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SEISMIC FINITE ELEMENT ANALYSIS OF AN EXISTING OLD CONCRETE STRUCTURE BY USING MULTIFIBER BEAMS: INTRODUCTION OF AN ADAPTIVE PUSHOVER METHOD

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Abstract. *The study of the seismic vulnerability of existing structures is an important issue. Many researches have been developed in order to investigate the structural behavior of these structures, and extract the basic informations needed to establish retrofitting guidelines in order to reduce the seismic risk to acceptable levels. The most accurate analysis procedure for the structures subjected to strong ground motions is the time-history analysis. This method is time-consuming though for application in all practical purposes. The necessity for faster methods that would ensure a reliable structural assessment or design of structures subjected to seismic loading led to the pushover analysis. Pushover analysis is a non-linear static analysis based on the assumption that structures oscillate predominantly in the first mode or in the lower modes of vibration during a seismic event. The present work deals with seismic vulnerability assessment of an old existing reinforced concrete structure – Perret tower – located in Grenoble, France. After a brief description of the structure in exam, a preliminary computation of the mass of the building and the definition of every existing section are performed. A simplified 3D numerical model is carried out using a finite element code based on multifiber beams approach. Firstly, a non-linear temporal dynamic analysis is performed, then a conventional and adaptive pushover analysis is carried out. The results obtained of the studied cases are then compared: it is observed that the conventional pushover analysis should be adjusted in order to take into account the change of dynamic characteristics due to the formation of plastic mechanisms. Finally, the tower critical levels in term of damage are highlighted.*

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1 INTRODUCTION

The seismic vulnerability assessment of existing structures and buildings is an important issue nowadays. In general terms, vulnerability expresses the propensity of a system of elements exposed to hazards to suffer damage. Many researches have been developed in order to investigate the behavior of these structures, and extract the basic informations needed to establish retrofitting guidelines in order to reduce the risks to acceptable levels. Numerous types of structures can be subjected to seismic risks and require a particular attention, such as buildings, towers, bridges, dams and nuclear power plants. In this matter, the seismic loading is due to well known "ground shaking" that can lead to a significant damage and/or collapse of the structure.

In seismic regions, a majority of old structures have been only designed to withstand gravity loads or to sustain seismic risks according to outdated seismic codes. These structures were designed and built without considering adequately earthquakes provisions constituting therefore an important source of risks. Recently, major earthquakes around the world have clearly proved that these old structures require an appropriate analysis to predict accurately their seismic behavior and evaluate their vulnerability. This allows proposing guidelines for the retrofitting procedure which is necessary for reducing seismic damage to allowable levels.

The process of analysis and design of a new structure is quite different from the case of an existing old structure. The design procedure objective is not the same in both cases indeed. When designing a new structure, the behavior is considered as ductile as the brittle failure modes are avoided. However, the failure modes characterizing an existing structure are mainly considered as brittle. This difference must be taken into account during the analysis phase.

The prediction of seismic behavior of existing structures has attracted some attention in the last decades [1, 2, 9, 11]. Several codes such as the Japanese Standard for Evaluation of Seismic Capacity of Existing Reinforced Concrete buildings [7] and Eurocode 8 [6] have addressed strengthening and rehabilitation of existing structures.

The numerical modeling represents a quite powerful and economic tool to assess accurately the seismic vulnerability of structures. The main objective of this paper is to present the numerical seismic analysis of the Perret tower located in Grenoble, France. Firstly, the description of the tower structure will be shown. Then it will be followed by a linear and non-linear analysis by mean of a 3D simplified finite element modeling using multifiber beams. Thirdly, the results of the non-linear temporal dynamic analysis will be compared with those obtained by carrying out a non-linear static pushover analysis. In this matter, an adaptive pushover method will be then described in order to improve the numerical behavior accuracy. And finally, conclusions and perspectives will be discussed.

2 PRESENTATION OF PERRET TOWER

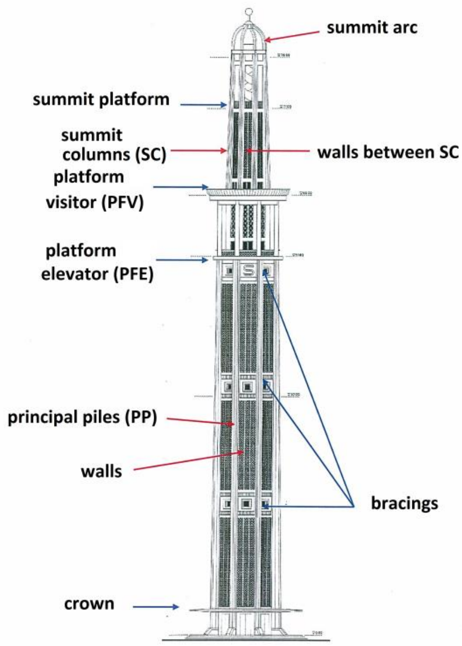
Perret tower is a reinforced concrete structure constructed between 1924 and 1925, currently located in the park Mistral in Grenoble, France. This tower, which is the first tall reinforced concrete building in Europe, with a height of 83 m approximately, was specifically built by the architect Auguste Perret. It has been completed in 1925 for the “Houille Blanche” and tourism exhibition in downtown Grenoble, France. Its main purpose was to be a panoramic observation point and was opened for public until 1960. It consists of a framework mainly made up of 8 piles joined together by three ring-shaped shear walls at 3 different levels (Figure 1). The screen walls are made of perforated concrete. The different constitutive levels of the tower are the following:

- A crown located at 3 m above the ground;

- At level 51.8 m, there is a platform elevator above it rises a second tower consisting of three terraces:
  - A platform for visitors at 60 m
  - A terrace at 71 m (summit platform); Figure 3.1 section at the base
  - terrace at 78 m (2 summit arcs and a ball)

Elevators and helical stairs give access to the platform, and another stair leads to the top of the tower. The tower is lying on a shallow foundation supported by 72 deep inclusions. At the level of the platform for visitors, a discontinuity in the section of the tower appears, where the columns above this level rest on a small cantilever beam. Figure 3.a shows clearly the details of this zone. Moreover, the columns are tapered and have a section quite similar to T shape (Figure 3.b).

(a)



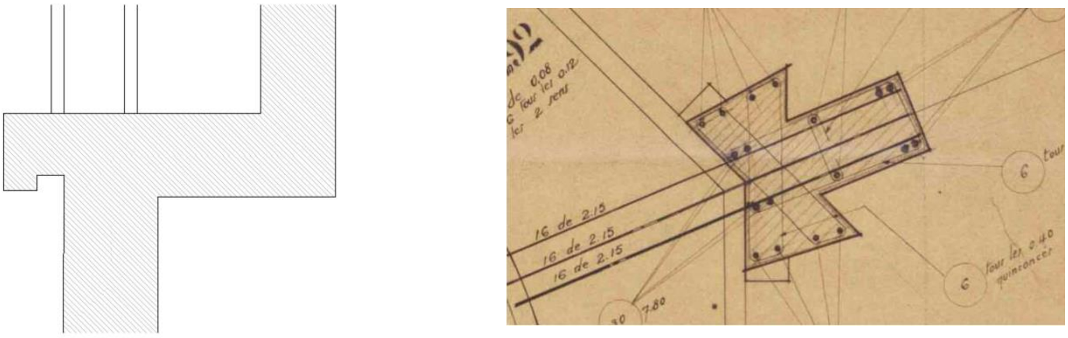
(b)

Figure 1: (a) Overview of the Perret tower in Grenoble, (b) the detailed structure's description

(a)

(b)

Figure 2: (a) Interior view showing the helicoidal stair and the elevators, (b) Screen walls



(a) (b)

Figure 3: (a) Interior view showing the helicoidal stair and the elevators, (b) Screen walls

3 NUMERICAL MODELING

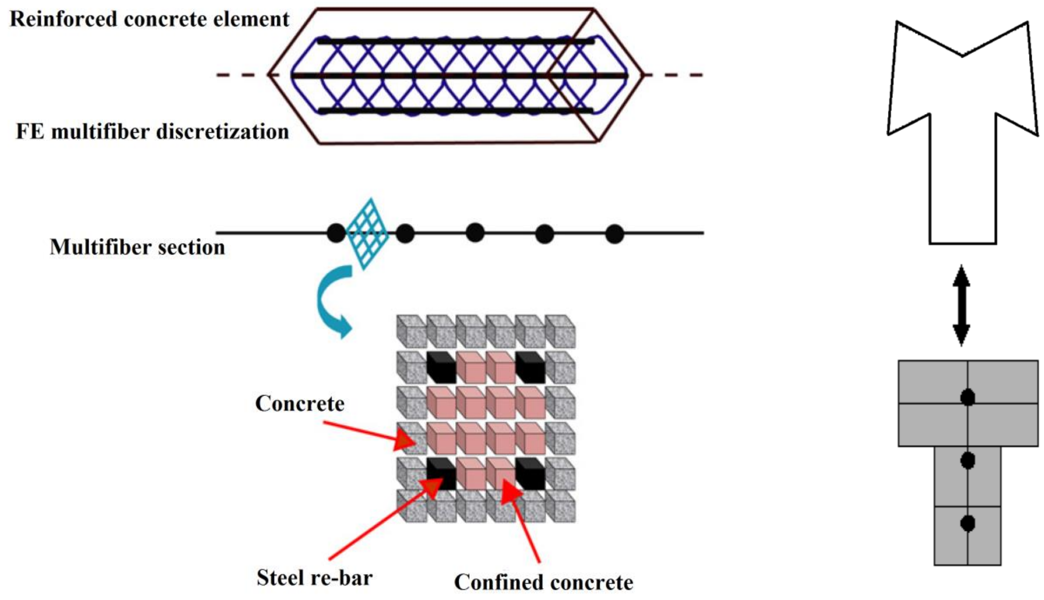
The structure is composed of eight piles confined together by three ring shaped shear walls which keep them together as unique structure. So the structure can be considered as a monolithic vertical beam. At the opposite, the screen walls seem to be only decorative as they are weakly linked to the columns. In this matter, the junctions contain very few corroded steel rebars. Thus, these elements do not participate to the structural stiffness. At the same time, there is an important concrete degradation of the spiral stairs which bring only mass to the tower and therefore their stiffness is negligible.

A numerical modeling is proposed hereafter to try reproducing the linear behavior of the tower as a first step which will be followed by a non-linear analysis. The eight piles will be considered as belonging to the same section of one beam clamped at the base and SSI was not taken into account. The total mass of the tower (1677tons) has been modeled by mean of concentrated masses distributed over 23 sections along the height of the tower. Numerical computations were performed with multifiber Timoshenko beam elements [10, 13], introduced in the finite-element code FEDEASLab (a MATLAB toolbox) [8]. This finite element strategy allows reproducing in a simplified manner the cyclic behavior of concrete as this numerical approach is suitable for slender structures. 1D non-linear constitutive behavior laws can be used for modeling concrete and steel fibers. The details of the numerical modeling data and results will be treated in the following paragraphs.

3.1 FE mulitifiber beam modeling

This approach is used to simplify the modeling of a structure versus a full 3D approach. Each element of the structure (columns, beams...) is decomposed into several beam elements with a node at each end. The section of a multifiber beam element is decomposed into several parallel fibers to the axis of the beam (Figure 4.a). For each fiber, a uniaxial constitutive law can be applied; this allows representing different materials in one section. The beam can follow Bernoulli or Timoshenko kinematics then each behavior law requires only a uniaxial writing.

The section of each column is discretized into 8 fibers of concrete and 3 fibers of steel (equivalent to the real section of steel), so at all the section of the beam contains 64 fibers of concrete and 24 fibers of steel (Figure 4.b). The simple model is equivalent to a simple beam fixed at the base in the ground whose section and stiffness vary with height. The model is discretized along the height into 22 elements and 23 nodes.



(a) (b)

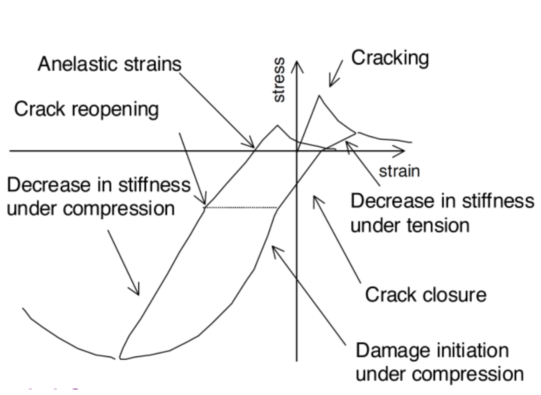
Figure 4: (a) Multifiber beam modeling, (b) Piles section discretization

3.2 Constitutive models for concrete and steel materials under cyclic loadings

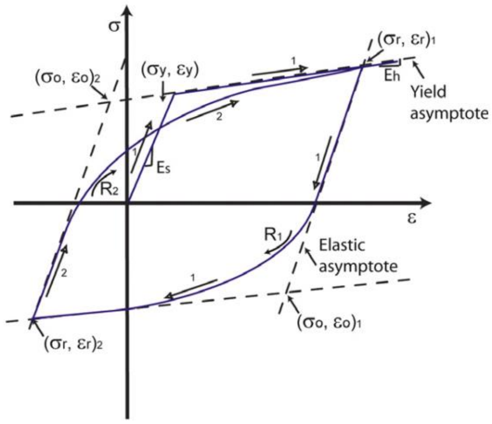
The major feature of the used material models is to describe each material only by one uniaxial law, which constitutes a significant simplification compared to a 2D or 3D description. The mechanical characteristics of the concrete have been determined by mean of experimental testing performed on specimen extracted from the tower. However, steel properties were not available at the time of the study so they were fixed on conservative values.

The behavior of concrete is described by the unilateral La Borderie model [12]. This local model based on damage mechanics follows the thermodynamics of irreversible processes and allows to take into account phenomena on opening and closure of cracks under cyclic loading for concrete (Figure 5.a). Quantification of damage distinguishing tension and compression through the corresponding two damage variables is a very interesting feature for the seismic analysis of a structure. The results obtained with this model in previous numerical simulations of structure under dynamic loading have also shown its good performance [5]. The model's general formulation is tridimensional (3D), but only the uniaxial (1D) version is used herein. The mechanical properties are fixed considering a compressive strength of 25 MPa measured in situ, tensile strength of 2 MPa calculated based on EC8, Young's modulus of 30 GPa and a poisson's ratio of 0.17.

The behavior of steel is represented by the modified model of Menegotto-Pinto [14] (Fig. 5.b). This uniaxial cyclic law reproduces a kinematic hardening and the buckling of the bars in compression when the transverse ones or stirrups are not sufficiently brought closer. Due to the lack of data about the steel properties used in the past when the tower was built, we supposed that the steel is smooth round of construction whose yield stress is supposed 235 MPa, Young's modulus about 190 GPa and a poisson's ratio of 0.3.



(a)



(b)

Figure 5: (a) La Borderie's 1D cyclic model, (b) Menegotto–Pinto's 1D cyclic model

3.3 Modal analysis using experimental data from ambient noise

The tower has been monitored by nine accelerometric stations (Figure 6). Four have been placed at the base of the structure, four additional at the level of the visitor platform and another one at the top of the tower. This in-situ modal analysis campaign has been performed on the Perret Tower in 2011 by the French company Miage Sarl by mean of the ambient noise technique [4]. The experimental results are provided into the table 1.

Modes	**EW direction f (Hz)**	**NS direction f (Hz)**
1st longitudinal	0.81	0.75
2nd longitudinal	2.88	2.56
3rd longitudinal	4.88	4.69

Table 1: Experimental modal analysis of the Perret tower: resonance frequencies in the different directions using the FDD method with ambient noise.

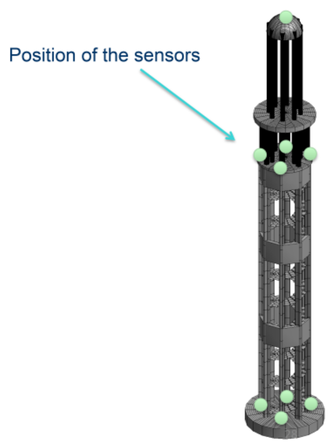


Figure 6: Positions of the accelerometric stations used in the ambient noise in-situ measurements

3.4 Numerical modal analysis

The results of the modal analysis carried out by taking into account only the structural parts of the tower (eight columns and three ring-shaped shear walls) are provided in table 2. The first three modes (Figure 7) correspond to bending effect. The results show that the numerical fundamental mode frequency is congruent with the value measured in-situ. However, the second and third modal frequencies obtained numerically are higher than in-situ values. This can be due to the numerical section rigidity which is higher than the reality due to the assumptions of the infinitely stiff three ring-shaped shear walls. By calculating the modal participation factor of the first 4 modes, we found that the fundamental mode is not predominant by large percentage (57.1%) as in the case of regular structures, and at the same time the higher modes have an important modal participation. The sum of modal participation factors for the first 4 modes is about 90%. This is due firstly to the non-uniform repartition of masses along the height of the structure which is heavier at the base than at higher levels, and secondly to the infinitely stiff braces in the model which promote the formation of bends and consequently activating the higher modes.

Modes	Numerical f (Hz)
1st longitudinal	0.78
2nd longitudinal	3.44
3rd longitudinal	7.81

Table 2: Numerical modal frequencies

A sensitivity analysis has been carried out by [3] in order to identify the most accurate numerical model to be used for the Perret tower. It allowed to better understand the contribution to the global stiffness given by the “secondary” structural components and mainly the screen walls. The numerical results showed that taking into account the screen walls allows to match the in-situ analysis results, with high accuracy. Although, the severe conditions of degradation of these decorative elements authorize to hypothesize, for them, a low-strength and fragile behavior under true seismic loading

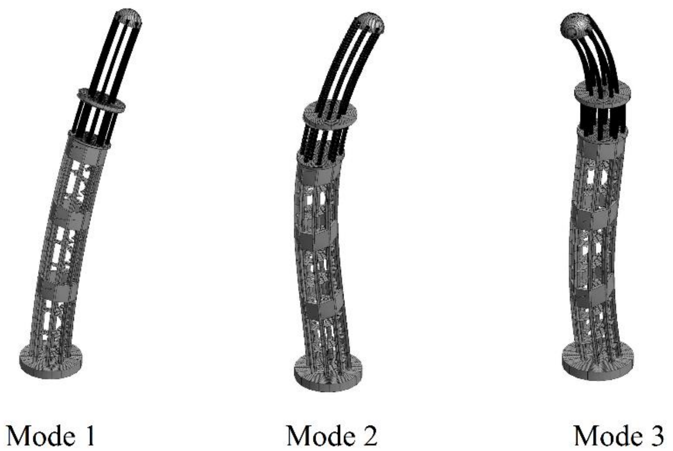


Figure 7: Representation of the three first fundamental modes of the Perret tower

3.5 Non-linear temporal dynamic analysis

The main idea of the study is to simulate the effects of a possible earthquake on the tower. Eurocode 8 spectrum has been used to calibrate the accelerograms assumed acting in 3 directions (Figure 8). Three difference sets of accelerograms have been used to have a better idea of the tower response. These signals have been artificially generated by the ISTerre (Institut des sciences de la terre) in Grenoble.

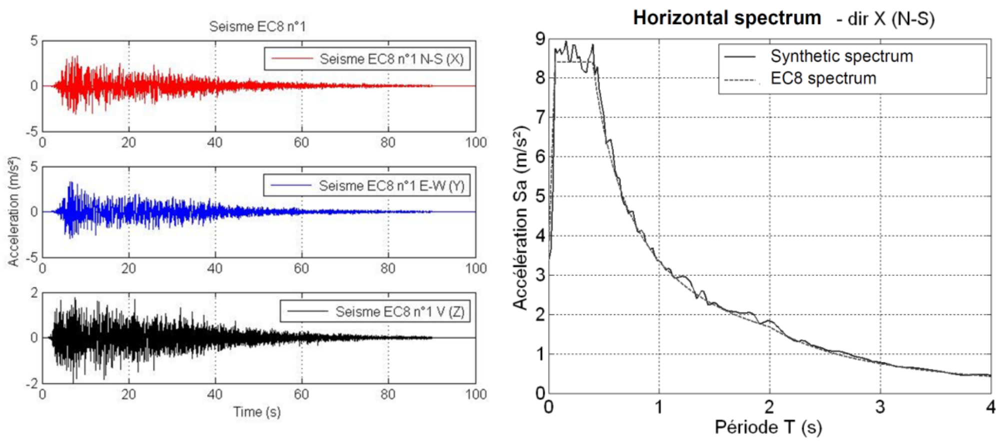


Figure 8: Set of accelerograms and spectrum of the X ones compared to the EC8 spectrum required

The amplitude of horizontal displacements of the numerical model of the Perret tower subjected to the EC8 earthquake in the EW direction reaches 20 cm while vertical displacements are negligible (< 6 mm) (Figure 9). In terms of solicitations at the base of the structure, the maximum shear force is 3087 KN and the maximum bending moment is 94390 KN.m.

Under the effect of the tri-directional earthquake the behaviour of the structure becomes nonlinear. Figures 10,11 and 12 show the results of numerical analysis in terms of damage and plastic strain in the steel rebars. Specifically, the behaviour of structure is mainly of flexion type, without adverse effects of torsion. This confirms that multi-fibre Timoshenko beams can be used.

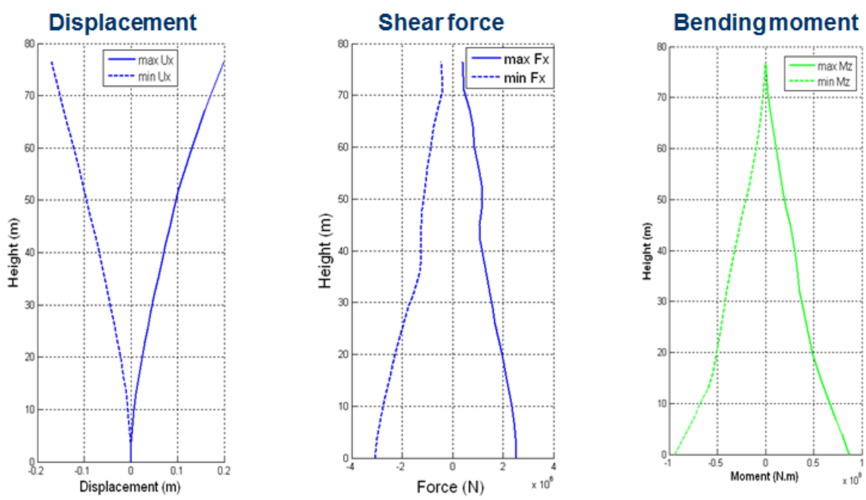


Figure 9: Non-linear temporal dynamic analysis results

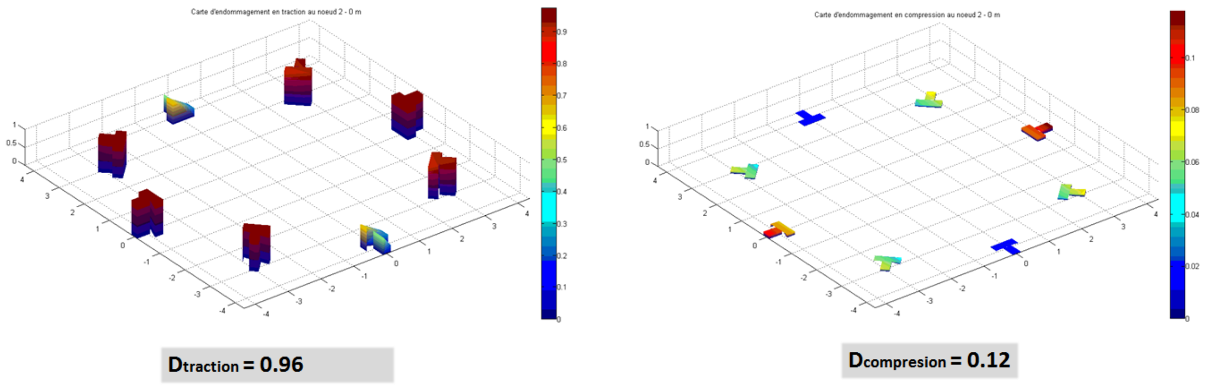


Figure 10: Tensile and compressive damages at 0 m level

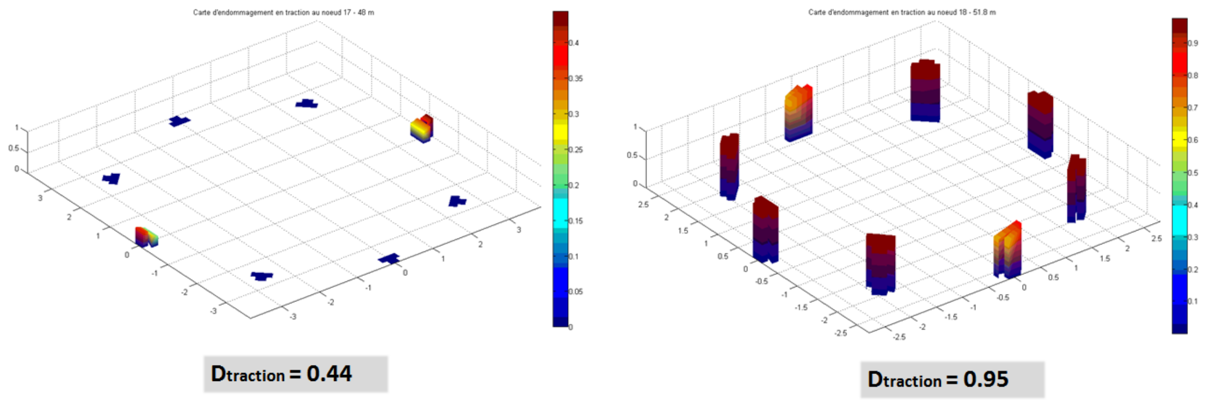


Figure 11: Tensile damage at 50.9 m (left) and 51.8 m (right) levels

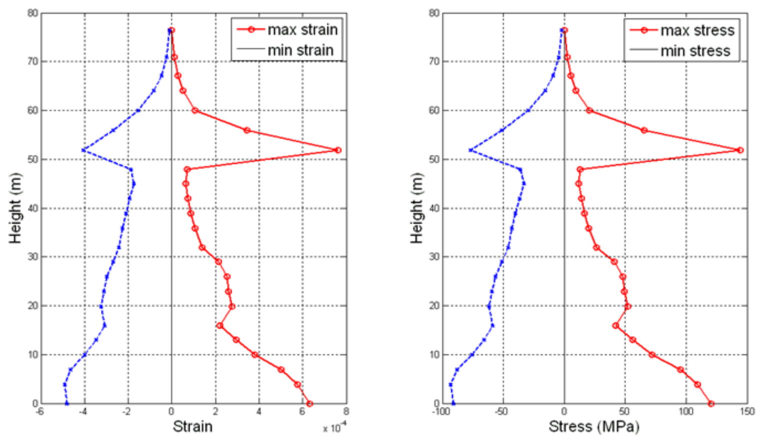


Figure 12: Repartition of maximum and minimum stresses and strains of rebars along the height of the tower

The objective of this work is the numerical evaluation of structural disorders which will be used to quantify the level of global damage of the structure. In other term, the damage is defined as irreversible degradations resulting from a dynamic stress modifying the later behavior of the structure. Within the framework of a reinforced concrete structure, such as Perret tower, whose dynamic behavior is expressed primarily in bending, the concept of damage relates to the cracking of the concrete in traction, spalling of the concrete in compression and the plasticization of the steel reinforcements, the three aspects being closely interrelated. With the numerical tools used, we have indicators of damage (damage variable in the sense of damage mechanics).

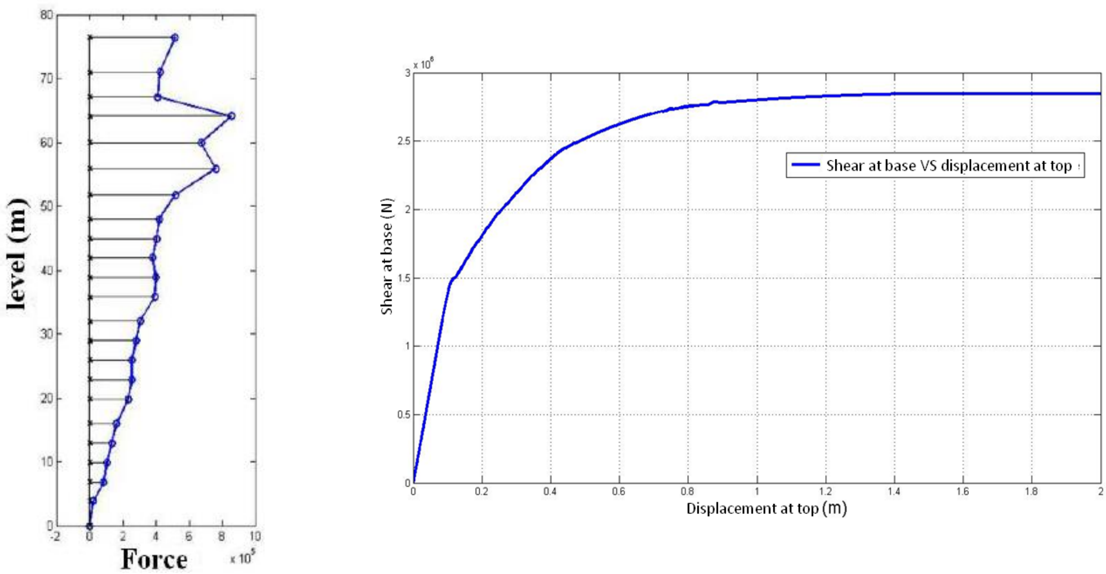
The damage in traction of concrete leads to the dissipation of energy, which can influence the behavior of the structure. Thus, the study of this damage brings an indication on the dynamic operation of the structure. A chart of damage makes it possible to visualize the damaged zones corresponding to a strong localization of the deformations "cracking of concrete". Those concentrate at the base of the tower and at the level of the discontinuity of section at 51.8 m. The maximum damage in traction is about: 0.96 at the base (Figure 10), 0.44 at 50.9 m just before the discontinuity (Figure 11) and 0.95 just after the discontinuity (Figure 11). We conclude that the critical zone at 51 m is strongly damaged in traction.

The main risk of the damage of the concrete in compression is related to spalling which induces the ruin by buckling of the steel reinforcements. However, this kind of risk is not observed on the structure of Perret tower as the maximum compressive is at the base and its value (0.12) is low (Figure 10).

By looking to Figure 12, we find that there is no plastification of steel rebars. The maximum stress and strain values are localized at the level of the section's discontinuity (51m) where the maximum stress is 145 MPa which is less than the yielding stress value (235 MPa), as for the maximum strain 0.76‰ which is less than the value of yielding stress (1.2‰). Therefore, the steel behavior remains in the elastic domain with no yielding,

3.6 Pushover analysis

The purpose of the pushover analysis is to assess the structural performance by estimating the strength and deformation capacities using static, nonlinear analysis and comparing these capacities with the demands at the corresponding performance levels. Traditionally, conventional pushover method has been used and implemented in design codes.



(a) (b)

Figure 13: Pushover analysis for the fundamental mode of Perret tower (a) lateral force distribution (b) capacity curve

Traditional pushover analysis is based on the assumption that the dynamic response of the MDOF (Multi-Degree-Of-Freedom) system is determined by a single mode only and that the shape of that mode is constant, throughout the time-history, regardless of the level of deformation. The response of the structure is supposed controlled by the first mode of vibration and mode shape, or by the first few modes of vibration. The structure is subjected to a lateral load that is equivalent approximately to the relative inertia forces generated at the locations of substantial masses such as floor levels (Figure 13.a). The outcome of the analysis is the global force-displacement curve or capacity curve of the structure (Figure 13.b). This capacity curve provides valuable information about the response of the structure because it approximates how it will behave after exceeding its elastic limit.

The plateau of the capacity curve is reached for an asymptote corresponding to a value of shear at base about 2800 KN. This plateau corresponds to the mechanism of plastification which appears in the structure by several aspects like cracking of the concrete, plastification of steel and apparition of plastic hinges.

Secondly, the capacity curve is transformed to the ADRS (Acceleration Displacement Response Spectrum) (Figure 14), and an iteration process must be carried to find the performance point of the structure which corresponds to the equality between the capacity and demand ductility of the structure

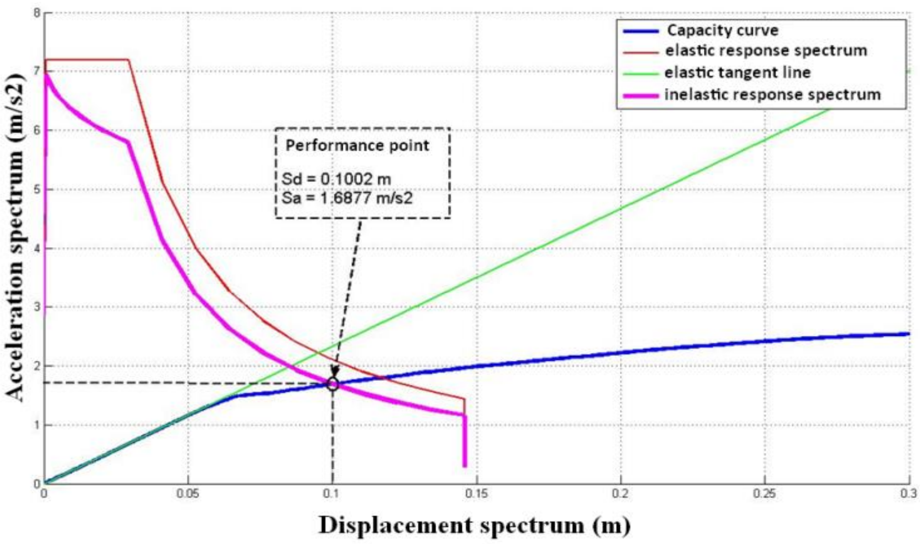


Figure 14: Pushover analysis of Perret tower: Finding the performance point of the structure

Table 3 shows the Pushover analysis results. By comparing the obtained values with those obtained in temporal dynamic analysis, we find that the results are not accurate especially in terms of shear force where the error is important (≈ 46 %). That means that carrying out pushover analysis for the fundamental mode only is not sufficient to provide accurate results and good estimation of dynamic behavior by an alternative static pushover approach. It can be explained also by the fact that the modal participation factor of the first mode is not high sufficiently (57 %) to govern the dynamic response of the tower. In other terms, higher modes are activated during the dynamic process and need to be taken into account during pushover analysis. In the following, a non-linear multimodal pushover analysis will be presented.

Displacement at top (cm)	Shear force at base (KN)	Bending moment at base (kN.m)
16.22	1661	80332

Table 3: Pushover mode 1 – values of top displacement and solicitations at the base of the tower

3.7 Non-linear multimodal pushover analysis

An improved non-linear multimodal pushover analysis method is presented to incorporate contributions of multiple modes and the effects of their interactions to the responses. The key step for the method is to obtain and equivalent seismic force for the important vibration modes. After launching pushover analysis for each mode apart, the results of different analysis must be combined to take into account the interactions between the responses. The combination is based on the SRSS method (Square Root of the Sum of the Square):

$$u_i^{SRSS} = \sqrt{\sum_{j=1}^{n} u_{i,j\ max}^2} \tag{1}$$

This formula can be applied for all variables such as displacements, forces and moments. The obtained results are shown in table 4.

	Displacement at top (cm)	Shear force at base (kN)	Bending moment at base (kN.m)
Combining 2 modes	17.04	1877	80353
Combining 3 modes	17.22	2219	83930
Combining 4 modes	17.23	2305	84000

Table 4: variation of displacement at top and solicitations at base with the number of modes included in the multimodal pushover analysis

Table 4 shows that by increasing the number of modes combined in multimodal pushover analysis, the values of displacement at top and solicitations at the base become closer to those obtained by the non-linear temporal dynamic analysis. However, the error still important in terms of base. By comparing the values of displacements, we found that 17.23 cm (value obtained by multimodal pushover analysis) represents about 85% of the value obtained by dynamic analysis (20 cm). This value was determined by considering the combination of the first 4 modes which have a modal participation of 90%. This difference between values can be interpreted by the phenomenon of damage evolution of the structure. The structure damage increases with the evolution of dynamic process under seismic loading, which can influence the dynamic characteristics of the structure (modal frequencies, modal shapes).

3.8 Effect of damage on the dynamic characteristics of the structure

The dynamic analysis is a complex phenomenon including several mechanisms (yielding, opening and closure of cracks...). The damage of the concrete in traction increases with time under seismic effect. By investigating the evolution of the tensile damage of concrete fibers during the non-linear dynamic temporal simulation, we found that in a specified phase (between 5s and 7.5s) (Figure 15) the damage of concrete fibers increases by an important rate. After carrying out modal analysis at each instant of dynamic process, we draw the evolution of modal frequencies of the first 4 modes (Figure 15), and we conclude that at the same phase there is a falling of the values of frequencies. In this matter, at each instant, we created a numerical model of the tower and we considered that the new value of Young's modulus of is affected by the tensile damage value D_1:

$$E_{damaged} = (1 - D_1)E_{initial} \tag{2}$$

Figure 15 shows clearly that the modal frequencies decreased by a percentage of 35% during the evolution of dynamic process, that's prove the effect of damage of concrete on the modal frequencies of the studied structure.

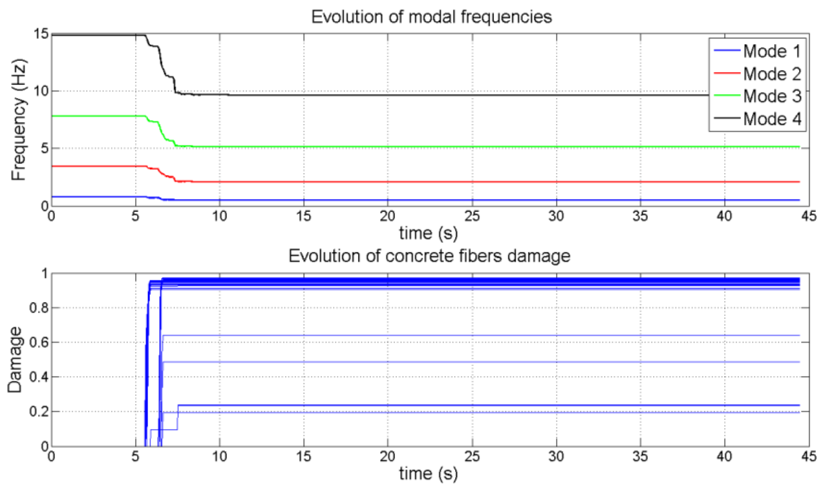


Figure 15: Evolution of damage in traction of concrete and of the modal frequencies during the non-linear dynamic simulation

3.9 Adaptive pushover analysis of Perret tower

Although conventional pushover analysis provides crucial information on response parameters of the structure, this method is not exempt from some limitations such as the inability to include progressive stiffness degradation. Therefore, the need for an adaptive procedure that overcomes the noted deficiency. The adaptive pushover analysis procedures are mostly concerned with an appropriate estimation of the force vector that is going to 'push' the structure at each static force increment. The monitoring of the change in the incremental force vector could ensure that the stiffness degradation or strength deterioration of the structure is counted to be more realistic than conventional nonlinear static analysis. Once the new lateral force distribution has been determined, the remaining steps of the adaptive pushover analysis follow those of the conventional.

After executing multimodal pushover analysis (for the first five modes) and taking into account the new distribution of lateral forces by considering the modal shapes of the damaged structure, we found the new values of displacement at top and solicitations at base (Table 5). We could conlude that the value of displacement at top became closer to those obtained by the temporal dynamic analysis. But in terms of shear force and bending moment at base of the tower, the differences between the corresponding dynamic and static pushover results are still important. This can be explained by the inability of adaptive pushover analysis used in the analysis to take into account the formation of plastic mechanism in the structure during dynamic process under seismic loading.

	Dynamic analysis	**Adaptive pushover**
Displacement at top (cm)	20	18.76
Shear force at base (kN)	3087	2490
Bending moment at base (kN.m)	94390	85616

Table 5: Comparison of the results of dynamic and static adaptive pushover analysis

4 CONCLUSIONS

The main objectives of this research were (i) to investigate numerically the seismic vulnerability of the Perret tower by mean of simplified 3D model and compare the linear behavior results with those obtained in-situ, (ii) to explore the critical structural zones of the tower which are the most affected by an earthquake, (iii) to discuss and access different methodologies for dynamic behavior of the tower like non-linear temporal dynamic analysis and pushover methodology, (iv) to explore the deficiencies presented in the conventional pushover method which is a practical engineering tool, and (v) to evaluate the level of damage of the tower under an earthquake.

The simplified 3D modeling used gives a good estimation of the behavior of the Perret tower in terms of solicitations and displacement. Under seismic loading, the concrete is almost cracked at the base and the level of section discontinuity (51m) with no yielding of steel which supports the efforts elastically.

Pushover analysis can provide good results as a static alternative of dynamic analysis. In terms of displacements and solicitations, the obtained results are still little bit far from the ones obtained by non-linear temporal dynamic analysis. It can provide an insight into the structural aspects which control performance during severe earthquakes, and data on the strength and ductility of the structure which cannot be obtained in an elastic analysis.

Furthermore, the non-linear static procedure based on pushover analysis is restricted with a single or few modes response. Then it is valid for low-rise buildings where the behavior is dominated by fundamental vibration mode. It is required to take into account of higher modes effects in pushover analysis of tall buildings. Moreover, conventional pushover procedure is based on a very restrictive assumption, i.e. invariant lateral force distribution without considering progressive stiffness degradation.

Therefore, the future research should continue developing the adaptive pushover analysis that overcomes the deficiencies above cited. It should take into account the structural weakness which may be generated when the structure's dynamic characteristics change after the formation of the first local plastic mechanism. The redistribution of inertia forces due to structural yielding should be investigated also in the adaptive procedure.

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