

# Chapter 7

## Analysis of the complete structural system

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The last task in this thesis work, and the final purpose of it, is the seismic analysis of a whole structure in 3D. The main points of interest are:

- Modelling of the 3D structure, with special consideration for the roof.
- Number and dimension of the bracing elements that are needed to keep the structure in the elastic behaviour.
- Study the possibility of reducing the complex 3D model into a simpler 2D one.

### 7.1 Modelling of the whole building

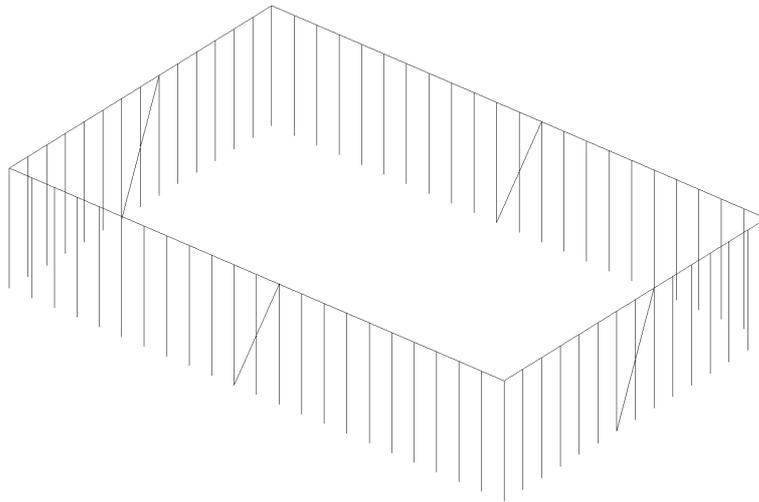
#### 7.1.1 First test: four façades

First of all, the shape and dimensions of the building on the ground plan have to be defined. The shape will be rectangular, which is the easiest to model and design, since it is meaningless to work with a complex structure if the response of the simplest is not known. Furthermore, the rectangular shape is the most recommended with regard to the seismic design of structures.

The dimensions are chosen to have a realistic value of the inner surface for a single-family house. The long façade will consist in 11 modules or frames, whereas the short one will have 7 modules. Therefore, the ground plan will have dimensions 13.2 x 8.4 m. and a surface of 110.88 m<sup>2</sup>. The height of the building has not been modified and it is equal to 2.75 m. The first assumption on the x-bracing diagonals is to place them in the middle of each façade. The final configuration of the building is shown in figure 7.1 and the dimensions are summarized in table 7.1

|                     |                       |
|---------------------|-----------------------|
| Length long façade  | 13.2 m./ 11 modules   |
| Length short façade | 8.4 m. / 7 modules    |
| Area                | 110.88 m <sup>2</sup> |
| Height              | 2.75 m.               |

**Table 7. 1** Dimensions of the 3D structure



Cast3m2001 Education Recherche : Structure 3D

**Figure 7. 1** Drawing of the 3D structure consisting in four façades

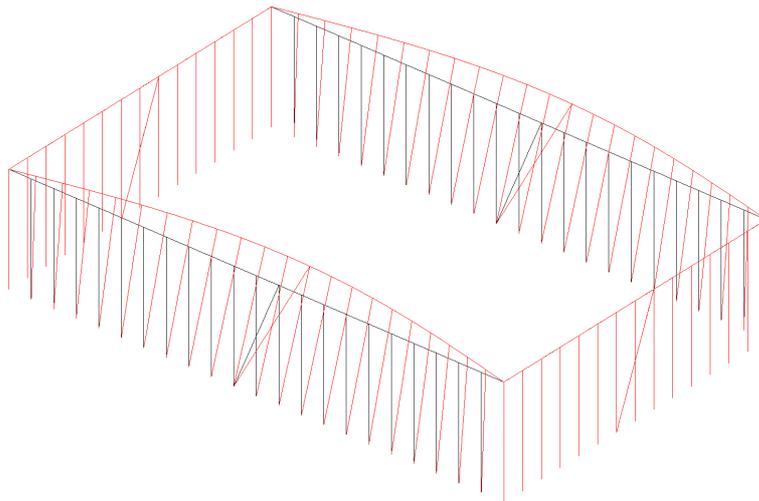
Like in the case of the façade, all joints are hinged, except the union between elements of the threshold, which are fixed. The result is a continuous beam, along each façade, that is simply supported on the columns.

In order to have the simplest geometry as possible, the addition of a roof is avoided. Therefore, it has to be substituted in some way, and it will be done like in the 2D analyses, by the addition of mass on the beam elements of the threshold, as in section 5.3. But this new mass is not loaded to all the boundary of the roof, just to the long façade. The reason is that the roof is supposed to rest on beams that are supported at their ends by exclusively the long façade. None of the other material and geometrical properties of the beams have been changed, so they can be looked up at tables 5.1 and 5.2.

The model of the structure is completely defined now, so the eigenvalues calculation is carried out to find the eigenmodes of vibration, but the obtained results are not satisfactory.

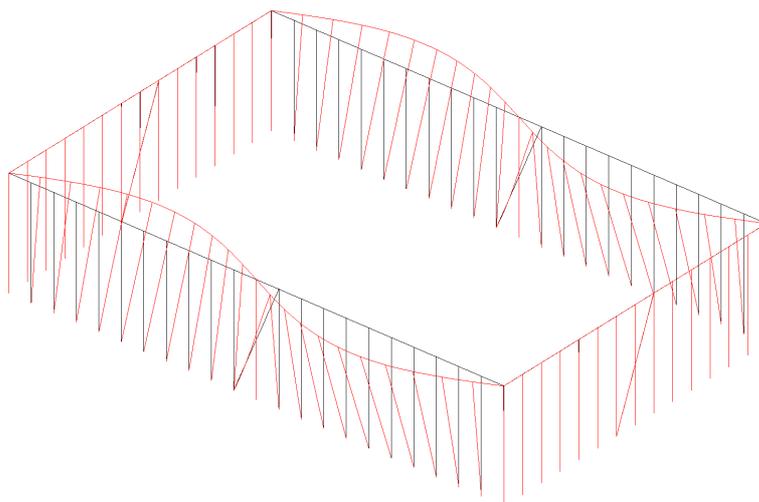
Figures 7.2 and 7.3 show the two lowest eigenmodes, in which an unexpected deformation of the threshold in the perpendicular direction to the façade is clearly observed. This behaviour is not a realistic response of a single-storey building, since the presence of the roof in the real construction would make the continuous beam much

stiffer in the perpendicular direction. Another consequence of this non-existing stiffness is that the lowest eigenfrequencies have values greatly differing from the ones obtained in the 2D façade, which are more similar to the usual values for houses and common edifices. As a general rule, it can be set that the first eigenfrequency of a building, in Hz, is  $10/n$ , where  $n$  is the number of storeys. Thus, for a single storey, and taking into account that the light-weight structure has less stiffness, the frequency should be less than 10 Hz, but not as little as  $5.48 \cdot 10^{-2}$  Hz. Furthermore, this approximation justifies the values obtained for the 2D façade, which were of the order of 3 Hz.



Cast3m2001 Education Recherche : Vibration mode corresponding to frequency 5.48333E-02 Hz

**Figure 7. 2** First eigenmode for the 3D structure consisting in four façades,  $f= 5.48333 \cdot 10^{-2}$  Hz



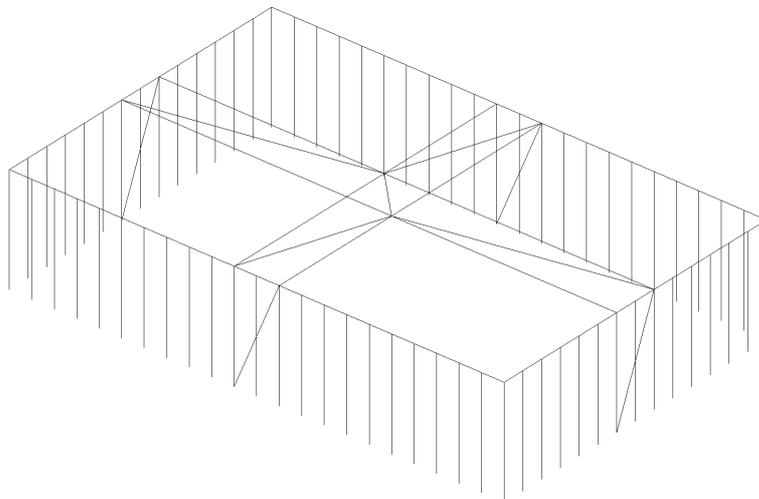
Cast3m2001 Education Recherche : Vibration mode corresponding to frequency 0.21908 Hz

**Figure 7. 3** Second eigenmode for the 3D structure consisting in four façades,  $f= 0.21908$  Hz

In a word, this model of the building is not a good one and some arrangements must be made.

### **7.1.2 Addition of stiffeners**

The second attempt in modelling the structure was done adding two horizontal trusses to stiffen the thresholds, each one consisting in two continuous beams and some diagonals. The trusses were placed connecting the braced frames of opposite façades and they were also joined together, sharing the diagonal at the centre of the roof. This geometry is seen in figure 7.4. Different suppositions were made on the values of the area and inertia of these beams. The diagonals were designed as straps of 65, 100 or 150 mm. width, whereas the first choice on the other profiles was to use the same beams as in the façade. Later, the area and inertia of these profiles was increased until ten times the original value, but the eigenmodes of vibration of the structure did not suffer any change. And this obtained response, in figures 7.5 and 7.6 for the two lowest eigenmodes, is again not representative of the real effect of the roof. It can be seen that only the central part of the threshold is stiffened, therefore the too large horizontal deformation has not been solved all along the beam. It is remarkable that there has been a big change in the lowest eigenfrequency, but the actual value, 0.35569 Hz, is still one order of magnitude lower than expected.

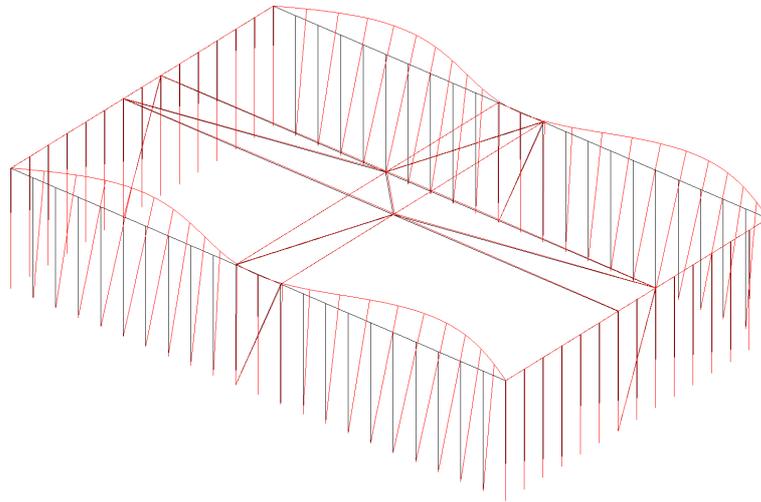


Cast3m2001 Education Recherche : Structure 3D

**Figure 7. 4** Drawing of the 3D structure consisting in four façades and roof trusses

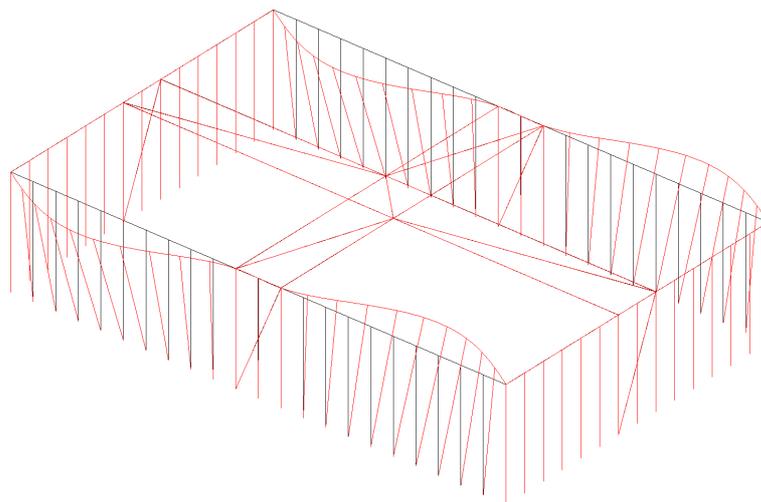
One last thing has to be said about the mass distribution in this structural model. When increasing the stiffness of the thresholds with the introduction of the trusses, the lowest eigenmodes were not the flexion of them anymore. They were the out-of-plane flexion of the diagonals on the façade, and the rest of the structure being motionless. Since this deformation may be avoided by the façade elements in the real building, we also do not want to take them into account in the mode superposition analysis. The

easiest way to do it is setting the density to zero, so no inertia forces will act on them. After comparing examples in two and three dimensions, it was seen that not considering the mass in the structural members did not lead to significant differences in the result, therefore, the only structural members with mass are those bearing the roof, the long threshold. This assumption will be used in all the analyses from now on.



Cast3m2001 Education Recherche : Vibration mode corresponding to frequency 0.35569 Hz

**Figure 7. 5** First eigenmode for the 3D structure with stiffeners,  $f= 0.35569$  Hz



Cast3m2001 Education Recherche : Vibration mode corresponding to frequency 0.39136 Hz

**Figure 7. 6** Second eigenmode for the 3D structure with stiffeners,  $f= 0.39136$  Hz

### 7.1.3 Modelling of the roof as a shell

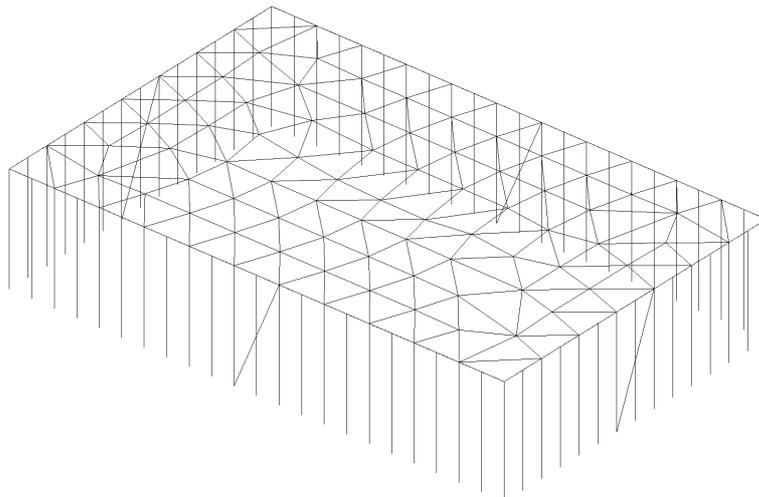
It has been discussed in the previous two sections some alternatives to not include the roof in the analysis, but they were not successful. Thus, the roof has to be modelled in some way, and it is simplified as a uniform shell element. Then, the complete structural system is formed by:

- Four façades made of beam elements, including the columns, thresholds and diagonals, all of them with zero-mass, to avoid the mentioned problems at the end of section 7.1.2. All the other features are like in the structural system in section 7.1.1. This is the resisting part of the structure.
- A horizontal shell element as the roof, introducing the mass (i.e. inertia forces) and lateral stiffness for the threshold. It is supposed to be made of concrete, and the required properties for the elastic analysis are summarized in table 7.2. Concerning the boundary conditions, it is simply supported along its four sides, implying that not only the long façade bears the roof, but also the short ones do. It might have some influence when trying to simplify the 3D analysis into a 2D.

|                       |                                 |
|-----------------------|---------------------------------|
| Young's modulus       | $E = 3 \cdot 10^{10}$ Pa        |
| Density               | $\rho = 2300$ kg/m <sup>3</sup> |
| Poisson's coefficient | $\nu = 0.25$                    |
| Dimensions            | 13.2 m x 8.4 m                  |
| Thickness             | $h = 0.06$ m                    |
| Weight                | 138 kg/m <sup>2</sup>           |

**Table 7. 2** Material and geometrical properties of the roof

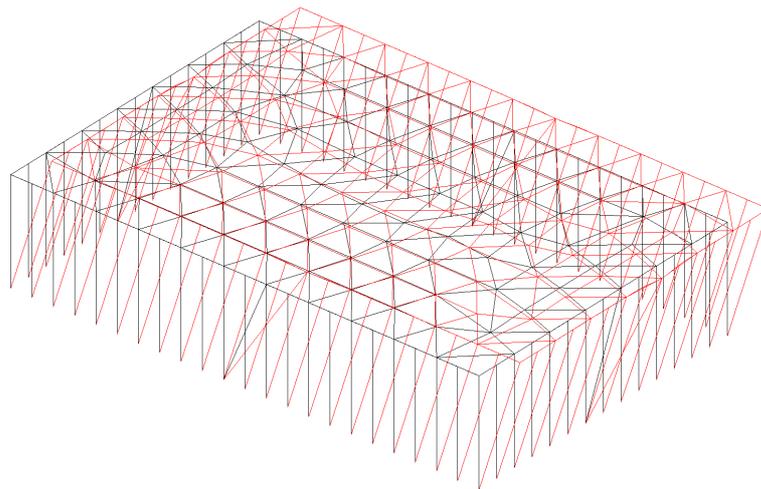
The final configuration of the structure and the roof's finite element mesh are drawn in figure 7.7.



**Figure 7. 7** Drawing of the 3D structure including the roof

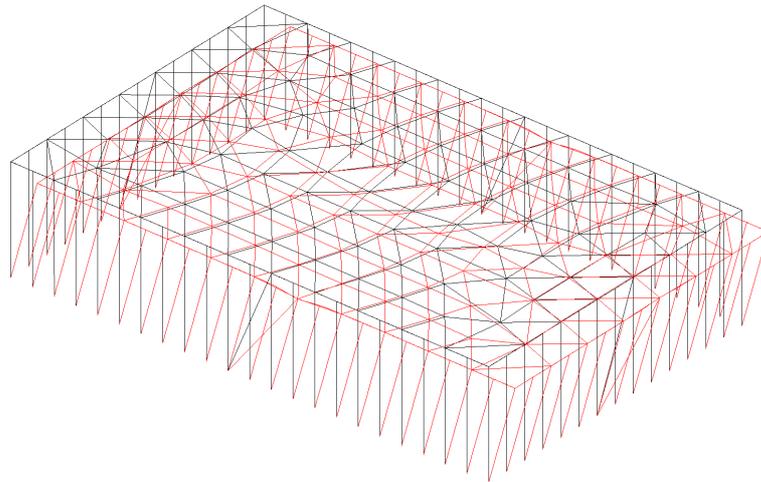
Starting with a diagonal width of 65 mm., we will check now that the dynamic behaviour of the structure is as expected, similar to a one degree of freedom system, like in all previous examples. An also that the eigenfrequencies have closer values to the usual for single-storey buildings.

The ten first vibration modes have been calculated, but not all of them are of interest. Most of the eigenmodes correspond to the flexion of the shell in the z-direction, and no deformation on the façades, therefore, they do not affect the structural response when applying horizontal ground motions. As a result, from the group of ten eigenmodes just three are considered, associated with the first, second and fourth eigenfrequencies. Modes 1 and 2, in figures 7.8 and 7.9, are the horizontal displacement of the roof in a direction non-coincident with the coordinate axes x and y, in which the façades are oriented, and the frequencies are very similar (1.3878 and 1.3881 Hz). These are the principal directions of the structure. The third mode, figure 7.10, is the rotation of the roof around the z-axis passing through its centre point. If the whole structure is assimilated to a cantilever with a punctual mass at the free edge, these three modes would correspond to the flexural and torsional modes of the beam.



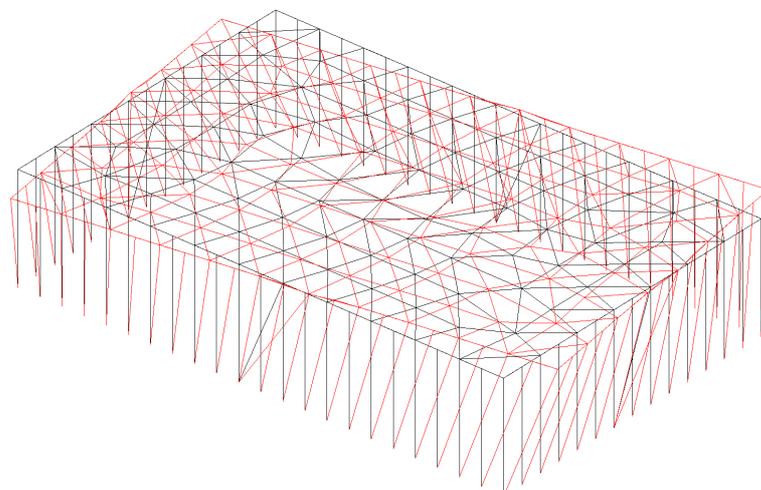
Cast3m2001 Education Recherche : Vibration mode corresponding to frequency 1.3878 Hz

**Figure 7. 8** First eigenmode for the 3D structure with the roof and diagonals of 150 mm width,  
 $f = 1.3878$  Hz



Cast3m2001 Education Recherche : Vibration mode corresponding to frequency 1.3881 Hz

**Figure 7. 9** Second eigenmode for the 3D structure with the roof and diagonals of 150 mm width,  
 $f = 1.3881$  Hz



Cast3m2001 Education Recherche : Vibration mode corresponding to frequency 2.4040 Hz

**Figure 7. 10** Fourth eigenmode for the 3D structure with the roof and diagonals of 150 mm width,  
 $f = 2.4040$  Hz

The calculation of the eigenvalues has been repeated for 100 and 150 mm. width of the diagonal, obtaining the same vibration modes with the related frequencies in table 7.3. This is done to show that the stiffness of the structure is controlled by the bracing diagonals, as desired.

| Diagonal Width<br>(mm) | Eigenfrequency (Hz)          |        |                 |
|------------------------|------------------------------|--------|-----------------|
|                        | Horizontal Displacement Mode |        | Rotational Mode |
| 65                     | 1.3878                       | 1.3881 | 2.4040          |
| 100                    | 1.6350                       | 1.6357 | 2.8323          |
| 150                    | 1.8728                       | 1.8768 | 3.2481          |

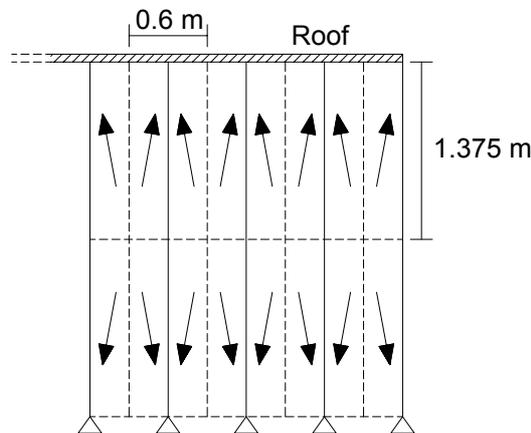
**Table 7. 3** Eigenfrequencies for the structure with roof and different width of the diagonal straps

The aim of this section has been achieved, a representative 3D model of the building have been created. Thus, we can now continue with the design of the bracing elements.

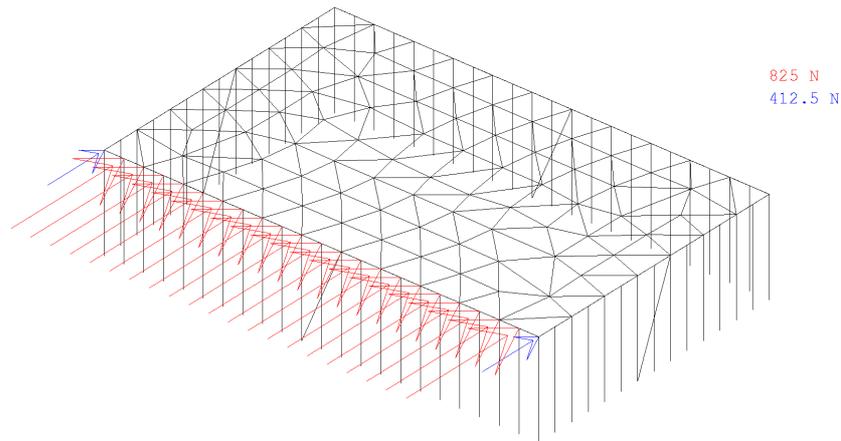
## 7.2 Preliminary design: wind load

Before starting with the design of the diagonals for the seismic load, a first guess on their dimension and number is made by means of a static analysis. A uniform wind load of  $1 \text{ kN/m}^2$  is applied perpendicular to the longest façade and the horizontal displacement of the roof must be lower than  $H/300$ , where  $H$  is the height of the building and equal to  $2.75 \text{ m}$ . This restriction is set by Eurocode 3 for single-storey buildings in the serviceability limit state (SLS). Besides, it is not expected in the SLS to have plastic behaviour of the structural members, thus, the tension in the diagonals is limited to the elastic limit of the steel,  $250 \text{ MPa}$ .

Since the deflection of the columns does not need to be checked and they are hinged at both ends, it is possible to substitute the uniformly distributed wind load for punctual loads acting on both ends of the column. It has to be noted that the forces acting on the base will be directly balanced by the horizontal reaction and will not cause any effect on the displacement field, therefore, these forces are ignored in the analysis. Each of the punctual forces has a value of  $825 \text{ N}$ , except those two placed on the corners, which are half of it,  $412.5 \text{ N}$ . The tributary areas used for the calculation of the forces can be found in the scheme in figure 7.11, and the position and orientation of the forces on the structure in figure 7.12



**Figure 7. 11** Tributary areas for the uniform wind load



Cast3m2001 Education Recherche : Punctual Wind Loads

**Figure 7. 12** Punctual wind loads acting on the structure

Only two attempts were needed to fulfil the requirements on the structural response. These results are included in table 7.4, where it is proved that using one diagonal of 150 mm. width in each façade is enough, and no other frame has to be braced.

| Width (mm) | Displacement (mm) | Limit H/300 (mm) | Tension (MPa) | Elastic Limit (MPa) |        |
|------------|-------------------|------------------|---------------|---------------------|--------|
| 100        | 11.2              | 9.2              | 227           | 250                 | Not OK |
| 150        | 8.5               | 9.2              | 151           | 250                 | OK     |

**Table 7. 4** Results from the static analysis acting a wind load

### 7.3 Design of the diagonals

When acting ground motions on the foundations of the structure it will be designed with regard to safety reasons and resistance of the structural members, i.e. ultimate limit state (ULS). In this particular case no limit values from building codes are used, since the dynamic analysis is linear, the requirement on the stress field is that the elastic limit of the steel, 250 MPa, cannot be exceeded. No limit is set for the displacements, because if the whole structure behaves in the elastic range it will go back to its initial configuration when the load stops, without causing any permanent structural effects. Anyway, displacements are expected of the order of centimetres or millimetres.

Another consideration before starting with the calculations is that, as the task is the design of some part of the structure, only the response spectrum from the Eurocode will be applied, figure A.4, with a peak ground acceleration of 4.8 m/s<sup>2</sup>.

A well-designed structure should equally resist ground motions from all possible directions, as well as simultaneous perpendicular excitations. First, that would yield to a deeper analysis and identification of the principal directions for each structure, which might be easy in the case of a rectangular building as we can see in figures 7.8 and 7.9 where the principal directions are obtained, and then apply the ground motions in this direction. Second, it supposes the addition of combination rules for seismic excitations in perpendicular directions, such as the 100/30 rule, the SRSS or the CQC3. More information on this topics can be found in the discussions from Computers & Structures, INC.

In this thesis, the aspects concerning the principal directions will not be taken into account, so the load will be applied in the x-y coordinate axes, which are also the orientation of the façades. To start with, the ground motions will be applied just in the direction of the long façade. Then, when a suitable design of the diagonals is achieved, the structure will be checked against two seismic excitations of the same magnitude in perpendicular directions, according to Eurocode 8. The combination rule for this case will be the SRSS, which is the same as the already discussed for the modal combination in section 3.5, but performing the summation in equation (3.24) over the directions instead of the eigenmodes.

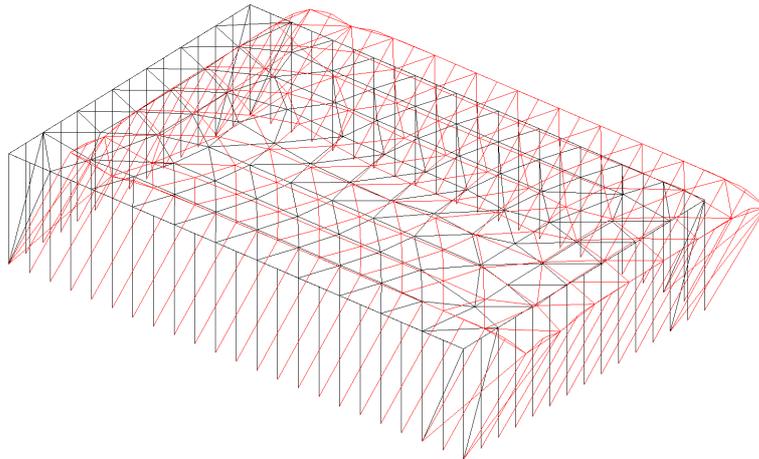
In the previous section, it was derived that at least one diagonal of 150 mm. width has to be in each façade. Therefore, this will be the first guess in the seismic design of these elements. But, as it is seen in the results summarized in table 7.5, it is not enough and façades with 2 or more braced frames have to be analysed. When designing with 2 x-braced frames they will be the ones at the ends of the façade, for 3 another one is added at the mid-frame and for 4 they are also as closest to the end as possible. As the reader might remember, it was proved in section 6.2 that the position of the braced frame or existence of openings in the façade do not cause any significant variation of the structural response. Consequently, the discussion below will be just focused on the number of diagonals and braced frames.

| Braced Frames per Façade | Crosses per Frame | Width (mm) | Total Area per Façade (mm <sup>2</sup> ) | Eigen-frequency (Hz) | Maximum Displacement in x-direction (mm) | Maximum Displacement in y-direction (mm) | Maximum Tension (MPa) |
|--------------------------|-------------------|------------|--|----------------------|--|--|-----------------------|
| 1                        | 1                 | 150        | 150                                      | 1.8728               | 64.4                                     | 0.8                                      | 1140                  |
| 2                        | 1                 | 65         | 130                                      | 1.8868               | 60.5                                     | 24.1                                     | 1353                  |
| 2                        | 1                 | 150        | 300                                      | 2.4934               | 40.6                                     | 24.7                                     | 720                   |
| 3                        | 1                 | 100        | 300                                      | 2.7404               | 37.4                                     | 13.9                                     | 755                   |
| 3                        | 1                 | 150        | 450                                      | 3.1204               | 27.8                                     | 12.9                                     | 493                   |
| 2                        | 2                 | 150        | 600                                      | 2.9560               | 26.9                                     | 20.1                                     | 348                   |
| 3                        | 2                 | 150        | 900                                      | 3.7364               | 18.1                                     | 11.4                                     | 235                   |
| 4                        | 2                 | 150        | 1200                                     | 4.3018               | 14.3                                     | 7.5                                      | 188                   |

**Table 7. 5** Structural response acting the Eurocode load in x-direction

Different comments have to be done about the structural responses obtained for the 3D building. Firstly, when more than one frame is braced, the ten first eigenmodes differ from the ones obtained with one diagonal per façade. The change consists in

having only one eigenmode with horizontal displacement of the roof, figure 7.13, instead of 2 principal directions on the x-y plane, figures 7.8 and 7.9. As shown in figure 7.13, the direction of vibration of this mode is not close to any of the façade directions, like they were in the case of one diagonal and two eigenmodes. On the other hand, the rotational eigenmode just differs in the related frequency, and the seven eigenmodes corresponding to a flexural vibration of the roof do not suffer any change, as they are not influenced by the diagonals. To complete the ten calculated eigenvalues a new mode appears, consisting in a uniform vertical displacement of the roof, always with an associated frequency a bit higher than the one for the horizontal vibration.



Cast3m2001 Education Recherche : Vibration mode corresponding to frequency 2.4934 Hz

**Figure 7. 13** Horizontal eigenmode of vibration for the structure with two braced frames per façade,  $f= 2.4934$  Hz

It is observed that the largest displacements occur in the same direction as the ground motions act, as expected. But, it is remarkable that, for buildings with more than one diagonal, also the displacements in the perpendicular direction are of the same order of magnitude than in x-direction, in some cases more than 50% of them. This is because the previously mentioned eigenmode of vibration, shown in figure 7.13, which is not parallel to any of the façade directions. Since we are used to work with static loads, this behaviour of the structure was not expected, but it will be proved in section 7.4 that the same structural response is obtained by means of a very different analysis, a direct time integration scheme. Therefore, it is possible to assure that the results can be trusted.

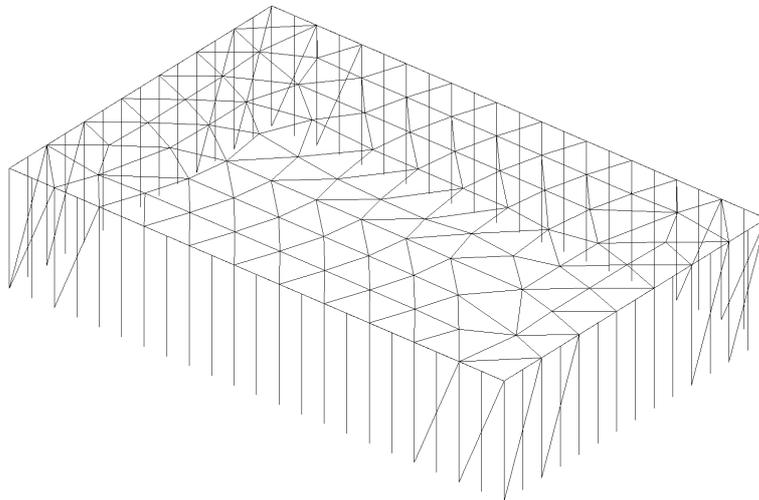
Concerning the stiffness of the structure, i.e. the value of the eigenfrequency, it is more influenced by the number of braced frames than the quantity of diagonals in each frame. For an equal amount of steel area of the straps it is stiffer to place them in many frames, and for the Eurocode seismic load, it will for sure lead to smaller displacements. But the tensional response is not improved.  $S_a$  is constant, so the total horizontal inertia force to be resisted will be the same, but its distribution to the diagonal members will not,  $A_i$  in equation (3.22) changes. Thus, the tension can increase for some members and decrease in the others. This discussion is clearly exemplified by the results in rows 1 to 4 in table 7.5.

After several attempts a configuration that fulfils the tensional requirement was found, consisting in three x-braced frames per façade and four diagonals of 150 mm. width in each one, two in tension and the other two not working. But this limitation was again exceeded when applying the same ground motions simultaneously in the x and y-directions, following the instructions from Eurocode, which is maybe too conservative. This new result is shown in table 7.6. As a consequence, a new frame had to be braced in order to have the maximum tension lower than 250 MPa, the elastic limit. The final configuration, plotted in figure 7.14, has two important consequences for the design of the whole building:

- In the short façade only three frames will be free to have an opening.
- Each braced frame will have two crosses, one joined on the inner part of the column and the other outside, as shown in the scheme in figure 7.15. Actually, it is the best way to do it, because the axial load from the strap will not be eccentrically applied on the column, so it will not create any torsional moment.

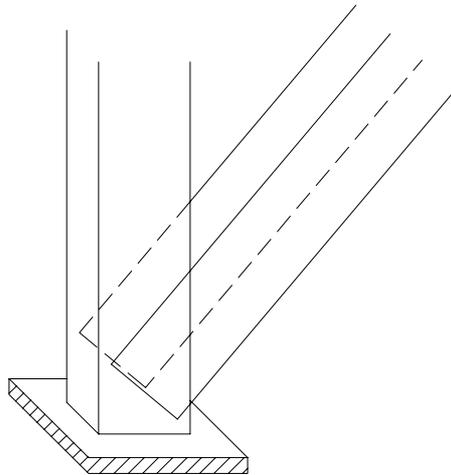
| Braced Frames per Façade | Crosses per Frame | Width (mm) | Total Area per Façade (mm <sup>2</sup> ) | Eigen-frequency (Hz) | Maximum Displacement in x-direction (mm) | Maximum Displacement in y-direction (mm) | Maximum Tension (MPa) |
|--------------------------|-------------------|------------|--|----------------------|--|--|-----------------------|
| 3                        | 2                 | 150        | 900                                      | 3.7364               | 21.3                                     | 21.9                                     | 284                   |
| 4                        | 2                 | 150        | 1200                                     | 4.3018               | 16.2                                     | 16.3                                     | 228                   |

**Table 7. 6** Structural response acting the Eurocode load in x and y-direction simultaneously



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**Figure 7. 14** Drawing of the 3D structure with 4 diagonals per façade

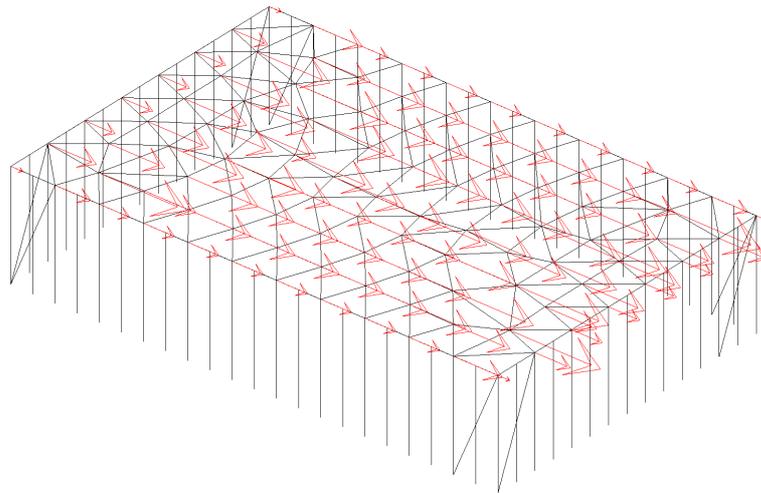


**Figure 7. 15** Scheme of the joint of two diagonals to the column

## **7.4 Checking of the behaviour with a step-by-step method**

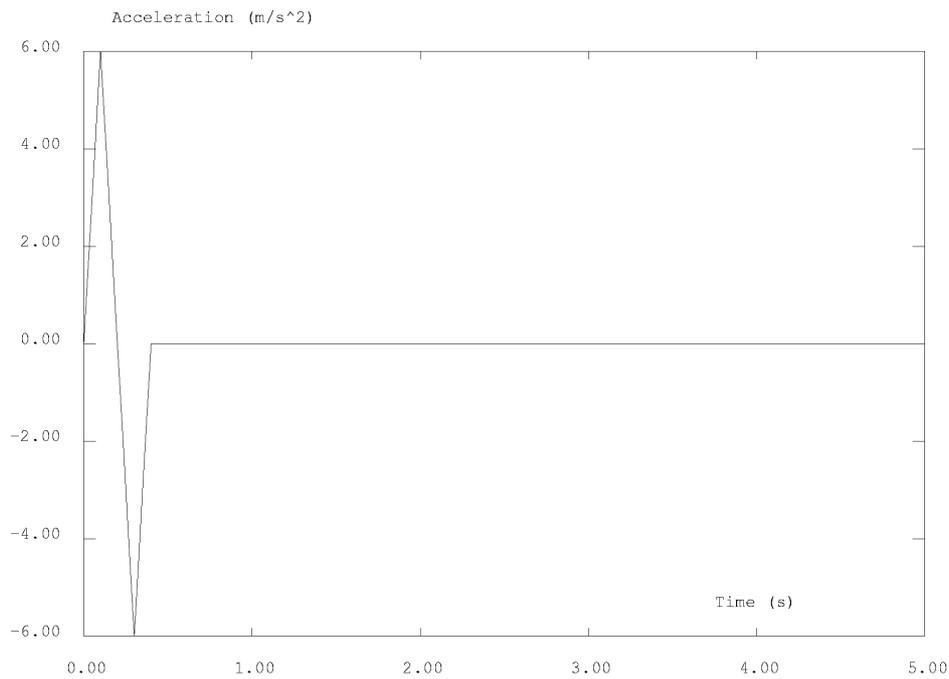
It was discussed in the previous section that the large displacements in the perpendicular direction were not expected a priori. In order to prove that this is the correct behaviour of the structure, a dynamic analysis by means of a direct time integration scheme is done. In this case the Centred Newmark Method is used, which was described in section 2.2.4. Since it is a direct integration procedure, no transformation of the equilibrium equations is performed, thus, no vibration modes are neglected. Furthermore, not only the maximum structural response is obtained, but all the time-history for displacements and tensions can be calculated. The unique assumption is on the variation of the displacements and velocities within each time step, which can be found in equations (2.16) and (2.17).

The structure analysed in this example is the building with two braced frames per façade and one diagonal of 150 mm. in each frame. The applied dynamic load is the mass of the roof multiplied by an acceleration in the x-direction, as shown in figures 7.16 and 7.17.



Cast3m2001 Education Recherche : Direction of the Forces

**Figure 7. 16** Direction of the forces acting on the roof

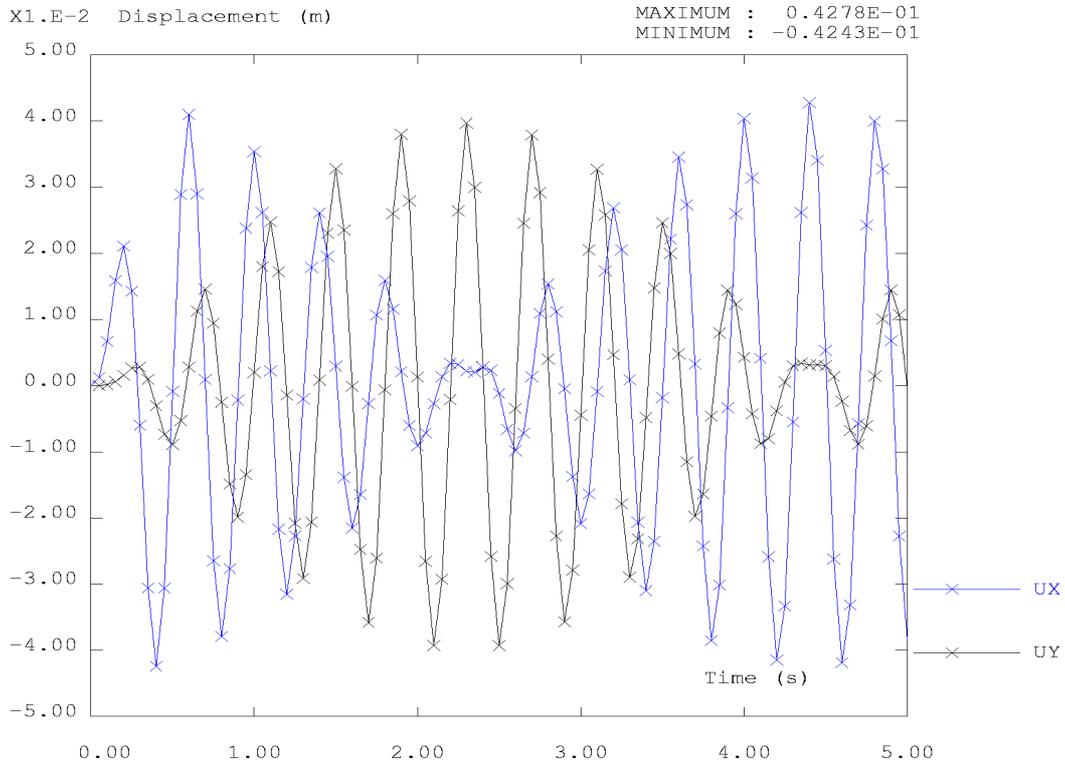


Cast3m2001 Education Recherche : Acceleration

**Figure 7. 17** Evolution of the acceleration applied on the roof.

As the roof is much stiffer than the columns in x and y directions it approximately moves as a rigid body, so all displacements are similar and only the results at the middle of the long façade are presented. Figure 7.18 shows the time-history of  $u_x$  and  $u_y$ . It is seen that when  $u_x$  increases  $u_y$  decreases, and the other way round. Thus, when one of them has a peak value the other variable is close to 0.

Moreover, the peak value for each displacement is not always the same, since it is also variable in time.

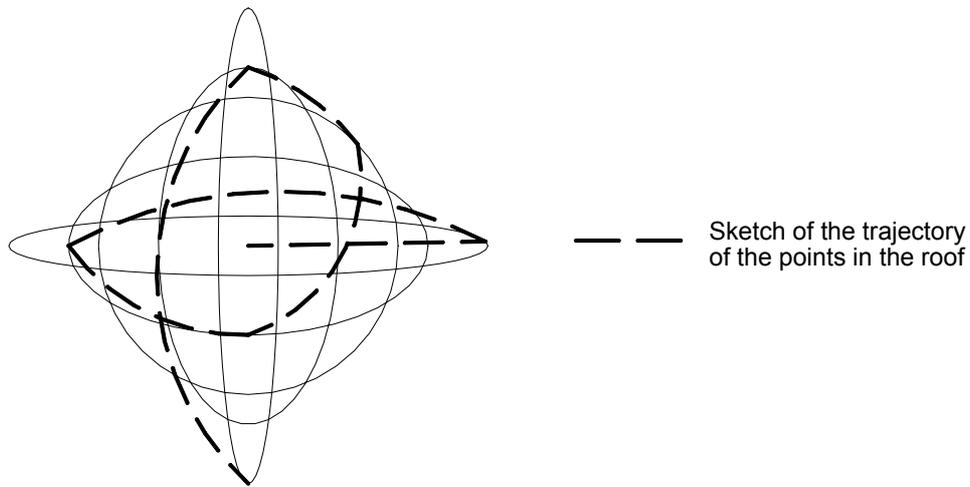


Cast3m2001 Education Recherche : Evolution of the displacements Ux and Uy

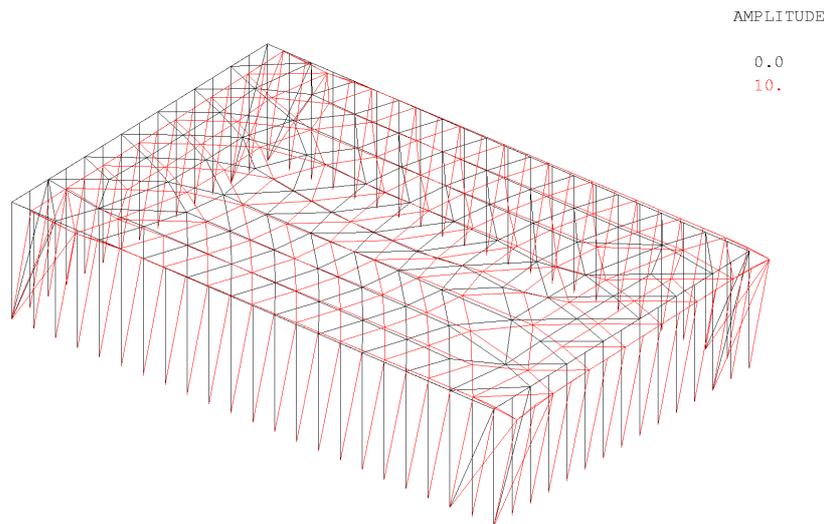
**Figure 7. 18** Evolution of the horizontal displacements:  $u_x$  and  $u_y$

These facts are the consequence of a not straight vibration of the roof, since it is the combination of elliptic trajectories in the x-y plane, as shown in the sketch in figure 7.19. The movement starts as an ellipse with the x-axis much greater than the y-axis,  $r_x \gg r_y$ . Then, the x-axis decreases and the y-axis increases, until the maximum value of  $u_y$  is reached and the relation of axis is  $r_y \gg r_x$ . After this moment, the evolution of the axes is the inverse, until the original ellipse. This behaviour is cyclical in time. Consequently, the deformed shape obtained in the response spectrum analysis will never occur in the structure, because the maximum displacements, shown in figures 7.20 and 7.21, are not simultaneous. So, working with only the maximum values will lead to conservative results.

To conclude, the response of the structure is not limited to the x-direction, as we wanted to prove.

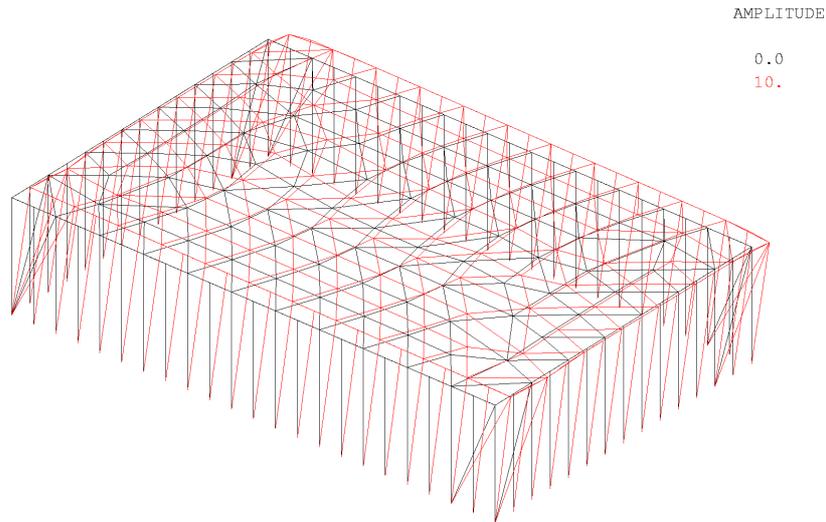


**Figure 7. 19** Sketch of the trajectory in the x-y plane of a point in the roof



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**Figure 7. 20** Maximum displacement of the roof in the x-direction. The scale factor for the displacements is 10.



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**Figure 7. 21** Maximum displacement of the roof in the y-direction. The scale factor for the displacements is 10.

## **7.5 Is the simplification into a 2D model possible?**

We will try to answer to this question by comparing the results for two of the complete 3D structures and their possible simplifications as 2D façades. If the vibration of the structure would have been mainly in the same direction as the application of the ground motions, then it would have been more clear that this simplification might be possible. But, it was seen in the previous section that also the response in the perpendicular direction is significant and not negligible.

As done in chapter 6, it is assumed that the threshold of the analysed façade bears half of the roof weight. In the 3D model it is not true, since the beams at the ends will support less mass because of the collaboration of the perpendicular façade. But this simplification is done in order to have a uniform mass distribution on the continuous beam. That fact has to be considered when discussing the response of the structural systems.

The 2D façades and the complete structure of the building have been subjected to the Eurocode ground motions, applied in either only one direction or simultaneously in two perpendicular directions. The results are summarized in tables 7.7 for the structure with four braced frames per façade and two diagonals in each one, and in table 7.8 for the one with three braced frames and also two diagonals each.

|              | Direction of the Seism | Eigenfrequency (Hz) | Maximum displacement of the roof (mm) | Maximum Tension in a diagonal (MPa) |
|--------------|------------------------|---------------------|---------------------------------------|-------------------------------------|
| Long Façade  | X                      | 4.0280              | 20.1                                  | 202                                 |
| Short Façade | Y                      | 4.1483              | 18.0                                  | 200                                 |
| 3D Structure | X                      | 4.3018              | (14.3, 7.5)                           | 188                                 |
| 3D Structure | X and Y                | 4.3018              | (16.2, 16.3)                          | 228                                 |

**Table 7. 7** Structural response of the complete building with 4 braced frames per façade and the equivalent 2D simplification

|              | Direction of the Seism | Eigenfrequency (Hz) | Maximum displacement of the roof (mm) | Maximum Tension in a diagonal (MPa) |
|--------------|------------------------|---------------------|---------------------------------------|-------------------------------------|
| Long Façade  | X                      | 3.5389              | 24.5                                  | 272                                 |
| Short Façade | Y                      | 3.5615              | 24.1                                  | 267                                 |
| 3D Structure | X                      | 3.7364              | (18.1, 11.4)                          | 235                                 |
| 3D Structure | X and Y                | 3.7364              | (21.3, 21.9)                          | 284                                 |

**Table 7. 8** Structural response of the complete building with 3 braced frames per façade and the equivalent 2D simplification

The first thing it may be noticed is the reduction in the eigenfrequency value when working with the simplified 2D façade, which is probably caused by the consideration of a larger roof mass than in reality, as previously mentioned in the actual discussion.

This lower eigenfrequency is a reason for having greater horizontal displacements in the façade, because the value of  $\frac{1}{\omega^2}$ , in equation (3.21), increases. But also that the little stiffness provided by the perpendicular façades is neglected. Anyway, the calculated displacements for the 2D models are larger than for any of the 3D models, so they are on the safe side.

However, the design of the building should be done with regard to the ultimate limit state, i.e. tension in the structural elements, for which two different cases should be distinguished.

- When the seismic load acts in only one direction, the maximum tension in the diagonal straps is larger in the 2D façade than in the 3D complete structure. This is because in the plane analysis the contribution from the diagonals in the perpendicular façade is neglected. Thus, the inertia forces from the roof mass have to be resisted by less diagonals. In this case the simplification into a 2D analysis would lead to a structural response in the safe side.
- But the Eurocode instructions force the analysis to be done with two perpendicular actions of the same magnitude. Then, the maximum tension for the 2D model is lower, so, it is not in the safe side. It has been shown that the application of a seismic action in the y-direction originates tension in the straps in the perpendicular façade. This tension is added, by a proper combination procedure, to the main one, that has been obtained with the ground motions in

the x-direction. As a result, the final tension is greater than just considering the façade.

When analysing the façade only the main tension in the diagonal member can be calculated, so the secondary tension to be added is unknown. Therefore, the simplification of the complete building into a 2D model cannot be done with enough reliability on the results until some good procedure to estimate this secondary tension is derived.