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MÉMOIRE DE MASTER

Sciences et Technologies
Génie Civil
Structure

Réf. : S58/2018

Présenté et soutenu par :
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Le : samedi 23 juin 2018

ANALYSIS AND EVALUATION OF A REINFORCED CONCRETE FRAME STRUCTURE USING PUSHOVER ANALYSIS METHOD

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Année universitaire : 2017 - 2018

Acknowledgements

Before starting the presentation of this work, I take this opportunity to thank God who has helped me and gave me patience and courage during these years of study. Moreover, I thank all those who contributed directly or indirectly to the completion of this graduation project.

I would like to express my sincere thanks to my great and respectful Prof. CHEBILI RECHID, for agreeing to be in charge of supervising my graduation project, as well as for his support, his pertinent remarks and his encouragement.

I wish to express my deep gratitude and all my thoughts of gratitude to the amazing MR. ABDESSALAM ISSAM who accompanied me closely with every single step during this work as well as for his availability, for the trust he has given me and the valuable advice he has given me throughout the realization of this project.

In addition, I would like to thank my dearest friend KHELIFI AHMED deeply for his invaluable help, his availability as well as for his moral and scientific contribution and along with his fruitful pieces of advice.

My thanks also go to all my teachers and all the people who have supported me until the end, and who have never stopped sharing their very important advice as a sign of recognition.

Anouar Hafiane

DEDICATION

Praise to Almighty God whose enlightenment has guided my way to achieve this successes.

I dedicate this work to my family and my friends who have supported me through the difficult moments, without which I would not have reached this stage of study, as well as each moment of happiness that they procured for me.

To all my colleagues, university teachers and especially to my dear friends far beyond the oceans.

I say thank you for helping me in the realization of this thesis.

Abstract

Although the progress of studies that have focused on limiting damages due to earthquakes, this least still be the biggest obstacle that faces the development of structural engineering. Recently for example, numerous reinforced concrete buildings have been collapsed or totally destroyed under the effect of soil movements due to the seismic forces, those degradations are very sensitive and directly linked more to displacement than to efforts. The capacity curve obtained by the Pushover analysis emphasize this and shows clearly that the appearance of degradations and the total collapse occur when the structure reaches a certain limit of displacement.

<u>Key words</u>: frame RC structure, capacity curve, Equivalent Static Method (ESM), Equivalent strut, Pushover Analysis Method, performance point, demand spectrum.

ملخص:

رغم تقدم الدراسات التي ركزت أبحاثهاعلى الحد من الخسائر الناجمة عن الزلازل، إلا ان هذا الأخير يبقى أكبر عائق يواجه تطور هندسة الهياكل. مؤخرا كمثال، العديد من مباني الخرسانة المسلحة انهارت أو تهدمت كليا تحت تأثير حركة الأرض الناجمة عن القوى الزلزالية، هاته التدهورات مرتبطة مباشرة ومتعلقة بالانتقال أكثر منه تعلقا بالقوى. منحنى الاستطاعة المتحصل عليه بتحليل " الدفع حتى الانهيار" يأكد هذا ويري بوضوح بأن ظهور التدهورات وكذا الانهيار التام يحدث عنما يبلغ الهيكل حد معين من الانتقال.

<u>الكلمات المفتاحية:</u> هياكل الخرسانة المسلحة، منحنى الاستطاعة، الطريقة السكونية المكافئة، العارضة المكافئة، طريقة تحليل " الدفع حتى الانهيار"، نقطة الأداء، طيف الطلب الزلزالي.

SUMMARY

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GENERAL INTRODUCTION

Over the centuries, some towns and cities have repeatedly been struck and sometimes devastated by major earthquakes, the difference between damage and devastation depends not only on the magnitude of the earthquake, but also on local geology, soil characteristics and on building techniques.

Poor construction techniques, where structural elements are not tied together correctly, for instance, makes buildings far more vulnerable to earthquake damage. Next, there is the choice of building design, buildings shake when the frequency of the seismic waves is close to the natural frequency of vibration of the building, an effect known as resonance. However, there is more to the issue of building design than simply the difference between careful and shoddy construction. For example, buildings with masonry infill where filler walls have been held in place with the correct mortar tend to survive much better. Several researchers insist on the positive role of the filler walls in the resistance of reinforced concrete frame buildings under the action of seismic forces. During the calculation of frame structures, the interaction between frames and filler walls is not taken into consideration precisely. It is considered that the filler walls have no effect on the stiffness and strength of the structures, while in reality it is quite the contrary.

The assigned object to this study is to analyze and evaluate the seismic performance of a multiple-storey RC frame structure in order to see the effect of masonry walls on the global behavior of the studied structure.

- In chapter I: Our work consists in the first place to introduce a literature review on the analysis methods, their principles and purpose of usage, conditions of application and finally their limitations.

- In chapter II: secondly, we will introduce a literature review on the Pushover analysis method.

- In Chapter III: In order to give evidence to the fact that infill walls have an observable effect on the structural behavior, we will apply an example of a regular RC frame structure braced with masonry infill walls with a soft storey situated in different levels.

Linear and nonlinear analyses will be made, where results will be shown in tables and curves. A comparison will be made and comments and conclusions will be concluded.

Finally, we finish our modest work with a General conclusion that sums up all what was made in this thesis.

CHAPTER I

Methods of structure analysis

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Methods of structure analysis

I.1. INTRODUCTION:

The uncertainties involved in accurate determination of material properties, element and structure capacities, the limited prediction of ground motions that the structure is going to experience and the limitations in accurate modeling of structural behavior make the seismic performance evaluation of structures a complex and difficult process [34]. However, most of the current seismic design codes belong to the category of prescriptive design procedures (or limit-state design procedures), where if a number of checks are satisfied, then the structure is considered safe since it fulfills the safety criterion against collapse. Existing seismic design procedures are based on the principle that a structure will avoid collapse if it is designed to absorb and dissipate the kinetic energy that is induced during a seismic excitation [44]. Most modern seismic norms express the ability of the structure to absorb energy through inelastic deformation using a reduction or behavior factor [2]. In the same context, this chapter exposes the general analysis methods of structures in a detailed way.

I.2. ANALYSIS OF REINFORCED CONCRETE FRAME STRUCTURES

Over the past few decades, intensive research activity in structural engineering has greatly increased our knowledge of the behavior of concrete structures under both shear and flexure. As a result, new analysis and design procedures have been developed and incorporated into design codes [25].

Occurring at the same time, advancements in computing technology have enabled structural engineers to analyze and design structures according to the new design codes quickly and easily. The analysis procedures are typically based on linear-elastic principles. Even though linear-elastic analyses cannot accurately predict all aspects of structural behavior, such as cracking of concrete and deformations under service loads, they are deemed sufficient if the structure is designed according to code. As a result, the structure will satisfy strength and serviceability requirements. The reinforcement is detailed so that the structure exhibits ductile behavior with a flexural failure mode. Currently, there are numerous easy-to-use software programs, which can perform such analyses and designs reasonably well [25].

I.3. METHODS OF REINFORCED CONCRE FRAME STRUCTURES ANALYSIS

Methods of structural analysis for RC framed structures (columns-beams) range according to their complexity, precision and objectivity [41]. These different methods of analysis simplify the structural model by constructing curves using functions of different types of seismic intensity providing sight information about the likely behavior of the structure [24, 19].

The seismic vulnerability assessment of buildings can be carried out using essentially two possible analysis methods: Linear-elastic methods and/or Non-linear (inelastic) methods.

I.3.1. *LINEAR-ELASTIC METHODS*

In the Algerian seismic code [43], the study of the structural response during an earthquake is conducted using linear methods (equivalent static method and/or response spectral analysis method) which are based on the principle of determining the forces that can be applied, then proceed to a verification of the structure movements [40].

I.3.1.1. Response spectrum analysis method

I.3.1.1.1. *Principle*

By this method, the maximum effects generated in the structure by the seismic forces represented by a calculation response spectrum is sought for each mode of vibration. These effects are subsequently combined to obtain the response of the structure [43].

I.3.1.1.2. *Conditions of application*

The response spectrum analysis method is used in all cases as well as in instances where the equivalent static method is not permitted to be used. Its modeling conditions are as follows:

- For regular plan structures with rigid floors, the analysis is done separately in each of the two principal directions of the building. The latter is then represented in each of the two calculation directions by a plane model, fixed at the base and where the masses are concentrated at the level of the centers of gravity of the floors with a single degree of freedom in horizontal translation [35].
- For irregular plan structures subject to torsion and with rigid floors, they are represented by a three-dimensional model, recessed at the base and where the masses

are concentrated at the center of gravity of the floors with three (03) DOF (2 horizontal translations and a vertical axis rotation) [36].

- For regular or non-rigid structures with flexible floors, they are represented by three-dimensional models fixed at the base and MDOF per level.
- The building model to be used should best represent the distributions of stiffness and mass taking into account all significant deformation modes in the calculation of seismic forces of inertia [36].
- In the case of reinforced concrete or masonry buildings, the stiffness of the load-bearing elements must be calculated considering the uncracked sections. If the displacements are critical especially in the case of structures associated with high values of behavior factor, a more precise estimate of the rigidity becomes necessary by taking into account of cracked sections [36].

I.3.1.1.3. *Procedure and formulation*

Designing codes provide a computational response spectrum that evaluates seismic forces for each mode of vibration. These forces are combined to obtain the response of the structure. The mathematical formulas which govern this spectrum of computation are in function of several parameters: the coefficient of acceleration of zone (A), the factor of quality (Q), the coefficient of behavior (R) , the percentage of critical damping (ξ), and the characteristic periods associated with the category of the site (T1, T2) where the building is located. The use of the response spectrum is possible only after a modal analysis where the results represent the values and the eigenvectors of the structure [43].

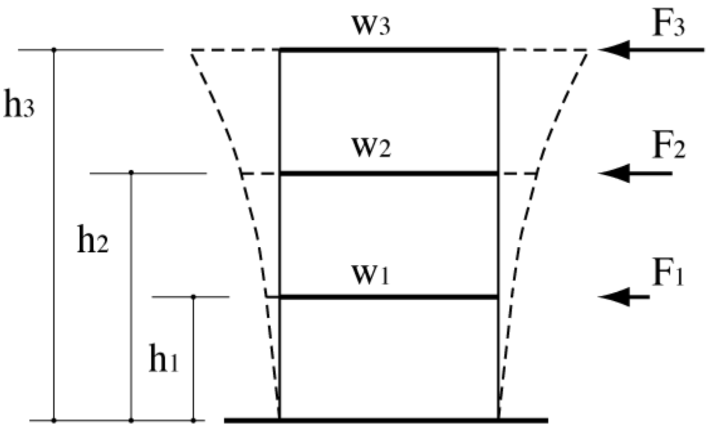
The seismic action is represented by the following calculation spectrum:

$$\frac{S_a}{g} = \begin{cases} 1.25A\left(1+\frac{T}{T_1}\left(2.5\eta\frac{Q}{R}-1\right)\right) & 0 \le T \le T_1 \\ 2.5\eta(1.25A)\left(\frac{Q}{R}\right) & T_1 \le T \le T_2 \\ 2.5\eta(1.25A)\left(\frac{Q}{R}\right)\left(\frac{T_2}{T}\right)^{2/3} & T_2 \le T \le 3.0s \\ 2.5\eta(1.25A)\left(\frac{T_2}{3}\right)^{2/3}\left(\frac{3}{T}\right)^{5/3}\left(\frac{Q}{R}\right) & T > 3.0s \end{cases} \tag{I.1}$$

I.3.1.2. Equivalent static analysis method

I.3.1.2.1. *Principle*

The static method is the oldest, the most used and the simplest one because it requires less computational efforts and it is based on a formulation given in the code of practice. The seismic forces are replaced by a system of equivalent horizontal static forces whose effects are considered equivalent to those of the seismic action. The static forces are applied successively in the two main directions of the horizontal plane of the structure. In the general case, the designer chooses these two directions [28].



$$F_x = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} V$$

Figure I.1: Calculation of lateral forces of each storey.

I.3.1.2.2. *Conditions of application*

I.3.1.2.2.1. Regularity in shape conditions

- Geometric regularity in Plan: The shape of the building shall be compact with a length / width ratio of the floor less than or equal to four. The sum of the dimensions of the re-entrant or projecting parts of the building in one direction shall not exceed 25% of the total dimension of the building in that direction.
- Structural regularity in Plan: At each level and for each direction of calculation, the distance between the center of gravity of the masses and the center of the rigidities shall not exceed 15% of the dimension of the building measured perpendicular to the direction of the seismic action considered.

- Regularity in plan: The floors must have sufficient rigidity with respect to vertical bracing to be considered as non-deformable in their plan. In this context, the total area of openings floor must be less than 15% of that of the latter.
- Geometric regularity in elevation: The bracing system must not contain any discontinuous vertical element in which the loads are not transmitted directly to the foundation.

1.3.1.2.2.2. Modeling conditions

- The model of the building to be used in each of the two directions of calculation is plane with the masses concentrated at the gravity center of the floors, and one degree of freedom in horizontal translation per level provided that the bracing systems in both directions can be decoupled.
- The lateral stiffness of the load-bearing members of the bracing system is calculated from uncracked sections for reinforced concrete or masonry structures.
- Only the fundamental mode of vibration of the structure is taken in consideration during the calculation of the total seismic force.

I.3.1.2.3. ***Procedure and formulation***

The total seismic force applied in the base of the structure is expressed by a mathematical formula; this formula is in function of the parameters A, Q and the behavior factor R, as well as of the total weight of the structure (W) and the average dynamic amplification factor (D). The parameter D is a function of category of the site, a depreciation correction factor (η) and the fundamental period T. The weight W is equal to the sum of the weights of all building levels, including both permanent loads and a fraction (β) of operating expenses [43]. The empirical formulas used for the estimation of the total seismic force, the distribution of the total seismic force according to the height of the building and the fundamental period are respectively given by the following equations:

$$V = \frac{ADQ}{R} W \tag{I.2}$$

$$F_i = \frac{(V - F_t) W_i h_i}{\sum_{i=1}^{n} W_i h_i} \tag{I.3}$$

$$T = C_T H^{3/4} \tag{I.4}$$

Where **hi** is the height of level **i** measured from the base, **Wi:** weight of level **i**,

$Ft = 0.07T \leq 0.25V$: Force concentrated at the top of the structure ($Ft = 0.0$ if $T \leq 0.7$ sec)

I.3.2. *NON-LINEAR (INELASTIC) METHODS*

The nonlinear behavior of the structure is taken into account by reducing the computation efforts by a behavior factor (R) that takes into account the dissipative behavior of the structure. Therefore, we can say that a very vital step towards a good performance estimation of the structure against earthquakes is the determination of correct load-deformation curve popularly known as “capacity curve” [21].

We consider two well-known general methods that simplifies the nonlinear structural model: Nonlinear Dynamic Analysis (Time History), Pushover Analysis Method.

I.3.2.1. Nonlinear dynamic analysis method (Time History)

I.3.2.1.1. *Definition*

Nonlinear dynamic analysis is also known as Time history analysis. It is an important method for structural seismic analysis especially when the evaluated structural response is nonlinear.

I.3.2.1.2. *Principle*

To perform this analysis, a representative earthquake time history data (figure I.2) is required for a structure being evaluated. Time history analysis is a step-by step analysis procedure of the dynamic response of a structure for a specified loading that may vary with time. Time history analysis is used to determine the seismic response of a structure under dynamic loading for a representative earthquake. Although the response spectrum analysis method, outlined in the previous section, is useful technique for the elastic analysis of structures, it is not directly transferable to inelastic analysis because the principle of superposition is no longer applicable. In addition, the analysis is subject to uncertainties inherent in the modal superimposition method. The actual process of combining the different modal contributions is a probabilistic technique and, in certain cases, it may lead to results not entirely representative of the actual behavior of the structure. The THA technique represents the most accurate method for the dynamic analysis for buildings. In this method, the mathematical model of the building is subjected to accelerations from earthquake records that represent the expected earthquake that may occur at the base of the structure. The method consists of a step- by- step direct integration over a time interval; the equations of motion are solved with the accelerations, velocities and displacements of the previous step serving as initial functions. The equation of motion can be represented as:

$$K\,x(t) + C\,\dot{x}\,(t) + m\,\ddot{x}\,(t) = P(t)$$

Where: *k* is the stiffness matrix, *c* is the damping matrix, and *m* is the diagonal mass matrix. In case of an earthquake, *P(t)* includes ground acceleration and the displacements, velocities and accelerations are determined relative to ground motion.

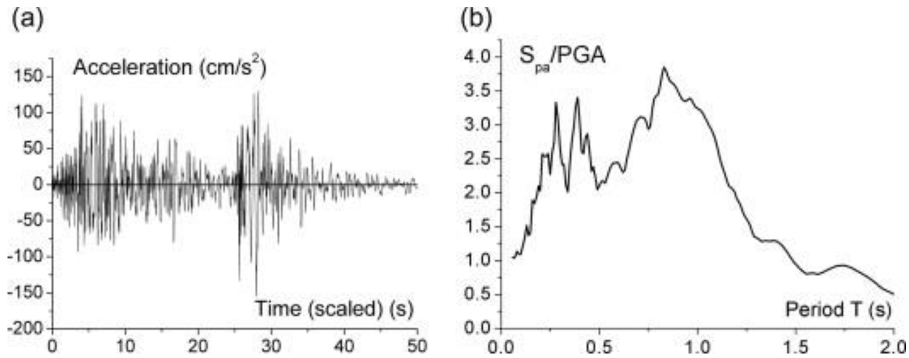


Figure I.2: Applied dynamic loading: (a) accelerogram and (b) elastic response specter.

I.3.2.1.3. ***Conditions of application***

The time-history method can be applied to both elastic and inelastic analysis. In elastic analysis the stiffness characteristics of the structure are assumed to be constant for the whole duration of the earthquake. In the inelastic analysis, the stiffness is assumed to be constant through the incremental time only. Modifications to structural stiffness caused by cracking, forming of plastic hinges, are incorporated between the incremental solutions. Even with the availability of sophisticated computers, the use of this method is restricted to the design of special structures such as nuclear facilities, military installations, and base-isolated structures [45].

I.3.2.1.4. ***Capabilities and limitations of time history analysis***

The potential and limitations of Time History Analysis includes the following:

- The same structure and load types are available as in the case of linear statics.
- The function of load variability may be defined for an arbitrary static load case with the exception of the moving load case. In order to model a dynamic impact of a

moving load, successive vehicle positions should be defined in separate load cases and use the time functions with the phase shift corresponding to the vehicle movement.

- Additional modeling options available in the linear static analysis can be used (such as releases, elastic connections, rigid links, and others).
- Case components may be used in combinations after generating an additional load case containing the analysis results for a given component.
- It allows adopting initial displacements from a selected load case, assuming simultaneously zero values of initial velocities and accelerations.
- It is solved only by using the modal decomposition method, which requires that modal analysis be carried out first.
- Only one time history function may be used to determine time variability of loads of a given load case. It is possible, however, to add time functions.
- There are a considerable number of facilitating options in the time history analysis.
- A graphical interface for introducing data, accompanied by the visualization of time function course.
- The possibility of reading a time function from and saving it to an easily editable text file.
- Scaling and phase shift of time functions.
- Calculation notes with all the pertaining data.
- The possibility of using the results from time history case components in combinations
- Perfected graphical presentation of the resultant values in diagrams. View diagram comparisons of several arbitrarily selected quantities in one viewer, with time function course displayed.
- Diagrams of a new resultant value - foundations shearing forces.

In order to obtain satisfactory results for time history analysis cases, it is required to carry out iterative analysis with multiple calculations for different case parameters. Modal analysis needs to be carried out again. In the case of a large-scale structure, the modal analysis itself may be time-consuming, as will the time history analysis. Therefore, it is necessary to select cases for calculations or at least to mark the modal analysis as calculated. This may also be useful in the case of seismic analysis [7].

I.3.2.2. Pushover analysis method

I.3.2.2.1. *Principle*

Pushover analysis is performed by subjecting a structure to a monotonically increasing pattern of lateral loads, representing the inertial forces, which would be experienced by the structure when subjected to ground shaking. Under incrementally increasing loads, various structural elements may yield sequentially. Consequently, at each event, the structure experiences a loss in stiffness. Using a pushover analysis, a characteristic nonlinear force displacement relationship can be determined [32].

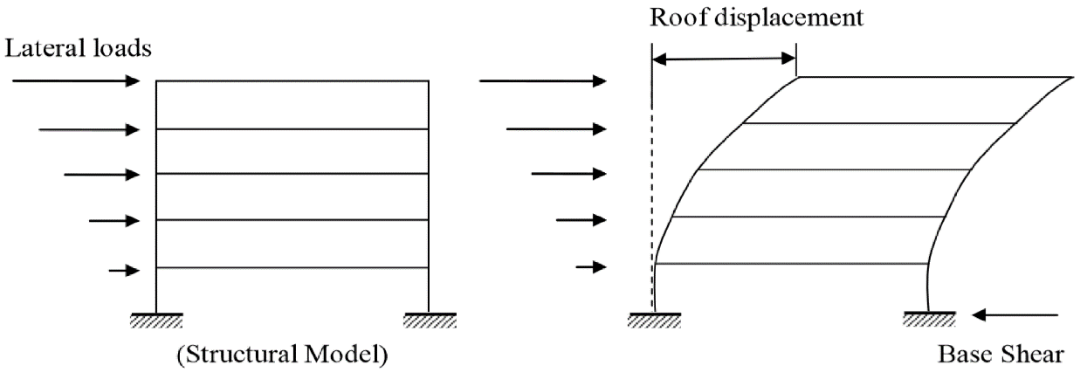


Figure I.3: Static Approximations in the Pushover Analysis.

I.3.2.2.2. *Conditions* of application

It is necessary to take into account the following considerations:

- Pushover analysis is a nonlinear static analysis used mainly for seismic evaluation of framed buildings.
- Seismic demands are computed by nonlinear static analysis of the structure which is subjected to monotonically increasing lateral forces with an invariant height-wise distribution until a target displacement is reached.
- It is also necessary for evaluating the seismic adequacy of existing buildings.

I.3.2.2.3. *Purpose*

The purpose of pushover analysis is to evaluate and estimate the expected performance of structural systems by estimating their strength and deformation demands in design earthquakes by means of static inelastic analysis and comparing these demands with available capacities at the performance levels of interest [32]. The evaluation is based on the assessment of important performance parameters, including global drift, inter-story drift, inelastic element deformations (either absolute or normalized with respect to a yield value), deformations between elements, and element connection forces (for elements and connections that cannot sustain inelastic deformations). The inelastic static Pushover analysis can be viewed as a method for predicting seismic force and deformation demands, which accounts in an approximate manner for the redistribution of internal forces that no longer can be resisted within the elastic range of structural behavior [26, 32].

This procedure is mainly used to estimate the strength and the seismic demand for the structure subjected to selected earthquake. This procedure can be used for checking the adequacy of new structural design as well. The effectiveness of pushover analysis and its computational simplicity brought this procedure in to several seismic guidelines (ATC 40 and FEMA 356) and various design codes in last few years.

Nonlinear static pushover analysis can provide an insight into the structural aspects, which control performance during severe earthquakes. The analysis provides data on the strength and ductility of the structure, which cannot be obtained by elastic analysis [27]. By pushover analysis, the base shear versus top displacement curve of the structure is obtained based on the capacity curve, a target displacement which is an estimate of the displacement that the design earthquake will produce on the building is determined. The extent of damage experienced by the structure at this target displacement is considered representative of the damage experienced by the building when subjected to design level ground shaking [32].

I.4. CONCLUSION

In this chapter, the following conclusions were made:

- Analysis methods give an idea about the overall behavior of the studied structures.
- The choice of the adopted method is based mainly on importance of the building that needs be analyzed and the accuracy of the needed results. Thus, the criteria are:
    - ✓ In smaller structures with little effort, response spectrum analysis or equivalent static analysis can be used to investigate the effects of seismic loading.
    - ✓ If very accurate and precise results are required, non-linear dynamic analysis should be carried out.

- A wrong model, simplified in the wrong places, can cause very different results compared to the real building. This is especially important in seismic loading, because when a section is designed to yield, and it turns out to be stronger than designed, it may cause the wrong part to yield, putting the whole structure into failure.
- Pushover method represents the first mode of the Time History analysis, in which it gives good results similar to those gotten from the Time History method and even simpler than this latter, that is based on Fine elements matrices.

In the next chapter, we will go further with explaining the pushover analysis method.

Chapter II

An overview on Pushover Analysis Method

CHAPTER II

An overview on Pushover Analysis Method

II.1. INTRODUCTION

The promise of performance-based seismic engineering (PBSE) is to produce structures with predictable seismic performance. The recent advent of performance-based design has brought to the forefront the non-linear static Pushover analysis procedure, which has been developed over the past thirty years and has become the preferred analysis procedure for design and seismic performance evaluation purposes, as this least is relatively simple and considers post-elastic behavior [22]. However, this method helps on giving sight information about the expected health status of the structure and the level of structural damage. Furthermore, this procedure has been shown to capture the essential structural response characteristics under a seismic action using the performance point obtained from the capacity curve mentioned in the previous chapter.

II.2. PUSHOVER ANALYSIS DEFINITION

Nonlinear static analysis also known as Pushover Analysis procedure is mainly used to estimate the strength and drift capacity of existing structure and the seismic demand for this structure subjected to selected earthquake. This analysis procedure can be used for checking the adequacy of new structural design as well. The effectiveness of pushover analysis and its computational simplicity brought this analysis procedure in to several seismic guidelines (ATC 40 and FEMA 356) and design codes (Eurocode 8 and PCM 3274) [5, 10, 23].

II.3. PRINCIPLE AND HYPOTHESIS OF THE PUSHOVER ANALYSIS

It is generally assumed that the behavior of the structure is controlled by its fundamental mode and the predefined pattern is expressed either in terms of story shear or in terms of fundamental mode shape. With the increase in magnitude of lateral loading, the progressive non-linear behavior of various structural elements is captured, and weak links and failure modes of the structure are identified. In addition, pushover analysis is also used to ascertain the capability of a structure to withstand a certain level of input motion defined in terms of a response spectrum.

In Pushover analysis, a static horizontal force profile, usually proportional to the design force profiles specified in the codes, is applied to the structure. The force profile is then increased by small incremental steps and the structure is analyzed at each step. As the loads are increased, the building undergoes yielding at a few locations. Every time such yielding takes place, the structural properties are modified approximately to reflect the yielding. The analysis is continued until the structure collapses, or the building reaches a certain level of lateral displacement.

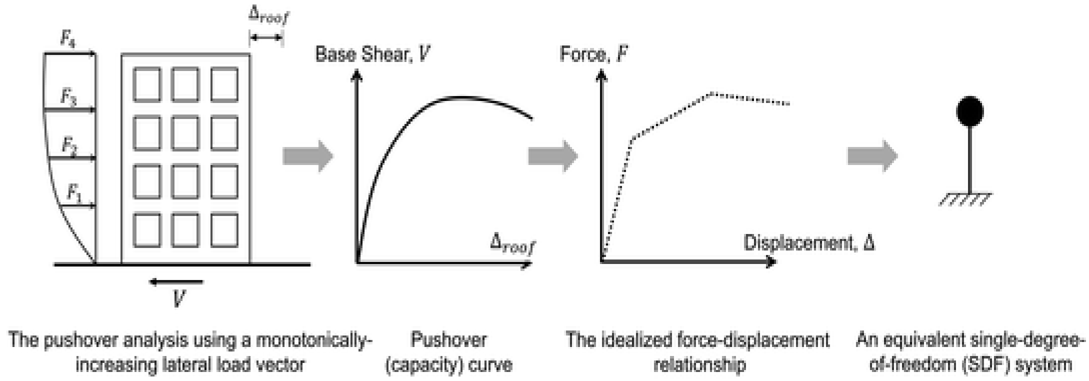


Figure II.1: The basic conversion of a detailed structural model in to an equivalent SDF system

II.4. NON-LINEAR STATIC PUSHOVER ANALYSIS ORIGIN

The term “pushover analysis” describes a modern variation of the classical collapse analysis method, as fittingly described. It refers to an analysis procedure whereby an incremental-iterative solution of the static equilibrium equations is carried out to obtain the response of a structure subjected to monotonically increasing lateral load pattern.

The Pushover method is based on the misconception that the response of a system that has an equivalent one degree of liberty implies that the response is fundamentally controlled by one mode of vibration, and the form of this mode remains constant during the seism.

Researchers demonstrated that the misconception gives good results concerning the seismic response (maximum displacement) given by the first mode of vibration of the simulated system of an equivalent linear system.

II.5. PUSHOVER ANALYSIS PROBLEMS AND LIMITATIONS

The Pushover analysis involves certain approximations and simplifications showing that some amount of variation is always expected to exist in seismic demand prediction of pushover analysis [39].

The accuracy and the reliability of the traditional pushover analysis in predicting global and local seismic demands for all structures has been a subject of discussion with some proposals to improve its methods in order to overcome the certain limitations of traditional pushover procedures. However, the improved procedures have proven to be so computationally demanding and conceptually complex that the use of such procedures are impractical in engineering profession and codes. As traditional pushover analysis is widely used for design and seismic performance evaluation purposes, its limitations, weaknesses and the accuracy of its predictions in routine application should be identified by studying the factors affecting the pushover predictions. In other words, the applicability of pushover analysis in predicting seismic demands should be investigated for low, mid and high-rise structures by identifying certain issues such as modeling nonlinear member behavior, computational scheme of the procedure, variations in the predictions of various lateral load patterns utilized in traditional pushover analysis, efficiency of invariant lateral load patterns in representing higher mode effects and accurate estimation of target displacement at which seismic demand prediction of pushover procedures are performed [39, 30, 27].

II.6. RECENT MODIFICATIONS TO THE PUSHOVER ANALYSIS

Recently, modifications to Pushover procedures have also been proposed in order to capture the contribution of structures with higher modes of vibration, changes in distribution of story shear subsequent to yielding of structural members, etc. Pushover procedure has gained popularity during the last few years, as appropriate analytical tools are now available (SAP-2000, ETABS) [17, 16].

II.7. PURPOSE OF NON-LINEAR STATIC PUSHOVER ANALYSIS

Pushover analysis is expected to provide information on many response characteristics that cannot be obtained from an elastic static or dynamic analysis [3]. The following are examples of such response characteristics:

• The realistic force demands on potentially brittle elements such as axial force demands on columns, force demands on brace connections [27], moment demands on beam to column connections, shear force demands in deep reinforced concrete spandrel beams, shear force demands in unreinforced masonry wall piers, etc.

• Estimates of the deformations demands for elements that must form inelastically in order to dissipate the energy imparted to the structure [27].

• The consequences of strength deterioration of individual elements in structural systems.

• Identification of the critical regions in which the deformation demands are expected to be high and that have become the focus through detailing [32].

• Identification of the strength discontinuities in plan elevation that will lead to changes in the dynamic characteristics in elastic range.

• Estimates of the inter-storey drifts that account for strength or stiffness discontinuities that can be used to control the damages and to evaluate P-Delta effects.

• Verification of the completeness and adequacy of load path, considering all the elements of the structural system, all the connections, the stiff nonstructural elements of significant strength and the foundation system [27, 3, 32].

II.8. LITERATURE REVIEW

The use of the nonlinear static analysis (pushover analysis) came in to practice in 1970's but the potential of the pushover analysis has been recognized in the last 10 or 15 years only.

These are some of the recent and most important studies concerning this method:

- In the 2003, M. Mouzzoun et Al [46] assessed seismic performance of a five-storey reinforced concrete building designed according to the Moroccan seismic code RPS2000. In the first time a set of dynamic analysis were carried out to compute the dynamic properties of the building (fundamental period, natural frequencies, deformation modes), in the second time a pushover analysis was performed to assess the seismic performance of the building and detect the locations of the plastic hinges. Pushover analysis was performed using SAP2000. The results obtained from the study showed that the building performs well under a moderate earthquake, but it was vulnerable under a severe earthquake.

- In 2008, A. Kadid et Al [48] made a study on three framed buildings of low-rise, medium rise and high-rise buildings with 5, 8 and 12 stories respectively; they analyzed these buildings using nonlinear static analysis software SAP 2000. They concluded that the failure of reinforced concrete during the Boumerdes earthquake could be attributed to the quality of the building materials and to the fact that most of buildings constructed in Algeria are of strong beam and weak column type.
- In 2013, Ms. Nivedita N. Raut et Al [47] presented a study on multi-story reinforced concrete framed building structures in urban India where buildings are constructed with masonry fills. Using nonlinear analysis results, they compared base shear and displacement between three different frames (bare frame, filled wall frame). After, the analysis concluded that at roof level, the displacement in bare frame was more than the other two frames and the displacement at ground floor in the weak story was more than the other two frames as well, because the plastic hinges in beam was stronger than in the column.
- In the FEMA (356) journal [23], detailed procedure and information about the Standard nonlinear static pushover analysis were described.

II.9. TYPES OF PUSHOVER ANALYSIS

Pushover analysis is defined in two main types: force controlled or displacement controlled.

II.9.1. Force controlled type

In this type, the total lateral force is applied to the structure in small increments.

II.9.2. Displacement controlled type

The displacement of the top story of the structure is incremented gradually, so that the required horizontal force pushes the structure laterally. The distance in which the structure is pushed is proportional to the fundamental horizontal translational mode of the structure [3].

In both types of pushover analysis, once the structure passes from the elastic state to the inelastic state the stiffness matrix of the structure may have to be changed for each increment of the load or displacement [3]. “The displacement controlled” Pushover analysis is generally preferred over “the force controlled” because the analysis can be carried out up to the desired level of the displacement.

II.10. LATERAL LOAD PATTERNS

A specific load distribution pattern is applied along the height of the building in pushover analysis procedure. The magnitude of the total force is increased but the pattern of the loading remains same until the end of the process. Pushover analysis results are very sensitive to the load pattern [38].

II.11. TARGET DISPLACEMENT

Target displacement can be defined as the displacement demand for the building at the control node subjected to the ground motion, which is considered for the analysis. It is an important parameter in pushover analysis procedure because the global and component responses (forces and displacement) of the building at the target displacement are compared with the desired performance limit state to know the building performance [27]. Therefore, the success of a pushover analysis largely depends on the accuracy of target displacement obtained [38].

II.12. PUSHOVER ANALYSIS FORMULATIONS

By the end of the analysis, the structure reaches a certain level of lateral displacement. It provides a load versus deflection curve of the structure starting from the state of rest to the ultimate failure of the structure. The load is representative of the equivalent static load of the fundamental mode of the structure. It is generally taken as the total base shear of the structure and the deflection is selected as the top-storey deflection. The selection of appropriate lateral load distribution is an important step.

The first step then is to select a displacement shape and the vector of lateral loads is determined as:

$$\{F\}=p[m]\{\Phi\} \tag{II.1}$$

Where $\{\Phi\}$ is the assumed displacement shape, and p is the magnitude of the lateral loads. From the equation, it follows that the lateral force at any level is proportional to the assumed displacement shape and story mass. If the assumed displacement shape was exact and remained constant during ground shaking, then the distribution of lateral forces would be equal to distribution of effective earthquake forces.

For pushover analysis of any structure, the input required is the assumed collapse mechanism, moment–rotation relationship for the sections that are assumed to yield, the fundamental mode shape, the limiting displacement, and the rotational capacity of the plastic hinges. In addition to data needed for usual elastic analysis, the non-linear force deformation relationship for structural elements under monotonic loading is also required. The most commonly used element is beam element modeled as line element. Seismic demand is traditionally defined in the form of an elastic acceleration spectrum Sae, in which spectral accelerations are given as a function of the natural period of structure T.

The structure is modeled as a SDOF system. The displacement shape is assumed to be constant. This is the basic and most critical assumption. The starting point is the equation of motion of planar MDOF model that explicitly includes only lateral translation degrees of freedom.

$$[m]\{u\} + \{R\} = [m]\{1\}x_g \qquad \text{(II.2)}$$

Where {u} and {R} are the vectors representing displacements and internal forces, {1} is a unit vector, and $\ddot{x}_g$ is the ground acceleration as a function of time. The displacement vector, {u} is defined as:

$$\{u\} = \{\Phi\}D_t \qquad \text{(II.3)}$$

Where $D_t$is the time dependent top displacement. For equilibrium, the internal forces, {R} are equal to statically applied external loads {F}. The equation of motion of equivalent SDOF is written as:

$$m^* D^* + F^* = -m^*\ddot{x}_g \qquad \text{(II.4)}$$

Where m* m is equivalent mass of the SDOF system, D* and F* are the displacement and force of the equivalent SDOF system, respectively. For simplification the force-displacement relationship is assumed to be elastic perfectly plastic for equivalent SDOF as shown in (Figure II.2).

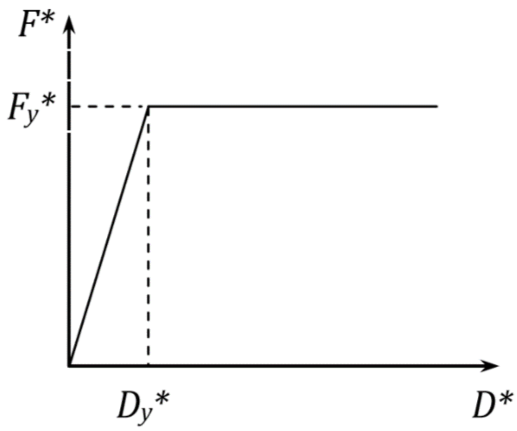


Figure II.2: Approximate elasto-plastic force-displacement relationships.

Determine the strength F_y*, yield displacement, D_y* and period T*. The T is given by

$$T^* = 2\,\pi\sqrt{\frac{m * D_{y^*}}{F_{y^*}}} \quad \text{(II.5)}$$

From the acceleration spectrum, the inelastic spectrum in acceleration-displacement format is determined. The capacity diagram in acceleration displacement (AD) format is obtained by dividing the forces in the force deformation diagram by m*.

$$S_a = \frac{F*}{m*} \quad \text{(II.6)}$$

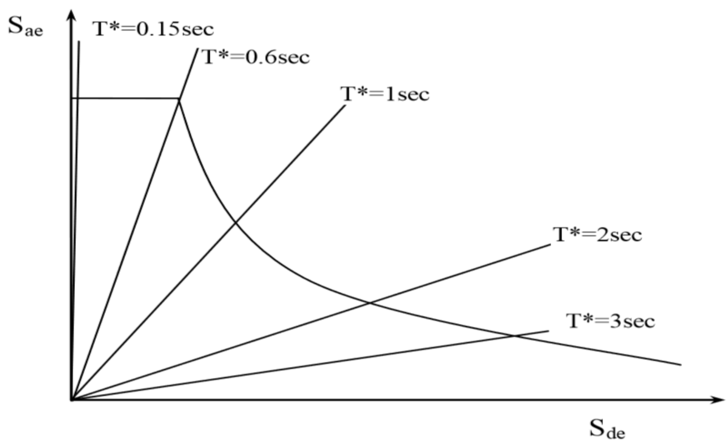


Figure II.3: Demand in the AD format.

The displacement demand for the SDOF model $S_d$is transformed into the maximum top displacement D_t of the MDOF system. The local seismic response (e.g. story drifts, joint rotations) can be determined by pushover analysis. Under increasing lateral loads with a fixed pattern, the structure is pushed to a target displacementD_t. Consequently, it is appropriate that the likely performance of the building under a push load is up to the target displacement. The expected performance can be assessed by comparing seismic demands with the capacities for the relevant performance level. Global performance can be visualized by comparing displacement capacity and demand.

II.12.1. Capacity curve and the performance point

The seismic performance of a building can be evaluated in terms of pushover curve, performance point, displacement ductility, plastic hinge formation etc. The base shear vs. roof displacement curve (Figure II.4) is obtained from the pushover analysis from which the maximum base shear capacity of structure can be obtained. This capacity curve is transformed into capacity spectrum by SAP as per ATC40 and the demand or response spectrum is also determined for the structure for the required building performance level. The intersection of demand and capacity spectrum gives the performance point of the analyzed structure. This is illustrated in the (Figure II.5).

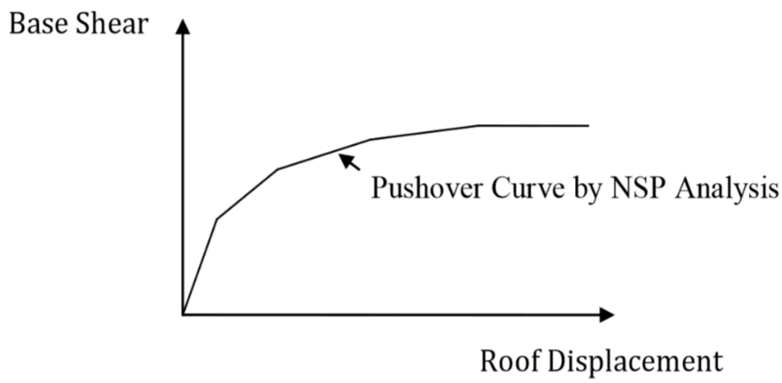


Figure II.4: Base shear vs. roof displacement.

At the performance point, the resulting responses of the building should then be checked using certain acceptability criteria. Consequently, the Performance Point obtained from pushover analysis is then compared with the calculated target displacement.

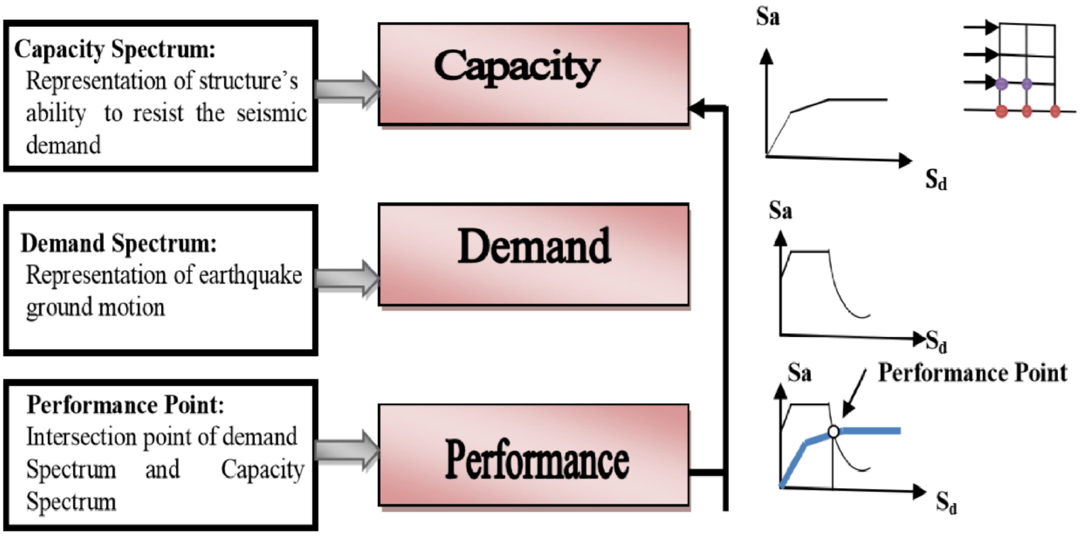


Figure II.5: Determination of performance point.

II.12.2. Procedures to find the performance point

There are three procedures described in ATC-40 to find the performance point.

-Procedure A: which uses a set of equations described in ATC-40. -Procedure B: is also an iterative method to find the performance point, which assumes that the yield point and the post yield slope of the bilinear representation remains constant. This is adequate for most cases; however, in some cases this assumption may not be valid.

-Procedure C: is graphical method that is convenient for hand as well as software analysis. SAP2000 uses this method for the determination of performance point. To find the performance point using Procedure C the following steps are used: First, the single demand spectrum (variable damping) curve is constructed by doing the following for each point on the Pushover Curve:

1) Draw a radial line through a point (P) on the Pushover curve. This is a line of constant period.

2) Calculate the damping associated with the point (P) on the curve, based on the area under the curve up to that point.

3) Construct the demand spectrum, plotting it for the same damping level as associated with the point 'P' on the pushover curve.

4) The intersection point (P') for the radial line and associated demand spectrum represents a point on the Single Demand Spectrum (Variable Damping Curve).

A number of arbitrary points are taken on the Pushover curve. A curve is then drawn by joining through these points. The intersection of this curve with the original pushover curve gives the performance point of the structure as shown in (Figure II.6) as follows:

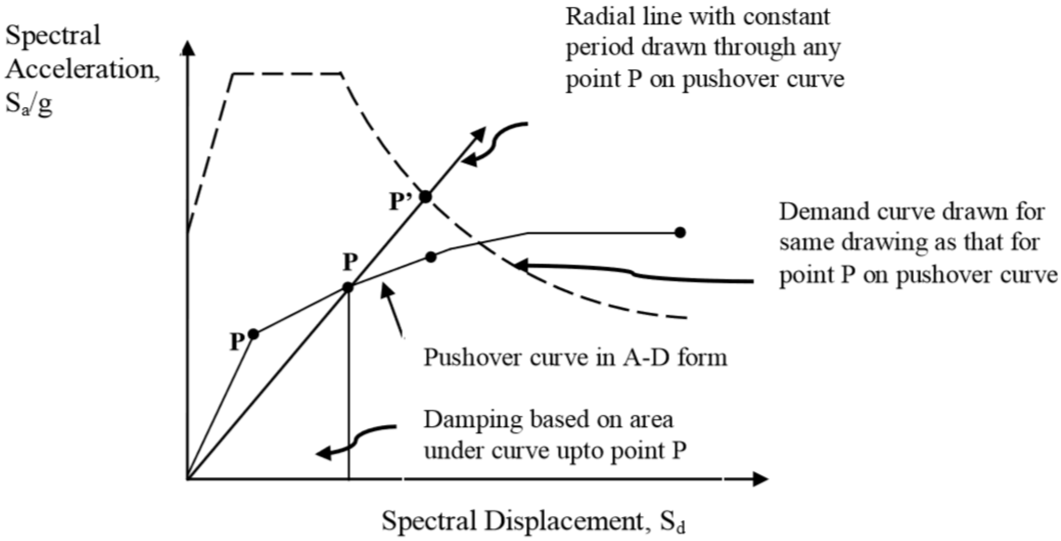


Figure II.6: Capacity Spectrum Procedure 'C' to Determine Performance Point [5].

II.12.3. Inter-story drift:

It has been recognized that the inter-story drift performance of a multistory building is an important measure of structural and non-structural damage of the building under various levels of earthquake motion. In performance-based design, inter-story drift performance has become a principal design consideration. The system performance levels of a multistory building are evaluated based on the inter-story drift values along the height of the building under different levels of earthquake motion. Inter-storey drift is defined as the ratio of relative horizontal displacement of two adjacent floors (δ) and corresponding storey height (h).

$$\text{Inter-story Drift} = \frac{\delta}{h} = \frac{\delta i - \delta_{i-1}}{h} \qquad \text{(II.7)}$$

II.13. PUSHOVER ANALYSIS METHODS

The most updated code procedures that use the pushover-based analysis are briefly summarized for clarity: i.e. the Eurocode 8 (prENV 1998-1, 1994), FEMA 356 (ASCE, 2000), ATC40 (ATC, 1996).

II.13.1. The Eurocode 8 (prENV 1998-1, 1994)

The NSP adopted in this seismic design code is the N2-procedure developed by Fajfar (1999) [22], which consists in the definition of a bilinear SDOFS (Figure) corresponding to the first vibration mode assuming that the contribution of the other modes is negligible [10]. The design displacement (or maximum displacement response) of an earthquake is defined as the displacement response spectrum at the elastic period of the SDOFS (T*) accounting for the system ductility, by means of an amplification factor, whereas the equivalent displacement approach between the linear system response and the response of the nonlinear one cannot be applied (i.e. short period range) [10].

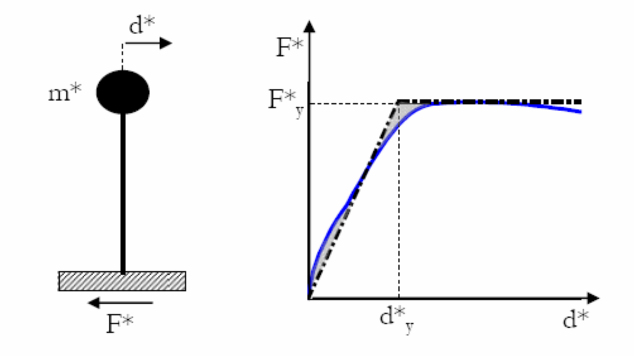


Figure II.7: Equivalent Single Degree Of Freedom System [22].

II.13.2. The Coefficient Method in FEMA 356 (ASCE, 2000)

The NSP adopted in FEMA 356 (the coefficient method) consists in the definition of an equivalent linear SDOFS considering an effective period Te generated from the initial period Ti, accounting for some loss of stiffness in the transition from the elastic to inelastic behavior. This procedure estimates the total maximum displacement of the SDOF oscillator by multiplying the elastic SDOFS response (assuming the initial linear properties, stiffness and damping) by one or more coefficients empirically derived (Figure II.8). These coefficients accounts for (i) the SDOF idealization (a shape factor scales the SDOFS response to the roof displacement of the building), (ii) the linear response assumed (conventionally characterized in terms of strength, ductility and period (R-D-T relationships)), (iii) stiffness and strength degradation, and (iv) the dynamic amplification of the response. It might be noted that the design displacement is defined by means of an iterative procedure until convergence of the linear SDOFS displacement amplitude to the response spectrum ordinate [23].

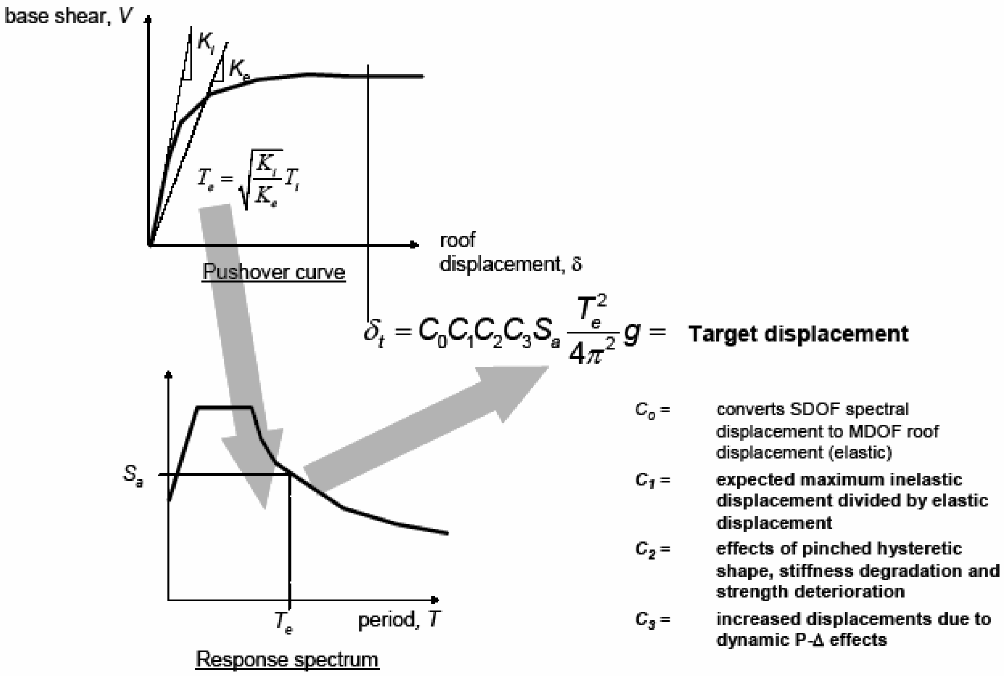


Figure II.8: Coefficient Method (ASCE, 2000) [23].

II.13.3. The Capacity Spectrum Method in ATC 40 (ATC, 1996)

The NSP adopted in this code is the capacity spectrum method proposed by Freeman [5]. This technique, following an equivalent linearization approach, estimates the maximum global displacement of the structure by means of an iterative graphical procedure (Figure). The basic assumption is that the maximum inelastic deformation of a nonlinear SDOFS can be approximated from the maximum deformation of a linear elastic SDOFS with a larger period and damping ratio than the initial values of the inelastic one [5].

According to this procedure, the capacity curve is converted into an equivalent SDOFS pushover response and plotted on the same axes as the seismic ground motion demand in the Acceleration- Displacement Response Spectrum (ADRS) format, assuming a trial-damping ratio. The secant period at the interception identifies the equivalent period of the elastic SDOFS with an equivalent viscous damping ratio proportional to the area enclosed by the capacity curve of the equivalent nonlinear SDOFS. Since both the period and damping are function of the displacement, the procedure requires iterations until the assumed damping is equal to the value computed at the design displacement [5].

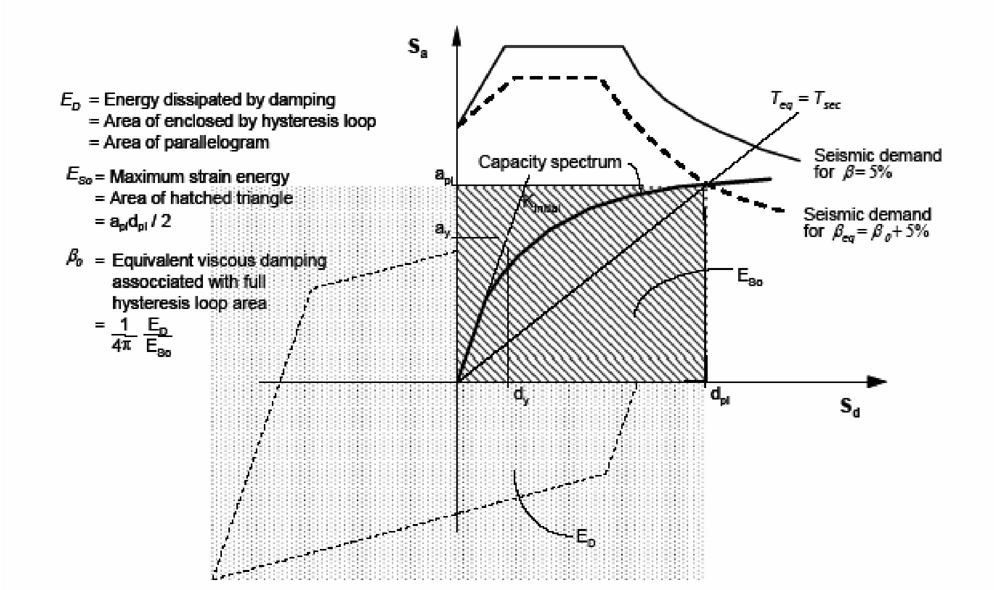


Figure II.9: Capacity Spectrum Method (ATC, 1996) [5].

Recently, the ATC-55 projects (ATC, 2005) demonstrate that the Coefficient Method (as proposed in FEMA 356) as well as the CSM (as proposed in ATC-40) show some inconsistencies in the prediction of the displacement demand. Thus, they propose a new formulation of both design approaches. The obtained updated design methods lead to approximately the same results with a significant improvement in the prediction of the displacement demand when compared to response history analysis results.

As it might be concluded from the discussion reported above, these procedures differ only in the approach used to estimate the global displacement demand (global response parameter, i.e. top floor displacement or the equivalent SDOFS displacement demand); instead, the pushover method adopted will affect not only the global response demand but also the local response parameters of interest, because both are related to the capacity curve obtained. For this reason, a more accurate prediction of the dynamic response by means of a pushover analysis is a fundamental element, and thus the call for further improvements in this field was increase in the last few years.

II.14. PUSHOVER ANALYSIS WITH SAP2000

Nonlinear static pushover analysis is a very powerful feature offered in the nonlinear version of SAP2000. Pushover analysis can be performed on both two and three-dimensional structural models. And can also consist of any number of pushover cases and each pushover case can have a different distribution of lateral load on the structure. A pushover case may start from zero initial conditions, or it may start from the end of a previous pushover case. However, SAP2000 allows plastic hinging during "Gravity" pushover analysis [16].

SAP2000 can also perform pushover analysis as either force-controlled or displacement-controlled. The "Push to Load Level Defined by Pattern" option button is used to perform a force-controlled analysis. The pushover typically proceeds to the full load value defined by the sum of all loads included in the "Load Pattern" box (unless it fails to converge at a lower force value). "The Push to Displacement Magnitude" option button is used to perform a displacement-controlled analysis. The pushover typically proceeds to the specified displacement in the specified control direction at the specified control joint (unless it fails to converge at a lower displacement value). An event-to-event solution strategy is utilized by SAP2000 pushover analysis and the parameters in the right-hand side of the "Options" area control the pushover analysis. The "Minimum Saved Steps" and "Maximum Total Steps" provide control over the number of points actually saved in the pushover analysis. Only steps resulting in significant changes in the shape of the pushover curve are saved for output [16]. "The Maximum Null Steps" is a cumulative counter through the entire analysis to account for the non-convergence in a step due to numerical sensitivity in the solution or a catastrophic failure in the structure. "Iteration Tolerance" and "Maximum Iteration/Step" are control parameters to check static equilibrium at the end of each step in a pushover analysis. If the ratio of the unbalanced-load to the applied-load exceeds the "Iteration Tolerance", the unbalanced load is applied to the structure in a second iteration for that step. These iterations continue until the unbalanced load satisfies the "Iteration Tolerance" or the "Maximum Iterations/Step" is reached. A constant "Event Tolerance" for all elements is used to determine when an event actually occurs for a hinge. Geometric nonlinearity can be considered through P-delta effects or P-delta effects plus large displacements.

Modal and uniform lateral load patterns can be directly defined by SAP2000 in addition to any user-defined static lateral load case. Modal load pattern is defined for any Eigen or Ritz mode while uniform load pattern is defined by uniform acceleration acting in any of the three global directions (accdir X, accdir Y and accdir Z)[16]. Nonlinear behavior of a frame element is represented by specified hinges in SAP2000 and a capacity drop occurs for a hinge when the hinge reaches a negative-sloped portion of its force-displacement curve during pushover analysis.

Such unloading along a negative slope is unstable in a static analysis and SAP2000 provides three different member unloading methods to remove the load that the hinge was carrying and redistribute it to the rest of the structure. In the "Unload Entire Structure" option, when the hinge reaches point C on its force displacement curve program continues to try to increase the base shear. If this results in increased lateral deformation the analysis proceeds. If not, base shear is reduced by reversing the lateral load on the whole structure until the force in that hinge is consistent with the value at point D on its force-displacement curve. All elements unload and lateral displacement is reduced since the base shear is reduced. After the hinge is fully unloaded, base shear is again increased, lateral displacement begins to increase and other elements of the structure pick up the load that was removed from the unloaded hinge. If hinge unloading requires large reductions in the applied lateral load and two hinges compete to unload, i.e., where one hinge requires the applied load to increase while the other requires the load to decrease, the method fails. In the "Apply Local Redistribution" option, only the element containing the hinge is unloaded instead of unloading the entire structure. If the program proceeds by reducing the base shear when a hinge reaches point C, the hinge unloading is performed by applying a temporary, localized, self-equilibrating, internal load that unloads the element [16]. Once the hinge is unloaded, the temporary load is reversed, transferring the removed load to neighboring elements. This method will fail if two hinges in the same element compete to unload, i.e., where one hinge requires the temporary load to increase while the other requires the load to decrease. In the "Restart Using Secant Stiffness" option, whenever any hinge reaches point C on force-displacement curve, all hinges that have become nonlinear are reformed using secant stiffness properties, and the analysis is restarted. This method may fail when the stress in a hinge under gravity load is large enough that the secant stiffness is negative. On the other hand, this method may also give solutions where the other two methods fail due to hinges with small (nearly horizontal) negative slopes. If "Save Positive Increments Only" option box (Figure) is not checked in a pushover analysis, steps in which hinge unloading occur are also saved to represent the characteristics of member unloading method on pushover curve. However, pushover curve will become an envelope curve of all saved points if "Save Positive Increments Only" option box is checked [16].

Although pushover curves obtained from each method have same base shear capacity and maximum lateral displacement, pushover analysis is generally performed by using "Unload Entire Structure" unloading method with "Save Positive Increments Only" option because "Unload Entire Structure" is the most efficient method and uses a moderate number of total and null steps. However, "Apply Local Redistribution" requires a lot of very small steps and null steps that the unloading branch of pushover curve could not be observed usually. "Restart Loading Using Secant Stiffness" is the least efficient method with the number of steps required increasing as the square of the target displacement. It is also the most robust (least likely to fail) provided that the gravity load is not too large [16].

II.15. CONCLUSION

In this chapter, the following conclusions were made:

- Nonlinear static analysis or pushover analysis represents a simplified method for seismic performance evaluation compared to the studied structure.
- Displacement-based procedures provide a more rational approach compared to force-based procedures by considering inelastic deformations rather than elastic forces. The analytical tool for evaluation process should also be relatively simple which can make it easier to capture critical response parameters that significantly affect the evaluation process.
- Pushover analysis is more appropriate for low to mid-rise buildings with dominant fundamental mode response. For special and high-rise buildings, pushover analysis should be complemented with other evaluation procedures since higher modes could certainly affect the response.

Chapter III

Modeling and calculation

CHAPTER III

Modeling and calculation

III.1. INTRODUCTION

The present study compares the seismic response of reinforced concrete frame buildings, from the modeling of masonry filling walls with a single equivalent diagonal strut versus those obtained by applying the recommendations of the current Algerian seismic code (RPA99 / version 2003) which takes into account the stiffness of the filling only through the response reduction factor R without struts. Using SAP2000 software, nonlinear pushover analyses and modal analyses of the Algerian response spectrum (Algiers) were carried out for different 3D configurations (G + 6), such as (1) bare frame, (2) frame with full-height fill panels from two sides only, (3) frame with full-height fill panels from the four sides and (4) frame models with panels of filling with a soft storey located at different levels of the structure. At the end of these analyses, the fundamental natural periods, base shears, storey displacements, inter-storey drift ratios and P-Δ effect are obtained for all the considered models and presented in a comparative way. Furthermore, a discussion is carried out focusing on the variation of the parameters, as well as the various mechanisms of collapse.

III.2. AN OVER VIEW ON MASONRY WALLS

Reinforced concrete structures with masonry infill are used in urban and rural areas all around the world. Brick masonry with cement mortar is the most common filling material because of its abundance, low cost, good acoustic and thermal isolation. Although masonry is very commonly used in constructions, its influence on the dynamic behavior is rarely taken into account in design standards. Indeed, the difficulty of calculations taking into account the masonry walls leads the consulting firms to consider them as static loads (constant loads having no influence on the dynamic behavior). As a matter of fact, experimental studies, numerical or post-seismic studies, however, have shown that their influence during an earthquake is decisive and can even be fatal to the structure, they interact with the frames during an earthquake and thus participate in the resistance to lateral forces caused by an earthquake. The difference between the mechanical characteristics of the frames material and those of the masonry filler walls especially in terms of the stiffness causes a complex behavior during the seismic excitation. Indeed, these walls create a close contact with the beams and columns of the surrounding frame and because of their stiffness in their plans, these filling walls can significantly influence the dynamic behavior of the whole structure such as its resistance, its stiffness and ductility during a seismic event.

III.2.1. Modeling of walls with masonry infill

The modeling of masonry walls under seismic loading is done with two types of approach, local and global. The local approach consists on using a 2D or 3D integration, laws of behavior based on the theory of damage or plasticity, elements of joints etc. Complex phenomena such as corner detachments and material degradation under cyclic loading can thus be taken into account. The complexity of modeling this approach makes it very demanding computing time and requires a great experiment on the part of the engineers.

The global approach is based on replacing masonry with simpler elements namely bar elements, usually one or two and sometimes more. These elements work alternately in compression to reproduce the same behavior of the panel, which is detached from two opposite corners according to the direction of the loading. The bar elements have the mechanical characteristics of the masonry, namely the modulus of Young, the ratio of Poisson, the compressive strength ...etc. If this aspect is unanimous among the experts, we cannot say as much about the geometrical characteristics the equivalent bar (strut), including the width.

III.2.2. The equivalent strut model of Mainstone

During Mainstone [37], Klingner, and Bertero [26], experimental tests on masonry-filled frames subjected to lateral loads, diagonal cracks appeared and had continued to develop in the center of the infill panel, and spacing were formed between the frame and the panel along the unloaded diagonal, while a complete contact was observed in the two corners of the loaded diagonal (Figure III.1) [1].

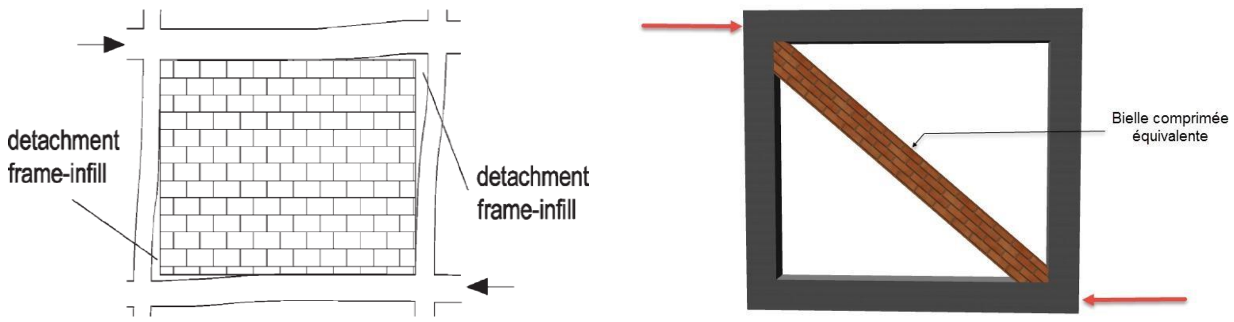


Figure III.1: frame with masonry infill under a lateral load replaced with an equivalent strut.

In 1966, Polyakov [42], introduced the macro-model method, also called the equivalent strut, replacing the masonry filler with an equivalent compressed masonry strut in order to study the overall response of RCC frame buildings with masonry filler. The main disadvantage of this method was the lack of precise modeling of the openings [37].

However, some progress has been made concerning the openings of filling walls where a number of struts can be used to stimulate the effect of the openings [20]. In this study, only the exterior fill walls are modeled like filling panel elements with no opening.

III.2.3. Geometrical characteristics of the equivalent strut

Many formulations have been developed in order to determine the width of the diagonal strut and the resistance of the panel. Some of the recommendations have been adopted in national codes, but they are not a unitary approach to the issue [20]. In this Memoire, the FEMA 356 [23] recommendations which adopt the Mainstone formulation will be followed to model the masonry fill walls.

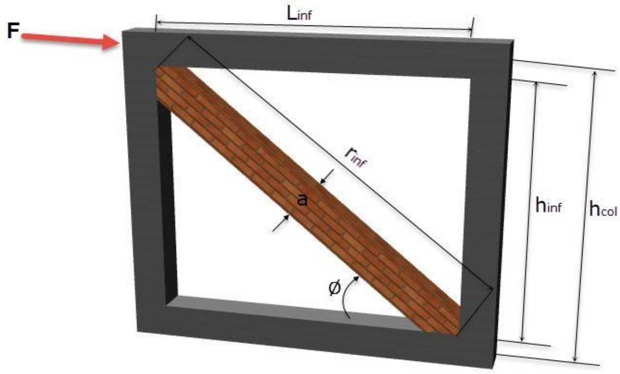


Figure III.2: Geometric characteristics of the equivalent strut.

III.3. AN OVERVIEW ON PLASTIC HINGES

III.3.1. Definition of plastic hinges

_In SAP2000 [16], nonlinear behavior is assumed to occur within a structure at concentrated plastic hinges. The default types include an uncoupled moment hinges, an uncoupled axial hinges, an uncoupled shear hinges and a coupled axial force and biaxial bending moment hinges.

III.3.2. Pre-dimensioning by verifying the criterion of plastic hinges for the structural elements

In our study, the properties of plastic hinges for beams and columns are determined using ATC-40 and FEMA-273. We will introduce beams, columns and struts with their laws of comportment, which are already defined by SAP2000v16 software as follows:

Beams: bending hinge (Default M3) at the start and the end of the element.

Columns: bending hinge (Default P-M2-M3) at the start and the end of the element.

Struts: bending hinge (user defined) in the middle of the element. In addition, no moment or torsion in both direction, and the strut hinge properties are as it shows in (figure II.3).

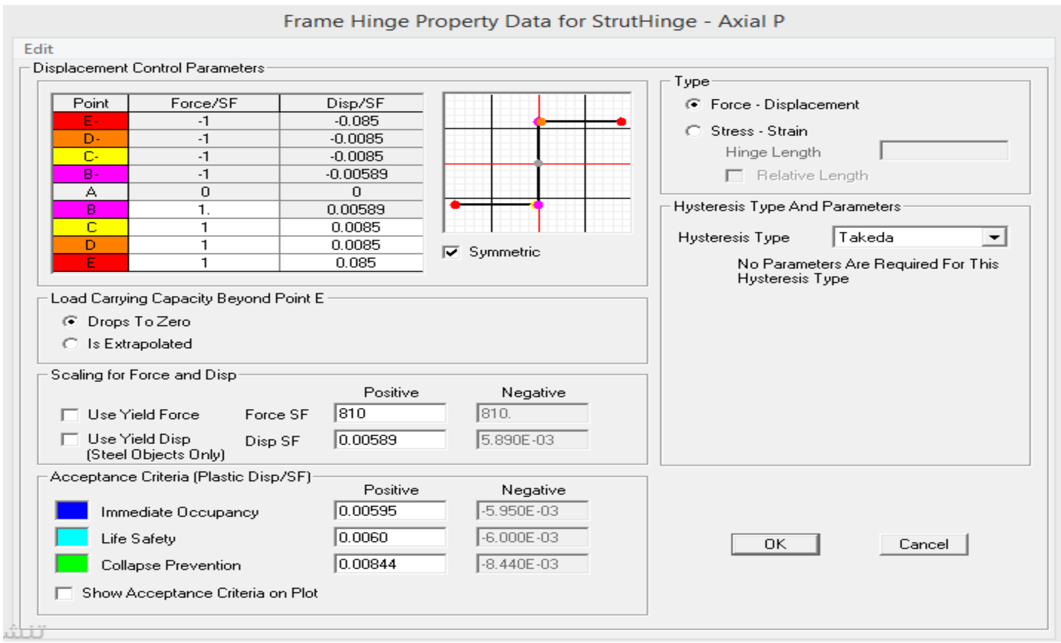


Figure III.3: Hinge properties of the diagonal equivalent strut.

III.3.3. Acceptance Criteria and Performance Level of the plastic hinges (Damage Levels)

FEMA Regulation 273 [23] defines three points to define the state of degradation of each section, as well as its degree of penetration in the plastic field (Figure II.5).

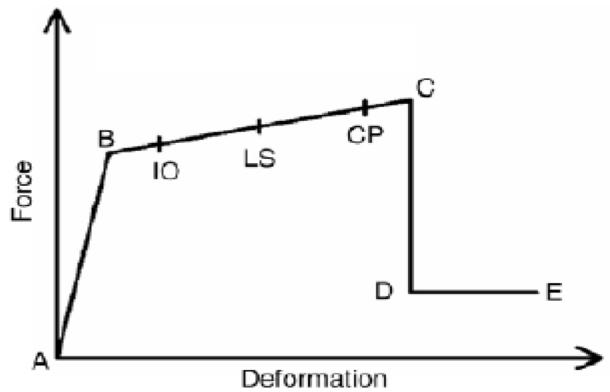


Figure III.4: Force-displacement or moment-rotation curve defining the hinges in SAP2000 (plastic deformation curve).

-Point (A) represents the origin

-Point (B) represents the state of plasticization

-Point (C) represents the ultimate capacity for Pushover analysis

-Point (D) represents residual resistance for Pushover analysis

-Point (E) represents total failure

Other additional points which have no effect on the behavior of the structure, and which are adopted by the calculation code, for the evaluation of static nonlinear analysis, and which are:

-Immediate occupancy IO: damage is relatively limited; the structure retains a significant portion of its original stiffness.

-Life safety level LS: substantial damage has occurred to the structure, and it may have lost a significant amount of its original stiffness. However, a substantial margin remains for additional lateral deformation before collapse would occur.

-Collapse prevention CP: At this level, the building has experienced extreme damage, if it is laterally deformed beyond this point, the structure can experience instability and collapse.

III.4. PRESENTATION OF THE STUDY VARIANT:

It is a residential building with six storeys and a ground floor. The bracing of the building is assured in elevation and in both horizontal directions, by reinforced concrete frames:

- 6 frames in the direction X.
- 5 frames in the direction Y.

-For the first phase of the study, the bracing will be considered as a frame without masonry filler panels.

-For the second phase of the study, the bracing will be considered as a frame with masonry filler panels.

The characteristics of the building

-Dimensions in plan: LX = 20m; LY = 12m.

- Total height: H = (3m × 8) = 21m <48m.

-The building is classified under group of usage 2 (H <48m).

-The building is located in an area of an intensive seismicity (zone III).

It is clear that the objective sought in our study is not the frame without masonry filling, but it is the one with arbitrary filling, so the question that can be asked at this level is: why are we going to make an application on the frame without masonry filling?

In fact, the answer to this question is very simple, it is to highlight the effect of the masonry on RC frames, in other words, to have a support of comparison.

The configurations to be studied

(1) Bare frame.

(2) Frame with full-height fill panels from two sides only.

(3) Frame with full-height fill panels from the four sides.

(4) Frame models with panels of filling with a soft storey located at different levels of the structure as follows:

-Model S1: soft storey located at the first level.

-Model S3: soft storey located at the third level.

-Model S5: soft storey located at the fifth level.

-Model S6: soft storey located at the seventh level.

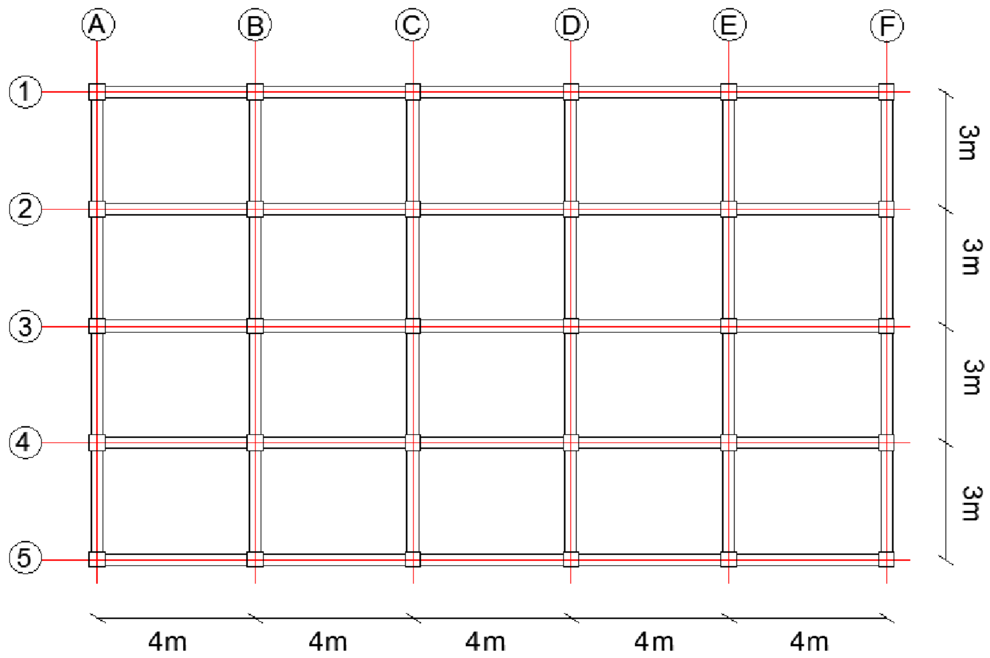


Figure III.5: Plan view of the structures to be studied.

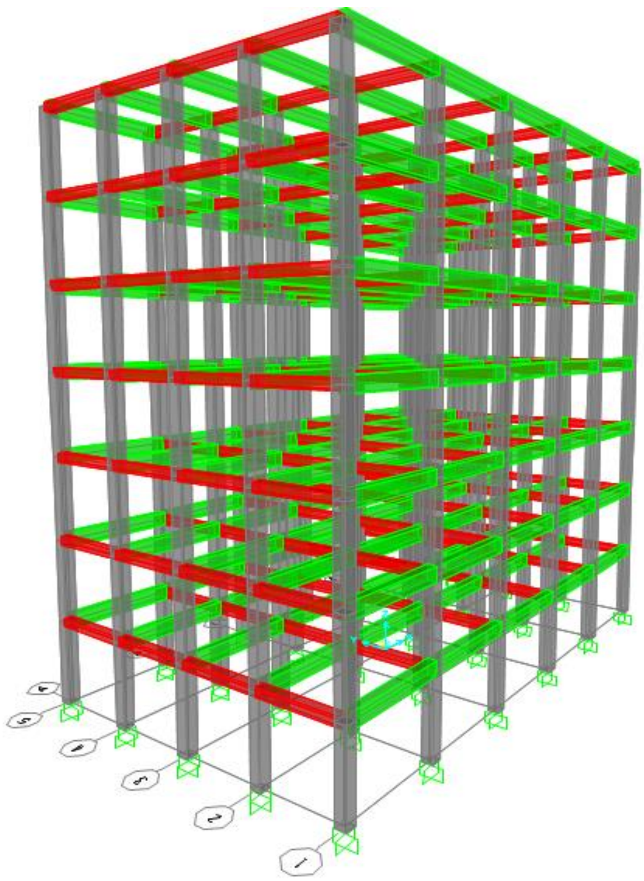


Figure III.6: Representative 3D photo of the structures to be studied (G+6).

III.5. DIMENSIONING AND REINFORCEMENT OF THE ELEMENTS OF THE STUDIED STRUCTURES

III.5.1. Width of the equivalent strut

According to FEMA 356 [8], masonry fill walls prior to cracking are modeled with an equivalent diagonal compression bar of width a . The thickness and modulus of elasticity of the strut are identical to those of the filling panel shown in Figure. The mathematical expression of the width of the equivalent strut, according to Mainstone [45] can be written in terms of the column height between the axes of the beams h_{col} and the length of the diagonal of the infill panel rinf and the coefficient λ_1 as follows:

$$a = 0.175(\lambda_1 h_{col})^{-0.4} r_{rif} = 0.175(0.92 \times 3)^{-0.4} \times 4.38 = 0.506 \text{ m}$$

Where r_{rif} is expressed according to equation

$$r_{rif} = \sqrt{{L_{inf}}^2 + {h_{inf}}^2} = \sqrt{3.6^2 + 2.5^2} = 4.38 \text{ m}$$

The coefficient λ_1 is calculated as a function of the height of the infill panel, the modulus of elasticity of the frame two materials E_{fe} and the material of the infill panel E_{me} , the moment of inertia of the columns I_{col} , the length h_{inf} of the panel filling, the thickness of the filling panel t_{inf} and the angle Φ formed by strut and the horizontal, according to equation below:

$$\lambda_1 = \left[\frac{E_{me} t_{inf} \sin 2\Phi}{4 E_{fe} I_{col} h_{inf}}\right]^{\frac{1}{4}} = \left[\frac{3550 \times 300 \times \sin(2 \times 34.77)}{4 \times 32164 \times 2 \times 0.00213 \times 2.5}\right]^{\frac{1}{4}} = 0.92$$

Note: the depth of the equivalent strut equals to the thickness of the exterior masonry wall, thus it is taken as follows: 30 cm

III.5.2. Pre-dimensioning and reinforcement of the beams and columns

The dimensioning of beams and columns (concrete section and steel section) is carried out considering the structure in its bare configuration.

By respecting the prescriptions of RPA99 / VERSION2003 (article 7-5-1) and (article 7-4-1) [26] the pre-dimensioning of the beams and the estimation of the cross sections of the columns are presented in figure as follows:

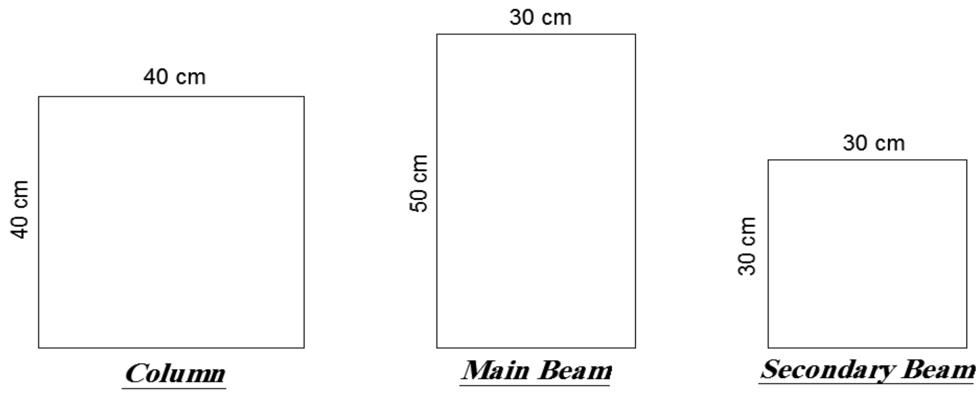


Figure III.7: dimensioning of columns and beams (for all levels including the ground floor).

The dimensioning of the structure is carried out according to the limit state concrete code BAEL91 [45] and the Algerian seismic regulation RPA99 / version 2003 (table 2).

Table III.1: Dimensioning and reinforcement of columns and beams of studied structures.

	Columns	Main beams		Secondary beams	
Level	Steel section	Steel section		Steel section	
		Top	Bottom	Top	Bottom
For all the levels	8 Ø16	3Ø14+3Ø12 8.01 cm^2	3Ø14 4.62 cm^2	3Ø14+3Ø12 8.01 cm^2	3Ø14 4.62 cm^2

III.5.3. Verification and reinforcement of the structure:

Before going into the reinforcement of the structure, the following checks must be made [44]:

-Art 4.3.4. RPA 2003 [26]: the number of modes of vibration to be retained in each of the two directions of excitation must be such that:

-The sum of the effective modal masses for the selected modes is equal to at least 90% of the total mass of the structure.

-All modes having an effective modal mass greater than 5% of the total mass of the structure must be retained for the determination of the total response of the structure.

-Art 4.3.6. RPA 2003 [26]: the resultant of the seismic forces at base Vt obtained by combining the modal values shall not be less than 80% of the seismic force resultant determined by the equivalent static method Vesm for a value of the fundamental period given by the appropriate empirical formula.

-Art 5.10 APR 2003 [26]: The relative lateral displacements of a storey relative to the adjacent storeys must not exceed 1.0% of the height of the storey.

III.6. CHARACTERISTICS OF THE MATERIALS

The properties of the construction materials are as follows:

III.6.1. Concrete characteristics

- Mass per unit volume (density): 2.5KN / m3
- Weight per unit volume: 25 KN / m3
- Modulus of Elasticity (Young's modulus): 32164 Mpa
- Specified concrete compression strength: 25 Mpa
- $v = 0$: The calculation of the loads considering the cracked concrete (ELU).
- $v = 0.2$: Calculation of deformations considering non-cracked concrete (ELS).

III.6.2.Steel characteristics

- The elastic limit of the steel used for the longitudinal and transverse reinforcement is equal to 400 MPa.

III.6.3. Masonry (Red hollow brick) characteristics

- Mass per unit volume (density): 1.5KN / m3
- Weight per unit volume: 15 KN / m3
- Modulus of Elasticity (Young's modulus): 3550 Mpa
- Specified brick compression strength: 6.46 Mpa

-Poisson Modulus: $v = 0$

III.7. APPLIED LOADS AND CONCENTRATED SEISMIC MASSES

III.7.1. Gravity loads

By modeling the slab as a "Deck" element, the software takes into account the loads due to the weight of the joists and the compression layer (concrete), we introduce an additional load that corresponds to the dead weight of the hollow cores, coating + sand + mortar, interior walls ... Etc.

The slab have a thickness of 20 cm (hollow cores + compression slab layer), gravity loads are evaluated as follows:

The gravitary loads due to the dead weight of the structural elements and the slab (the permanent load G) are evaluated at 6.2 KN/m^2 for each storey, and that of the terrace of 6.5 KN /m^2.

Overload (operating loads Q) are estimated at 1.5 KN /m^2 for all storeys, and that of the terrace floor of 1 KN /m^2.

Note: the masonry walls are estimated at 2.89 KN /m^2, they are taken into account only as vertical load applied on the structure, their contribution in the stiffness and strength of the frame is neglected.

III.7.2. Seismic overload

The analysis consists in applying to the structure a gradual incremental lateral force distribution until the top displacement reaches the target displacement or the structure becomes unstable, the latter is carried on using SAP2000 software [10].

III.7.3. Concentrated seismic mass of buildings considered

The concentrated seismic mass for every single storey is calculated as follows:

At a level i (storey i) of the building, this mass is given by the formula:

$$M_i = M_{Gi} + \beta M_{Qi}$$

M_{Gi}: Mass relative to permanent loads (G).

M_{Qi}: Mass relative to operating loads (Q).

β: Weighting coefficient of operating loads, in our case the building is addressed for residential use, hence β = 0.2 (RPA99 / Version 2003. Table 4.5) [26].

III.7.4. Behavior factor R:

In accordance with RPA99 / 2003 version [43], the values of the overall design comportment factor R as well as those of the damping coefficient ξ for the models studied are summarized in Table 1 below. It should be noted that the value of R = 5 adopted for all models with Mainstonne

[45] filling modeled with diagonal struts is justified by the fact that the choice of the other values of R worth 3.5 and 2 would have amplified the rigidity of these models in addition to that of which is provided by the equivalent strut modeled as much as the bar element working uni- axially in the direction of the diagonal.

Table III.2: Values of R and ξ of the models studied in the two configurations.

	RPA recommendations		**Mainstone**	
	R	**ξ**	**R**	**ξ**
Model (WFW)	3.5	7	5	6
Model (NFW)	5	6	5	6
Model (S1)	2	7	5	6
Model (S3)	2	7	5	6
Model (S5)	2	7	5	6
Model (S7)	2	7	5	6

III.8. NUMERICAL APPLICATION OF THE EQUIVALENT STATIC METHOD

III.8.1. Building characteristics

It is assumed that the 6 storeys building models considered are located in a zone of high seismicity “Algiers“ (zone III according to RPA99 / 2003 version) [26], and are addressed for residential use (groupe 2).and rest on a ground type S3 (ferm soil), and last but not least bracing will be considered as a frame with masonry infill.

III.8.2. Calculating of the fundamental period of the structure

$T = 0.09\, h_N / \sqrt{d}$

With $h_N$the total height of the building, $h_N = 21$ m

d: the length of the building in the considered direction.

$d_x = 20$ m $\Longrightarrow$ $T_x = 0.42$ sec

$d_y = 12$ m $\Longrightarrow$ $T_y = 0.545$ sec

III.8.3. Calculation of the total seismic force

$$V = \frac{A.D.Q}{R} W$$

- Acceleration coefficient of zone A :

Seismic zone III and usage group 2, then $A = 0.25$

- Dynamic amplification coefficient D :

$$D = \begin{cases} 2.5.\eta \\ 2.5.\eta.(T_2/T)^{2/3} \\ 2.5.\eta.(T_2/T)^{2/3}.(3/T)^{5/3} \end{cases}$$

T_2: Characteristic period, associated to the category of the site $T_2 = 0.4$ sec

N: correction factor given by the formula

$$\eta = \sqrt{7/2 + \zeta} \geq 0.7$$

Where ξ(%) is the percentage of critical amortization in terms of the constitutive material of the type of the structure and the importance of frame filling.

ξ(%) = 7 %

$$\eta = \sqrt{7/2 + \zeta} = 0.88$$

$T2 \leq T_x \leq 3.0$ sec $\Longrightarrow$ $D_x = 1.637$

$T2 \leq T_y \leq 3.0$ sec $\Longrightarrow$ $D_y = 1.384$

- Factor of quality Q :

Q=1+∑Pq

Q=1.15

- Coefficient of the global behavior of factor of the structure R :

-Freestanding frame with masonry infill.

$R_x = R_y = 3.5$

- Total weight of the structure W :

$W_i = W_{Gi} + \beta W_{Qi}$

Building addressed for residential use $\beta = 0.2$

-Surface of the shell :

$S = [(3.7 \times 2.7) - 0.5^2 \times 4] \times 20 = 179.8 \text{ m}^2$

-Global shell weight:

$[(6.2 + 0.2 \times 1) \times 179.8] + [(5.1 + 0.2 \times 1.5)] \times 179.8 \times 6$ = 6976.24 KN

Dead weight of the structural elements:

-Columns: $(0.4 \times 0.4 \times 25) \times 3 \times 30 \times 7 = 2520$ KN

-Beams:

Direction X-X

$$(0.3 \times 0.5 \times 25) \times 3.6 \times 5 \times 5 \times 7 = 2362.5 \text{ KN}$$

Direction Y-Y

$$(0.3 \times 0.3 \times 25) \times 2.6 \times 4 \times 6 \times 7 = 982.8 \text{ KN}$$

-External walls weight:

$$w_{X-X} = 2.5 \times 3.6 \times 2.89 \times 5 \times 7 \times 2 = 1820.7 \text{ KN}$$
$$w_{Y-Y} = 2.7 \times 2.6 \times 2.89 \times 4 \times 7 \times 2 = 1136.11 \text{ KN}$$

1820.7+1136.11=2956.82 KN

-Total weight of the structure W:

W= 6976.24 + 2362.5 + 982.8 + 2520 + 2956.82 =15798.356 KN

III.8.4. Base shear in the two directions:

$$V_x = 2124.37 \text{ KN}$$

$$V_y = 1797.08 \text{ KN}$$

$T \leq 0.7$ sec $\Longrightarrow$ $F_i = V - \frac{W_i.h_i}{\sum_{j=1}^{n} W_j.h_j}$

$$\begin{cases} W_{i,terace} = 337.5 + 140.4 + 360 + 422.4 + 970.92 = 2231.22 \text{ KN} \\ W_{i,storey} = 337.5 + 140.4 + 360 + 422.4 + 1150.72 = 2124.37 \text{ KN} \\ \sum_{j=1}^{n} W_j.h_j = 2124.37 \times (3 + 6 + 9 + 12 + 15 + 18) + 2231.22 \times 21 \end{cases}$$

$$= 21191198.28 \text{ KN.m}$$

Table III.3: distribution of the seismic forces all along the height of the building.

F_x (KN)		F_y (KN)	
$F_{x,1}$	74.372	$F_{y;1}$	62.9139
$F_{x,2}$	148.744	$F_{y,2}$	125.8277
$F_{x,3}$	223.116	$F_{y,3}$	188.741
$F_{x,4}$	297.488	$F_{y;4}$	251.655
$F_{x,5}$	371.86	$F_{y,5}$	314.5698
$F_{x,6}$	446.232	$F_{y,6}$	377.48
$F_{x,7}$	562.556	$F_{y,7}$	475.886

III.9. DETERMINATION OF THE SEISMIC LOAD

III.9.1. Response Spectral analysis

In order to define the seismic load applied to the studied structures, a response specter of calculation gotten from RPA99 software was defined as shown in figure (III.8), This specter will be integrated in Sap2000 software and participate in the modeling of the studied structures.

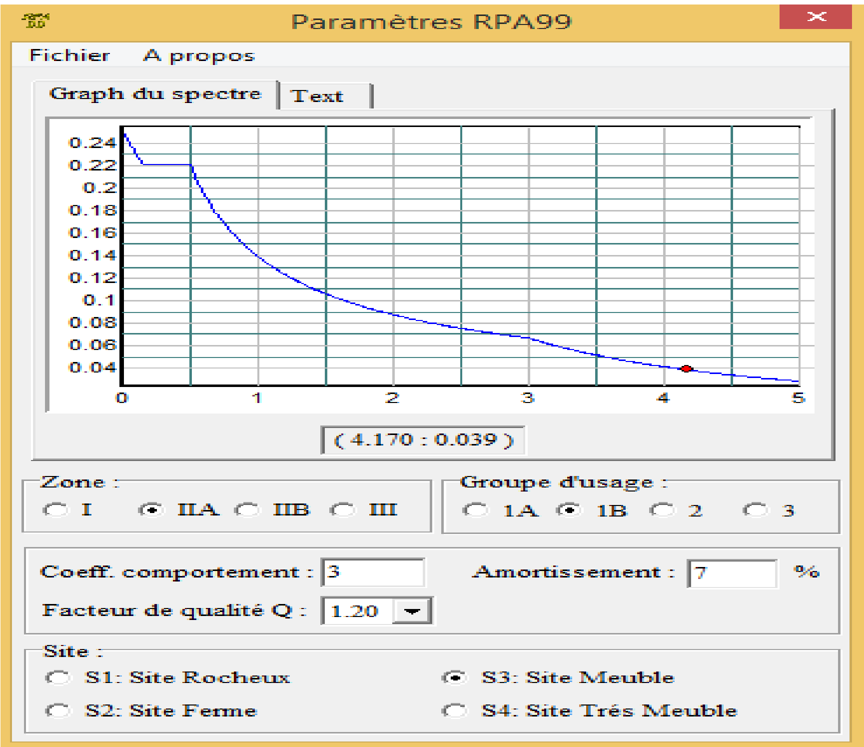


Figure III.8: response specter according to RPA99/2003version.

III.10. MODELING, RESULTS AND AUTOMATIC CALCULATION OF THE STUDIED STRUCTURES

III.10.1. Results of linear analysis (structure with no masonry infill)

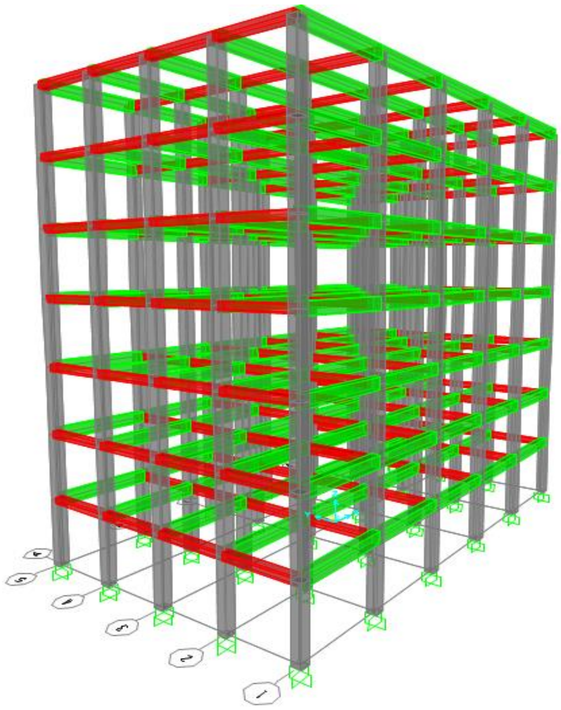


Figure III.9: A representative photo of the studied structure (with no masonry infill).

A / Periods, frequencies and modal participation factors:

Table III.4: Periods, frequencies and modal participation factors of the studied structure.

Mode	Period	Frequency	UX	UY	SumUX	SumUY
Unitless	Sec	Cyc/sec	Unitless	Unitless	Unitless	Unitless
1	0.865199	1.1558E+00	0.00000	0.80810	0.00000	0.80810
2	0.584670	1.7104E+00	0.83174	0.00000	0.83174	0.80810
3	0.273479	3.6566E+00	0.00000	0.10383	0.83174	0.91192
4	0.191800	5.2138E+00	0.09917	0.00000	0.93090	0.91192
5	0.149862	6.6728E+00	0.00000	0.04166	0.93090	0.95359
6	0.111716	8.9513E+00	0.03572	0.00000	0.96662	0.95359
7	0.097265	1.0281E+01	0.00000	0.02319	0.96662	0.97678
8	0.078066	1.2810E+01	0.01801	0.00000	0.98463	0.97678
9	0.069630	1.4362E+01	0.00000	0.01375	0.98463	0.99053
10	0.060140	1.6628E+01	0.00964	0.00000	0.99427	0.99053
11	0.054439	1.8369E+01	0.00000	0.00726	0.99427	0.99779
12	0.050038	1.9985E+01	0.00451	0.00000	0.99878	0.99779

Notes:

-We notice in this sub-application that 93% of the modal masses participation are reached in the 4th mode in the x direction, and the adjacent mode does not exceed 5%.

-We notice in this sub-application that 91% of the modal masses participation are reached in the 3th mode in the y direction, and the adjacent mode does not exceed 5%.

C / Base reactions:

Table III.5: Base reactions of the studied structure.

OutputCase	**CaseType**	**StepType**	**GlobalFX**	**GlobalFY**	**GlobalFZ**
Text	Text	Text	KN	KN	KN
EX	LinRespSpec	Max	1678.803	1.120E-10	5.574E-13
EY	LinRespSpec	Max	9.860E-11	1274.510	1.109E-12

D / Displacements and inter-storey drifts in direction X of frame number 1:

Table III.6: Displacements and inter-storey drifts in direction X of frame number 1.

Storey	**Charge**	**Displacement (m)**	**Drift X (m)**
1	EX	0.002937	**0.0009**
2	EX	0.006821	**0.0012**
3	EX	0.010468	**0.0012**
4	EX	0.013647	**0.0010**
5	EX	0.016216	**0.0008**
6	EX	0.018062	**0.0006**
7	EX	0.019111	**0.0003**

III.10.2. Results of the linear analysis (structure with full height masonry infill at two sides)

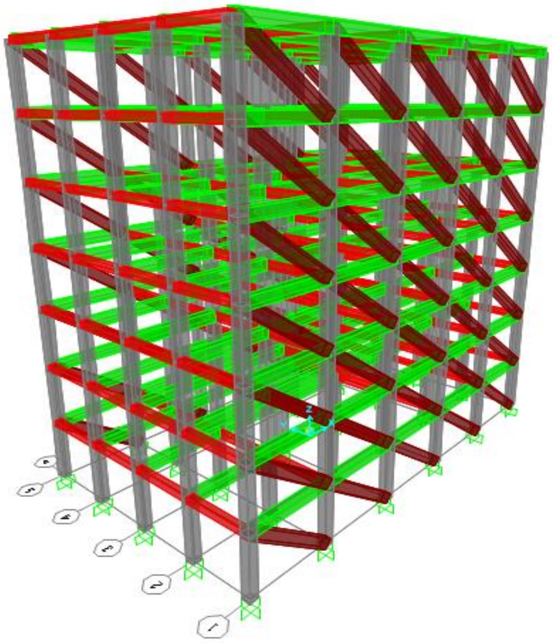


Figure III.10: A representative photo of the structure (with masonry infill at two sides).

A / Periods, frequencies and modal participation factors:

Table III.7: Periods, frequencies and modal participation factors of the studied structure.

Mode	Period	Frequency	UX	UY	SumUX	SumUY
Unitless	Sec	Cyc/sec	Unitless	Unitless	Unitless	Unitless
1	0.927053	1.0787	0.00000	0.80826	0.00000	0.80826
2	0.534244	1.8718	0.83612	0.00000	0.83612	0.80826
3	0.293328	3.4092	0.00000	0.10376	0.83612	0.91202
4	0.176559	5.6638	0.09933	0.00000	0.93545	0.91202
5	0.160804	6.2188	0.00000	0.04166	0.93545	0.95368
6	0.104431	9.5757	0.00000	0.02316	0.93545	0.97684
7	0.103957	9.6193	0.03418	0.00000	0.96963	0.97684
8	0.074803	13.368	0.00000	0.01372	0.96963	0.99056
9	0.073836	13.544	0.01666	0.00000	0.98629	0.99056
10	0.058513	17.090	0.00000	0.00724	0.98629	0.99780
11	0.057767	17.311	0.00867	0.00000	0.99496	0.99780
12	0.050278	19.889	0.00000	0.00220	0.99496	1.00000

Notes:

- We notice in this sub-application that 93 of the modal masses participation are reached in the 4th mode in the x direction, and the adjacent mode does not exceed 5%.

-We notice in this sub-application that 91% of the modal masses participation are reached in the 3rd mode in the y direction, and the adjacent mode does not exceed 5%.

C / Base reactions:

Table III.8: Base reactions of the studied structure.

OutputCase	CaseType	StepType	GlobalFX	GlobalFY	GlobalFZ
Text	Text	Text	KN	KN	KN
EX	LinRespSpec	Max	2077.032	2.220E-10	2.152E-13
EY	LinRespSpec	Max	2.451E-10	1419.014	2.648E-13

D / Displacements and inter-storey drifts in direction X of frame number 1:

Table III.9: Displacements and inter-storey drifts in direction X of frame number 1.

Storey	Charge	Displacement (m)	Drift X (m)
1	EX	0.002745	**0.0009**
2	EX	0.006162	**0.0011**
3	EX	0.009338	**0.0010**
4	EX	0.012112	**0.0009**
5	EX	0.014366	**0.0007**
6	EX	0.015998	**0.0005**
7	EX	0.016941	**0.0003**

III.10.3. Results of the linear analysis (structure with full height masonry infill at the four sides)

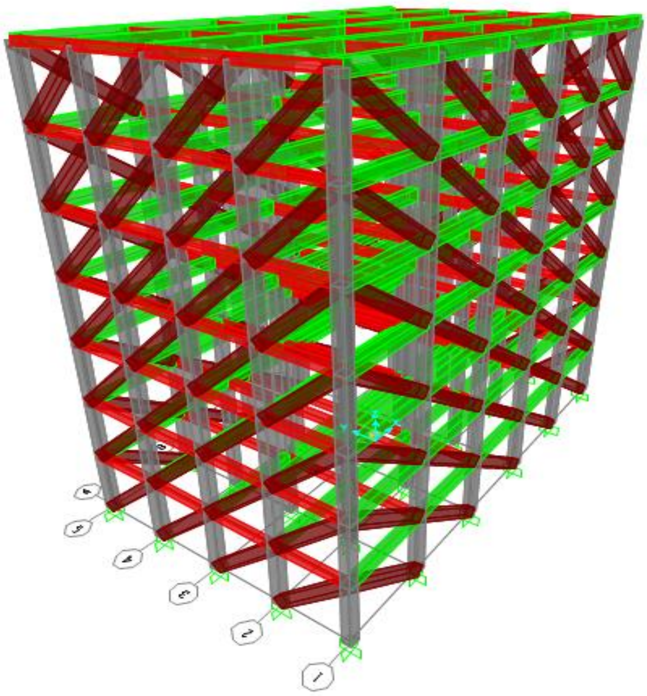


Figure III.11: A representative photo of the structure (with masonry infill at the four sides).

A / Periods, frequencies and modal participation factors:

Table III.10: Periods and factors of modal participation of the studied structure.

Mode	Period	Frequency	UX	UY	SumUX	SumUY
Unitless	Sec	Cyc/sec	Unitless	Unitless	Unitless	Unitless
1	0.760647	1.3147	4.939E-06	0.81822	4.939E-06	0.81822
2	0.555350	1.8007	0.83622	4.677E-06	0.83622	0.81823
3	0.246163	4.0624	5.475E-08	0.10295	0.83622	0.92118
4	0.183552	5.4481	0.09924	5.738E-08	0.93546	0.92118
5	0.139476	7.1697	1.662E-08	0.03809	0.93546	0.95926
6	0.108097	9.2510	0.03417	2.371E-09	0.96963	0.95926
7	0.094258	10.609	1.668E-08	0.02053	0.96963	0.97979
8	0.076777	13.025	0.01666	7.751E-09	0.98629	0.97979
9	0.069735	14.340	4.002E-09	0.01201	0.98629	0.99180
10	0.060069	16.648	0.00867	1.383E-10	0.99496	0.99180
11	0.055769	17.931	9.734E-10	0.00630	0.99496	0.99810
12	0.050593	19.765	0.00398	8.721E-12	0.99894	0.99810

Notes:

- We note in this sub-application that 93% of the modal masses participation are reached in the 4th method in the x direction, and the adjacent mode does not exceed 5%.

-We notice in this sub-application that 92% of the modal masses participation are reached in the 3rd method in the y direction, and the adjacent mode does not exceed 5%.

C / Base reactions:

Table III.11: Base reactions of the studied structure.

OutputCase	CaseType	StepType	GlobalFX	GlobalFY	GlobalFZ
Text	Text	Text	KN	KN	KN
EX	LinRespSpec	Max	2190.178	6.073	2.932E-13
EY	LinRespSpec	Max	6.073	1759.851	1.294E-12

D / Displacements and inter-storey drifts in direction X of frame number 1:

Table III.12: Displacements and inter-storey drifts in direction X of frame number 1.

Storey	Charge	Displacement (m)	Drift X (m)
1	EX	0.002872	**0.0009**
2	EX	0.006449	**0.0011**
3	EX	0.009775	**0.0011**
4	EX	0.012684	**0.0009**
5	EX	0.015049	**0.0007**
6	EX	0.016762	**0.0005**
7	EX	0.017753	**0.0003**

-Interpretation of the results :

-We notice that the existence of struts decreased significantly the periods of vibrations.

- Base shear forces have increased in a significant way compared to the structure with no masonry infill; this was expected anyway because the struts stiffen the structure.

- We notice also that the existence of struts decreases inter-storey drift displacements as shown in the following figure:

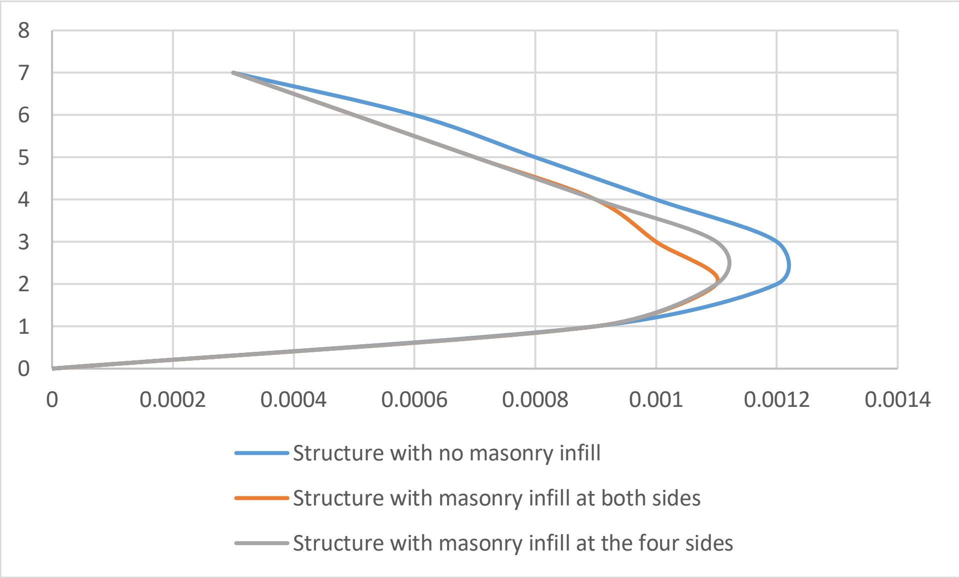


Figure III.12: Curves of inter-storey drifts in direction X.

III.10.4. Results of nonlinear analysis (structure without filling)

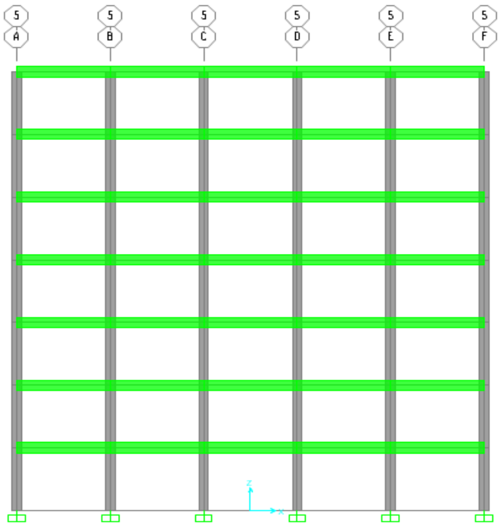


Figure III.13: plan view of the frame (number1) to be studied (structure without filling).

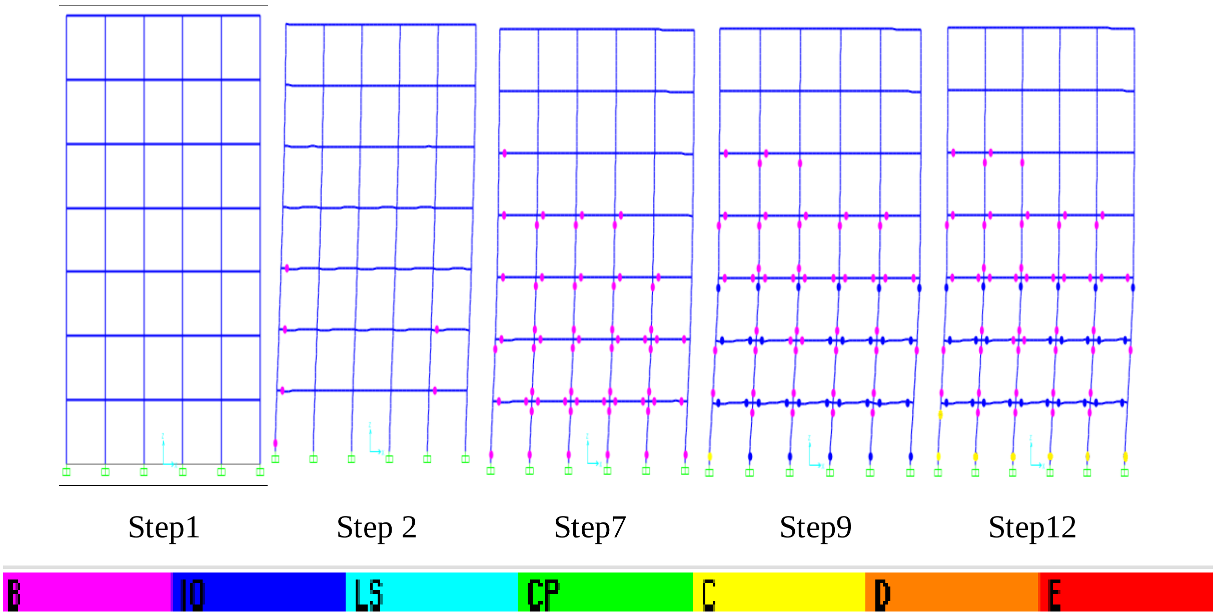


Figure III.14: the different steps of the formation of the plastic hingess of the structure without filling.

-Commentary:

- In the 1st step no plastic hinges appearance in the frame.

-At the 2nd steps, appearance of the hinges of type B at the ends of the beams of the 2nd and 3rd and 4th level and at the foot of the 1st column of the 1st level, which means that there is no deformation at the level of the hinges, because all the elastic deformations are ignored.

- At the 7th level, appearance of the hinges of type B at the ends of columns and beams of the 5 first levels of the frame which means that there is no deformation at the level of the hinges, because all the elastic deformations are ignored.

- At the 9th step, we can notice the development of the new hinges of type IO at the ends of beams and columns of the 1st and 2nd and 3rd levels, and appearance of a hinge of type C at the head of the first column of the 1st level, this means that the damage is relatively limited.
-In the 12 th stage, more hinges of type C have developed at the feet of the columns of the first level, which This means that a minor damage is likely to develop.

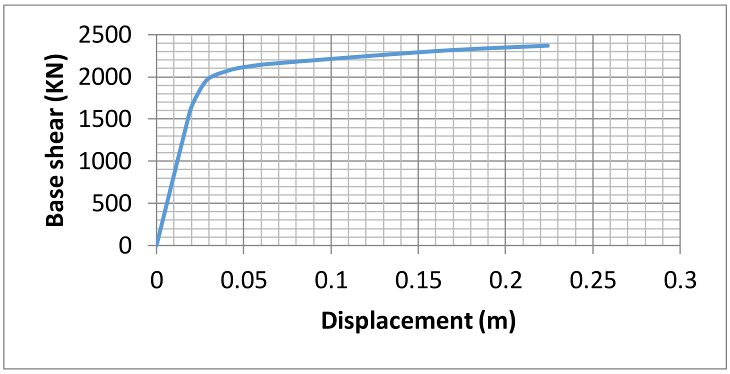


Figure III.15: Capacity curve (base shear-displacement) of the structure without filling.

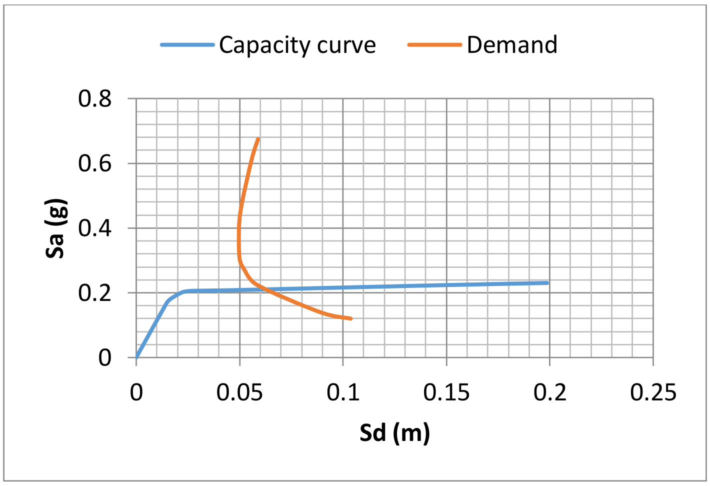


Figure III.16: capacity curve (acceleration spectrum - displacement spectrum) of the structure without filling.

III.10.5. Results of the non-linear analysis (structure with filling at both sides)

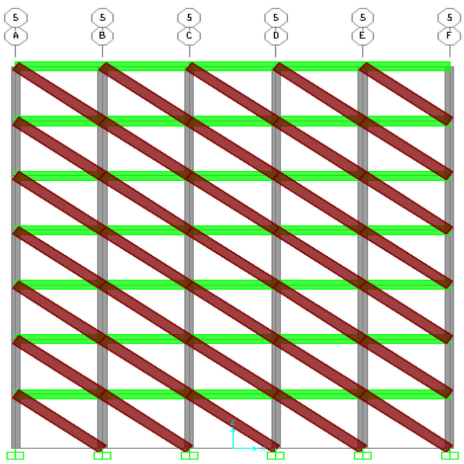


Figure II.17: plan view of frame number1 to be studied (structure with filling on both sides).

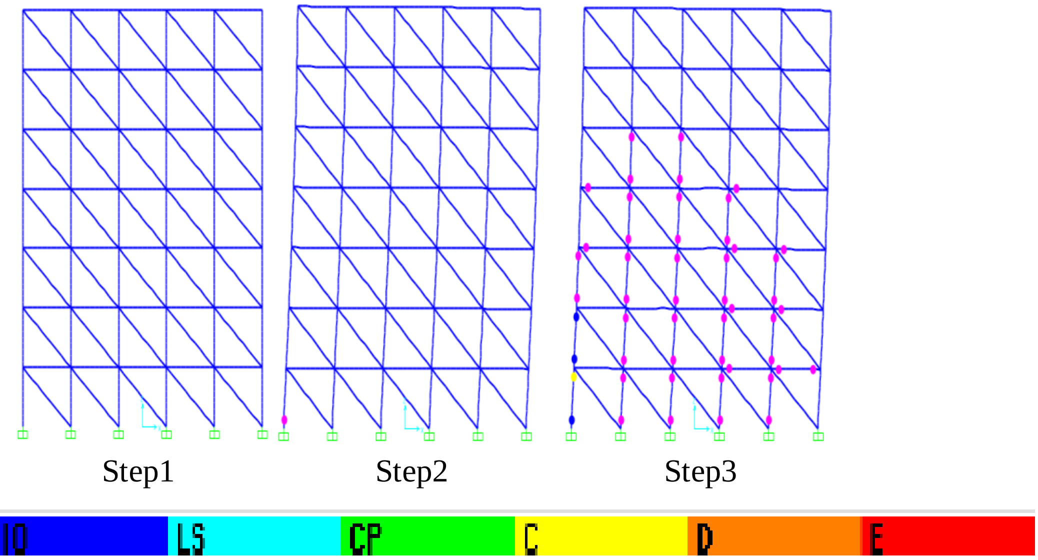


Figure III.18: the different steps of the formation of the plastic hinges of the structure with filling at both sides.

-Commentary:

- In the 1st step no plastic hinges appearance in the frame.

-In the 2nd step, appearance of one hinge of type B at the foot of the 1st beam of the 1st level, this means that no deformation at the level of the hinge, because all the elastic deformities are ignored.

-In the 3rd step, the number of the hinges of type B doubled and reached the ends of columns and beams of the 6 first levels of the frame, and development of hinges type IO at the end of the first columns of the first 2 levels and also we can notice the appearance of a hinge of type C on the head of the first column of the first level, all of the latter means that a minor damage is likely to develop.

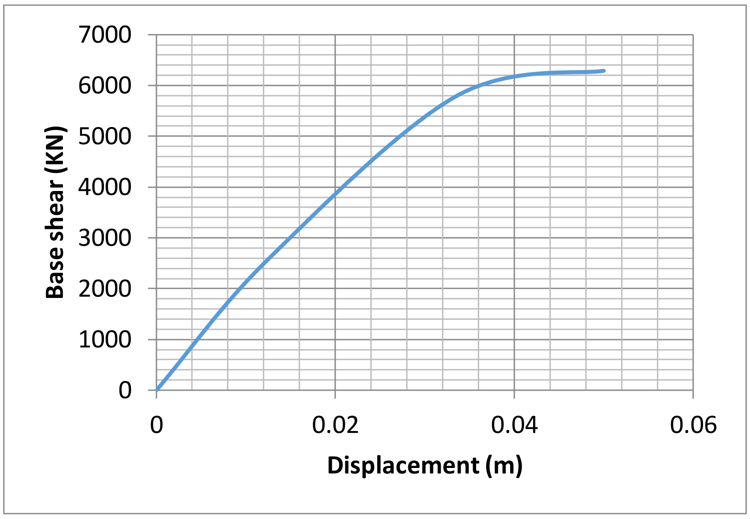


Figure III.19: Capacity curve (base shear-displacement force) of the structure with filling at both sides.

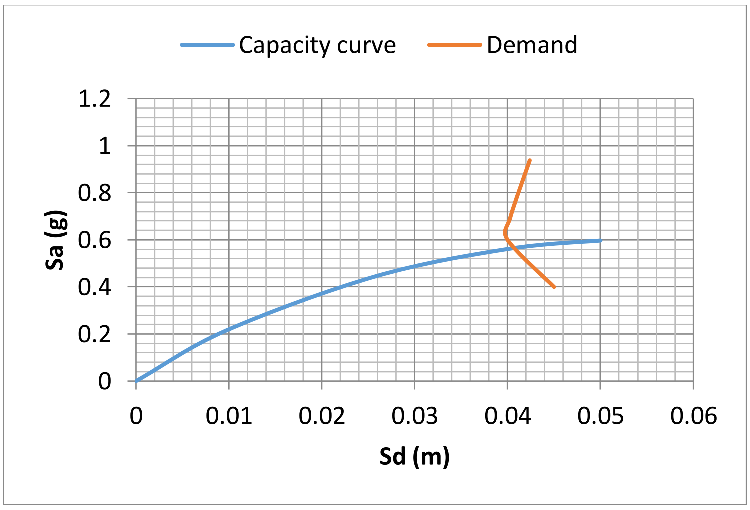


Figure III.20: capacity curve (acceleration spectrum - displacement spectrum) of the structure with filling at both facades.

III.10.6. RESULTS OF MASONRY INFILL FRAME MODELS WITH A SOFT STOREY LOCATED IN DIFFERENT LEVELS

III.10.6.1. model 1: masonry infill frame with a soft storey at the 1st level

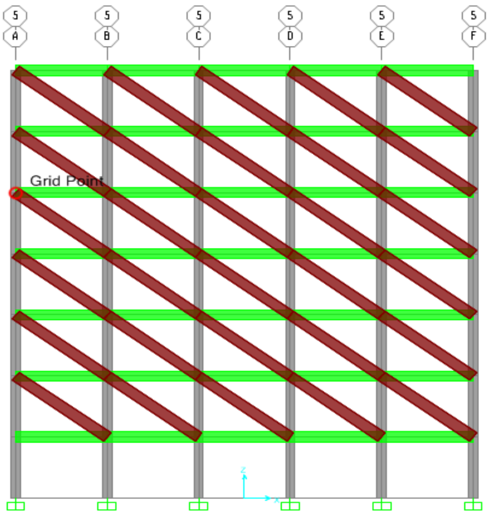


Figure III.21: plan view of frame number 1 to be studied (model 1).

A / Inter-storey displacements in X direction of frame number 1 (model 1)

Table III.13: Inter-storey displacements in the X direction (model 1).

Storey	Charge	Displacement (m)	Drift X (m)
1	EX	0.004090	0.0013
2	EX	0.007377	0.0011
3	EX	0.010407	0.0010
4	EX	0.013028	0.0009
5	EX	0.015131	0.0007
6	EX	0.016522	0.0005
7	EX	0.017440	0.0003

B / Base reactions (model 1)

Table III.14: Base reactions of the studied structure direction (model 1).

OutputCase	CaseType	StepType	GlobalFX	GlobalFY	GlobalFZ
Text	Text	Text	KN	KN	KN
EX	LinRespSpec	Max	2050.099	2.723E-10	1.630E-12
EY	LinRespSpec	Max	2.895E-10	1403.400	4.158E-13

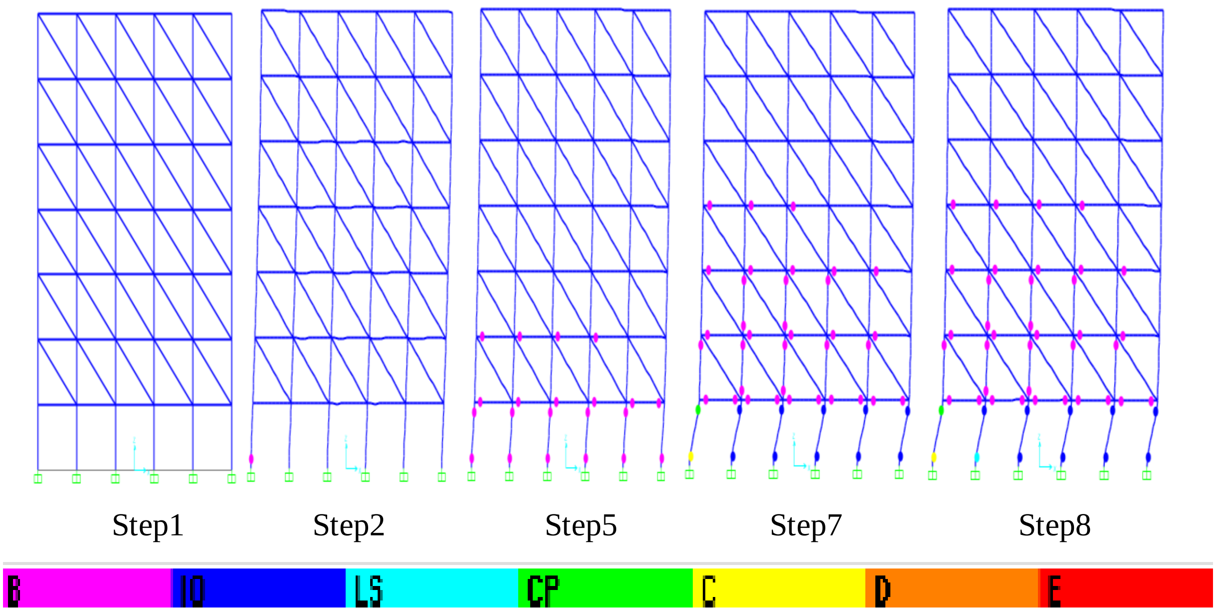


Figure III.22: the different steps of the formation of the plastic hinges (model 1).

-Commentary:

- In the 1st step no plastic hinges appearance in the frame.

-In the 5th step, appearance of hinge of type B at the ends of beams and columns of the 3 first levels, this means that no deformation at the level of the hinges, because all the elastic deformities are ignored.

-In the 3rd step, the number of hinges of type B doubled and reached the ends of columns and beams of the 5 first levels of the frame, and development of hinges type IO at the ends of the columns of the first level, and also we can notice the appearance of a hinge of type CP and a hinge of type C on the head and the foot of the first column of the first level, all of the latter means that a minor damage is likely to develop.

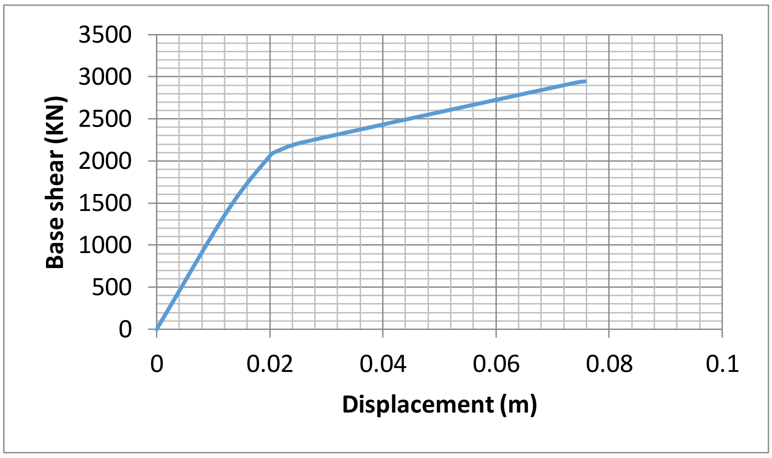


Figure III.23: Capacity curve (base shear-displacement) -model 1.

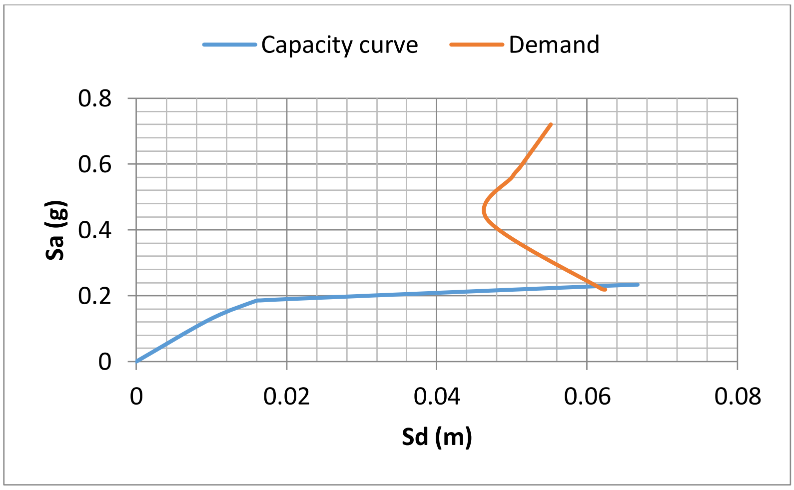


Figure III.24: capacitance curve (acceleration spectrum -spectrum displacement) -model 1.

III.10.6.2. model 2: masonry infill frame with a soft storey at the 3rd level

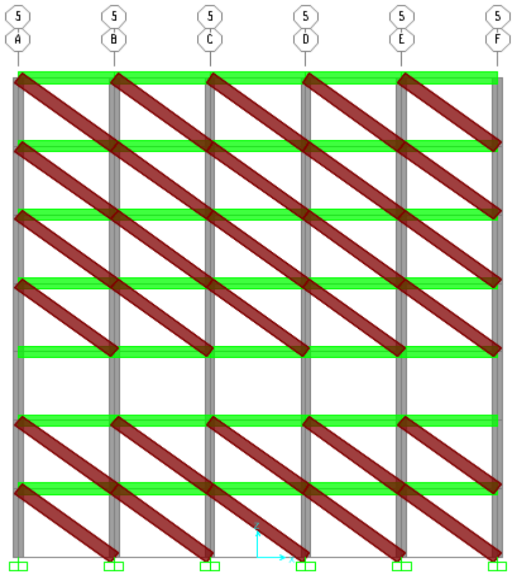


Figure III.25: plan view of the frame number 1 to be studied (model 2).

A / Inter-storey displacements in the X direction of frame number 1 (model 2)

Table III.15: Inter-storey displacements in the X direction (model 2).

Storey	*Charge*	*Displacement (m)*	*Drift X (m)*
1	EX	0.00268	**0.0009**
2	EX	0.005677	**0.0010**
3	EX	0.009007	**0.0011**
4	EX	0.012128	**0.0010**
5	EX	0.014531	**0.0008**
6	EX	0.016322	**0.0006**
7	EX	0.017540	**0.0004**

B / Base reactions (model 2)

Table III.16: Base reactions of the structure (model 2).

OutputCase	*CaseType*	*StepType*	*GlobalFX*	*GlobalFY*	*GlobalFZ*
Text	Text	Text	KN	KN	KN
EX	LinRespSpec	Max	1976.319	3.319E-09	2.665E-12
EY	LinRespSpec	Max	3.341E-09	1390.996	1.466E-12

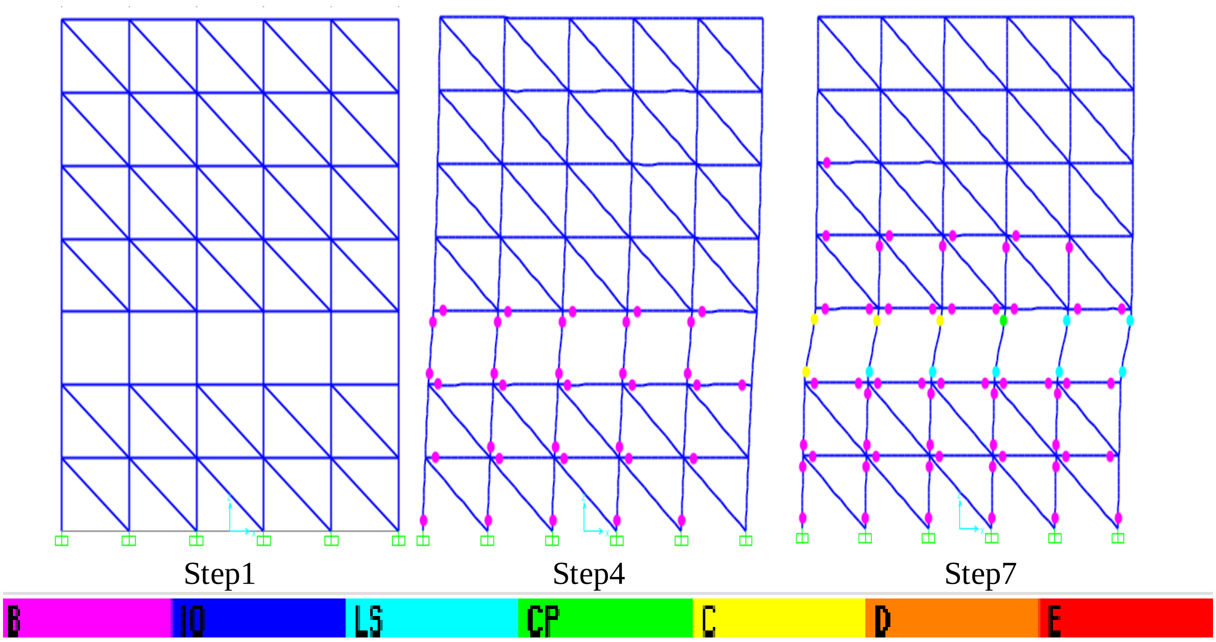


Figure III.26: the different steps of the formation of plastic hinges (model 2).

-Commentary:

- In the 1st step no plastic hinges appearance in the frame.

-In the 4th step, appearance of hinge of type B at the ends of beams and columns of the 4 first levels, this means that no deformation at the level of the hinges, because all the elastic deformities are ignored.

-In the 7th step, development of hinges of type LS at the ends of the columns of the soft storey (3rd level), and also we can notice the appearance of a hinge of type CP and hinges of type C at the ends of the columns of the soft storey too, which will cause the shearing of the columns of the soft storey.

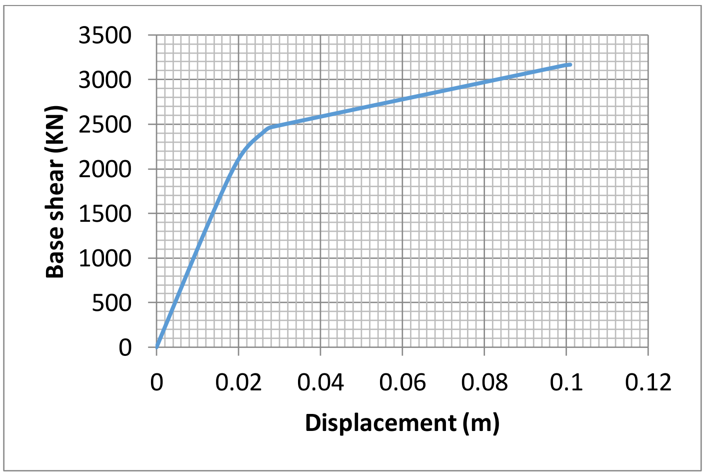


Figure III.27: Capacity curve (base shear-displacement) -model 2.

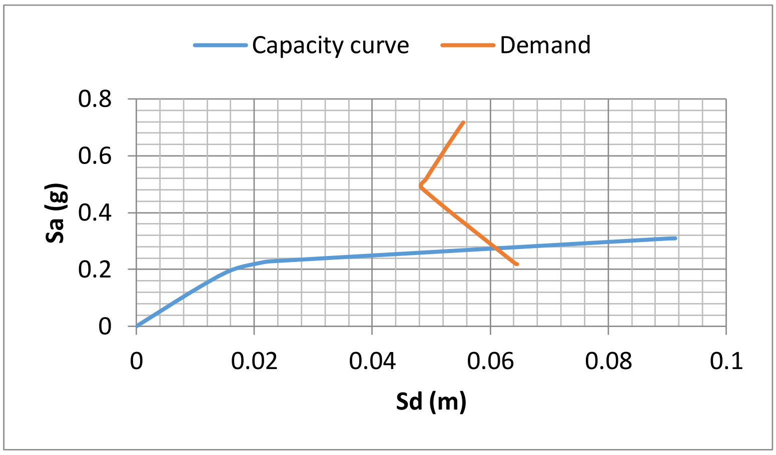


Figure III.28: capacitance curve (acceleration spectrum -spectrum displacement) -model 2.

III.10.6.3. model 3: masonry infill frame with a soft storey at the 5th level

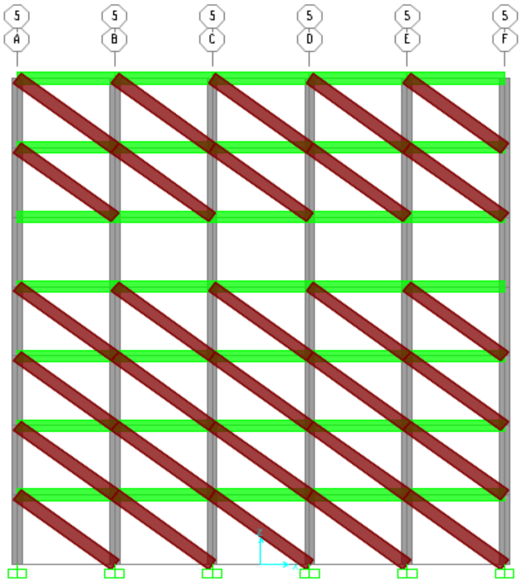


Figure III.29: plan view of the frame number 1 to be studied (model 3).

A / Inter-storey displacements in the X direction of gantry number 1 (model 3):

Table III.17: Inter-storey displacements in the X direction (model 3).

Storey	Charge	Displacement (m)	Drift X (m)
1	*EX*	*0.002472*	***0.0008***
2	*EX*	*0.005195*	***0.0009***
3	*EX*	*0.008178*	***0.0010***
4	*EX*	*0.011466*	***0.0011***
5	*EX*	*0.014415*	***0.0010***
6	*EX*	*0.016511*	***0.0007***
7	*EX*	*0.017433*	***0.0003***

B / Base reactions (model 3):

Table III.18: Base reactions of the structure (model 3).

OutputCase	*CaseType*	*StepType*	*GlobalFX*	*GlobalFY*	*GlobalFZ*
Text	*Text*	*Text*	*KN*	*KN*	*KN*
EX	*LinRespSpec*	*Max*	*2022.161*	*3.702E-10*	*1.022E-12*
EY	*LinRespSpec*	*Max*	*3.798E-10*	*1399.541*	*4.954E-13*

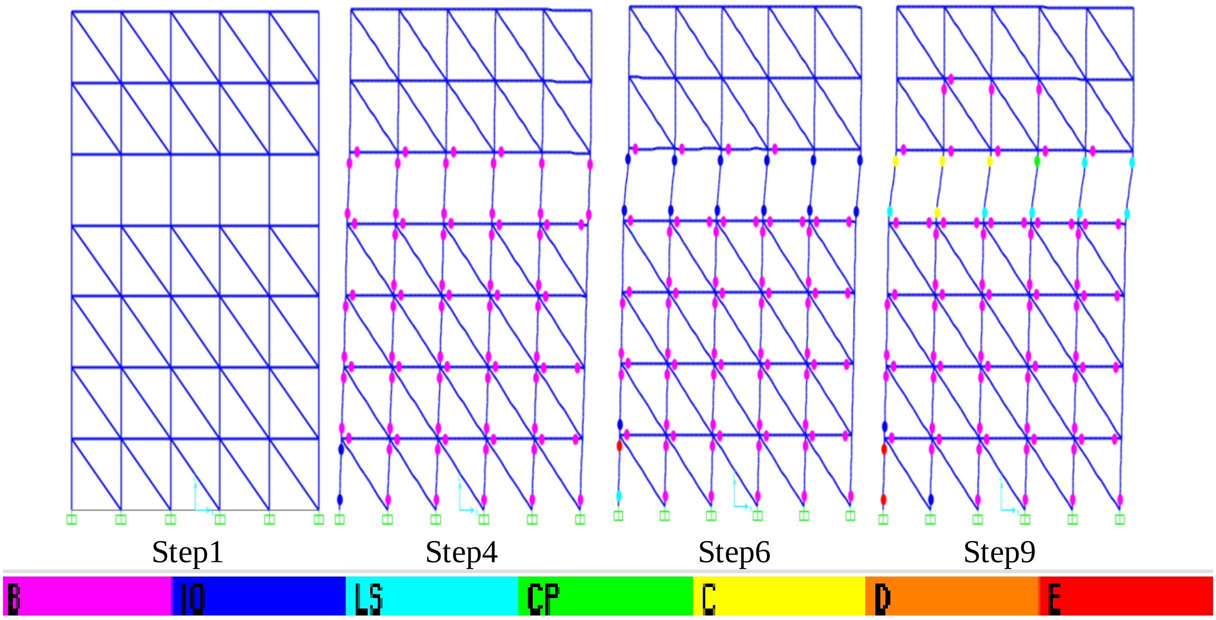


Figure III.30: the different stages of the formation of plastic hinges (model 3).

-Commentary:

- In the 1st step no plastic hinges appearance in the frame.

-In the 4th step, appearance of hinge of type B at the ends of beams and columns of the 6 first levels, this means that no deformation at the level of the hinges, because all the elastic deformities are ignored.

-In the 6th step, development of hinges of type IO at the ends of the columns of the soft storey (6th level), and also we can notice the appearance of a hinge of type LS and a hinge of type E on the head and the foot of the first column of the first level, all of the latter means that a minor damage is likely to develop.

-In the 9th step, development of hinges of type LS at the ends of the columns of the soft storey (6th level), and also we can notice the appearance of a hinge of type CP and hinges of type C on the head and the foot of the first column of the soft storey too, which will cause the shearing of the columns of the soft storey.

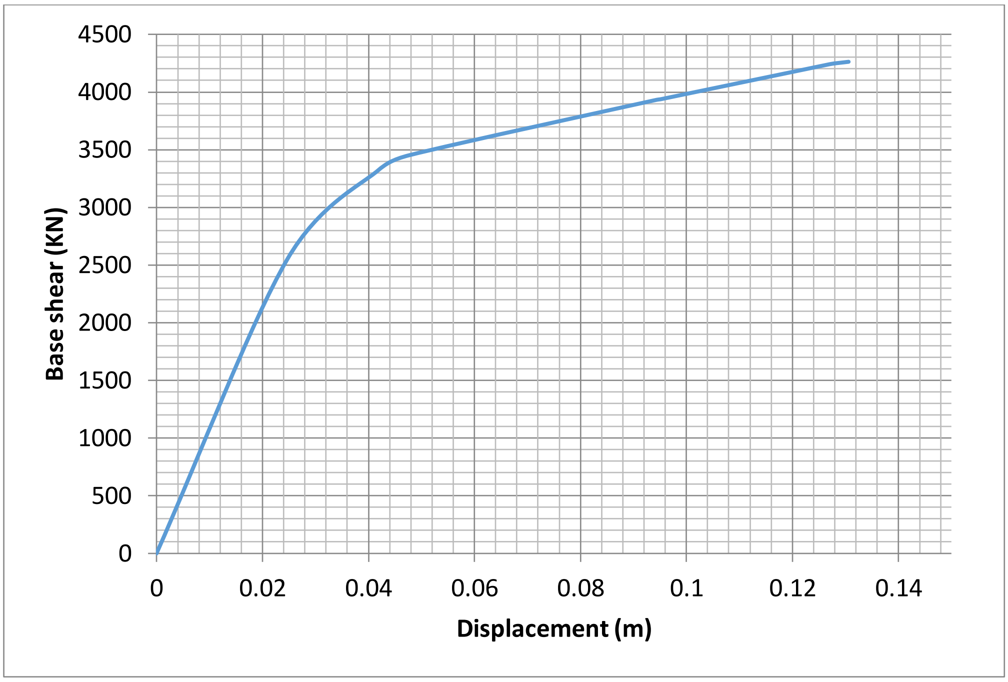


Figure III.31: Capacity curve (shear-displacement) -model 3.

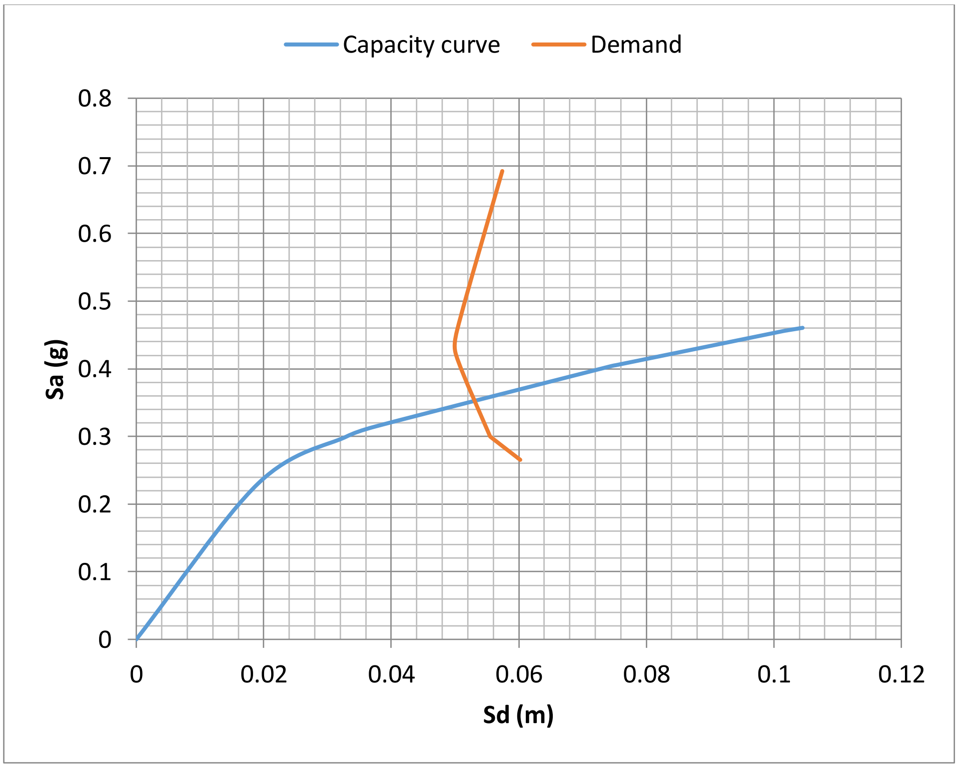


Figure III.32: capacitance curve (acceleration spectrum -spectrum displacement) -model 3.

III.10.6.4. model 4: masonry infill frame with a soft storey at the 7th level

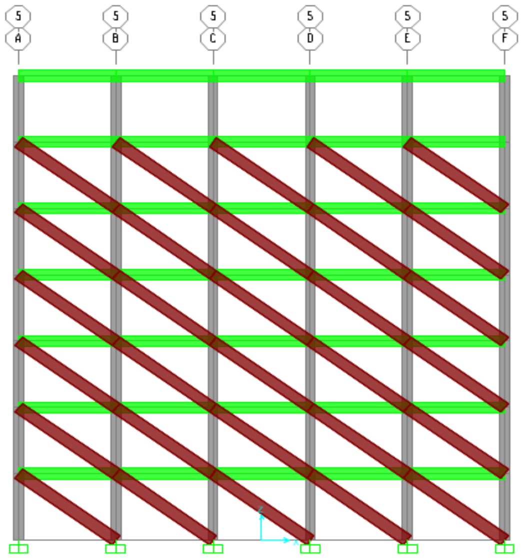


Figure III.33: plan view of frame number 1 to be studied (model 4).

A / Inter-storey displacements in the X direction of frame number 1 (model 4):

Table III.19: Inter-storey displacements in the X direction (model 4).

Storey	*Charge*	*Displacement (m)*	*Drift X (m)*
1	*EX*	*0.002717*	***0.0009***
2	*EX*	*0.006096*	***0.0011***
3	*EX*	*0.009026*	***0.0010***
4	*EX*	*0.011745*	***0.0009***
5	*EX*	*0.014131*	***0.0008***
6	*EX*	*0.016212*	***0.0007***
7	*EX*	*0.018089*	***0.0006***

B / Base reactions (model 4):

Table III.20: Base reactions of the structure (model 4).

OutputCase	*CaseType*	*StepType*	*GlobalFX*	*GlobalFY*	*GlobalFZ*
Text	*Text*	*Text*	*KN*	*KN*	*KN*
EX	*LinRespSpec*	*Max*	*2056.800*	*1.482E-10*	*4.427E-13*
EY	*LinRespSpec*	*Max*	*1.346E-10*	*1409.853*	*1.285E-12*

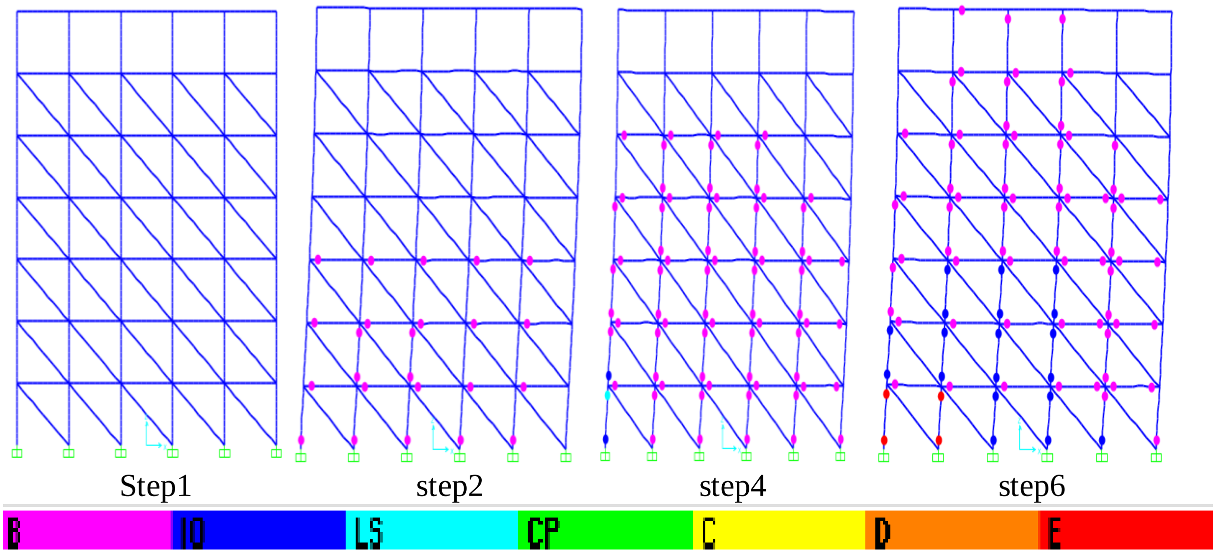


Figure III.34: the different steps of the formation of plastic hinges (model 4).

-Commentary:

- In the 1st step no plastic hinges appearance in the frame.

-In the 2nd step, appearance of hinge of type B at the ends of beams and columns of the 4 first levels, this means that no deformation at the level of the hinges, because all the elastic deformities are ignored.

-In the 4th step, the number of hinges of type B doubled and reached the ends of columns and beams of the 6 first levels of the frame, and a development of hinges of type IO at the feet of the columns of the 2 first levels, and also we can notice the appearance of a hinge of type CP on the head the first column of the first level, all of the latter means that a minor damage is likely to develop.

-In the 6th step, appearance of hinges of type B in the 6th level and on the columns of the soft storey (7th level) development of hinges of type IO at the ends of the columns of the first three levels, and also we can notice the appearance of hinges of type E on the head and the foot of the first and the second column of the first level, all of the latter means that our structure is at the limit of resistance (no resistance capability).

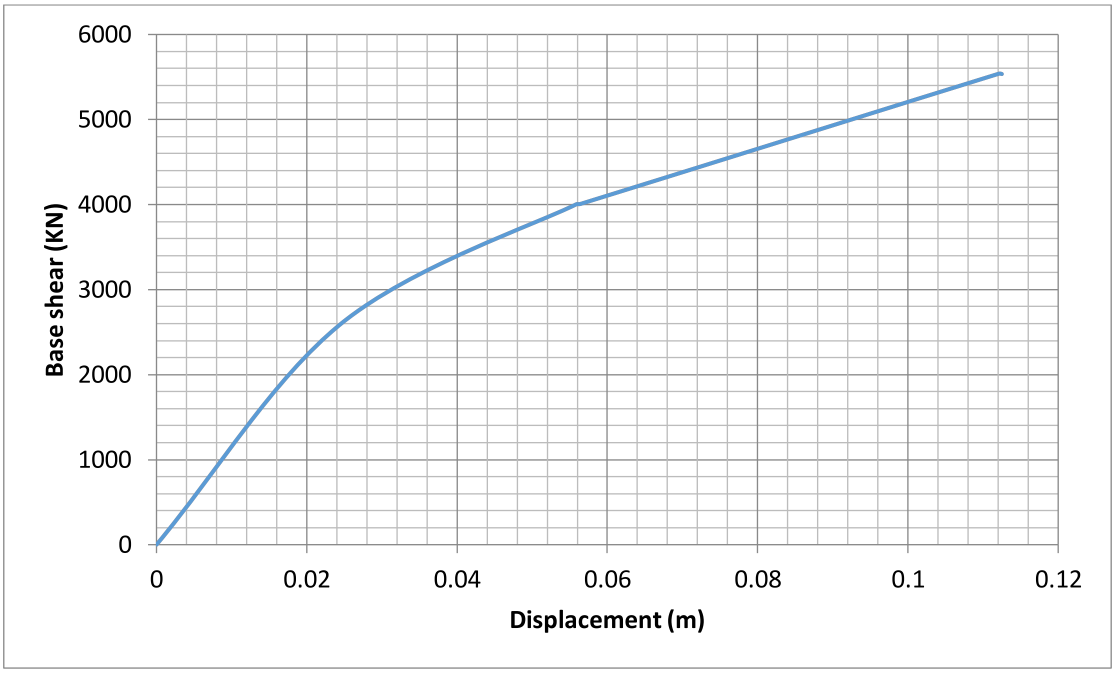


Figure III.35: Capacity curve (base shear-displacement) -model 4.

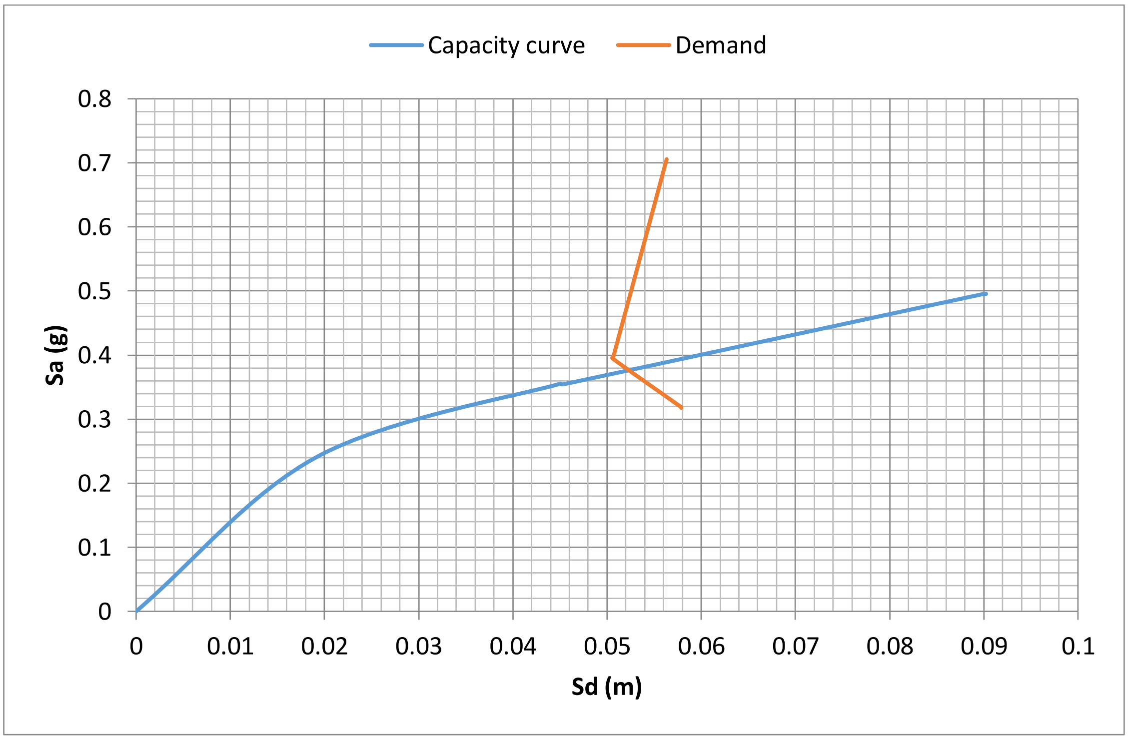


Figure III.36: capacity curve (acceleration spectrum -spectrum displacement) -model 4.

Results interpretation:

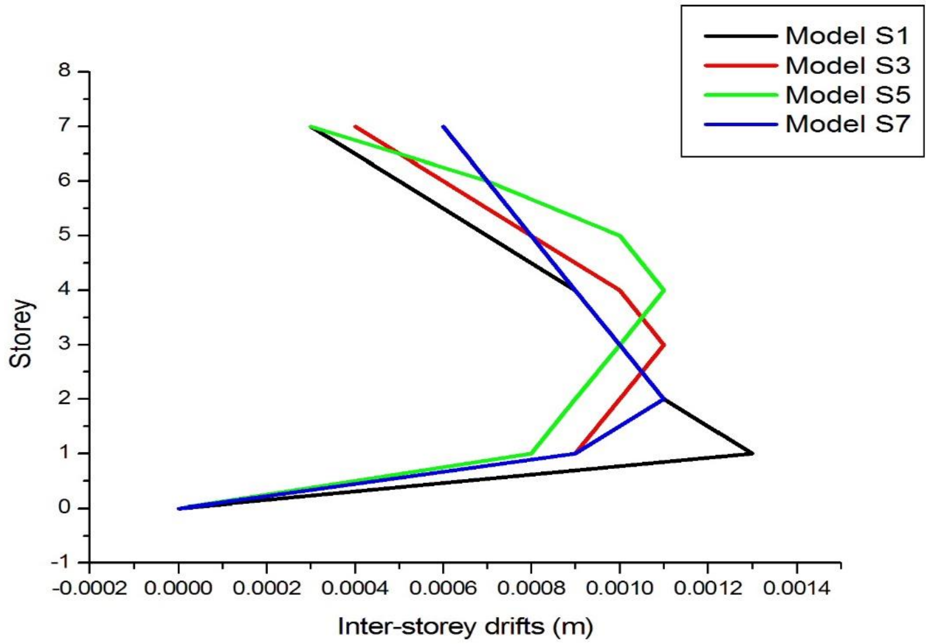


Figure III.37: The curves of inter-storey displacements in the direction X of the models.

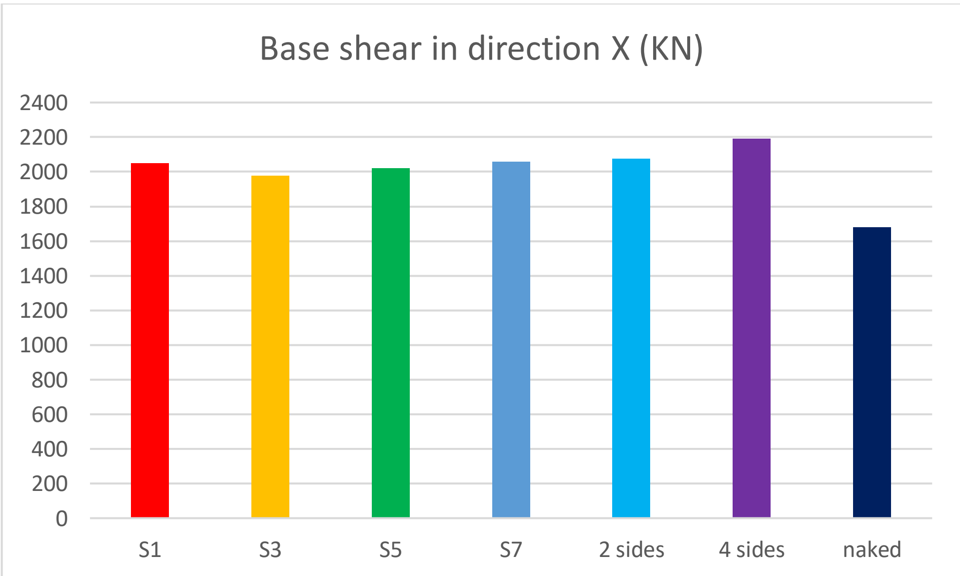


Figure III.38: Graph of the base shear in the direction X of the models.

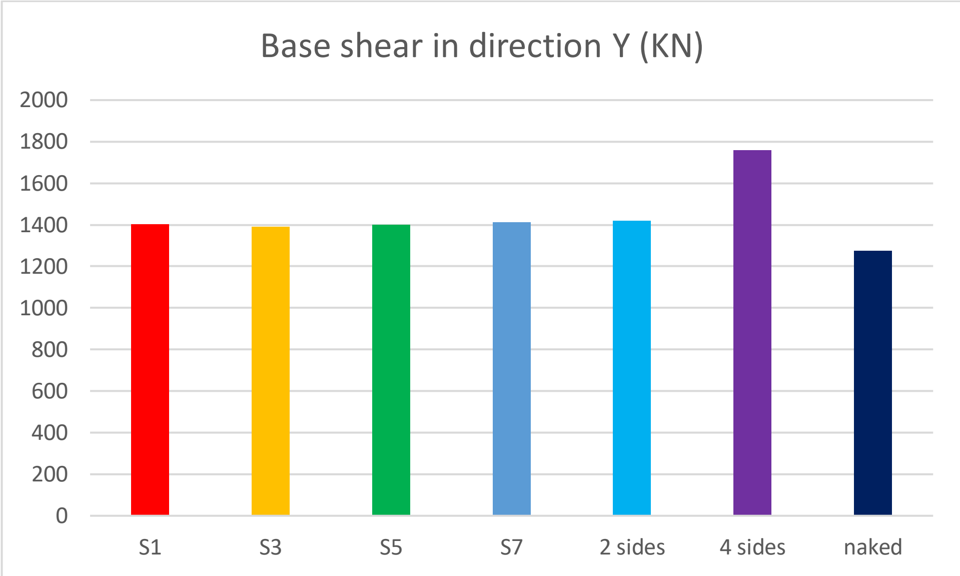


Figure III.39: Graph of the base shear in the direction Y of the modes.

III.11. CONCLUSION:

In this chapter concerning the effect of filling walls on the behavior of reinforced concrete frame structures in seismic zones.

Seven structural models were studied to confirm the effect of the filler walls based on the principle of the compression diagonal that replaces the wall, and the following relevant conclusions should be emphasized:

- In the linear analysis of the structures (structure without filling, structure with filling at both sides and structure with filling at the four sides) we recorded three remarks:

- We noticed that the existance of struts decreases inter-storey displacements.
- We note that the existance of struts significantly reduces the periods of vibration.
- The forces at the base of the frames with filling have increased significantly compared to the frame without filling; this was expected anyway because the struts increase the stiffness of the structure.

- Then, comparing the results of the linear analysis with those obtained from the nonlinear analysis of the structures (structure without filling, structure with filling at both sides and structure with filling at the four sides) we notice that:

- the increase of the shear force at the base in the nonlinear static study compared to the linear study. Thus, we show the effect of the nonlinear analysis allowing working the materials to the economic limits.

- Concerning the analysis of models (1, 2, 3 and 4):
  - we notice that the displacement has increased in the soft storey.
  - The soft storey at the base is more dangerous than the soft storeys at the top.

- In the end this study we gave possibility to see the interest of the filling walls in the frames and also the distribution of the walls in the building. It will be interesting in the future to quantify the gains in terms of strength and stiffness of the use of the filling walls and even the effect of the openings (soft storeys) on the compression diagonal.

General Conclusion

The object of the present work was to study the effect of masonry infill walls on the global behavior of RC frame structures; this was done by analyzing and evaluating an exemplary structure under the effect of a response specter in order to determine its capacity and performance point. For this, two main steps were made; the first was dedicated for a theoretical detailed research, where a big importance was given to the different procedures of the Pushover analysis and their equations of prediction, which models the degradation degrees of the whole structure. Furthermore, a brief research also was made on diagonal struts, where it based on giving an overview on them and determining their geometrical and material characteristics, and defining the hinge properties.

The second step focused on validating the theoretical aspect, where a comparative case of study was carried out for different models configurations of the same structure, one with struts, the second without struts and the remained models have soft storeys located in a different level each time. From the results of the linear modal analysis, it has been observed that masonry infill lowers the period of vibration of the structure compared to the naked structures .Calculation by Pushover method was adopted too in this study in order to show the efficiency of this analysis in predicting the comportment of RC frame structures with masonry infill. Nonlinear static analysis « Pushover » executed on the tridimensional structures have showed the significant influence of the existence of masonry infill walls on the dynamic characteristics, strength, the overall stiffness, energy dissipation and so on the seismic performance of buildings. Structures with diagonal struts give more realistic results and more representative than those of frame-masonry interaction given by the Algerian seismic code (RPA99/version 2003). Finally, from a personal point of view, the work that was held has given me the opportunity to improve and deepen my knowledge in the field of RC structures, and particularly over masonry infill walls, their modeling and characteristics.

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