SEISMIC RESPONSE OF DUAL SYSTEM IN REINFORCED CONCRETE BUILDING

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Approval sheet of the Thesis

This is to certify that we have read this thesis entitled **"Seismic response of dual system in reinforced concrete building"** and that in our opinion it is fully adequate, in scope and quality, as a thesis for the degree of Master of Science.

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ABSTRACT

Seismic response of reinforced concrete buildings

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Past and recent earthquakes have highlighted the seismic vulnerability of masonry buildings by promoting, over the centuries, the development of alternative solutions such as with reinforcement in order to improve its ductile characteristics and force distribution. One of these ideas is to merge with each other structural elements on the wall, in this case leading to "dual systems". For example, the systems "Gaiola Pombaline" or "baraccata Casa" (originally developed in Portugal and Italy), are based on the idea of reinforced structures through walls of wood - framed, then, since the beginning of the twentieth century, by author, Ph.D., S.Cattari and S. Lagomarsino and the usage rise of reinforced concrete (RC) has led to the adoption of solutions of mixed masonry buildings with reinforced concrete. The aim of this diploma is the study of structural system used in Albania, especially the comperisson in between dual system and frame system, distribution of seismic forces in shear walls as well as the effect of shear walls on the structure.

Keywords: Dual systems; Elastic Response; Plastic hinges.

ABSTRAKT

Reagimi sizmik I sistemeve duale ne ndertesat betonarme

Alushi, Flavio

Master Shkencor, Departamenti i Inxhinierisë se Ndertimit

Udhëheqësi: Prof. Dr. Hektor Cullufi

Termetet e kaluara dhe e koheve te fundit kane nxjerre ne pah dobesine sizmike te ndertesave me murature duke promovuar, gjate shekujve, zhvillimin e zgjidhjeve alternative me ane te armimit me qellim permiresimin e karakteristikave duktile te saj dhe shperndarje te ndryshme te forcave. Nje nga idete me kryesore te ketyre strategjive eshte ajo e bashkimit te elemente te tjera strukturore ne mur, duke çuar keshtu ne "sisteme duale”. Ne fakt, keto zgjidhje kane sjelle progress konstruktiv dhe evolucionin teknik. Per shembull, sistemet "gaiola Pombalina" ose "baraccata Casa" (zhvilluar fillfillimisht ne Portugali dhe Itali), jane te bazuara ne idene e strukturave te perforcuara permes mureve te drurit, i pershtatur, pastaj, qe nga fillimi i shekullit te njezete, nga autori, Ph.D., S.Cattari dhe S. Lagomarsino dhe perhapja e shpejte e betonit te armuar (RC) ka çuar ne miratimin e zgjidhjeve te ndertesave me murature mikse betonarme. Synimi I diplomes eshte te nxjerr ne pah diferencen teknike te sistemeve duale dhe atyre te thjeshta rama.

Fjalët kyçe: Sisteme duale; Spektri elastik; Cerniera plastike.

Dedicated to…………….…

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b_w	Width of web or wall thickness, mm.
a_g	Acceleration
Δ	Displacement (deflection)
F	Inertial force
K	Elastic stiffness
M	Mass
T	Period
ω	Circular frequency
β	Damping ratio
a_{max}	Maximum acceleration
v_{max}	Maximum velocity
Δ_{el}	Maximum displacement
F_{el}	Elastic forces
F_y	lateral forces
Δ_u	Inelastic displacement
M_t	Torsional moment
e	Eccentricity
CM	Center of weight (mass)
e1	Case eccentricity
L_j	Size of building floor
R	Relative rigidity
Φ	Angle of relative torsion
R1k	Relative rigidity of wall k
Xk	Distance
Ik	Moment of inertia of wall k

J	Polar moment of inertia
ρ_k	Distribution coefficient
λ	Multiplier factor
H	Horizontal force
δ	Relative displacement of corresponding measure H
Ew	Elastic modulus
t	Thickness of masonry
d	Diagonal length
θ	Diagonal angle
η	Coefficient
q	Behaviour factor

CHAPTER 1

INTRODUCTION

In Italy, after the earthquake in 1908 Messinadisa regulation (Royal Italian Decree 1909) advised the adoption of masonry that is limited only by reinforced concrete walls as the anti - seismic structure, reconstruction of some part of cities, such as Messina that proves implementation of this suggestion. However, in Italy, this explicit proposal has been neglected by successive codes, including present ones, and it is used in a very limited area. In contrast, in other countries such as those in South America (e.g. Chile, Argentina, Mexico), the enclosure reinforce concrete walls finds wide application in nowadays.

Despite this, together with structures designed for a special role for the earthquake behavior, the spread of reinforced concrete technology has led of mixed solutions inspired more by functional aspects rather than structural. In most cases these configurations start from existing buildings in which, for example:

- Reinforced concrete walls replace the interior walls,
- Reinforced concrete walls are entered to build ladders

The variety of all these cases presents difficulties not only for their typological classification, but also to standardize the structure of their scheme. Furthermore, since these interventions usually grow from a spontaneous construction, any design capacity or ductility concept is neglected and they are designed to maintain only vertical loads. It also seems important to note that in many cases these interventions are not only residential buildings but also schools and hospitals.

All previous issues make a valid vulnerability assessment. And also national and international codes (e.g. 2005 Euro code 8 - Part 1 and 3 and Italian codes for structural design of 2008) provide only short recommendations not only for their structural idealization, but even fot the criteria to be adopted for seismic safety. In addition, although there is a steady background taken from a non-linear analysis for mixed reinforced concrete buildings, setting numerical tools able to study their interaction effects are quite limited.

In this regard, existing buildings represent another issue fundamental: while new buildings can be designed and built to comply with the assumptions on which it is based model, in the case of existing structures, the possibility of simulating the actual conditions of structure presents fundamental issues. This approach starts from the main idea (supported by surveys of earthquake damage) that, referring to the reaction we plan the complex building with walls openings, it is possible to recognize two major structural components: piers and spandrels. Piers are vertical key elements for seismic loads resistant and permanent; spandrels, which are intended to be those parts of the walls between two vertical openings, are secondary horizontal elements.

1.1. Theoretical Treatment Systems Used In Building Construction

In reinforced concrete buildings the retaining structures can be classified as follows.

1.1.1. Key Retaining Structures

1.1.1.1. Systems with Frames

Is called the system in which the vertical and lateral loads are afforded mainly by frames. Frames may be, dimensional or plans. But we should be noted that the concept of plans frames is an abstraction that we make and is used only in cases where we have a complete symmetry in plan and even in height. Only in these cases we can calculate a structure in plan. In all other cases we must calculate frames dimension.

1.1.1.2. Systems with walls

This is a system where the vertical and lateral loads are afforded from vertical walls that are also called "structural diaphragm." These are designed according to schemes with ground installation.

In these systems walls can work in separate (wall system - slab), i.e. independent from each other, but can work also connected to each other by means of ductile beams that achieve their cooperation (wall system - beam), thus reducing moments in them.

In these systems walls can work as separate (wall system - slab) independent from each other, but also can work connected to each other by means of ductility beams cooperation to realize their (wall system - beam), thus reducing moments in them.

According to Euro code 8 (EC8 -2002) as a special type of wall system is distinguished the case when the walls are with large sizes transversal cross-section and lightly armored. Unlike the case with ductile walls that in conception and calculation of this type is provided a very limited inelastic behavior of the walls of this system (so we have a limited ductility).

1.1.1.3 Dual or mixed system

Is that system that withstands vertical loads helped mainly by plans frames, and in resisting lateral loads contribute partly frames system and partly structural walls.

Depending on the contributions that each of them gives dual systems are partly considered as:
-Equivalent frames, when more than 50% of the horizontal forces are afforded by frames.
-Equivalent walls, when more than 50% of the horizontal forces are afforded by walls.

1.1.1.4 Systems with core

This may be a system with walls or dual system that has very low rigidity in torsion. So we consider this type as flexible frames combined with walls concentrated near the center of the building. This system as a result of walls connection with each other has a good standing against lateral forces but it has a bug that wants a symmetrical scheme. This system is used more in the realization of high buildings

1.1.1.5"Inverted Pendulum" systems

This system is when at least 50% of its mass is located in 1/3 height of the building (from above) or when energy consumption is made on the basis of a single structural element.

It should be noted that the classification on structural types of above is simplified and incomplete.

1.1.1.6 Membranes or bridge systems etc

We note that some types or structural forms are not mentioned because they are not considered appropriate systems to use in seismic areas. The main reason for this is the large side displacements that can suffer these structures during these seismic operations.

During these strong seismic actions, linkages between columns and slabs are subjected to large bending moments and therefore they must be ductile. For this purpose, specific engineering solutions are proposed.

Side deformation or bending of structures is also considered as their classification criteria.

Side deformation is assessed by his own oscillating period.

• Often rigid structures are considered with T < (0.2 - 0.3 sec)

• Semi-solid or semi-flexible structures 0.35 <T <1.0 sec

• Flexible or elastic structures T> 1.0 sec, but again, this separation should be considered simplified and conventional.

1.1.2 Flexible structures

Some advantages of flexible structures are:

• Respond well to low periods shakings.

• easily accomplished the necessary ductility and seismic analysis is performed easily.

Some disadvantages of flexible structures are:

• React with greater constraints and distortions to earthquakes with large periods.

• is difficult to make structural elements in these buildings

1.1.3 Rigid structures

Some advantages of rigid structures are:

• Respond well to large periods shakings.

• Structural elements are easily done.

Some disadvantages of rigid structures are:

• Considerable reaction to low period’s shakings.

• for these systems, seismic analysis is performed with difficulty.

CHAPTER 2

GENERAL KNOWLEDGE FOR TYPES OF CANTILEVER SYSTEMS AND THEIR COMPONENT

2.1. Horizontal diphragm

Horizontal diaphragms are whole rigid horizontal elements that make possible the transfer of lateral seismic forces in resistant vertical elements of a building.

Depending on the conception and realization of horizontal diaphragms, they can be divided into:

-Flexible, which are characteristic in the case of relatively light slabs. They are usually made of wood or metal and rarely with reinforced.

-Solid (rigid), which are mainly characteristic in cases of reinforced concrete slab.

From static point of view flexible diaphragms behave as simple beams, independent from each other, based on walls or other vertical retaining systems that we consider to be solid. On this basis is calculated the distribution of seismic forces on those systems.

This distribution does not depend on the stiffness of walls. So the size of the horizontal forces applied on vertical structural elements does not depend on resistance and stiffness of those elements. It should be noted that the flexible diaphragms cannot be "trusted" loading and transmission of torsional moments, which occur especially in the irregular layout structures.

The rigid diaphragms can be analyzed as continuous beams with very great rigidity, which rely on flexible horizontal link "represented" from the walls or other vertical carrying systems. The rigid horizontal diaphragms are non-deformable in their plan, and in this plan they have much greater stiffness than vertical retaining structures. The presence of gaps as openings, holes, etc. weakens diaphragms and creates opportunities for constraints concentrations, this especially in places where there are structural continuity fractures.

Nowadays the realization of rigid diaphragms at all levels and floor buildings, including the cover is considered as a necessary measure and effective to anti-seismic. Rigidity of a diaphragm depends on many factors and the more important are the construction materials used, their forms and method of implementation.

Long and narrow diaphragms behave more like flexible, regardless of the material used. Their rigidity can be done by redesigned deployment and distribution of resistant carrying elements of the building or making sharing with joints, in order not to work as a long diaphragm.

It happens that the diaphragm itself to be rigid but assessed as flexible because the vertical carrying system is much more rigid [Pojani et. al., 2013].

2.2. Frames

2.2.1. Plan frames

Study of a system with frames plans requires that the scheme is absolutely symmetrical; also the method of loading should be only in one direction (*Fig. 1*). Besides these, the solution with plans frames takes no account of other frames effect and this is a very big flaws.

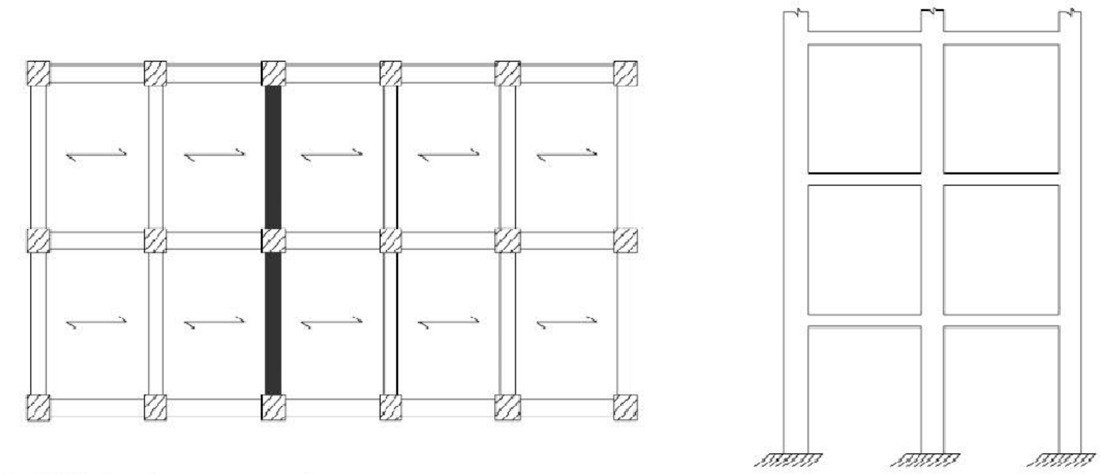


Figure 1. Plan frames system.

2.2.2. Dimensional frames

In seismic zones frames are designed such that their elements to be able to resist bending moments M and other internal forces (Q, N) occurring during the earthquake action.

Nowadays concepts of frames designing resistant to seismic actions are based on procedures Design by Capacity (Capacity Design). The basic principle of this procedure is that of "strong columns and weak beams." According to this principle is intended to create conditions in the case of very strong seismic activities, plastic hinges can be formed mainly in beams but not in columns or maximum in 1/4 of the number of columns. This is because the plasticity of any part of the column would endanger the stability of any structure. Also plastics hinges should not be formed inside the frame nodes. Nodes, being very solid, absorb great strength and so they are so much vulnerable, so special measures must be taken to the construction and reinforcement of them, not to become places of the appearance of plastic hinges [Pojani et. al., 2013].

Avoiding major deformations by shear also is part of the Design under Capacity. This is because the structural elements working to shear ductility can be achieved practically only at low levels and if the item will be destroyed from shear will not allow exploit ductile reinforcement properties.

As for columns, reinforced concrete structures are recommended that they do not settle so very dense. Most recommended are the spaces of (5 ÷ 7) m, and the distance of not more than 10 m. A further important recommendation is that of symmetry according to two axes (x, y) of the building plan. Cases of the favorite are those space systems frames "auto stabile" which form framed cross.

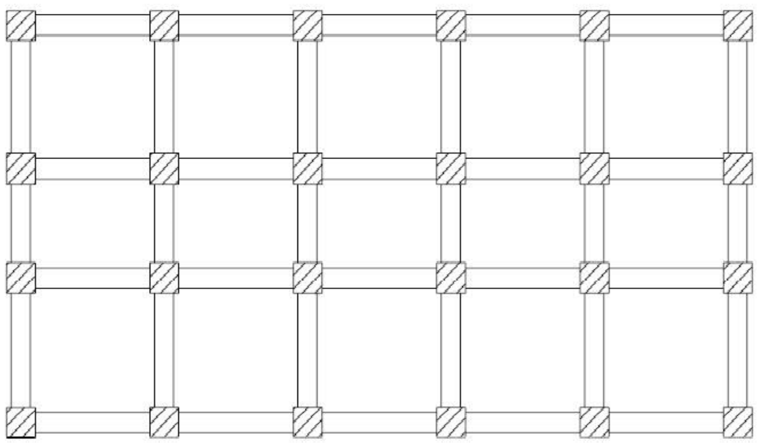


Figure 2. Dimensional frames system.

Frame type systems statically indeterminate have a greater ability to absorb and extinguish large amounts of seismic energy through plastic deformation (*Fig. 2*). This for the simple fact because before the destruction possible, in those frames should create a very large number of plastic hinges.

It has special significance the regularity of frames in their own height, and avoiding flexible floors well known as "soft story" (*Fig. 3*). Generally, the anti-seismic design is intended to avoid significant and immediate changes of the floors heights because in these cases we change rigidities as well as interruption of continuity. In case of earthquakes on these floors we have a large concentration of stresses and thus we can get what is called "soft story", so there is a shifting of the building that leads to destruction. "Soft story" can be noticed even on the upper floors that have different heights from the other floors as well as on first floor that has a height greater than the other floors. In these cases, it should definitely take special measures or increasing rigidity on the first floor (*Fig. 4*).

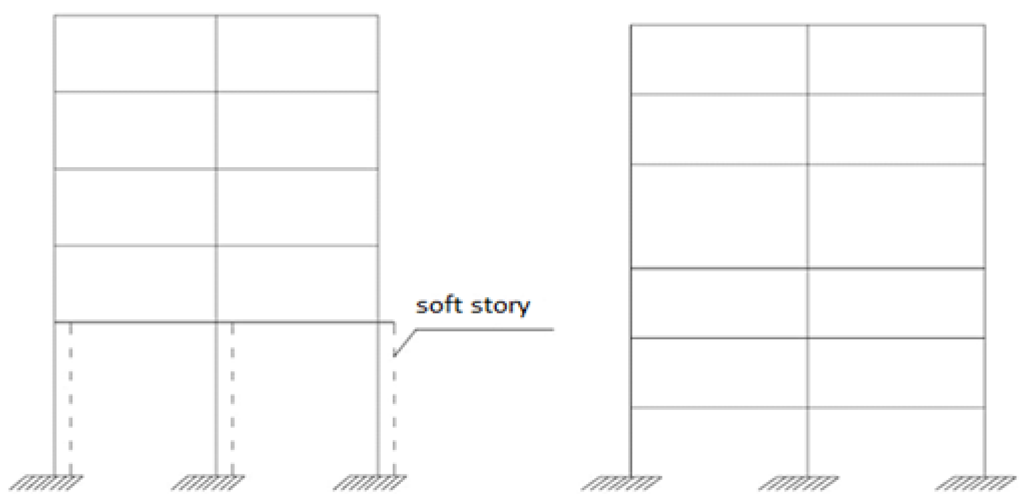


Figure 3. Systems that need to be avoided.

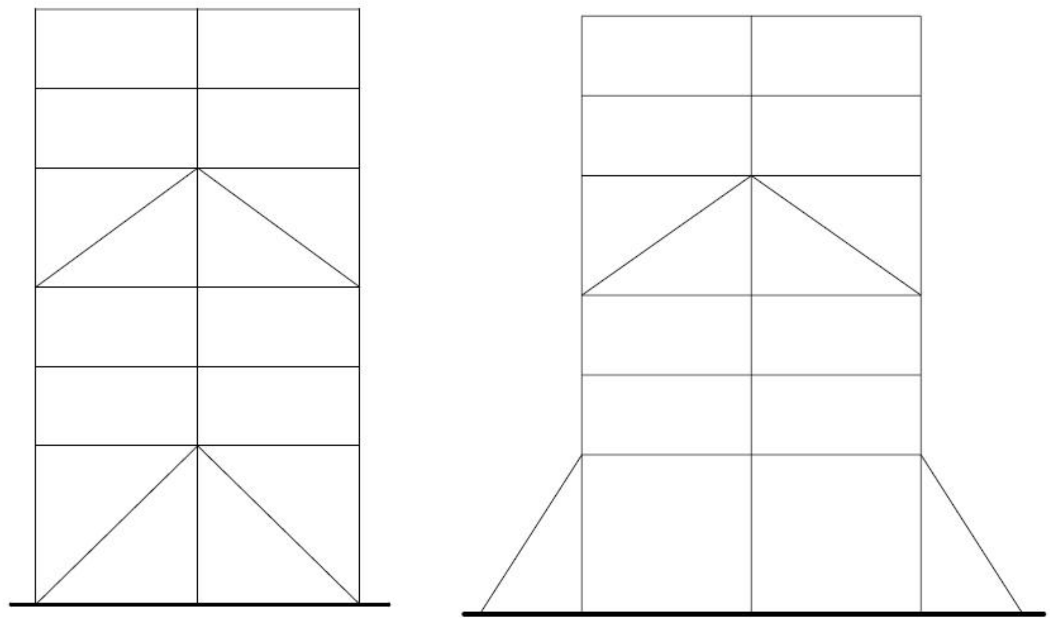


Figure 4. Possible solution for levels with different height.

When choosing the retaining systems with frames, walls etc. should be noted that, during seismic movement, free oscillation periods can be in their favor or not of a seismic appropriate response. To avoid resonance with the land, flexible structures that have a large vertical period (i.e. frames without interaction with rigid walls fillings) must not be placed on weak land. Resonance brings considerable amplification of acceleration and consequently very large deformities, often irreversible, which may lead to the destruction of non-structural elements.

Flexible constructions are recommended mainly in generally strong lands.

Non-structural elements of the frames should be designed such that tolerate without damage, deformations of the main structure. In the case of high buildings, it may be necessary to reduce the flexibility of framed structures. This is achieved by bracing columns.

Regarding frame-framework structures with masonry filling it is determined that their reaction in many cases has been unsatisfactory in weak soil and even in strong soil.

2.2.3. Shear walls

Vertical walls also known as vertical concrete diaphragms (or masonry) are designed to stand the lateral forces working mainly in shear. However, in a flexible wall is also noticed a significant bending. When walls have low height / width (H / D) ratio there is shear duty thus providing a lower level of ductility. When walls have greater height / width (H / D) ratio, bending prevails and the wall may have high ductility. The 3 cases are shown below in (*Fig. 6*) [Augenti et. al., 2009].

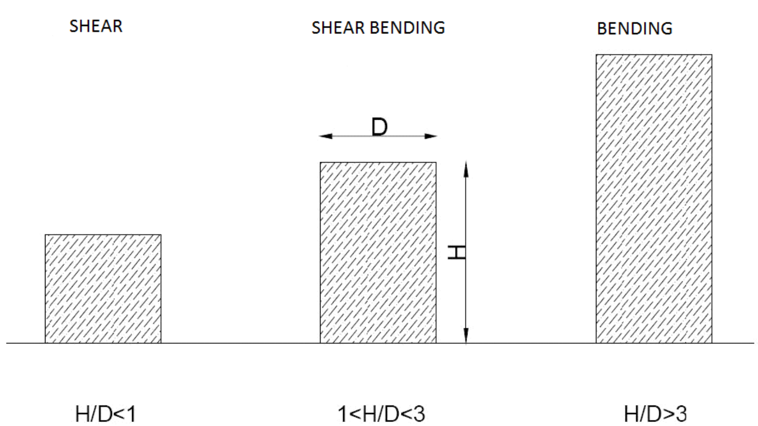


Figure 3. Duty of the walls depending on the ratio H/D.

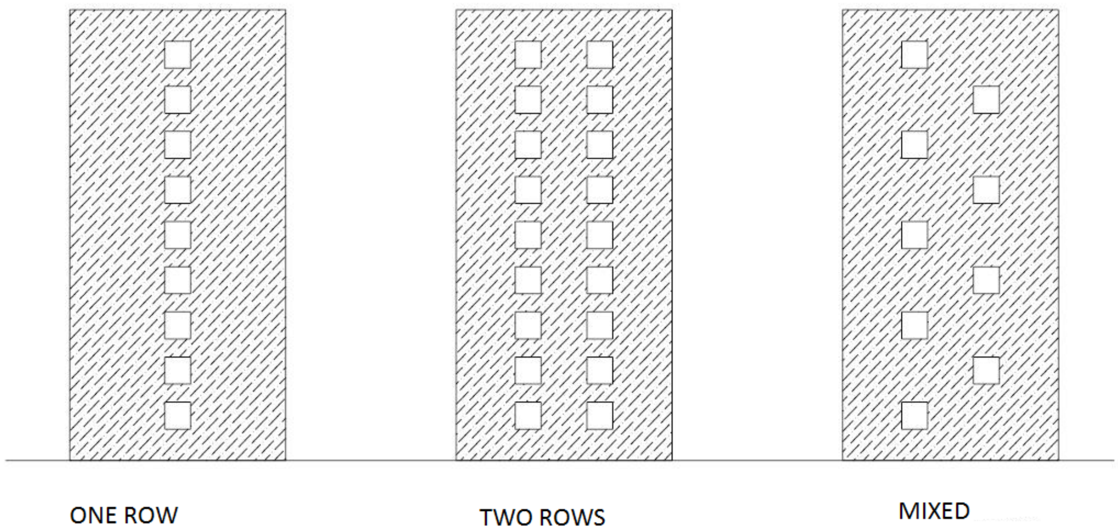


Figure 4. Forms of openings in walls.

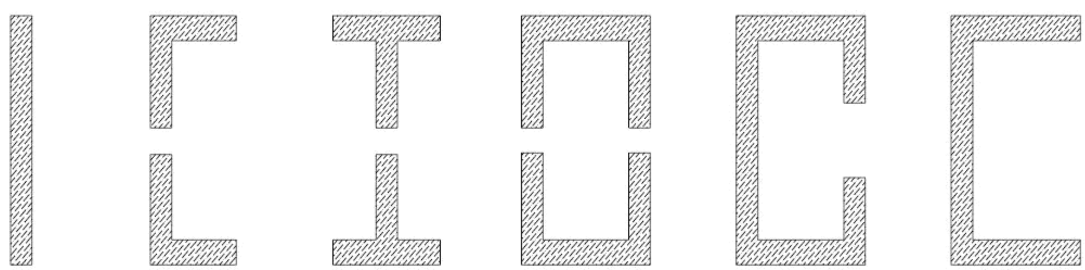


Figure 5. Wall forms.

Generally, walls are working in shear so are designed to resist shear forces. Buildings where as the basic structure are vertical walls can be shaped and varied where linear and rectangular forms are widespread. These forms due to their simplicity and symmetry are perfect for seismic activity. In the case of structures with connected walls or coupled, transfer of forces from wall to wall is made through the weak parts that work like beam binding element. Non-uniform conception of a coupled walls system and especially his eventual termination in any given floor by establishing full wall without space would be dangerous. This because during an earthquake in that floor would be focus greater strain. It may occure rupture of the complete wall which will lead to the destruction of the whole system of combined wall. This is why in these schemes should utilized the possibility of seismic energy absorption via plasticizer of connecting beams. This is achieved by designed them so as to have a high ductility level. The ductility needed is easily achieved through reinforcement with rods placed in a diagonal shape sets associated with joints to avoid their dislocation. This kind of reinforcement is verified that it is quite efficient compared to binding beams with "classic" orthogonal reinforcing. Orthogonal classic reinforcement does not provide to concrete any high ductility, which during the earthquake has led to cracks in the shape of a cross. Generally, on coupled walls systems appear two cases: bending and shear. Isolated walls connected between them beams with small bending stiffness through the action of lateral forces are working as vertical console. The distribution of shear forces F in systems with connecting walls becomes proportional with moments of inertia to the transverse sections of the wall system, based on the same character of deformation and displacement of compound walls.

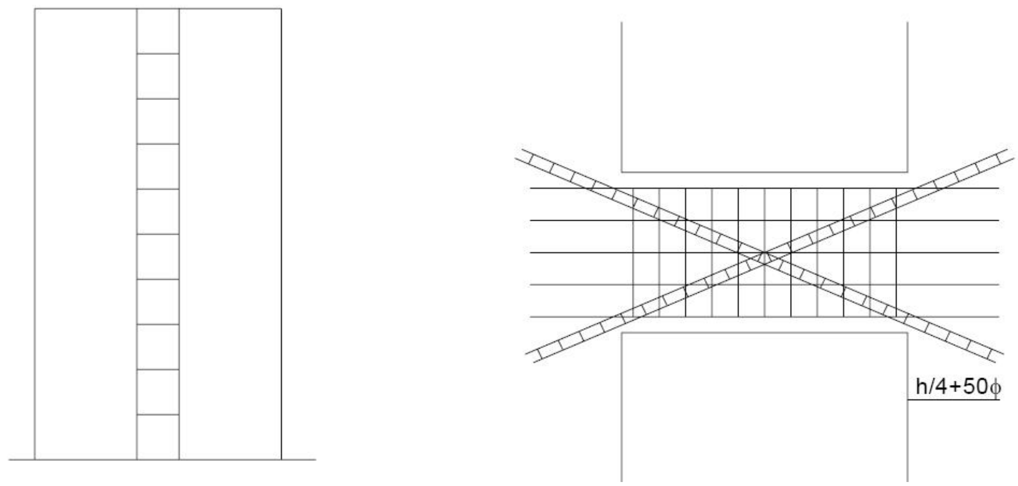


Figure 6. Coupled walls system (left), connecting beams reinforcement (right).

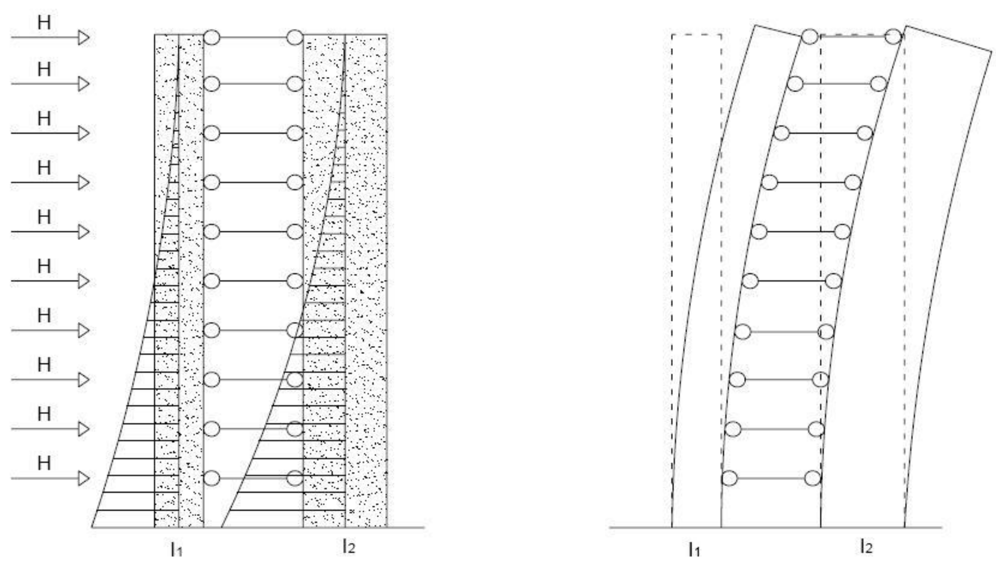


Figure 7. The reaction of a system of walls against horizontal loads, moment’s diagram (left), and displacement of floors (right).

2.2.4. Dual System

To make more rigid structures with greater side flexibility are used mixed framed structures and walls. Systems create so called "dual system". Dual systems become necessary to be applied to anti-seismic design. When resistance from lateral forces is obtained by the combination between frames and wall structures is common to refer to them as dual systems or hybrid structures. Dual systems can combine the advantages of their constituent elements.

The concept of mixed structure includes structural system that represents the vertical masonry elements and in high technology (concrete, iron, wood, etc.) situated in altitude in the same plane or in different plans. Most mixed-walled structures masonry buildings with elements represented by the technology established in the rest of the same plans with masonry.

The role of horizontal elements for the purpose of distribution of seismic action between connecting walls, used for masonry structures, is important even in cases of mixed structures. Plates solid in the plan itself fails to transfer actions horizontal walls in masonry proportionally with the hardness of each slat, each column, therefore, not participate in the absorption of seismic action, contrary deformable plates do not redistribute the seismic actions, which are distributed the vertical elements at the base of the contact surface. In cases when horizontal joints are made from wood these can be considered solid in order to be filled with reinforced concrete wall about 5 cm, which performs functions and acoustic isolation. In (*Fig. 11)* is shown how the buildings responds to (*a*) and (*b*) case which are cases thet have to be avoided and (*c*) and (*d*) wich refers to (*g*) response wgich is the ideal one. Ductile frames that are interacting with walls, where necessary, can provide a considerable amount of energy distribution. On the other hand, due to the great stiffness of the walls during an earthquake can be achieved a good control of the avoidance, including columns as shown in (*Fig. 11)* (*e*) and (*f*), it can be easily avoided [Paulay et. al., 1992].

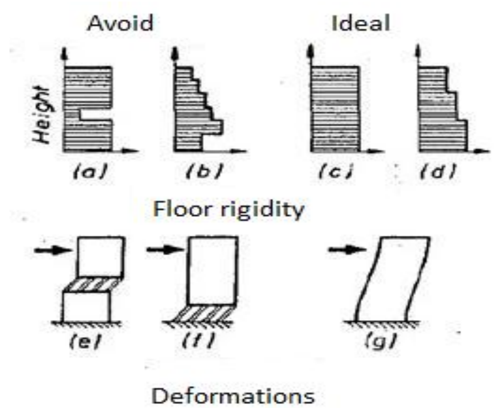


Figure 8. Interaction wall-frame.

Despite the attraction and spread of dual systems, only nowadays the researches in this field are making progression in this anti seismic designing methodology. This research, including analytical studies of existing buildings and experimental work, using static movable tables to perform tests showed a good potential to not elastic seismic response. Under the action of lateral forces, a structure can deform, while a wall can act as a vertical console with primary bending deformations, as shown in (*Fig. 12)* (*b*) and (*c*).

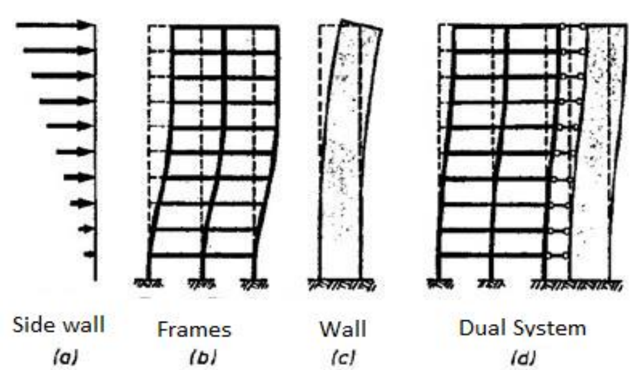


Figure 9. Distortions due to lateral forces in a frame element, wall and dual system.

Same deformations in structures require that and retaining walls at each level have the same lateral displacement, (*Fig. 12)* (*d*). Because of the proper displacement way of two elements shown in (*Fig. 12)* (*b*) and (*c*) which is modified, it shows that scatter the walls and structures of the resistance force lower floors in the lower floors, but tend to oppose each other in the higher floors. Dismissal of the resistance to lateral forces between the walls and frames of a dual system is also strongly influenced by the characteristics of response and dynamic development of connections plastic along a great seismic event and it may be quite different from that provided by a result of the subsistence elastic analysis in cases of dual systems, simplified elastic analysis can be misleading. In particular, the common practice of sharing part of the lateral force between frames and the remaining walls, each of these are analyzed independently. Although several variants affect the interaction of frames and walls are controversial, it is not possible to examine all possible combinations [Paulay et. al., 1992].

Dual systems are classified:

− With walls alternating with frames, (*Fig. 13).*

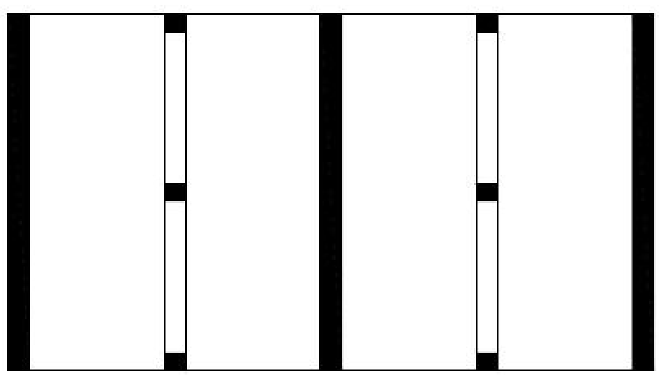


Figure 10. System with distributed walls throughout the facility by integrating the frames in plan.

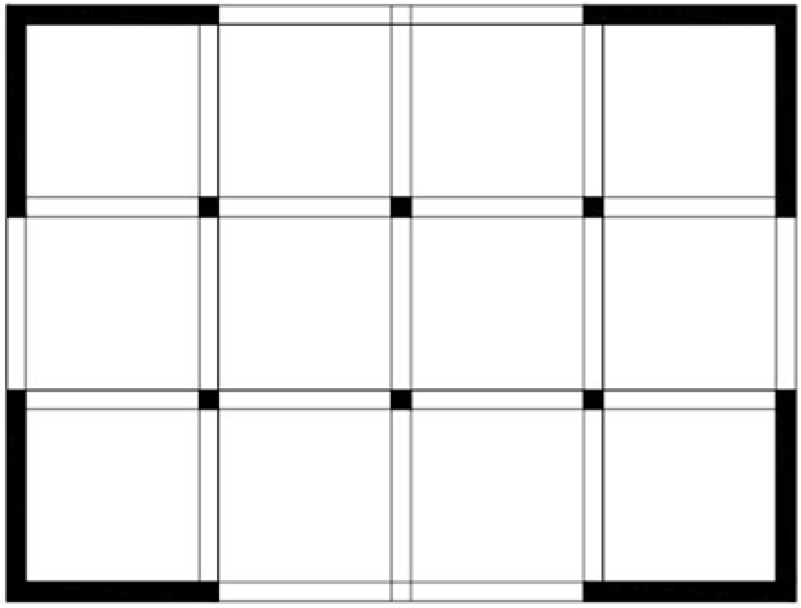


Figure 11. System with walls combined with each other in the center of the structure forming a core.

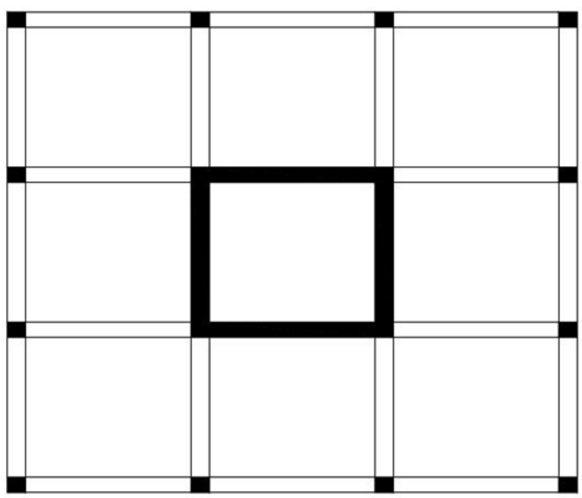


Figure 12. Centered wall

In seismic areas should be avoided systems that have open or transparent the bottom floor or any other floors in height of the building. This is used to avoid problematic flexible floors, soft and weak that cause large concentration of stresses, especially in the edges of columns. But on the other hand it is acceptable that a building can contain in the first floors wall and the other floors frames [Paulay et. al., 1992]. This solution also allows decreasing the gravitational center of the building. Dual systems are characterized by essential changes in terms and deformations of tension to their constituent components. Frames deformations are much lower on the upper floors which are compensated by a large displacement of the walls there. The characteristics of this interaction can be summarized like this: in the lower floors, the wall "holds" the frame, while on the upper floors the opposite occurs, there the frame prevent the large wall displacement. We can say that in a dual system, the function of the wall resembles of a vertical console which is recessed in the basement and has an elastic support on the upper level. If we describe in principle the method of seismic response of mixed systems it can be said that the walls which were originally due to their high stiffness, taking almost all the load side. Further, after the emergence of plasticized areas in walls, a large part of the load passes to frames. The frames ductility makes that in them is losing a significant amount of energy. Walls provided dual systems simultaneously characterized by high resistance but by a good capacity for ending (absorption) of energy thing that owes presence of frames.

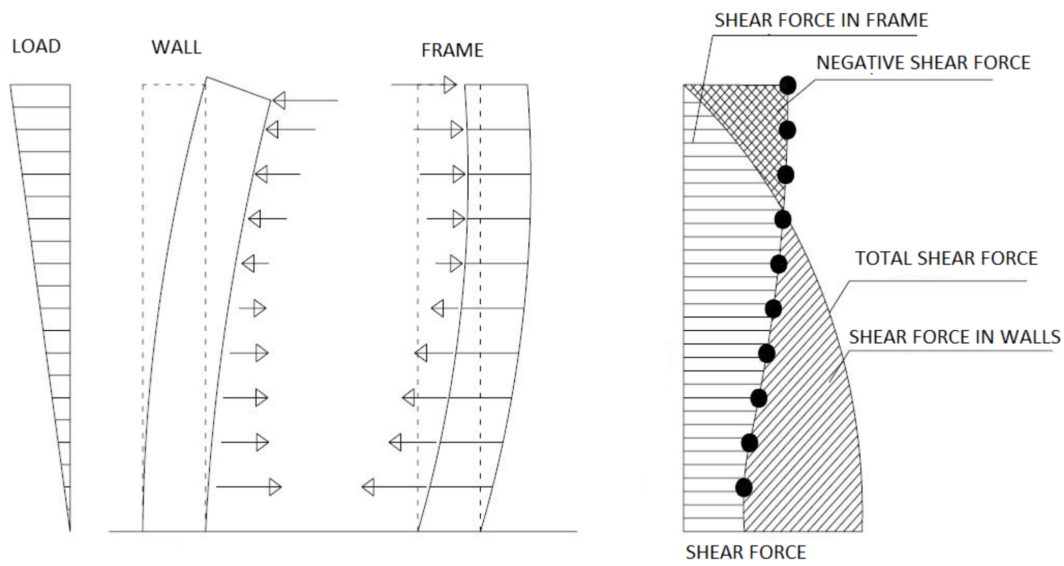


Figure 13. Dual system, frame-wall interaction.

Dual system is effective in cases where the distribution of the walls is symmetrical and uniform, even when the binding beams system between walls and frames possesses a good ductility. As is seen in (*Fig. 16)*, mixed systems can combine the advantages of their own elements. Frames conjugation with walls, can provide a significant amount of energy absorption that is required in particular on the upper floors of a building. On the other hand, as a result of greater stiffness of the walls it can be achieved a good control of dislocation of the floor during an earthquake, and can avoid the development of mechanisms that forms hinges on the floors columns. As noted above that under the action of lateral forces, a frame will be distorted first by shear force, while a wall will act as a vertical console with a mainly bending deformation. Consequently, in the case of a mixed system, simplified elastic analysis can lead do wrong paths. Especially the common practice of giving a portion of the lateral forces to the frames and the rest of forces to the walls, and then to analyze after each of them separately, is totally wrong. (*Fig. 17)*

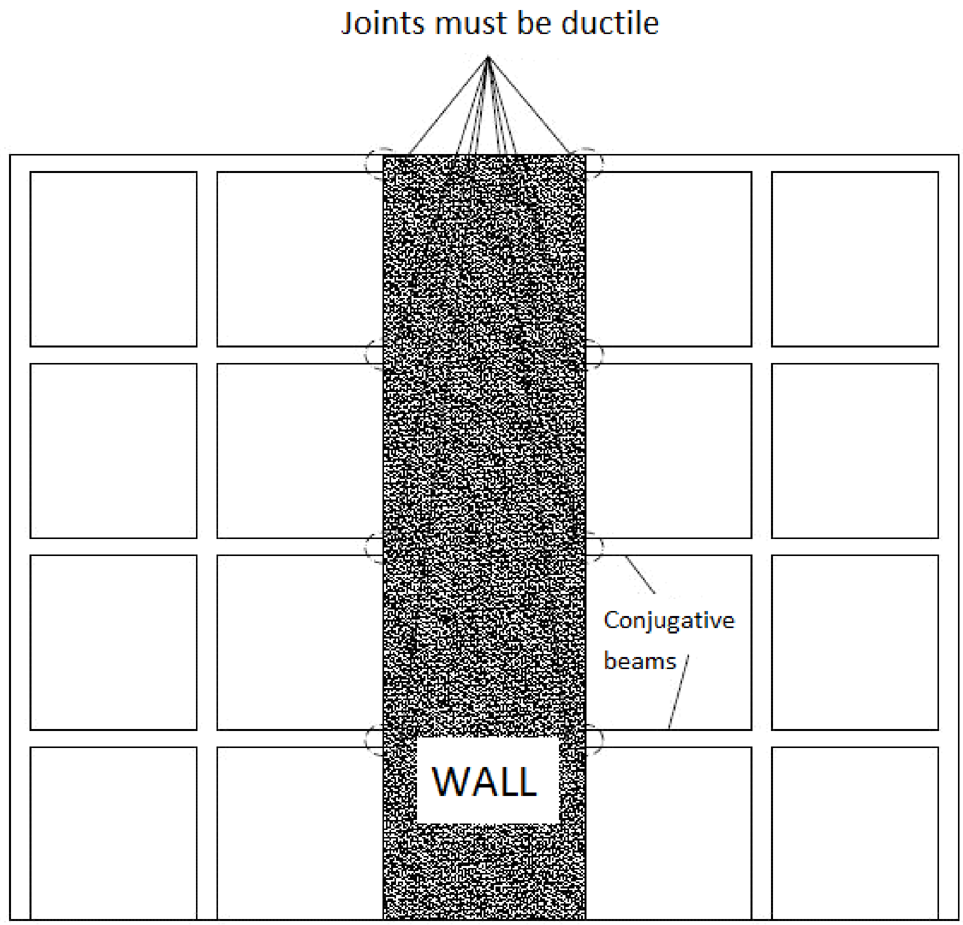


Figure 14. Dual symmetric system conjugating ductile beams.

2.2.5. Some types of mixed system

- ***Scheme 1***

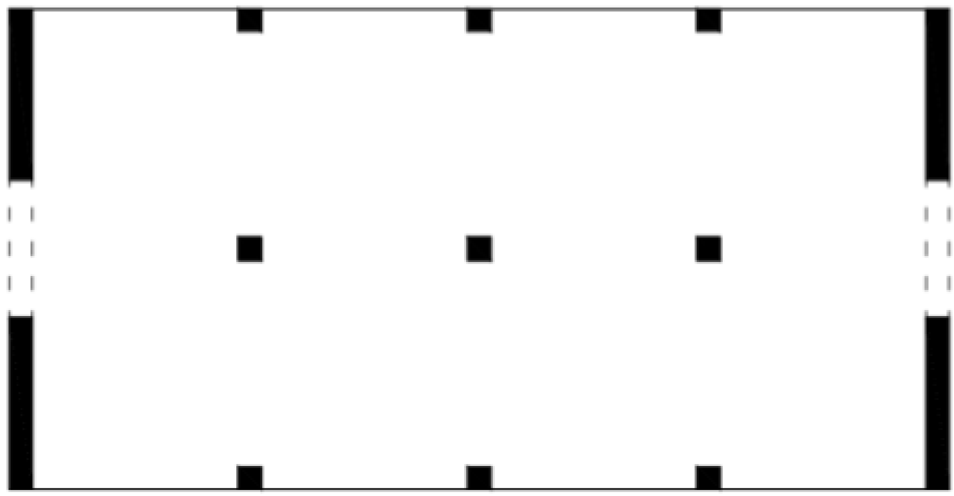


Figure 15. First scheme

In this scheme the walls can afford bending in one plan. Frames operate in both plans. Testing of rigidity in the torsion this can be considered a good scheme. (*Fig. 18)*

- ***Scheme 2***

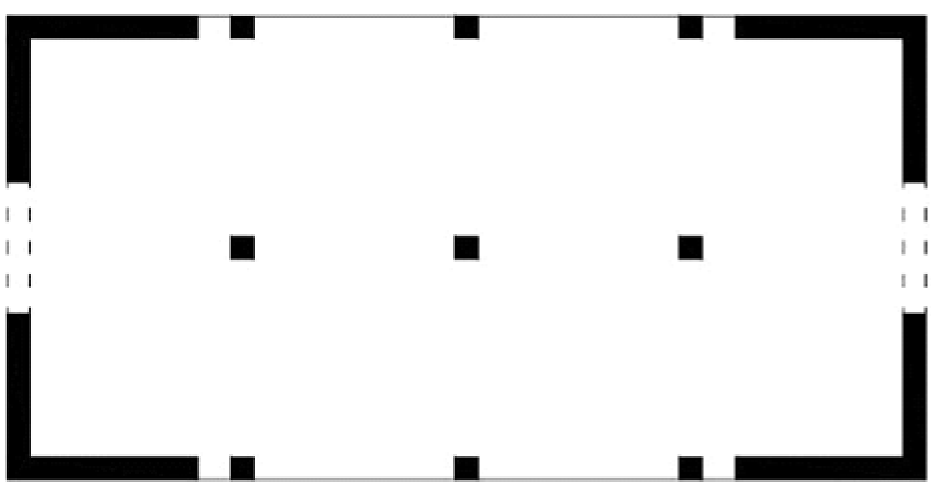


Figure 16. Second scheme

In this scheme the walls can afford bending in two plans. This scheme has good rigidity in torsion (*Fig. 19)*.

- ***Scheme 3***

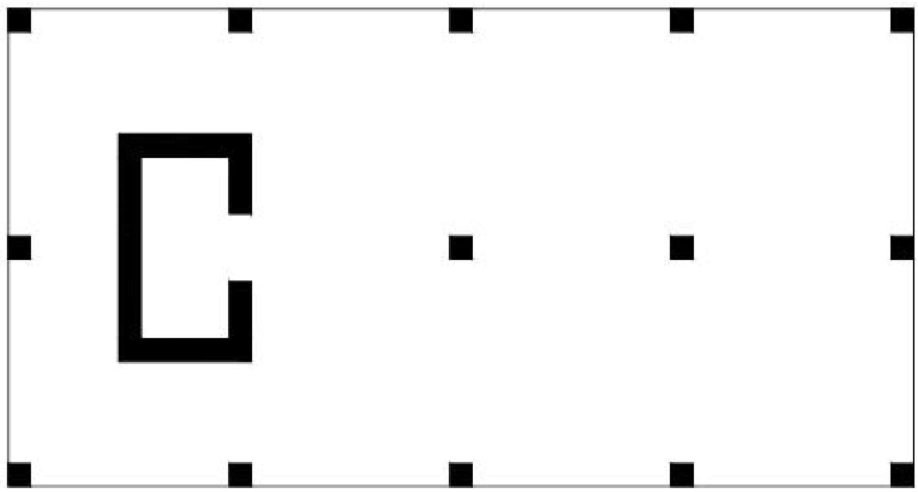


Figure 17. Third scheme

In this scheme the walls can afford moments in two directions. For rigidity in torsion this scheme has flaws, so it is not good (*Fig. 20)*.

- ***Scheme 4***

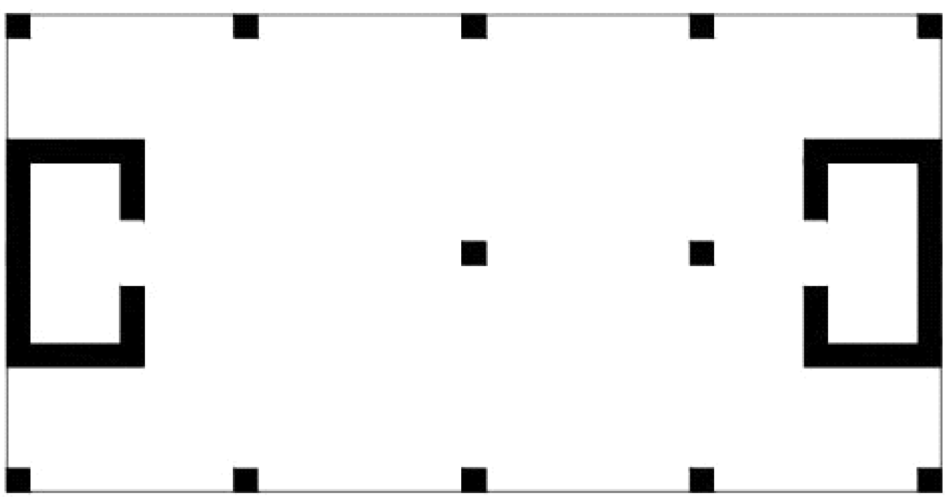


Figure 18. ***Forth scheme***

In this scheme the walls can afford the moments in two directions. This is a good scheme for torsion rigidity (*Fig. 21)*.

2.2.6. Ductile frame and walls attached to the beam

These structures can be designed as frames in which the beams conjugated with walls are elongated from both sides with infinite rigidity attached with the center of wall gravity.

Before finalizing the design of each of the elements, it is necessary to clearly identify the position in beams and columns in which plastic hinges are foreseen, to make it possible to apply designed capacity procedures.

Hinges of the beams can be held in the in both sides of the wall, and attached to it.

However, the designer may decide to allow forming plastic hinges in beam or column (*Fig. 22)* [Augenti *et. al.*, 2009].

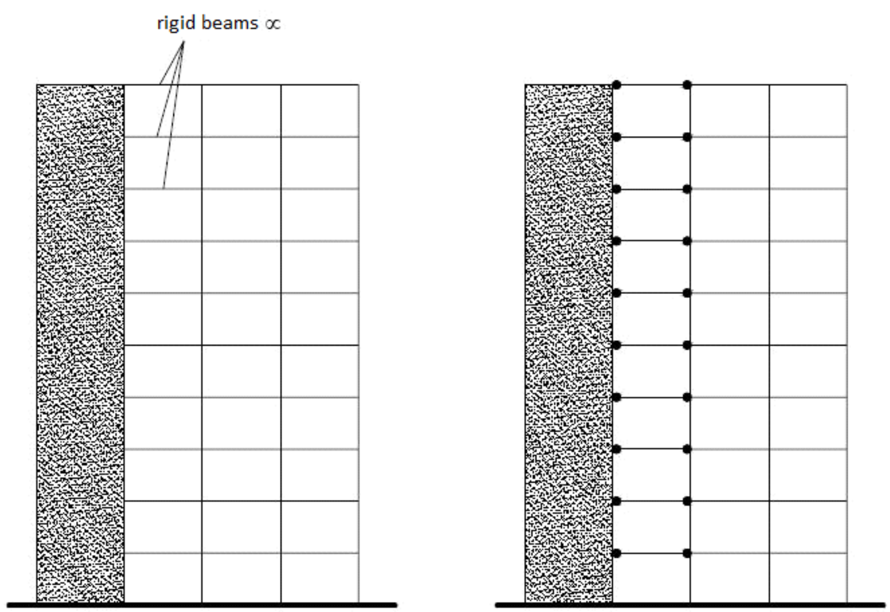


Figure 19. Rigid beams

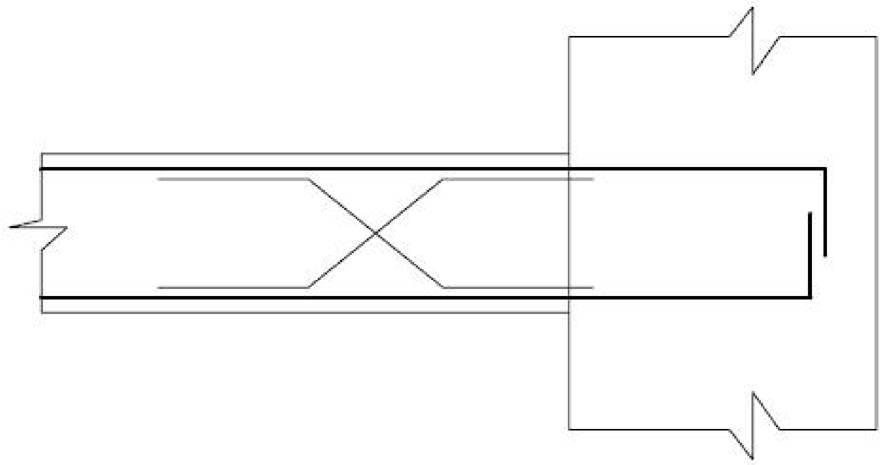


Figure 20 Reinforcement area where is supposed to form plastic hinges.

2.2.7. The walls that rotate and three-dimensional effects

It can happen that the wall can lose the connection with his own support if this support reacts badly during the movements. In the extreme case, the wall rotates as a solid element around a point near most stressed side. This can have considerable effect on the behavior of the mixed system. Due to rotation as a solid element of the wall, in each floor will happen the rotation during the height of the wall, with the same order as the rotation of the foundation raising the demand for ductility in beams frames from the side of the wall that is in stressed (*Fig. 24)*. Concrete stressed deformations alongside plastic hinges small compared with deformations of strain in the other part of the wall. Deformations of plastic wall are different in both sides resulting in a final rotation as rigid body with the upper levels and very similar to those that occur in the case of walls that rotate. This allows torsion effect to occur in near flooring systems and, in particular, will cause increase of those transversal beams that are related with the rib that works in tension [Pojani et. al., 2013].

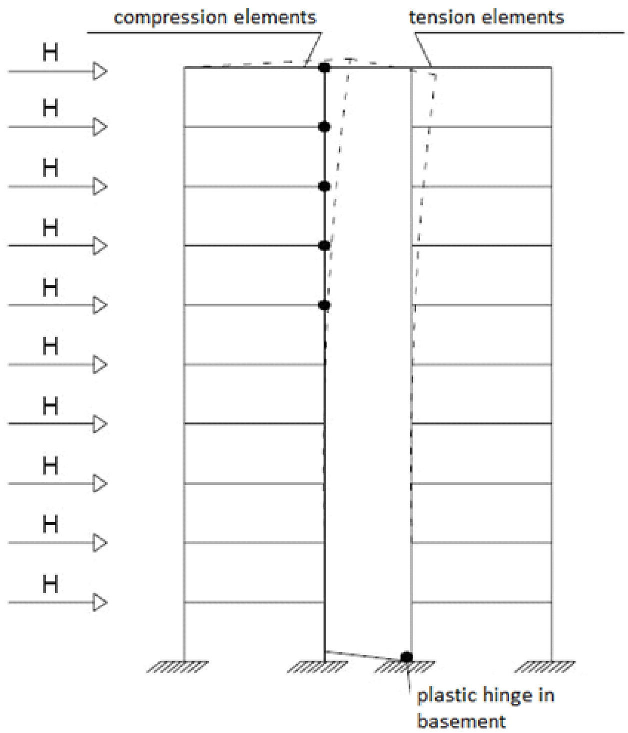


Figure 21. Different elements in in tension and compression

The most important effects of the wall rotation to be taken into consideration in mixed systems during design are:

1. Deformations induced in the transverse beams to walls, should be considered, if it is not given any primary role in the earthquake resistance. It is important that they be treated with care in the detailing of reinforcement.

2. When transversal beams have considerable bending capacity they can cause in or near rib wall a considerably eccentric force on each floor. Surface reinforcement beams will work in compression when the wall rises and in tension when we have a decrease wall. This increase in bending strength due to beams necessarily leads to increased shear

strength which must be taken into account if, in accordance with the principles of design by holding capacity should prevent a premature destruction in shear.

3. Axial force increased in the walls may need for a review of the restrictive transverse reinforcement required in sections of the wall in length plastic hinges.

4. Walls that rise (as solid) can mobilize additional mechanisms of absorption of energy, eg in beams transversal hinges plastic throughout beams to climb side withdrawn wall in his plan can take place subject to the rotational plastic large.

5. Shear forces in beams arising induce within the axial tensile force to the columns the other end of those beams. If this power of attraction is ignored, then plastic hinges may develop in those columns which cannot be properly detailed.

6. When attention is paid to the 3D effect mentioned above, the contribution of the walls that rotate in overall seismic response is favorable.

2.2.8. Frames interacting with partial-height walls

There are times when the walls for architectural or other reasons, finish without reaching the roof. This means you will have an interruption of continuity of the total inertia or the total rigidity of the building and it will bring some concerns in response building. These interruptions have consequences that can consistently be provided by elastic analysis for static forces. It is suspected that the regions (areas) of non-continuous can preserve a premature damage and the need for ductility during large earthquakes exceeds the ability of components b / a in the area to be deformed in plastic borders without significant loss of strength. On the other hand, elastic lateral force analysis shows that the structural walls of the building upper structural not intended for use. In other words, the termination of the wall before it reaches the roof can reduce the need for strength in frames the top floors, as can be imagined, even in walls interrupted height width of the wall affects the modification of moving between floors. As well as ways of capturing the wall to the bottom (the land) affects the displacement of floors. An interesting fact is that the lack of walls on the upper floors leads to the reduction of displacement between floors 2-3 floors above the wall termination. Even in terms of the values of the beam moments, varying the conditions of the earth connection to the wall for b_w <<, do not have any great effect on the beams and columns with the exception of 2-3 floors above (last). If b_w >> This impact is greater. Conclusion, for the same conditions with regard to soil walls, the contribution of partial-height walls as the flexural and in that shear forces on the lower half side in critical structure not significantly affected by the height of the walls.

CHAPTER 3

THE REACTION OF STRUCTURES FROM EARTHQUAKES

3.1. History of earthquakes in mixed reinforced concrete buildings

Seismic design of masonry structures became an issue after 1933 in Long Beach, from earthquakes in California in which the school buildings have suffered damage that could have been fatal if there will be students in schools [Augenti et. al., 2009].

At that time, a lateral seismic load equal to the product of a seismic coefficient and the weight of the structure should be taken into account in those areas like California known that could have been seismically active. Instruments that can measure peak ground of acceleration or displacement have been developed at the time, and in fact, the first strong accelerogram is scored during the 1933 Long Beach earthquake. However, there were widely used different types of accelerogram like the ground motion acceleration recording, which was measured in El Centro Imperial Valley during the 1940 earthquake in southern California.

Data for El Centro 1940 became popular and is still used by many researchers to study the effect of earthquakes on structures. With the availability of the information on how the ground moves, it was possible to determine the response of simple structures modeled as systems with one degree of freedom. Once computers were available in 1960 it was possible to develop more complex models to analyze the response of larger structures.

The arrival of computers has also had a major impact on the ability to predict the risk of movement in the terrain, and in particular, on possible risk provisions to which seismic risk model is based (NBCC)

3.2. The effects of the earthquake in mixed reinforced concrete buildings.

Mixed buildings have been used for centuries and reinforced concrete buildings have existed for about 100 years. Such buildings use masonry or reinforced concrete walls or frames to resist resist vertical loads and to resist movements from earthquakes. The floor and the roof can be made of concrete or wood. Reinforced concrete buildings built according to modern design standards are able to resist earthquakes, while some older buildings of reinforced concrete have seismic deficiencies. Modern mixed reinforced concrete buildings are also expected to be resistant by earthquakes. Free masonry buildings with reinforced concrete often called URMS, built up till 1933 in California and still being built in other parts of the America, are dangerous to earthquakes. Mixed reinforced concrete buildings have dynamic characteristics (mass, stiffness, and strength) that impact as much as tremble in the ground by the earthquake response (*Fig. 25)* [Augenti et. al., 2009].

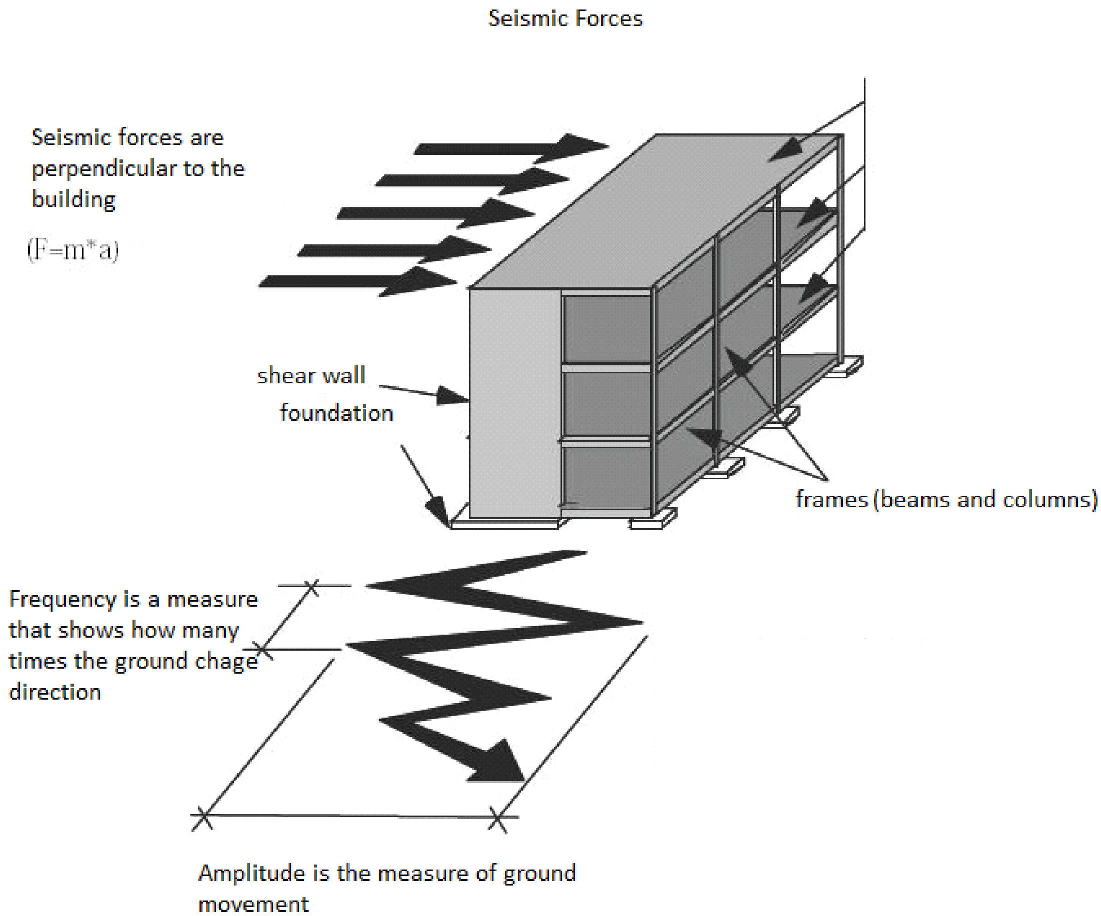


Figure 22. Distribution of seismic forces [Augenti et. al., 2009]

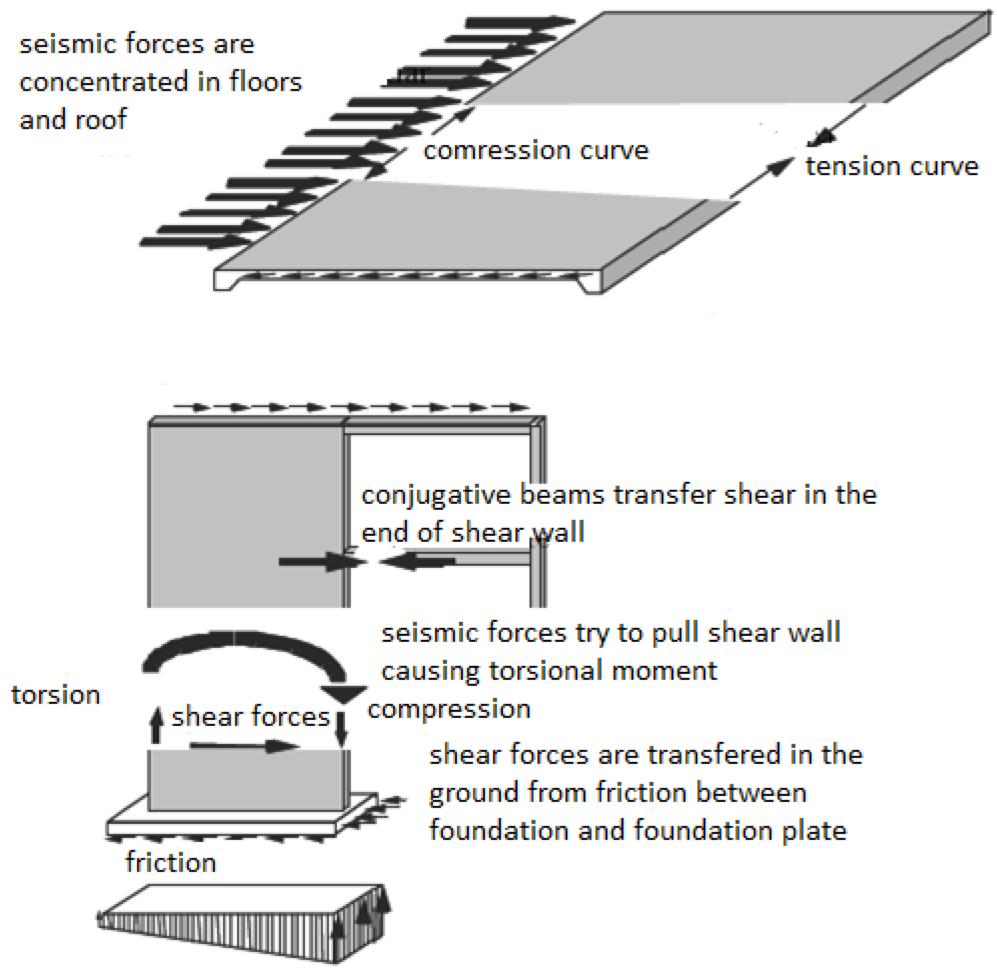


Figure 23. Seismic forces in mixed reinforced concrete buildings. [Augenti et. al., 2009]

Every building has a natural tendency to oscillate in its fundamental frequency. If one of the frequency components of the movement of the earth is near the basic frequency of the building, accelerations (forces) are amplified in buildings in the form of resonance. The direction of forces inside the building changes as the direction of land movements, causing shaking or vibrations in buildings (*Fig. 26).*

A building well built and designed has a reliable way of the load that transfers these forces through this structure in the foundation where the soil may be resist them.

Because the floor and roof elements (diaphragm) are relatively heavy, a large part of the mass of the building is focused on these elements. For the purposes of structural analysis, other measures of building components, including walls, beams, columns, furniture, and other parts of the building are normally considered focusing on slabs and roof. Horizontal earthquake forces usually have resistance from the walls or the frame elements. Diaphragmas, walls, frames and the foundation of a building are important elements that together with the engineer create a cargo way through the structure.

3.3. Generality on seismic response and energy processes

Seismic response of structures is expressed through change in time or maximum values that are taken through the action of earthquakes for different size reaction. Such are the angular or linear displacement, curvature of sections, seismic inertial forces, internal forces (bending moments, cutting forces, normal force, etc.) These are taken as a result of seismic analysis. In analytical calculation, earthquake action can be given directly through seismometer ag (t) real, artificial or synthetic representing seismic risk. But this risk can be given between the response spectrums that refer to a particular earthquake. For engineering purposes, the spectral response often given in standard forms in the different codes of anti-seismic design.

According usage of the structural model calculation, seismic response refers to two main models: elastic and inelastic. The first assumes that the structure remains with a linear elastic behavior at all times that the seismic action accures, while the second shows a real behavior that is nonlinear, inelastic, according to accepted models.

Analytical treatments of elastic and inelastic seismic response are similar between them and this is reflected in similar states of the respective equations of motion. But, of course, there are substantial differences.

3.4. Elastic response

When an earthquake hits, the base of a building is subject to lateral displacement, while the upper part of the structure is initially at rest. The forces generated in relative displacement between the structure of the upper base and cause acceleration of the upper displacement. Lateral forces on each floor seek to accelerate weight of components obtained from the floor of the vertical forces. Forces in between floors are inertial forces, not outside forces such as wind loads, and exist only as long as there is no movement in the structure

The earthquakes shake the ground for a short time, 15 to 30 seconds strong shakings, even though the shakings may continue for several minutes. The motion is cyclical and structural response can only be determined by considering the dynamics of the problem. Some important dynamic concepts are discussed below.

Let us consider a simple one store building with wall and a flat roof. The building can be represented as a system with one degree of freedom as shown in (*Fig. 27)*. Measure M that is on top of the system represents the roof and the walls weight while the column represents the combined wall hardness, K, in the direction of ground movement during the earthquake. If an earthquake causes a deflection side, Δ, in the top of the building, (*Fig. 27)*, and if the reaction of the building is the elastic stiffness, K, then force inertial side, F, acting mass M will be:

$$F = K \cdot \Delta \qquad (1)$$

The time that a structure completes a full vibration cycle is called the period, T, this for the system with one degree of freedom is:

T= 2 π √ (M/K) (2)

Instead of the period, the term natural frequency, ω, is often used in designing seismic period. It is related to the successive period as follows:

$\omega_{cps}= \frac{1}{T} = \frac{\omega}{2\pi}$ (3)

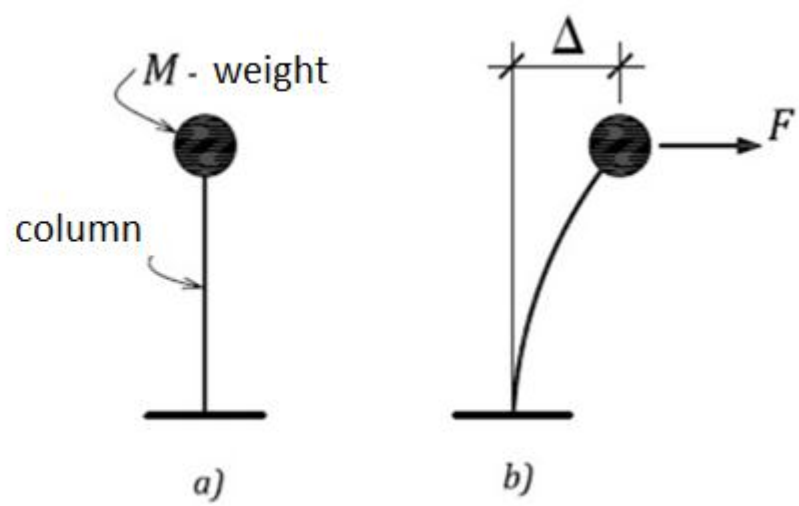


Figure 24. System with one degree of freedom: a) first position model; b) after move.

Once the structure is shaken, there is always a loss of energy which will cause a decrease in amplitude over time. This phenomenon is called damping. Extending in a building depends on building materials, its structural system and detailed, and the presence of architectural components such as partition walls, ceilings and exterior walls.

In buildings seismic designs, the damping is usually expressed on the basis of an extinguishing report, β. Damping is defined as critical viscous damping level which bring a system shifted to rest in a minimum time without shaking [Pojani et. al., 2013].

One of the most used concepts in seismic designs is response spectrum. When a structure is subjected to an earthquake movement in one moment relative displacement in the soil and absolute acceleration reaches a maximum, respectively Δ max and amax. (*Fig. 28)* shows the maximum displacement. The period of this structure is T_1. If the dynamic characteristics like weight or stiffness are changed, the period will change, and will be shown like T_2. As a result, the maximum displacement will change when structure is subjected to the same movement of terrain and the earthquake, as shown in (*Fig. 28, b*).

The repetition of the the process above for many different values of a period, and then the connection of the points as is shown in the figure below, is called the spectrum shif reaction. The spectrum corresponds to a specific movement of earthquakes and has a specific ratio of damping. The same type of graph will be constructed for maximum acceleration amax, instead of moving, and will be called acceleration response spectrum.

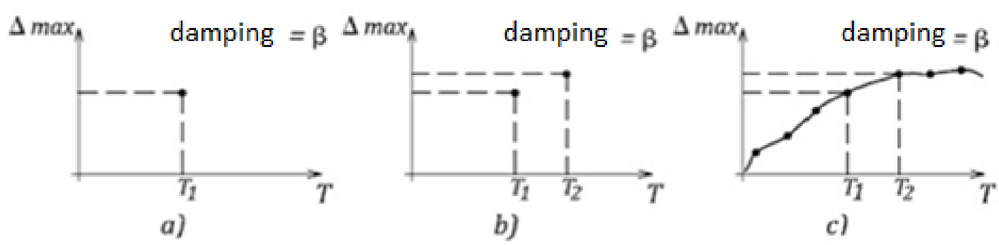


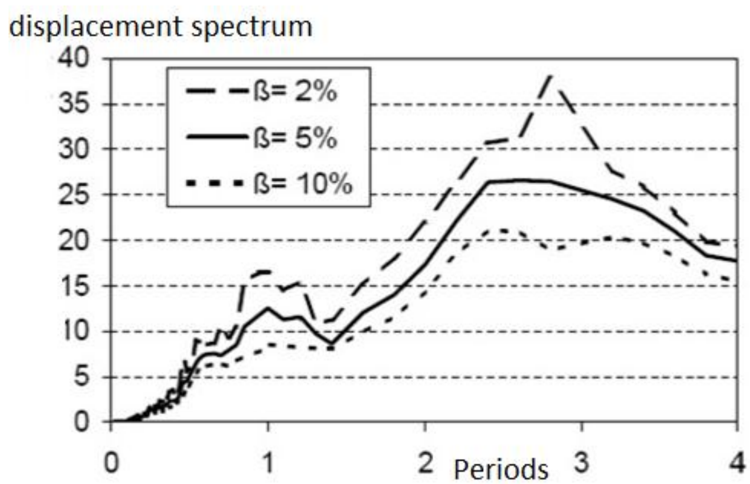
Figure 25. The development of response spectrum – maximum displacement for different periods T: a) $T = T_1$; b) $T = T_2$; c) different units of T.

(*Fig. 29, a*) shows the displacement of response spectrum for the El Cento 1940 earthquake in different levels of extinction. It is noted that displacements are reduced by the increase of damping ratio β, from 2% to 10%. (*Fig. 29, b*) shows the response spectrum acceleration for the same terms. For a small amount of damping present in the structure, maximum acceleration, amax, occurs simultaneously with maximum displacement, Δmax, and these two parameters can be related as follows:

$$a_{max}=\left(\frac{2\pi}{T}\right)^2 \ \Delta_{max} \tag{4}$$

Thus, knowing the acceleration of the spectrum, it is possible to calculate the values of displacement of the spectrum and vice versa. It is also possible to generate a response spectrum for maximum speed. Except for very short periods and very long ones, speed, Vmax, is approximated by:

$$v_{max}=\left(\frac{2\pi}{T}\right)^2 \ \Delta_{max} \tag{5}$$



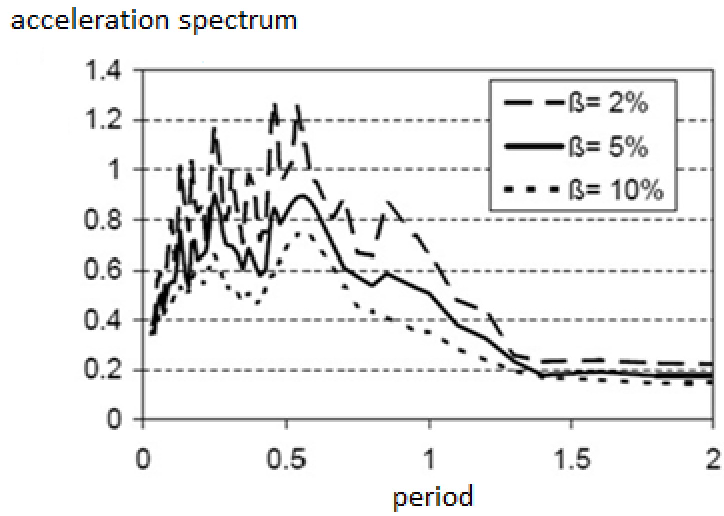


Figure 26. Spectrum of response to the 1940 earthquake EL CENTRO different levels of damping:
a) Displacement of the response spectrum; b) the acceleration response spectrum.

The response spectrum can be used to determine the maximum response of a SDOF structure, given its fundamental period and termination, for a special recording of earthquake acceleration. Different earthquakes produce widely different spectra and so it was practical to select several earthquakes and to use average score of the response spectrum as a general design spectrum. For years, the provisions seismic NBCC have used this procedure where spectrum design for a site are described by one or two parameters.

Recently, probabilistic methods are used to determine the values of the spectrum in one place for different structural periods. (*Fig. 30*) shows the 5% acceleration damping response spectrum used in the development of NBCC Vancouver

2005. This is a dangerous uniform acceleration response spectrum eg, spectrum accelerations corresponding to different periods are based on the same probability to be exceeded, that is, 2% in 50 years [Augenti et. al., 2009].

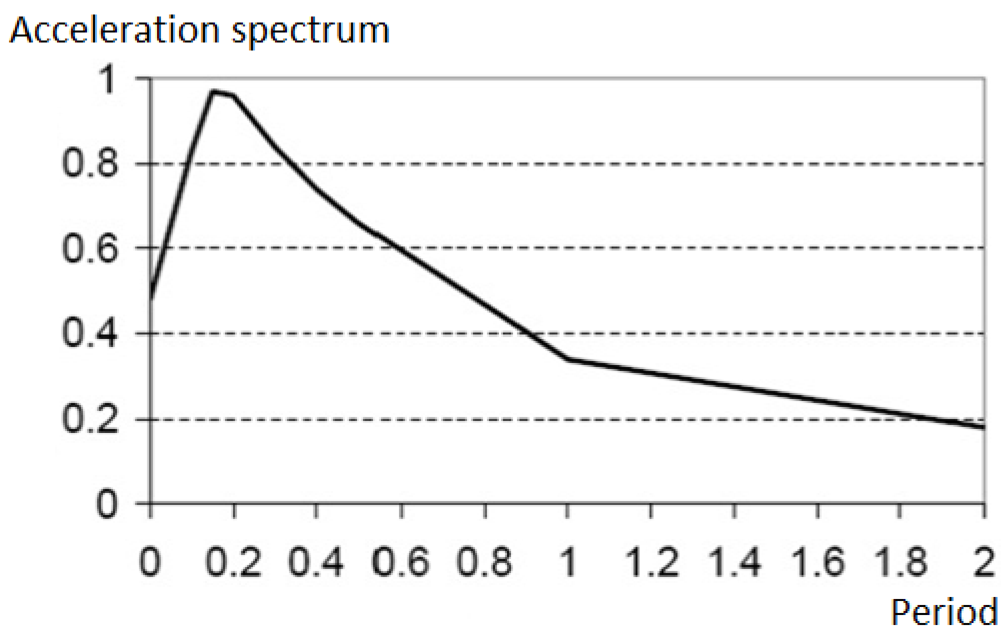


Figure 27. Dangerous uniform acceleration response spectrum for Vancuver, 2% probability in 50 years, 5% excintion**.**

3.5. Inelastic response

For any given terrain movement of earthquake and flexible system it is possible to define the maximum acceleration and the corresponding inertial forces, F_{el} elastic force, and the maximum displacement, Δ_{el}. If the structure does not have sufficient force to resist elastic forces, F_{el}, then it will give us a lower level of inertia force, that is noted in the level of lateral force, F_y. It was noted in many studies that a nonlinear structure with a cycle of force-displacement has the same reaction to that shown in

(*Fig. 31, b*), will have a maximum displacement that is not very different from the maximum displacement elastic. This is shown in (*Fig. 31, c*) where displacement inelastic (plastic), Δ_u, is only slightly greater than the elastic displacement, Δ_{el}.

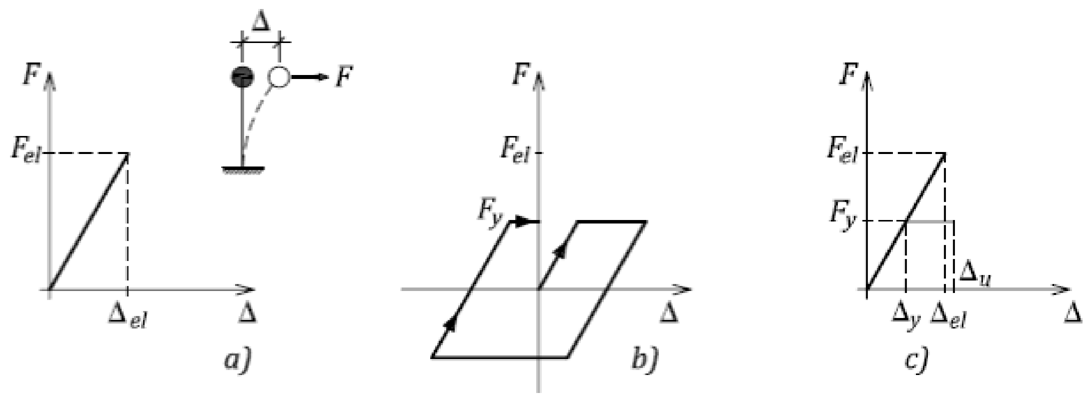


Figure 28. The relationship force-displacement: a) elastic response; b) inelastic response; c) equal displacement rule.

There are limits beyond that which can not be included in the rule of equal displacement. In structures with small period, nonlinear displacment are greater than elastic displacements, and for the structures with greater period, the maximum displacement is equal to the displacement of the terrain. However, the rule of equal displacements, in most cases, is the base of seismic provisions in which most of building codes are designed. But there is always a change in between low given forces to greater non-linear deformations. This can be noted from the (*Fig. 31, c*). At the figure we can notice the change in between non-linear displacement Δu and given displacement Δy, that are inelastic deformations, will increase if the force decrease. Non-elastic deformation in general is connected with the amount of the damage, and the one who designed it must be sure that the force will not go worse too soon in the upcoming cycles, and the simplest failure must be stoped.

This can be achived throught the seismic adding, detailed for structural component, which is usually described by the standard material. For example, in reinforced concrete structures, seismic detailing consists in adding special reinforcements that secures the good ductile performance in the most critical areas such as beams, columns and walls. In mixed reinforced concrete structures, it is difficult to ensure the isolation with similar details, and so the limits are set for limiting the reinforced space, in joints levels, and in the limits set in the masonry structural components such as walls, wich offer resistance from seismic loads.

CHAPTER 4

SEISMIC RESPONSE OF IRREGULAR STRUCTURES

4.1. General knowledge

In anti-seismic design, static and dynamic analysis of the structures are done by concepting plan models by two main orthogonal directions x,y. But it must be noted that this simplified concept is only available for the structures that have geometric symmetry and from the rigidity point of view against each axis. In these cases the torsional movement's results independent between them. Particulary in these buildings, seismic forces in direction of orthogonal axes couse only translational displacement in corresponding directions. In the cases of non-symmetrical structures we can expect union of translational displacements with torsional ones. For these structures it is necessary threedimensional analysis. So in this case the most important thing to do is to specify for every floor where the center of rigidity is. It is at this point that the responses of the structure elements are based. The difference of the seismic force and inertial force creates a situation that for the given floor, we have two forces acting on it, total seismic force F that is acting on vertical elements and the torsional moment Mt. Measure of Mt is taken as a product of force F and eccentricity "e" between CM and CS (*Fig. 32*) [Takeda et. al., 1992].

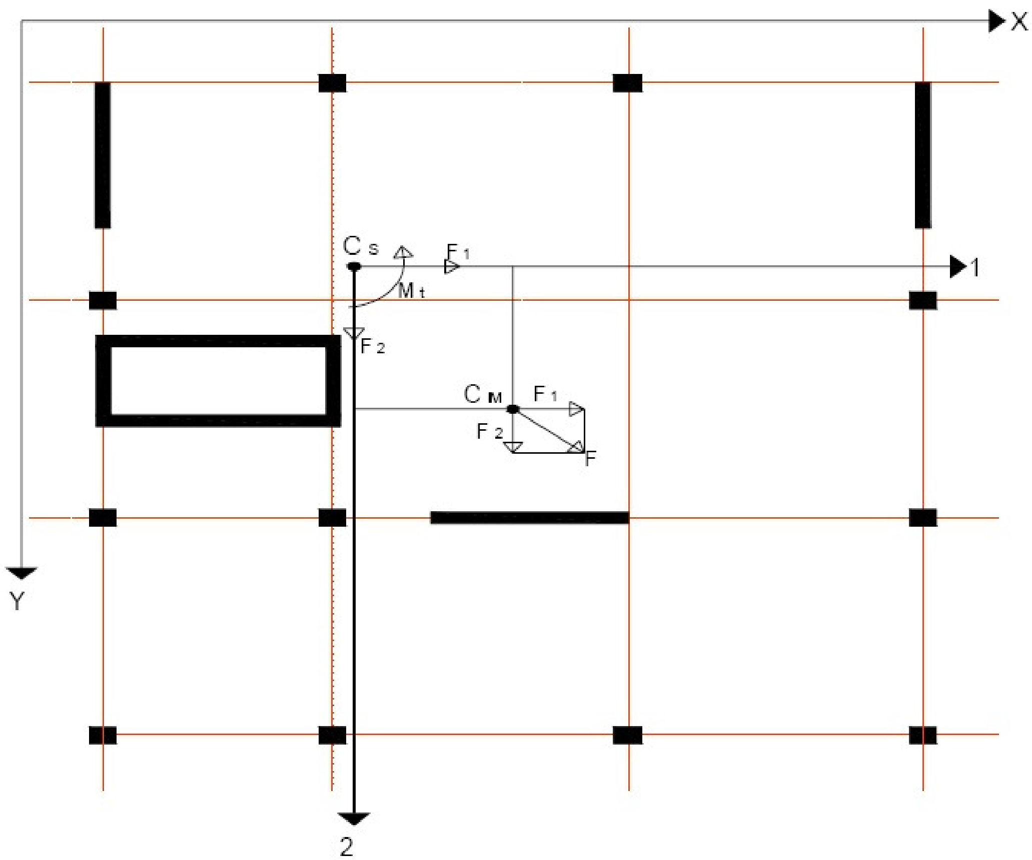


Figure 29. Plan with X and Y orthogonal directions

When the centres of CM matched with reative rigidity of the floor CS, theoretically in that floor we have no torsion. But in nowdays design practice, even in cases of buildings with full symmetry it is taken in consideration an accidental event of torsional moment. The showing of this accidental event is caused by non-homogeneity of materials used in construction, variability of actual loads, potential errors during work proceses etc. While analyzing torsional effect in anti-seismic analysis we have to take in consideration that:

1. In general, the line that connects the centres of rigidities CS in different floors of a building is a curved line in space. But when vertical elements of resistance have same characteristics in their height, this line is supposed as straight vertical line.

2. To avoid a large asymmetrically, consequently, greater torsional moments, while designing high structures we can use with effectively resistant vertical elements such as vertical diaphragms.

3. To reduce negative effect of torsion, the plan placement of resistance vertical elements must be such that the line of centres of rigidity of floors is as much as clores to corresponding wheight centers.

4.1. Accidental torsional effects

According to contemporary codes and Eurocode 8, even structures that are defined as regular in plan and in height, center of weight CM of each floor "j" is considered accidentally diplaced in the direction of initial anti-seismic analysis. The size of this displacement is taken as case eccentricity and is marked with "e_1". According to Eurocode 8 the size of "e_1" is taken in every floor "j" is equal to:

e1j = ±0.05 Lj, where Lj is the size of the building (floor) in perpendicular direction with the considerated seismic action. In every case, size of "e_1" should be added to the current eccentricity of the floor [Pojani et. al., 2013].

$$e_x = \pm\, 0.05\, L_2 \tag{6}$$

$$e_y = \pm\, 0.05\, L_1 \tag{7}$$

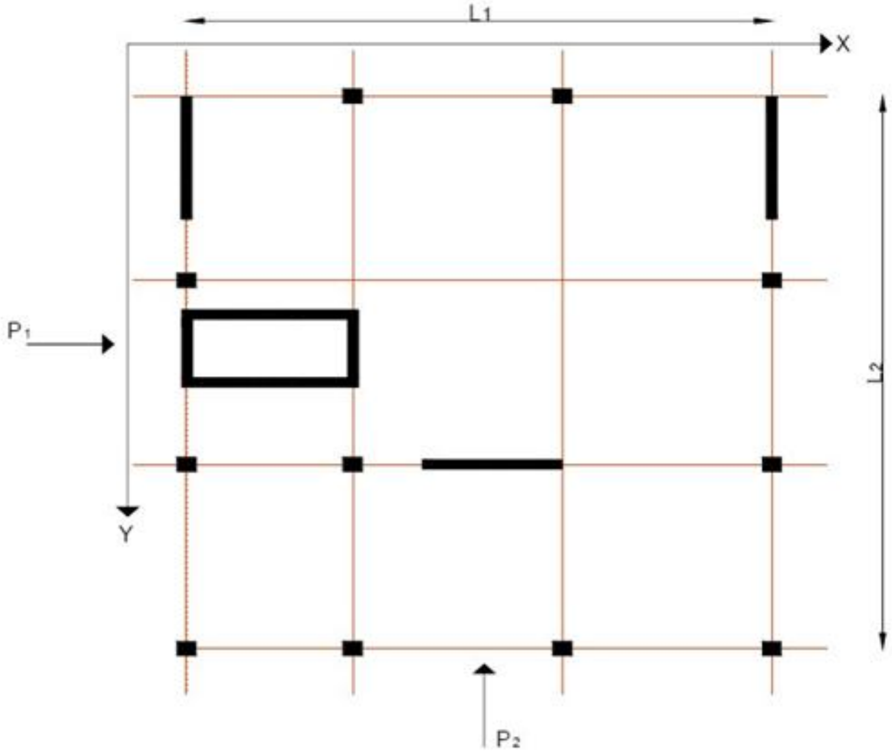


Figure 30. Accidental torsional effects

4.2. The distribution of forces in special cases

Let us examine the case when vertical elements of structures, despite of their cross-section, square, rectangular, Tprofile, I profile etc, they have their central axis parallel with two orthogonal axis x, y.

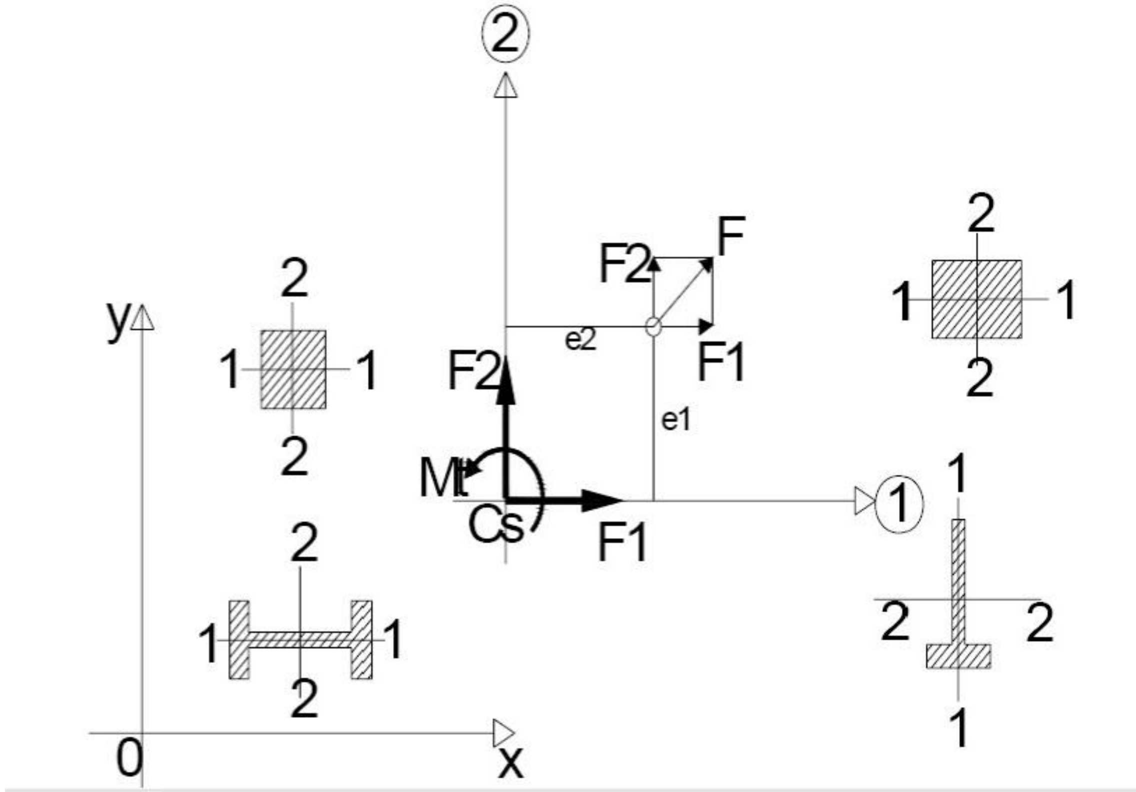


Figure 31. Relative rigidity of R1 floor

Let remember firstly that, the notion of relative rigidity R by a certain direction 1 or 2 of any vertical element such as wall of columns, placed in between two levels of buildings, is concepted as equals with shear force. For the case of (*Fig. 34*) we can say that relative rigidity of R_1 floor is taken from the sum of each element, obtained by direction of 1-1 axis. Same as this will be obtained rigidity of floor R_2 by 2-2 axis. R_1 and R_2 are forces that respond to same displacements. Their evaluation is taken from:

$$R = M_t/\phi \quad (8)$$

In this formula ϕ is the angle of relative torsion of the floor that we are analyzing, while Mt is the torsion moment that derive from F_1 and F_2 forces applied in center CM. so, taking in consideration (*Fig. 34*) we have:

$$M_t = F_1\, e_2 + F_2\, e_1 \tag{9}$$

Relative rigidity R of structural elements is propotional to section rigidities e_l (e - module of elasticity and I - moment of inertia). This fact will be reflected in the expressions below, calculating and determined in conventional manner instead of R.

Sipmlifing the scheme considerating that seismic motion is given only by direction 1-1 and the structure is only with walls in that direction as (*Fig. 35*).

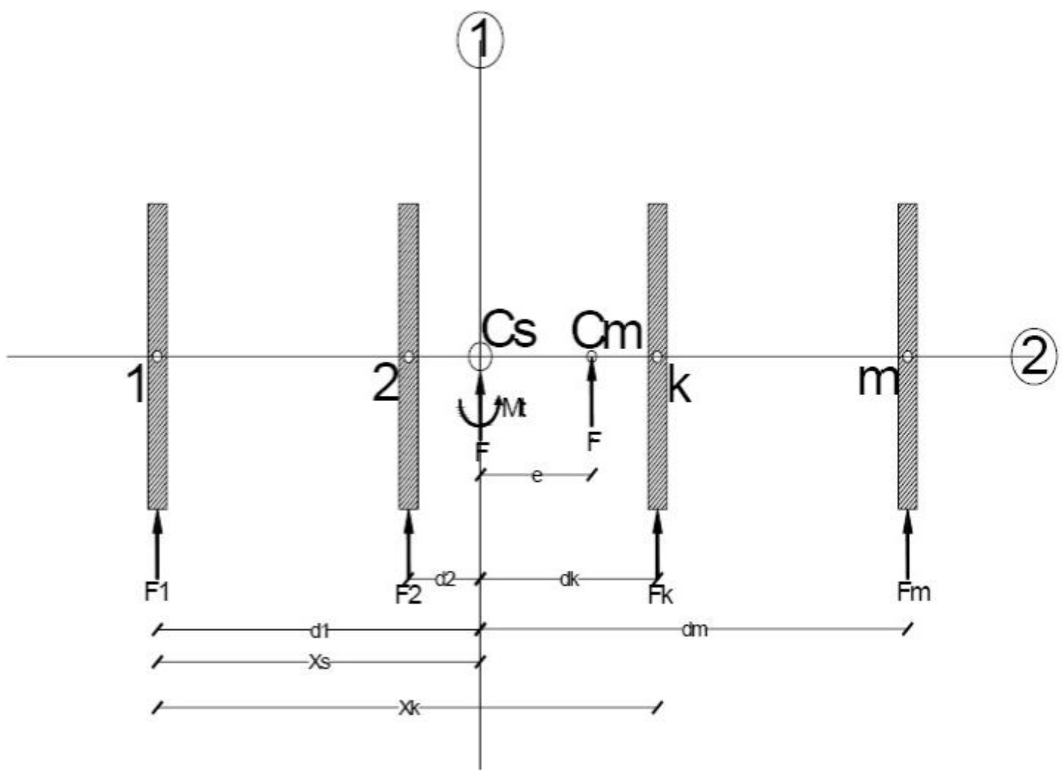


Figure 32. Scheme simplified

For the given case, supposing the rigidity of 2-2 axis is equal to 0 (R_2=0), we can notice that the position of center of rigidity (torsion) in in symmentry axis of 2-2 in distance equal with:

$$X_s= \frac{\sum_{k=1}^{m} R1kXk}{\sum_{k=1}^{m} R1k} = \frac{\sum_{k=1}^{m} IkXk}{\sum_{k=1}^{m} Ik} \quad (10)$$

In the expression above with K=1,2…. M is any of vertical walls.

R1k - relative rigidity of wall k.

Xk – distance of wall K from left side wall.

Ik – Moment of inertia of wall K versus horizontal axis x.

Polar moment of inertia of all structural vertical elements is:

$$J= \sum_{k=1}^{m} R_{1k}\, {d_{1k}}^2 \quad (11)$$

Torsional moment is:

$$Mt = Fe \quad (12)$$

Where F – Shear force.

The distribution of shear forces F in the active forces Fk exerted on the walls of a special "k" are made in a propotionally way according to the principle of rigidities:

$$F_K= \frac{Rk}{\sum_{k=1}^{m} Rk} * F + \frac{Rk*dK}{\sum_{k=1}^{m} Rkdk2} * Fe \quad (13)$$

The term in brackets in the last expression can be positive or negative, depending on the relative position of Cs and CM centers. We mark this term with ρK:

$$\rho k= \left(\frac{Ik}{\sum_{k=1}^{m} Ik} + e * \frac{\sum_{k=1}^{m} Ikdk}{\sum_{k=1}^{m} Ikdk2}\right) \quad (14)$$

Finally, force Fk that is “discharged" in a wall "k" can be written simply as:

$$FK = \rho k\ F \quad (15)$$

Measure of ρK can be considered as the distribution coefficient of shear forces on the structural element "K" taking into account the simultaneous action of translational and rotational motion of the verticals structural diaphragmas.

We note that ρK is an algebraic measure and can result positive or negative.

4.3. The effect of irregularities from masonry fillings

4.3.1. Irregularities in plan

When the distribution of masonry fillings in a concrete frame structure is fairly uniform, existing irregularities can be considered by adding accidental eccentricity with multiplier factor λ=0.2. But, when irregularities are caused by placing non-summetrical fillings, structural analysis is done by using spatial models that takes in consideration the impact of fillings in rigidity distribution. Masonry fillings can be considered as the model in (*Fig. 36*).

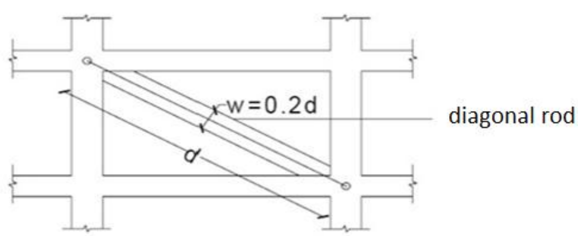


Figure 33. Diagonal rod

$$K_W = \frac{H}{\delta} = \frac{Ew * W * t * \cos 2\theta * e}{d} \tag{16}$$

H- Horizontal force;

δ - Relative displacement of corresponding horizontal measure H

EW – The elasticity module of masonry;

$w \approx 0.2d$;

t – Thickness of masonry

d- The diagonal length

θ – The diagonal angle against the horizontal axis

4.3.2. Irregularities in height

In the case of significant irregularities in height, when there are immediate reductions of fillings in one or several floors, in these floors should be applied a local increase of seismic effects. Counter effects on the structure without filling grow by a multiplier η determined by the expression:

$$\eta = 1 + \frac{\Delta VRw}{\sum VSD} \tag{17}$$

Where:

"q" is the behavior of the structure factor

ΔVRw –is the total reduction of filled wals on given floors making a comparison with full fillings.

$\sum$VSD- is the sum of seismic shear forces that act in all vertical elements in the given floor. In case where η <1:10, is not necessary a such a modification effects.

Regarding to wall filling should be noted that the seismic analisys of the structure can be done without taking in consideration masonry flling [Takeda et. al., 1992].

CHAPTER 5

Shear walls effects in the structure

5.1. Description of the case study building.

Building under study is a mixed reinforced concrete building 8-storey. The building will serve as offices. The building has 2 floors under basement and 6 upper floors. The total height of the building on the foundation is 19m. The height of the first floor is 4 meters (from 0 to level 1), while other heights are 3m floors. In basemant there are peripheral walls. Floor dimensions of the first floor are 30mx21m, however, the floor surface to other floors (above 0) are the same, approximately 30mx14m.

Reinforced concrete walls will be placed in several positions to study their effect on the main parameters of the building as periods, relative displacement, shear forces in basement etc that will be one of the main parts of the study in this degree. To reduce as much as possible the load received by the beams we have to attribute them in the shortest direction. The slabs of the first floor are realized with monolithic while the floor above with ribbed slabs because of architectural reasons. This is realized by placing ceramic bricks type ‘sap’ with γ=800 kg/m^3. They are 12 cm width and 50cm distance from each other. Slabs are horizontal diaphragms undeformed in their plans.

Concrete for columns must be 300kg/cm^2 (B-30) C20/25, while for horizontal and walls are 250 kg/cm^2 (B- 25) C25/30.

Steel is with Ra = 3600kg/cm^2.

Calculation of horizontal and vertical elements is done automatically with the help of computeric programme ETABS 9.0.7. The calculation scheme of the structure is spatial. Such a scheme allows the three-dimensional structure modeling and taking into consideration of all the factors that actually operate in structure. So we can mention modeling the horizontal seismic forces etc.

5.2. Preliminary sizing of the structure

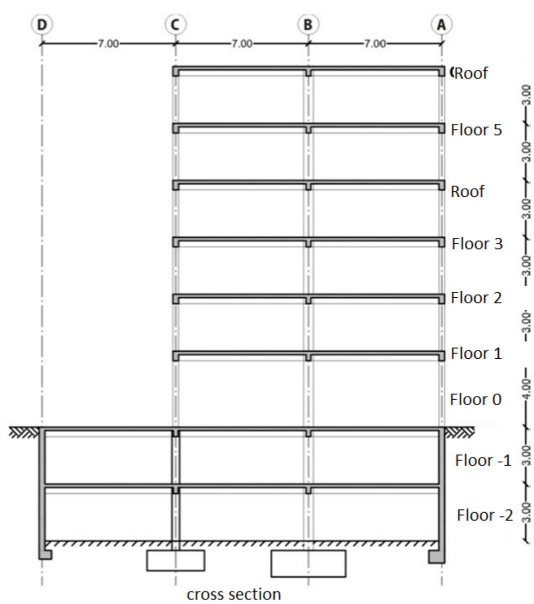


Figure 34 . Cross-section scheme of the building. Units are in metres.

5.2.1. Sizing of frames

This kind of sizing is done through experience in other object like this.

$$h = (\frac{1}{10} / \frac{1}{12}) * l \tag{18}$$

$$b = (\frac{1}{2} / \frac{1}{3}) * h \tag{19}$$

Frame sizing is done through architectural requirements to have a flat ceiling. So in this case we can say that we are obliged to use frames that hide quite good inside the slab. But is olso taken in consideration the fulfillements of reductions and resistance. They are generally provided thin beams with height 25cm.

Loaded width beams is accepted 80cm, and for other connecting beams 30x50cm. High beams are used to join the lateral columns (to create a closed contour).

5.2.2. Sizing of Colums

Columns works in compression and bending simultantly so we will have moment by two plans (x-x and y-y) due to the action of seismic force wich have equal probability to happen in both two directions. Due to the action of seismic forces which gives shear and bending moment on both sides of the column, we have to reinforce columns on both of their sides. The columns are provided with section 40x70 cm, decreasing in height, for the foundation and 30x70 for floors above 0. Columns change their size in height. Concrete for columns must be M = 300 kg / cm^2. For other structures concrete should be taken not less than M = 250 kg / cm^2.

5.2.3. Sizing of slabs

Slabs are provided with beams 30cm of total thickness. In calculation of office slabs is taken in consideration that above them will act a force of p = 200 kg/m^2.

To the weight of slab is added even the load of internal walls. The slabs of shoping floor are calculated with an overload of 500kg/m^2. The slab of the roof is calculated with an overload of 200kg/m^2. Slabs are undeformed horizontal diaphragms in their plan. The height of the slab is depending on the light space between columns

$$h = \left(\frac{1}{25} / \frac{1}{30}\right) * l \qquad (20)$$

Is accepted a column with h=20cm

Slabs of the stairs are provided as monolithic with 15cm thickness and are calculated for an overload 300kg/m^2. Slabs of balconies are predicted to be built with reinforced ceramic with 30cm thickness and calculated for an overload of 300kg/m^2. In all the slabs in every 2-3 metres are predicted connector beams with dimensions of 20x30cm.

5.3. Loads acting on a structure

5.3.1. Dead loads

Permanent loads

Monolithics slab

• Tiles 0.01*2500 = 25 daN/m^2 * 1.35 = 33.75 daN/m^2

• Leveling layer 0.05*2200 = 110 daN/m^2 * 1.35 = 148.5 daN/m^2

• Slab 0.2*2500 = 500 daN/m^2 * 1.35 = 675 daN/m^2

• Plaster 0.02*1800 = 36 daN/m² * 1.35 = 48.6 daN/m² gn= 671 daN/m² gll= 905.85 daN/m²

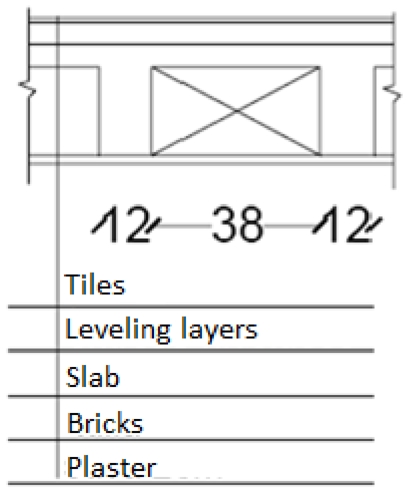


Figure 35. Element of permanent load

In one direction:

• Tiles 0.01*2500 = 25 daN/m² * 1.35 = 33.75 daN/m²

• Leveling layers 0.05*2200 = 110 daN/m² * 1.35 = 148.5 daN/m²

• Slab

(0.05*1*1+2*0.12*0.2*1)*2500=245 daN/m² * 1.35 = 269.5 daN/m²

• Bricks

2 * 0.38 * 0.2 * 800=121.6 daN/m² * 1.35 = 330.35 daN/m²

gn= 501.6 daN/m² gll= 782.5daN/m²

In two directions (change the load of the slab not the layers):

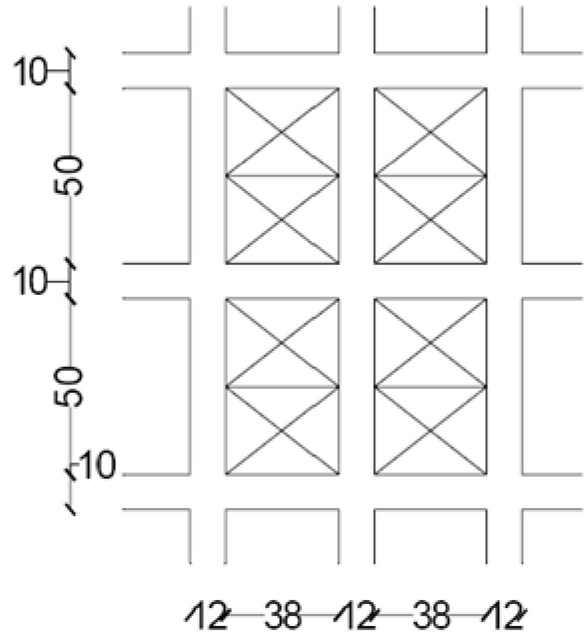


Figure 36. Elements of permanent load.

• Tiles 0.01*2500 = 25 daN/m² * 1.35 = 33.75 daN/m²

• Leveling layers 0.05*2200 = 110 daN/m² * 1.35 = 148.5 daN/m²

• Slab 0.05*2500 = 125 daN/m² * 1.35 = 168.75 daN/m²

• Ribbed

0.2*1*1*2500 – 4*0.2*0.25*0.38*2500- 2*0.2*0.3*0.38*2500 = 196 daN/m² * 1.35 = 264.6 daN/m²

• Bricks

4*0.2*0.25*0.38*800 + 2*0.2*0.3*0.38*800 = 97.28 daN/m² *1.35 = 131.33 daN/m²

gn= 553.28 daN/m² gll= 746.33 daN/m²

5.3.2. Walls

5.3.2.1. External Walls (30cm)

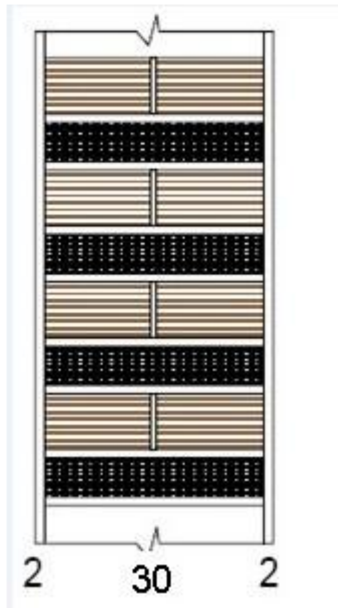


Figure 37. External wall

• Bricks 0.3*1*1*800 = 240 daN/m2 wall *1.35= 324

• Plaster 0.04*1*1*1800 = 72 daN/m2 wall *1.35=97.2

• Mortar 5*0.3*0.01*1*2200 =33 daN/m2 wall *1.35= 44.55 gn

m30= 345 daN/m² wall

5.3.3. Stairs

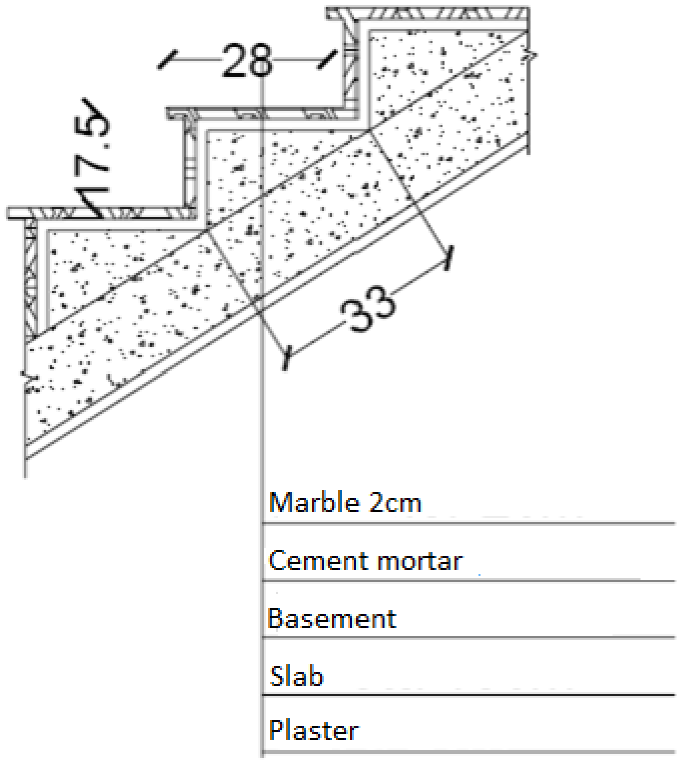


Figure 38. Stairs

Inclined part

A. permanent

• Marble (0.175+0.28)*0.02*1*2500 = 22.75 daN/baz *1.35 = 30.71 daN/baz

• Mortar (0.175+0.28)*0.02*1*2000 =18.2 daN/baz *1.35 = 24.57 daN/baz

• Base (0.175*0.28)*0.5*1*2300 =56.35 daN/baz *1.35 =70.07 daN/baz

• Slab 0.15*0.33*1*2500*cosα = 105.2daN/baz *1.35 = 142.02 daN/baz

• plaster 0.02*0.33*1*2000* cosα = 11.22 daN/baz *1.35 = 15.15 daN/baz gn= 213.72 daN/baz gll= 282.52daN/baz

$gn=213.72/0.28 = 764 daN/m^2$ $gll= 282.52/0.28 = 1009\ daN/m^2$

Lobby

A.permanent

- Marble $0.02*1*1*2500 = 50\ daN/m^2 *1.35 = 67.5\ daN/m^2$
- Mortar $0.02*1*1*2000 = 40$ -||- $*1.35 = 54\ daN/m^2$
- Slab $0.15*1*1*2500 = 375$ -||- $*1.35 = 506.25\ daN/m^2$
- Plaster $0.02*1*1*1800 = 36$ -||- $*1.35 = 48.6\ daN/m^2$

$gn = 501 daN / m2$ $gll= 676.35\ daN/m^2$

Live loads

- Office p=200daN/m2
- Shops p=500daN/ m2
- Balconies p=300daN/ m2
- Roof p=400daN/ m2
- Stairs p=300daN/ m2

5.3.4. Special loads (seismic)

Included in this group of loads are wind load which will not be considered and the seismic load from earthquake. Soils of the area where the object will be built based on requirements of geologic-engineering is part of the 3rd category and considering that the object is located in Vlora, with the help of seismic map we have to calculate the object through a seimic intensity I=8 Richter.

According to the KTP, the seismic response spectrum for horizontal seismic action is:

$$S_a = k_E * k_R * \Psi * \beta * g \tag{21}$$

β - Dynamic Coefficient is given:

1. for 1st category soils (strong):

$$0.65 \leq \beta = \frac{0.7}{T} \leq 2.3 \tag{22}$$

2. for 2nd category soils (medium):

$$0.65 \leq \beta = \frac{0.8}{T} \leq 2.0 \tag{23}$$

3. for 3rd category soils (weak):

$$0.65 \leq \beta = \frac{1.1}{T} \leq 1.7 \tag{24}$$

Seismic forces shall be defined by means of modal analysis of dynamic response which is determined in the form of general formula.

$$E_{ki} = k_E * k_r * \Psi * \beta_i * \eta_{ki} * Q_k \tag{25}$$

Where E_{ki} is the seismic force in floor (k) which responds to (i) of oscillations. k_E- is the seismic coeffition wich is taken from soil categories.

3rd category I=8.5 Richter kE=0.34

k_r –building importance coefficient.

Massive buildings kr=1.0

Ψ- Coefficient of constructive solutions. Ψ=0.28 Mixed type structure.

βi for dynamic coefficient of oscillation form is taken by the chart above

$0.65 \leq \beta = \frac{1.1}{T} \leq 1.7$ 3rd category soil (24)

$\eta_{ki} = \Phi_{ki} * \frac{\sum_{j=1}^{n} Qj * \Phi ji}{\sum_{j=1}^{n} Qj * \Phi_{ji}^{2}}$ form oscillations coefficient (26)

$Q_k = m_k * g$ weight in (k) level of the structure (27)

Seismic antion in according to EuroCode 8 is taken from the spectre of elastic response.

- Maximal seismic acceleration in strong soil is agR.
- agR = 0.25g. g=9.81 m/s2
- Values of periods (TB, TC, and TD) and the land factor (S), outlining the shape of the elastic response spectrum is:
- TB = 0.15s, TC = 0.5 s, TD = 2, 0 s and S = 1.2.
- TB, TC→limits of spectral constant acceleration.
- TD→ value that determines the start order of the reaction with constant displacement in the spectrum.

The building is classified as class II and coefficient γ= 1.0

The ground seismic acceleration is equal:

$ag = \gamma * agR = 0.25g$ (28)

According to EuroCode 8elastic response spectrum is taken as 5% of damping. For designing the building is used the designed response spectrum (spectre of elastic response reduced by behavior factor q). Determination of the behavior factor q, which depends on the type of structural system, regularity in elevation and plan, and ductility, it is 3.0.

MS→ magnitude of earthquake.Ms2: 5.5 ⟹1st types of earthquakes determined by EuroCode 8 of designing acceleration are determined:

For $0 \leq T \leq T_B \quad \rightarrow$ Sd (T) $= a_g * S * [1 + \frac{T}{T_B} * (\frac{2.5}{q} - 1)]$ (29)

For $T_B \leq T \leq T_C \quad \rightarrow$ Sd (T) $= a_g * S * \frac{2.5}{q}$ (30)

For $T_C \leq T \leq T_D \quad \rightarrow$ Sd (T) $= \begin{cases} ag * S * \frac{2.5}{q} * (\frac{Tc}{T}) \\ \geq \beta * a_g \end{cases}$ (31)

For $T \geq T_D \quad \rightarrow$ Sd (T) $= \begin{cases} ag * S * \frac{2.5}{q} * (\frac{Tc*Td}{T^2}) \\ \geq \beta * a_g \end{cases}$ (32)

In our case T1=0.72, Tc=0.5, TD=2.0 so:

β→ so called factor ofbottom border is recommended to be taken = 0.2

For $T_C \leq T \leq T_D \quad \rightarrow$ Sd (T) $= \begin{cases} ag * S * \frac{2.5}{q} * (\frac{Tc}{T}) \\ \geq \beta * a_g \end{cases}$ (33)

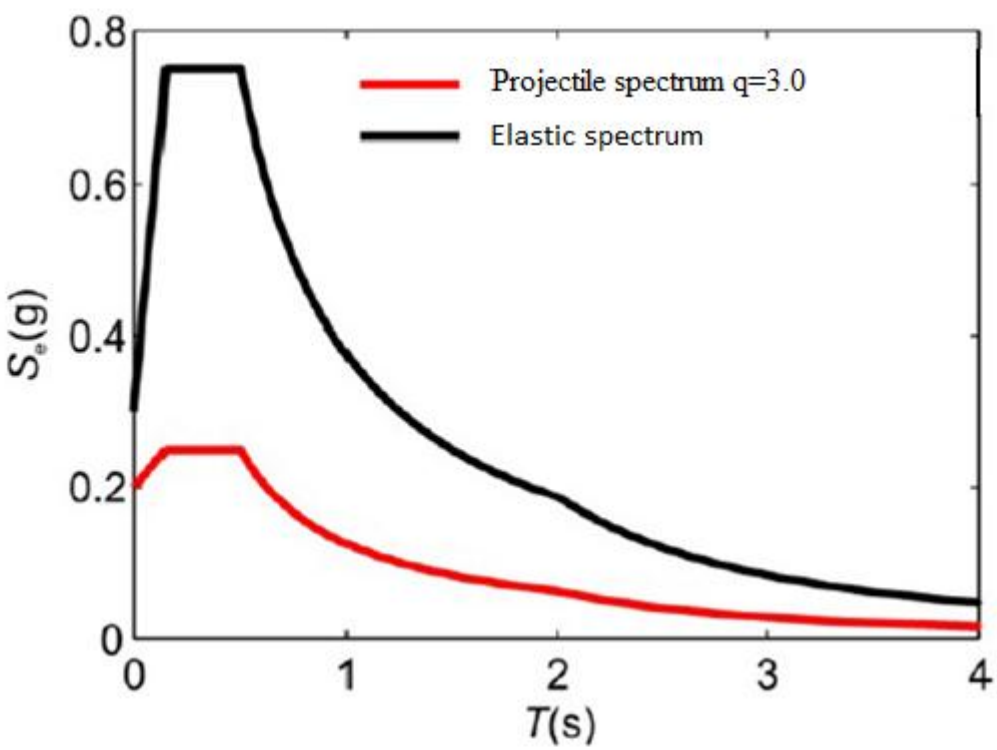


Figure 39. ***The design spectrum and elastic spectrum.***

5.4. Structural model

ETABS programme in used to analyse. It is used a three dimensional structural model. The axes used are all shown in figure. Structural model fulfills all the requirements of EuroCode 8.

Characteristics of the model are shown below:

- All elements, including walls, are modeled as straight elements
- Effective width of the beams is calculated according to EuroCode. There are used for two different widths for internal beams and two external beams
- Stiffness for connector beams and colums have been ignored
- Infinite stiffness of the elements is used only in connection with the walls (walls M2 and M5 in sections 1 and 6).
- All Elements are fixed entirely on the foundation (at -2).
- Frames and the walls are connected together through rigid diafragmaes (in the horizontal plane) on each floor. These tiles are not modeled.
- Moments and measures of inertia of each floor are concentrated in the center of loads. They are calculated from vertical loads. Only loads of upper peripheral walls are taken in consideration
- Fillings are not taken in consideration in this model.

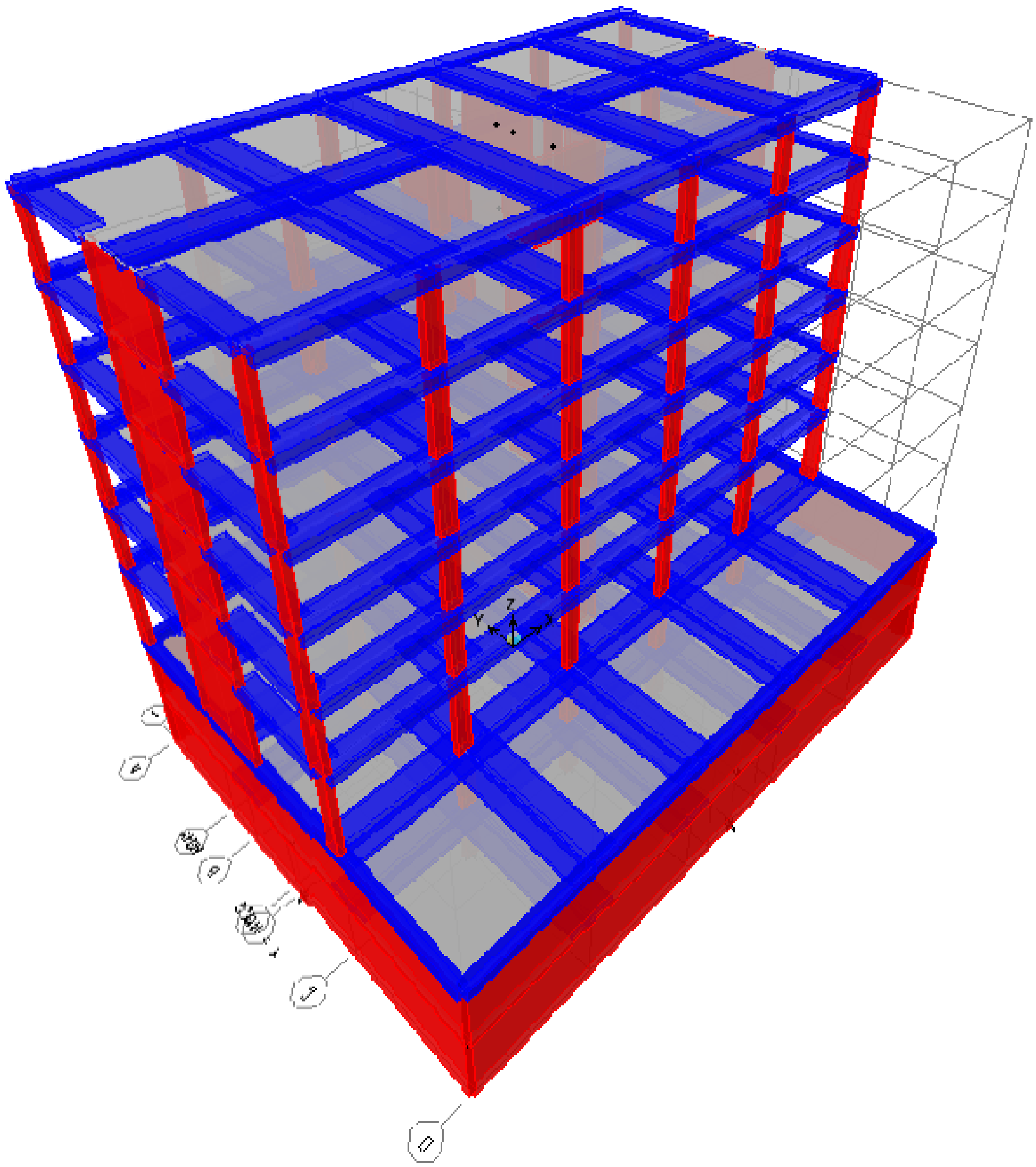


Figure 40. Structural dual system model.

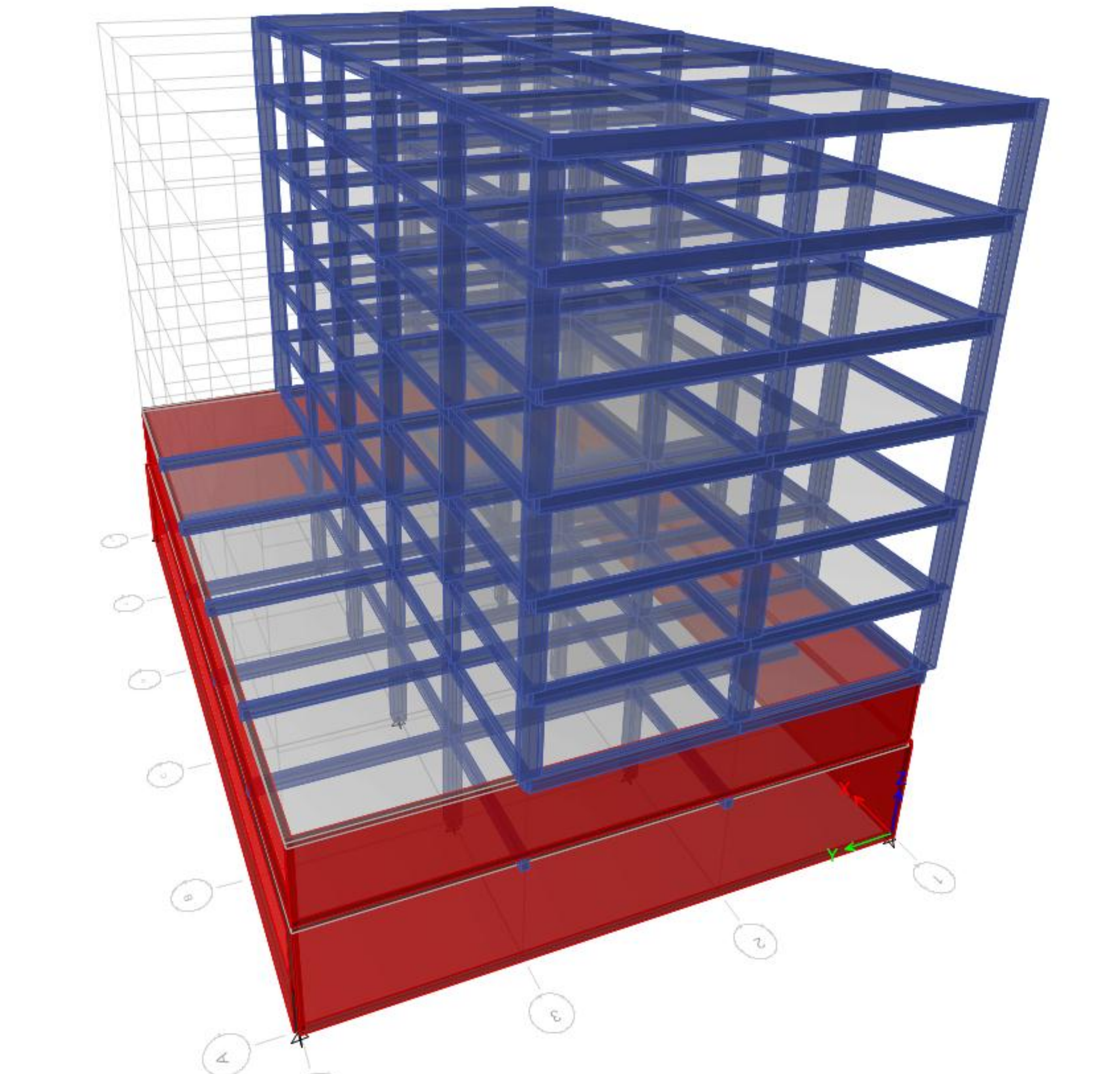


Figure 41. Structural frame system model.

5.5. The effective width of beams

The effective width of beams b_{eff} is calculated according to EuroCode. It must be defined two different internal beams width (Beam T1 and beam T2 fig 43) and two different external beams width (External beam T1 and T2)

We will take a constant width for the space. In this case the value of b_{eff} for the space will be taken according to the EuroCode. Corresponding length lo (The distance between the two points where the moment is 0) it is taken as 70% of the length of the element. Effective width values beff are about 5cm (*Fig. 44*).

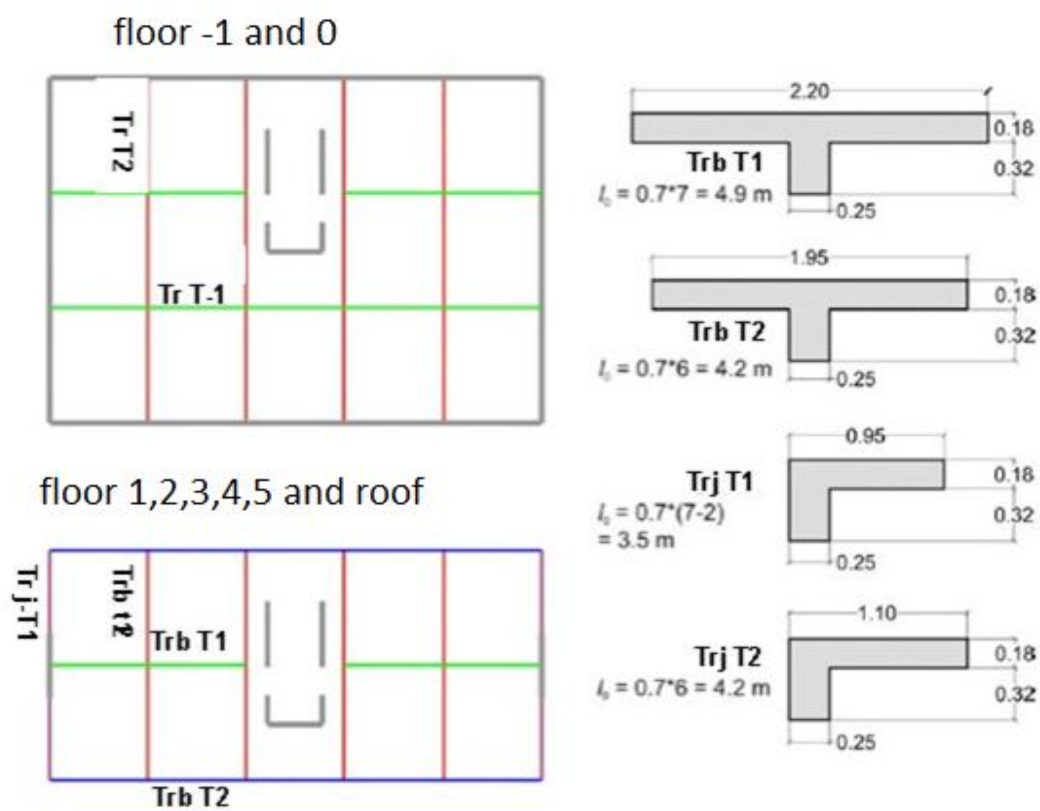


Figure 42. Effective widths of beams

5.6. Modeling of perimetral walls

Perimetral walls are modeled with straight elements and with rigid beams at the top of each element.Rigid beams (RB orientation in ETABS) are modeled as elements with quadrangular cross-section.The greater value of beam rigidity is obtained by multiplying all the characteristics (area, moment of inertia, torsion constant) to a factor 100. Eight fictitious columns are oriented in the direction X (WP1), four Colums in the direction Y (WP2) and four columns in the corner (WPC, (*Fig. 44*) are used for modeling the perimeter walls. For each column, the surface, the moment of inertia about the main axis are calculated as part of the respective characteristics of the whole perimeter wall in the direction selected (Wp1 * in direction X, Wp2 * in direction Y). Cross-section of the walls are 30 * 0.3 m and 21 * 0.3 in case of Wp1 * and Wp2. The moment of weak axis is determined using the effective with of the fictional column. We assume that the effective width for columns WP1 and WP2 runs until 4 m, which is equal to the width of the walls M1 - M4 in floors above basement. The torsion stiffness of columns is ignored. In the case of WP1 column, the surface, and the moment of inertia about the strong axis represent one fifth of the entire corresponding values of WP1 * wall, whereas in the case of column WP2, they reach 1/3 of the wall values WP2 *. For columns in the corner (WPC), Surface represents the sum of the proportional values to the two walls (WP1 * and WP2 *), and the moment of inertia about the axis 3 with origins from the wall WP1 *, while the moment about the axis 2 is derived from the wall WP2 *. Local axes (2 and 3) of all the columns are oriented in such a way, that axis 2 coincides with the global X and axis 3 with the global axis Y (*Fig. 45*).

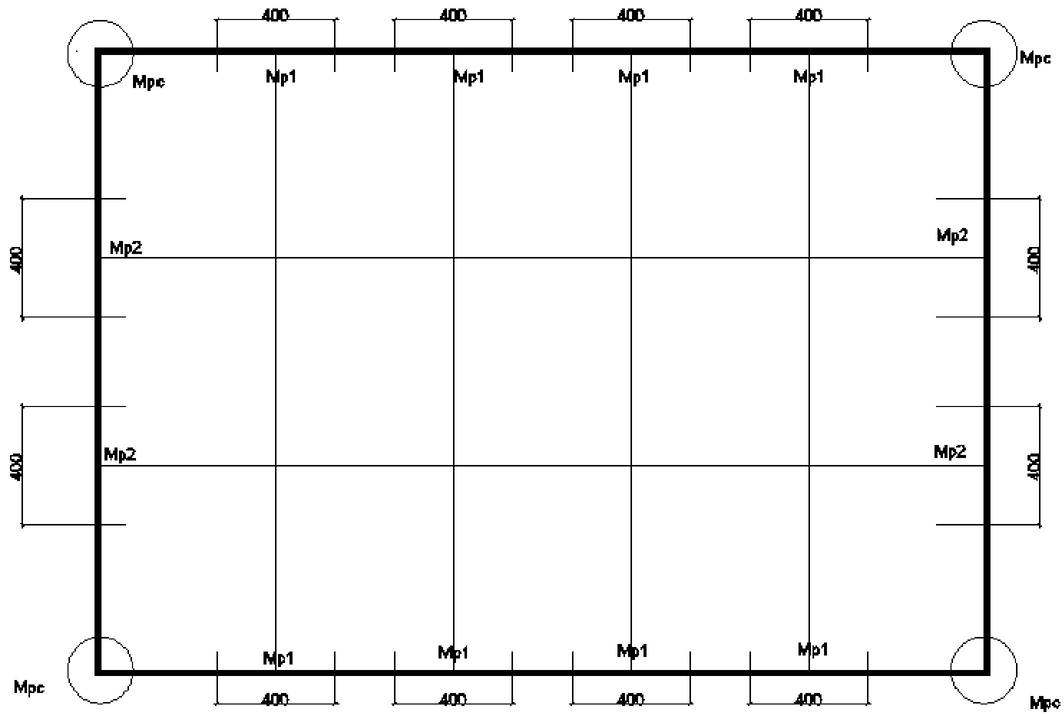


Figure 43. Modeling of perimetral walls

5.7. Regularity of the structure

The regularity of the structure (in height and in plan) affects the necessary structural model (planar or spatial), the method of analysis required and the value of the behavior factor q. Structure under study can be categorized as regular in height and in plan. The regularity or irregularity in plan may affect the size of the seismic action (through load factor αu / α1). In the case of building that I take in sudy, the load factor does not apply and there is no difference between the seismic action for a regular building in plan or irregular building in plan. Structure under study is regular in height, if we don't consider irregularity due to basement. For a regular structure in plan and in height, it can be used planar model and can be performed with lateral force model. Furthermore, it can be used as the reference value based on the behavior factor qo.

5.8. Criteria for regularity in plan

In general, the regularity in plan can be controlled when structural model is defined. Criteria for regularity in plan:

- Longitudinal flexibility of the building will be no higher than 4 (λ = Lmax / lm)
- Structural eccentrity will be less than 30% of the radius of inertia ($e_{0X} \leq 0.30r_X$, $e_{0Y} \leq 0.30r_Y$) and
- radius of inertia will be greater than the mass moment of inertia of in floor plan ($r_x \geq ls$, $r_y \geq ls$)

Flexibility of the structure under study is less than 4.0.

$$\lambda = \frac{30\text{m}}{21\text{m}} = 1.43 \text{ for basement floors} \quad (34)$$

$$\lambda = \frac{30\text{m}}{14\text{m}} = 2.14 \text{ on the upper floors} \quad (35)$$

Two other conditions (structural eccentrity is less than 30% of the radius of inertia and inertia radius is greater than the mass moment of inertia of floors) are also conducted in each level floor in both directions. The building is categorized as regular in plan in both directions.

Table 1. Criteria for irregularity in plan according to EuroCode. All units are in M.

floor	Direction X						Direction	
	|eox| <	0.3 rx	rx >	ls	|eoy| <	0.3 ry	Y ry >	ls
roof	0	3.81	12.71	9.56	0.93	4.96	16.54	9.56
floor 5	0	3.8	12.66	9.56	1.06	5.1	16.99	9.56
floor 4	0	3.78	12.59	9.56	1.25	5.27	17.56	9.56
floor 3	0	3.77	12.57	9.56	1.49	5.52	18.38	9.56
floor 2	0	3.81	12.69	9.56	1.77	5.9	19.65	9.56
floor 1	0	3.96	13.21	9.56	2.09	6.46	21.44	9.56
floor 0	0	5.76	19.21	1057	0	4.75	15.82	10.57
floor -1	0	5.54	18.48	10.57	0	4.77	15.91	10.57

Table 2. Modal participating mass ratios of frame analysis.

Case	Mode	RX	RY	RZ	Sum RX	Sum RY	Sum RZ
Modal	1	0.0012	0.5703	0.0147	0.0012	0.5703	0.0147
Modal	2	0.541	0.0006	0.015	0.5422	0.571	0.0297
Modal	3	0.0233	0.0022	0.3737	0.5655	0.5731	0.4034
Modal	4	2.749E-06	0.0583	0.0011	0.5655	0.6314	0.4045
Modal	5	0.0641	1.209E-05	0.003	0.6296	0.6314	0.4075
Modal	6	0.0045	3.985E-06	0.0522	0.6341	0.6314	0.4597
Modal	7	2.48E-06	0.0029	0.0005	0.6341	0.6343	0.4603
Modal	8	0.0072	0	0.0009	0.6413	0.6343	0.4612
Modal	9	0.0001	0.0073	0.0031	0.6414	0.6415	0.4643
Modal	10	0.0002	0.002	0.0211	0.6416	0.6435	0.4854
Modal	11	2.062E-06	0.0043	0.0001	0.6416	0.6479	0.4855
Modal	12	0.0139	1.013E-06	0.0002	0.6555	0.6479	0.4857

5.9. Determination of structural eccentricity (e_{0X} and e_{0Y})

Structural eccentrity in two orthogonal directions (e_{0X} and e_{0Y}) represents the distance between the center of rigidity (X_{CR}, Y_{CR}) and the center of mass (X_{CM}, Y_{CM}). In general, it should be calculated for each floor. The center of mass coincides with the origin of the global coordinate system in floors above 0. Determination of structural eccentricity:

$$e_{ox,i} = \frac{Rzi(Fxi=1)}{\mathrm{Rz,i(Mi=1)}} \quad \text{and} \tag{36}$$

$$e_{oY,i} = \frac{Rz,i(Fy,i=1)}{\mathrm{Rz,i(Mi=1)}} \tag{37}$$

- where $R_{z\,,\,i}$ ($F_{Y\,,\,i}$ = 1) –is the floor rotation around the vertical axis due to static load $Fy,_{i}$ = 1 in the direction Y.
- $R_{z,\,i}$ ($F_{X,\,i}$ = 1) - the floor rotation due to load $F_{X,\,i}$ = 1 in the direction X
- $R_{z,\,i}$ (M = 1) - is the rotation due to torque moment around vertical axis.

forces $F_{X\,,\,I}$ and $F_{Y\,,\,i}$ and moment M are applied at the center of mass of floor i.

This can occur because is supposed that floors are rigid. Structural spatial model is needed for orientation of eccentricity used. In the case of the building being studied by 8 floors, static load cases were defined. Results are presented in Table 2. Unit loads are:

$F_{X,\,i} = F_{Y,\,i}$ = 106 kN and M = 106 kNm. Acquired coordinates of the center of rigidity are measured from the center of mass.

$$X_{CR\,,\,i} = X_{CM\,,\,i} + e_{0X\,,\,I} \tag{38}$$

$$Y_{CR\,,\,i} = Y_{CM\,,\,i} + e_{0Y\,,\,I} \tag{39}$$

In general, $e_{0X\,,\,i}$ and e_{0Y} , i may be positive or negative sign, but to control the absolute regularity plan are used in absolute value.

Table 3 . The coordinates of the center of mass (X_{CM}, Y_{CM}), rotation R_Z because of forces F_X = 106K, F_Y = 106 kN and M = 106 kNm, structural eccentricity (E_{OX} and e_{OY}) and the coordinates of the center of rigidity (X_{CR}, Y_{CR})

Floor	Xcm	Ycm	Rz(Fx)	Rz(Fy)	Rz(M)	eox	eoy	Xcr	Ycr
	(m)	(m)	(rad)	(rad)	(rad)	(m)	(m)	(m)	(m)
roof	0	0	-0.0761	0	0.0818	0	-0.93	0	-0.93
Floor 5	0	0	-0.057	0	0.5037	0	-1.06	0	-1.06
Floor 4	0	0	-0.0418	0	0.0333	0	-1.25	0	-1.25
Floor 3	0	0	-0.0277	0	0.0186	0	-1.49	0	-1.49
Floor 2	0	0	-0.0151	0	0.0086	0	-1.77	0	-1.77
Floor 1	0	0	-0.0059	0	0.0028	0	-2.09	0	-2.09
Floor 0	0	-3.5	0	0	0.0002	0	0	0	-3.5
Floor -1	0	-3.5	0	0	0.0001	0	0	0	-3.5

Table 4. Modal participating mass ratios frame model.

Case	Mode	Period sec	UX	UY	UZ	Sum UX	Sum UY	Sum UZ
Modal	1	0.903	0.5037	0.0011	0	0.5037	0.0011	0
Modal	2	0.871	0.0005	0.4948	0	0.5042	0.4958	0
Modal	3	0.675	0.0027	0.023	0	0.5069	0.5188	0
Modal	4	0.285	0.0682	2.392E-05	0	0.575	0.5188	0
Modal	5	0.25	9.8E-06	0.0744	0	0.575	0.5932	0
Modal	6	0.206	0	0.0046	0	0.575	0.5977	0
Modal	7	0.164	0.0294	1.589E-06	0	0.6044	0.5977	0
Modal	8	0.133	0	0.0322	0	0.6044	0.6299	0
Modal	9	0.114	0.0162	0.0002	0	0.6206	0.6302	0
Modal	10	0.112	0.0041	0.0007	0	0.6247	0.6309	0
Modal	11	0.088	0.0149	3.481E-06	0	0.6396	0.6309	0
Modal	12	0.085	1.794E-06	0.023	0	0.6396	0.6539	0

5.10. Orientation of radius of inertia

Radius of inertia $r_x(r_y)$ is taken as the square root of the ratio of the rigidity (K_M) with lateral rigidity in a direction K_{Fy} (K_{Fx})

$$r_{x,i}=\sqrt{\frac{Km,i}{K\,Fy,i}} \quad \text{and} \tag{40}$$

$$r_{y,i}=\sqrt{\frac{Km,i}{K\,Fx,i}} \tag{41}$$

The procedure for determining the direction of torsion and lateral rigidity is similar to determining the structural eccentricity that static cases of the structure are determined for each floor and the loads are represented by F_{TX}, F_{TY} and M_T, respectively from forces and moments that are applied in center of rigidity. Resistance in torsion and lateral rigidity for both directions are calculated as above:

$$K_{m,I}=\frac{1}{Rz,i(Mt,i=1)}\ ,\ K_{Fx,i}=\frac{1}{Uz,i(F\,Tx,i=1)}\ ,\ K_{Fy,i}=\frac{1}{Uy,i(F\,Ty,i=1)} \tag{42}$$

Where:

- $R_{z,\,i}$ ($M_{T,\,i}$ = 1) – is rotation of the floor in vertical axis
- $U_{X,\,i}$ ($F_{TX,\,i}$ = 1)- It is floor level shift in direction X due to force unit FTX
- Uy, i ($F_{TY,\,i}$ = 1) - Is the shifting in Y direction due to force unit F_{TY}.

$F_{TX,\,i}$ = $F_{TY,\,i}$ = 106 kN and $M_{T,\,i}$ = 106 KNm.

Table 5. Displacements (U_X, U_Y) and rotation (R_Z) due to forces F_{TX} = 106 kN, F_{TY} = 106 kN and M_T = 106 kNm, stiffness torque in both directions (K_{fx}, K_{FY}) and the radius of inertia (r_x, r_y)

floor	Ux(Ftx)	Uy(Fty)	Rz(Mt)	Kfx	Kfy	Kmt	Rx	ry
	(m)	(m)	(rad)	(kN/m)	(kN/m)	(kNm/rad)	(m)	(m)
roof	22.37	13.22	0.0818	$4.47*10^4$	$7.57*10^4$	$1.22*10^7$	12.71	16.54
floor 5	15.51	8.61	0.0537	$6.45*10^4$	$1.16*10^5$	$1.86*10^7$	12.66	16.99
floor 4	10.26	5.28	0.0333	$9.74*10^4$	$1.89*10^5$	$3*10^7$	12.59	17.56
floor 3	6.27	2.93	0.0186	$1.59*10^5$	$3.41*10^5$	$5.39*10^7$	12.57	18.38
floor 2	3.3	1.38	0.0086	$3.03*10^5$	$7.26*10^5$	$1.17*10^8$	12.69	19.65
floor 1	1.29	0.49	0.0028	$7.75*10^5$	$2.04*10^6$	$3.56*10^8$	13.21	21.44
floor 0	0.05	0.07	0.0002	$2.22*10^7$	$1.51*10^7$	$5.56*10^9$	19.21	15.82
floor -1	0.02	0.03	0.0001	$4.78*10^7$	$3.55*10^7$	$1.21*10^{10}$	18.48	15.91

5.10.1. Determination of inertia radious of mass of the floor in plan (ls)

For control criteria for plan regularity, it is necessary the definition of inertial radius for mass of floor (I_S). In case of the quadrilateral cross-section with dimensions l and b and with mass non-uniform in this floor, I_s is taken from:

$$Is= \frac{(i^2+b^2)}{12} \qquad (42)$$

In our case IS =10.57 m for 2-floors under basement and 9.56 for floors above ground 0.

5.11. Criteria for regularity in height

The structure under study significantly fulfills all the requirements for height regularity according to EuroCode provided that only the upper part mentioned above is considered.

5.12. Structure type of building and behavior factor

The mathematical model (structural) is necessary for determining the structural type of building. According to EuroCode8, the building under study is considered as a dual system with a horizontal direction. Structural system is considered as a wall system, when 65 %(or more) of shear resistance in building base is taken from the walls. EuroCode8 allows shear resistance to be substrituted by shear force. In our case, shear force in basement, taken from the walls, reaches about 72% of the shear force of all structural system in X direction and 92% in Y direction. Behaviour factor q for each horizontal direction is calculated from equation:

Q=qo*kW (2.6) (43)

- Where q0 - behavior factor
- k_w- is factor related to the prevention of destruction in structural all system.
- Factor q_0 reaches 3.0. Factor q0 also depends from irregularity in height. Because the structure is considered as regular in height, unit q0 remains unchanged. If the structure is classified as irregular in height, q_0 factor will be reduced to 20%. k_w factor is equal to 1.0, so the behaviour factor is equal to the basic value of the behavior factor $q = q_0 = 3.0$.

5.13. Modal analysis of response spectrum

- Response spectrum analysis (abbreviation as RSA) is carried out independently for ground movements in both horizontal directions.

- The design spectrum (*Fig. 41*) is used in both horizontal directions.

- Modal analysis results in both horizontal directions are combined rule of square root of the sum of squared SRSS

- The torsion side effects are taken into considerationn by means of torsional moments around the vertical axis.

- Weight combination of gravity and seismic loads are considered by Eurocode.

5.14. Periods, effective measures and forms of oscillating

Modal basic characteristics of the building are summarized in Table 4. The three main periods of vibration of the building (considering cracked sections element) are 0.92, 0.68 0.51 s. Effective measures show that the first form of vibrations is translational movement to the X direction, second form is translational movement by Y direction and the third form of vibration is torsion. Three main forms of vibrations of the structure are shown in (*Fig. 46*). The modal analysis of response spectrum is taken into account all forms of vibration. (The sum of the effective modal masses reaches in 100% of the total mass of the structure)

Table 6. Period (T), effective measures and moments of effective measures (Meff)

Form of oscillations	T (s)	Meff,Ux (%)	Meff,Uy (%)	Meff,Uz (%)
1	0.92	80.2	0	0.2
2	0.68	0	76.3	0
3	0.51	0.2	0	75.2
4	0.22	15	0	0.2
5	0.15	0	18.5	0
6	0.12	0.2	0	17.6
	∑Meff=	95.7	94.7	93.1

Table 7. Joint reactions of frame model.

Story	Joint Label	Load Case/Combo	FX kN	FY kN	FZ kN	MX kN-m	MY kN-m	MZ kN-m
Base	1	Dead	18.9772	4.8505	3644.6049	0	0	0
Base	1	Live	3.4804	1.1901	881.6084	0	0	0
Base	1	LCase1 Max	52.1062	10.9695	2573.4985	0	0	0
Base	1	Comb1	32.1367	8.6949	6513.0203	0	0	0
Base	3	Dead	-0.8449	9.8645	3055.7253	0	0	0
Base	3	Live	-0.0785	2.2814	783.1666	0	0	0
Base	3	LCase1 Max	16.3246	8.5893	1959.9725	0	0	0
Base	3	Comb1	-1.3084	17.4604	5531.082	0	0	0
Base	4	Dead	0.5117	10.2606	3035.4119	0	0	0
Base	4	Live	-0.0054	2.2441	775.7209	0	0	0
Base	4	LCase1 Max	16.8692	8.5599	2211.4957	0	0	0
Base	4	Comb1	0.7077	17.9553	5490.73	0	0	0
Base	5	Dead	-15.4124	5.556	3444.3035	0	0	0
Base	5	Live	-4.0028	1.144	875.2056	0	0	0
Base	5	LCase1 Max	50.4728	11.4892	3156.7355	0	0	0
Base	5	Comb1	-27.982	9.6088	6222.3539	0	0	0
Base	7	Dead	-947.5582	-581.9236	9171.8602	0	0	0
Base	7	Live	-240.9625	-89.1913	1853.2386	0	0	0
Base	7	LCase1 Max	4546.9448	4651.2014	8982.0627	0	0	0
Base	7	Comb1	-1712.1215	-957.3992	15805.7862	0	0	0
Base	12	Dead	929.3443	-478.5766	8426.7111	0	0	0
Base	12	Live	239.8405	-100.0292	1835.2263	0	0	0
Base	12	LCase1 Max	4583.4183	4945.7894	9379.9764	0	0	0
Base	12	Comb1	1684.8267	-830.0539	14733.7576	0	0	0
Base	13	Dead	37.3463	10.6954	6360.9982	0	0	0
Base	13	Live	7.2856	2.5058	1628.7989	0	0	0
Base	13	LCase1 Max	60.871	13.444	883.704	0	0	0
Base	13	Comb1	63.9418	18.9829	11511.4757	0	0	0
Base	14	Dead	-2.8112	15.4418	6400.7185	0	0	0
Base	14	Live	-0.4317	3.6216	1693.643	0	0	0
Base	14	LCase1 Max	7.6585	18.616	1388.5301	0	0	0
Base	14	Comb1	-4.6263	27.413	11670.8346	0	0	0
Base	15	Dead	2.4891	15.4297	6412.9717	0	0	0
Base	15	Live	0.4708	3.6483	1693.3705	0	0	0
Base	15	LCase1 Max	7.5226	19.4655	1416.8526	0	0	0
Base	15	Comb1	4.2379	27.4389	11687.5532	0	0	0
Base	16	Dead	-31.5226	10.7636	6268.117	0	0	0
Base	16	Live	-8.05	2.5603	1653.6395	0	0	0
Base	16	LCase1 Max	61.0962	15.5516	1040.4876	0	0	0
Base	16	Comb1	-57.0116	19.1655	11421.187	0	0	0
Base	19	Dead	125.4667	531.3348	3951.9989	0	0	0
Base	19	Live	10.0636	78.3296	718.9442	0	0	0
Base	19	LCase1 Max	2805.6814	2361.3291	5009.5058	0	0	0
Base	19	Comb1	191.7551	869.1961	6683.1093	0	0	0
Base	20	Dead	-115.9859	446.3033	3770.2247	0	0	0
Base	20	Live	-7.6098	91.6954	727.4375	0	0	0
Base	20	LCase1 Max	2744.8075	2737.0018	4820.0062	0	0	0
Base	20	Comb1	-174.556	771.5373	6442.2146	0	0	0

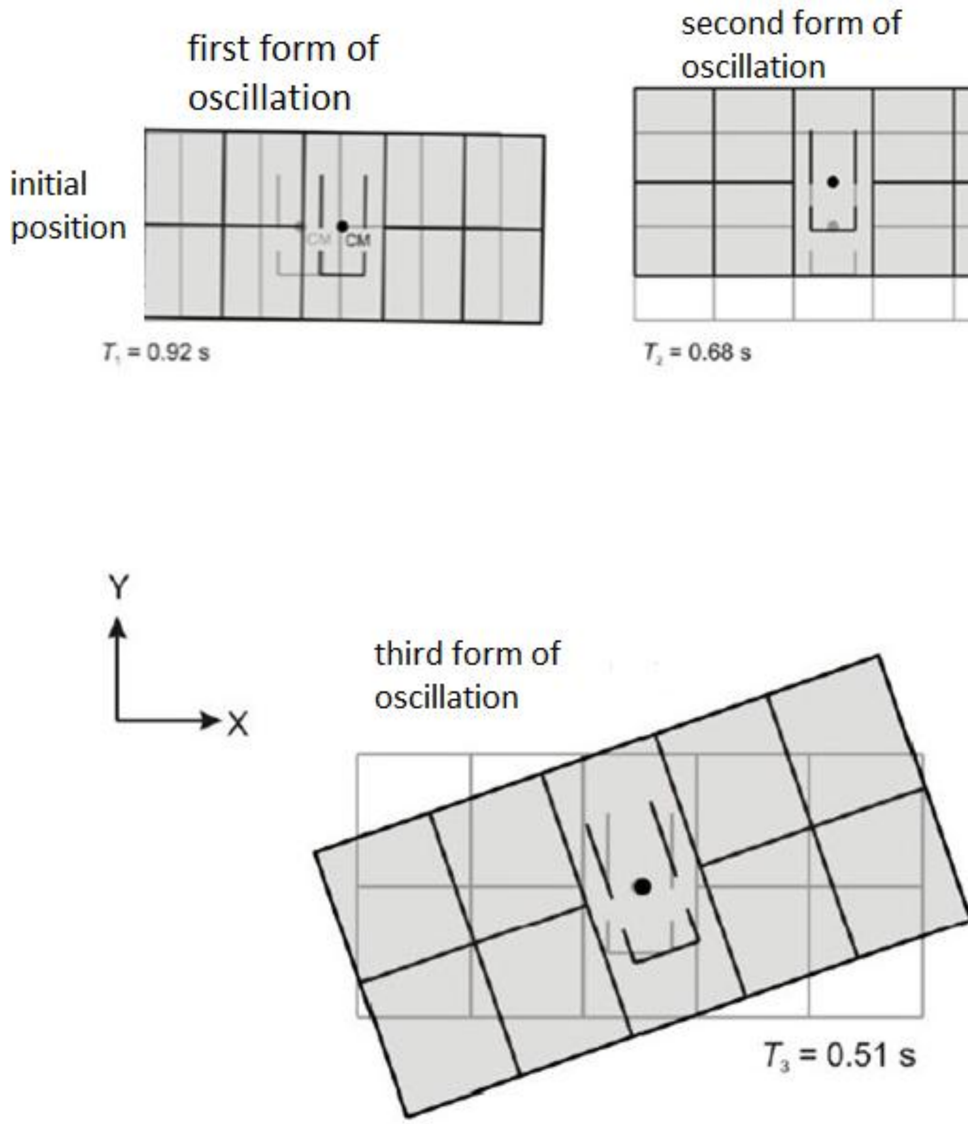


Figure 44. The main forms of oscillating.

5.15. The effects of accidental torsion

Torsion effects are considered by means of torsional moments (M_{Xi} dhe M_{YI}) around the vertical axis. They are defined as a product of horizontal forces in any horizontal direction (F_{XI} and F_{YI}) and respective accidental eccentricity (e_{xi} and e_{Yi}).Accidental eccentricity are equal to 5% of the size of the floor (L_{XI} dhe L_{Yi}). It will be considered only torques above floor 0.

$M_{xi}=F_{xi}*e_{yi}$ and (44)

$M_{Yi}=F_{Yi}*e_{xi}$ (45)

Table 8. Torsional moments

Floor	Lxi	Lyi	exi	eyi	Fxi	Fyi	Mxi	Myi
	(m)	(m)	(m)	(m)	(kN/)	(kN)	kN(m)	(kNm)
roof	30	14	1.5	0.7	703	951	492	1426
Floor 5	30	14	1.5	0.7	630	852	441	1278
Floor 4	30	14	1.5	0.7	512	692	358	1039
Floor 3	30	14	1.5	0.7	394	533	276	799
Floor 2	30	14	1.5	0.7	276	373	193	559
Floor 1	30	14	1.5	0.7	162	220	114	329

Table 9 Story forces of frame model.

Story	Load Case/Combo	Location	P kN	VX kN	VY kN	T kN-m	MX kN-m	MY kN-m
Story8	Comb1	Top	11874.7013	0	0	0	83122.9089	-174255
Story8	Comb1	Bottom	12288.2791	0	0	0	86017.9538	-180459
Story7	Comb1	Top	24162.9804	0	0	0	169140.8627	-354714
Story7	Comb1	Bottom	24576.5582	0	0	0	172035.9075	-360917
Story6	Comb1	Top	36451.2595	0	0	0	255158.8164	-535172

Story	Load Case/Combo	Location	P kN	VX kN	VY kN	T kN-m	MX kN-m	MY kN-m
Story6	Comb1	Bottom	36864.8373	0	0	0	258053.8613	-541376
Story5	Comb1	Top	48739.5386	0	0	0	341176.7702	-715631
Story5	Comb1	Bottom	49153.1164	0	0	0	344071.815	-721835
Story4	Comb1	Top	61027.8177	0	0	0	427194.7239	-896089
Story4	Comb1	Bottom	61441.3955	0	0	0	430089.7688	-902293
Story3	Comb1	Top	73316.0968	0	0	0	513212.6777	-1076548
Story3	Comb1	Bottom	73729.6746	0	0	0	516107.7225	-1082752
Story2	Comb1	Top	90612.152	0	0	0	689572.6037	-1335989
Story2	Comb1	Bottom	92824.1832	0	0	0	712857.714	-1369169
Story1	Comb1	Top	109172.8456	0	0	0	884518.6684	-1614399
Story1	Comb1	Bottom	113713.1045	0	0	0	932269.7636	-1682503
Story8	Dead	Top	6561.9295	0	0	0	45933.5064	-95667.8298
Story8	Dead	Bottom	6857.3422	0	0	0	48001.3955	-100099
Story7	Dead	Top	13419.2717	0	0	0	93934.9019	-195767
Story7	Dead	Bottom	13714.6844	0	0	0	96002.7911	-200198
Story6	Dead	Top	20276.6139	0	0	0	141936.2974	-295866
Story6	Dead	Bottom	20572.0267	0	0	0	144004.1866	-300297
Story5	Dead	Top	27133.9561	0	0	0	189937.693	-395965
Story5	Dead	Bottom	27429.3689	0	0	0	192005.5822	-400396
Story4	Dead	Top	33991.2984	0	0	0	237939.0885	-496064
Story4	Dead	Bottom	34286.7111	0	0	0	240006.9777	-500495
Story3	Dead	Top	40848.6406	0	0	0	285940.4841	-596163
Story3	Dead	Bottom	41144.0533	0	0	0	288008.3732	-600594
Story2	Dead	Top	50322.9657	0	0	0	381671.8598	-738278
Story2	Dead	Bottom	51902.988	0	0	0	398304.0814	-761978
Story1	Dead	Top	60700.604	0	0	0	490679.0488	-893942
Story1	Dead	Bottom	63943.646	0	0	0	524786.974	-942588
Story8	LCase1 Max	Top	0	3917.8521	3891.5055	69442.1473	0	0
Story8	LCase1 Max	Bottom	0	3917.8521	3891.5055	69442.1473	11674.5166	11753.5564
Story7	LCase1 Max	Top	0	7369.1075	7218.3309	129313.5592	11674.5166	11753.5564
Story7	LCase1 Max	Bottom	0	7369.1075	7218.3309	129313.5592	33158.282	33665.8264
Story6	LCase1 Max	Top	0	10221.3979	9965.5523	178590.5081	33158.282	33665.8264
Story6	LCase1 Max	Bottom	0	10221.3979	9965.5523	178590.5081	62671.4866	63960.1324
Story5	LCase1 Max	Top	0	12418.8703	12096.1342	216722.6281	62671.4866	63960.1324
Story5	LCase1 Max	Bottom	0	12418.8703	12096.1342	216722.6281	98374.6058	100648.1659
Story4	LCase1 Max	Top	0	13870.4437	13567.9667	242777.0332	98374.6058	100648.1659
Story4	LCase1 Max	Bottom	0	13870.4437	13567.9667	242777.0332	138320.8299	141528.6949
Story3	LCase1 Max	Top	0	14471.1633	14316.4144	255414.0445	138320.8299	141528.6949
Story3	LCase1 Max	Bottom	0	14471.1633	14316.4144	255414.0445	180439.907	184219.6497
Story2	LCase1 Max	Top	0	14505.5785	14378.6219	256420.8158	180439.907	184219.6497
Story2	LCase1 Max	Bottom	0	14505.5785	14378.6219	256420.8158	223001.3106	227260.0823
Story1	LCase1 Max	Top	0	14518.2829	14401.2843	256778.7848	223001.3106	227260.0823
Story1	LCase1 Max	Bottom	0	14518.2829	14401.2843	256778.7848	280096.8447	284915.491
Story8	Live	Top	1680	0	0	0	11760	-25200
Story8	Live	Bottom	1680	0	0	0	11760	-25200

Story	Load Case/Combo	Location	P kN	VX kN	VY kN	T kN-m	MX kN-m	MY kN-m
Story7	Live	Top	3360	0	0	0	23520	-50400
Story7	Live	Bottom	3360	0	0	0	23520	-50400
Story6	Live	Top	5040	0	0	0	35280	-75600
Story6	Live	Bottom	5040	0	0	0	35280	-75600
Story5	Live	Top	6720	0	0	0	47040	-100800
Story5	Live	Bottom	6720	0	0	0	47040	-100800
Story4	Live	Top	8400	0	0	0	58800	-126000
Story4	Live	Bottom	8400	0	0	0	58800	-126000
Story3	Live	Top	10080	0	0	0	70560	-151200
Story3	Live	Bottom	10080	0	0	0	70560	-151200
Story2	Live	Top	12600	0	0	0	97020	-189000
Story2	Live	Bottom	12600	0	0	0	97020	-189000
Story1	Live	Top	15120	0	0	0	123480	-226800
Story1	Live	Bottom	15120	0	0	0	123480	-226800

Two options of combination are shown. In the first option, the amount of effects from four groups of torsional moments (+ M_{Xi}, - M_{Xi}, + M_{YI} - M_{YI}) is added (SRSS) combined results of seismic operations in two orthogonal directions taken by the modal analysis of the response spectrum. Torsional moments due to horizontal loading in the direction Y (M_{YI}) are greater than those in the direction X (M_{Xi}). Therefore, the final effects of torsional moments defined as more torque M_{YI} with positive and negative signs loading. In the second option, firstly, the effects resulting from torque moments due to the action of seismic in a single direction with positive and negative sign of the loading are combined with the results of modal analysis of response spectrum to the same horizontal component of the seismic actio. Then, the results for both directions included with torsional effects are combined under Rule SRSS.

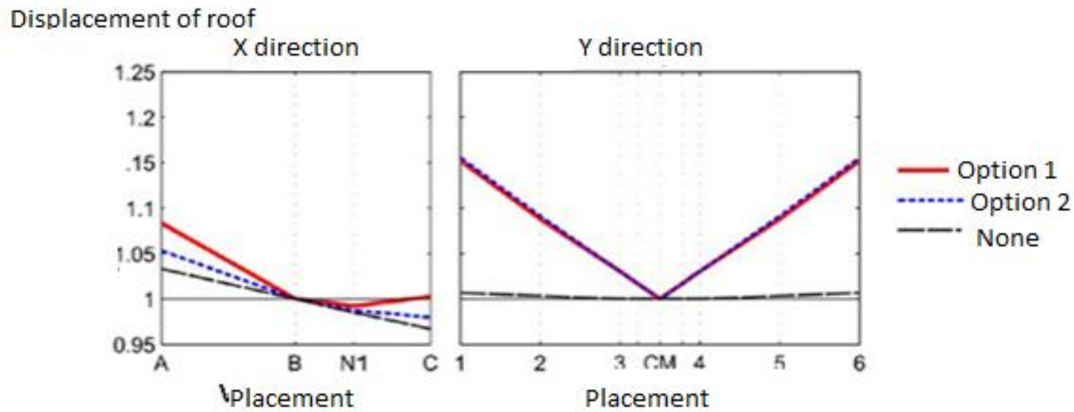


Figure 45. Torsional Effects due to movement of the roof in both directions.

5.16. Shear forces

Shear forces at the base of the structure derived from the analysis of the modal response spectrum for direction X reaches F_{bX} = 2693 kN. The ratio of shear forces in base with total weight of the structure of the floor above 0 is equal 2693/(2363 * 9,81) = 12%. For direction Y, the main shear force is F_{bY} = 3452 kN and the ratio of the shear force in base versus weight of the total structure is 3452/(3452 * 9,81) = 15%.

$$Fbi=(Sa1/g)*W \tag{46}$$

S_{a1}→spectral acceleration referring to period T_1

W*=WE_1→ effective weight of the building to the first form of oscillating.

E1=(0.7+0.8) for multi-storey reinforced concrete buildings.

It is notable that the shear forces in floors in two basement levels are equal to those in level 1, because the basement measures are ignored.

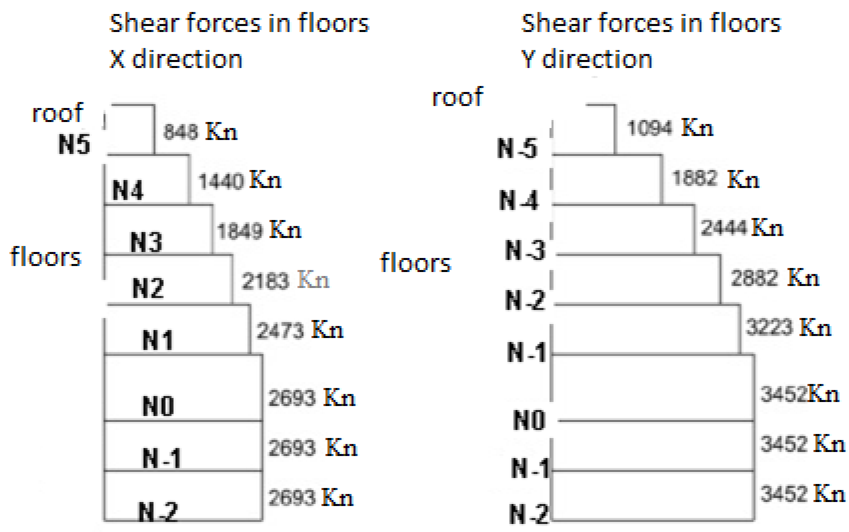


Figure 46. Shear forces in floors in height in both directions taken by the modal analysis of the response spectrum.

A quick check on the basis of calculated shear force can be done by comparing it with the greatest value shear force, which can be determined by multiplying the total mass with spectral acceleration of the design.

Considering M = 2362 tons and SD (T = 0.92 s) = 0.14g in direction X and SD (T = 0.68s) = 0.18g in direction Y, are taken the values of shear forces.

Lower values presented in the table can be obtained in a similar manner, but considering effective mass for the respective effective final mode (80.2% and 76.3% the total mass of the basement in the direction, X and Y) instead of the total mass.

Table 10. shear forces in basement

Shear force		Lower value		Great value		Medium value	
Direction X		2602 Kn		3244 kN		2693 kN	
Direction y		3182 kN		4171 kN		3452 kN	

5.17. Displacement

According to EuroCode8 actual displacement of a point of the structural system (dS) it shall be calculated as the product of the behavior factor q with displacement at the same point (de) obtained from modal analysis of the response spectrum (including torsional effects). In our case, factor q is 3.0. Displacements in center of mass (CM) are given in table 7. The ratio of actual displacement in center of mass and total height of the building above basement is 0.118/0.19=0.6% and 0.089/19=0.5% for direction X and Y.

Table 11. Displacements in center of mass in height (de and ds) in both directions

Floor	de (m)		ds=de*q (m)	
	Direction X	Direction Y	Direction X	Direction Y
roof	0.039	0.03	0.118	0.089
Floor 5	0.033	0.024	0.1	0.073
Floor 4	0.027	0.019	0.08	0.056
Floor 3	0.02	0.013	0.06	0.04
Floor 2	0.013	0.008	0.039	0.024
Floor 1	0.007	0.004	0.02	0.011
Floor 0	0	0	0.001	0.001
Floor -1	0	0	0	0

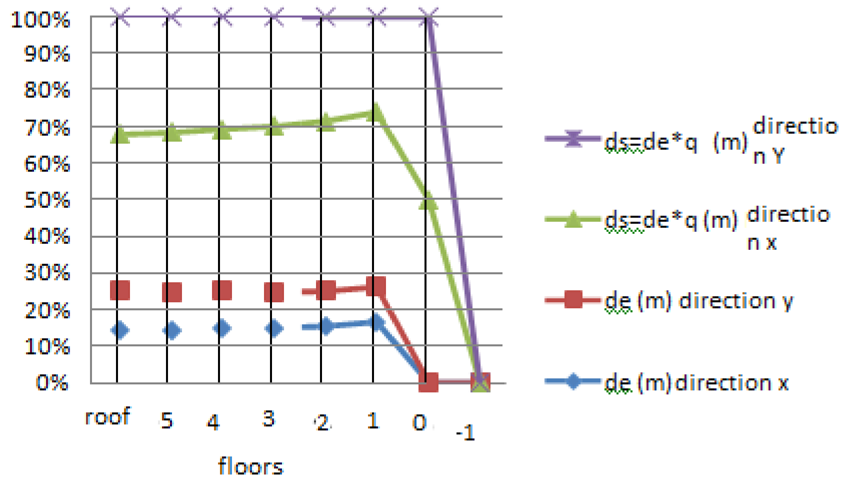


Figure 47. Actual displacements in center of mass (ds) in both directions X and Y

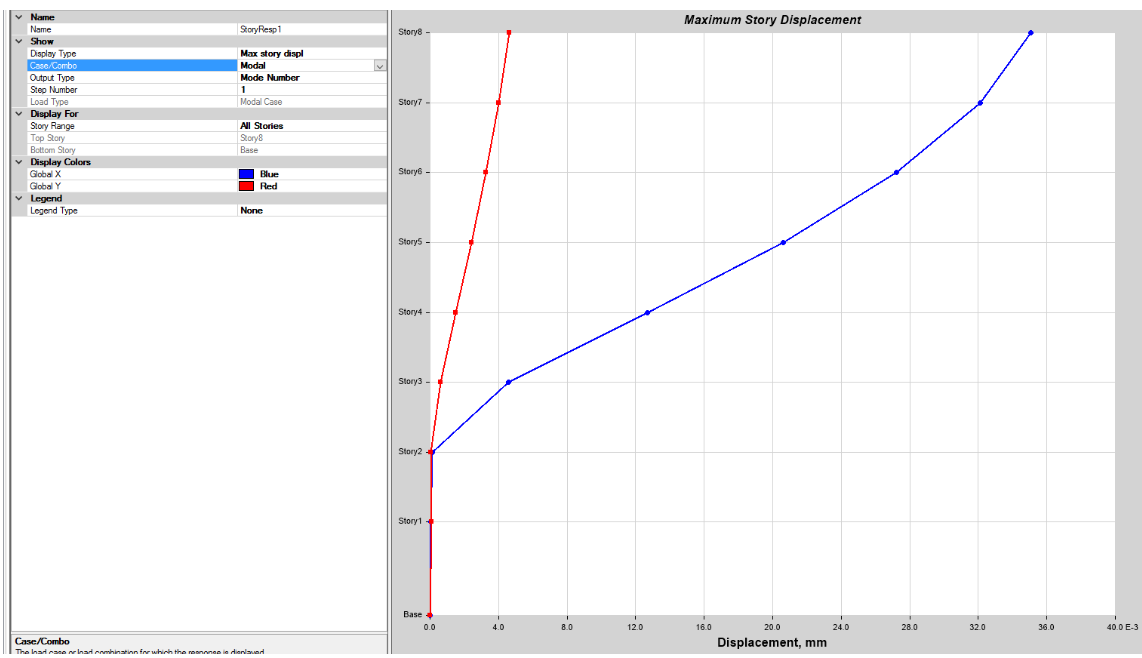


Figure 48. Displacement of frame model stories.

5.18. Limits of damages

According to EuroCodit 8 inelastic displacement $\Delta = q * \Delta el$ at any point due to seismic action indicated by the symbol dr. Displacements dr are taken as a result of the multiplication of displacements resulting from standard linear elastic analysis for the design of seismic action including the behaviour factor q.

$$dr = de * q \tag{47}$$

$$d_r * \nu \leq \alpha * h \rightarrow \frac{d_r}{h} \leq \frac{\alpha}{\nu} \tag{48}$$

Relative displacemnt dr is defined as the difference of the average lateral displacement ds in CM at the beginning and end of the floor. In the center of mass values shall be taken as the average value. Relative displacements taken shall be determined for each vibration mode and combined as rule of combination.

- h It is the height of the floor.

- ν is the reduction factor that takes in consideration the lowest period of the seismic action. It depends on the importance of building classes. Building under study is classified as class II importance and have factor reduction ν = 0.5. α- a factor which takes into account the type of non – structural elements and their role in the structure. α takes values (0.005 , 0,0075 and 0.01)

Table 12. Control of the relative displacement in both directions floors

floor	dr(m) in	CM	h	ν	ν*dr/h		(a)	α	(c)
	direction X	direction Y	(m)		direction X	direction Y		(b)	
roof	0.019	0.016	3	0.5	0.0031	0.0027			
floor 5	0.021	0.017	3	0.5	0.0034	0.0028			
floor 4	0.022	0.017	3	0.5	0.0036	0.0028	0.005	0.0075	0.01
floor 3	0.022	0.016	3	0.5	0.0036	0.0026			
floor 2	0.02	0.013	3	0.5	0.0033	0.0022			
floor 1	0.02	0.01	3	0.5	0.0025	0.0013			

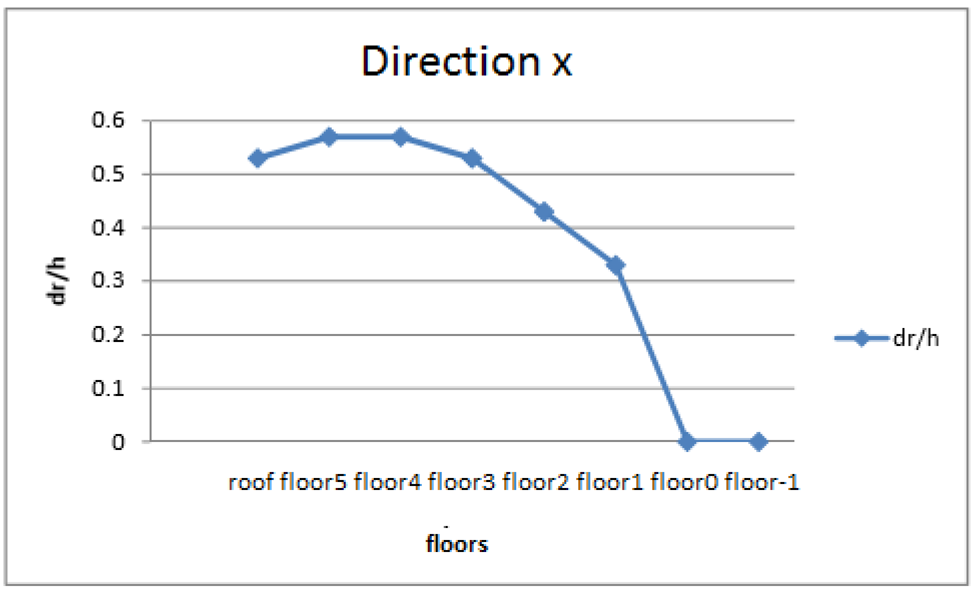


Figure 49. Check of shifting floors in both directions.

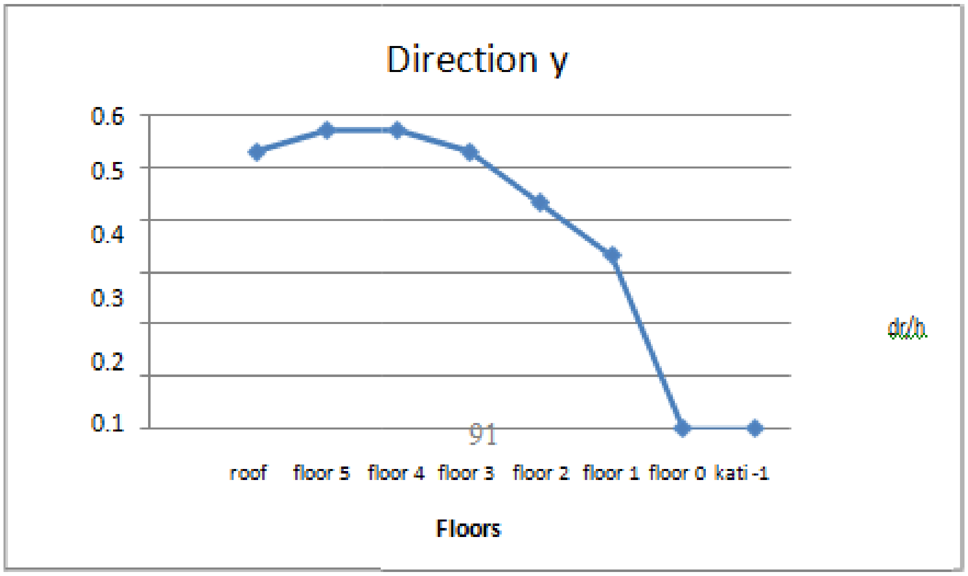


Figure 50. Check of shifting floors in both directions.

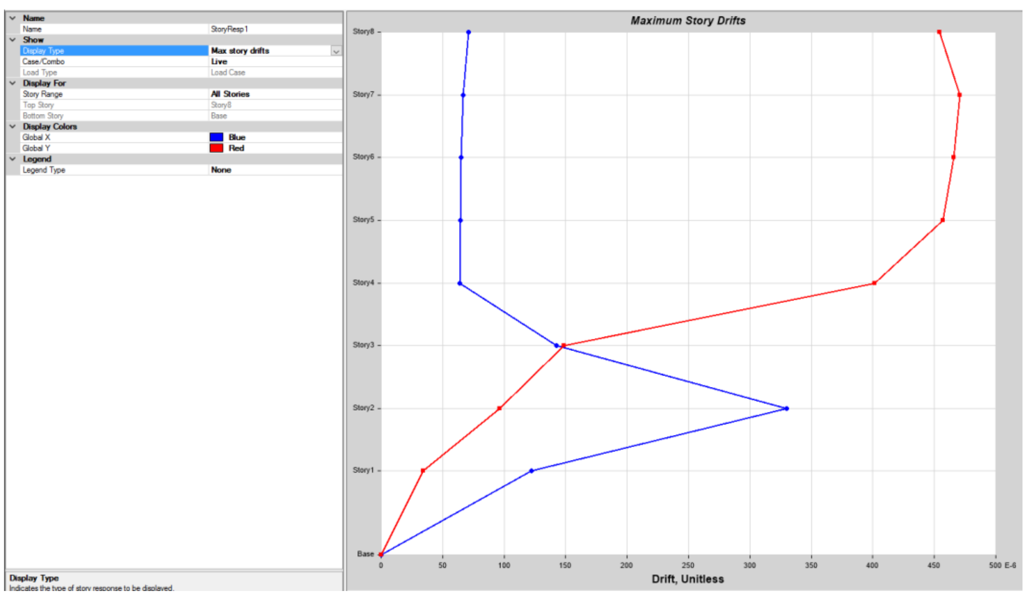


Figure 51. Maximum story drifts in frame model.

5.19. Accidental torsion criteria

The criteria taking into account the side effects are based on the relative displacement of the coefficient e which deals with the equation:

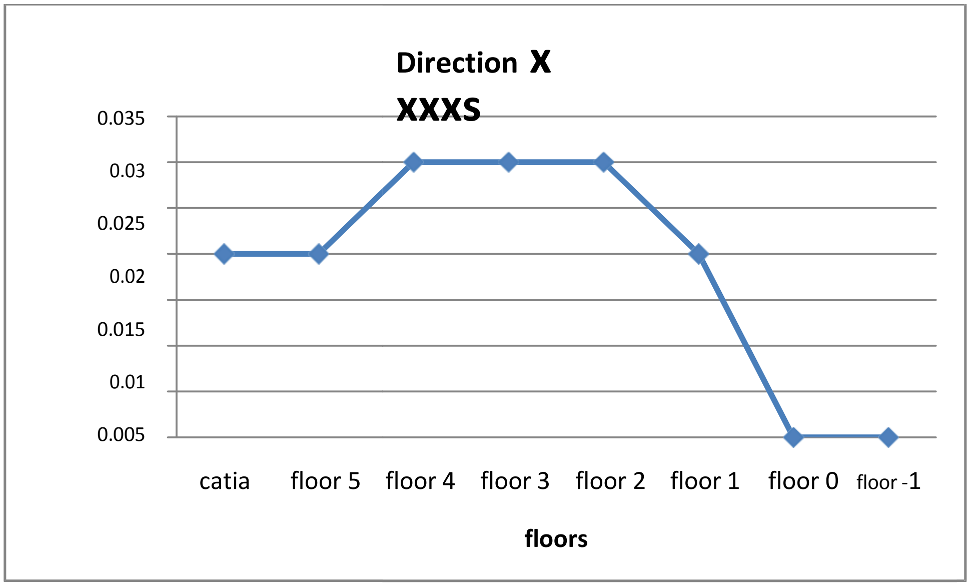
$$\theta = P_{tot}d_r/(V_{tot*h}) \tag{49}$$

- where dr is the relative displacements of the floor,
- h is the height of the floor
- Vtot The total seismic force of the floor taken from model analysis of the response spectrum
- Ptot is the total load of the gravity in and above the considerate floor in seismic design situation (G + 0.3Q).

In our case, secondary side effects are ignored; because sensitive coefficient of the displacement of the floor e is lower than 1.1 in all the floors in both directions.

Table 13. Determination of the sensitivity angle of relative displacement θ

Floor	Ptot	h	V tot	(kN)	dr (m) ne	CM	θ	
	(kN)	(m)	drejtimi X	drejtimi Y	(kN/)	(kN)	Drejtimi X	Drejtimi Y
roof	3650	3	848	1094	0.019	0.016	0.03	0.02
floor 5	7659	3	1440	1882	0.021	0.017	0.04	0.02
floor 4	11669	3	1849	2444	0.022	0.017	0.05	0.03
floor 3	15678	3	2183	2882	0.022	0.016	0.05	0.03
floor 2	19688	3	2473	3223	0.02	0.013	0.05	0.03
floor 1	23817	4	2693	3452	0.02	0.01	0.04	0.02



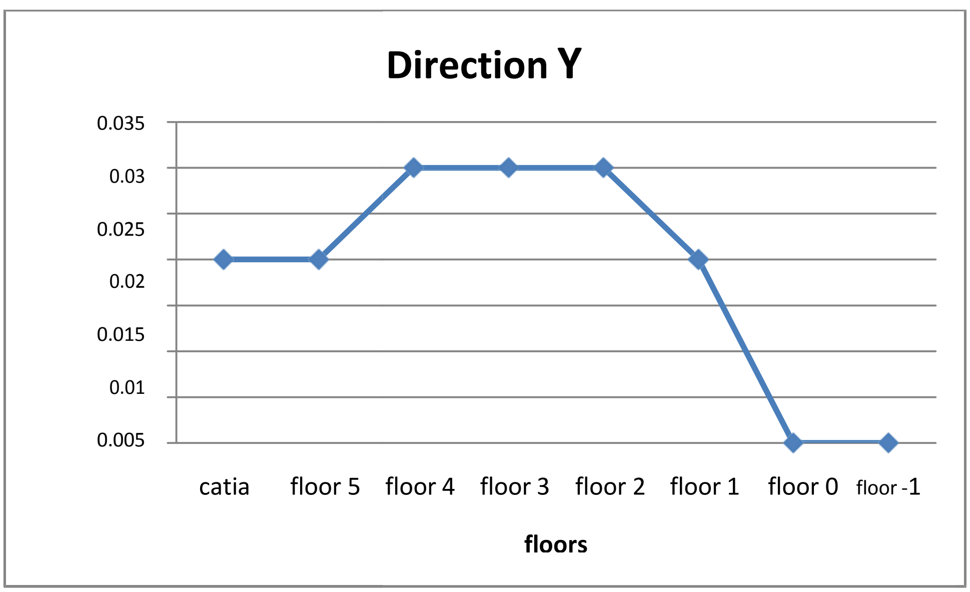


Figure 52. Check of slab displacement in both directions

5.20. Seismic design situations

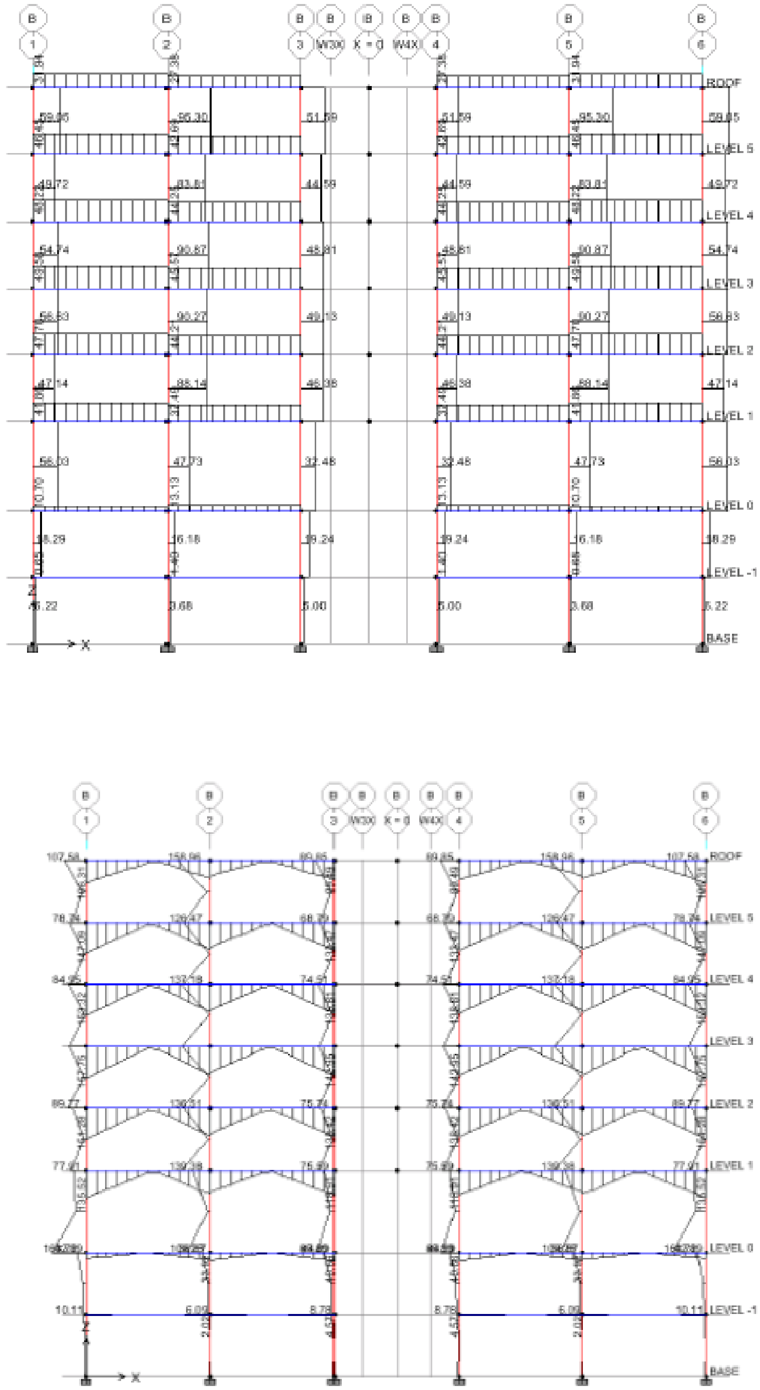
$$1.0*G+\Psi_{2i}*Q\pm E_{XY}(\pm M_a) \quad (50)$$

Where G - They are permanent gravity loads (own weight and permanent loads)

- Q- Is temporary load which is reduced by a factor Ψ_{2i}=0.3(for office building)
- E_{XY}- is seismic action combined in both directions taken from the modal analysis of the response spectrum including torsional effects ($\pm M_a$).

5.21. Internal forces

Shear forces and bending moments obtained by modal analysis and response spectrum given in the figures below.



(a)

(b)

Figure 53 Shear forces (a) and bending moments (b) for internal frame B in direction X taken from modal analysis of response spectrum. All units are in kN.

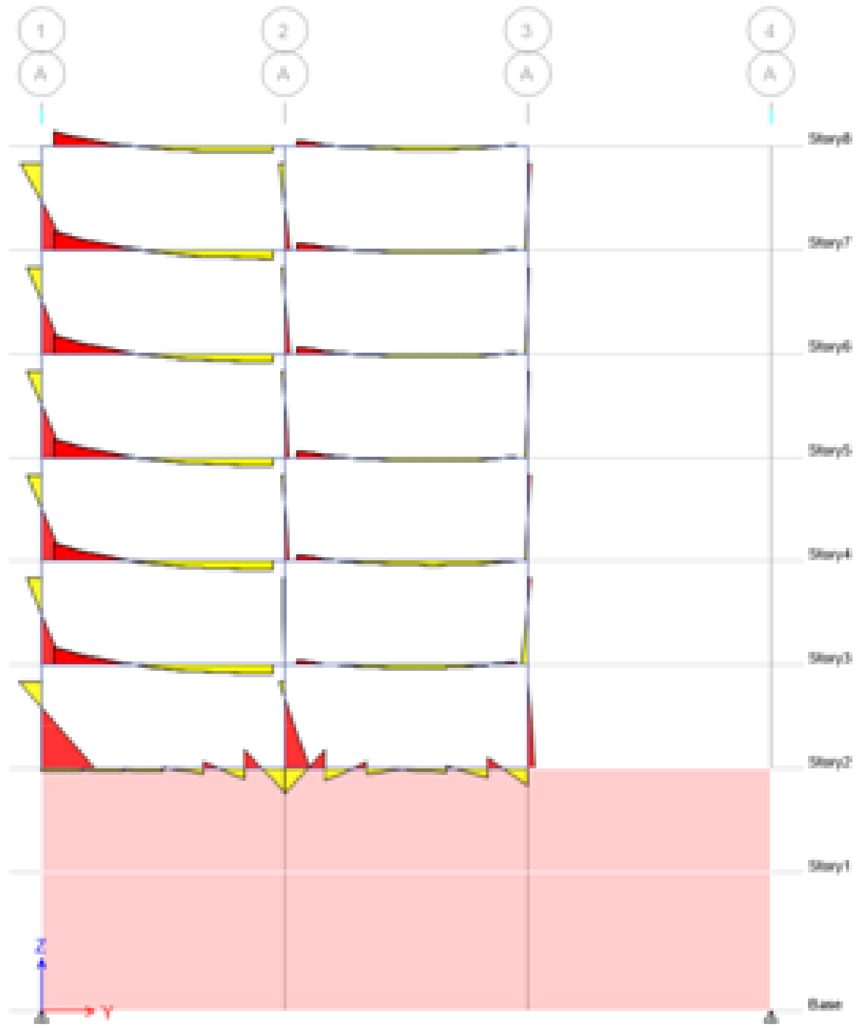
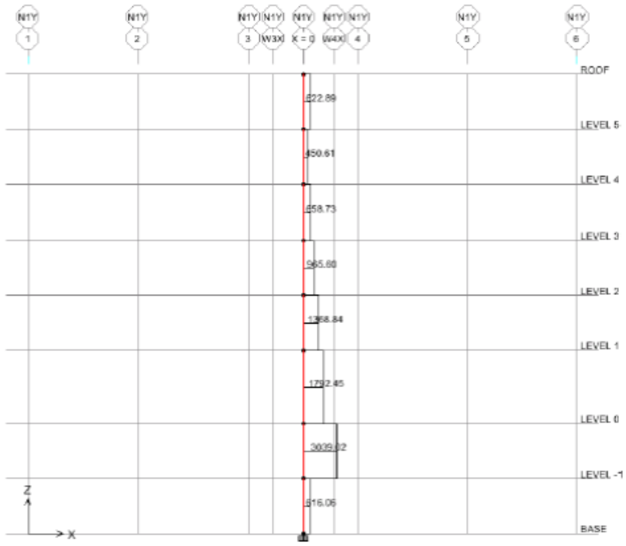
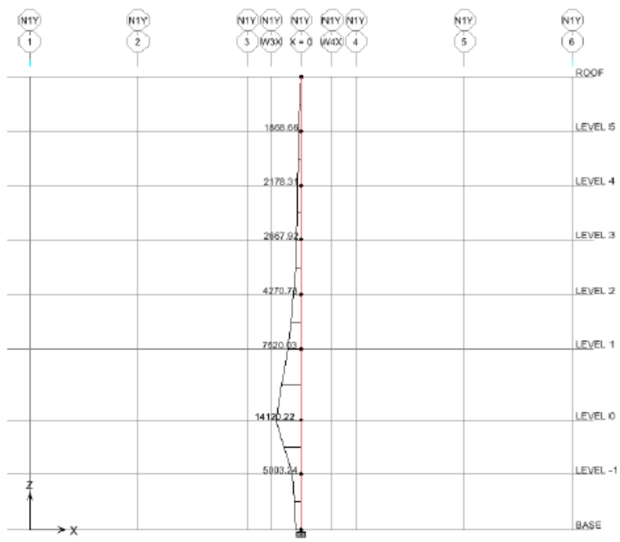


Figure 54. Shear forces and bending moments in frame model analysis.



(a)



(b)

Figure 55. Shear force (a) and bending moment (b) for wall N1 in direction X taken from modal analysis of response spectrum.

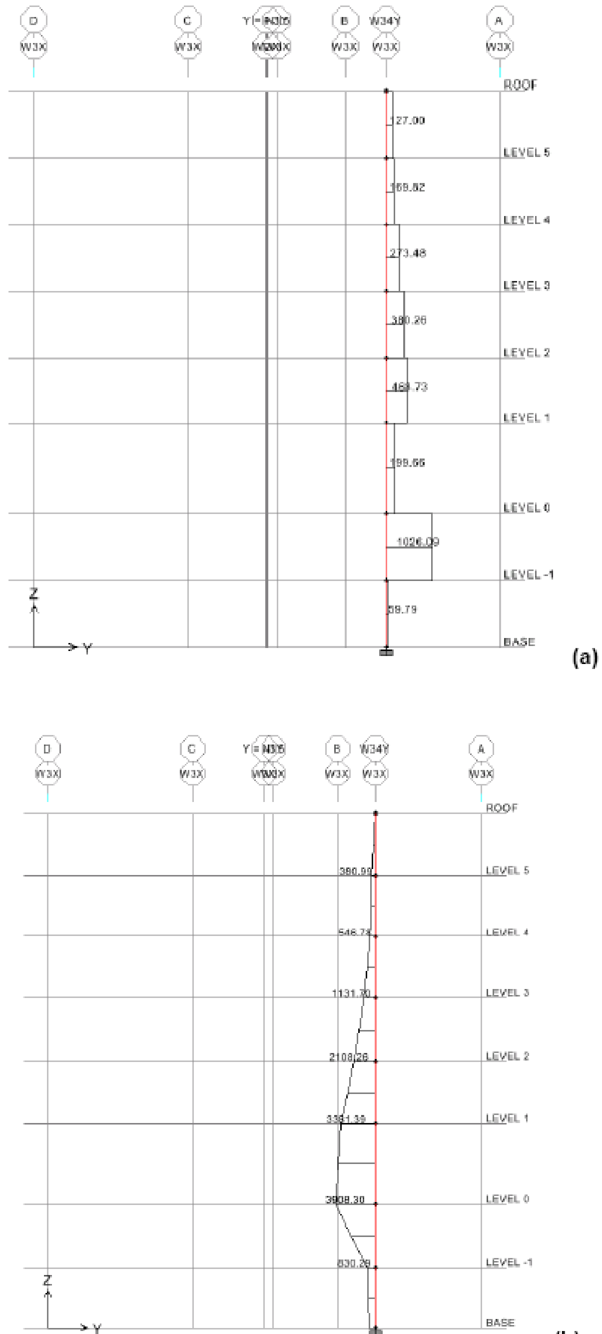


Figure 56. Shear force (a) and bending moment (b) for wall W3 in direction X taken from modal analysis of response spectrum

5.22. Lateral forces and analysis methods

The structure is categorized as regular in height and main modal periods in both directions (TX=0.92 s and TY = 0.68 s) are smaller than the minimum of 2 s and 4Tc, where Tc = 0.5 s.

The main period T1 for each direction is calculated according to the equation of Reiyel:

$$T_1=2\pi * \sqrt{\frac{\sum_{i=1}^{n}(m_i * s_i^2)}{\sum_{i=1}^{n}(f_i * s_i)}} \tag{51}$$

- where n=6 is the floor number above 0
- mi- floor weight(only weight above 0 are taken in account)
- fi- horizontal forces that are acting on floor 1 in center of mass of the floor
- si- displacements of weight are caused by horizontal forces fi. Table 10 Forces that are needed to determine main period T1 according to Relejit method.

Table 14 Main period is T1=0.91s and 0.72s for direction X and Y

Floor	fxi=fyi	Sxi	Syi	mi
	(kN)	(m)	(m)	(ton)
roof	1900	0.1051	0.0599	372
floor 5	1600	0.0891	0.0491	396
floor 4	1300	0.0715	0.038	396
floor 3	1000	0.053	0.0268	396
floor 2	700	0.0346	0.0164	396

5.23. Shear forces in basement and distribution of horizontal forces in height.

Seismic shear force Fb for each direction is determined by:

$$Fb=Sd*(T1)*m*\lambda \tag{52}$$

- where m-total weight above basement m=2362 ton
- T1- is the main period in X and Y direction ($T_{1,x}$=0.92s and $T_{1,Y}$=0.68s)
- Sd(T1)- It is the coordinates of the design spectrum for the period T1(Sd($T_{1,X}$=0.92s)=0.14g and Sd($T_{1,Y}$=0.68)=0.18g
- λ=0.85(the building has more than 2 floors and T1≤2Tc in both directions,Tc=0.5s)
- Base shear force is Fb,X=2676 kN(12% The total mass of the basement in the directions X and Y).

Horisontal force in floor 1 Fi is determined:

$$Fi=Fb * \frac{z_i * m_i}{\sum z_{j*m_j}} \tag{53}$$

- where mi,mj- are measures of the floors
- zi,zj- are the heights above level 0.

Table 15. Determination of horizontal forces in both directions (F_{xi} and F_{Yi})

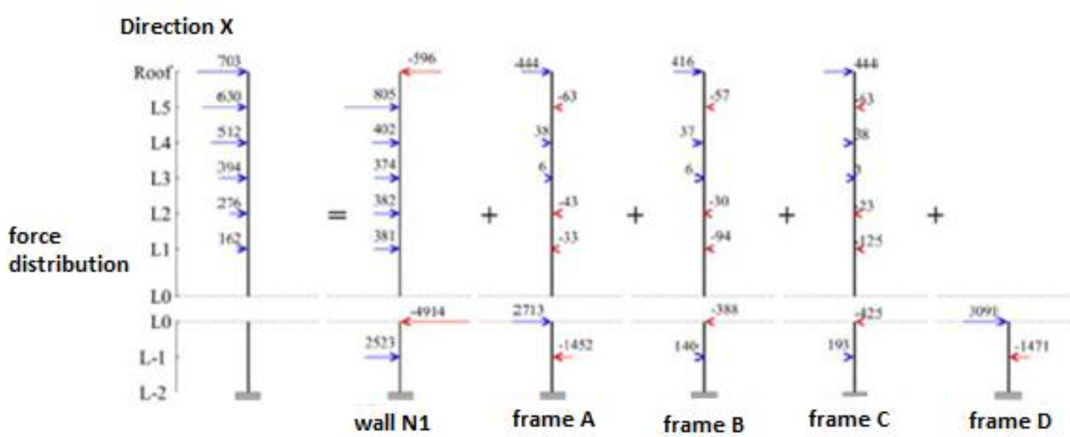
Floor	zi	mi	mi*zi	Fxi	Fyi
	(m)	(ton)		(kN)	(kN)
Roof	19	372	7063	703	951
floor 5	16	396	6329	630	852

floor 4	13	396	5142	512	692
floor 3	10	396	3956	394	533
floor 2	7	396	2769	276	373
floor 1	4	408	1631	162	220
		∑Meff=	26890	2676	3621

5.24. The distribution of horizontal and shear forces in each frame and wall

Distributions of internal forces are quite irregular because the structure consists of special elements (frames and walls), which are characterized by different forms of deformation. Great irregularity occurs at ground level (level 0), where loads are transferred from the peripheral walls with greater stiffness.

Irregularities will be reduced if we do not take into account the deformation of the plate.



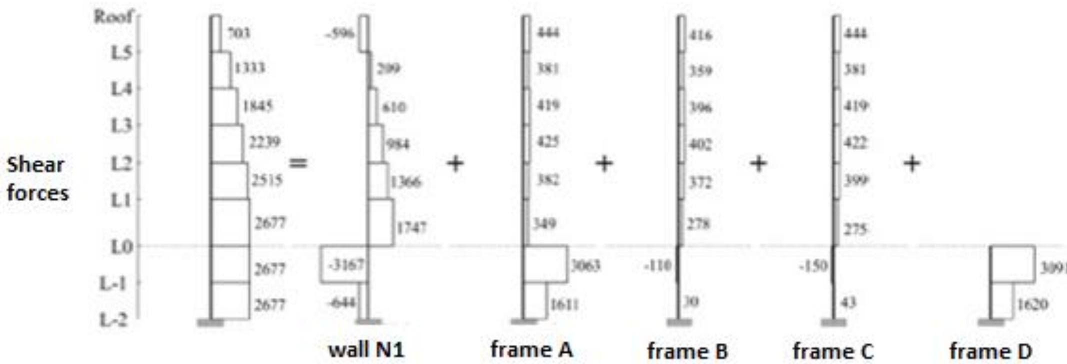


Figure 57. The distribution of horizontal forces and shear forces for each frame and walls in the direction X

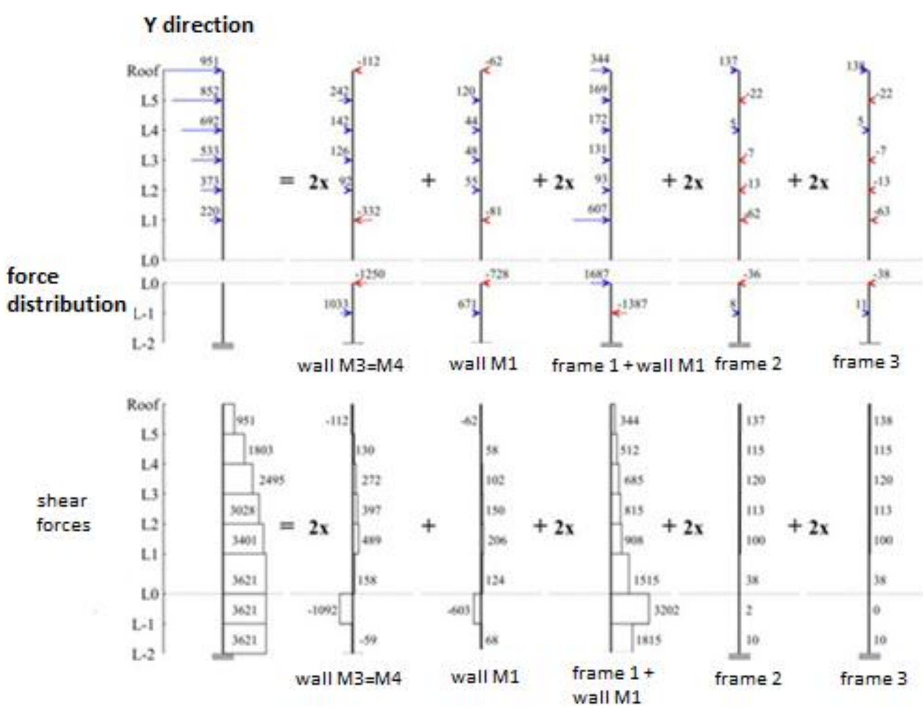


Figure 58. The distribution of horizontal forces and shear forces for each frame and walls in the direction Y.

CHAPTER 6

RECOMMANDATIONS OF EUROCODE8 FOR USE OF SHEAR WALLS IN THE STRUCTURE

6.1. Ductile wall

Wall fixed in base in a way that prevent the relative rotation of the base against the other part of the structural system, designed and detailed to delete energy in bending plastic hinges area, that has no space just above his base.

6.1.1. Oversized wall with light reinforce

Oversized wall cross section (horizontal size lw at least equal to 4,0m as two-thirds of the height hw of the wall, is considered –the one which is lower), because of which is expected to occur limited cracks and inelastic behavior in seismic design situation.

6.2. Geometrical requirements

6.2.1. For ductile walls

• Thickness *b*wo The rib wall would have to satisfy the condition:

$b_{wo} \geq \max\ \{150\ \text{mm},\ h_s/20\}$

(57) where :

h_s is height “ clear” of the floor.

6.2.2. For oversized walls

Light reinforced is applied the same demand.

6.3. Special measures for ductile walls

- Uncertainties in the analysis and post-elastic dynamic effects should be taken into account at least through an appropriate simplified method. If there is not any more precise method, for this purpose can be used rules that are given in the following points.

- Is allowed to mass redistribution till 30% of the seismic action effects between primary seismic walls, provided that the request of the total strength is not reduced.

It should be redistributed shear forces together with bending moments, in such a way that the ratio of the shear forces ("shear ratio") of special walls are not significantly affected. Walls subject to fluctuations (changes) of large axial forces (normal), eg in the coupled walls, It would still have moments and shear forces to be redistributed from the wall (walls).

- In coupled walls it is allowed redistribution till 30% of the mass of seismic action effects inbetween conjugative beams of different floors provided that the axial sismic force at the base of each special wall do not have to me touched.

•	Must be compensated (covered) the uncertainties associated with the distribution of the moment in height of temporary walls seismic primary (ratio hw / lw, the height versus length, greater than 2.0).

•	Considered that the request (4) above is complemented by applied, regardless of the type of analysis used, the following simplified methodology:

Appropriate moment diagram in wall height must be obtained by the diagram of bending moments taken from the analysis, by displacing it vertically (tension shift). This can be supposed as linear if the structure does not show significant non-continuity of the mass, rigidity or the height of its strength. this displacement would have to be consistent with the inclination type rod ("strut inclination") that is considered the control (verification) for cutting by the Ultimate Limit State (ULS - Ultimate Limit State), with model potential screw-type or blower ("fan- type pattern") of bars (trusses) near the base.

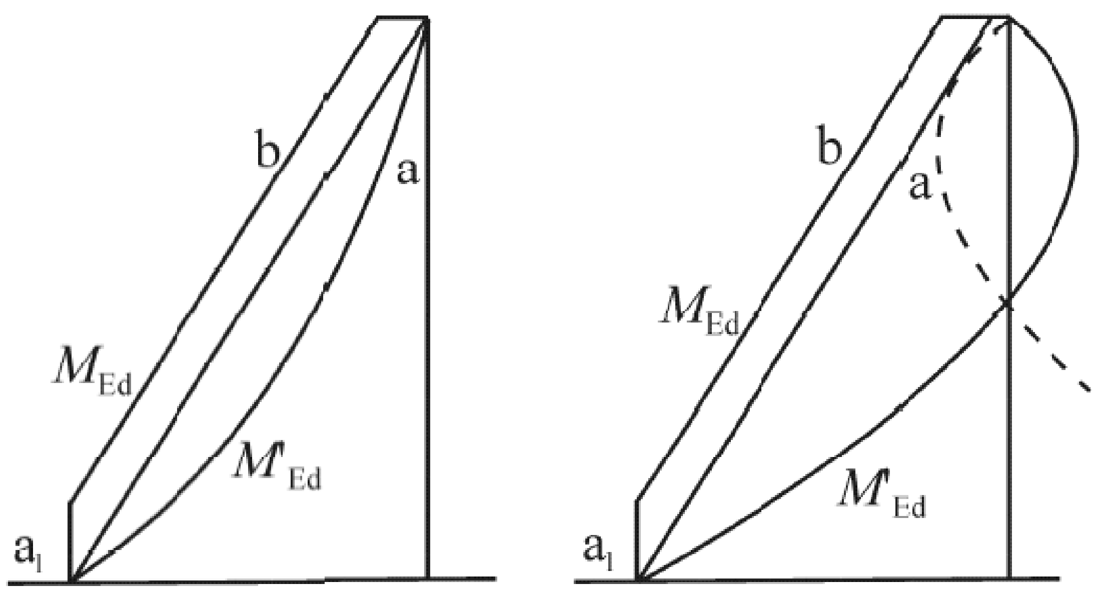


Figure 59. Bending moments wrapping design in bending walls (left, systems with wall; right: dual systems).

- It should be taken into account the possible increase of shear forces after emergence of fluency at the base of a primary seismic wall.

- Dual systems that form the flexible walls to take into account of the uncertainties due to the effects of the higher vibrational forms would have used shear forces wrapping design, built in accordance with Fig.59

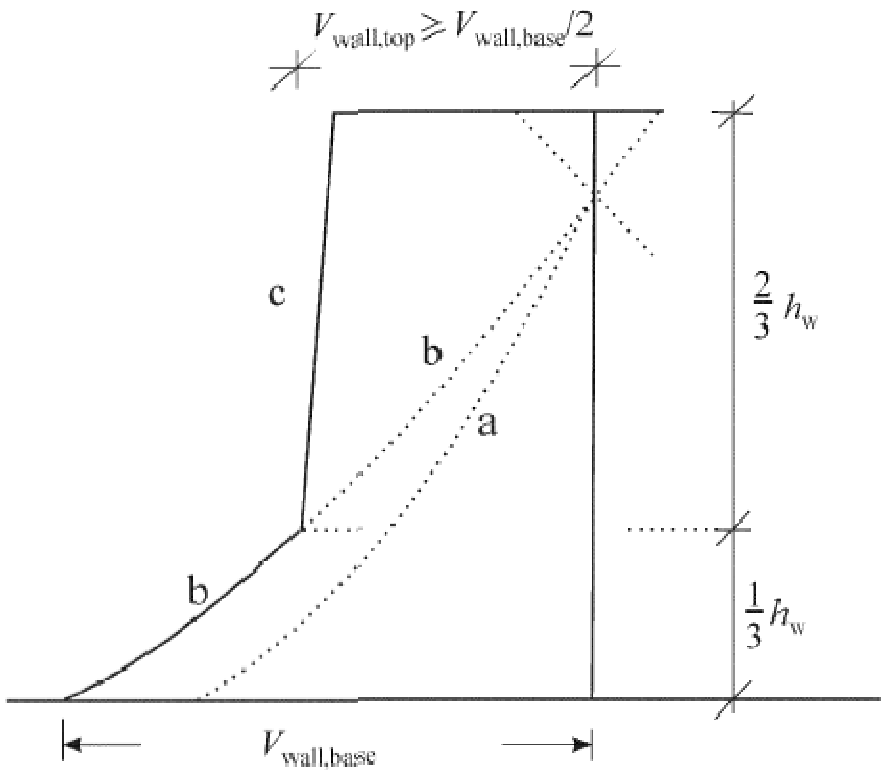


Figure 60. Shear forces in dual system walls

Legend:

a: diagram of shear forces from analysis

b: diagram of shear forces

6.4. Shear and bending resistance of ductile walls

- Shear and bending resistance are calculated according to Eurocode 2-EN 1992 1-

1:200X. for this purpose, is used the value of the axial force resulting from the analysis for the seismic design situation.

- In primary seismic walls would have the value of the normalized axial force νd shall not exceed 0.4.

- In calculating the bending resistance of sections of walls should not be considered vertical rib reinforcement.

- Composite sections of walls, which consist of rectangular segments associated or crosses between them (setions L-, T-, U-, I-, ore more) It would have to be considered as an integral unit (full), which consist of a few ribs or parallel ribs (approximately) to the direction of active seismic shear forces and from a tile or more tiles equal with. For the calculation of bending resistance, the effective width of tile in each rib site would have to be taken such that the edge of the site lies in a minimum determined by sizes:

a) The actual width of tile;

b) Half the distance from the neighbor wall rib; and

c) 25% of the total height above the level of the considered wall

CHAPTER 7

CONCLUSION

In this diploma thesis is developed seismic response of mixed reinforced concrete buildings, including deformation and damage that may incur in structures under earthquake action. It is studied the effect of shear walls in a building with 2 floors basement and 6 floors that serve as offices. Also it has been observed the distribution of seismic forces on the shear walls through the non-linear elastic analysis. From the study I made, some conclusions are derived.

Mixed reinforced concrete structures represent a suitable constructive system for small or medium costructions for economic advantages that they show. The principle, that gives the horizontal action to the element of the same composition, in our case, resulted approved and coherent with response from the horizontal loads of mixed structure.only when the dimensions of reinforced elements become comparable with walls, structural behaviour become mixed and for for the horizontal loads results necessary the verification with non-linear analysis (static or dynamics) as given from EuroCode8. The limit imposes to have at least 75% of the vertical loads in in resistance walls from horizontal loads, according to EuroCode8.

Although recent innovations in seismic and construction engineering, in recent years the rate of mortality is increased along with the losses of an economic due to the earthquake, the main causes are the presence and construction of structures more vulnerable in areas with high risk seismic.

Seismic actions include 2 main conditions: 1- the ultimate limit

2- Serviceability limit

Proper design of the building must be properly predict structural performance for each level of seismic action. For seismic action the structure must resist:

- Small Intensity actions without being damaged.

- Medium intensity actions with small repairable damage.

- High intensity actions without reaching collapse.

A proper design should provide the following steps:

1- The criteria for the required performance

2- Definition of structural verification criteria

3- The choice of structural model

4- Choosing the proper structural analysis

5- Definition of procedures for verification

- The criteria for the required performance

In general it is reasonable to assume that the structures must endure, to resist external action without incurring damage, whereas for rare events with high intensity it is possible to tolerate a possible damage but to be guaranteed stability of generalized building. An exceptionis made for special buildings.The criterias for required performance are:

- Safety of human life.

- Preventing the collapse of the building.

- Limitation of damages in the building.

- Limiting losses of historical and architectural assets

- Definition of structural verification criteria

Structures may be targets of high or medium intensity earthquakes without incurring any damage, damage reaching of the partial collapse or total one. Is obvious that the level of damage depends on the intensity of the earthquake: medium intensity earthquakes occures more often. High intensity earthquakes can occur 1-2 times during the life of the structures, so less often. to guarantee an appropriate structural response we must take treatment on more levels.

Treatments at two levels (according to Eurocodes)

- Ultimate limit, structures must stay flexible or able to undergo minor cracking. Structural elements should not suffer damage.

- Serviceability limit, structures can take advantage of their ability to be deformed beyond the elastic limit. Structural elements can not be damaged.

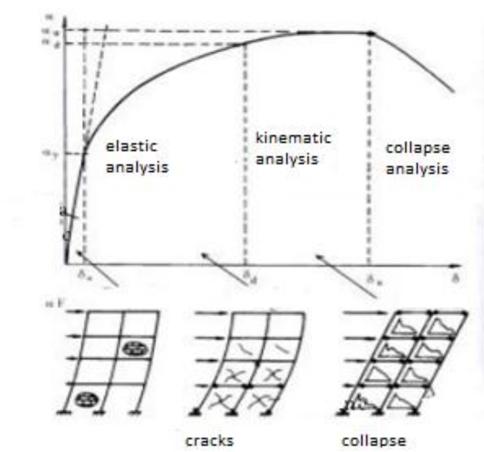


Figure 61. Elastic analysis, kinematic analysis and collapse.

Constructions must be equipped with a different level of seismic design in view of their importance and their use. For this purpose, various categories of importance are included.

- Description of the seismic actions

The reference model describing the seismic action under a point of floor surface is taken from the elastic response spectrum.

By studying how the acceleration value varies in function of time, can be noted that for T = 0, for rigid Structure, maximum acceleration of structure leads to acceleration of ground motion. With the increase of the period of absolute acceleration system Sa increase reaching values T=T. The maximum value is achieved when in free float system is characterized by the neutral period close to the value Tprevail, so resonance occures.

The response spectrum of displacement can be taken from direct transformation of elastic response spectrum of accelerations using the expression:

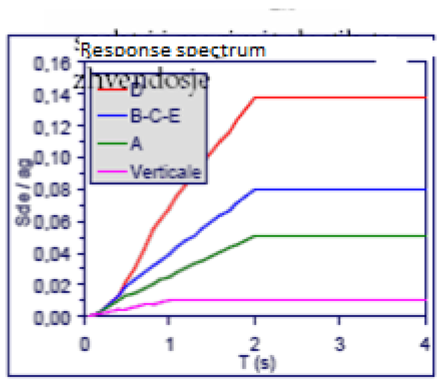


Figure 62. Response spectrum

Many existing structures have not received a proper defense, even though anti-seismic norms are stricter today, and the population growth in seismic areas is the cause of increased risk. To to breakdown this trend, reducing the number of victims of earthquake and economic damages, is necessary, improving the design of structures and to intervene in these buildings.

According to research made in previous earthquakes it shows that, in the same area where earthquakes have occurred, buildings with the same structure have suffered various damages.

Many structures previously designed have not a deformation capacity with proper distribution of power to resist horizontal forces in order to provide a resistance to exclude a collapse of the structure.

Studies of seismic vulnerability of the damages caused by the earthquakes in these regions are moderate risk, in which urban development assumes a greater threat..

Evaluation of seismic vulnerability is necessary in studies of seismic risk and it must be considered in any intervention to reduce the risk sometimes. In general, there are different methods for the evaluation of seismic risk. To evaluate the seismic risk and seismic response follow these steps:

1- Describes the main conceptual aspects and methodologies related to the seismic response in mixed reinforced concrete buildings.
2- Seismic actions are defined through response spectrum and seismicmetres for different seismic intensities.
3- Described a method for the evaluation of seismic response which is based on non-linear static analysis.
4- In mixed reinforced concrete buildings is analyzed structural model of the building under study taking into account the effects of fillings in total response of the structure.
5- In the end is made a nonlinear elastic analysis to verify some of the overall results obtained by method demonstrated

- Regarding the seismic action:
  - In this study, the seismic action, according to the methods used, is defined and used in spectral terms.
- As the methodology for determining the seismic response of mixed reinforced concrete building:

- Is described the method used for evaluation of the seismic response of structure taken in study based on nonlinear elastic analysis. This method has allowed defining the relative structure of the reaction spectrum through and displacement design spectrum.

- For damages caused :

This methodology we are using has 5 conditions:

- no damage
- minor damage
- medium damage
- Some not very big damage
- The structure collapse

$\Delta_y \rightarrow$ gives average condition and the point of last capacity.

$\Delta_u \rightarrow$ define the state of total collapse.

Damages not very big deal as $0.7\Delta_y$ and $\Delta_y+0.25(\Delta_u-\Delta_y)$

Experimental stability analysis shows that a 20% shift to state that the damage can cause slight variations with 70% of the state of medium damage at a critical point.

Experimental stability analysis shows that a 20% shift to state that the damage can cause slight variations with 70% of the state of medium damage at a critical point.

- With regard to the analysis of seismic damage in mixed reinforced concrete buildings

From the study is noticed that how base buildings have a great resistant capacity and even though results as most damaged ones. This occurs from structural rigidity that we can see from minor spectral displacements and have small or medium damages. It is interesting to note the impact of the response spectrum form in the damages caused. In reality the typical response spectrum in cities are expected to be in a band too narrow of constant accelerations; this is important because we can concentrate damages in lower and more rigid buildings.

Some conclusions for mixed reinforced concrete buildings:

1- All load factors of the buildings are bigger than the minimum of the overload factor which is taken from Eurocode.

2- Shear forces in base and lateral structural deformations are taken by using EuroCode values in both X and Y directions.

3- Reducing of modification factor of seismic response can cause greater seismic load in the structure.

4- We can have great loads from earthquake in the building, great lateral structures deformations

5- The dual system buildings designed by EuroCode will have the best performance and the building will have a better ductility.

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