two values will be used: 0.10g (in linear range [9]) and 0.30g (in non linear range [8]). Another reason to use a higher peak ground acceleration in this study is related to the importance of the building and its return period.

3. DESCRIPTION AND MODELLING OF THE BUILDING

3.1 Description of the building

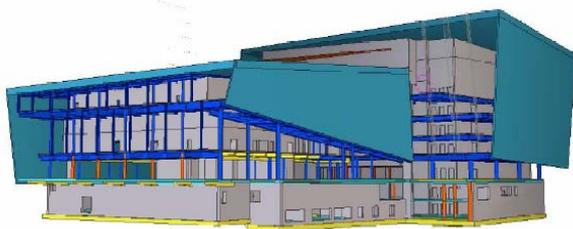


Fig. 5: Tekla's web viewer screenshot of CCC [18]

The Figure 5 shows a screenshot from Tekla web viewer [18] file of CCC. The Concert and Conference Centre building is separated, on the left with the Conference Hall, the Rehearsal Hall and 4th Hall (West Building) and on the right with the Concert Hall (East Building). This separation is

an acoustic joint due hall's functionality. Then, the analysis of the buildings is made independently as two totally separated structures. In this paper, just the west building (Conference Hall) is analyzed.

3.1.1 Foundations

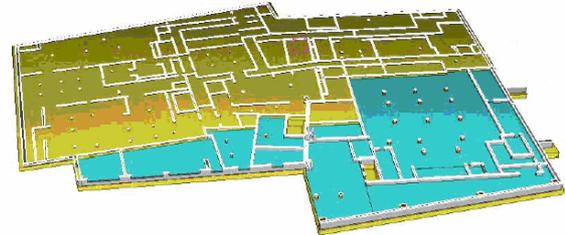


Fig. 6: CCC Bottom's Slab [18]

The dimension of the envelope of the bottom slab is approximately 106.2m x 83.6m. Even though the halls are analyzed separately, they are cast together without any separation (Figure 6).

3.1.2 Basement

The Figures 6 and 7 show the bottom level below the ground level.

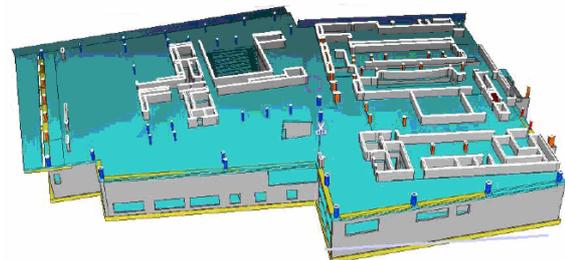


Fig. 7: The Basement [18]

The bottom slab (Figure 6) and walls together with the level 1 slab (Figure 7) form a rather stiff box structure equalizing the deflections to a rather uniform level, valid for the total building above. The bottom slab distributes bending moments and the vertical load transferred from the above walls and columns to the ground fill. Thus, the highest pressures are below the walls and columns, and the lowest inside the basement rooms.

3.1.3 Ground Level

The level 2 slab carries out the mission of interconnect the parts in the building and distribute horizontal forces (Figure 8). This slab transfers

horizontal forces to the staircase tower and other stabilizing walls from level 1 to level 2. Then, this level has more columns and much less walls than the level above (Figure 9).

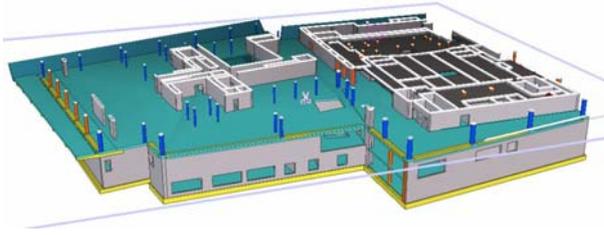


Fig. 8: View from Level 1 and all elements that support the Level 2 [18]

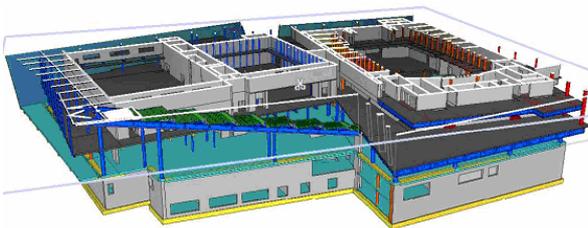


Fig. 9: Overview of the level 1, level 2 slab (grey) and the wall of the halls [18]

3.1.4 The Halls

The Concert Hall, the Rehearsal Hall, the Conference Hall and the smaller 4th Hall have “a box structure” (Figure 10). All have walls, bottom and top slabs, that make them able to take up horizontal and vertical loads and transfer these to their supports that permit being used also to make them stand without support along their entire wall centerlines [10].

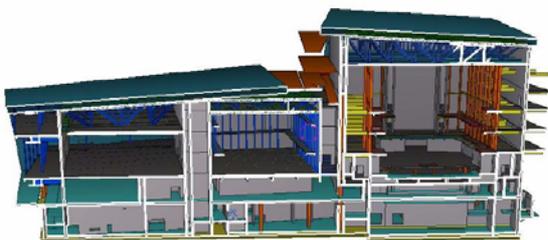


Fig. 10: Conference Hall on left, Rehearsal Hall on the middle and Concert Hall on the right [18]

3.2 Modelling of the Building

The first step to start the modelling was to make some simplifications of the 3D AutoCAD [2] drawings, in order to import a simple model from AutoCAD to SAP2000 [12].

3.2.1 From Tekla Viewer to AutoCAD

With Tekla Viewer, all the main levels were identified in order to build the grid in SAP2000 (Figure 11). It was done increasing the height and looking for all the changes in plan (new slabs, new walls and new steel elements). Were identified fourteen grid levels in the West building.

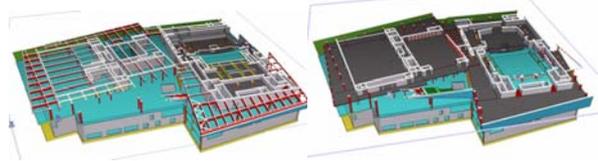


Fig. 11: Examples of two Tekla's grid levels [18]

3.2.2 From AutoCAD to SAP2000

In AutoCAD [2], two layers were created (points and frames) in order to import to SAP2000 lines and points separately from all the fourteen 2D drawings (SAP2000 only allow to import AutoCAD .dxf files this way).

Some levels are approximately at the same weight (one level could be the steel beam structure and the other level is the correspondent concrete slab which lies above the steel beams), then some levels were aggregated. In the final, there were just ten AutoCAD levels (Figure 12).

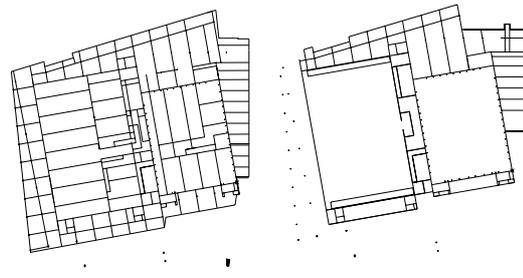


Fig. 12: Examples of two AutoCAD grid levels [18]

3.2.3 Assumptions done in AutoCAD

- For the bottom slabs (Basement) it was assumed an average height (z). There are some slabs that start at different height, but that difference is not so much relevant to be considered. This assumption was made in some levels in order to simplify the modelling, but without lose accuracy.
- The roof was considered horizontal, due its lower slope.

- The balconies were replaced by concentrated mass. It could be done because they do not contribute for the global stiffness, just for mass.

3.2.4 Assumptions done in SAP2000

- The bottom slab (1m thick) was not modelled. All the walls and columns above the slab were assigned with joint restraints (Figure 13).

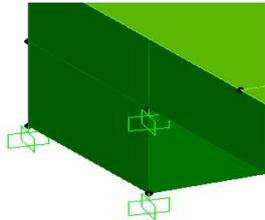


Fig. 13: Examples of AutoCAD [18]

- The West (Conference Building) and the East (Concert Building) buildings are separated due acoustic reasons. But there is one big wall (shown by blue colour in Figure 14) where there is a connection on its top with the East building. To simulate that connection were applied joint restraints (horizontal displacement perpendicular to the wall) to the top joints (shown by red colour in the same figure).

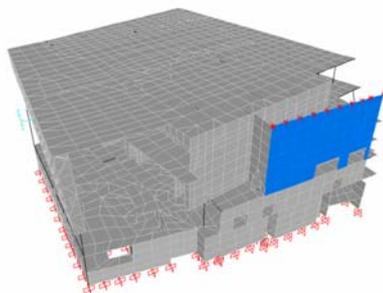


Fig. 14: SAP2000 screenshot showing the joint restraints (red) and the shared wall (blue) [18]

3.2.5 Roof

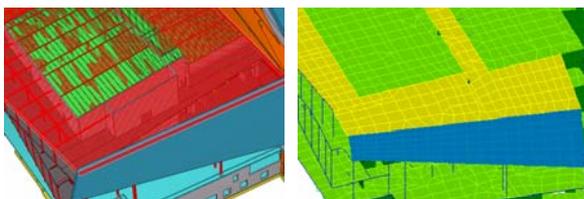


Fig. 15: Tekla viewer and SAP2000 screenshot of the modelling of the above roof (blue) [18]

The roof specifications were provided in a meeting with Eng. Martin Munck from *Rambøll*. The self-

weight provided is 1.5kN/m^2 and the roof stiffness is $EI = 50000\text{kNm}^2$.

3.2.6 Facades

It was decided to apply only the mass of the facades in their vertical support. In this paper is shown just one example of a facade of the West building. The North facade was chosen and is presented in Figures 16 and 17.

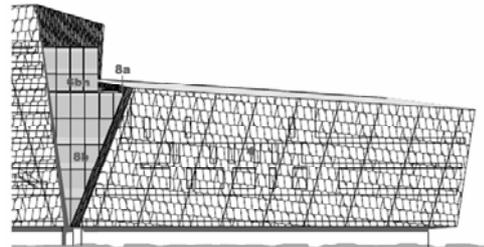


Fig. 16: Architectural elevation – north view [18]

- **Facade 4** – Vertically the facade is supported solely at level 1.

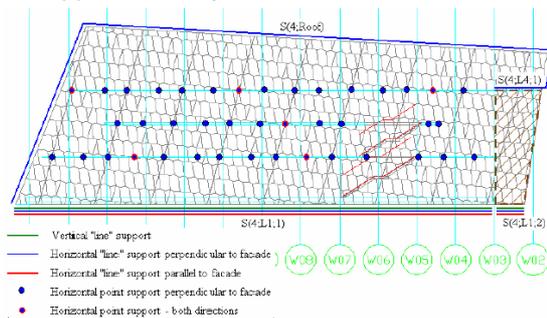


Fig. 17: Supports for facade 4 [11]

Table 1 – Vertical load on facade 4 support

Reference	S(4;L1;1)	S(A;L1;2A)	S(4;L1;2B)
Vertical Load (kN/m)	34	45	30

The Table 1 shows the vertical load applied on facade 4 support.

The only exception was the South Quasi-Brick facade, which was completely model with a shell element (Figure 18).

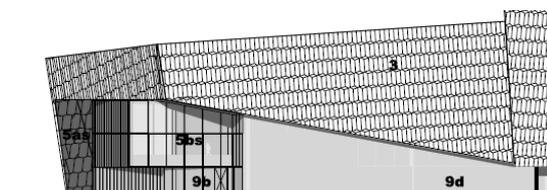


Fig. 18: Architectural elevation – South view [11]

• **Facade 3**

The South Quasi-Brick facade was modelled in SAP2000 (Figure 19) because part of the roof is supported by the Quasi-Brick facade, and this one is supported by the ramp. SAP2000 will calculate the correct distribution of loads.

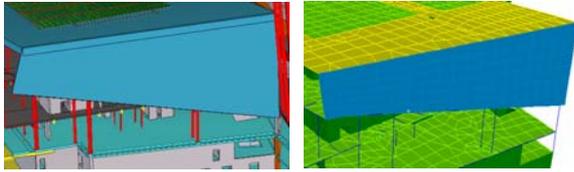


Fig. 19: Tekla viewer and SAP2000 screenshot of the modelling of the Quasi-Brick facade (blue) [18]

3.2.7 Loads

The distribution of the dead and imposed loads per floor was provided by Rambøll and used in the analysis. It can be consulted in detail in appendices A1 and A2 from [8].

The effects of the seismic action shall be evaluated by taking into account the presence of all gravity loads appearing in the following combination of actions:

$$\sum G_{kj} + \sum \psi_{Ei} \cdot Q_{ki} \quad (1)$$

where ψ_{Ei} is a combination coefficient for variable action i , G_{kj} is a characteristic value of permanent action and Q_{ki} is a characteristic value of variable action.

4. SEISMIC ANALYSIS

The irregularity of the main structural system of the building both in plan and in elevation means that the equivalent lateral force method is not applicable. Due to this irregularity, a spatial multi modal response spectrum analysis was performed.

The Earthquake load is calculated for this study according with the recent version of EC8 [3]. To perform a multi-modal response spectrum analysis a spatial model was built in a finite element program, SAP2000. Rambøll used LUSAS [1].

4.1 Modal Analysis

To evaluate the response of all modes of vibration with significant contribution to the global response, are taken into account either of the following conditions should be satisfied for each directions [3]:

- The effective modal masses for the modes considered shall at least be 90% of the total mass of the structure.
- All modes with effective modal masses greater than 5% of the total mass are considered

4.2 Assumptions

- The CCC is classified as a structure that belongs at class III. The importance factor is given by $\gamma_{III} = 1.2$ [3, p.53].
- The building is characterized by having a ground type A. The standard values of subsoil class A are: $S=1.1$, $T_B=0.15s$, $T_C=0.4s$, $T_D=2.0s$. A soil parameter $S=1.1$ – instead of 1.0 – is chosen to take into account of the minor areas with thicknesses of the gravel pad exceeding 5m on top of the bedrock.
- The reference peak ground acceleration, a_{gR} , is 0.10g, for a reference return period of 500 years. As the importance factor is $\gamma_I = 1.2$, the design ground acceleration, a_g , is 0.12g. This value will be used for the linear analysis. Anyway, higher peak ground acceleration (0.3g) will be used for the non linear analysis. This higher value is used because is not clear that the value given by the iso-acceleration curves (Figure 4) should be used. Other argument that supports this higher value is related with the importance of the building. A new level of reliability of the building can be used and a higher value of Importance factor would multiply the reference ground acceleration.
- The damping, ξ , is estimated as 3.5% of critical damping. This value being chosen as a value between on the one hand the standard value - 5% - according to EC8 [3], for a structure with large deformations and ductility capacities, in which structural damage would contribute greatly to the overall damping, and the value 2%, corresponding

to the structure remaining elastic during an earthquake.

- The upper limit value of the behaviour factor q , to account for energy dissipation capacity, shall be derived for each design direction as follows [3, pag.82]:

$$q = q_o k_w \geq 1.5$$

However, this value is reduced because of the large irregularities in the floor plans and elevations of the structure created by the irregular architecture of the building. The resulting behaviour factor is: $q=1.2$.

4.3 Multi-Modal Response Spectrum Analysis

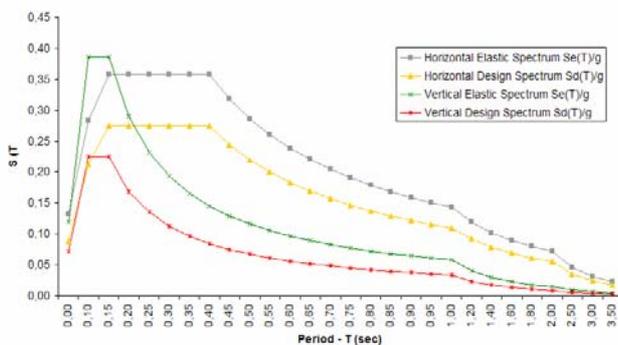


Fig. 20: Design and Elastic Acceleration Spectra

The horizontal elastic response spectrum, the design spectrum and the vertical response spectrum are taken as defined in EC8 [3].

4.4 Non Linear Time History Analysis

For a non linear time history analysis, ten synthetic accelerograms were created, based on the response spectrum.

When a synthetic accelerogram is created, the goal is that the response spectrum obtained from the synthetic accelerograms match as close as possible the defined target response spectrum (general the ones proposed in seismic codes). There are several ways to do this. An example methodology can be found in Clough and Penzien [4].

In this project, the generation of the synthetic accelerograms was done employing Gosca Software [5]. It uses routines developed in University of Bristol.

An example of one synthetic accelerograms with its matching response spectrum can be seen in Figures 21 and 22, respectively.

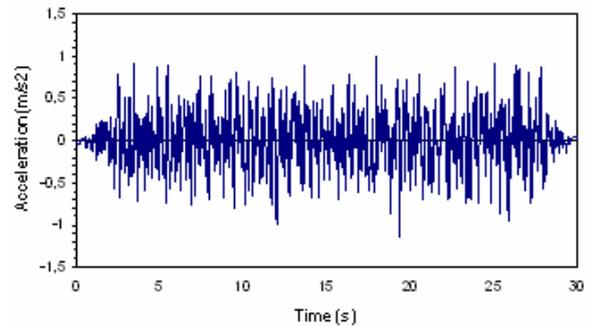


Fig. 21: Example of a Synthetic Accelerogram

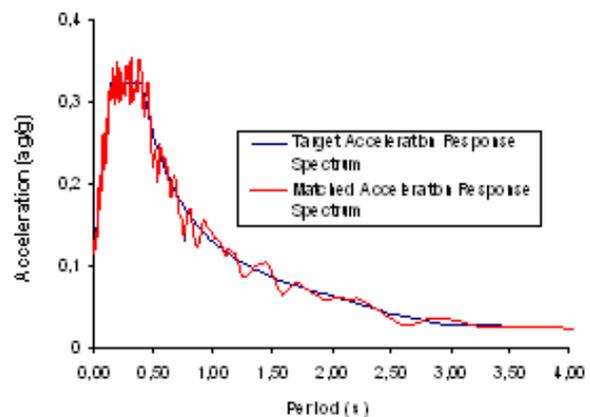


Fig. 22: Matched Vs. Target Acceleration Response

As is possible to see in Figure 22, the acceleration response spectrum defined from the synthetic generated accelerogram is very close to the target acceleration response spectrum.

The number of accelerograms used in the non linear time history analysis (ten) is higher than the number used in the linear time history analysis (three), because the sequence of peaks affects the non linear response of the structure. Moreover, the instant when the acceleration peak occurs is important for the response of the structure. In the non linear time history analysis is different if the acceleration peak occurs when the structure is in a linear range or if it is yet in a plastic range. Then, it is recommend to do the average of many accelerograms.

In a linear time history analysis it is indifferent the instant when the response peak occurs, because the response of the structure is always in the linear range.

5. NON LINEAR SEISMIC ANALYSIS RESULTS

In this thesis, when it is said Non Linear Analysis, the author means design response spectrum analysis and non linear time history analysis. In fact, as it was mentioned above, the response spectrum analysis is a linear analysis, and the non linearity is given by the behaviour factor q .

5.1 Reinforcement

In order to perform the non linear time history analysis, the most stressed elements were identified in [9]: three small walls in the first level and the column which supports the quasi-brick facade and the ramp. These elements are the critical locations of the linear analysis.

As it was referred, for the non linear time history analysis and for the design response spectrum analysis, a higher seismic acceleration is used, 0.3g.

5.2 Multi-Linear Force-Deformation definition

For the modelling of the structure, linear elastic beam and shell elements were used in all locations, except for those in which plastic hinge formation were expected. These zones were modelled using shell elements and as it is known, SAP2000 does not permit plastic hinges in this type of elements. Then, a simplified shell/frame model was created in order to overcome this software limitation.

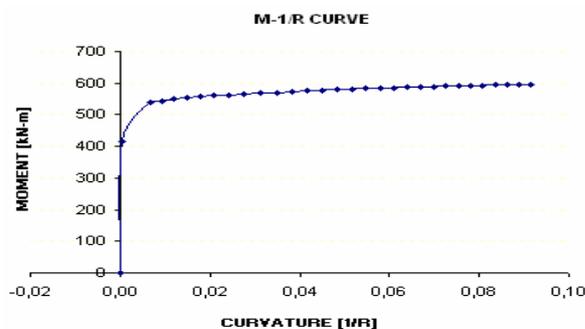


Fig. 23: Graphic of the relation M-1/R for the study section

The multi-linear force-deformation (Figure 23), in both horizontal directions, was defined to get a Bi-Linear Hysteretic Plastic Hinge Model (Figure 24). Specific software developed in Department of Civil Engineering and Architecture of *Instituto Superior Técnico of Lisbon* was used.

5.3 Bi-Linear Hysteretic Plastic Hinges Model

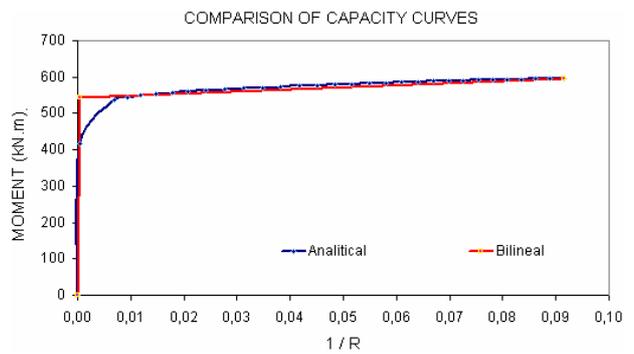


Fig. 24: Graphic of the bi-linear hysteretic plastic hinge model

Bi-Linear Hysteretic Plastic Hinge Model considers that the initial slope and stiffness of the analytical capacity curve is near from the slope of the idealized curve, stiffness after yielding is non zero, and uses the equality energy principle.

5.4 Plastic Hinges in SAP2000

The plastic hinges were simulated by Links, they can simulate their hysteretic behaviour.

Link element can only be applied on linear elements. In order to overcome this limitation, it was decided to replace the most forced shell elements by rigid frames and apply on it the plastic hinges (Figure 25).

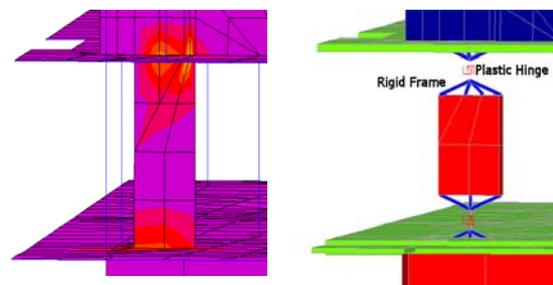


Fig. 25: Shell element before plastic hinges (left) and after plastic hinges (right)

5.5 Non Linear Analyses

In order to confront the results from a design response spectrum analysis and from a non linear time history analysis, displacements and rotations were extracted from various locations of the building. To organize the results, four main alignments are defined (Figure 26).

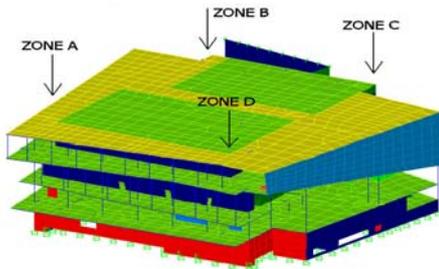


Fig. 26: Four main alignments from the building

The design response spectrum analysis and the non linear time history analysis were performed on X, Y and Z direction, separately.

In this paper, is just given the example of displacements, rotations and inter floor drifts in Zone A, caused by an X direction motion. (Figures 27 to 29). The complete list of results is presented in detail in [8].

- DISPLACEMENT IN ELEVATION

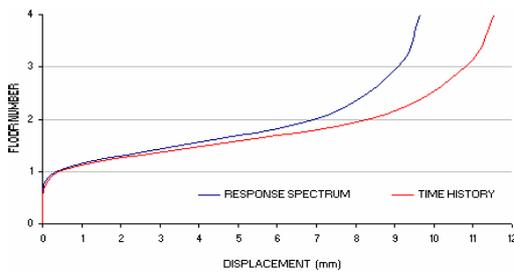


Fig. 27: X direction displacement caused by an X direction motion – Zone A

- ROTATION IN ELEVATION

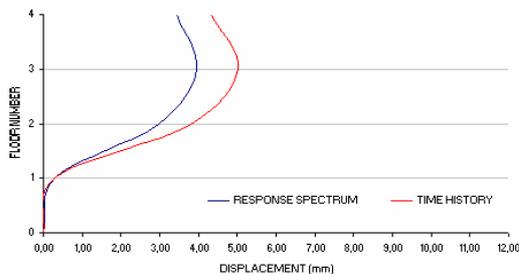


Fig. 28: X direction rotation caused by an X direction motion – Zone A

- DRIFT [%] IN ELEVATION

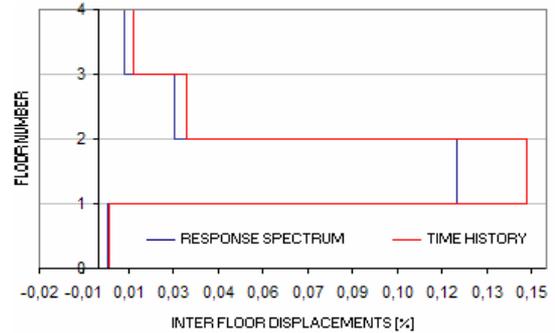


Fig. 29: X direction drift caused by an X direction motion – Zone A

For each kind of bending moments (M_{11} and M_{22} in shells and M_2 and M_3 in frames) and shear forces (V_{13} and V_{23} in shells and V_2 and V_3 in frames), caused by a motion on X, Y and Z direction, is just shown in the critical areas where the Shell/Frame bending moments and shear forces are high. As the force maps are different, different numbering were chosen.

Some zones of the structure have high forces due structural irregularities [9]. The Figure 30 shows the zones of the building where the bending moment M_{11} , for a motion on X direction, is higher. In Table 2, the correspondents bending moment values are presented. The complete list of results is presented in detail in [8].

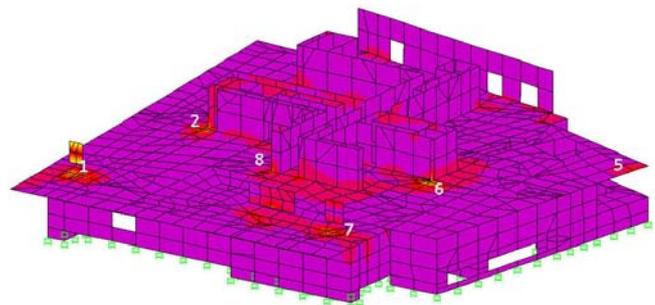


Fig. 30: Most forced zones in the structure, 1st level (forces for Motion on X direction, M_{11} moment)

In the table, DRS means Design Response Spectrum Analysis and NLTH means Non Linear Time History Analysis.

Table 2 – M₁₁ caused by a motion on X direction in zone 1 to 8

Point	DRS (kNm/m)	NLTH average value (kNm/m)
1	134.25	168.89
2	117.46	147.36
3	158.47	191.78
4	215.72	268.51
5	42.39	68.91
6	137.35	146.73
7	126.56	157.65
8	98.73	126.24

6. CONCLUSIONS

The use of the non linear dynamic methodology has helped to quantify the inelastic response of the structure and may be used as a powerful means for design assessment.

Comparing the two non linear analyses is easily checked that the response spectrum analysis always gives results below the time history analysis.

For displacements and rotations, those differences are more evident for the first level, the place where the plastic hinges were included.

For bending moments and shear forces, the difference between the two analyses is evident.

Non linear time history analysis values are 10-30% higher than design response spectrum analysis values, for all the structure and for all types of elements (shells and frames).

Based on these results, is possible to conclude that the response spectrum analysis (performed by Rambøll for 0.1g) is not conservative for the seismic intensity considered in this chapter. Then, a non linear time history analysis is recommended in this structure for this level of action. In fact, the irregularities of the structure may be responsible for the difference found in both analyses.

This conclusion is important because the design response spectrum analysis may not always lead to a conservative solution for an irregular structure, and it is beholden on the design to ascertain the magnitude of such load effects.

Nevertheless, it is important to emphasize that the response spectrum technique is only an

approximate method in the case of a multi degree of freedom system. In the case of a complex three dimensional structure with many closely spaced modes, such as the Conference Hall building, great caution should be done in applying the response spectrum analysis. Then, the results of the non linear time history analysis present in this thesis are considered much more reliable than the results of the response spectrum analysis.

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