

Application of knee-bracing system on high-rise buildings

by

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Diploma Civil Engineering
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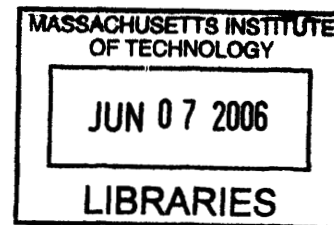
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Massachusetts Institute of Technology

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Abstract

The need and ambition of humanity to go higher and higher is something that is amplified as time evolves. It is the same need that leads engineers to push their structures to higher limits.

However, when engineers design higher structures their knowledge and their abilities are challenged. In that sense, during the design process of a high-rise building all the strange phenomenon of the behavior of the structure must be considered.

A very significant factor that affects the limits of today's high-rise construction is the wind loading. Bracing the building in a clever and more efficient way was always a difficult task for designers and engineers. This thesis deals with a bracing system called the knee-bracing system. The application of knee-bracing system for high-rise buildings is not yet fully determined and this study will try to describe the problem and provide some solutions. Knee-bracing will be checked and the possibility of providing the required results using the minimum amount of material and giving the maximum space for use from the residents or workers of the building will be examined.

Several different cases of loading and knee-bracing systems are considered and an optimization for the design of such systems is described. The last part of the document describes the idea of adaptive stiffness, something new for high-rise buildings. Reinforcing these ideas, solutions for the construction of these bracings are also provided.

Thesis Supervisor: Jerome J. Connor

Title: Professor of Civil and Environmental Engineering

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I am deeply grateful to my supervisor Jerome J. Connor. For me, his advice on everything was always an invaluable guide for all my choices. I consider myself very fortunate having met and worked with a person like him. His example is always going to be a pharos for my life.

My family Thomas, Efi and Despina is one of the main reasons why I am in this place. Their upbringing makes me very confident and their support throughout my whole life gives me strength and courage to continue improving myself.

My friends and especially Dimitrios Iliadelis who was more like a brother to me during our year in MIT helped me go through a lot of difficult situations. Petros Englezos and Dimitrios Andritsos, my closests, have been an incredible, everyday support. I am sure and happy that this deep friendship will keep strengthening.

I am also very indebted to my friends from the program who made this year special. I will always look back upon my time at MIT with great memories with Jeremiah, George, Despina, Aurelie, Takis, Naeem, Alessia, Roberta, Violeta, Milan, Juan, Lauren, Ricardo, Adrian, Nick, Claire, Samar, Maria, Rosella, Sinan, Paul.

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1) Introduction

The race towards new heights has not been without challenges. After the invention of the elevators by E.G. Otis and the use of structural steel as the material for the structure, skyscrapers have continued to go higher and higher where they started to face strange wind effects and very high wind loading. Unfortunately, these advances in height were often combined with increase in flexibility and lack of stiffness or damping, increasing their vulnerability to the actions of the wind.

Lateral loading was a loading condition that structural engineers considered relatively late. It was the damaging force of nature in the form of wind or earthquake that made the engineers think that the lateral load could indeed produce very significant problems in the structure.

Therefore, the idea that permitted structural engineers to design high rise buildings was the bracing of the building with several different ways. The use of the stiffest geometrical form, which is the triangle, was the dominant idea. Triangular form can be integrated in the geometry of the building with infinite ways.

Figure 1 illustrates different schemes of integrating the triangular shape inside a square.

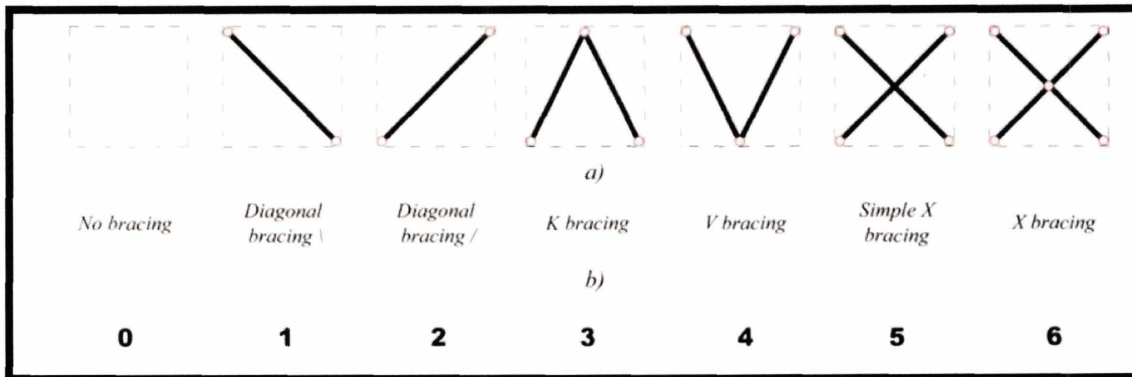


Figure 1. Image from “Parameterized versus Generative representations in structural design : an empirical comparison.”

The selection of each of the above shown geometries depends of course on the loading problem, the overall geometry, the desired result, the availability of the material and on several other reasons. All the above bracings, when used for wind loadings, are called wind bracings.

Wind bracing provides the lateral stability of the building. It consists of cross bracing located in roof and sidewalls, in one or more bays depending on the loadings and the height of the building.

Although the bracings shown above have been mostly used throughout the history of high rise buildings, they have a common disadvantage. There is no possibility of having openings in the square, therefore the option of having windows, doors or any kind of opening is not

feasible anymore. That is why engineers came up with a scheme of bracing, the so-called knee bracing.

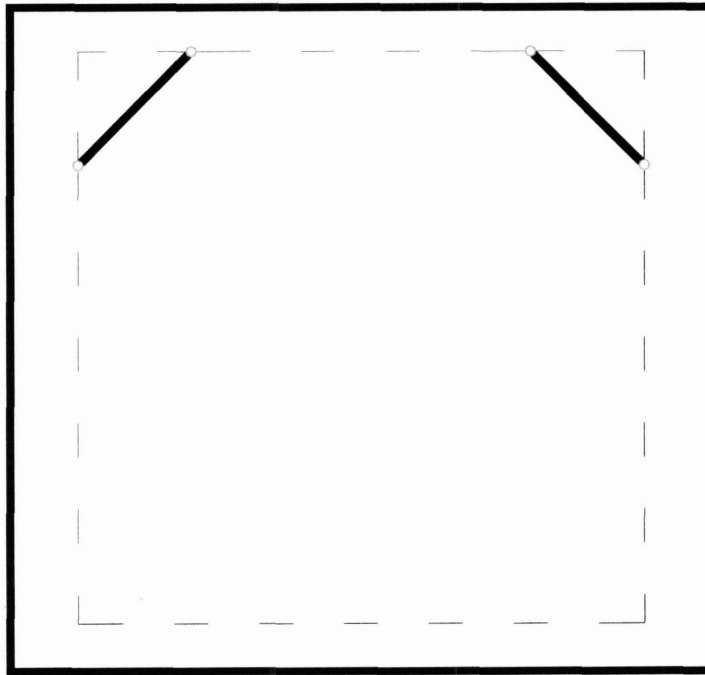



Figure 2. *Knee bracing*

The knee bracing provides the same stiffness as all the above bracings, without having the disadvantage of blocking the space inside the square of beams and columns. As shown in figure 2, the only area that is blocked by the bracing elements is the area around the connection of columns and beams which is usually already hidden by false ceiling because of mechanical purposes.

The structural result is to stiffen significantly the column-beam connection whose flexibility usually produces the overall flexibility of the building. It must be noted that the result of the knee bracing depends on whether the column-beam connection is a moment connection or a braced connection.

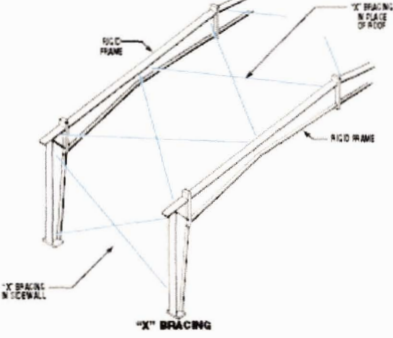
This idea was applied in many cases but for low loading conditions and only in low-rise buildings. In figure 3 below one can see the application of knee bracing by Ceco Building Systems, on a one story building.



Ceco Building Systems

BRACING

ROD/CABLE "X" BRACING



"X" BRACING

Longitudinal wind/seismic forces, transferred from the top of the end wall columns into the roof system, are then transmitted to the eave lines by either diaphragm action of the roof sheets, when applicable, and/or by trusses in the plane of the roof. Trusses are composed of rigid frame rafters for top and bottom chords, purlins for compression struts, and "X" rods for tension diagonals. From the eave lines, the longitudinal forces are transferred to the foundation either by diaphragm action of the wall sheathing and/or by "X" rod bracing in the side wall.

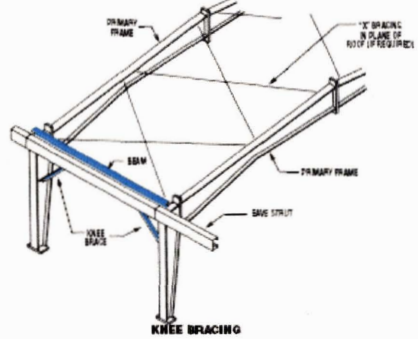
At Ceco's option or at customer's request, cables may be substituted for rods.

Diaphragm action and rod bracing are used in combination to resist longitudinal wind/seismic loads. However, if cables are used in lieu of rods, they will be designed to carry the entire longitudinal wind/seismic load without using any diaphragm action from the sheathing.

When "X" bracing is used to resist longitudinal forces, all braced bays share equally in transmitting these forces to the foundation. Each braced bay is designed to take its proportional share of the wind/seismic loads.

KNEE BRACING

The standard procedure to transfer longitudinal wind/seismic forces from the eave line to the foundation is to utilize diaphragm action of the wall sheathing and/or provide "X" rod (or cable bracing) in the side walls. Should building requirements preclude the use of the standard procedure, Ceco Buildings provide three alternate methods of bracing, one of which is a Knee Brace system; however, this system is not available for bracing buildings that have Flush Wall Primary Frames.



KNEE BRACING

This system consists of a knee brace and beam arrangement which, when connected to Primary Frame columns, provides economical bracing to resist moderate eave forces. The diagonal knee braces, located at each end of the beam, consist of double angle members which will intersect the column 4'0" below the beam member on buildings up to 14'0" eave height and 5'0" below on buildings with eave heights greater than 14'0". The beam is an extra member added, parallel to the eave strut, in knee braced bays.

The number of knee braced bays provided in any side wall will be as required by design. The structural design of the knee brace is based on the beam being pinned at both ends, with the Primary Frame column designed for weak axis bending.

Knee Bracing is designed to carry the entire longitudinal wind/seismic eave line load without sharing any of this load with any other bracing system.

16

Figure 3. Application of knee bracing by Ceco Building Systems. Image taken from Ceco website

According to their website:

"KNEE BRACING: This system consists of a knee brace and beam arrangement which, when connected to Primary Frame columns, provides economical bracing to resist moderate eave forces. The diagonal knee braces, located at each end of the beam, consist of double angle members which will intersect the column 4'0" below the beam member on buildings up to 14'0" eave height and 5'0" below on buildings with eave heights greater than 14'0".

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Knee Bracing is designed to carry the entire longitudinal wind/seismic eave line load without

sharing any of this load with any other bracing system.”

The application of Ceko deals with small buildings and low wind loads. The effect of the application of knee bracing in high-rise buildings still remains not widely applied. This thesis will try to investigate what are the effects of using knee bracing in high-rise buildings.

Chapter 2. Stiffness problem

2.1 Bracing on a 20 floor building – description of the building

- Geometry

The material in this section demonstrates that small elements located close to the connections of beams and columns (knee bracing), provide adequate lateral stiffness for a high rise building structure. Based on the results of this analysis, the rest of the chapter focuses on how to distribute these diagonals throughout the height of the building to optimize their performance. A lot of factors were taken into account such as different sections for the diagonals, different loading conditions, different locations and others.

For the purpose of the analysis a 20 floor steel frame building was selected. The geometry of the building is shown in the following images.

A simple building was considered to be more appropriate for the analysis, since a complicated geometry would be comparatively intensive. The software used for all the analysis and simulation is SAP 2000. Furthermore, the effect on a 20-story building would provide insight on the behavior of taller buildings.

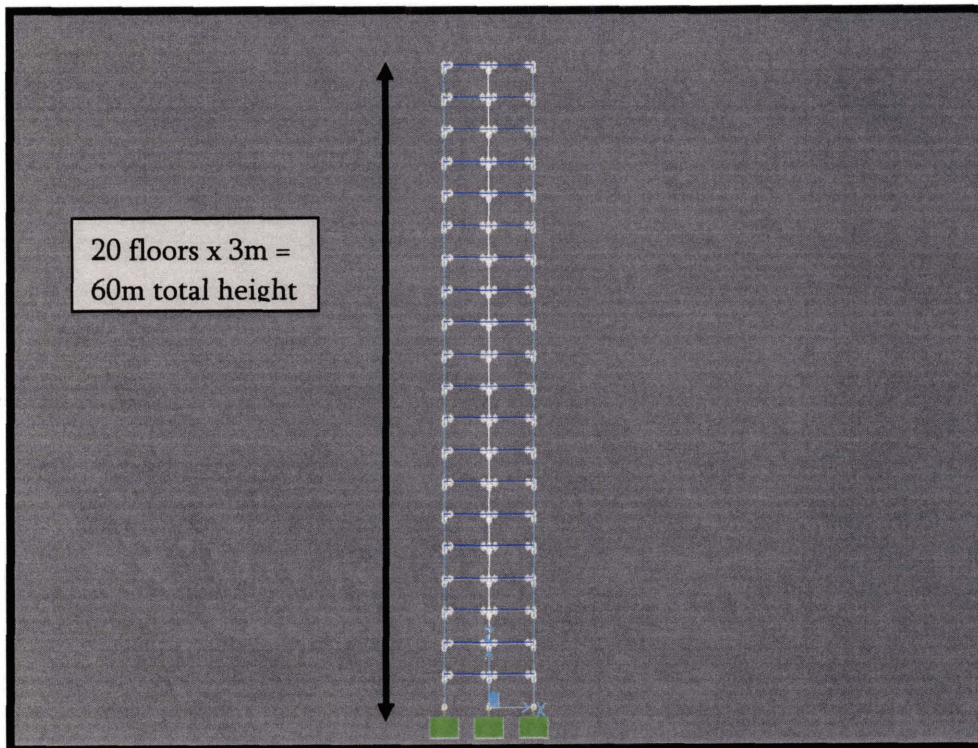


Figure 4. *Front view of the model in SAP*

The plan view of the building is shown in figure 5.

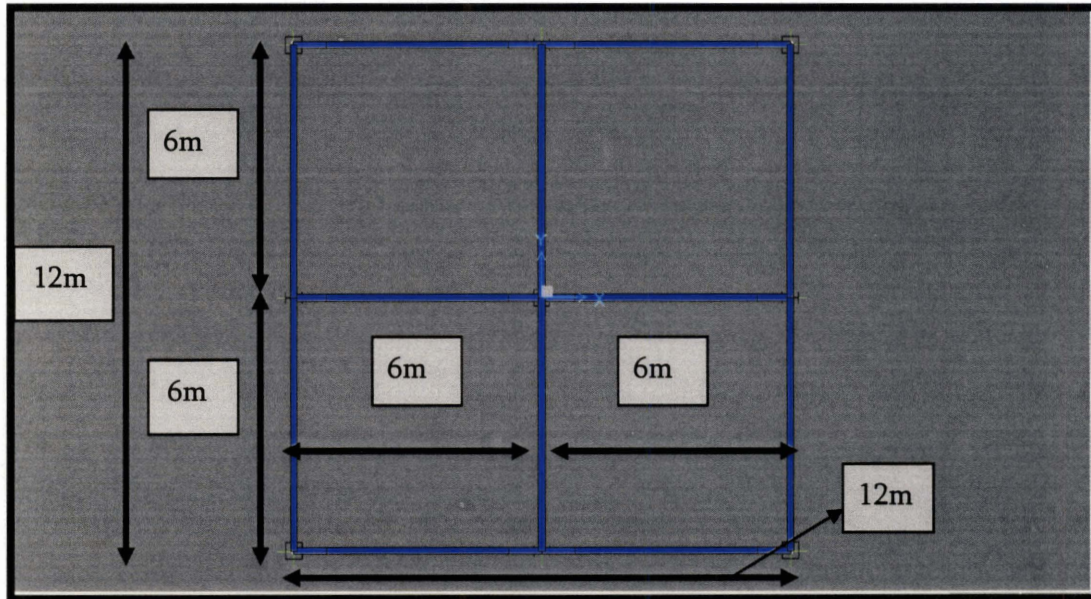


Figure 5. Plan view of the model in SAP

The building is a steel frame building which is supported by 4 corner columns which are box sections, a middle column which is also a box section and another 4 columns which are I sections. The cross section of the building is a square and this was selected in order to have symmetry in the two different wind directions. For that reason only one of the wind directions was analyzed and not both of them. The dimensions of the elements as shown in figure 5 result in an aspect of :

$$\boxed{\text{base/height} = 12/60 = 1/5}$$

which is typical of non-slender buildings.

Some dimensions were input for the elements themselves. The beams are I sections. The dimensions of these sections, in millimeters are shown in the following figures:

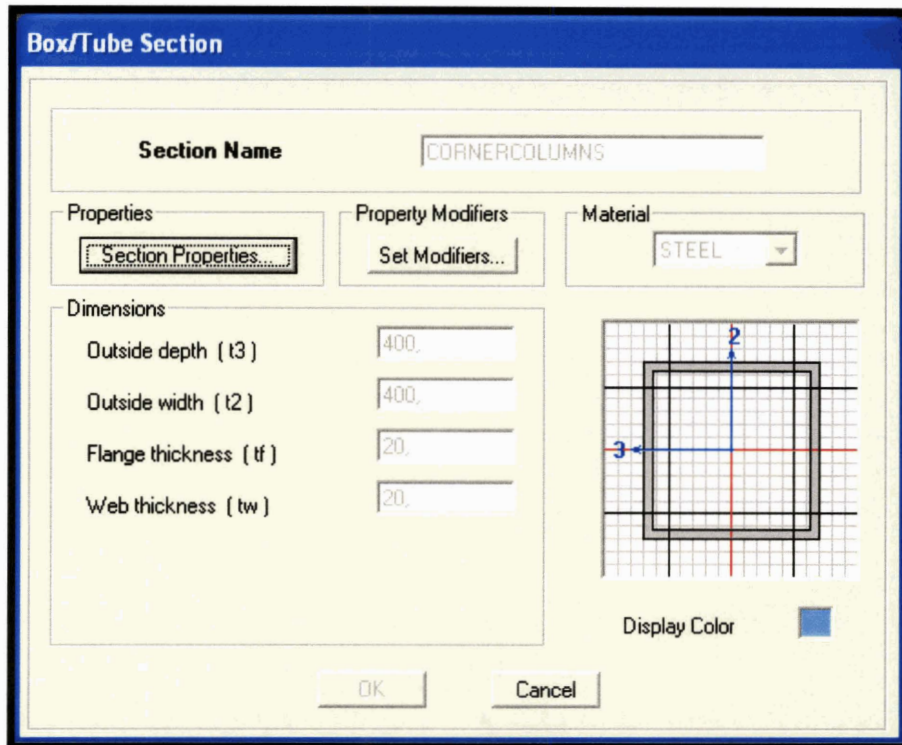


Figure 6. Corner columns section as input in SAP

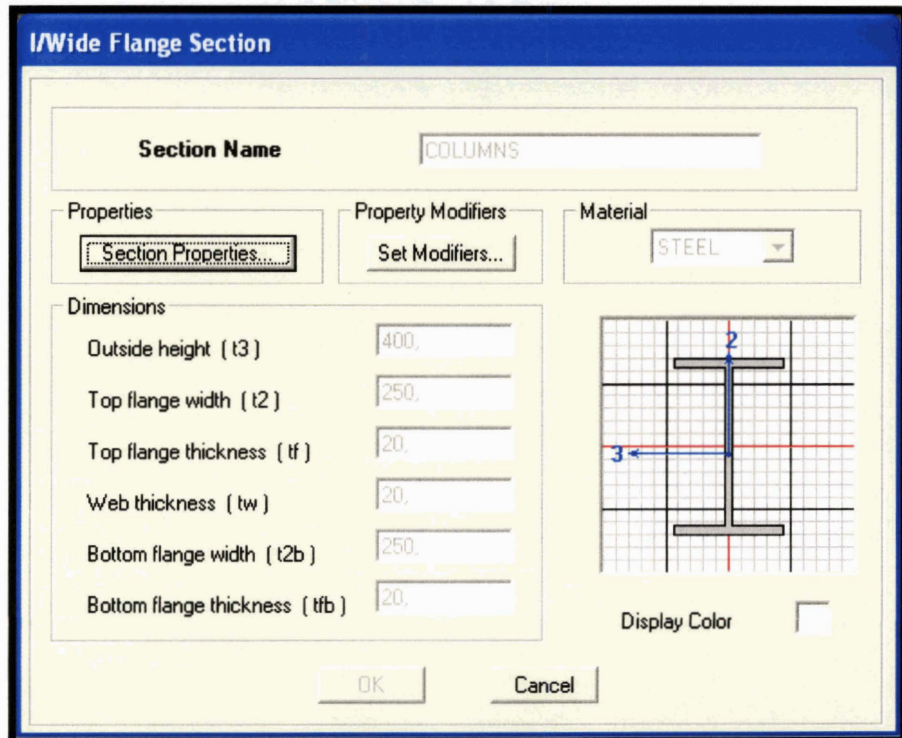


Figure 7. Columns section as input in SAP

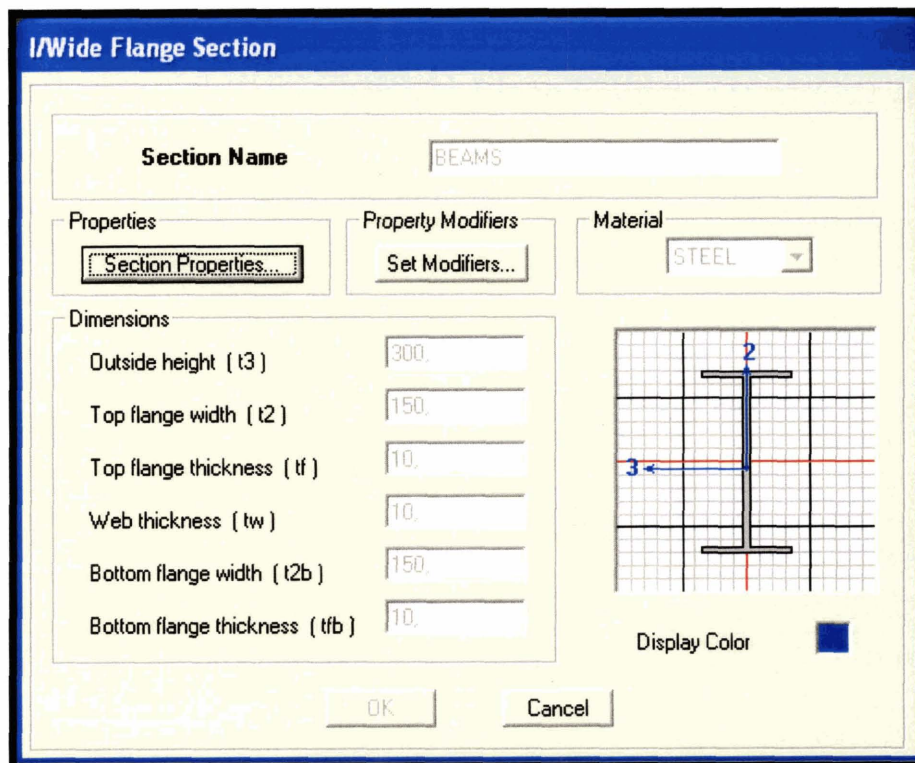


Figure 8. Beams section as input in SAP

The sections of the columns were considered to be uniform throughout the height of the building. In a more realistic case it is possible to have changes in the different heights of the building. Nevertheless, for a 20 story building, the use of uniform columns sections is considered to be realistic.

- Loads

The critical loading that the structure will be evaluated for is the wind loading. Although there are a lot of different factors that affect the application of the load in a dynamic way, the load for this first analysis will be considered to be static. In case of a more time flexible schedule the effect of making the knee braces also working as dampers would be analyzed.

It is proven that the wind load on a structure follows a pattern which can be shown in figure 9 below. However, for the purposes of this thesis a linear increase of the wind load throughout the height of the building was considered to be accurate enough.

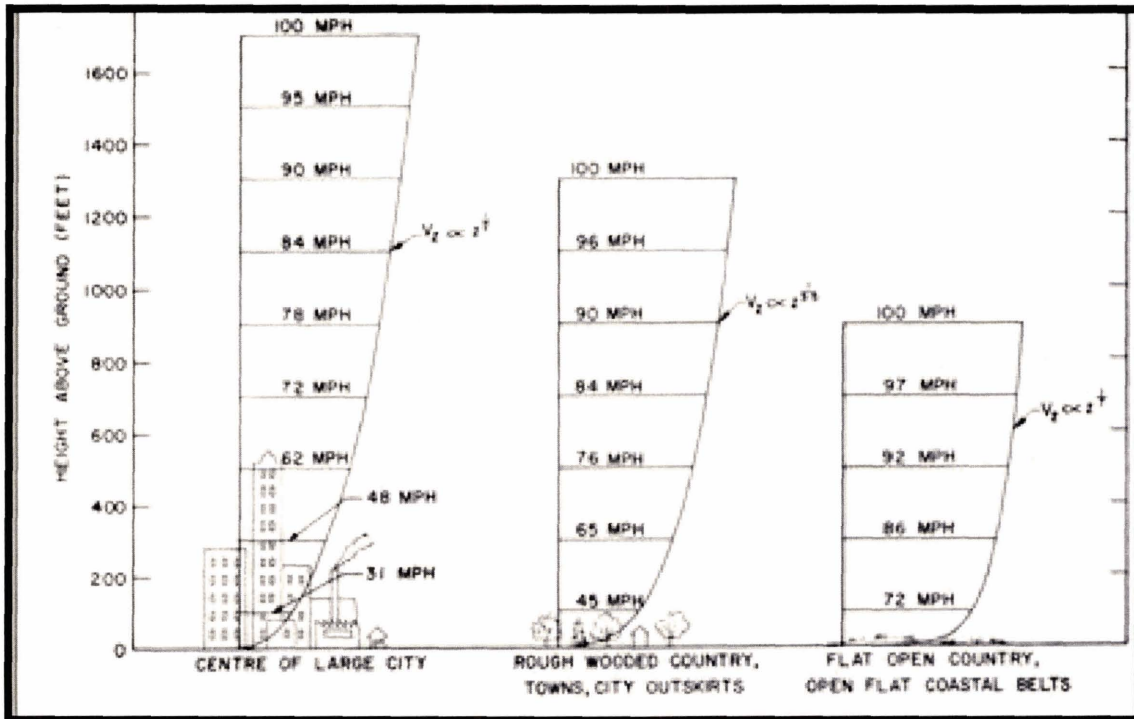


Figure 9. Ideal wind load distribution. Image taken from <http://www.geocities.com/lpittack/cae614proj.html>

The wind load that was considered for the model is shown in figure 10 on the next page. This wind load is not scaled or calculated in detail, so using the load combination factor can be changed so that real values can be reached.

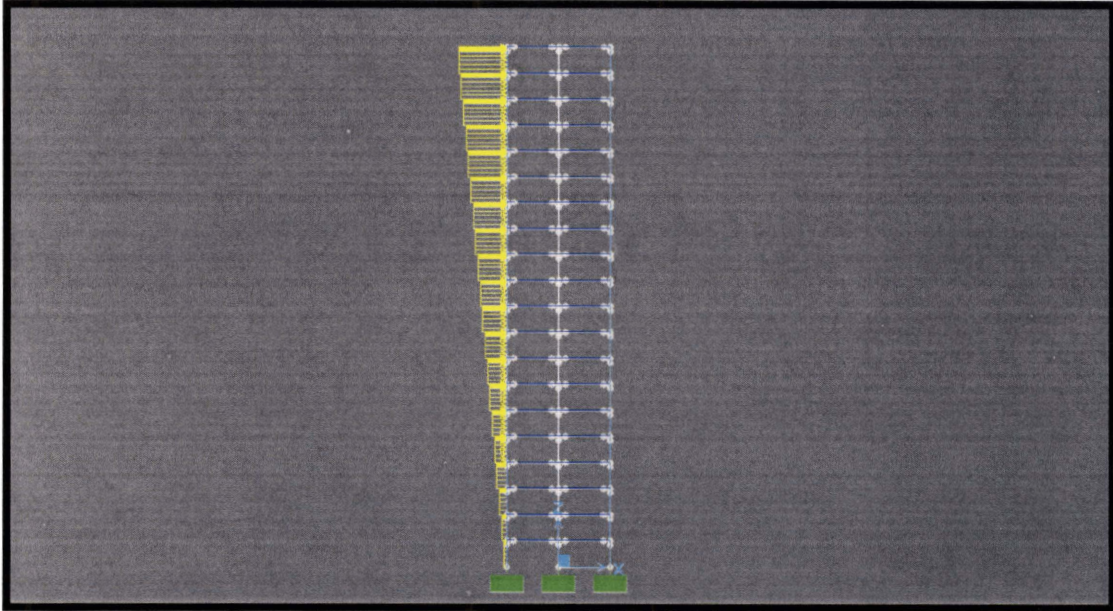


Figure 10. *Wind load distribution in the model*

The applied forces on the building as a first approach are input on the columns of one side of it as shown in figure 11.

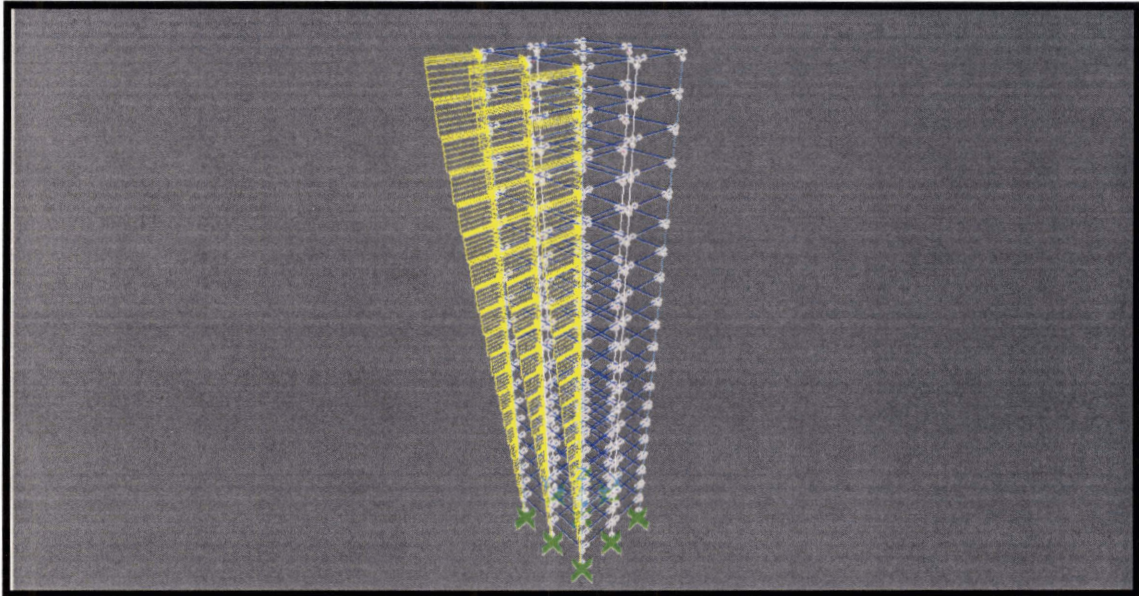


Figure 11. *Wind load distribution in the 3d model*

2.2 The bracing system

The bracing system that is going to be analyzed in this building does not consist of the usual big diagonal elements that would be connected from the base of a column to the head of the adjacent one or like any other bracing that is described in the introduction. The bracing system that was designed in the particular building consists of small diagonal braces that are connected close to the joints of columns and beams. The unbraced building is shown in the figure 12 below.

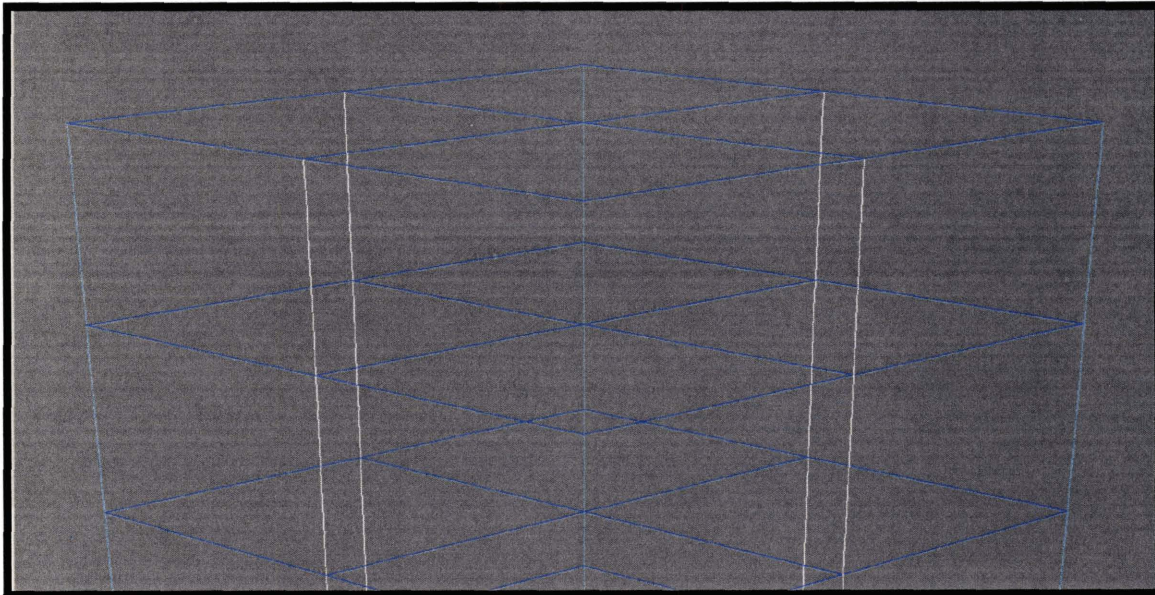


Figure 12. *Unbraced building geometry*

A wind load applied on this unbraced building produces the moment diagram, which governs the design of all the elements and mainly the columns and the beams, as shown in figure 13. The highest values appear close to the beam-column connection. Taking into account the fact that each element must be a uniform element the designer is forced to design each element with this very high value which appears at the two ends of the element. The rest of the element is not working with the same efficiency, since there is much more material than needed.

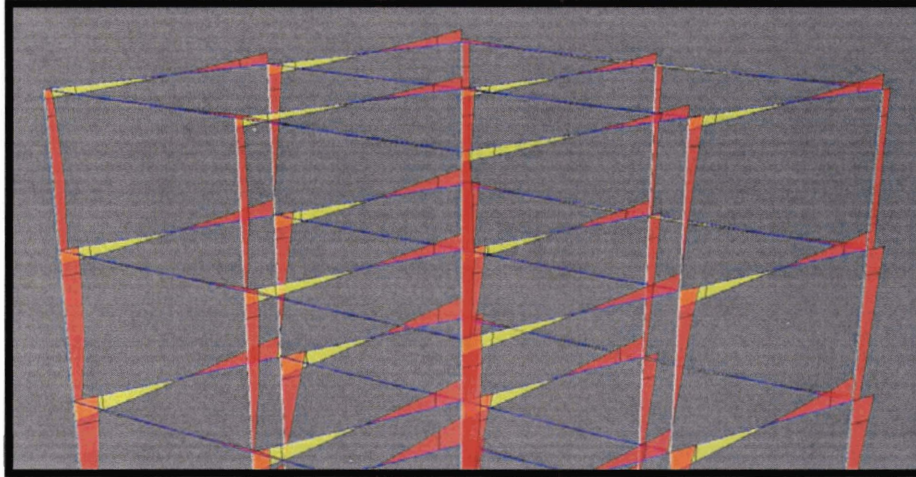


Figure 13. *Unbraced building moment of diagrams under linear wind load*

Applying the normal bracing solutions to this geometry to avoid this moment diagram with the diagonal bracing as shown in figure 14 does not change the shape at all.

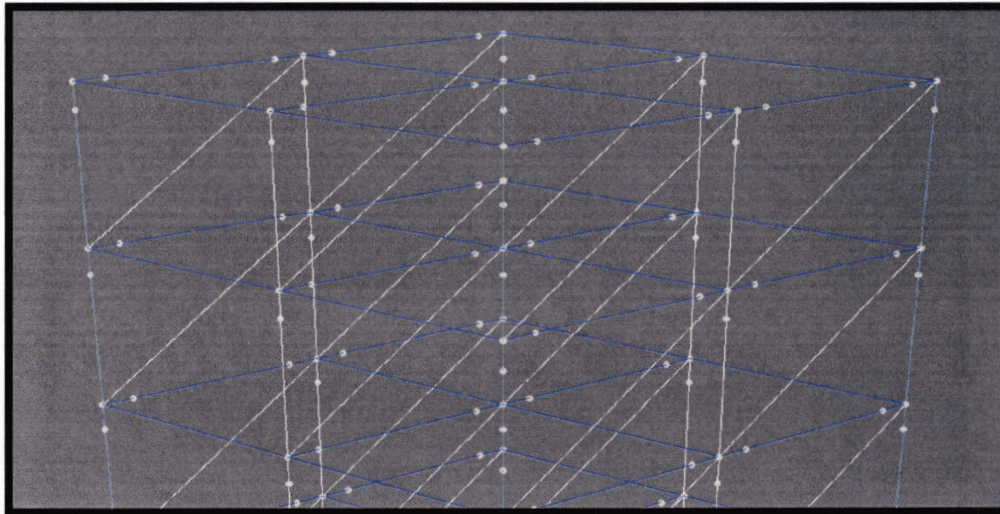


Figure 14. *Diagonally braced building*

In figure 15, the moment diagram of the diagonally braced building is shown. It is obvious that the shape of the diagram is not at all lowers from the unbraced building. It is true though that the values of the moments in the plane, perpendicular to the loading are now much higher than the values in the longitudinal plane (parallel to the loading).

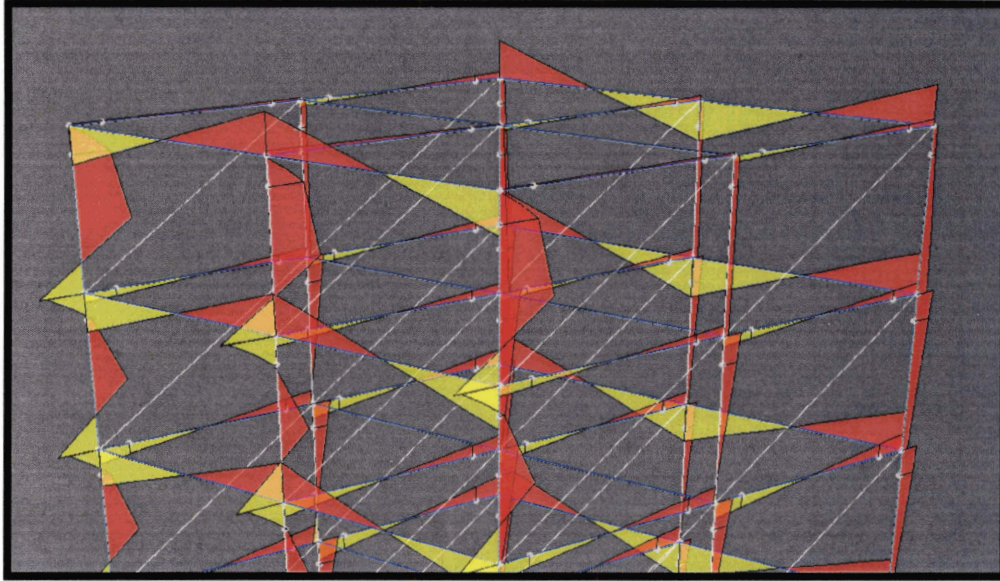


Figure 15. *Moment diagram of the diagonally braced building*

The only way to make the triangular moment diagram a more uniform diagram is to design a knee bracing system. A system like this is shown in figure 16. The bracing elements are short approximately 1m long elements that are connected 80cm inside the beam and half a meter (50cm) below the beam-column connection, on the column. This creates a geometry like the one shown in the figure below, which makes the beam-column connection much stiffer than the unbraced building.

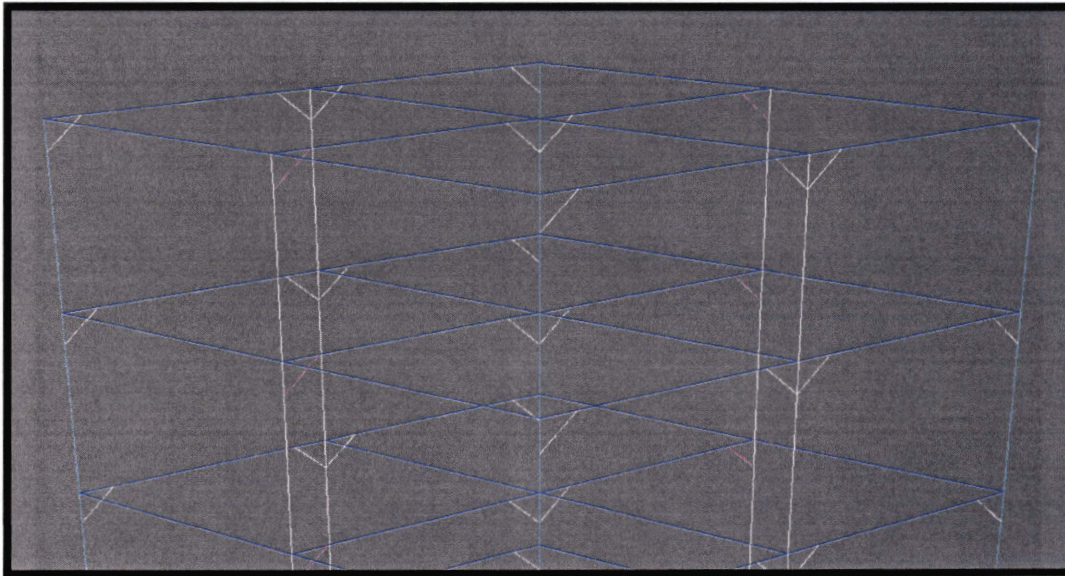


Figure 16. *Knee bracing*

The bracing elements as a first approach were selected to be pipe sections shown in figure 17. The dimensioning of the bracing elements and the distribution of their stiffness is described in a later chapter.

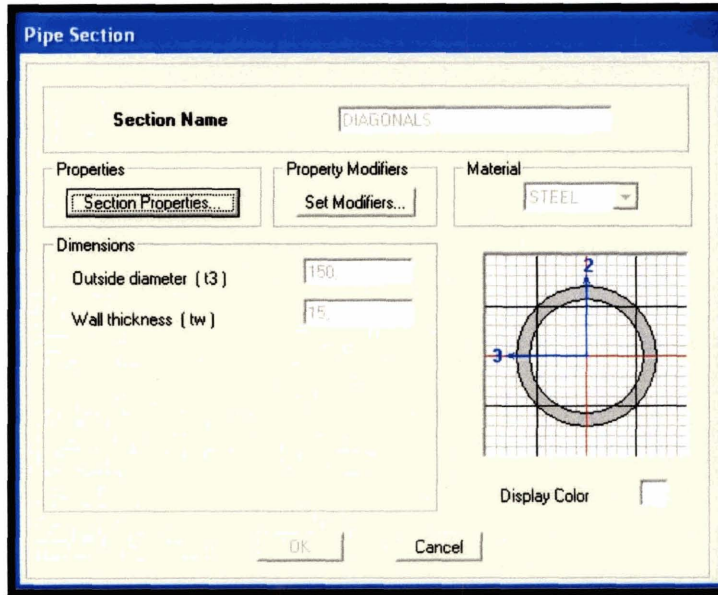


Figure 17. *Bracing elements section characteristics*

These elements are connected to the beams and the columns, with pins. There is no moment transfer through the bracing elements. They only transfer axial loads.

Figure 18 shows the moment diagram of the knee-braced building. The first goal which was to reduce the maximum bending moments at the two ends of all the elements is now achieved since the bracing elements have the effect of changing the diagram in to a more uniform one.

The maximum moment value of the beam or column now appears at the point of the connection of the bracing element, a point which can be also easily locally reinforced with stiffeners. Thus, the governing design moment can be even more decreased.

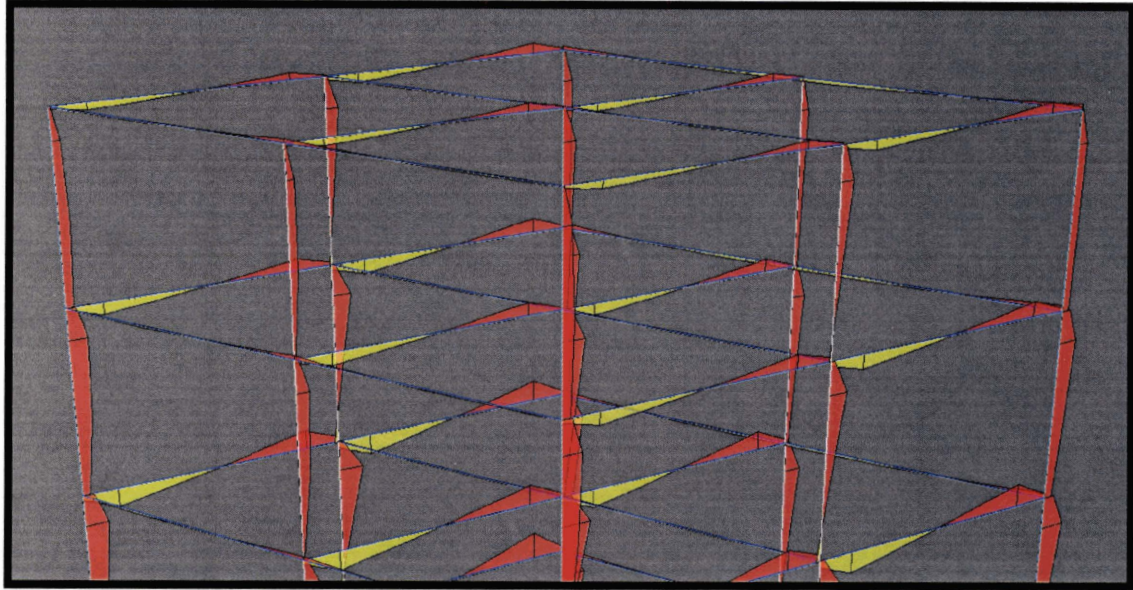


Figure 18. *Moment diagram of a knee-braced building*

2.3 Results

The main goals of any bracing system are to make the structure stiffer and to reduce moments and forces in the rest of the elements. What follows are the results showing this effect of the knee-bracing system on displacements and moments of the beams and columns.

2.3.1. Displacements

A major function of the bracing system in any structure is to reduce the displacements of the structure. In a high-rise building in particular the displacements play a very important role since they can produce discomfort for the residents or workers that work at high floors.

By inputting this bracing system the deflections at the top and centre of the building are decreased as shown in the table below:

	Displacement at the top and centre of the building (mm)	Ratio: Displacement/height of the building
Unbraced	276,34	1/217
Knee-braced	201,13	1/300
Percentage decreased	27,2%	

Table 1. Displacements at the top of the building of the unbraced and the knee-braced building

The percentage that the displacements are decreased is significant and it proves that the bracing system works indeed and it is crucial for bringing the displacements inside the allowable limits. The deformed shape of the main column is shown in the following diagram.

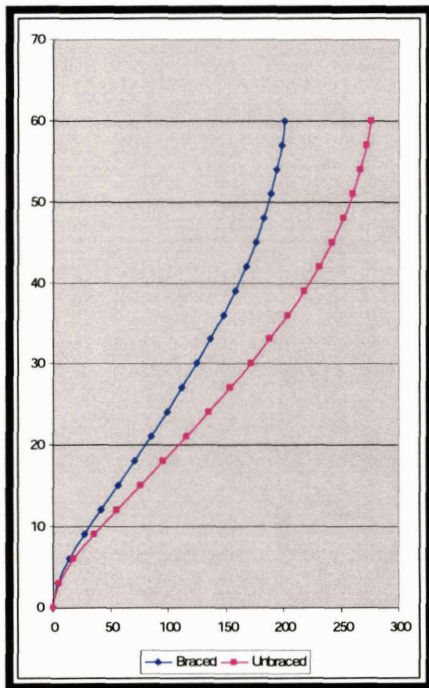


Figure 19. Deformed shape of the centre column building

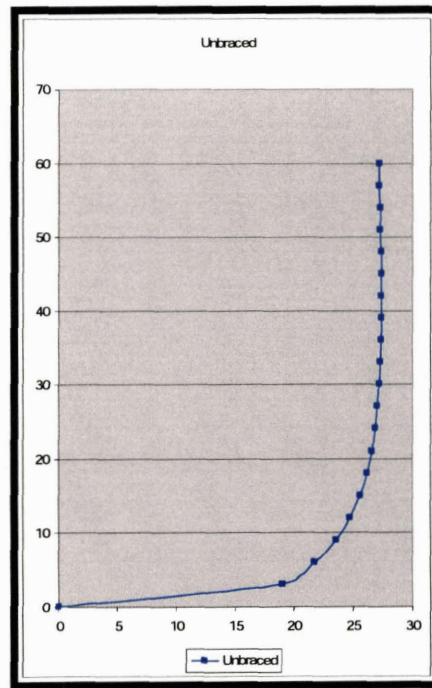


Figure 20. Percentage decrease of the displacements with the height of the building

It is obvious that the centre column of the building behaves as a fixed cantilever.

In figure 20 the percentage of the decrease for the displacements is presented. It can be noted that the effect of the bracing starts at the fourth floor approximately where the percentage decrease of the displacement is around 25%.

In addition one can see that the percentage above the seventh floor remains constant around 26-27% although the loading is still rising. A first conclusion that one can draw from this graph is that the knee-bracing elements affect the displacements in a uniform way. The effects of the knee-bracing system in the lower floors are not so significant and maybe they could be avoided. For the lower floors, say below the seventh floor figures 21 and 22 compare deformed shapes for the braced and partially braced cases.

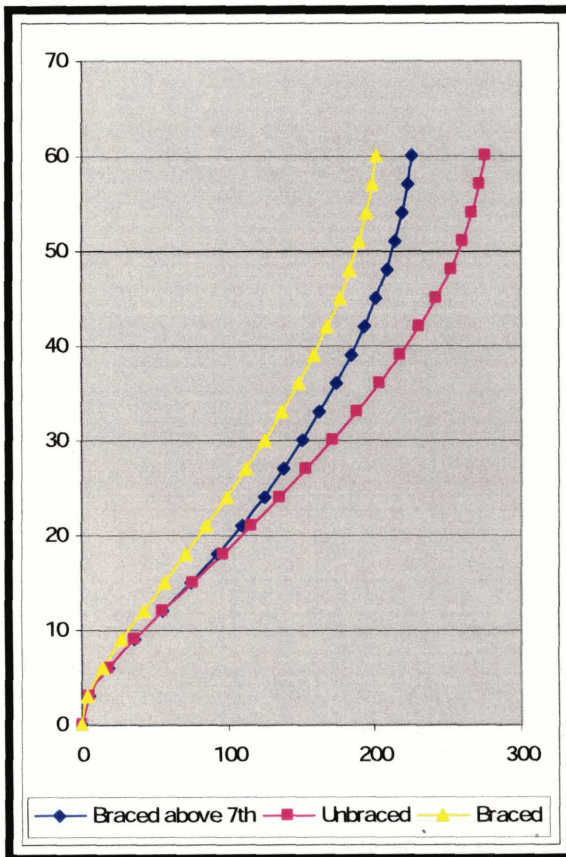


Figure 21. Deformed shape of the centre column building

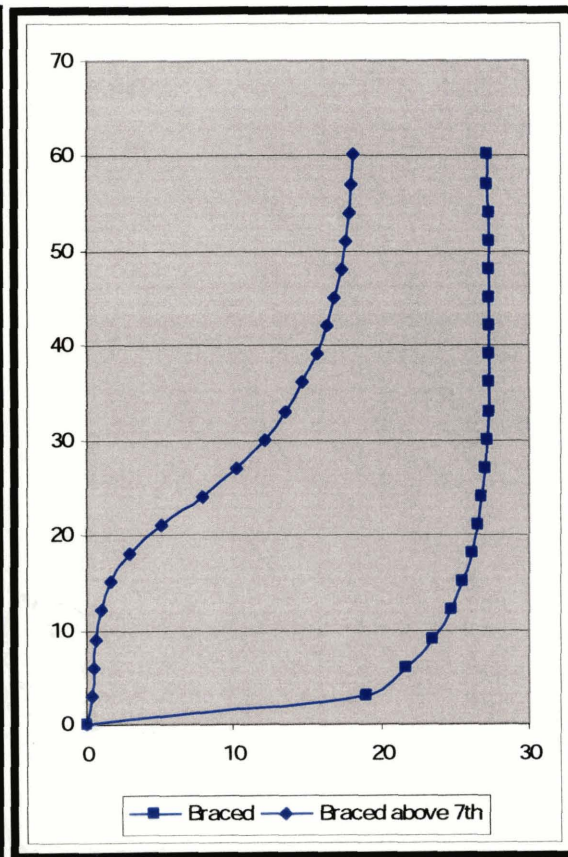


Figure 22. Percentage decrease of the displacements with the height of the building

One can see that the displacements are higher than for the fully braced building. In addition there is this transition part of the building until the bracing operates fully. In figure 21, it is obvious that the absence of the bracing below the seventh floor does not affect the building. However, there is again a part of the building above the seventh floor in which the percentage is increasing until it uniform now at the 17th floor (51m).

This transition part for the half braced building is longer now and it covers approximately from the 4th floor (12m) to the 17th (51m), in total 39m, while for the fully braced building this part is only 4 floors = 12m.

A second conclusion that comes out of these graphs is that bracing the building from the ground floor is something that provides extra stiffness to the building which is not negligible since it permits the bracing to start working at a lower height and decreases the displacements all along the building.

2.3.2. Bending moments on the columns

One very important characteristic of a high-rise building that affects the design and cost, is the bending moment values and diagram of the columns. It is usual that a lot of details depend on the section and the dimensions of the columns section. So, in addition to the main load carried by the column which is the axial load, the bending moments due to lateral loads such as wind or earthquake play a very important role in the overall design of the building.

In order to get a good perspective of the nature of the bending moment diagram for the columns of the building under the applied wind load, the centre column was considered. Figures 23 and 24 show the bending moment diagrams for that column corresponding to the unbraced and the braced condition.

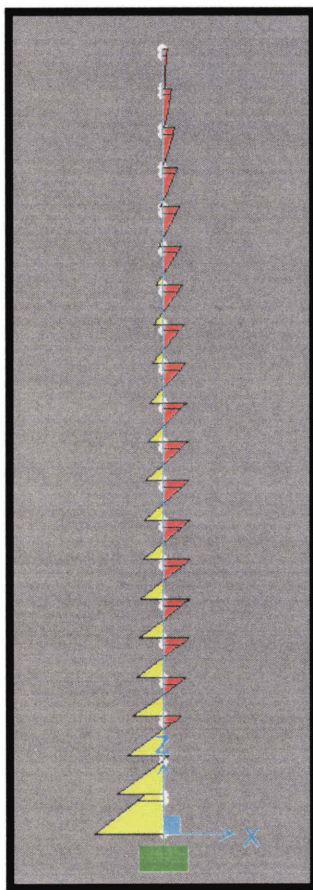


Figure 23. *Bending moments of the unbraced building*

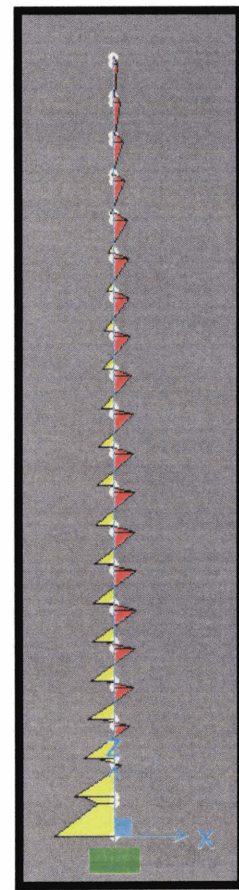


Figure 24. *Bending moments of the braced building*

It must be noted that as described in the geometry chapter, the effect of knee-bracing on the bending moment diagram is the reduction of the high values that appear at the two ends of the column and the result is a more uniform diagram.

This can be easily noticed through a closer look at the base part of the columns which is shown in figures 25 and 26. It is obvious that there is a change between the two cases which creates a more uniform and more efficient diagram for the braced condition. The strength of the column can be better used in the braced case while in the unbraced condition the column is designed for the maximum value which appears only at the end of the column.

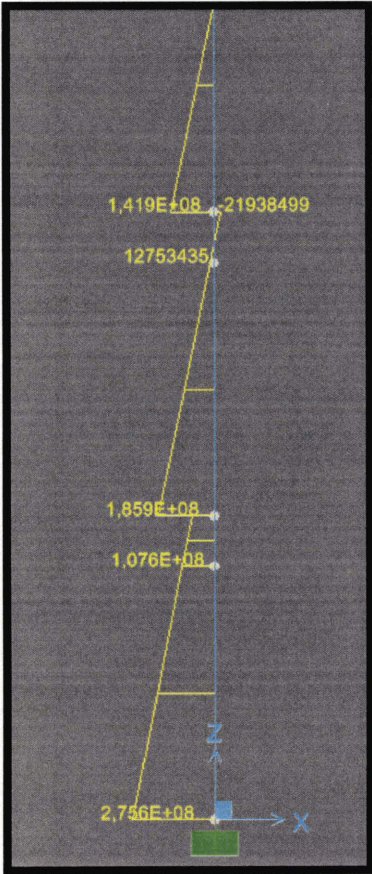


Figure 25. *Bending moments of the unbraced building*

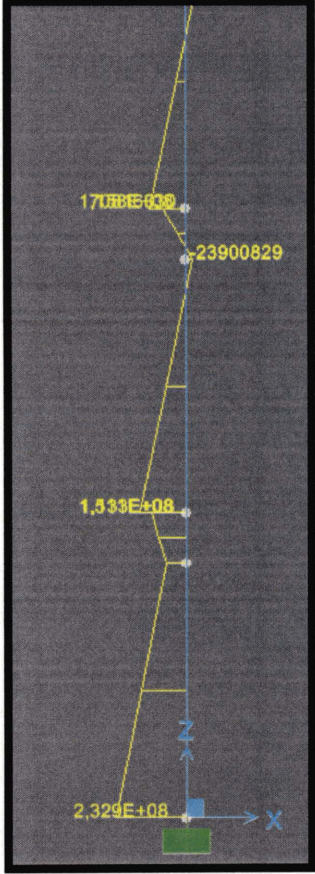


Figure 26. *bending moments of the braced building*

In more detail the difference between the two bending moment values is shown in table 2 which follows on the next page.

Braced	Unbraced	% difference
14,61	18,88	22,6165254
28,19	36,72	23,2298475
36,97	48,68	24,0550534
44,04	58,08	24,1735537
50,13	65,99	24,0339445
55,58	72,91	23,7690303
60,52	79,09	23,4795802
65	84,63	23,1950845
69,03	89,57	22,9317852
73,61	93,91	21,6164413
75,74	97,62	22,4134399
78,38	100,63	22,1107026
80,47	102,8	21,7217899
81,85	103,81	21,1540314
82,17	103,03	20,2465301
80,55	99,15	18,7594554
74,9	89,35	16,1723559
60,12	67,76	11,2750885
23,9	21,94	-8,9334549

Table 2. Bending column moment values for the braced and the unbraced case and the percentile difference

One can notice that the decrease in the moment values follows again a uniform pattern above the seventh floor. This can be noticed easily in figure 27 (next page). Until that there is a part that the percentage decrease is increasing until reaching the average value, which is around 20%.

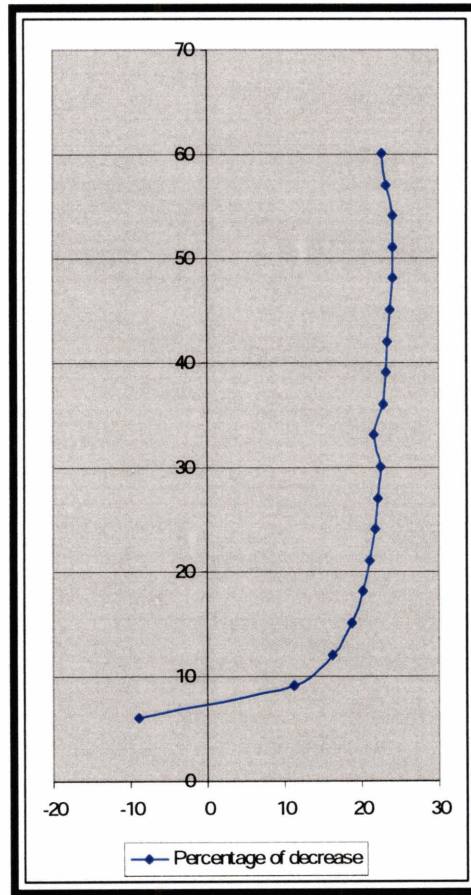


Figure 27. Percentile column moment values decrease as a function of height

According to the code then, having reduced the applied moment by 20% and calculating the w_{pl} for the column:

$$M_{sd} < M_{pl,Rd} = W_{pl,UNBRACED} \cdot \frac{f_y}{1,1}$$

$$0,8 \cdot M_{sd} < W_{pl,BRACED} \cdot \frac{f_y}{1,1}$$

$$\Rightarrow \frac{M_{sd}}{0,8 \cdot M_{sd}} < \frac{W_{pl,UNBRACED} \cdot \frac{f_y}{1,1}}{W_{pl,BRACED} \cdot \frac{f_y}{1,1}} \Rightarrow$$

$$W_{pl, BRACED} < 0,8 \cdot W_{pl, UNBRACED}$$

One can see that there is a 20% decrease in the required w_{pl} of the column. This can be translated in weight as shown in the following table.

	Necessary section (w_{pl} in mm^3)	Weight of section (kgrs/m)
Unbraced building	2.548.000	124,88
Knee-braced building	2.038.400	135,0
Percentile decrease	20%	8%

Table 3. *Weight decrease of the column*

The effect of knee-bracing on the columns is significant since they can be reduced as much as 8% from their initial weight. This means an immediate decrease in the dimensions of the column which can provide even more space for the working area of the building.

2.3.3. Bending moments on the beams

One main factor that affects the total behaviour of the structure is how the beams are connected to the columns. The two different ways in which this connection is realised are:

1. Moment bearing connection = Moment Frames
2. Pin connection = Braced frames

So far, all the analysis that has been done and presented incorporated moment frame connections for the beams. This does not mean that this is the only way though. In this chapter the braced frames will be analysed and the comparison between moment and braced frames will be presented.

1. MOMENT FRAMES

In case of wind load applies on the structure, the shape of the bending moment diagram for the beams and for the braced and unbraced structure is like the the one shown in figures 28 and 29 below.

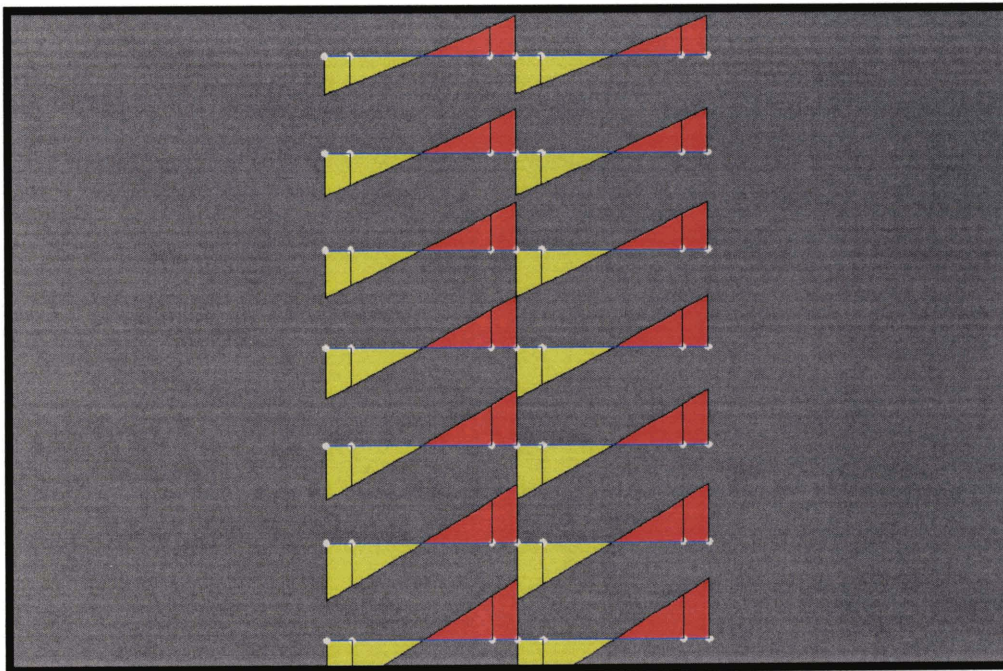


Figure 28. *Beam moment diagram for the unbraced building*

One can notice once again the triangular shape of the diagram, which means as mentioned that there is not very high efficiency in the design process for the beam, since the design is governed by the high end value of the diagram.

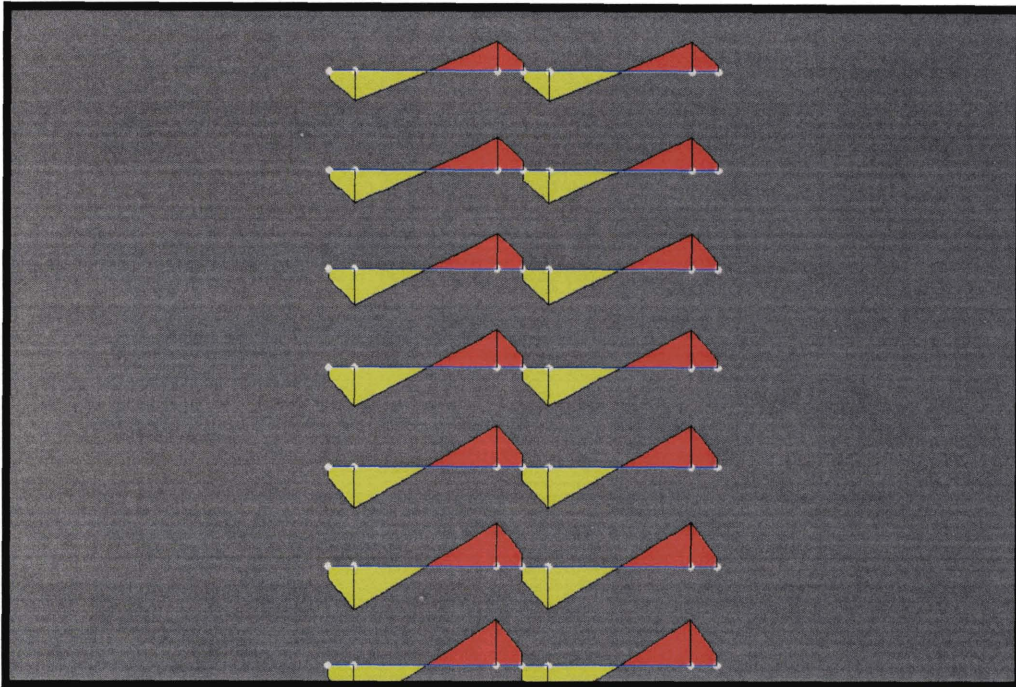


Figure 29. *Beam moment diagram for the braced building*

Bracing the building immediately changes the shape of the moment diagram for the beams. It creates a more uniform shape having the maximum value in the connection of the bracing element with the beam. The values of the moments for the two conditions and the percentile difference can be shown in table 4 below.

Braced	Unbraced	% difference
46,05	57,45	19,843342
65,7	83,45	21,2702217
72,96	94,73	22,9811042
74,89	98,75	24,1620253
74,37	99,03	24,901545
72,6	97,23	25,3316878
70,08	94,17	25,5813953
67,02	90,23	25,7231519
63,51	85,58	25,7887357
58,6	80,33	27,050915
55,29	74,53	25,815108

Braced	Unbraced	% difference
50,61	68,22	25,8135444
45,56	61,41	25,8101286
40,16	54,14	25,8219431
34,44	46,46	25,8717176
28,42	38,44	26,0665973
22,21	30,27	26,6270235
16,01	22,27	28,1095644
10,35	15,21	31,9526627
6,55	10,74	39,0130354

Table 4. *Bending beam moment values for the braced and the unbraced case and the percentile difference*

These values are plotted as a function of the height of the building in figure 30. It is noticed that the same pattern appears. Below 21m (seventh floor) there is the transition part of the building where the percentile decrease of the moments is increasing until reaching the average value of decrease, which is 27%.

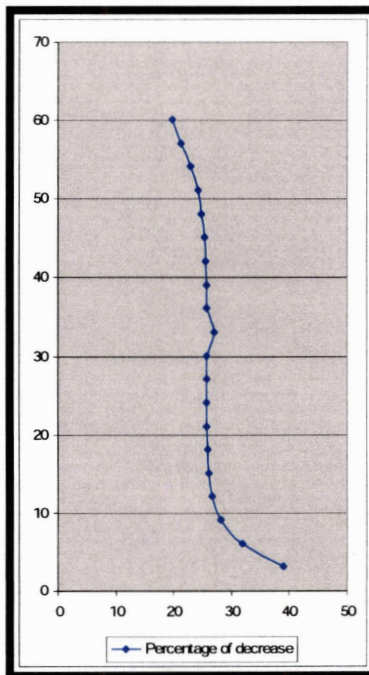


Figure 30. *Percentile beam moment values decrease as a function of height*

Applying the code again, having reduced the applied moment by 27% and calculating the w_{pl} for the column:

$$M_{sd} < M_{pl,Rd} = W_{pl,UNBRACED} \cdot \frac{f_y}{1,1}$$

$$0,73 \cdot M_{sd} < W_{pl,BRACED} \cdot \frac{f_y}{1,1}$$

$$\Rightarrow \frac{M_{sd}}{0,73 \cdot M_{sd}} < \frac{W_{pl,UNBRACED} \cdot \frac{f_y}{1,1}}{W_{pl,BRACED} \cdot \frac{f_y}{1,1}} \Rightarrow$$

$$W_{pl,BRACED} < 0,73 \cdot W_{pl,UNBRACED}$$

One can see that there is a 27% decrease in the required w_{pl} of the column. This can be translated in weight as shown in the following table.

	Necessary section (w_{pl} in mm^3)	Weight of section (tons/m)
Unbraced building	631.000	45,5
Knee-braced building	460.630	40,4
Percentile decrease	27%	11,2%

Table 5. Weight decrease of the beam

The effect of knee-bracing on the beams proves to be more significant than the columns since they can be reduced as much as 11,2% from their initial weight. This means an immediate decrease in the dimensions of the column which can provide even more space for the working area of the building.

Table 6 contains a summary of the two decreases in weight (columns and beams), along with the increase in the total weight of the structure by inputting the bracing elements.

	Columns	Beams	Bracing elements
Total weight of the unbraced elements (other columns and corner columns, in m)	135,02/238,64	45,53	-
Percentile decrease/increase (%)	8%	11,2%	-
Total length of elements (other columns and corner columns, in m)	240+300	1440	226,416
Absolute weights (tons)	104	65,6	
Absolute decrease/increase	8,32	7,3	20,1
Total weight of the braced elements (kgr/m)	95,68	58,3	20,1

Table 6. Calculation of the weight of all elements of the structure

From the above we have a total increase in the weight of the structure of about:

$$(95,68 + 58,3 + 20,1) - (104 + 65,6) = 4,48kgr$$

This means that not only the building is stiffer and the displacements are limited, but also the total weight of the building is increased by a very small number. As a percentage the increase of the weight of the building is:

$$\frac{4,48}{104 + 65,6} = 0,026$$

This means that there is a increase of approximately 2,6% of the total weight of the steel structure.

2. BRACED FRAMES

Another way to connect beams and columns is with pin connections. This way is preferred since these connections are much easier to be constructed than moment frame connections. However, the overall stiffness of such a building is none (unbraced).

The first approach to the braced frame problem was simply to input the frame releases at the ends of all the beams and run the analysis keeping all the rest of the characteristics of the sections and the geometry the same. However, when this analysis run, the results for the unbraced “braced” frame were extremely out of the normal spectrum because this building is a mechanism. When the same geometry including the bracing run, the results were logical.

It has to be noted here that a frame building with pin connections at the end of its beams is an unstable building. That is why when putting knee-bracing the building becomes incredibly stiffer than before. Something that proves that bracing is much more important in braced frames.

However, in order to start with good results the sections of the elements of the structure were changed. All the above are summarized in table 7 below.

	Displacement at the top and centre of the building (mm)	Percentile decrease
Unbraced (moment frame)	276,34	-
Knee-braced (moment frame)	201,13	27%
Unbraced (braced frame-same geometry)	Several meters ($\approx 69\text{m}$)	-
Knee-braced (braced frame)	207,26	Very high
Unbraced (braced frame-changed geometry)	5205	-
Knee-braced (braced frame-changed geometry)	80	98%

Table 7. Displacements at the top of the building for the unbraced and the knee-braced building (moment and braced frame)

A first conclusion that can be extracted from the table is, as mentioned above that the effect of knee-bracing becomes much more important in a simply braced building. It is very interesting that the same geometry and the same sections with the only difference the connections at the end of the beams produce in the moment frame a deflection of 276mm, and for the simply braced a huge deflection. At the same time when the simply braced building is braced with the knee-bracing, it immediately comes in normal values and very

close to the moment frame building. Something that leads us to the conclusion that in case a knee-bracing system is used a simply braced frame system must be preferred and there is no need for moment bearing connections between the beam and the column.

Regarding the moment diagram for the beams, the effect of knee-bracing is again much beneficial. In figures 31 and 32 the beam moment diagrams are shown for the unbraced and the braced “simply braced building”.

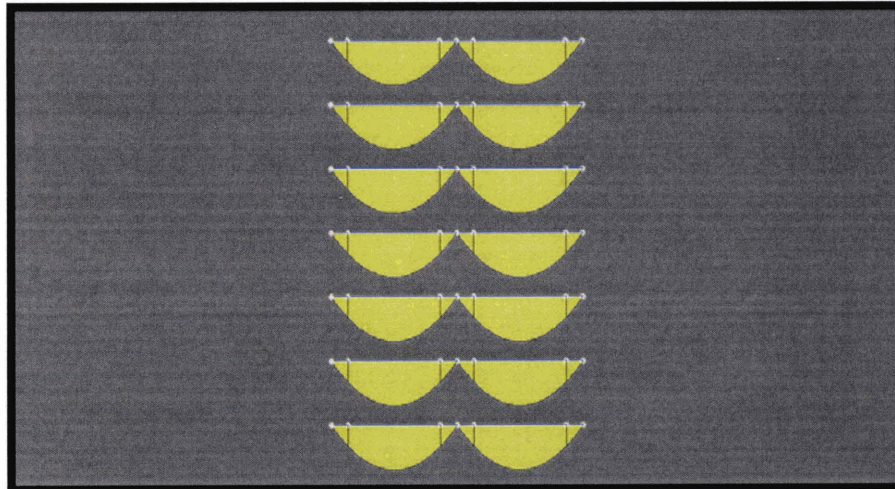


Figure 31. *Beam bending moments for the unbraced “simply braced” building*

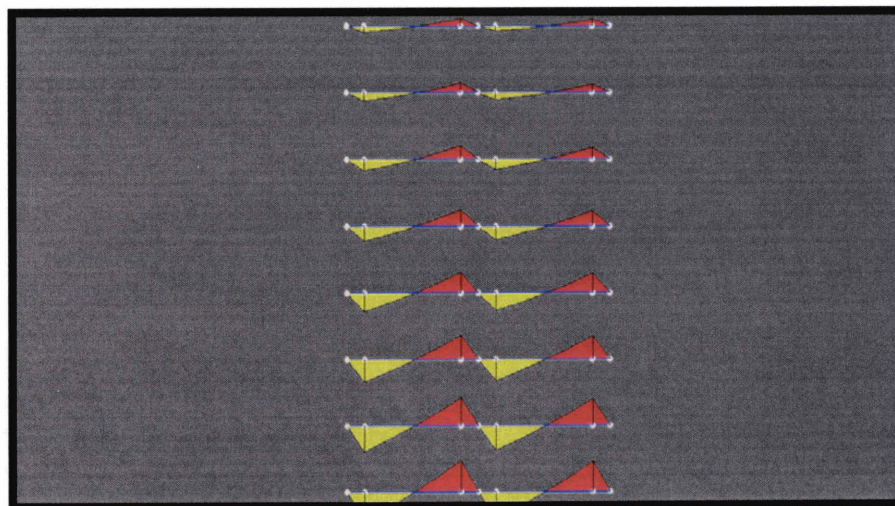


Figure 32. *Beam bending moments for the braced “simply braced” building*

It must be noted here though that the maximum values of the moments for the beams are much lower for the unbraced condition. This happens because the beams in the unbraced condition do not take part in the structural system of

the building that reacts to the wind load. On the other hand, when braced, they become a part of the structure that actively plays a role bearing lateral loads.

That is why the diagram for the unbraced condition is a parabola. It is only because of the dead loads, there is no moment transfer in the ends of the beam. In the braced case there is moment transfer because of the bracing elements.

2.4 Variable sizes of the diagonals

A very important factor when designing the knee-bracing system is the section of the diagonal bracing elements. For the purpose of the analysis a pipe section was used initially as described in previous chapters. However, the dimensions and characteristics of the section is something so far not defined and at the same time is something that can change a lot the behaviour of the bracing system. For that reasons several different sections were used and the structure was analysed after each change. According to the results and the best behaviour of the system, the optimized section will be selected in the end of the chapter. In table 8 below all the different sections are presented:

	Size of diagonal (pipe in mm, radius/thickness)
Analysis #1	100/10
Analysis #2	100/12
Analysis #3	100/15
Analysis #4	100/20
Analysis #5	120/10
Analysis #6	120/12
Analysis #7	120/15
Analysis #8	120/20
Analysis #9	150/10
Analysis #10	150/12
Analysis #11	150/15
Analysis #12	150/20

Table 8. *Different sections of diagonals checked*

Three different diameters for the pipe section were selected and for each diameter four different thicknesses were also selected, in total 12 sections. After running the analysis for all these different cases the displacements at the top of the building were extracted. In figures 33 and 34 these displacements are shown in graphs firstly as groups of the same diameter and secondly as a function of the weight of the diagonals.

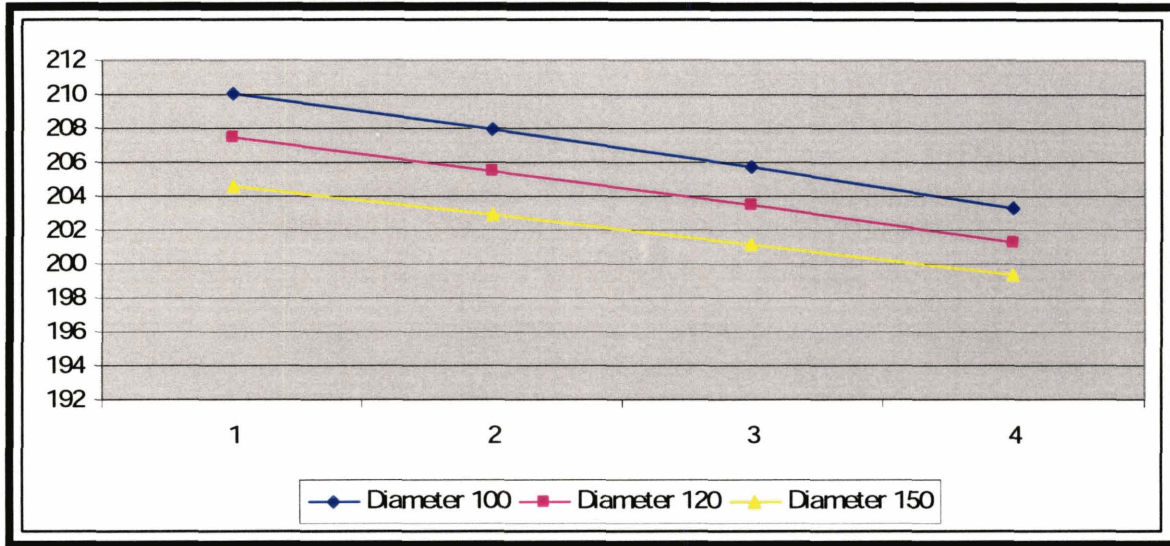


Figure 33. Displacements at the top of the building as groups for the same diameter

From figure 33 one can notice that there is a logical decrease in the deflection of the building as the stiffness of the diagonal increases, either because of diameter increase or of thickness increase. On the other hand figure 34 shows a rather not uniform pattern of decreasing the deflection with the increase in the weight of the diagonals, which means that the optimization of the diagonal section does not only depend on the weight, thus the area of the section, but also on other characteristics such as the moments of inertia. Optimizing the section of the diagonal also must include the result that comes out of a change in the diagonals. For instance a decrease of 5% in the deflection (from 210mm to 199,3mm) is translated into an increase in the weight of the diagonals of as much as 300%. Increasing the weight of the diagonals 300% just for a minor decrease in the deflection is certainly not the most efficient way of doing it.

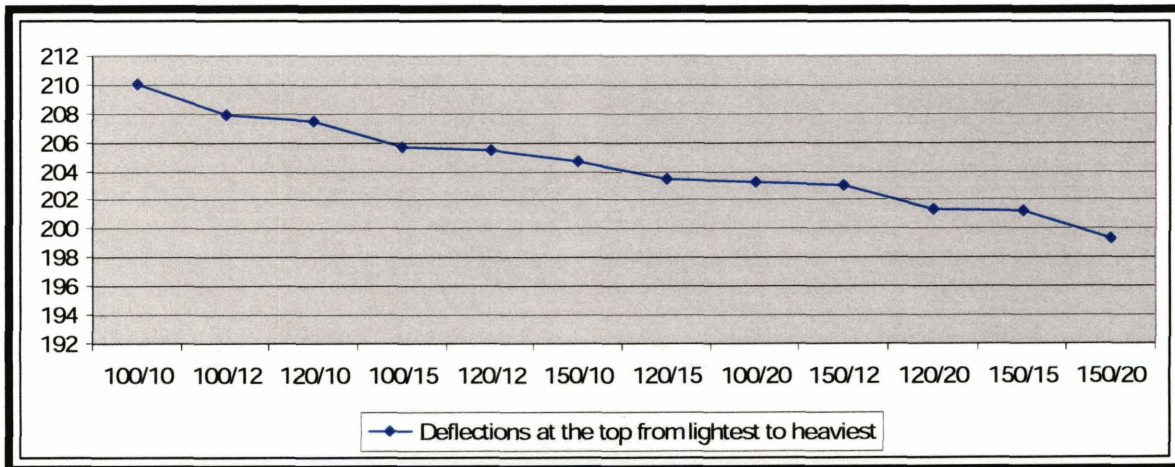


Figure 34. Displacements at the top of the building as a function of the weight of the diagonals

Table 9 below shows all the different sections and their characteristics.

Diagonals	Deflection at the top (mm)	Area (mm ²)	Length (mm)	Weight (kgr)
100/10	210	2827,433388	943,4	20,93909517
100/12	207,9	3317,521842	943,4	24,56853833
100/15	205,62	4005,530633	943,4	29,66371816
100/20	203,22	5026,548246	943,4	37,22505808
120/10	207,4	3455,751919	943,4	25,59222743
120/12	205,4	4071,504079	943,4	30,15229704
120/15	203,4	4948,008429	943,4	36,64341655
120/20	201,23	6283,185307	943,4	46,5313226
150/10	204,6	4398,229715	943,4	32,57192582
150/12	202,9	5202,477434	943,4	38,52793511
150/15	201,13	6361,725124	943,4	47,11296413
150/20	199,3	8168,140899	943,4	60,49071938

Table 9. *Different sections and their characteristics*

Considering the fact that all the deflections are very close, compared to the initial not braced building, it is reasonable to try to compare the total increase in the tonnage of the structure due to the bracing. The heaviest diagonal used from the above table is the diagonal with a diameter of 150mm and a thickness of 20mm. The total increase in the structure's self weight is calculated and shown in table 10 below. It has to be noted that the self weight of the building without any bracing is around 167,37tons.:

Number of single diagonals	200
Number of double diagonals	40
Total number of diagonals	280
Total weight of diagonals	16940kgr=17tons
Increase in the tonnage of the building	≈10%

Table 10. *Total increase of self weight of the structure due to diagonal bracing (the worst case)*

In table 10 there are two types of diagonals described. Single diagonals and double diagonals. This will be analysed in detail in chapter “adaptivity-active stiffness” but a note must be written here as well. The concept was to put double diagonals in places that could have the space and the possibility to do that. In figure 35 below, these diagonals are shown in a different colour. The only difference in the simulation of these diagonals was that they only double the area of the section, so they can take double the axial force. The reason why these diagonals appear at that points is explained in chapter “adaptivity-active stiffness”. In a more detailed simulation two actual elements should be included in the model.

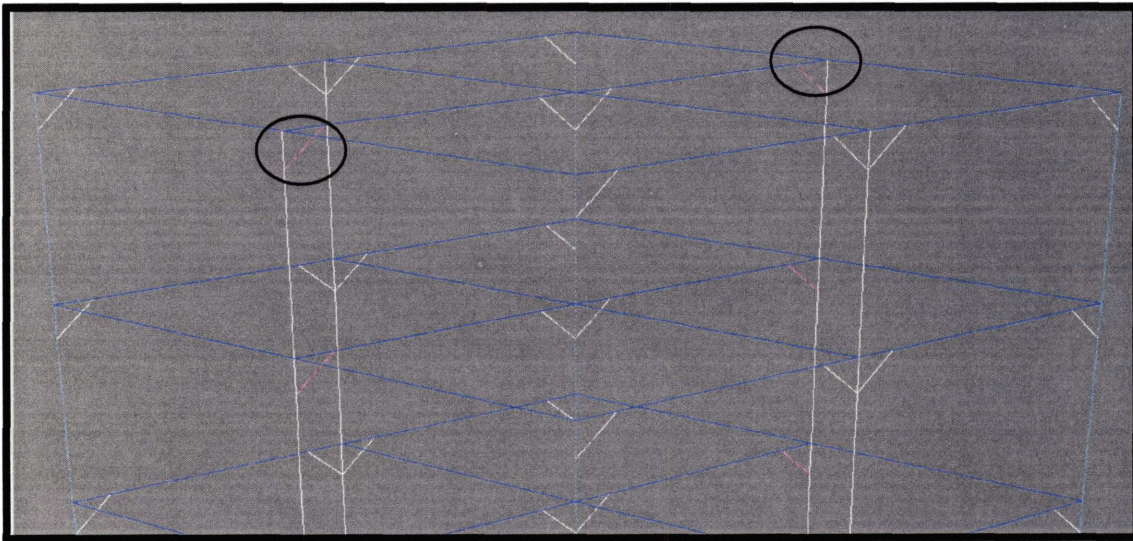


Figure 35. *Places of the double diagonals*

It is shown from table 10, that the maximum increase is about 10% of the self weight of the unbraced building. This number is considered to be very low, since the benefits of using these elements are very important and create a much stiffer structure than the unbraced one.

Having all these results of sections with different diameters and different thicknesses was very helpful and gives a very good perspective of how the knee-bracing works, but at the same time creates some confusion. A better way of approaching the problem would be to change one of the two variables to a constant number so that it would be easier to understand the behaviour of these elements. So, in order to find out better and more reliable results, a further analysis was conducted. It was decided, considering also constructability constraints, that a reasonable diameter of the diagonals could be 200mm. Having this as a constant, different thicknesses were examined to see the comparison between the decrease in the deflection of the building and the increase in the total tonnage of it.

Table 11 on the next page shows the different diagonals used for all the analyses, the deflection at the top came out of the results, the weight of the diagonals, the percentiles differences in the tonnage of the structure, the deflection at the top of the building as

compared to the one of the unbraced structure and the change in the period of the structure using all these different diagonals.

Diagonals (diameter/thickness)	Deflection at the top (mm)	Area (mm ²)	Length (mm)	Weight (kg)	Increase in tonnage (%)	Decrease in deflection (%)	Period (sec)
200/5	208,96	3063,05	943,4	22,68402	3,794832819	24,38300644	1,782
200/6	206,69	3656,81	943,4	27,08123	4,530446566	25,20445828	1,787
200/7	204,96	4244,29	943,4	31,43191	5,25827604	25,83049866	1,793
200/10	201,66	5969,03	943,4	44,20476	7,395058827	27,02467974	1,809
200/12	200,3	7087,43	943,4	52,48733	8,780659323	27,5168271	1,819
200/15	198,89	8717,92	943,4	64,56221	10,80067802	28,0270681	1,833
200/20	197,45	11309,73	943,4	83,75638	14,01169041	28,5481653	1,857
200/25	196,58	13744,47	943,4	101,7873	17,02809598	28,86299486	1,878
200/30	196	16022,12	943,4	118,6549	19,84989475	29,07288123	1,898
200/35	195,61	18142,70	943,4	134,3592	22,4770867	29,21401172	1,917
200/40	195,31	20106,19	943,4	148,9002	24,90967184	29,32257364	1,934
200/45	195,1	21912,61	943,4	162,278	27,14765017	29,39856698	1,949
200/50	194,94	23561,94	943,4	174,4925	29,19102169	29,45646667	1,96

Table 11. Different diagonals used with the same diameter and results

As shown above, many different thicknesses were examined, starting from a very thin profile of 5mm and reaching a very thick one of 50mm. Figure 36 below shows the increasing weight of the diagonals.

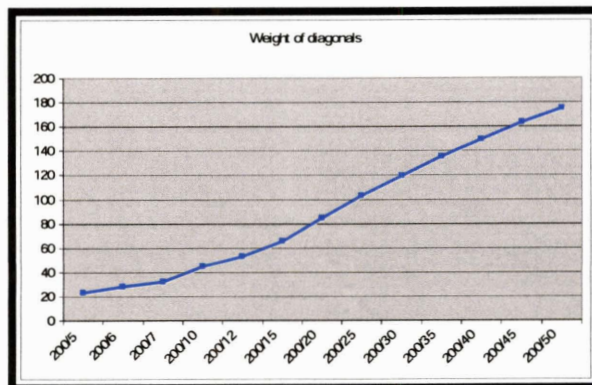


Figure 36. Weight of diagonals

Projecting the results that appear in table 11, the following graphs came out. In figure 37 the deflections at the top of the building are shown as a function of the thickness of the diagonal bracing elements. It can be noticed that small changes in the thickness of the diagonals, when the thickness is still low, produce significant decreases in the deflection of the building, while changes in the thickness of the diagonals in high thicknesses hardly affect the deflections of the building. **It must be noted here that the decreasing tangent of the curve of figure 37 shows that the thicker the diagonal becomes the less impact rate can have on the deflection of the building.** That is why the optimum thickness that can be extracted from that graph is around 20mm, since above that thickness there is much weight increase and less stiffness increase.

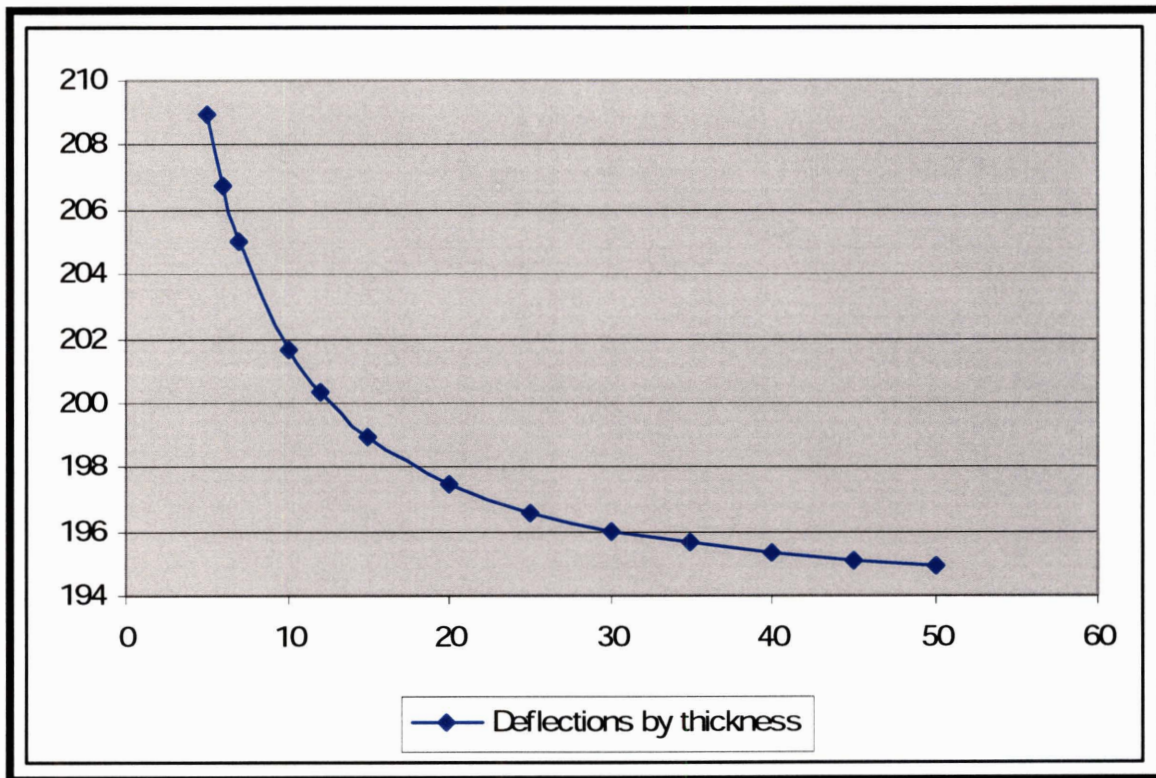


Figure 37. *Deflections at the top of the building as a function of the thickness of the diagonals*

This can be also shown from the graph of figure 38 which show the same curve of figure 37 in addition to the curve which shows the increase of the weight of the diagonals as functions of the thickness increase. It is noticed that the deflections are affected until a thickness of around 20mm. Above that number the deflections are almost invariant while the weight of the diagonals logically increases. Based on that graph one can clearly say that the optimum thickness is 20mm. **Something that must be added here is that only a graph presenting deflections and weight can give enough data and evidence for optimizing the thickness and stiffness of the diagonal bracing.**

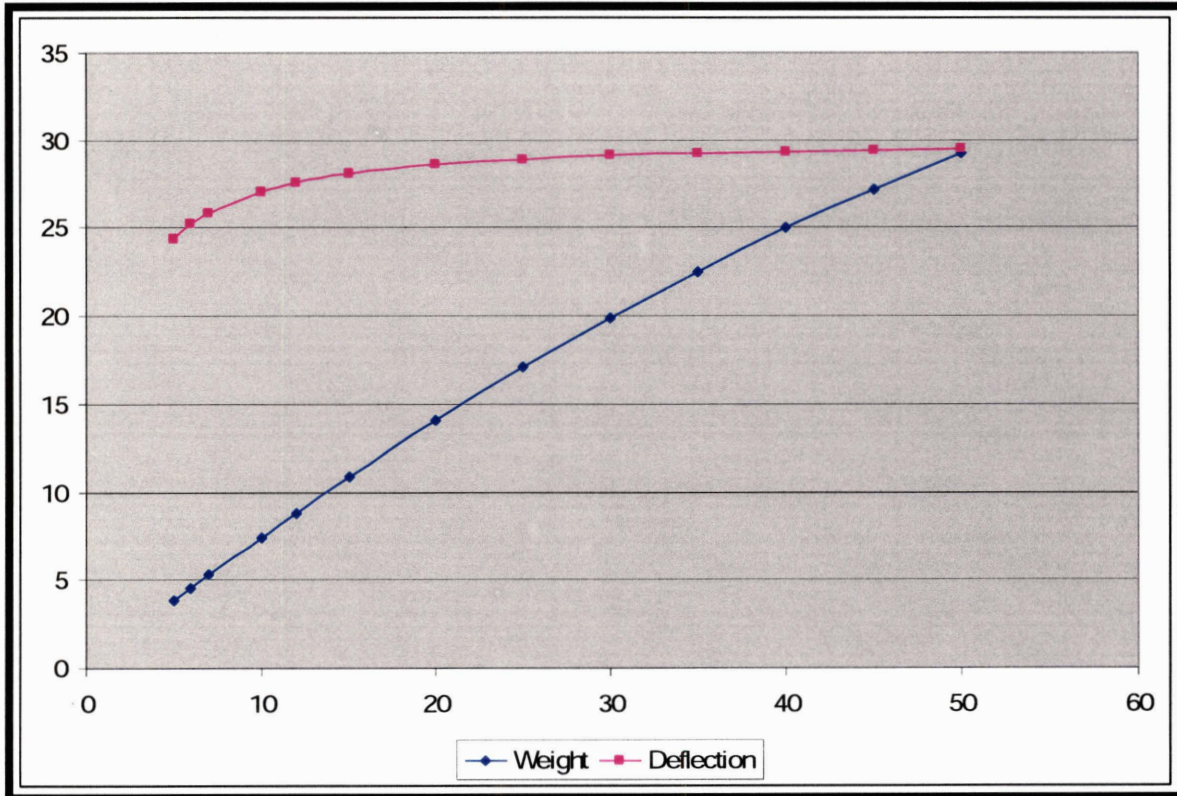


Figure 38. *Deflections at the top of the building and weight of the diagonals as functions of the thickness of the diagonals (all percentages)*

An important characteristic of the structure that is affected by the change in the section of the diagonal is the period of the structure. In figure 39, the fundamental period of the building is presented. However, it must be noted that the first mode of the structure is not the most relevant with the stiffness of the diagonals since it deforms the building laterally to the knee-bracing. The second mode of the structure is again an irrelevant shape and it deforms the building in torsion as height increases. The third mode of the structure deforms the building along the knee-bracing. In figure 40 the periods of the third mode are plotted as a function of the thickness of the diagonals.

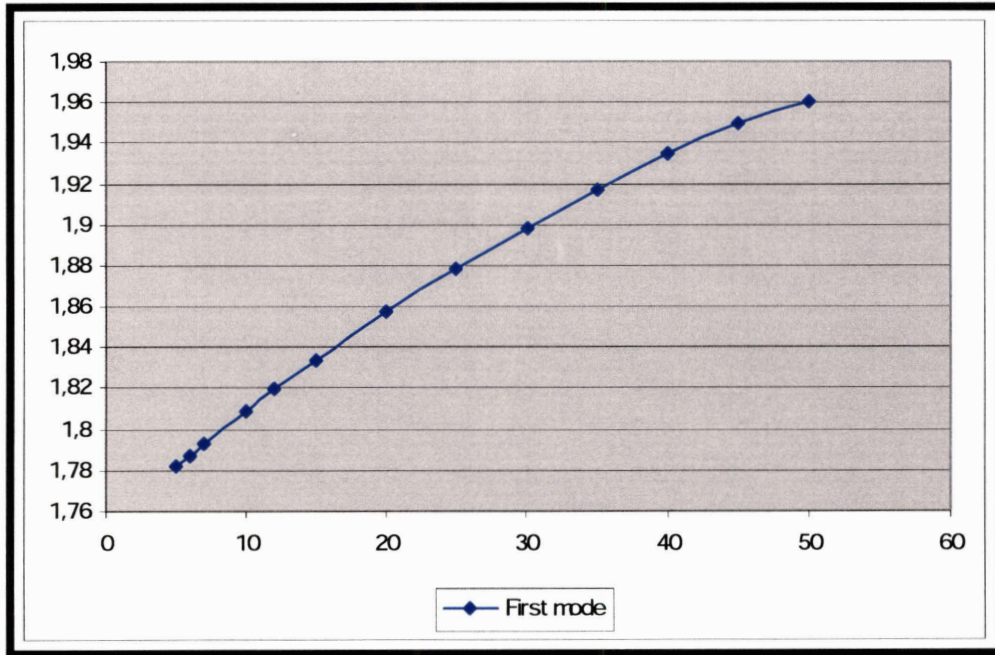


Figure 39. Fundamental periods of the building as a function of the thickness of the diagonals

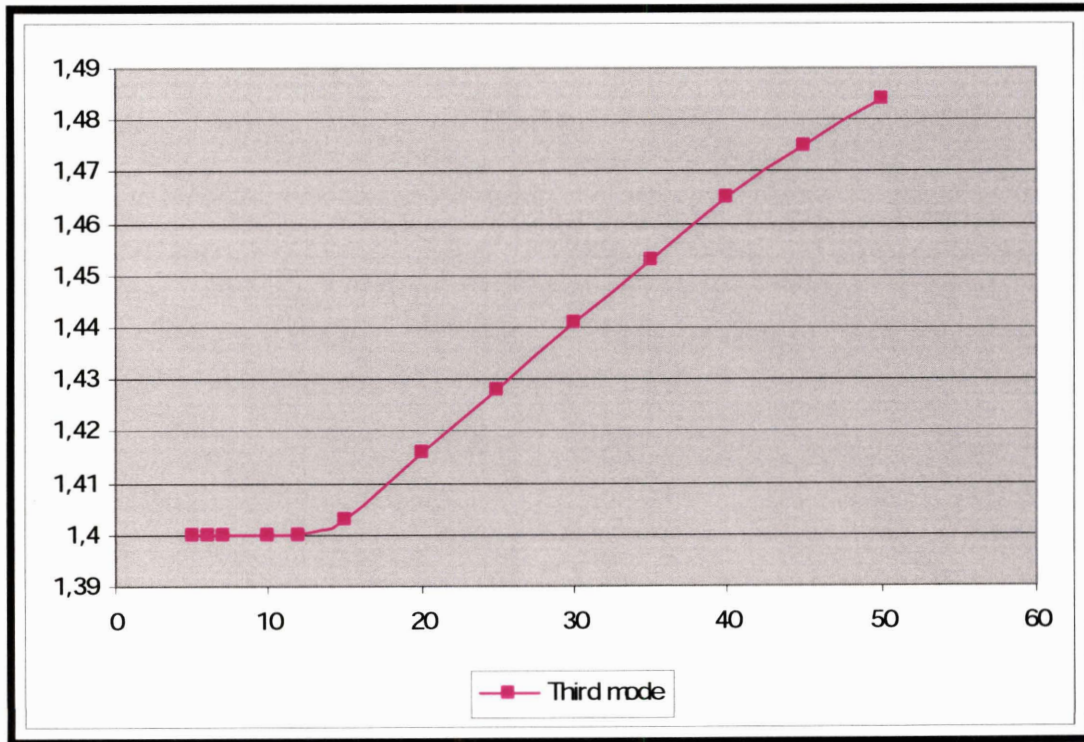


Figure 40. Periods of the third mode for the building as a function of the thickness of the diagonals

Although the fundamental periods of the building seem to increase in a linear way (something that does not lead us to any conclusions because the fundamental mode is not critical), the periods for the third mode of the structure start to increase above a certain value of the thickness of the diagonal (around 12mm).

Above that value of thickness, the period starts to rise. The fundamental period of the unbraced structure is 1,577sec which is much higher than the braced one. This in a simple way means that according to the formula,

$$T = 2\pi \sqrt{\frac{m}{K}}$$

there is more stiffness input in the building than mass. However, as the thickness of the diagonals increase, the period tends to reach the period of the unbraced building, which cancels the positive effect of the knee-bracing. According to that criteria then, the thinnest the diagonal the better it is for the period. Of course this criteria depends a lot on the applied frequency of the wind load for the . In case the wind load's frequency is low then a high eigenmode period is preferred. In addition other factors that affect this criteria are: the foundation of the building which can affect the period, or in case of an earthquake area, the soil conditions and the applied earthquake load period.

Taking into account all the above mentioned, it was considered that the optimum diagonal for this building is the 200/20.

This diagonal knee-bracing provides a decrease in the deflection at the top of 28,5% and an increase in the total tonnage of the structure of 14%.

2.5 Optimizing the distribution of the diagonals

2.5.1. General

Although designing the diagonals according to the criteria described in chapter 2.4, it is common sometimes not to use one section for all the diagonals of the building. That is more efficient because not all diagonals work under the same load, so it is not an efficient way of designing to select one section for different places in the building where the forces are different. The proof to that comes from figures 41 and 42 where the axial forces of the diagonals throughout the height of the building are shown. Figure 41 shows the axial forces from dead load, where the distribution is almost uniform, while figure 42 shows the distribution of the axial forces from the wind load combination.



Figure 41. Axial forces on diagonals under dead loads

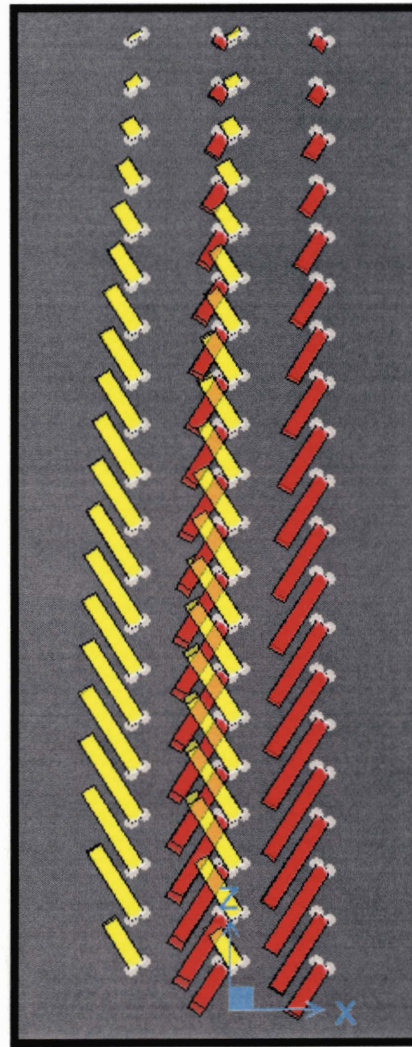


Figure 42. Axial forces on diagonals under wind loads

Figure 43 shows the variation of the diagonal forces for each series with height. Each of the different series of the diagonals represent diagonals that are in a vertical line. There are 12 locations in plan view where diagonals are placed, so there are 12 series of diagonals. As expected half of the diagonals work in compression and half of them in tension.

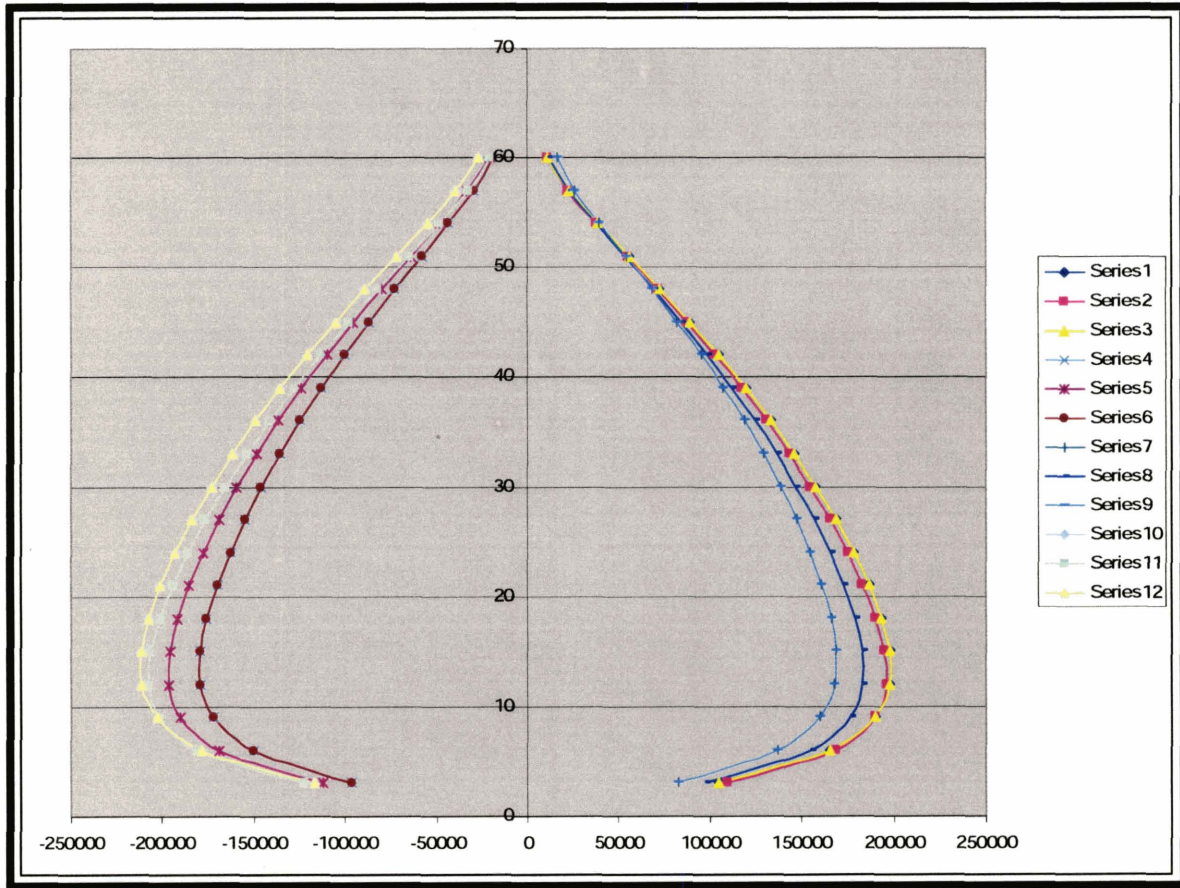


Figure 43. Axial forces of the diagonals throughout the height of the building

The plot shows that the diagonal forces seem to reach their maximum value at a height of 10-30m, while the logical thing to expect would be to have the maximum values at the base of the building. This graph brings up again the pattern that was noticed in the graph of figure 20 where the diagonal seemed to start working above a certain height. It must be noted here that this graph occurs under triangular wind load and the expected diagram would be a parabolic diagram. In order to comprehend this diagram and how the building and the knee-bracing behaves, different load cases were considered.

2.5.2. Different loads

Having a triangular load condition on a building with a knee-bracing system and extracting values for the diagonal elements of the bracing system is not very straightforward as to how this system works. For the better understanding of its behaviour the model was also analysed under different and easier loadings that are presented in the following figures:

i) Triangular load

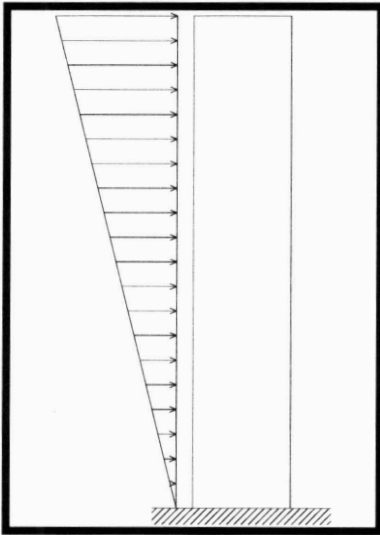


Figure 44. *Triangular load*

ii) Uniform load

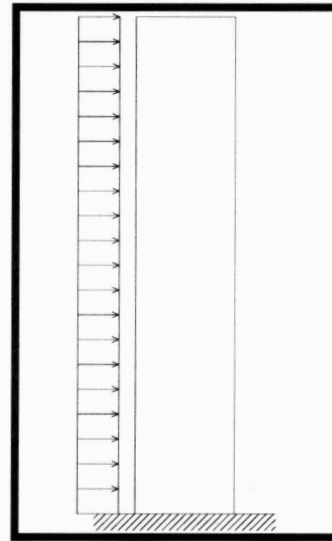


Figure 45. *Uniform load*

ii) Concentrated load at the top

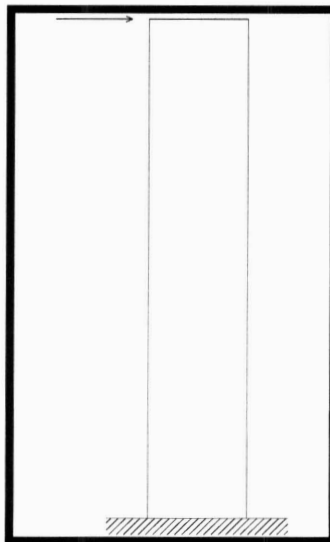


Figure 46. *Concentrated load at the top of the building*

The model was analysed under the three loading conditions and the results were obtained for the axial forces in the diagonals, which are plotted in figure 47.

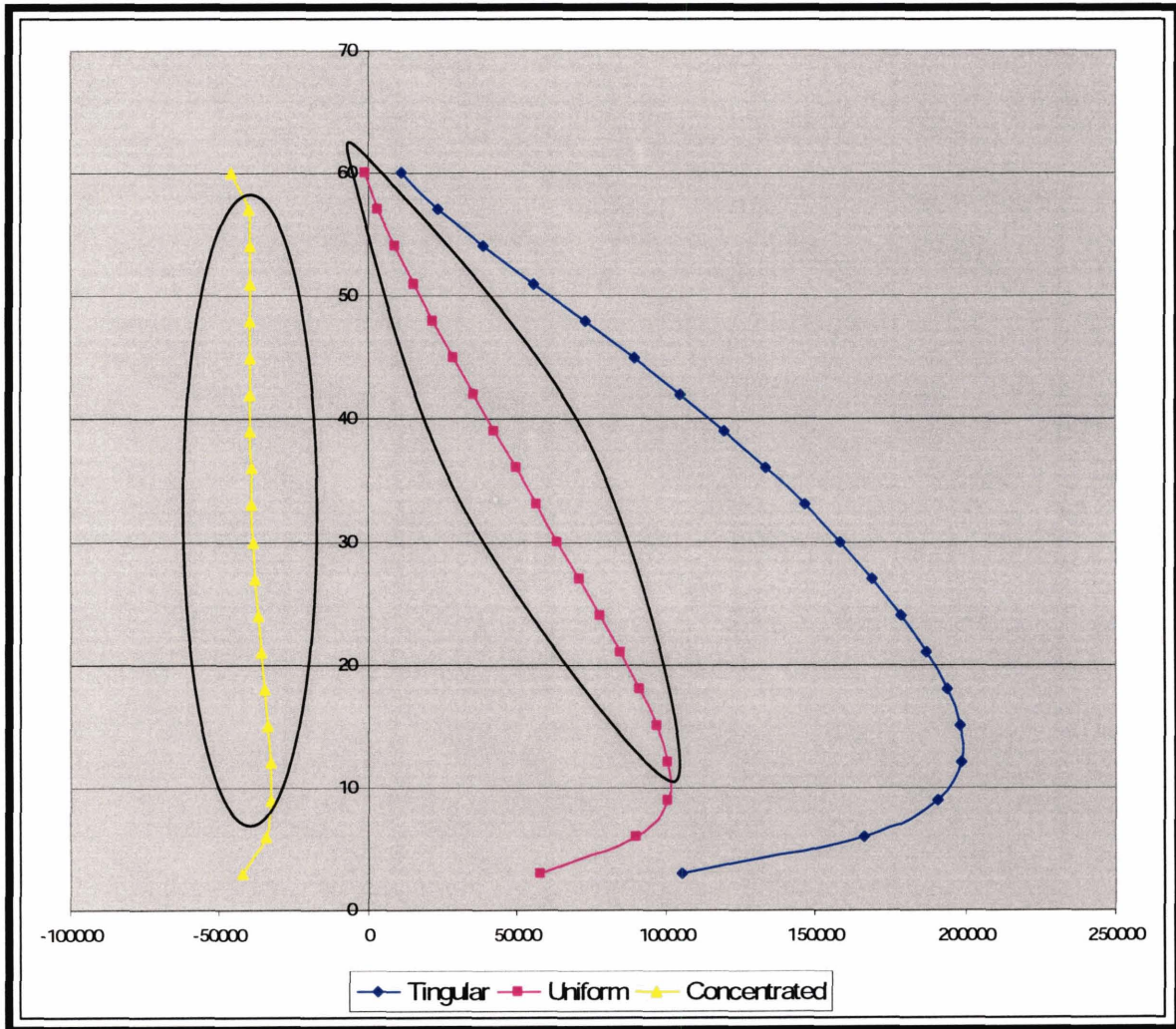


Figure 47. Results of the three loading conditions

It is obvious from this graph that the diagrams are following an expected and logical way. Firstly, regarding the concentrated load, the values of axial forces of the diagonals (or otherwise considered as the shear of the whole building) are almost constant throughout the height of the building, while for the uniform load it is linearly growing to the base of the building. That explains the parabolic shape of the shear for the triangular load as well. Regarding the strange curve at the bottom of the building, a conclusion that can be extracted from this diagram, comes out from the concentrated load diagram in combination with all the others. That is that the reaction in a building like this does not appear in the ground floor but as can be seen from above, starts to affect the forces in the building at the third or fourth floor. This height is proportionally the $1/5$ - $1/7$ of the height of the building.

2.5.3. Position of diagonals

The previous results refer to a corner column of a building. Thus, in order to eliminate the maximum amount of variables for the results and to ensure that all the conclusions and results can be applied to the knee-bracing system in general, values for the axial forces of the diagonals were extracted for the centre column as well. As presented in figure 48 below, the diagonals that are attached to the centre column follow the same pattern as the ones attached to a corner column.

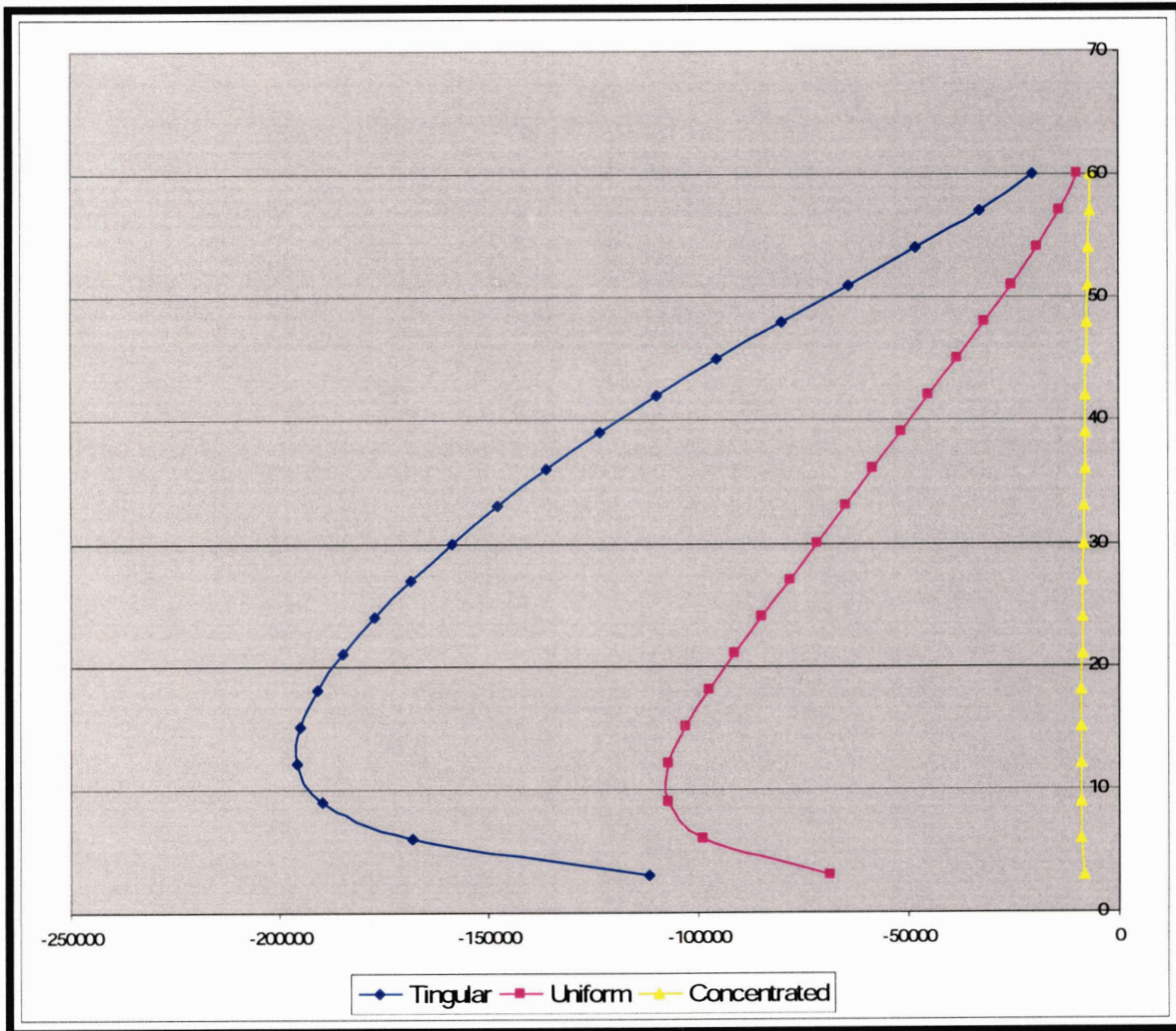


Figure 48. Results of the three loading conditions for the center column

This means that the shape of the axial forces diagram for the diagonal elements of the bracing system does not depend on their location in the building but on the loading.

2.5.4. Local change of stiffness

The previous results indicated that, there was a need to change this design so that the either the axial forces of the diagonals would be more uniform or the diagonals should be designed in a more effective and independent way. That is why the idea of changing the sections of the diagonals depending on their height came up.

The part of the building that appeared to have the maximum values of the axial forces of the diagonals was the height between 10m and 30m. So the diagonals from 10m to 30m height were changed and then results for only one group were examined.

The first change was to increase their thickness, thus increasing the stiffness. This change resulted in more force, indicated by figure 49 the results that plotted are for diagonals that are attached to a corner column.

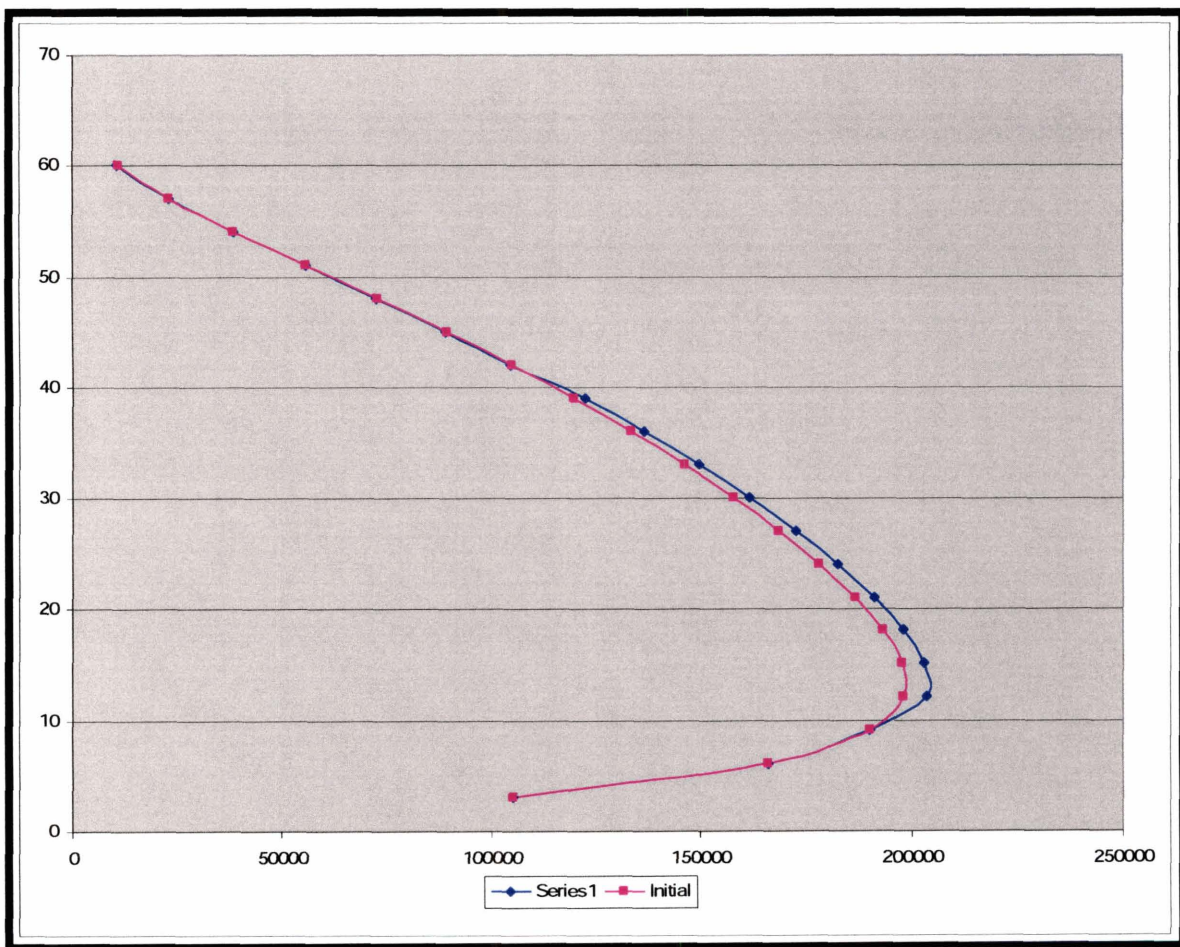


Figure 49. Increasing the thickness of the diagonals locally

However, the goal of a designer would be to have a more uniform axial force diagram in the whole height of the building. So, the next step was to decrease the stiffness of these members between heights 10-30m so that the other diagonals could be better exploited.

In the following graph it is obvious that a change like this immediately creates a non uniform diagram with some steps at the points of change.

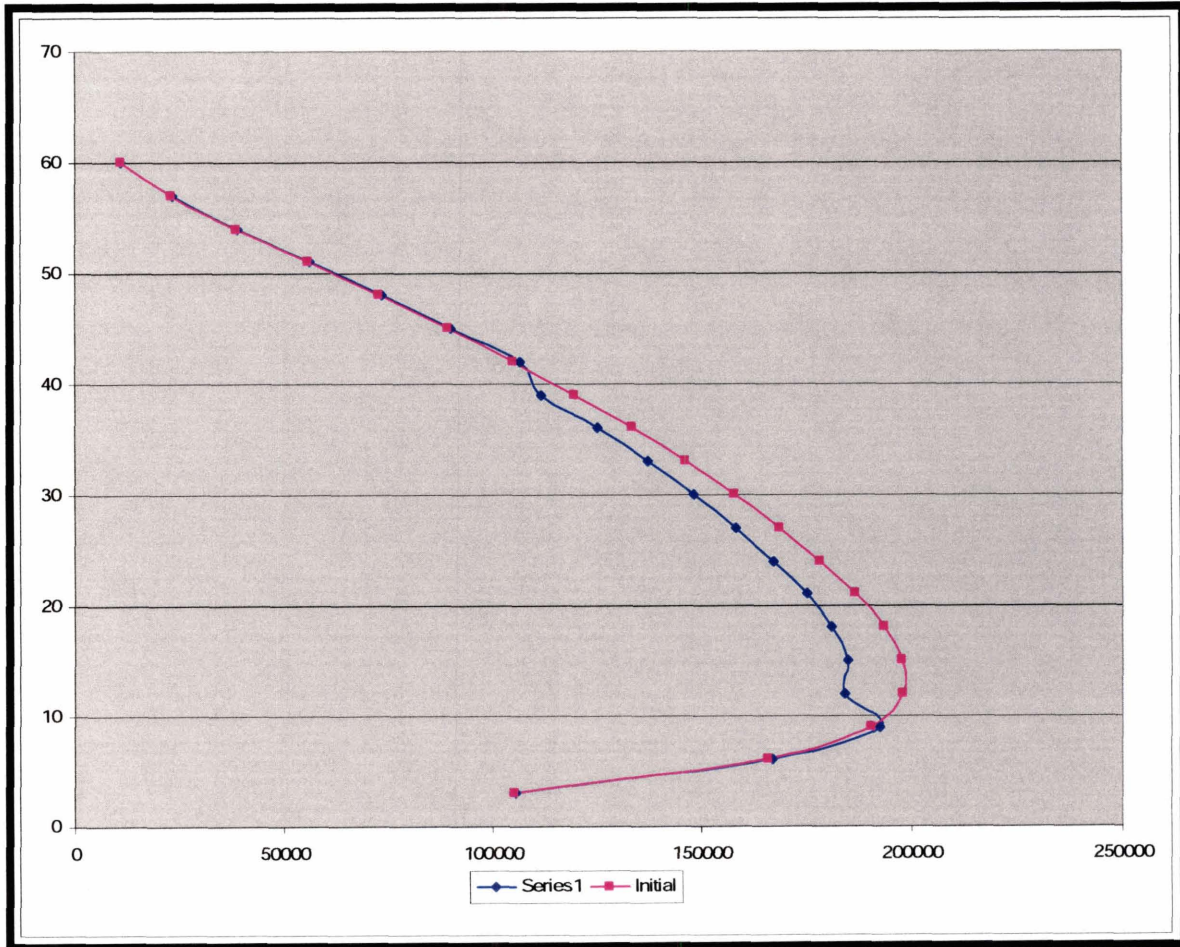


Figure 50. *Decreasing the thickness of the diagonals locally*

Concluding from the above two graphs one can say that the best way to distribute the stiffness of the diagonals is to follow the pattern of their axial loads, thus to design a parabolic stiffness pattern throughout the height of the building. This would optimize the use of the diagonals in the best way, since the needed stiffness for every point there would be designed.

2) Adaptivity-active stiffness

This chapter will try to describe an idea that the author and his supervisor had regarding the option of an active and adaptive system of stiffness for a building as the one analyzed above.

An innovative approach to the bracing of a high-rise building is definitely using the knee-bracing system. As proven the knee-bracing system can result in an effective way of bracing a high-rise building since it can limit deflections, make the design of the columns and beams more effective and with a careful design these advantages can become bigger and bigger.

However, another idea that could lead to an even more effective use of a knee-bracing system could be to have an active way of stiffening the building. This idea of adaptive structures is a field of interest that nowadays researchers are very interested in. In that way, there have been a lot of cases where active dampers have been used. This way of designing dampers makes them much more efficient and cheaper since they can really adapt themselves for every different loading that hits the building. In contrast to tuned dampers, these devices have the ability to recognize several important characteristics of the applied load, such as the frequency or the amplitude and adapt themselves likewise to produce the optimum response of the building.

Although research regarding damping has evolved and these devices are already being produced there is not much about adaptive stiffness of a building. It is considered that knee-bracing's other advantage is that there is the possibility of making the system active so that less elements are needed for the bracing of the building.

The traditional and logical way in which a high-rise building would be designed, would be to analyze the wind loads and design a bracing system, one for each main direction of the wind. In case of a symmetry the same amount and characteristics of the elements used for the bracing in one direction would be used for the bracing of the other direction as well. This means that the overall increase in the tonnage of the building would be calculated if all the elements of the bracing system would be considered.

However the way the wind loading is designed in all the load codes requires that the wind should be considered to be in both directions with its maximum value. This means that the wind would load the building either in the x direction (with the maximum value), or in the y direction (again with the maximum value). This fact immediately concludes that when the x wind will hit the building, the bracing which reinforces the building in that direction will fully work, while the bracing elements which reinforce the building in the y direction will not work at all. So, in any case there is always much more material on the structure than it will be ever needed.

The solution to this problem seems to be either finding a bracing system that braces the building equally in two directions with half of the weight and elements of the building described above, or to creating a mechanism in the structure which could move bracing

elements or change the geometry of the building in order to adapt every time to the load applied.

The idea that came up about knee-bracing was closer to the second option. It was considered that with the appropriate installations half of the bracing elements could be avoided. Taking for example a corner column, under normal and traditional design solutions, the knee-bracing elements for that column would be two as shown in figures 51 and 52 below.

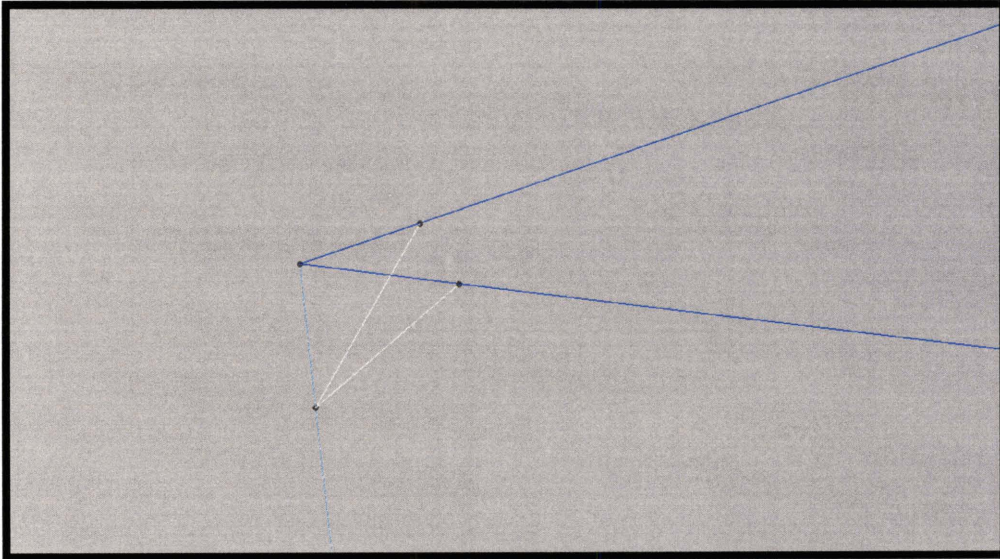


Figure 51. *Corner column with all the bracing elements*

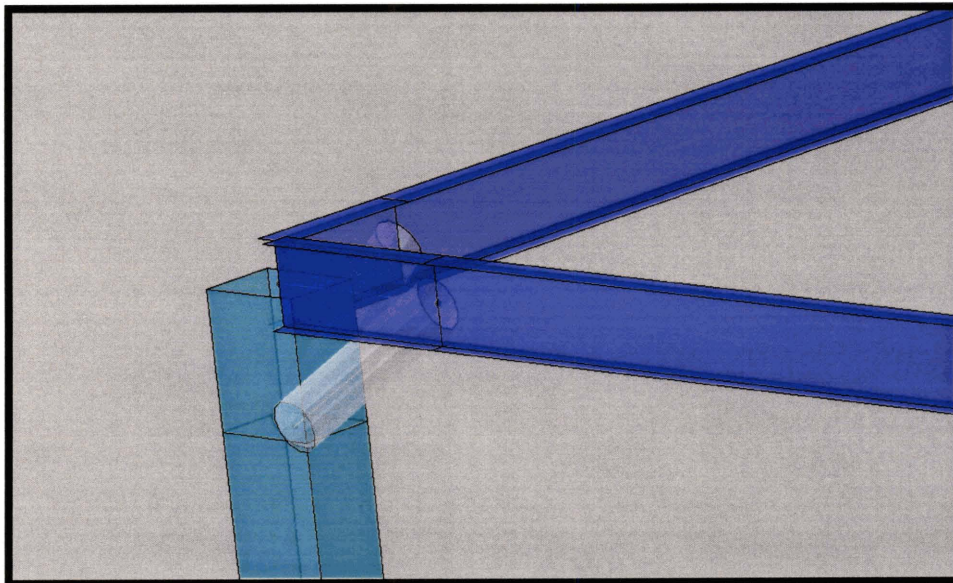


Figure 52. *Corner column with all the bracing elements*

In the case of an active stiffness system one of these bracing elements could be avoided. There would be the need only for one. There are two solutions in which this could be realized.

1. The first solution would be to create an element like a spring that could extend from the initial position close to the column and reach its final position. This element would follow already placed tracks at the bottom flange of the beams while it would be permanently pinned on the column. The fact that the ends of the diagonals are pins helps in that sense because the diagonal could roll on the track below the beam and reach the final position. When there is no need for the diagonal to be in the final position, it would be by the column, where using a device that could turn around the column, it would be ready to turn to the critical wind direction at any time. In figures these devices could be as the ones shown in figures 53 and 54.

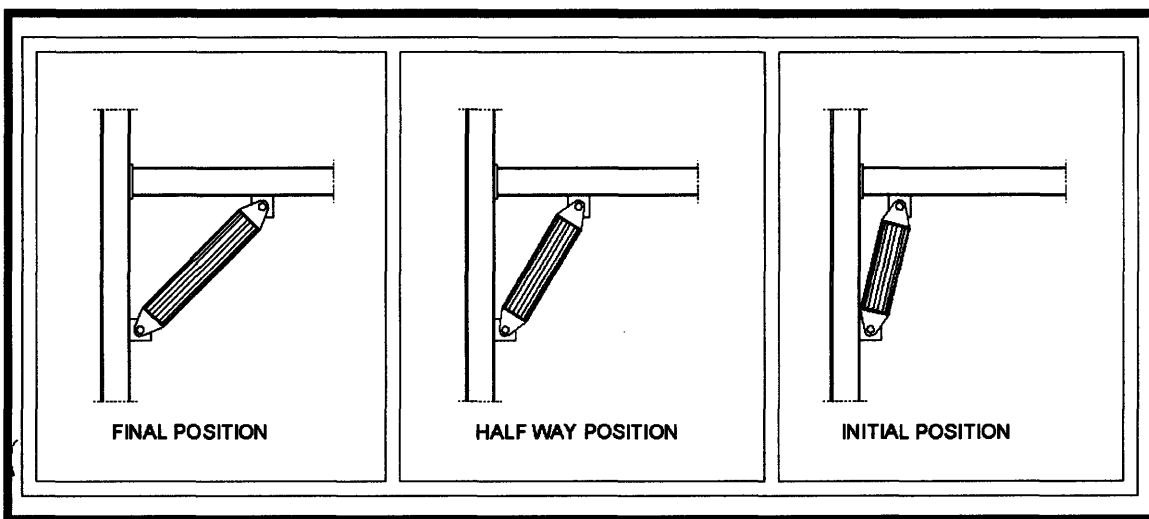


Figure 53. *Extension of the diagonal*

The different lengths of the diagonal that are shown above could be achieved by having one part of the diagonal going inside the other. This technology is used a lot in cranes.

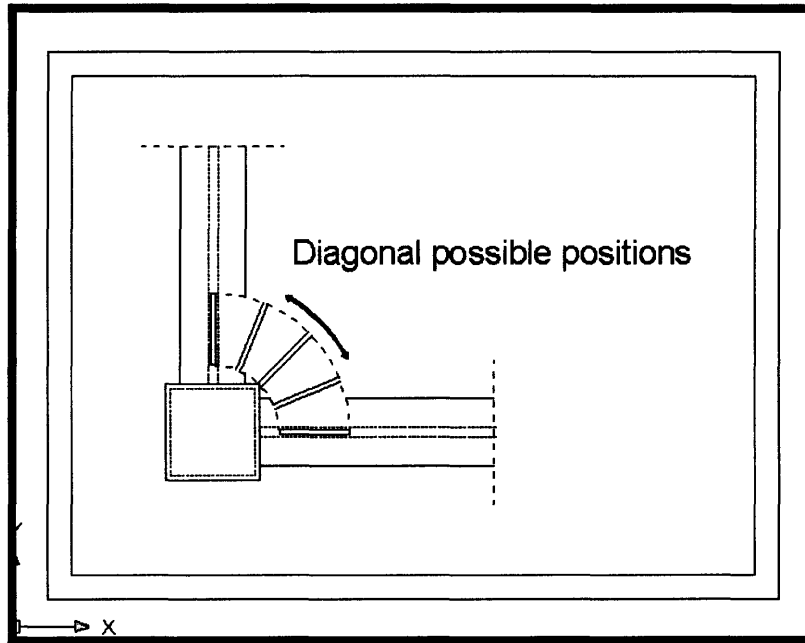


Figure 54. *Plan view of the system*

2. The second solution would be to have the diagonal permanently in the final position and not with the possibility to extend or to shrink. This would mean that a big circular plate would provide the area for this diagonal to rotate around the column. The only change from solution 1, is that here the circular plate is also at the top of the diagonal. The big advantage of the second solution is that the diagonals can also perform in an intermediate position and not only below the two beams. Something that allows the knee-bracing system to be every time completely aligned to the wind direction.

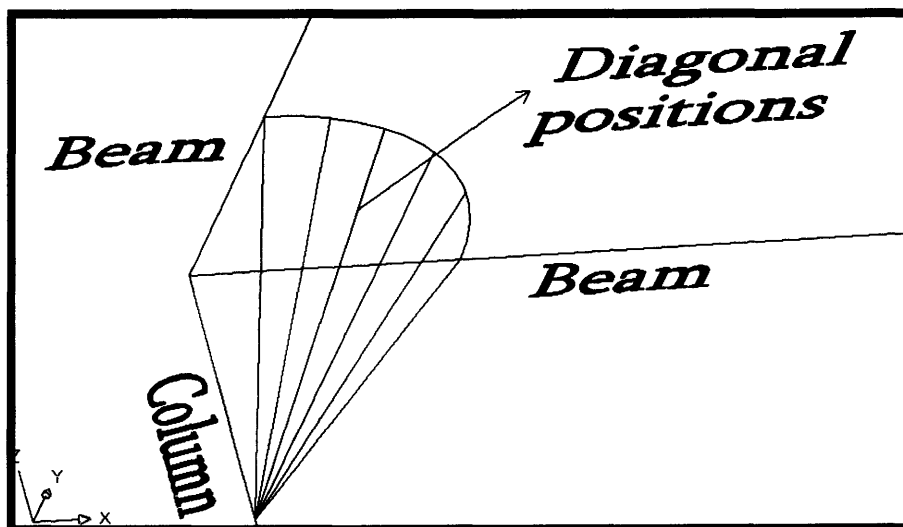


Figure 55. *Solution #2*

3) Constructability

Having described all the above about the knee-bracing system, it is necessary to give also a solution regarding the construction of a building like that.

Normally the way these high-rise buildings are constructed is starting from the foundation and then by building columns and attaching the beams they rise to the sky.

Trying to put all these small diagonals on site it could create a very tiring and difficult task. That is why a very good solution for solving this problem would be to create pieces in the shop that would include these diagonals, like a column tree. A piece like this can be shown in figure 56 below. This piece would then arrive at the site as a whole and connect to the adjacent ones. This would speed up a lot the procedure and create a very easy way to construct the building.

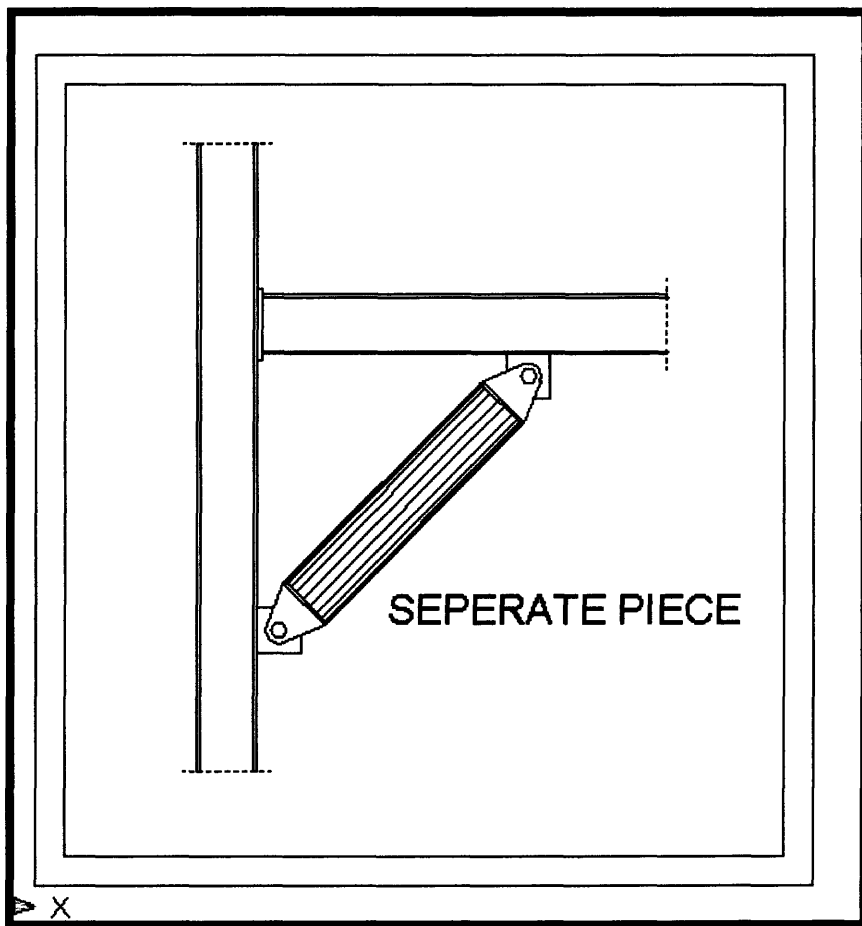
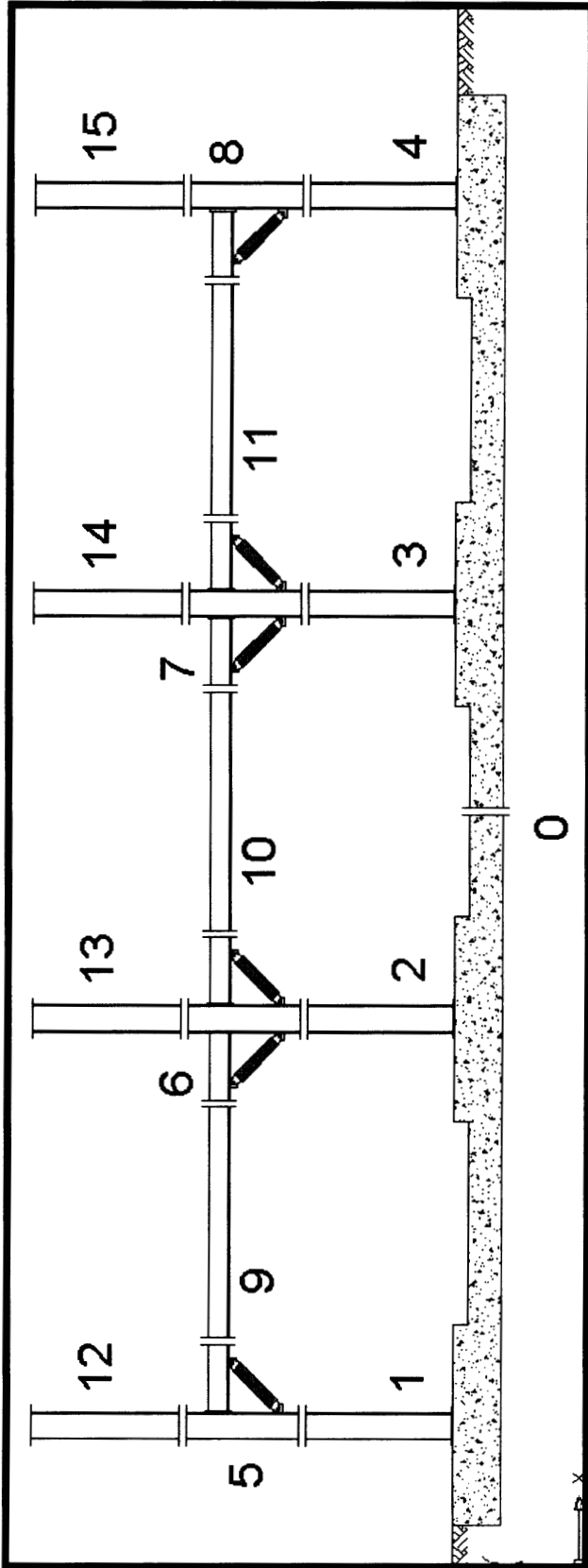


Figure 56. *Separate piece during construction*

Figure 57 shows the construction process for the ground floor. The same procedure is followed for the rest of the building.



4) Conclusions

The main conclusion that can be drawn from all the analyses carried out here is that **knee-bracing can provide adequate bracing functions for a high-rise building**. The data was gathered using a 20 floor building which was analysed for a triangular wind load. The main conclusions that all the analyses showed are the following:

1. **The deflections of the building were decreased by 30%, compared to the unbraced building. Furthermore the bending moments of the columns were decreased by 20% and the bending moments of the beams by 27%. This immediately means a decrease in the total weight of these elements, for the columns 8% and for the beams 11,2%.**
2. **The bracing of the building is achieved with a minor increase in the total weight of the structure of 2,6%.**
3. **Another conclusion is that knee-bracing is much more important for buildings with no moment capacity in their beam-column connections (unstable buildings). The effect of the knee-bracing in these buildings is very big, so in case a knee-bracing system is going to be used, there is no need for moment bearing connections at the frame of the building.**
4. **It was proved that the reactions of the building start to affect the forces inside the building from a height of 10m approximately. That is why the forces of the knee-bracing system are highest at the height of 1/5 to 1/7 of the total height of the building (from the ground).**
5. **In addition, the shape of the axial forces diagram for the diagonal elements of the knee-bracing system was proven to be independent of the position of the diagonals on the building.**
6. **The best way to distribute the stiffness of the diagonals is to follow the pattern of axial forces of the diagonals which is parabolic.**

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