6. DIAPHRAGM DESIGN

6.1. Interaction between diaphragm and frames

A detailed explanation on how to calculate the diaphragm has been presented: how to calculate its resistance, its deformation. A third point, maybe the most important, should not be forgotten and it is to check that the diaphragm *really* actuates as a diaphragm.

When the horizontal loads act on the long lateral side of the building (following the direction of the frames). These loads are immediately distributed to the foundations and a part of it is transmitted to the diaphragm. The part transmitted to the diaphragm depends on two factors: the relative flexibility of the diaphragm and the frame, and the position of the stiffened portals in the building.

A building that is too long or a diaphragm that is too flexible will not absorb any force, and consequently the frames (which maybe are not designed for it) will absorb all the lateral force.

A method to calculate this interaction, and an analysis of the results in different cases realised by the author are provided in the next pages.

6.1.1. Interaction diaphragm-frames in a symmetric building

6.1.1.1. Method to calculate the distribution of loads

The building is modelled as it can be seen in figure 5.1:

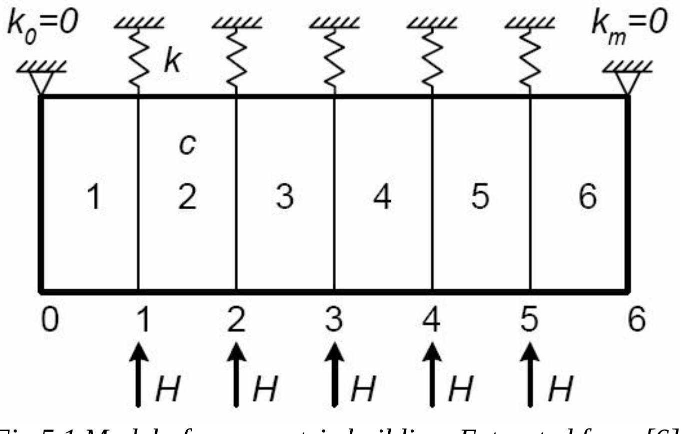

Fig 5.1 Model of a symmetric building. Extracted from [6]

The following parameters are defined:

- *c* is the flexibility of the diaphragm. It can be obtained by the calculation explained in the chapter III. And it is equal to the flexibility of the "shear cell"
- *k* is the flexibility of the frame. It can be calculated by means of elastic analysis of the frame.

- *r=c/k* is the relative flexibility of the diaphragm respect to the flexibility of the frame. It is an important value as it will be seen the distribution of forces do not depend on the absolute values of c and k if not in the relative flexibility r.
- *n* is the number of bays in a building. The number of frames in a building with n bays is n+1
- H_i as the horizontal force acting in each frame

The calculation to obtain R_j (force on frames), S_i (diagram forces) and F_j (force on diaphragm is the following (extracted from The SBI174 [6], example 3) is the following:

It is defined f_i and h_i as the distribution factors to the right with the following definition:

$$\begin{cases} f_n = 0 \\ h_n = 1 \end{cases}$$

$$\begin{cases} \text{for } i = n-1 \text{ to } 0 \\ f_i = \dfrac{k_i}{k_i + c_{i+1} + k_{i+1}(1 - f_{i+1})} \\ h_i = 1 - f_i \end{cases}$$

With f_n=0 it is assumed that the end frame is rigid. f_i is an evaluation of the flexibility k_i with respect to all the flexibilities of the elements on the right side of the building.

The same procedure can be done, but this time for all the elements on the left side of the, after i. The parameters g_i and q_i are defined as follows:

$$\begin{cases} g_0 = 0 \\ q_0 = 1 \end{cases}$$

$$\begin{cases} for\ i = 1 \text{ to } n \\ g_i = \dfrac{k_i}{k_i + c_i + k_{i-1}(1 - f_{i-1})} \\ q_i = 1 - g_i \end{cases}$$

Like in the factors to the right, the end frame is considered rigid g_0=0

A number of transfer factors are defined as follows:

$$for\ j = 1 \text{ to } (n-1)$$

$$p_{v_{j,j}} = \frac{k_{j-1} \cdot q_{j-1} + c_j}{k_j}$$

$$p_{h_{j,j}} = \frac{k_{j+1} \cdot q_{j+1} + c_{j+1}}{k_j}$$

$$m_{r_{j,j}} = \frac{p_{v_{j,j}} \cdot p_{h_{j,j}}}{p_{v_{j,j}} \cdot p_{h_{j,j}} + p_{h_{j,j}} + p_{h_{j,j}}}$$

$$m_{v_{j,j}} = -\frac{p_{h_{j,j}}}{p_{v_{j,j}} \cdot p_{h_{j,j}} + p_{h_{j,j}} + p_{h_{j,j}}}$$

$$m_{h_{j,j}} = -\frac{p_{v_{j,j}}}{p_{v_{j,j}} \cdot p_{h_{j,j}} + p_{h_{j,j}} + p_{h_{j,j}}}$$

Two matrices can now be created: Q defined as forces on frames and V defined as shear force in diaphragm, both for each action H.

The global calculation calculates the effect of one load H_j how affects the rest of the building, and adding the effect of all the individual loads. The iteration to create the matrices is as follows:

For j=1

$$V_{j,j} = m_{h\,j,j} \cdot H_j$$
$$V_{j-1,j} = m_{v\,j,j} \cdot H_j$$
$$Q_{j,j} = m_{r\,j,j} \cdot H_j$$
$$\begin{cases} for\ \text{i} = 1 \text{ to } (n-j) \\ V_{j+1,j} = f_{j+i} \cdot V_{j+i-1,j} \\ Q_{j+1,j} = h_{j+i} \cdot V_{j+i-1,j} \end{cases}$$
$$\begin{cases} for\ \text{i} = 1 \text{ to } j \\ Q_{j-1,j} = -q_{j-i} \cdot V_{j-i,j} \end{cases}$$

For j= 2 to (n-1)

$$V_{j,j} = m_{h\,j,j} \cdot H_j$$
$$V_{j-1,j} = m_{v\,j,j} \cdot H_j$$
$$Q_{j,j} = m_{r\,j,j} \cdot H_j$$
$$\begin{cases} for\ \text{i} = 1 \text{ to } (n-j) \\ V_{j+1,j} = f_{j+i} \cdot V_{j+i-1,j} \\ Q_{j+1,j} = h_{j+i} \cdot V_{j+i-1,j} \end{cases}$$
$$\begin{cases} for\ \text{i} = 2 \text{ to } j \\ V_{j-1,j} = g_{j-i+1} \cdot V_{j-i+1,j} \end{cases}$$
$$\begin{cases} for\ \text{i} = 1 \text{ to } j \\ Q_{j-1,j} = -q_{j-i} \cdot V_{j-i,j} \end{cases}$$

The vector R_i (force absorbed in each frame) is the sum of the values of the row(i) of the matrix Q,
The vector S_i (shear flow in each shear cell) is the sum of the values of the row(i) of the Matrix V,
F is the difference between the force on frames and the external horizontal force, would mean the force absorbed by the shear connectors or the purlin connections in the rafter

$$j = 0\ to\ n$$
$$R_i = \sum \left(Q^T\right)^{<i>}$$
$$S_i = \sum \left(V^T\right)^{<i>}$$
$$F_j = H_j - R_j$$

An excel sheet has been programmed (can be found in the CD attached to the Dissertation) with the previous schema. An analysis of the schema for different conditions has been made and is the combination of the following:

- Buildings of 3 to 12 frames
- Relative rigidities from 0,01 to 100 including 35 different values in between
- Constant force of 1 KN per frame, in all the frames

The results of this calculation are presented in the table III.1 of annex III. There is also a table in the ECCS where it is given for the symmetric problem the values to share to the frame and to the diaphragm (and the results are the same, of course).

6.1.1.2. <u>Example</u>

This is the result of the output for a 8 bay building, with k=1 mm/KN and c=0.3 mm/KN. Let's imagine that the horizontal constant Load is of 40 KN in each frame. The output is then:

c=0,3	k=1	r=0,3

j	Hj	Rj	Qj	Fj	
0	0	53,74	-53,74	-53,75	
1	40	16,128	-29,87	23,88	
2	40	25,08	-14,9	14,91	
3	40	29,57	-4,53	10,43	
4	40	30,93	4,53	9,07	
5	40	29,57	14,95	10,43	
6	40	25,08	29,87	14,91	
7	40	16,12	53,74	23,88	
8	0	53,74	0	-53,75	

Table 5.1. Output for an 8 bay building

How can these values be placed in the building?

The figure 5.2 shows the actions on the building. The second (fig 5.3) are the forces on the frame and the third one (fig 5.5) is the action on the diaphragm, with the shear field forces in each diaphragm.

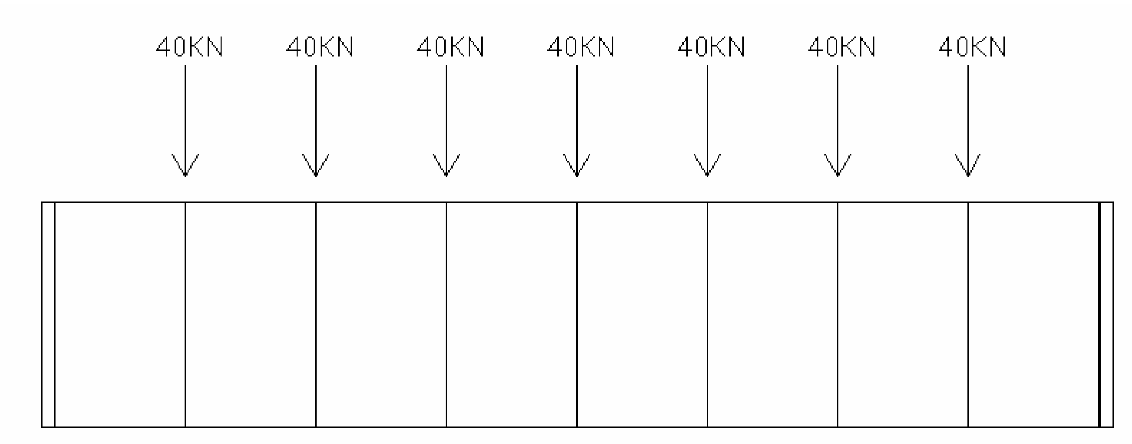

Fig 5.2. Actions on the building

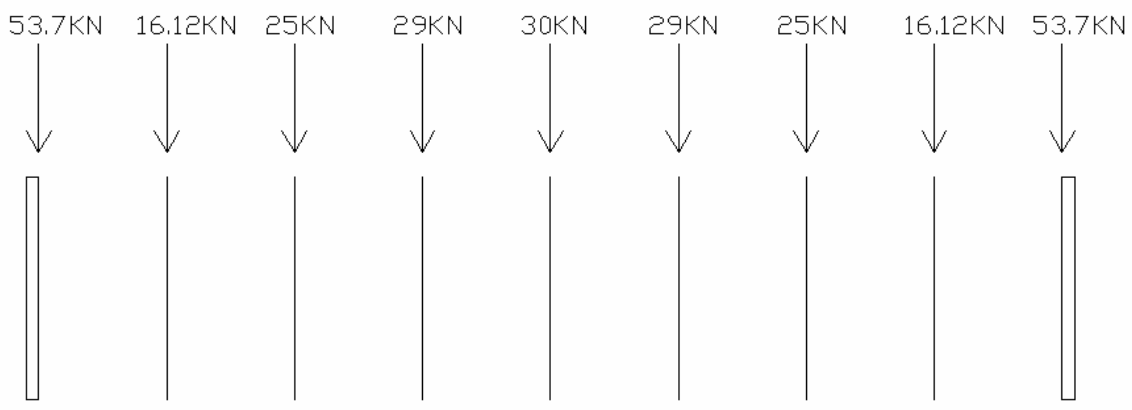

Fig 5.3. Actions on the frames

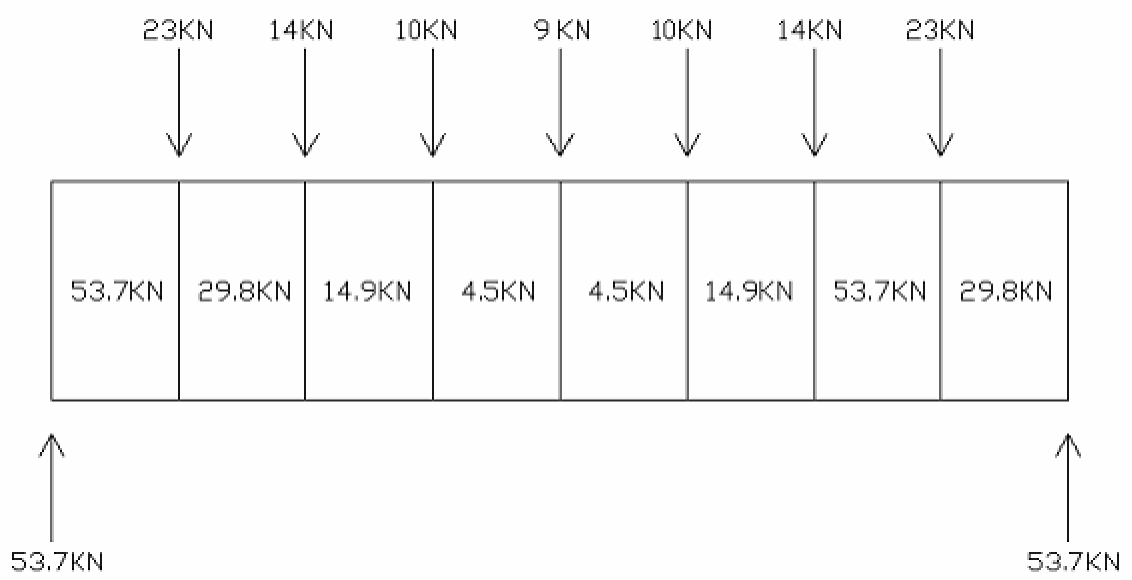

Fig 5.4. Actions on the diaphragm

6.1.1.3. <u>Deformation of the building</u>

- Flexibility in the middle point if the frame would not absorb any force (frame much more flexible than the diaphragm):

$$\Delta = \frac{n^2}{8} \cdot c \cdot qa$$

n=number of bays in the building
c=flexibility of the diaphragm
qa=load per frame(KN)

In our case this value would be:

$$\Delta = \frac{8^2}{8} \cdot 0{,}3 \cdot 40 = 96mm$$

- Flexibility in the middle point if the frame would absorb all the force (diaphragm much more flexible than the diaphragm)

$$\Delta = k_{frame} \cdot H_{frame}$$

In our case

$$\Delta = 1 \cdot 40 = 40mm$$

- Flexibility if the compatible deformation of the frame and the diaphragm is considered. The total deformation can be calculated, assuming that if the force on the frames and its flexibility is known. The deformation in each frame is the flexibility multiplied with R_i .The first two frames have k=0, there is no deformation there:

j	Rj	Uj(mm)=k_j R_j
1	53,74	0
2	16,13	16,13
3	25,08	25,08
4	29,57	29,57
5	30,93	30,93
6	29,57	29,57
7	25,08	25,08
8	16,13	16,13
9	53,74	0

Table 5.2. Deformation in the 8 bay building of the example

The maximum deformation would be 30,93 mm

It can be observed that, the deformation when assuming the compatibility is between the extreme cases. The closer to one extreme or the other will depend on the relative flexibility.

So as a conclusion:

-The absolute deformation depends on the relative flexibility r, and the absolute value of c and k

- The distribution of the forces to the frames and the values of shear stress in the diaphragm depends on the relative flexibility r
- The absolute value of these forces depends on the flexibility r and the absolute value of the incoming forces.

6.1.1.4. <u>Calculation's example from the values provided in the table III.1</u>

How to perform the previous analysis with data provided in the Table III.1 from annex III; The following values would be found for the r=0,3 and 8 bays.

8	1	0,403
	2	0,627
	3	0,739
	4	0,773

Table 5.3. Value extracted from table II for an the 8 bay building of the example

1) Place first the values of the external actions in the H_j and the sum of the H_j
2) Multiply H_j with the reduction values of the table. The number is how far is the frame from a stiffened portal
3) The force on the extremes of the portals are the total force on the side of the building, minus all the values filled before, divided by two
4) The values of $F_j=H_j-R_j$
5) As a last step Q_j can be filled as follows:

$Q_0=F_0$

$Q_j=Q_{j-1}-F_j$

c=0,3	k=1	r=0,3

j	Hj	Rj	Qj	Fj	
0	0	(280-172.38)/2=53,81	-53,81	0-53,81=-53,81	
1	40	40x0,403=16,12	-53,81+23,88=-29,9	40-16,12=23,88	
2	40	40x0,627=25,08	-29,9+14,92=-15,01	40-25,08=14,92	
3	40	40x0,739=29,53	-15,01+10,47=-4,57	40-29,53=10,47	
4	40	40x0,773=30,92	-4,97+9,08=4,51	40-30,92=9,08	
5	40	40x0,739=29,53	4,51+10,47=14,98	40-29,53=10,47	
6	40	40x0,627=25,08	14,98+14,92=29,9	40-25,08=14,92	
7	40	40x0,403=16,12	29,9 + 23,88=52,81	40-16,12=23,88	
8	0	(280-172.38)/2=53,81	0	0-53,81=-53,81	
Total	280				

Table 5.4. Calculation out of the table 5.3 values the 8 bay building of the example

6.1.2. Interaction diaphragm-frames in a non-symmetric building

The previous formulation ,as it is a symmetric calculating procedure, it is not useful to calculate when the building it is not symmetric. The example of this kind of building is a building, where the dilatation joint is in the middle of the building. And it is not possible to stiffen the end bay.

Another approximation has been taken in this case, as the previous calculating schema was not appropriate for this calculation

6.1.2.1. <u>Method to calculate the distribution of loads</u>

The building is modelled as it can be seen in figure 5.5:

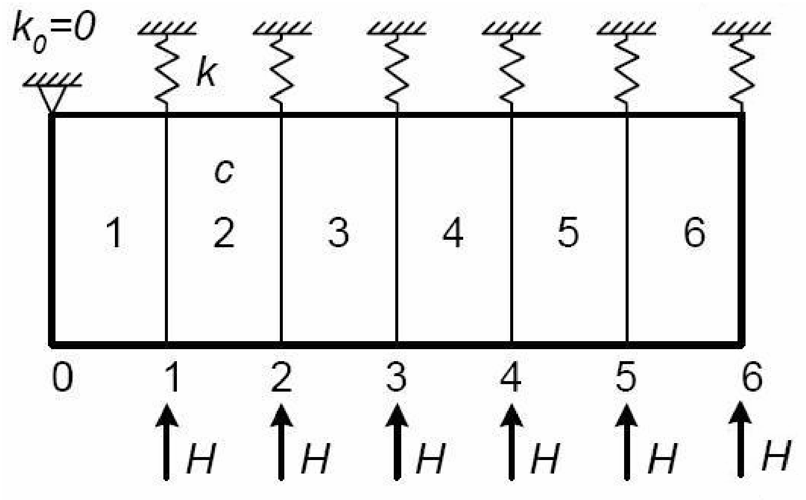

Fig 5.4. Actions on the diaphragm

The following parameters are defined:

- *c* is the flexibility of the diaphragm.
- *k* is the flexibility of the frame.
- $r=c/k$ is the relative flexibility of the diaphragm respect to the flexibility of the frame.
- *n* is the number of bays in a building. The number of frames in a building with n bays is n+1
- H_i as the horizontal force acting in each frame

The calculation procedure is based in the assumption that the deformation of the frame and the diaphragm are the same in each frame.

Each frame absorbs a Force $H_i\alpha_i$, with α_i as the distribution factor of the forces to the frames or the diaphragm.

The distribution of the forces absorbed by the frame

$$i = 0$$

$$R_0 = \sum_{i=1}^{n} H_i - \sum_{i=1}^{n} (H_i \cdot \alpha_i)$$

$$For \text{ i} = 1 \text{ to n}$$

$$R_i = H_i \cdot \alpha_i$$

The distribution of the forces absorbed by the diaphragm is then:

$$i = 0$$

$$F_0 = -R_0 = -\sum_{i=1}^{n} H_i + \sum_{i=1}^{n} (H_i \cdot \alpha_i)$$

$$For \text{ i} = 1 \text{ to n}$$

$$F_i = H_i \cdot (1 - \alpha_i)$$

The Shear stress in each part i of the diaphragm is then

$$i = 1$$

$$S_1 = -R_0 = -\sum_{i=1}^{n} H_i + \sum_{i=1}^{n} (H_i \cdot \alpha_i)$$

$$For \text{ i} = 2 \text{ to n}$$

$$S_i = -R_0 + \sum_{j=1}^{i-1} (H_j \cdot \alpha_j)$$

The deformation in each frame i is then:

$$For \text{ i} = 1 \text{ to n}$$

$$\delta_i^f = k \cdot H_i \cdot \alpha_i$$

The deformation of the diaphragm in each frame i is:

$$For \text{ i} = 1 \text{ to n}$$

$$\delta_i^d = -c \cdot \sum_{j=1}^{i} S_i$$

It is imposed that both deformations are equal then:

$$For \text{ i} = 1 \text{ to n}$$

$$\delta_i^f = \delta_i^d$$

$$k \cdot H_i \cdot \alpha_i = -c \cdot \sum_{j=1}^{i} S_i$$

$$H_i \cdot \alpha_i = -r \cdot \sum_{j=1}^{i} S_i$$

Written in a matrix form, the linear system looks like this:

$$\begin{pmatrix} (1+r)H_1 & rH_2 & rH_3 & 1rH_4 & \dots & rH_i & \dots & rH_n \\ rH_1 & (1+2r)H_2 & 2rH_3 & 2rH_4 & \dots & 2rH_i & \dots & 2rH_n \\ rH_1 & 2rH_2 & (1+3r)H_{31} & 3rH_4 & \dots & 3rH_i & \dots & 3rH_n \\ rH_1 & 2rH_2 & 3rH_3 & (1+4r)H_4 & \dots & 4rH_i & \dots & 4rH_n \\ \vdots & \vdots & \vdots & \vdots & \ddots & \vdots & & \vdots \\ rH_1 & 2rH_2 & 3rH_3 & 4rH_4 & \dots & (1+i\cdot r)H_i & \dots & i\cdot rH_n \\ \vdots & \vdots & \vdots & \vdots & & \vdots & \ddots & \vdots \\ rH_1 & 2rH_2 & 3rH_3 & 4rH_4 & \dots & i\cdot rH_i & \dots & (1+nr)H_n \end{pmatrix} \cdot \begin{pmatrix} \alpha_1 \\ \alpha_2 \\ \alpha_3 \\ \alpha_4 \\ \\ \alpha_i \\ \\ \alpha_n \end{pmatrix} = \cdots$$

$$\ldots = r \cdot \begin{pmatrix} H_1 + H_2 + H_3 + H_4 + \cdots + H_i + \ldots + H_n \\ H_1 + 2H_2 + 2H_3 + 2H_4 + \cdots + 2H_i + \ldots + 2H_n \\ H_1 + 2H_2 + 3H_3 + 3H_4 + \cdots + 3H_i + \ldots + 3H_n \\ H_1 + 2H_2 + 3H_3 + 4H_4 + \cdots + 4H_i + \ldots + 4H_n \\ \\ H_1 + 2H_2 + 3H_3 + 4H_4 + \cdots + i\cdot H_i + \ldots + i\cdot H_n \\ \\ H_1 + 2H_2 + 3H_3 + 4H_4 + \cdots + i\cdot H_i + \ldots + n\cdot H_n \end{pmatrix}$$

When the system is solved the parameters alpha are obtained, and all S, F, R can be obtained with the formulation calculated previously

As is was done for the symmetric building a table with the transfer factors alpha is provided in the Table IV.1 Annex IV. The values presented there assume the following hypotheses:

- Compatibility of the deformation of the building and the diaphragm.
- Building of n bays,
- Forces H_1 to H_{n-1} = 1 KN , H_n = 0,5 KN acting on the side of the building.
- The diaphragm has a constant flexibility c and all the frames have a flexibility k except the first one which is zero (rigid frame)

6.1.2.2. Example

This is the result of the output for a 4 bay building, with k=1 mm/KN and c=0.3 mm/KN. Let's imagine that the horizontal constant Load is of 40 KN in each frame except in the last one where it is 20 KN. The output is then:

c=0,3	k=1	r=0,3

j	Hj	Rj	Qj	Fj	
0	0	50,96	-50,96	-50,96	
1	40	15,28	-26,24	24,72	
2	40	23,16	-9,4	16,84	
3	40	26	4,6	14	
4	20	24,6	0	-4,6	

Table 5.5. 4 bay non-symmetric building

How can these values be placed in the building?

The figure 5.5 shows the actions on the building. The second are the forces on the frame and the third one is the action on the diaphragm, with the shear field forces in each diaphragm.

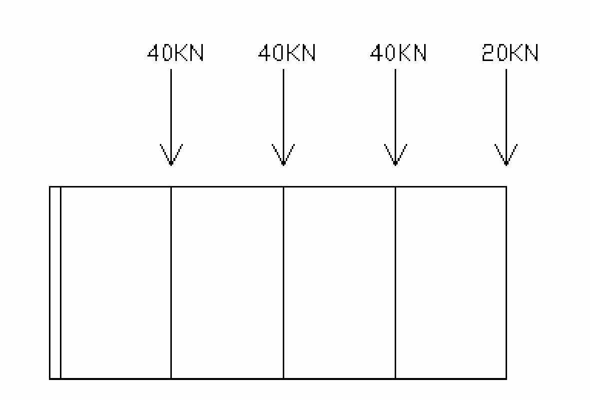

Fig 5.5. Actions on the building

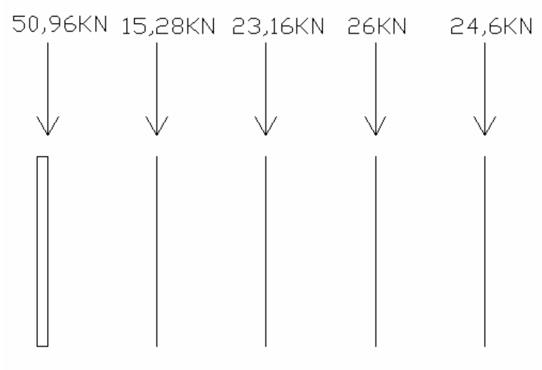

Fig 5.6. Actions on the frames

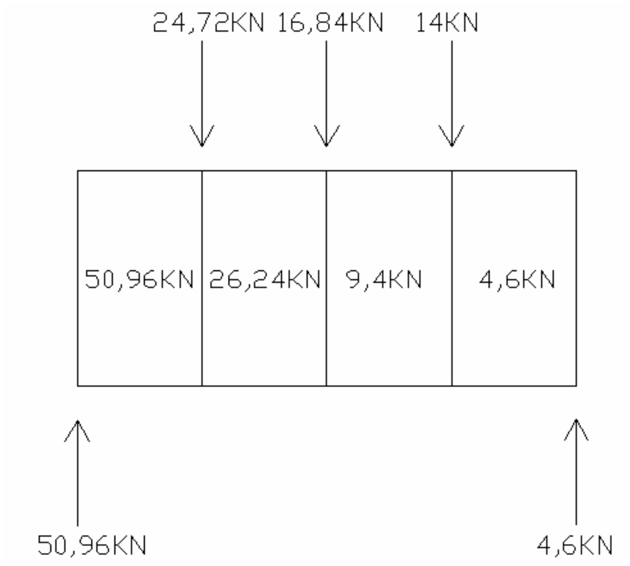

Fig 5.7. Actions on the diaphragm

6.1.2.3. Deformation of the building

- Flexibility in the bay before the end bay, if the frame would not absorb any force (frame much more flexible than the diaphragm):

$$\Delta = (100 + 60 + 20) \cdot 0{,}3 = 54mm$$

- Flexibility in the bay before the end bay, if the frame would absorb all the force (diaphragm much more flexible than the diaphragm)

$$\Delta = k_{frame} \cdot H_{frame}$$

In our case

$$\Delta = 1 \cdot 40 = 40mm$$

- Flexibility if the compatible deformation of the frame and the diaphragm is considered. The total deformation, assuming that the force on the frames and its flexibility is known. The deformation in each frame would be the force in the frame multiplied by the frame flexibility. The first frame has k=0, there is no deformation there:

j	Rj	Uj(mm)=k_j H_j
0	50,96	0
1	15,28	15,28
2	23,16	23,16
3	26	26
4	24,6	24,6

Table 5.5. Deformation of the 4 bay example building

The maximum deformation would be 26 mm, and 24,6 mm in the end bay.

The position of the most deformed frame is not always the same frame as it was on the symmetric building, here depending on the r this will be the next to last, the one before that.

6.1.2.4. Calculation example from the values provided in the table

How to perform the previous analysis with data provided in the Table IV.1 from annex IV? the following values for r=0,3 and 4 bays are given:

4	1	0,382
	2	0,579
	3	0,650
	4	1,230

Table 5.6. Values on the table for the 4 bay example building

1) Place first the values of the external actions in the H_j and the sum of the H_j
2) Multiply H_j per the corresponding alpha parameter.
3) The force on the extremes of the portals is the total force on the side, minus all the values filled before,
4) The values of $F_j=H_j-R_j$
5) As a last step Q_j can be filled as follows:
$Q_0=F_0$
$Q_j=Q_{j-1}-F_j$

c=0,3		k=1		r=0,3	
j	Hj	Rj	Qj	Fj	
0	0	(140-89,04) =50,96	-50,96	0-53,81=-50,96	
1	40	40x0,382=15,28	-50,96+24,72=-26,24	40-15,28=24,72	
2	40	40x0,579=23,16	-26,24+16,84= -9,4	40-23,16=16,84	
3	40	40x0,650=26	-9,4+14=4,6	40-26=14	
4	20	20x1,230=24,6	4,6-4,6=0	20-24,6=-4,6	
Total	140				

Table 5.6. Calculation of the reaction out of the values in table 5.6

6.1.3. Analysis of the effectiveness of the diaphragm design

6.1.3.1. Effectiveness of the diaphragm design

The first analysis is to check which is the maximum relative flexibility (r) that can be allowed in the different buildings in order to consider that they are stiffened with the diaphragm. The building has the same portals over the whole length; as a consequence the diaphragm will be effective if all the frames are restrained. What is an effective diaphragm?
It is considered that an effective diaphragm is the one that absorbs at least the 40% of the horizontal load on the frames.
Let’s examine the tables for a symmetric and non-symmetric building.

n	fn	0,01	0,02	0,03	0,04	0,06	0,08	0,1	0,2	0,3	0,4	0,5	0,6	0,7	0,8	0,9	1	1,1
3	1	0,010	0,020	0,029	0,038	0,057	0,074	0,091	0,167	0,231	0,286	0,333	0,375	0,412	0,444	0,474	0,500	0,524
	1	0,015	0,029	0,043	0,056	0,082	0,106	0,129	0,225	0,301	0,362	0,412	0,454	0,490	0,521	0,548	0,571	0,593
4	2	0,020	0,039	0,057	0,075	0,109	0,140	0,170	0,296	0,392	0,468	0,529	0,580	0,622	0,658	0,688	0,714	0,737
	1	0,020	0,038	0,056	0,073	0,104	0,134	0,160	0,268	0,347	0,407	0,455	0,494	0,526	0,554	0,579	0,600	0,619
5	2	0,029	0,057	0,083	0,108	0,155	0,198	0,237	0,390	0,497	0,576	0,636	0,684	0,721	0,752	0,778	0,800	0,819
	1	0,024	0,047	0,068	0,088	0,124	0,157	0,186	0,298	0,375	0,432	0,477	0,513	0,543	0,569	0,591	0,611	0,629
	2	0,039	0,074	0,108	0,139	0,196	0,246	0,291	0,457	0,563	0,638	0,692	0,734	0,767	0,793	0,815	0,833	0,849
6	3	0,043	0,084	0,121	0,156	0,219	0,275	0,325	0,506	0,620	0,698	0,754	0,795	0,827	0,852	0,873	0,889	0,902
	1	0,029	0,055	0,079	0,101	0,141	0,176	0,207	0,319	0,393	0,447	0,488	0,522	0,551	0,575	0,597	0,615	0,632
	2	0,048	0,091	0,130	0,167	0,231	0,286	0,334	0,502	0,603	0,672	0,721	0,758	0,787	0,811	0,830	0,846	0,860
7	3	0,057	0,109	0,156	0,199	0,274	0,339	0,394	0,585	0,695	0,765	0,814	0,849	0,875	0,895	0,911	0,923	0,933
	1	0,033	0,063	0,089	0,113	0,156	0,191	0,222	0,333	0,403	0,454	0,494	0,527	0,554	0,578	0,599	0,617	0,634
	2	0,056	0,106	0,151	0,191	0,260	0,318	0,367	0,532	0,627	0,690	0,735	0,770	0,796	0,818	0,836	0,851	0,864
	3	0,070	0,132	0,187	0,237	0,321	0,390	0,449	0,637	0,739	0,802	0,844	0,874	0,896	0,913	0,926	0,936	0,945
8	4	0,075	0,141	0,199	0,252	0,341	0,414	0,475	0,670	0,773	0,835	0,875	0,903	0,923	0,938	0,949	0,957	0,964
	1	0,037	0,070	0,098	0,124	0,168	0,204	0,235	0,342	0,409	0,458	0,497	0,529	0,556	0,579	0,599	0,618	0,634
	2	0,065	0,121	0,170	0,213	0,285	0,344	0,393	0,552	0,641	0,700	0,743	0,775	0,800	0,821	0,838	0,853	0,865
	3	0,083	0,154	0,216	0,270	0,360	0,431	0,490	0,672	0,766	0,822	0,860	0,886	0,906	0,920	0,932	0,941	0,949
9	4	0,092	0,171	0,239	0,298	0,396	0,474	0,536	0,727	0,820	0,873	0,906	0,929	0,944	0,956	0,964	0,971	0,976
	1	0,041	0,076	0,106	0,133	0,177	0,213	0,244	0,348	0,413	0,461	0,499	0,530	0,556	0,579	0,600	0,618	0,634
	2	0,073	0,134	0,186	0,231	0,305	0,364	0,412	0,565	0,650	0,706	0,746	0,777	0,802	0,822	0,839	0,854	0,866
	3	0,095	0,175	0,241	0,299	0,392	0,464	0,521	0,695	0,781	0,833	0,867	0,892	0,910	0,924	0,934	0,943	0,950
	4	0,109	0,199	0,274	0,338	0,441	0,521	0,583	0,763	0,847	0,893	0,922	0,941	0,954	0,963	0,970	0,976	0,980
10	5	0,113	0,206	0,285	0,351	0,458	0,539	0,603	0,785	0,867	0,911	0,938	0,954	0,966	0,974	0,980	0,984	0,987
	1	0,045	0,082	0,114	0,141	0,185	0,221	0,251	0,351	0,415	0,462	0,499	0,530	0,557	0,580	0,600	0,618	0,634
	2	0,081	0,146	0,201	0,247	0,322	0,380	0,427	0,573	0,654	0,709	0,748	0,779	0,803	0,823	0,840	0,854	0,866
	3	0,107	0,193	0,264	0,323	0,418	0,489	0,545	0,709	0,790	0,839	0,871	0,894	0,912	0,925	0,935	0,944	0,951
	4	0,124	0,224	0,305	0,373	0,478	0,557	0,618	0,787	0,863	0,904	0,930	0,946	0,958	0,967	0,973	0,978	0,981
11	5	0,133	0,239	0,325	0,397	0,508	0,590	0,653	0,823	0,895	0,932	0,953	0,967	0,975	0,981	0,986	0,989	0,991
	1	0,049	0,088	0,120	0,147	0,192	0,227	0,256	0,354	0,416	0,463	0,500	0,530	0,557	0,580	0,600	0,618	0,634
	2	0,088	0,157	0,214	0,261	0,335	0,392	0,437	0,578	0,657	0,710	0,749	0,779	0,803	0,823	0,840	0,854	0,866
	3	0,118	0,210	0,284	0,345	0,439	0,508	0,562	0,719	0,795	0,842	0,873	0,896	0,912	0,925	0,936	0,944	0,951
	4	0,139	0,247	0,333	0,402	0,508	0,585	0,644	0,803	0,872	0,910	0,934	0,949	0,960	0,968	0,974	0,978	0,982
	5	0,152	0,269	0,361	0,436	0,549	0,629	0,690	0,847	0,911	0,943	0,961	0,972	0,980	0,985	0,988	0,991	0,993
12	6	0,156	0,276	0,371	0,447	0,562	0,644	0,704	0,861	0,922	0,952	0,969	0,979	0,985	0,989	0,992	0,994	0,995

Table 5.7. Distribution factors for symmetric buildings. r= 0.01 to 1.1

The colour schema is the following:

- Red: Diaphragms that absorb less than the 25% of the external actions.
- Green: Diaphragms which absorb between 25-40% of the external actions
- White: Diaphragms which absorb between the 40-75 %
- Yellow: the ones that absorb more than the 75% of the external loads.

n	Fn	0,01	0,02	0,03	0,04	0,06	0,08	0,1	0,2	0,3	0,4	0,5
3	1	0,024	0,046	0,066	0,085	0,119	0,149	0,176	0,277	0,347	0,400	0,442
	2	0,038	0,073	0,105	0,134	0,186	0,231	0,270	0,410	0,498	0,559	0,605
	3	0,085	0,163	0,233	0,296	0,408	0,502	0,582	0,850	0,997	1,084	1,140
	1	0,033	0,061	0,087	0,110	0,149	0,183	0,211	0,314	0,382	0,433	0,474
	2	0,056	0,104	0,146	0,184	0,248	0,300	0,344	0,491	0,579	0,639	0,684
	3	0,069	0,129	0,180	0,226	0,301	0,361	0,411	0,567	0,650	0,701	0,737
4	4	0,147	0,272	0,379	0,472	0,624	0,743	0,838	1,112	1,230	1,288	1,316
	1	0,041	0,075	0,104	0,129	0,171	0,205	0,233	0,334	0,399	0,448	0,488
	2	0,072	0,131	0,180	0,223	0,291	0,346	0,390	0,534	0,618	0,676	0,719
	3	0,094	0,170	0,232	0,285	0,370	0,434	0,486	0,641	0,722	0,774	0,810
	4	0,107	0,192	0,262	0,319	0,410	0,478	0,530	0,676	0,743	0,781	0,805
5	5	0,221	0,396	0,537	0,653	0,830	0,959	1,055	1,294	1,374	1,401	1,407
	1	0,048	0,086	0,117	0,143	0,185	0,219	0,247	0,344	0,408	0,456	0,494
	2	0,087	0,154	0,207	0,251	0,321	0,375	0,418	0,557	0,638	0,693	0,735
	3	0,116	0,204	0,274	0,330	0,417	0,481	0,532	0,681	0,759	0,809	0,843
	4	0,137	0,239	0,318	0,382	0,478	0,546	0,598	0,741	0,808	0,847	0,873
	5	0,149	0,259	0,343	0,409	0,507	0,575	0,624	0,750	0,799	0,825	0,840
6	6	0,305	0,527	0,695	0,825	1,013	1,138	1,226	1,416	1,460	1,464	1,453
	1	0,055	0,095	0,127	0,153	0,195	0,228	0,255	0,350	0,412	0,459	0,497
	2	0,100	0,172	0,228	0,273	0,342	0,394	0,436	0,569	0,648	0,702	0,743
	3	0,136	0,233	0,306	0,363	0,449	0,511	0,560	0,703	0,778	0,826	0,859
	4	0,164	0,278	0,362	0,428	0,523	0,590	0,640	0,777	0,842	0,880	0,906
	5	0,183	0,309	0,400	0,470	0,569	0,636	0,684	0,807	0,858	0,887	0,905
	6	0,194	0,326	0,420	0,491	0,588	0,652	0,696	0,798	0,832	0,848	0,857
7	7	0,394	0,658	0,844	0,982	1,167	1,282	1,357	1,496	1,511	1,497	1,477
	1	0,061	0,103	0,135	0,161	0,202	0,234	0,260	0,353	0,415	0,461	0,499
	2	0,112	0,187	0,244	0,288	0,355	0,406	0,446	0,577	0,654	0,707	0,746
	3	0,154	0,256	0,330	0,387	0,471	0,531	0,577	0,716	0,789	0,835	0,867
	4	0,188	0,310	0,396	0,461	0,554	0,618	0,666	0,798	0,861	0,898	0,922
	5	0,214	0,349	0,444	0,514	0,611	0,675	0,721	0,840	0,891	0,919	0,937
	6	0,231	0,376	0,475	0,547	0,644	0,706	0,748	0,849	0,888	0,908	0,921
	7	0,241	0,391	0,491	0,562	0,656	0,713	0,751	0,829	0,851	0,861	0,866
8	8	0,488	0,785	0,982	1,120	1,294	1,394	1,456	1,548	1,541	1,516	1,488

Table 5.8. Distribution factors for non-symmetric buildings. r= 0.01 to 0.5

The colours in this table are the same as in the symmetric one, in this case the last portal is not coloured, as the load received is the other portals' half, so the one before is usually the limiting one. For values of r > 0.2 the portal before can be the most solicited, but these ones lay anyway on the red zone.

From both tables the maximum relative necessary flexibility of the building so that the diaphragm is effective (at least 40% external forces are absorbed on the more demanded frame) is:

Number of bays	Maximum r necessary for a symmetric building	Maximum r necessary for a non symmetric building
3	1,5	0,4
4	0,6	0,2
5	0,4	0,1
6	0,2	0,08
7	0,2	0,06
8	0,1	0,04
9	0,1	-
10	0,08	-
11	0,08	-
12	0,08	-

Table 5.9. Maximum r so that the diaphragm absorbs at least the 40% of the external forces

It was shown in the previous chapter that the usual flexibilities ranges from 0.1 to 0.8 mm/KN. Let's see the equivalences k corresponding for the selected r:

	C							
R	0,1	0,2	0,3	0,4	0,5	0,6	0,7	0,8
1,5	0,07	0,13	0,20	0,27	0,33	0,40	0,47	0,53
0,6	0,17	0,33	0,50	0,67	0,83	1,00	1,17	1,33
0,4	0,25	0,50	0,75	1,00	1,25	1,50	1,75	2,00
0,2	0,50	1,00	1,50	2,00	2,50	3,00	3,50	4,00
0,1	1,00	2,00	3,00	4,00	5,00	6,00	7,00	8,00
0,08	1,25	2,50	3,75	5,00	6,25	7,50	8,75	10,00
0,04	2,50	5,00	7,50	10,00	12,50	15,00	17,50	20,00

Table 5.10.Values of k given c and r

What is the analysis that can be extracted from these three different tables?

- With a relative flexibility r>1,5 no building can be considered to be stiffened by the roof.
- For r < 0,1 the frame has to be flexible, (with k>1 mm/KN); for 0,1< r <0,6 the frame can be rigid if the diaphragm is stiff enough c < 0,5 mm/KN
- The symmetric approach is effective with much more stiffener frames as the non-symmetric approach. The length of the building that can be stiffened with the diaphragm is much higher in a symmetric case as in a non-symmetric one.
- For symmetric buildings with more than 7 bays, flexible portals and stiffen diaphragms are required. For shorter buildings the portals can be rigid.
- For a non-symmetric building with more than 4 bays flexible portals are required, and it would be difficult for rigid frames to be stiffened.

6.1.3.2. <u>Bear resistance of the shear cells</u>

Now that the minimum relative flexibilities are placed into the table, the next step is to check which is the maximum allowed horizontal forces on the side of the building, so that the diaphragm can bear it without collapsing.

It is assumed that the flexibility of the shear cell in the whole length of the building is the same. That it is true either if the whole width of the building is a diaphragm (fig 5.9) or if there are two different lines of diaphragms (fig 5.8), as the flexibility of each of them is the same.

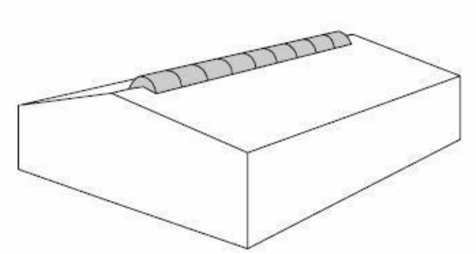

Fig 5.8. Building with a non-continuous diaphragm over the ridge ($b_{diap}=b_{building}/2$) Extracted from [35]

When the wind is acting on the long side, if there is one diaphragm, it is considered that the diaphragm acts as explained. If there are two different and equal diagrams, they function separately and they share the action due to wind on the long side.

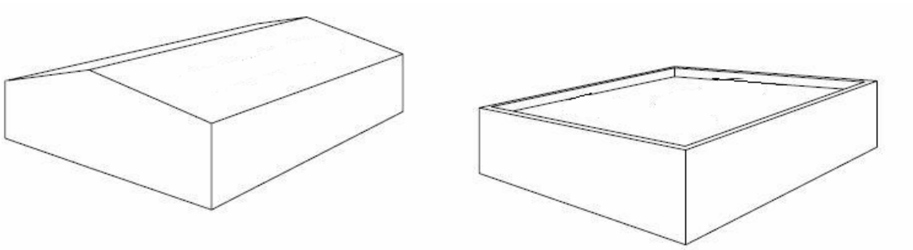

Fig 5.9. Building with a continuous diaphragm over the ridge ($b_{diap}=b_{building}$) Modified from [35]

Let's examine then the maximal shear force. In the table 5.11 there is the total force per frame H , on the side of the building. The values of shear force showed are the ones in the most loaded shear cell (next to the stiffened frame). The diaphragm distribution is the one of two diaphragms (fig 5.8), it is the maximum shear force in each of the two diaphragms in the building. If the building would have a whole diaphragm (fig 5.9) the shear force would be the double as showed in the table. The complete list can be found in annex III, Let's examine one of the parts, the one in which the total ultimate load due to wind is H=30 KN

	Number of bays										
r	3	4	5	6	7	8	9	10	11	12	H(KN)
0,02	15	22	29	35	41	47	52	57	62	66	30
0,04	14	21	27	33	38	42	46	50	53	55	30
0,08	14	20	25	29	33	36	38	40	41	43	30
0,1	14	19	24	28	31	33	35	37	38	38	30
0,2	13	17	20	22	24	25	26	26	26	27	30
0,4	11	14	15	16	17	17	17	17	17	17	30
0,8	8	10	10	11	11	11	11	11	11	11	30
1	8	9	9	9	9	9	9	9	9	9	30
1,5	6	7	7	7	7	7	7	7	7	7	30

Table 5.11.Values of S in half diaphragm given r, n and H

Let's analyse this depending on the building bays.

- From 10 to 12 bays: The shear force (S) ranges from 40-43 KN that is over the allowed shear loads, it was seen on the previous chapter, the limit was 40 KN
- From 6-9 bays: 20-30 KN of ultimate shear effort are required. This shear effort can be easily reached with a direct diaphragm. This would be difficult to reach with a non-direct system.
- 5 or less: Less than 20 KN are required, what can be easily reached.

The following analysis can be performed in the whole annex III table III.2 .Obtaining the following approximate results:

Number of bays	Direct (KN)	Indirect (KN)
3	>60	>60
4	>60	>60
5	60	55
6	50	40
7	50	35
8	35	25
9	30	25
10	25	20
11	25	20
12	25	20

Table 5.12.Maximum horizontal external load in case there is direct or indirect fastening of the diaphragm

Short buildings would allow a force superior of 60 KN, then the 5-7 bay buildings with acceptable force round 50 KN, for longer buildings the value decreases to a constant value of 25 KN

The same analysis can be performed for a non-symmetric building. Let's examine one of the parts, the one in which the total load is H= 20 KN (extracted from annex IV)

	Number of bays						
r	3	4	5	6	7	8	H(KN)
0,04	23	31	37	43	48	51	20
0,06	21	27	32	36	38	40	20
0,08	20	25	28	31	33	34	20
0,1	19	23	26	27	28	29	20
0,2	18	21	23	25	26	26	20
0,4	14	16	17	17	17	18	20
0,8	10	11	11	11	11	12	20
1	6	6	6	6	6	6	20

Table 5.13.Values of S in half diaphragm given r, n and H

Let's analyse it depending on the building bays.

- 8 bays: The shear efforts range from 40-43 KN that is over the shear loads allowed for the studied diaphragm, which is 40 KN

- 6-7 bays: 20-30 KN of ultimate shear effort are required. This shear effort can be easily reached with a direct diaphragm. This would be difficult to reach with a non-direct system.
- 5 or less: Less than 20 KN are required, what can be easily reached.

The following analysis can be performed in the table IV.2 in annex IV in total obtaining the following approximate results:

Number of bays	Direct(KN)	Indirect(KN)
3	55	40
4	35	25
5	30	20
6	25	15
7	20	15
8	15	10

Table 5.14 Maximum horizontal external load in case there is direct or indirect fastening of the diaphragm

Short buildings would allow forces up to 30 KN, for longer buildings the value decreases quickly to the 15-20 KN

6.1.3.3. Deformation pattern on the buildings

6.1.3.3.1. Symmetric building

The frames deformation of a building would look like this:

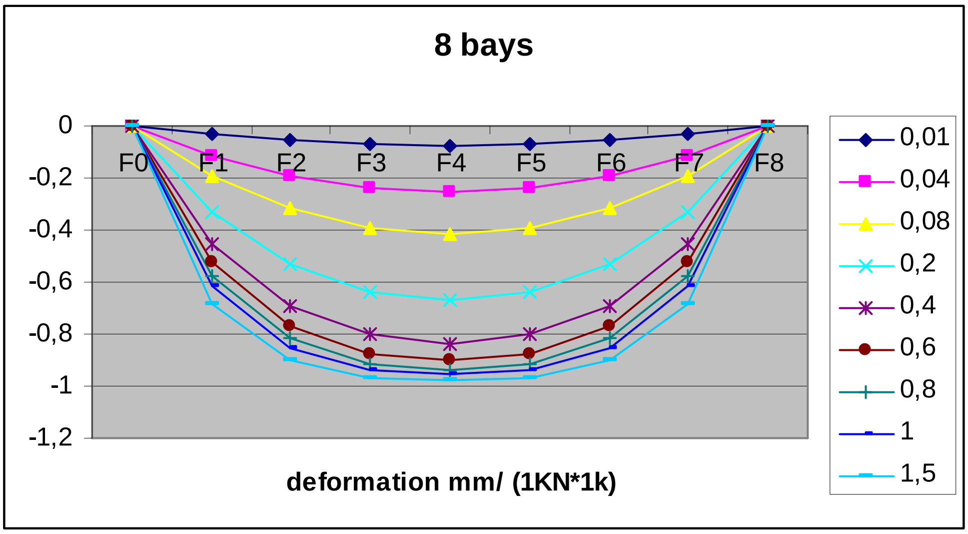

Fig 5.10. Deformation of an 8 bay building

In the graphic it can be observed that there are two differentiate parts, which can be found in all the other buildings lenghts (see graphics in Annex III)

- In the example there are the values from 0,4< r <1,5. For this case, it can be seen that the diaphragm it is not effective, the deformation in the central bays shows that the major part of the deformation is due to the portals that absorb all the external force. On the bays closer to the extremes, the diaphragm starts to be effective though.

- The second set of graphics are the ones 0,4<r<0,01 where as the flexibility of the diaphragm respect to the frames is lower (let's remember that in the graphic k=1mm/KN and H=1 KN) then the diaphragm turns to be more effective reducing the deformation. The maximum deformation is always in the middle point of the building

6.1.3.3.2. Non symmetric building

The frames deformation of a non-symmetric looks like :

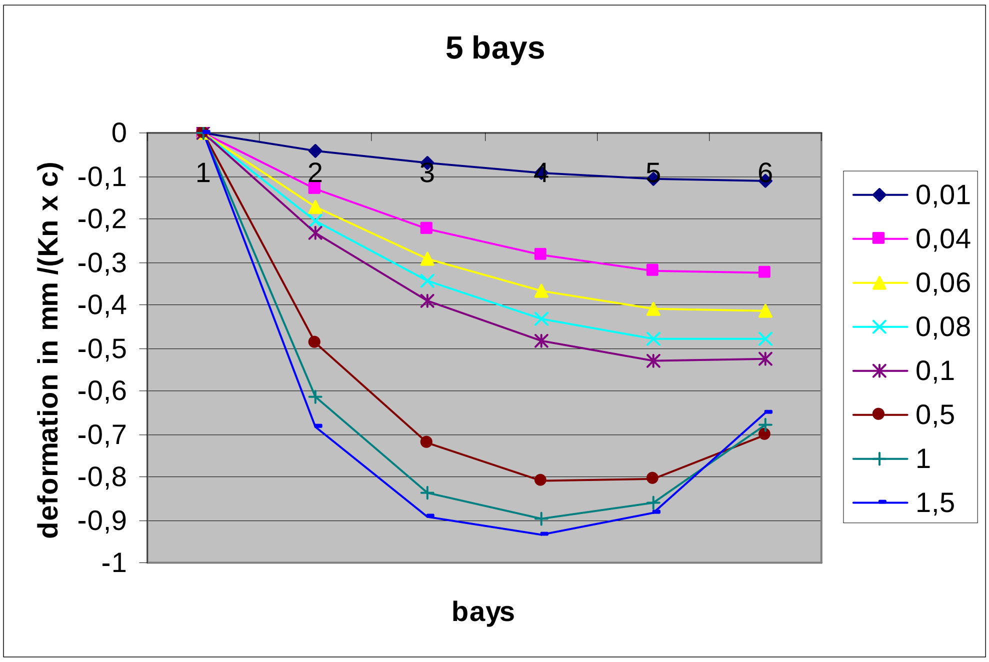

Fig 5.11. Deformation of a non-symmetric for different r 5 bay building

In the graphic it can be observed that there are two differentiate parts that can be found in all the other buildings lengths (see other graphics in Annex IV)

- In the example there are the values from 0,1<r<1,5. For this case, it can be seen that the diaphragm is not effective, the deformation in the 3-5 bays shows that the major part of the deformation is due to the portals that absorb everything, On the bays closer to the extremes, the diaphragm is a bit effective though.

- The second set of graphics is the ones 0,01<r<0,1 where there is a progressive deformation, until the deformation is stabilised towards the last bays.

6.1.3.4. Maximum absolute deformation

6.1.3.4.1. Symmetric

On the previous figures the deformation patterns of the two typologies have been showed. How about the absolute value of the deformation? To compare it with the values of ultimate shear force, the assumption done is the following:

- The force H acting on the frames is an ELU force (multiplied by a security factor 1.5). H_{tot}^* is defined as $H_{tot}^*=H/1,5$
- Each diaphragm absorbs the half of the H_{tot}^*; then $H^*=H/3$

The values showed are the ones of maximum deformation (central frame) due to an external force H per frame on the side of the building. As an example for H=40KN → H^*=13.3KN (per diaphragm). If there would be a whole diaphragm the values would be the double, as the external force would be the double:

r	c	k	3	4	5	6	7	8	9	10	11	12
0,08	0,1	1,25	-1,2	-2,3	-3,3	-4,6	-5,6	-6,9	-7,9	-8,7	-9,8	-10,7
0,08	0,2	2,5	-2,5	-4,7	-6,6	-9,2	-11,3	-13,8	-15,8	-17,4	-19,7	-21,5
0,08	0,3	3,75	-3,7	-7,0	-9,9	-13,8	-16,9	-20,7	-23,7	-26,0	-29,5	-32,2
0,08	0,4	5	-4,9	-9,4	-13,2	-18,3	-22,6	-27,6	-31,6	-34,7	-39,3	-42,9
0,08	0,5	6,25	-6,2	-11,7	-16,5	-22,9	-28,2	-34,5	-39,5	-43,4	-49,2	-53,6
0,08	0,6	7,5	-7,4	-14,0	-19,8	-27,5	-33,9	-41,4	-47,4	-52,1	-59,0	-64,4
0,08	0,7	8,75	-8,6	-16,4	-23,1	-32,1	-39,5	-48,3	-55,3	-60,7	-68,8	-75,1
0,08	0,8	10	-9,9	-18,7	-26,4	-36,7	-45,2	-55,2	-63,1	-69,4	-78,7	-85,8

Table 5.15. Deformation in mm if a external force H=40 KN per frame acts on the side of the building

6.1.3.4.2. Non symmetric

For the non-symmetric case the assumption is the following:

- The force H acting on the frames is an ELU force (multiplied by a security factor 1.5). H^* is defined as $H_{tot}^*=H/1,5$
- Each diaphragm absorbs the half of the H_{tot}^*; then $H^*=H/3$

The values showed would be the ones of maximum deformation (different frames, see schemas to see which one it is) for an external force H on each frame of the building, except the last frame where there is H/2. As an example for H=40KN → H^*=13.3KN (per diaphragm):

r	c	k	3	4	5	6	7	8
0,1	0,1	1,0	-4,2	-6,2	-8,0	-9,6	-10,8	-11,9
0,1	0,2	2,0	-8,3	-12,4	-15,9	-19,1	-21,7	-23,7
0,1	0,3	3,0	-12,5	-18,5	-23,9	-28,7	-32,5	-35,6
0,1	0,4	4,0	-16,7	-24,7	-31,9	-38,2	-43,4	-47,4
0,1	0,5	5,0	-20,9	-30,9	-39,9	-47,8	-54,2	-59,3
0,1	0,6	6,0	-25,0	-37,1	-47,8	-57,3	-65,1	-71,1
0,1	0,7	7,0	-29,2	-43,3	-55,8	-66,9	-75,9	-83,0
0,1	0,8	8,0	-33,4	-49,4	-63,8	-76,4	-86,7	-94,8

Table 5.16. Deformation in mm if a external force H=40 KN per frame acts on the side of the building

6.1.3.4.3. Shear force and maximum deformation

The complete values for the case H=1 KN can be found in annex III and IV. Which are the allowed values on a building? According to the height the following maximum deformations in the frames can be considered:

Height (mm)	Maximum deviation: h/100	Maximum deviation: h/200
7000	70 mm	35 mm
10000	70 mm	50 mm
13000	70 mm	65 mm

Table 5.17. Deformation in mm of the central frame according to the height

Which are then the external ELU forces that can be allowed in order to limit the lateral displacement of the frames in the buildings with the conditions mentioned in the previous chapters? In the following tables with be showed with an *ok* if the deformation is lower than 35 mm. And with a value of maximum r it does not fulfil the maximum imposed by the previous chapter's limitations.

H(KN)	3	4	5	6	7	8	9	10	11	12
60	Ok	Ok	Ok							
55	Ok	Ok	Ok							
50	Ok	Ok	Ok	Ok	Ok					
45	Ok	Ok	Ok	Ok	Ok					
40	Ok	Ok	Ok	Ok	Ok					
35	Ok	Ok	Ok	Ok	Ok	Ok				
30	Ok	Ok	Ok	Ok	Ok	Ok	Ok			
25	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	0,04	0,08
20	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok
15	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok
10	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok

Table 5.18. Check if def< 35 mm with the limitant conditions: Ok mean deformation is lower, and a r value indicates the maximum r necessary for the given H .Symmetric building

H	3	4	5	6	7	8
55	Ok					
50	Ok					
45	Ok					
40	Ok					
35	Ok					
30	Ok	Ok	Ok			
25	Ok	Ok	Ok	Ok		
20	Ok	Ok	Ok	Ok	0,04	
15	Ok	Ok	Ok	Ok	Ok	0,04
10	Ok	Ok	Ok	Ok	Ok	Ok

Table 5.19. Check if def< 35 mm with the limiting conditions: Ok mean deformation is lower, and an r value indicates the maximum r necessary for the given H .Non Symmetric building

It can be seen that the deformation is not a limiting parameter, and that if the building fulfils the ELU in the shear cells will fulfil the deformation conditions in ELS.

6.1.3.5. <u>Lateral actions due to wind</u>

The question now is to associate this ELU wind lateral force with the lateral intensity that can be allowed in design. The hypotheses to change from the *q* on the building to the H in the building are the following:

The wind acts on the side of the building as it can be seen in the figure 5.12

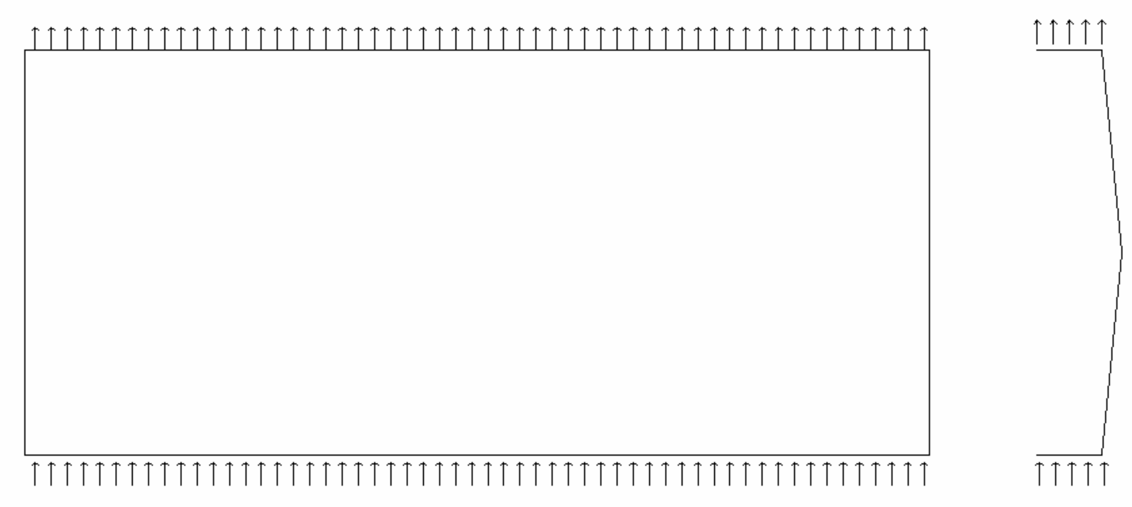

Fig 5.12Wind acting on the long sides of the building

A coefficient of 0.8 on the wind side and a value of 0.4 on the other side are supposed.

The following assumptions will be made in order to convert this to the "force per frame" H_i

- The force H on top of the column can be calculated as indicated in the frame (fig 5.13)

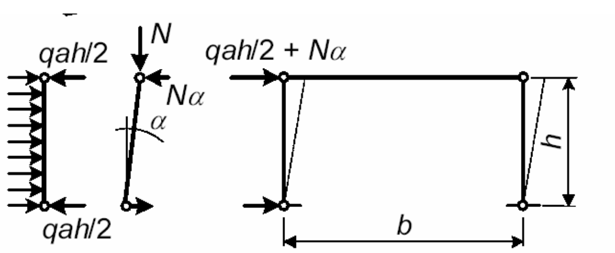

Fig 5.13Equivalent H due to q in a nominally pinned frame. Extracted from [6]

$$H = \frac{q \cdot a \cdot h}{2} + \left(\frac{q_{\sup} \cdot b_{roof}}{2} \right) \cdot \alpha \approx \frac{q \cdot a \cdot h}{2}$$

$$\alpha \approx 0.005$$

- The two forces on one side and the other can be considered as only one force placed in one of the sides of the building.
- The c parameters are 0,8 on one of the sides and 0,4 on the other

With those assumptions the following tables can be created

If height is 7 meters

	Portal Spacing (mm)	Wind load (KN/m2)	Wind side c	Other wall c	V wind side (KN)	V other side (KN)	V total (KN)
D1L	6000	0,75	0,8	0,4	12,6	6,3	18,9
D2L	7000	0,75	0,8	0,4	14,7	7,35	22,05
D3L	8000	0,75	0,8	0,4	16,8	8,4	25,2
D4L	9000	0,75	0,8	0,4	18,9	9,45	28,35
D1M	6000	1,25	0,8	0,4	21	10,5	31,5
D2M	7000	1,25	0,8	0,4	24,5	12,25	36,75
D3M	8000	1,25	0,8	0,4	28	14	42
D4M	9000	1,25	0,8	0,4	31,5	15,75	47,25
D1S	6000	2	0,8	0,4	33,6	16,8	50,4
D2S	7000	2	0,8	0,4	39,2	19,6	58,8
D3S	8000	2	0,8	0,4	44,8	22,4	67,2
D4S	9000	2	0,8	0,4	50,4	25,2	75,6

Table 5.20. Equivalent H due to a wind load q. Height 7 meters

If height is 10 meters

	Portal Spacing (mm)	Wind load (KN/m2)	Wind side c	Other wall c	V wind side (KN)	V other side (KN)	V total (KN)
D1L	6000	0,75	0,8	0,4	18	9	27
D2L	7000	0,75	0,8	0,4	21	10,5	31,5
D3L	8000	0,75	0,8	0,4	24	12	36
D4L	9000	0,75	0,8	0,4	27	13,5	40,5
D1M	6000	1,25	0,8	0,4	30	15	45
D2M	7000	1,25	0,8	0,4	35	17,5	52,5
D3M	8000	1,25	0,8	0,4	40	20	60
D4M	9000	1,25	0,8	0,4	45	22,5	67,5
D1S	6000	2	0,8	0,4	48	24	72
D2S	7000	2	0,8	0,4	56	28	84
D3S	8000	2	0,8	0,4	64	32	96
D4S	9000	2	0,8	0,4	72	36	108

Table 5.21. Equivalent H due to a wind load q. Height 10 meters

Ic can be seen that the range of uses will be approximately from loads up to 1.25 KN/m^2 of surface. Depending on the height of the building.

6.2. Design considerations

6.2.1. Building classification

Building class III: The steel structure should be designed to carry the total loads, provided that the sheeting possesses sufficient deformation capacity, this will follow the movements, if the roof designed is too stiff, then it will absorb part of the horizontal force, for which it is not planned.

Building class II: A second function is added to the previous case. The metal sheet has to grant that it is able to brace the purlins, (the compression section of the purlins is stiffened).

Building class I: The roof is used as restrain of the whole building and also used as stressed skin to absorb part of the lateral forces.

- Hinged structure:

  The task of the bracing has been replaced by the action of the diaphragm of the sheeting. It should be proved that the diaphragm can fulfil this function. Temporary bracing will be probably needed during erection. Usually does not lead to a saving cost of the wind bracing.

- Rigid frames:

  - The external load will be partly taken by the structure and partly by the sheeting. The proportion depends on the stiffness of both elements.
  - If the frame is too rigid, the frames will only absorb the external load. Which a lead to heavier sections, there has to be enough deformation possibility of the sheeting if the diaphragm effect is considered.
  - Used as bracing against lateral torsional buckling. Eurocode 3.1 part 3 (independent from the diaphragm effect)

6.2.2. Calculations

In order to calculate the diaphragm design the following points have to be observed:

1. Design the sheeting (and end supporting) members for its primary purpose as cladding

2. Determine the in-plane loads on the sheeting and the shear force and maximum bending moment in the deep plate girder
3. Calculate the shear flexibility and the ultimate shear strength, of a shear paned
4. Check that:
   - The combined stresses in the supporting member are acceptable
   - The ultimate shear strength of the shear panel is adequate
   - The in-plane deflection of the diaphragm is acceptable.

6.2.3. Recommendations

Some points have not been treated in this dissertation, but have to be observed when designing diaphragms:

1. *Edge members should be designed to absorb axial forces*: If the stressed skin design is provided, the sheet will absorb shear loads, but the purlins will also absorb axial forces, and those forces have to be considered in design.
2. *Transmission to the diaphragm forces to the foundations*: The basis of the diaphragm is that there are stiffen portals able to transmit the shear forces from the roof to the foundations.
3. *It can be also used to transmit forces of small cranes* (load in the diaphragm less than 30% of failure load of the diaphragm). The procedures to calculate the shear efforts in this chapter can be also used in case there is just one punctual load on the building due to a crane.

4. *Specify clearly in the building that the diaphragm have been used*: as the stability of the building relies on the roof, and openings reduce the strength and increases the flexibility, it should be clearly marked that the design has been used.
5. *Openings can be neglected if less than 3%*, every thing that allows a big disruption should be treated separately: In the suppositions of the dissertation the buildings' roof where divided in both sides. Openings can reduce the capacities as mentioned: in the fig 5.14 some examples of non allowed lightening options are showed

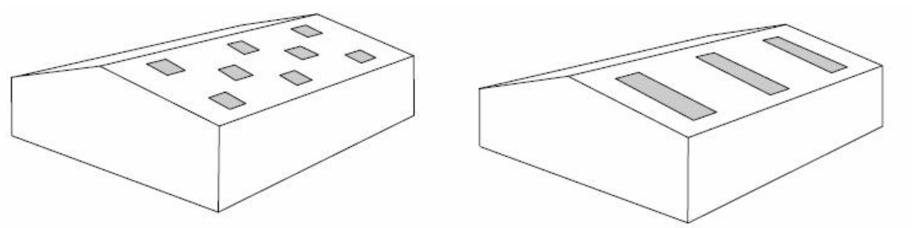

Fig 5.14. Building with a lighting distribution that either reduce or not allow the use of diaphragm effect. Extracted from [35]