NATIONAL TECHNICAL UNIVERSITY OF ATHENS

School of Civil Engineering

Institute of Steel Structures

INVESTIGATION OF THE SEISMIC BEHAVIOR OF A NEWLY CONSTRUCTED 7-STORY REINFORCED CONCRETE BUILDING WITH A ROOF WATER TANK

MASTER THESIS

MSc in Analysis and Design of Earthquake Resistant Structures

DIMITRIOS MOURLAS

EMK ME 2023/5

Supervised by

D. VAMVATSIKOS, Associate Professor NTUA

Athens, July 2023

Mourlas Dimitrios (2023).

Investigation of the seismic behavior of a newly constructed 7-story reinforced concrete building with a roof water tank

Master Thesis "EMK ME 2023/5"

Institute of Steel Structures, National Technical University of Athens, Greece

Contents

Abstract

A range of seismic assessment methods is employed to evaluate the seismic behavior of a newly constructed building and to estimate its seismic losses. For this scope, a 7-story building with a basement constructed in Kallithea, Greece in 2022 was studied. In addition, we studied the seismic behavior of an added roof water tank, this being the typical practice for several areas outside of Athens with low or inadequate water pressure. As such non-structural components are rarely designed by structural engineers, the aim is to investigate whether there will be damage due to the (typically amplified) roof top acceleration.

The Rapid Visual Inspection method was used in order to classify the building in a priority category for further checking. For the second level rapid seismic assessment, the methods proposed by Dritsos S., Vougioukas E. and the new FEMA P-2018 guidelines were applied. In addition, nonlinear static analyses were performed based on the Greek code (KANEPE), Eurocode 8-Part 3, and ASCE/SEI 41-17. The Rapid Visual Inspection method revealed that the building belongs to the category with low priority for further checking, as expected of a newly designed structure. Regarding the results of the second level rapid seismic assessment methods and the nonlinear static analyses, considerable scatter is observed, with KANEPE being the most conservative approach.

Furthermore, the analysis of the roof water tank showed that it failed at a peak ground acceleration PGA of 0.067g, or 40% of the design PGA, corresponding to an earthquake with a 35-year return period rather than the needed 475 years. Therefore, this water tank has a high probability of collapse during the lifetime of the building (50 years), potentially causing severe water damage to the roof, stairwell, and floor contents.

Moreover, the fragility curves of the building were estimated using the SPO2FRAG software for both KANEPE and EC8, further showing the conservatism of KANEPE. Finally, the repair cost for various levels of building damage was estimated using the PACT software developed by FEMA P-58. The results showed that in all cases, the critical elements are generally the infills, the walls, and the roof water tank, although the average annual loss is low, reaching only 0.66% (KANEPE) to 0.16% (EC8) of the total building replacement cost.

Acknowledgments

As I complete this thesis, I feel the need to thank everyone who has contributed to it and especially my supervisor, Associate Professor Dimitrios Vamvatsikos, for the continuous support that he offered, his patience, motivation, enthusiasm and immense knowledge. Thanks to the valuable knowledge he imparted to me, this thesis was a valuable teaching experience.

Finally, I must express my very profound gratitude to my parents, Vasileios and Maria for providing me with unfailing support and continuous encouragement throughout my years of study and through the process of researching and writing this thesis. Thank you!

List of Figures

➢ Chapter 1

➢ Chapter 2

➢ Chapter 3

➢ Chapter 4

➢ Appendix B

List of Tables

➢ Chapter 1

➢ Chapter 2

➢ Chapter 3

➢ Chapter 4

➢ Appendix A

➢ Appendix C

➢ Appendix D

Chapter 1

Introduction

1.1. General

The new Eurocode[1] made the structures much safer as they now focus on resisting deformation rather than forces. This was achieved by having dense stirrups in columns and walls (confinement), strong column–weak beam mechanism, stirrups in the column-beam joint, etc. Moreover, most new structures have many walls to resist shear forces, unlike the old ones, where only columns appear, which increases the probability of having a “soft story” mechanism.

However, all this refer to structures with regular configurations without many torsional effects, while most of the buildings have many irregularities. Moreover, the design code[1] takes into account a design earthquake with a 10% probability of exceedance (poe) in 50 years by applying spectral analysis, which is a static method, while the earthquake is a dynamic phenomenon.

If we take these things into account, it is clear that it is worthwhile to examine a new structure to see if there will be problems. Some pre-earthquake assessment methods (first and second level) are used to quickly verify how a structure will behave in a future earthquake before performing nonlinear analyses with a software. In this work, we compared many rapid methods (Greek, European, and American) to see the differences and draw appropriate conclusions. Subsequently, the software Seismobuild[2] was used to apply the third level pre-earthquake assessment method, using the Greek Code of Intervention (KANEPE)[3], the EN 1998-3[4], and the American ASCE/SEI 41-17[5]. To perform this procedure, a static nonlinear analysis (pushover) was used to find the target displacement according to each code and performance level. In addition, the water tank behavior was studied finding the horizontal force acting in this structure (according to EN 1998-1[6]), which is located high above the ground, to investigate whether it will collapse in a future earthquake. The seismic hazard was then assessed using the classical PSHA (Probabilistic Seismic Hazard Analysis)[7], determining the uniform hazard spectrum from EFEHR[8] and comparing the results with the EC8 spectrum. Finally, the fragility curves of the building were calculated with the software SPO2FRAG[9] and the annual repair losses of the structural and non-structural components for different damage states were determined from the results obtained using the FEMA P-58[10] method with PACT[11] software.

This chapter briefly describes the rapid assessment methods to better understand the results that are presented next.

1.2. First Level Pre-Earthquake Assessment Method

Pre-earthquake assessment methods aim to assess the structural vulnerability of the building against the maximum probable expected seismic actions in the area where they are located. The first level pre-earthquake assessment method or Rapid Visual Inspection[12] aims to assess the seismic capacity of the building through data collected and completed in a special Inspection Report. The fields for filling in the form are designed to estimate a first indicator of seismic capacity. A significant factor in ensuring the reliability of the data collected is to find and use the building design before carrying out the check. In addition to completing the inspection report, the engineers who have inspected the building shall draw the floor plan and a typical section on an A4 sheet of paper and attach a photograph of the building's facade. The fields to be completed on the inspection report are divided into eight (8) sections:

- Section A: Enter the location of the building (address, municipality, district and coordinates), the owner's details, its use and the maximum number of persons accommodated in it and who is carrying out the check.
- Section B: Include the stories of the building, the square meters of the floor plan and the total built-up area, the year of construction, any interventions that have been carried out and the significance of the building according to the Greek code E.A.K 2000[13].
- Section C: General seismological and geotechnical data such as the seismic hazard zone according to the E.A.K 2000[13] and at the time of design of the building and the E.A.K 2000[13] soil category shall be completed.
- Section D: This section concerns the distinction of the structural type of the building according to the structural system (whether it is a reinforced concrete structure or a prefabricated building or a building with load-bearing masonry or a steel structure) and the date of construction, which leads directly to the applicable Design Codes.
- Section E: Contains the data concerning the general vulnerability of the building to earthquakes, such as: whether the design of the building was carried out without the application of Seismic Regulations, the change of use of the building, which implies a change of importance, the existence of previous seismic loads that have not been remedied by the design of construction, the poor condition due to lack of maintenance, the possibility of impact of large

buildings, the existence of a soft story, the irregular arrangement of the infills on each story, the great height of the building, its irregularity both horizontally and vertically, the possibility of significant torsional deformation and the existence of short columns.

- Section F: It concerns the elements of "additional" vulnerability, i.e., the cases of arbitrary elements that have not been studied and need to be assessed during the final rating and classification of the building.
- Section G: Includes additional causes affecting the final vulnerability of the structure, such as change of use of the building, unsuitable soil due to subsidence, sliding, fire and flooding.
- Section H: The final rating and classification of the building is carried out, considering all the above sections.

After completing the sections, the calculation of the structural rating follows according to Table 1.2 after having selected the table corresponding to the structural type of the building. The table used for the needs of this work includes the reinforced concrete structural types, which are presented in detail in Table 1.1 below. The final score of the building is obtained by summing up the values in Table 1.2. Therefore, the building is classified as one of the priority categories of inspection according to Table 1.3.

Table 1.1 Structural type of buildings

Structural Type	**Description of Structural Type**	**Regulations**
RCa	Reinforced Concrete Buildings	Greek Anti-Seismic Regulation 1959[14] / Concrete Regulation 1954[15] / Pre 1985 Buildings
RCb	Reinforced Concrete Buildings	GASR 1959[14] + extra articles 1985[16] / Concrete Regulation 1954[15] / Buildings between 1986 and 1995
RCc	Reinforced Concrete Buildings	E.A.K 2000[13] / E.K.O.S 2000[17] / Buildings after 1995

Table 1.2 Final building rating[12]

Parameter	**Reinforced Concrete**		
	RCa	**RCb**	**RCc**
Basic rating, depending on the Structural Type	6.0	7.0	8.0
Seismic Zone I	-0.5	-0.1	-0.5
Seismic Zone II or III	-1.5	-1.5	-1.0
Ground Type B (for A: -0.1)	-0.3	-0.3	-0.3
Ground Type C or D	-0.6	-0.6	-0.6
Ground Type C or D and building $>$ 5 stories	-0.8	-0.8	-0.8
Ground Type X	-0.8	-0.8	-0.8
Without Anti-Seismic Regulation	-0.5	-	-
Previous seismic loads, problems	-1.0	-0.5	-0.5
Bad condition	-0.5	-0.5	-0.5
Impact with adjacent buildings	-0.5	-0.5	-
Pilotis and/or Short Columns	-1.5	-1.5	-0.5
Irregular infill arrangement in plan	0.5	0.5	-
High height	-1.0	-0.5	-0.5
Irregularity in elevation	-1.0	-0.5	-0.5
Irregularity in plan	-1.0	-0.5	-0.5
Torsion (intense)	-0.5	-0.5	-0.5
Operating intensity	0.2 or 0.5	0.2 or 0.5	0.2 or 0.5
Number of users $\leq$ 9	-0.2	-0.2	-0.2
Number of users 10 - 99	-0.4	-0.4	-0.4
Number of users $\geq$ 100	-0.6	-0.6	-0.6

Table 1.3 Priority categories of buildings[12]

Rating	Category
F.R. $\leq$ 4.0	High Priority
4.0 $<$ F.R. $<$ 5.5	Middle Priority
F.R. $\geq$ 5.5	Low Priority

1.3. Second Level Pre-Earthquake Assessment Method

1.3.1. Dritsos S. Method

This method[18] is an approximate procedure for assessing the seismic capacity and adequacy of existing reinforced concrete buildings with respect to seismic requirements as defined in modern code provisions.

When applying this method[18], it is necessary to find any existing documentation on the construction of the building, in particular the structural study, any studies of subsequent interventions and corresponding drawings of the building's formwork. Once the study is available, the seismic capacity of the building is assessed and evaluated based on the verification of critical geometric elements (section dimensions, reinforcements, etc.), some non-destructive tests of the building materials (e.g., concrete strength) and some simple approximate numerical calculations (e.g., base shear). If this documentation is not possible, then an imprint of the load-bearing structure and the infills is required. Regarding the quality of the materials, only the determination of the concrete strength is required, optionally using random checks, mainly on the vertical elements, alternatively by taking the “in absentia” representative values of the KANEPE code[3]. Thus, a general picture of the condition of the building under inspection is created.

The purpose of this rapid method for assessing the seismic capacity of reinforced concrete buildings is to determine the approximate “Priority Index λ”, provided that the seismic demand of the structure and the corresponding seismic resistance at the base of the building have been calculated, taking into account the additional seismic criteria that affect its load-bearing capacity. This index denotes (in an approximate way) the degree of inadequacy for the specific building in terms of structural capacity and consequently the order of priority for the third level of assessment, i.e., the preparation of valuation studies and redesign (reinforcement) of a limited number of buildings according to the budget capacity of the relevant body. In the present methodology, thirteen (13) criteria were considered[18], which describe vulnerability factors that have a decisive influence on the seismic behavior of the building (Table 1.4). The criteria are graded with a whole number β_i from one (1) to five (5) [except the first three (3) criteria, where it begins with zero (0)], with one (1) being the highest level of influence equivalent to a reduction in the building's seismic performance and five (5) being the lowest. The score assigned to each criterion results in a reduction coefficient β in the shear strength at the base of the building.

The final step is to determine the seismic category (K) of the building, which is practically the maximum assessment target that the building can achieve for performance level “Significant Damage”.

Table 1.4 Seismic vulnerability criteria that affect the load-bearing capacity of the building

Order Number	**Criterion**	**Grade β_i**						**Weight Factor σ_i**
		0	1	2	3	4	5	
1	Static failure damage							0.10
2	Reinforcement oxidation							0.10
3	Relative axial force on ground floor							0.05
4	Regularity in plan	-						0.05
5	Stiffness distribution in plan - Torsion	-						0.10
6	Regularity in elevation	-						0.05
7	Stiffness distribution over the height of the building	-						0.15
8	Mass distribution over the height of the building	-						0.05
9	Short columns	-						0.15
10	Vertical discontinuities	-						0.05
11	Path and Transfer of forces in the building	-						0.05
12	Adjacent buildings	-						0.05
13	Malfunctions - Injuries	-						0.05

More specifically, in order to determine the “Priority Index λ” and the seismic category (K) of the building, the following steps need to be applied:

- **Calculation of Seismic Demand (V_{req})**

Seismic demand is the total magnitude of seismic loads applied to a building and is given by the following equation:

$$V_{req} = MS_d(T) \tag{1.1}$$

where:

M = building total mass based on the static loads for the combination $G+\psi_2 \cdot Q$, with $\psi_2 = 0.30$

T = building eigenperiod based on Greek Code of Interventions (KANEPE[3]) (for more details see Appendix A[18])

$S_d(T)$ = spectral design acceleration (Appendix A[18])

q = behavior factor according to Table A.3 of the Appendix A[18] (to be entered in the Design Spectrum)

- **Calculation of Seismic Resistance (V_R)**

The seismic resistance is given in terms of the base shear resulting from the geometric properties and the action forces of the columns at the base of the building in combination with the degradation due to the seismic vulnerability criteria (Table 1.4). The equations used are the following:

$$V_R = \beta V_{R0} \tag{1.2}$$

$$\beta = \sum_{1}^{13} \frac{\sigma_i \beta_i}{5} \tag{1.3}$$

$$V_{R0} = \alpha_1 \sum V_{Ri}^{Col} + \alpha_2 \sum V_{Ri}^{Wall} + \alpha_3 \sum V_{Ri}^{Short\ Col} + \sum V_{Ri}^{Infills} \tag{1.4}$$

where:

β = reduction coefficient defined in Eq. (1.3), where β_i is the criterion grade and σ_i a weight factor taken from Table 1.4

V_{R0} = approximate total shear strength at the base of the building without taking into account the seismic vulnerability criteria (Table 1.4) and using some reduction factors (Table 1.5)

V_{Ri} = shear strength of each vertical element (column, wall, short column) at the base of the building derived from KANEPE[3]

$V_{Ri}^{Infills}$ = shear strength of infills at the base of the building (not considered in this work because there are not infills at the ground floor)

Table 1.5 Shear strength reduction factors[18]

Type of Vertical Elements	α_1	α_2	α_3
Columns + Walls + Short Columns	0.50	0.70	0.85
Columns + Walls	0.70	0.85	-
Columns + Short Columns	0.70	-	0.85
Columns	0.85		

A short column is defined as a column with $l/h \leq 5$ where:

l = clear height of column

h = depth of the cross-section in the direction of flexure of the plastic hinge

Moreover, a wall is defined when $L_w/b_w \geq 4$ where:

L_w = length

b_w = width

Nevertheless, the presence of short columns is considered when the grade β_i (according to criterion 9) (Table 1.4) is smaller than 3.0. In addition, the presence of walls in each direction is considered when the degree $a_T > 0.10$. According to this method[18], a_T is given by the equation:

$$a_T = \frac{\sum V_{Ri}^{Wall}}{V_{R0}} \tag{1.5}$$

where:

ΣV_{Ri}^{Wall} = total shear strength of all walls at the base of the building in the corresponding direction

V_{R0} = approximate total shear strength at the base of the building as described above

To conclude, for instance, if $\beta_i \geq 3.0$ (criterion 9) and there are short columns at the base of the building ($l/h \leq 5$) and if $a_T \leq 0.10$ and there are walls at the base of the building ($L_w/b_w \geq 4$), then only one reduction factor (Table 1.5) will be used ($\alpha_1 = 0.85$) for all the elements.

Analytically, the calculation of the shear strength of a vertical element (column, wall, short column) is based on the relations of KANEPE[3] and is given by the equation:

$$V_{Ri} = \min\left(V_{Rd}, V_M\right) \tag{1.6}$$

where:

V_{Rd} = shear strength of vertical element based on the relations presented in Appendix 7C of KANEPE[3]

V_M = shear force corresponding to flexural strength of vertical element equal to:

$$V_M = \frac{M_y}{L_s} \tag{1.7}$$

where:

M_y = flexural strength of vertical element based on the relations presented in Appendix 7A of KANEPE[3]

L_s = shear length of vertical element equal to:

$$L_s = \frac{L_k}{2}, \quad L_k = \begin{Bmatrix} h_{clear} & \rightarrow \text{columns} \\ h_w & \rightarrow \text{walls} \end{Bmatrix} \tag{1.8}$$

where:

h_{clear} = clear height of the column on the story under consideration

h_w = distance of the wall base cross-section on the story under consideration from the top of the building

More specifically, the shear strength V_{Rd} of a vertical element according to KANEPE[3] (Appendix 7C) is given by the following equation [units in MN, m and MPa]:

$$\begin{aligned} V_{Rd} = \frac{h-x}{2L_s} \cdot \min\left(N, 0.55 A_c f_c\right) + \left(1 - 0.05 \min\left(5, \mu_\theta^{pl}\right)\right) \times \\ \left[0.16 \max\left(0.5, 100\rho_{tot}\right)\left(1 - 0.16 \min\left(5, \alpha_s\right)\right)\sqrt{f_c} A_c + V_w\right] \end{aligned} \tag{1.9}$$

where:

h = depth of the cross-section in the direction of flexure of the plastic hinge

$x = \xi_y d \rightarrow$ height of the compression zone at yielding (ξ_y = relative compression height ay yielding, d = effective depth of the cross-section)

L_s = shear length as described in Eq. (1.8)

N = axial force of vertical member based on the static loads for the combination $G+\psi_2 \cdot Q$ (positive for compression, zero for tension), with $\psi_2 = 0.30$

A_c = cross-section area of vertical member equal to:

- $b_w d$ (Orthogonal Sections), where d = effective depth of the cross-section
- $\pi D_c^2/4$ (Circular Sections), where D_c = core diameter with the stirrups

α_s = shear ratio of vertical member (L_s/h)

f_c = compressive strength of concrete

μ_θ^{pl} = plastic ductility factor at flexural failure equal to $[\theta_u/\theta_y - 1]$ (θ_u = ultimate chord rotation, θ_y = chord rotation at yielding) with values ranging from 0.5 – 5.0 (new structures [after 2000] are close to or higher than 5.0 while old ones [before 1985] are close to 0.5 or even 0.0)

ρ_{tot} = reinforcement ratio of total longitudinal bars (tension, compression, middle)

V_w = contribution of the transverse reinforcement to the shear strength of vertical member equal to:

$$\begin{aligned} V_w &= \rho_w b_w z f_{yw} \rightarrow \text{Orthogonal Sections} \\ V_w &= \frac{\pi}{2}\frac{A_{sw}}{s} f_{yw}\left(D-2c\right) \rightarrow \text{Circular Sections} \end{aligned} \tag{1.10}$$

where:

ρ_w = reinforcement ratio of shear bars (stirrups) in the direction under consideration

z = lever arm of internal forces equal to d-d′ (columns) and 0.8h (walls)

f_{yw} = yield strength of shear reinforcement

A_{sw} = cross-section area of the circular stirrup

s = distance between stirrups

c = concrete cover

For the calculation of relative compression zone at yielding, the following relation is used:

$$\xi_y = \left(\alpha^2 A^2 + 2\alpha B\right)^{1/2} - \alpha A \tag{1.11}$$

where:

$\alpha = E_s/E_c$, Young's modulus ratio of steel to concrete

A, B = parameters depending on which yields first, reinforcement or concrete, so there are two (2) cases:

1) Reinforcement yielding

$$A = \rho + \rho' + \rho_v + \frac{N}{bdf_y}, \qquad B = \rho + \rho'\delta' + 0.5\rho_v\left(1+\delta'\right) + \frac{N}{bdf_y} \tag{1.12}$$

2) Concrete yielding

$$A = \rho + \rho' + \rho_v - \frac{N}{1.8\alpha bdf_c}, \qquad B = \rho + \rho'\delta' + 0.5\rho_v\left(1+\delta'\right) \tag{1.13}$$

where:

ρ, ρ', ρ_v = reinforcement ratio of longitudinal bars (tension, compression, middle) [in that order], with [$\rho_{tot} = \rho+\rho'+\rho_v$]

b = width of the cross-section

$\delta' = d'/d$, where d' = distance from the center of the compression reinforcement to the end of compressive concrete fiber and d = effective depth of the cross-section

N = axial force as described above

$\alpha = E_s/E_c$, Young's modulus ratio of steel to concrete

f_y, f_c = yield strength of longitudinal reinforcement and compressive strength of concrete, respectively

Thus, after ξ_y has been calculated for both two (2) cases, the next step is to determine the yield curvature $(1/r)_y$. The expression is the following:

1) Reinforcement yielding

$$(1/r)_y = \frac{f_y}{E_s\left(1-\xi_y\right)d} \tag{1.14}$$

2) Concrete yielding

$$(1/r)_y = \frac{1.8 f_c}{E_c \xi_y d} \tag{1.15}$$

where:

f_y, f_c = yield strength of longitudinal reinforcement and compressive strength of concrete, respectively

E_s, E_c = Young's modulus of steel and concrete, respectively

d = effective depth of the cross-section

ξ_y = relative height of compression zone as described above in Eq. (1.11) for both cases

The final result for the yield curvature $(1/r)_y$ is the lower value from Eq. (1.14) and Eq. (1.15) and for the value of ξ_y, the one corresponding to the final curvature is selected.

As described above, in order to calculate μ_θ^{pl}, θ_u and θ_y must first be examined. According to §7.2.2 of KANEPE[3] the chord rotation at yielding is given by the following equation [f_c and f_y in MPa]:

$$\theta_y = (1/r)_y \frac{L_s + a_v z}{3} + 0.0014\left(1 + 1.5\frac{h}{L_s}\right) + \frac{(1/r)_y d_b f_y}{8\sqrt{f_c}} \rightarrow \text{Beams and Columns} \quad (1.16)$$

$$\theta_y = (1/r)_y \frac{L_s + a_v z}{3} + 0.0013 + \frac{(1/r)_y d_b f_y}{8\sqrt{f_c}} \rightarrow \text{Walls} \quad (1.17)$$

where:

L_s = shear length as described in Eq. (1.8)

z = lever arm of internal forces equal to d-d′ (columns) and 0.8h (walls)

$(1/r)_y$ = yield curvature taken as the lower value from Eq. (1.14) and Eq. (1.15)

h = height of the vertical element

d_b = mean diameter of tension reinforcement

f_y, f_c = yield strength of longitudinal reinforcement and compressive strength of concrete, respectively

α_v = parameter with value 0 or 1, depending on whether shear cracking V_{Rc} is smaller or higher than the shear corresponding to flexural yielding V_M (Eq. (1.7)). So, according to this:

$$a_v = \begin{Bmatrix} 0 \rightarrow V_{Rc} > V_M \\ 1 \rightarrow V_{Rc} \leq V_M \end{Bmatrix} \quad (1.18)$$

The shear cracking V_{Rc} according to §7.2.2 of KANEPE[3] is given by the equation [f_c in MPa]:

$$V_{Rc} = \left\{ \max\left[180(100\rho_L)^{1/3}, 35\sqrt{1+\sqrt{\frac{0.2}{d}}} f_c^{1/6} \right] \left(1 + \sqrt{\frac{0.2}{d}}\right) f_c^{1/3} + 0.15\sigma_c \right\} b_w d \quad (1.19)$$

where:

b_w = width of the cross-section

d = effective depth of the cross-section

f_c = compressive strength of concrete

ρ_L = reinforcement ratio of tension longitudinal bars

$\sigma_c = N/A_c \leq 0.2f_c$, where N = axial force of the vertical member and A_c = cross-section area of the vertical member as described above

Regarding the calculation of V_M (Eq. (1.7)), flexural strength M_y must first be examined. According to KANEPE[3] (Appendix 7A), the equation used is the following:

$$M_y = (1/r)_y \left\{ E_c \frac{\xi_y^2}{2}\left(0.5(1+\delta') - \frac{\xi_y}{3}\right) + \left[\left(1-\xi_y\right)\rho + \left(\xi_y - \delta'\right)\rho' + \frac{\rho_v}{6}\left(1-\delta'\right)\right]\left(1-\delta'\right)\frac{E_s}{2}\right\} \times b_w d^3 \tag{1.20}$$

where:

ρ, ρ', ρ_v = reinforcement ratio of longitudinal bars (tension, compression, middle) [in that order], with [$\rho_{tot} = \rho+\rho'+\rho_v$]

$(1/r)_y$ = yield curvature taken as the lower value from Eq. (1.14) and Eq. (1.15)

ξ_y = relative height of compression zone

$\delta' = d'/d$, where d' = distance from the center of the compression reinforcement to the end of compressive concrete fiber and d = effective depth of the cross-section

E_s, E_c = Young's modulus of steel and concrete, respectively

Having computed the chord rotation at yielding θ_y, the next step is the calculation of ultimate chord rotation θ_u to determine the ductility factor. According to §7.2.4 of KANEPE[3], the ultimate chord rotation is given by the following equation [f_c in MPa]:

$$\theta_u = 0.016\left(0.3\right)^{\nu}\left[\frac{\max\left(0.01,\omega'\right)}{\max\left(0.01,\omega-\omega'\right)} f_c\right]^{0.225} \left(\alpha_s\right)^{0.35} 25^{\left(a\rho_s \frac{f_{yw}}{fc}\right)}\left(1.25^{100\rho_d}\right) \rightarrow \text{Columns} \tag{1.21}$$

$$\theta_u = 0.625 \cdot 0.016\left(0.3\right)^{\nu}\left[\frac{\max\left(0.01,\omega'\right)}{\max\left(0.01,\omega-\omega'\right)} f_c\right]^{0.225} \left(\alpha_s\right)^{0.35} 25^{\left(a\rho_s \frac{f_{yw}}{fc}\right)}\left(1.25^{100\rho_d}\right) \rightarrow \text{Walls} \tag{1.22}$$

where:

α_s = shear ratio of vertical member (L_s/h)

ν = relative axial force ($\nu = N/bhf_c$)

ρ_s = reinforcement ratio of shear bars (stirrups) in the direction under consideration

ρ_d = reinforcement ratio of diagonal bars if existed ω, ω΄ = mechanical reinforcement ratio of total longitudinal bars and compression longitudinal bars, respectively

f_c = compressive strength of concrete

α = confinement effectiveness factor equal to:

$$\alpha = \left(1-\frac{s_h}{2b_o}\right)\left(1-\frac{s_h}{2h_o}\right)\left(1-\frac{\sum b_i^2}{6h_o b_o}\right) \tag{1.23}$$

where:

s_h = distance between stirrups

b_o, h_o = dimension of confined core to the centerline of hoops

b_i = centerline distance between longitudinal bars laterally restrained by a stirrup corner or a cross-tie along the perimeter of the cross-section

The Eq. (1.21) and Eq. (1.22) is used for buildings constructed after 1985. For buildings before 1985 but with continuous deformed (high bond) bars, these equations must be divided by 1.20.

In walls, the shear strength V_{Rd} as described in Eq. (1.9) must not exceed $V_{R,max}$, which is the maximum shear force that can be sustained by the member, limited by crushing of the compression struts. Additionally, in columns where a_s (shear ratio) is less than 2.0, V_{Rd} must be checked again not exceeding the $V_{R,max}$. The relationships for calculating $V_{R,max}$ for columns and walls are the following [units in MN, m and MPa]:

$$V_{R,\max} = 0.85\left(1-0.06\min\left(5,\mu_\theta^{pl}\right)\right)\left(1+1.8\min\left(0.15,\frac{N}{A_c f_c}\right)\right)\left(1+0.25\max\left(1.75,100\rho_{tot}\right)\right)\times$$
$$\left(1-0.2\min\left(2,a_s\right)\right)\sqrt{f_c}\,b_w z \rightarrow \text{Walls} \tag{1.24}$$

$$V_{R,\max} = \frac{4}{7}\left(1-0.02\min\left(5,\mu_\theta^{pl}\right)\right)\left(1+1.35\frac{N}{A_c f_c}\right)\left(1+0.45\left(100\rho_{tot}\right)\right)\sqrt{\min\left(40,f_c\right)}\times$$
$$b_w z\sin\left(2\delta\right) \rightarrow \text{Columns} \tag{1.25}$$

where:

N = axial force as described above in the calculation of V_{Rd} (Eq. (1.9))

A_c = cross-section area of vertical member as described above

a_s = shear ratio of vertical member (L_s/h) as described above

f_c = compressive strength of concrete

μ_θ^{pl} = plastic ductility factor at flexural failure

ρ_{tot} = reinforcement ratio of total longitudinal bars (tension, compression, middle)

z = lever arm of internal forces equal to d-d′ (columns) and 0.8h (walls)

δ = compression strut angle ($\tan \delta = h/2L_s = 0.5/a_s$)

Furthermore, in walls, consideration shall also be given to the possibility of sliding at the base or at any other cross-sections where the longitudinal reinforcement is likely to yield. The shear resistance in sliding $V_{R,SLS}$ can be obtained according to KANEPE[3] (Appendix 7C) from the expression:

$$V_{R,SLS} = V_i + V_f + V_d \tag{1.26}$$

where:

V_i = contribution of inclined bars to the shear strength

V_f = contribution of friction to the shear strength

V_d = dowel action of the vertical reinforcement

The relations used to determine the shear resistance in sliding are the following:

$$V_i = \sum A_{si} f_{yi} \cos(\varphi) \tag{1.27}$$

$$V_f = \min\left(\mu\left[\left(\sum A_{sv} f_{yv} + N\right)\xi + \frac{M_y}{z}\right], 0.3 f_c A_{compr}\right) \tag{1.28}$$

$$V_d = 1.6 \sum A_{sv} \sqrt{f_c f_{yv}} \leq \sum \frac{A_{sv} f_{yv}}{\sqrt{3}} \tag{1.29}$$

where:

ΣA_{si} = sum of cross-sections areas of all the inclined bars in both directions

φ = angle of the inclined bars with respect to the plane of potential slip

ΣA_{sv} = sum of cross-sections areas of the longitudinal bars in the wall web

f_{yi}, f_{yv} = yield strength of inclined bars and yield strength of longitudinal reinforcement in the wall web, respectively

f_c = compressive strength of concrete as described above

μ = friction coefficient with recommended value 0.80

N = axial force as described above in the calculation of V_{Rd} (Eq. (1.9))

M_y = flexural strength of vertical element based on the relations presented in Appendix 7A of KANEPE[3] (Eq. (1.20))

z = lever arm of internal forces equal to d-d′ (columns) and 0.8h (walls)

A_{compr} = the area of the compression zone (equal to bξd in walls of rectangular cross-section)

ξ = relative height of compression zone equal to:

$$\xi = \xi\left(\mu_\theta\right) = \xi_y - \frac{\xi_y - \xi_u}{\mu_\theta^{\,fl} - 1}\left(\mu_\theta - 1\right) \tag{1.30}$$

where:

μ_θ = ductility factor

μ_θ^{pl} = ductility factor at flexural failure equal to θ_u/θ_y (θ_u = ultimate chord rotation, θ_y = chord rotation at yielding) [Eq. (1.17) and Eq. (1.22)]

ξ_y = relative height of compression zone at yielding as described above in the Eq. (1.11) (taking the value corresponding to the lower yield curvature $(1/r)_y$)

ξ_u = ultimate relative height of compression zone equal to:

$$\xi_u = \frac{\left(A_s f_y - A_s' f_y + N\right)\left(1 - \frac{d'}{d}\right) + A_{sv} f_{yv}\left(1 + \frac{d'}{d}\right)}{\left(1 - \frac{\varepsilon_{co}}{3\varepsilon_{cu}}\right) b\left(d - d'\right) f_c + 2A_{sv} f_{yv}}, \quad \text{if} \quad A_s' f_y - A_s f_y + \frac{A_{sv} f_{yv} d'}{\left(d - d'\right)}\left(d_s \frac{\varepsilon_{cu} + f_{yv}/E_s}{\varepsilon_{cu} - f_{yv}/E_s} - d\right) + \ldots$$

$$\ldots + \frac{\varepsilon_{cu} - \varepsilon_{co}/3}{\varepsilon_{cu} - f_{yv}/E_s} f_c b d' \le N$$

different:

$$\left[\left(1 - \frac{\varepsilon_{co}}{3\varepsilon_{cu}}\right) b d f_c + A_{sv} \frac{\left(E_s \varepsilon_{cu} + f_{yv}\right)^2 d}{2E_s \varepsilon_{cu}\left(d - d'\right)}\right]\xi_u^{\,2} - \left[N + A_s f_y - A_s' E_s \varepsilon_{cu} + A_{sv} \frac{f_{yv} d + E_s \varepsilon_{cu} d'}{d - d'}\right]\xi_u - \ldots$$

$$\ldots - \left[A_s - \frac{A_{sv} d'}{2\left(d - d'\right)}\right] E_s \varepsilon_{cu} \frac{d'}{d} = 0 \tag{1.31}$$

where:

A_s, A_s' = cross-section area of tension and compression reinforcement bars, respectively

A_{sv} = cross-section area of longitudinal bars in the wall web

E_s = Young's modulus of steel

d = effective depth of the cross-section

N = axial force as described above in the calculation of V_{Rd} (Eq. (1.9))

f_y, f_{yv}, f_c = yield strength of longitudinal reinforcement (tension + web) and compressive strength of concrete, respectively

d_s = mean diameter of longitudinal tension bars

ε_{co} = 0.002

ε_{cu} = ultimate strain of the extreme fiber of the compression zone taken as:

$$\varepsilon_{cu} = 0.004 + 0.4\frac{\alpha\rho_{sx}f_{yw}}{f_{cc}} \tag{1.32}$$

where:

f_{yw} = yield strength of shear reinforcement

ρ_{sx} = reinforcement ratio of shear bars (stirrups) in the direction under consideration

f_{cc} = compressive strength of confined concrete equal to:

$$f_{cc} = f_c\left[1 + 3.5\left(\frac{\alpha\rho_{sx}f_{yw}}{f_c}\right)^{3/4}\right] \tag{1.33}$$

α = confinement effectiveness factor as described above (Eq. (1.23))

- **Calculation of "Priority Index λ"**

According to this method[18], the calculation of "Priority Index λ" in both directions is given by the equation:

$$\lambda_x = \frac{V_{req,x} + 0.30V_{req,y}}{V_{R,x} + 0.30V_{R,y}}, \quad \lambda_y = \frac{V_{req,y} + 0.30V_{req,x}}{V_{R,y} + 0.30V_{R,x}} \tag{1.34}$$

where:

$V_{req,x}$, $V_{req,y}$ = seismic demand in both directions (Eq. (1.1))

$V_{R,x}$, $V_{R,y}$ = seismic resistance in both directions (Eq. (1.2))

The final “Priority Index λ” is taken as:

$$\lambda = 100\max\left(\lambda_x, \lambda_y\right) \tag{1.35}$$

- **Determination of Seismic Category (K) of the Building**

As previously mentioned, the final step in this method[18] is to determine the seismic category (K) of the building, which is practically the maximum assessment target that the building can achieve for performance level “Significant Damage”. This is done based on Table 1.6 after calculating the δ coefficient from the following equation:

$$\delta = \min\left(\frac{1}{\lambda_x}, \frac{1}{\lambda_y}\right) \tag{1.36}$$

where:

λ_x, λ_y = “Priority Index λ” in both directions as described above

Table 1.6 Building classification to seismic category (K)

Περίοδος Επαναφοράς (έτη)	Πιθανότητα υπέρβασης σεισμικής δράσης εντός του συμβατικού χρόνου ζωής των 50 ετών	δ	**ΣΕΙΣΜΙΚΗ ΚΑΤΗΓΟΡΙΑ (Κ)**
2475	2%	$1.80 \leq \delta$	**K0**
975	5%	$1.30 \leq \delta < 1.80$	**K1⁺**
475	10%	$1.00 \leq \delta < 1.30$	**K1**
225	20%	$0.75 \leq \delta < 1.00$	**K2⁺**
135	30%	$0.60 \leq \delta < 0.75$	**K2**
70	50%	$0.45 \leq \delta < 0.60$	**K3⁺**
40	70%	$0.35 \leq \delta < 0.45$	**K3**
20	90%	$0.25 \leq \delta < 0.35$	**K4⁺**
<20	>90%	$\delta < 0.25$	**K4**

So, for a newly constructed building, the category must be K1 and above, since the design earthquake according to Eurocode 2[1] has 10% probability of exceedance (poe) in 50 years.

1.3.2. Vougioukas E. Method

This is a rapid assessment method of the seismic capacity of existing buildings based solely on the Greek Code of Interventions (KANEPE[3]). More specifically, it mostly concerns buildings designed and built before 1985, which have strong beams and weak columns and where the “soft story” mechanism is usually applicable. The shear resistance of each vertical element is calculated on the ground floor using the equations of

KANEPE[3]. The seismic capacity of the building is obtained by adding the corresponding shear strength of all the elements together.

In order to determine the shear strength of each column, Eq. (1.6) is used where:

V_{Rd} = shear strength according to Eq. (1.9)

V_M = shear corresponding to flexural strength according to Eq. (1.7)

If $V_{Rd} \leq V_M$, this means that the element has brittle behavior and the corresponding flexural strength M_y (Eq. (1.20)) has to be reduced according to the ratio V_{Rd}/V_M.

1.3.3. FEMA P-2018 Method

According to American regulations, older concrete buildings not in compliance with the strength and detailing requirements of the 1976 or later editions of the Uniform Building Code[19] and not otherwise determined to have acceptable seismic performance through evaluation or retrofit to locally accepted standards, could be susceptible to significant structural damage in an earthquake and should be considered high seismic risk buildings. In contrast, buildings constructed after 1976 have lower relative risk of collapse, termed lower seismic risk buildings.

This evaluation methodology[20] is intended for use in determining the relative risk of collapse within an inventory of buildings, prioritizing buildings for further evaluation and guiding prudent risk-reduction policy decisions. It is not intended for use in developing or implementing seismic retrofits to improve the seismic performance, or reduce the risk of collapse, of an individual building.

This methodology is applicable to reinforced concrete buildings 160 feet (≅ 50 m) or less in height, with concrete diaphragms and with or without structural load-bearing walls, reinforced concrete shear walls, or masonry infill walls. Reinforced concrete elements that can be evaluated using this methodology include:

- frame lines (or bays) that are designed to resist gravity loads, including beam-column systems, slab-column systems, or joist-column systems
- frame lines (or bays) that are designed to resist gravity plus lateral loads, regardless of the level of ductile detailing
- frame lines with masonry infill walls of hollow clay tile, brick masonry, or concrete masonry units (CMU), with or without openings (note: infill masonry panels that have been previously retrofitted or reinforced are outside the scope of application of the infill procedures in this methodology)
- wall lines with structural reinforced concrete walls, generally 4 inches or greater thickness and with a horizontal reinforcement ratio equal or greater than 0.0015

- concrete elements retrofitted using materials and components that are compatible with concrete behavior and strength calculation procedures
- post-tensioned concrete systems with reinforced concrete columns. In such cases, procedures for determining column capacity as a function of the beam or slab capacity and any other calculation related to the capacity of beams and slabs, must be appropriately determined from ACI 318-14[21]

Additional requirements for use of this methodology include:

- a complete load path that includes diaphragms and adequate connections between lateral force-resisting elements, including adequate reinforcing at construction joints in walls and at column and wall connections to foundations
- beam-column joints with limited eccentricity, in which the centerline of the beam is located within the width of the column and at least some of the beam longitudinal reinforcement passes within the column core (as defined by the boundaries of the column longitudinal reinforcement). Up to 10% of the beam-column joints may exceed this requirement if it is judged that overall building response will not be adversely affected

The evaluation methodology is not applicable to precast frame or wall structures in which the capacity of a system or component is limited by its connection to other structural elements.

This method requires specific knowledge of the as-built configuration and condition of in-place materials and components. Documentation of the as-built configuration of a building is necessary for implementation of the evaluation methodology. Required as-built information includes: (a)building size and configuration, (b) structural component size, reinforcement and detailing, material properties and (c) site and foundation information. This information is best obtained from complete structural design drawings or as-built drawings. Other potential sources of information include construction specifications, geotechnical reports, structural calculations, and shop drawings.

In the case of older concrete buildings, sources of as-built information are often not available. Because it is potentially misleading to make assumptions regarding reinforcement details and member proportioning, it is recommended that any concrete building for which detailed structural drawings are not available should be considered an exceptionally high seismic risk building by default, unless structural details can be confirmed by other means (e.g., destructive and non-destructive site investigations).

A site investigation consists of visual observation of the condition of the building to verify that as-built information is representative of the existing conditions. The following should be confirmed as part of a site investigation: (a) building configuration, including the presence (or not) of structural additions, alterations, or modifications, (b) layout, proportioning and condition of structural components, (c) site characteristics, (d) foundation conditions and (e) where present, infill characteristics (e.g., infill panel locations, materials, thickness/number of wythes and size of openings).

Seismic hazard due to ground shaking is defined as an acceleration response spectrum based on either a probabilistic or deterministic assessment of hazard presented in United States Geological Survey (USGS) seismic maps. The recommended seismic hazard level for evaluation is the ASCE/SEI 41-17[5] Basic Safety Earthquake BSE-2E, which corresponds to a 5% probability of exceedance in a 50-year period.

Properties of cast-in-place concrete, reinforcing steel and masonry infill materials shall be taken from design drawings or other as-built information. Physical testing of in-situ materials is not required as part of the methodology.

Concrete and reinforcing steel properties used in component strength calculations shall be expected material properties, determined as follows:

- expected compressive strength of concrete, f_{ce}', taken as the specified or nominal compressive strength, f_c', multiplied by 1.5
- expected yield strength of reinforcement, f_{ye}, taken as the specified or nominal yield strength, f_y, multiplied by 1.25

The methodology assumes that the building and structure are in generally good condition. Although site investigations do not require destructive investigation or detailed condition assessment, visual observations should include assessment of the general condition of significant structural component in accessible areas. If the overall building or significant structural components are judged to be in poor condition, then the evaluation results should be adjusted to characterize the potentially weakened state of the building.

The evaluation methodology assumes the existence of a complete load path. Prior to implementing the methodology, it is important to understand the load path for lateral and vertical forces. Significant gaps in the load path can lead to catastrophic failure even if most structural components would remain undamaged. Assessment of the load path is primarily qualitative in nature, based on identification of gaps in the seismic-force resisting system rather than quantitative analysis of structural elements.

Except for buildings classified as frame structures, buildings should be evaluated for adequate connection between the diaphragms and shear walls. In general, it is assumed that frame structures have distributed lateral systems and that there is sufficient interconnection between the frames and floor slabs to provide at least a nominal load path for transferring diaphragm forces to the frame elements.

Penthouses and other rooftop structures need not be explicitly evaluated in this methodology if the weight is no more than 25 percent of the effective seismic weight of the main roof level and the area of the footprint is no more than 30 percent of the total area of the main roof level.

The purpose of this rapid method is to determine the “Building Rating (BR)” so as to see if the structure has a high or low likelihood of failure. This index (BR) ranges from 0.1 to 0.9, with 0.1 indicating a low risk of failure while 0.9 indicating a high risk of failure.

More specifically, in order to determine the “Building Rating (BR)”, the following steps need to be applied:

- **Calculation of Seismic Spectrum**

As described above, the recommended seismic hazard level corresponds to a 5% probability of exceedance in a 50-year period. There are three seismic zones in Greece with PGA ranging from 0.16g to 0.36g, but these values refer to an earthquake with a probability of exceedance 10% in a 50-year period, so we have to multiply with a factor γ to determine the appropriate result. According to Eurocode 8 – Part 1[6], this factor is given by the equation:

$$\gamma = \left(\frac{475}{T_R}\right)^{-\frac{1}{3}} \tag{1.37}$$

where:

T_R = return period corresponding to the required earthquake (in this case 5% in a 50-year period) equal to:

$$T_R = \frac{1}{-\ln\left(1 - poe\right)/50} \tag{1.38}$$

where:

poe = percentage of exceedance (in this case 0.05)

After calculating this factor, we use the same equations from Eurocode 8 – Part 1[6], just replacing the a_{gR} value with $\gamma \cdot a_{gR}$.

- **Calculation of Components Strength**

For most concrete components, strengths are determined in accordance with ASCE/SEI 41-17[5] and ACI 318-14[21].

For the purposes of this methodology, the determination of expected gravity loads on concrete columns, concrete walls, concrete slab-column connections and masonry infill panels shall be determined first. Vertical loads P_g due to expected gravity load effects are calculated as follows:

$$P_g = P_D + 0.25P_L \tag{1.39}$$

where:

P_D = axial load due to tributary dead loads

P_L = axial load due to unreduced tributary live loads

Consideration of column and wall component axial loads due to earthquake is required only where explicitly specified. Column axial load P_{eq} due to earthquake overturning effects at each story x is calculated as:

$$P_{eq} = \frac{V_y\left(h_{eff} - h_x\right)}{L} \tag{1.40}$$

where:

V_y = base shear strength of the structure

h_{eff} = effective height of the building, defined as the height from the base to the centroid of lateral forces (same as the effective height of an equivalent single-degree-of-freedom system, which may be taken as 0.7 h_n in multistory buildings having uniform distribution of effective weight over the building height and h_n in single-story buildings)

h_n = height from the base of the building to the highest level of the seismic force-resisting system (i.e., level n)

h_x = height from the base of the building to level x (i.e., the bottom of story x)

L = plan dimension between the outermost frame columns in the direction of interest at story x

Axial load P_{eq} is taken as positive in compression and can be distributed in proportion to contribution of each frame line to total building V_y. Tension loads due to earthquake overturning are not critical and need not be considered. For columns located in a story above the effective height of the equivalent single-degree-of-freedom system, or for interior columns in a frame line (i.e., not the outermost columns in the frame line), P_{eq} may be taken as zero. If h_{eff} falls within a story, P_{eq} should be computed for the columns within that story.

Component strengths used in this evaluation methodology shall be taken as expected strengths. Expected strengths shall be calculated using expected material properties as described above. Column shear strength V_n is calculated based on ASCE/SEI 41-17[5], as follows:

$$V_n = k\left(\frac{A_{sv} f_{ye} d}{s} + \lambda\left(\frac{6\sqrt{f_{ce}^{'}}}{l_{\inf}/d}\sqrt{1+\frac{P_g}{6\sqrt{f_{ce}^{'}}A_g}}\right)0.8A_g\right) \tag{1.41}$$

where:

k = factor related to displacement ductility demand, can be taken as 1.0 for the purpose of calculating column shear strength in this methodology

s = spacing of transverse reinforcement

A_{sv} = area of transverse reinforcement within spacings

d = effective depth of the column section

f_{ye} = expected yield strength of reinforcement

f_{ce}' = expected compressive strength of concrete

λ = 0.75 for lightweight concrete and 1.0 for normal weight concrete

$l_{\inf}$ = clear height of an equivalent cantilever column from the face of a joint to the point of inflection (or zero moment), in a typical story, $l_{\inf}$ may be taken as half of the column clear height l_u

P_g = column axial load (in compression) due to expected gravity load effects (equal to zero if in tension)

A_g = gross area of concrete column section

It is permitted to assume $d = 0.8h_c$, where h_c is the overall dimension of the column in the direction of shear. The ratio $l_{\inf}/d$ should not be taken greater than 4, nor less than 2. For columns satisfying the detailing and proportioning requirements of ACI 318-14[21] Chapter 18, the shear strength equations of ACI 318-14[21] Chapter 22 are permitted.

The expected flexural strength of a column section M_n is calculated in accordance with ACI 318-14[21] Chapter 22 using expected material properties and $\phi = 1.0$. The axial load for determining flexural strength shall be the expected gravity loads on the column.

For the purpose of determining the effective base shear strength and controlling building mechanism that will be presented next, if the lap splices in the vertical reinforcement of the columns do not meet the minimum length requirements in

ASCE/SEI 41-17[5] Section 10.3.5, the expected flexural strength shall be modified to account for a reduction in the strength of vertical reinforcement in accordance with ASCE/SEI 41-17[5] Section 10.5.3.

Beam-column joints satisfying the following conditions may be assumed to have sufficient strength to develop the flexural strength of the beams and columns framing into the joint:

- interior beam-column joints where beams frame into all four faces of the joint
- any joint with hoop reinforcement within the joint at a spacing not exceeding the lesser of: h/3, where h is the overall dimension of the column in the direction of shear, or 8 inches.

For other joints, shear strength should be calculated in accordance with ASCE/SEI 41-17[5] Section 10.4.2.2, except that the joint shear strength V_{nj} need not be taken less than:

$$V_{nj} = 10\lambda\sqrt{\frac{h_c}{h_b}}\sqrt{f_{ce}'}A_j \tag{1.42}$$

where:

h_c = overall dimension of the column in the direction of shear

h_b = overall depth of the beam

A_j = effective cross-sectional area of the beam-column joint

The ratio h_c/h_b shall be taken less than or equal to 1.0. The effective area of the joint A_j can be taken as $h_c \cdot (b_c + w_b')/2$, where b_c is overall width of the column in the direction perpendicular to joint shear and w_b' is the web width of the beam, excluding portions of the web that extend beyond the width of the column. Joint shear strength calculated in accordance with ASCE/SEI 41-17[5] is typically less than the value in the exception noted above. Thus, V_{nj} calculated using Eq. (1.42) is generally expected to control.

In calculating expected shear and flexural strengths of concrete walls, expected material properties shall be used and the strength reduction factors ϕ should be taken as unity (i.e., $\phi = 1.0$).

The shear strength of a wall section shall be calculated as the expected shear strength using the procedures of ACI 318-14[21] Section 18.10, except that the restriction on spacing, reinforcement ratio and number of curtains of reinforcement need not apply. If the transverse reinforcement ratio ρ_n is less than 0.0015, the contribution of wall reinforcement to the shear strength of the wall may be computed using $\rho_n = 0.0015$.

The shear strength of a wall shall be limited by the sliding strength at horizontal construction joints. Sliding strength shall be taken as twice the value determined in accordance with the shear friction provisions in ACI 318-14[21] using expected material properties and taking the friction coefficient and strength reduction factor as unity (i.e., $\mu = \varphi = 1.0$). Shear friction capacity shall be modified to account for the effects of non-conforming lap splice lengths as described below.

For the purpose of determining the effective base shear strength and controlling building mechanism that will be presented next, if the lap splices in the vertical reinforcement of a wall do not meet the minimum length requirements in ASCE/SEI 41-17[5] Section 10.3.5, the expected flexural strength shall be modified to account for a reduction in the strength of vertical reinforcement in accordance with ASCE/SEI 41-17[5] Section 10.3.5.

For columns in a typical story, the column capacity-limited (plastic) shear strength V_p is equal to:

$$V_p = \frac{M_{cT} + M_{cB}}{l_u} \tag{1.43}$$

where:

M_{cT} = flexural strength at the top of the column

M_{cB} = flexural strength at the bottom of the column

l_u = clear height of the column

M_{cT} and M_{cB} are taken as the lesser of the expected flexural strength of the column section M_n, calculated in accordance with ACI 318-14[21] Chapter 22 and the flexural strength controlled by the beams or slabs at the top or bottom of the column, respectively. The column clear height l_u shall account for translational or rotational restraint provided by concrete spandrel beams, partial height concrete walls and masonry infill panels. Where the moment is controlled by the beams or slabs, l_u shall be replaced by the story height.

Alternatively, if the beams are shear-controlled, the expected flexural strength of the beam can be limited by the shear strength of the beam. Additionally, the flexural strength at the top or bottom of the column need not be taken greater than the capacity associated with the joint shear strength, taken as $V_{nj} \cdot h_b/2$, where V_{nj} is the expected shear strength of the joint determined in accordance with Eq. (1.42) and h_b is the overall depth of the beam.

- **Structural Classification**

The seismic risk presented by different types of buildings varies widely. Many buildings have limited ductility under lateral loading, particularly those governed by members failing in shear. Lateral strength also varies and buildings with structural walls are generally stronger than buildings with frames. However, some frame systems designed for high gravity loads, such as those used in warehouse occupancies, also have relatively high strength and frames with round columns and spiral transverse reinforcing can be quite ductile in response to earthquake shaking.

Regardless of the type of structural system, the performance of many buildings is largely influenced by the building configuration. Tall first stories, used to create lobbies or first-floor commercial spaces, are often also soft and/or weak stories that concentrate lateral displacement at that level. Other vertical irregularities, such as discontinuous walls supported on columns or girders, or columns supported on transfer girders, can be a local collapse risk that could lead to global collapse. Plan irregularities, especially in buildings with walls, induce torsional response that can amplify lateral displacement demands on frame and wall lines.

For the purposes of this methodology, a reinforced concrete column is a vertically oriented concrete component with an elevation aspect ratio of:

$$l_u / h_{\max} \geq 2.0 \tag{1.44}$$

where:

l_u = clear height of column

h_{max} = largest cross-sectional dimension

For the purposes of this methodology, vertically oriented reinforced concrete components that are not otherwise classified as columns and that possess the following characteristics, shall be considered as structural concrete walls:

1. Thickness of at least 4 inches, or thickness at least 1/25 of the distance between supporting or enclosing members.
2. Ratios of distributed longitudinal and transverse reinforcement to gross concrete area perpendicular to that reinforcement of at least 0.0015.
3. Sufficient anchorage to the floor diaphragms such that they can be considered as altering the lateral behavior of the structural system.
4. Sufficient strength to significantly impact other structural members (e.g., a spandrel that reduces the clear height of a column).

Classification of buildings into structural systems of similar seismic response is necessary for implementation of the evaluation methodology. Each principal horizontal direction in a building can have a different structural system classification. The structural systems that this method considers are the following:

- Frame Systems
- Frame-Wall Systems
- Bearing Wall Systems
- Infilled Frame Systems

Frame and frame-wall systems are generally the most common in modern structures. Regarding to bearing wall systems, are systems configured such that the majority of gravity loads are supported on structural concrete bearing walls. Isolated columns supporting up to 25% of the gravity load in the building can be present in a bearing wall system. If columns are used to support more than 25% of the gravity load, the structure shall be classified as a frame-wall system. Infilled frame systems are systems that satisfy the requirements for frames, or frame-walls and also include one or more masonry infill panels confined within the beam and column framing.

- **<u>Determination of Plastic Mechanisms</u>**

The effective yield strength V_y of a structure is defined as the base-shear strength under static lateral loading, considering expected member strengths calculated along each principal direction of the building. It is permitted to calculate the effective yield strength V_y using established principles of mechanics.

The nonlinear static procedure of ASCE/SEI 41-17[5], with a lateral force distribution in accordance with Eq. (1.45), is an acceptable procedure for calculating the effective yield strength. Alternatively, it is permitted to estimate the effective yield strength based on the plastic mechanism base shear strength V_{p1} using the steps outlined next.

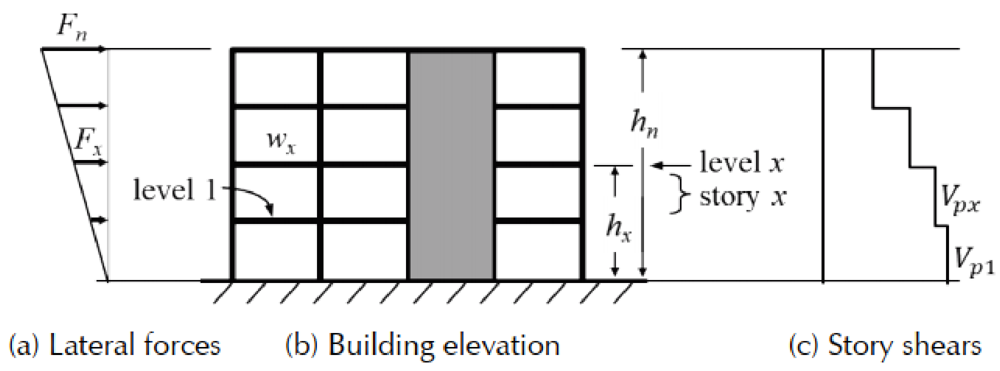


Figure 1.1 Distribution of lateral forces

The plastic mechanism base-shear strength V_{p1} is calculated under static lateral loading, not including P-delta effects. The plastic mechanism base-shear strength V_{p1} is the sum of lateral forces F_x that are concentrated at each floor level x. Values at each floor level are defined by:

$$F_x = C_{vx} V_{p1} \tag{1.45}$$

where:

V_{p1} = plastic mechanism base-shear strength

C_{vx} = vertical distribution factor for story forces determined as:

$$C_{vx} = \frac{w_x h_x}{\sum_{i=1}^{n} w_i h_i} \tag{1.46}$$

where:

x = level under consideration, with level 1 designating the first level above the base

w_i and w_x = the portion of the total effective seismic weight of the structure (W) located or assigned to level i or x

h_i and h_x = the height (ft or m) from the base to level i or x

n = designation for the uppermost level in the main portion of the building

For the lateral loading defined by Eq. (1.45), the story shear in story x (i.e., the story below level x) is related to the story shear in the first story through the following expression:

$$V_{px} = \left(\sum_{i=x}^{n} C_{vi} \right) V_{p1} \tag{1.47}$$

There are four (4) mechanisms to calculate the base-shear strength V_{p1}. More specifically:

- **1st Mechanism**

Mechanism 1 assumes that the building strength is controlled by the strength of structural elements in the first story. It requires calculation of the strengths of the columns and walls in the first story (Figure 1.2).

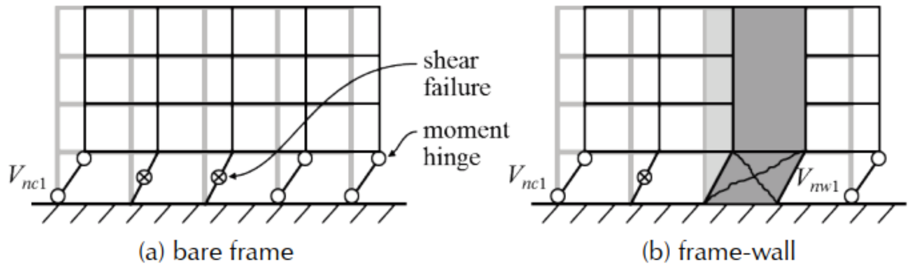


Figure 1.2 Mechanism 1 for calculation of plastic mechanism base-shear strength

The lateral strength of an individual column in the first story is the smaller of the column shear strength and the shear associated with development of the column flexural strength, that is:

$$V_{nc1} = \min\left(V_{nc}, \sum M_{nc} / l_u\right) \tag{1.48}$$

where:

V_{nc1} = lateral strength of column in first story

V_{nc} = shear strength of column in first story, calculated according to Eq. (1.41)

M_{nc} = flexural strength of column, where the summation applies to the flexural strengths at the top and bottom of a column, considering the strength of the connection with the foundation where that condition applies

l_u = clear height of column

The shear strength of the walls V_{nw1} in the first story is calculated as described above, but not exceeding the shear corresponding to development of flexural strengths at the top and bottom of the wall. The plastic mechanism base-shear strength V_{p1} in the first story corresponding to Mechanism 1 is equal to:

$$V_{p1} = \sum V_{nc1} + \sum V_{nw1} \tag{1.49}$$

- **2nd Mechanism**

Mechanism 2 assumes that the vertical elements have sufficient strength to force yielding throughout the building height. This mechanism (or mechanisms approaching this mechanism) may occur in frames with walls or in frames having columns much stronger than the beams. Mechanism 2 should be checked in frames with walls (Figure 1.3(a)) and in pure frames having $\Sigma M_{nc} / \Sigma M_{nb} \geq 1.5$ in typical stories within the lower half of the building height (Figure 1.3(b)).

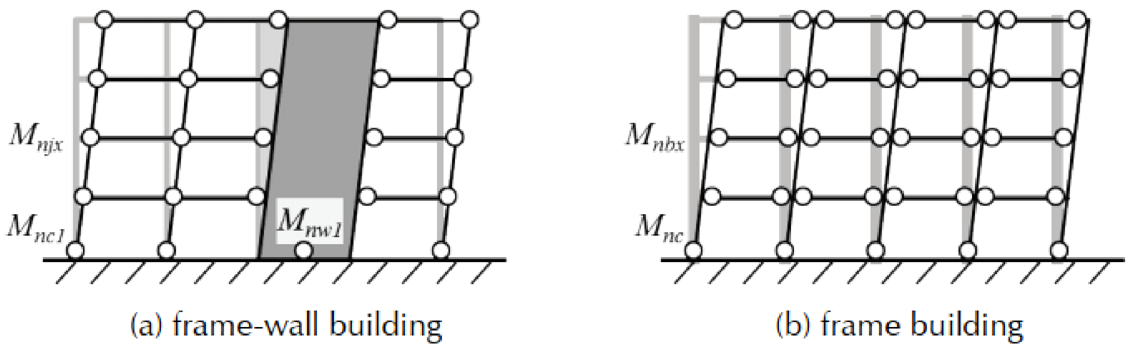


Figure 1.3 Mechanism 2 for calculation of plastic mechanism base-shear strength

To determine the base-shear strength according to this mechanism, it is necessary to calculate the framing strengths resolved at column-footing connections, beam-column (and slab-column) connections and beam-wall (slab-wall) connections, plus moment strengths of the walls in the first story, as follows:

- M_{nc1} = flexural strength at the bottom of a column, considering the strength of the connection with the foundation where that condition applies, but not exceeding the moment corresponding to column shear failure
- M_{njx} = flexural strength of beam-column, slab-column, beam-wall and slab-wall connections(joints) at level x
- M_{nw1} = flexural strength at the bottom of a wall, considering the strength of the connection with the foundation where that condition applies

At a beam-column connection, M_{njx} is the smallest moment that can be developed at the joint at level x, as limited by the column flexural strength and the beam flexural strength, that is:

$$M_{njx} = \min\left[\sum M_{nc}, \sum M_{nb}\right] \tag{1.50}$$

where:

M_{nc} = flexural strength of columns, summed above and below the beam-column joints at level x, but not exceeding the moments corresponding to column shear failure

M_{nb} = flexural strength of beams, summed on both sides of the beam-column joint in the direction of framing at level x, it is acceptable, but not required, to limit the beam moments to values corresponding to beam shear failure

In buildings with combinations of beam-column, slab-column and beam-wall framing, values of M_{njx} are determined at each beam-column, slab-column and beam-wall joint and the results are then summed. The plastic mechanism base-shear strength corresponding to Mechanism 2 is equal to:

$$V_{p1} = \frac{\sum M_{nc1} + \sum M_{njx} + \sum M_{nw1}}{h_{eff}} \tag{1.51}$$

where:

M_{nc1} = flexural strength at the bottom of a column, considering the strength of the connection with the foundation where that condition applies, but not exceeding the moment corresponding to column shear failure

M_{njx} = values of M_{njx} from Eq. (1.50) or from connections between beams and walls

M_{nw1} = flexural strength at the bottom of a wall, considering the strength of the connection with the foundation where that condition applies

h_{eff} = effective height of the building, defined as the height from the base to the centroid of lateral forces (same as the effective height of an equivalent single-degree-of-freedom system, which may be taken as $0.7h_n$ in multistory buildings having uniform distribution of effective weight over the building height and h_n in single-story buildings)

3rd Mechanism

Mechanism 3 applies only to buildings with an obvious strength irregularity in which one story or multiple stories have story shear strengths that are significantly reduced relative to adjacent stories (Figure 1.4). This may occur due to reduction in strength of columns, walls, or both. Mechanism 3 need only be considered where the rate of reduction in story shear strength exceeds the rate of reduction in the story shear demand by more than 20%. Note that Mechanism 3 is similar to Mechanism 1, except the weak story is in the upper stories of a building. Therefore, the steps to determine base-shear strength for Mechanism 1 and Mechanism 3 are similar.

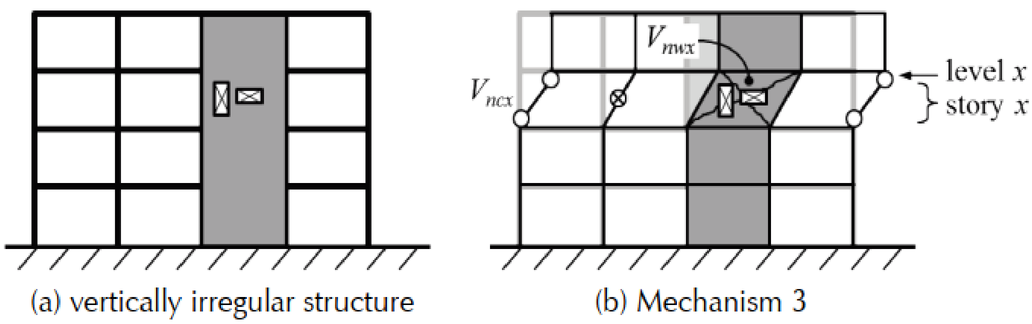


Figure 1.4 Mechanism 3 for calculation of plastic mechanism base-shear strength

To determine the plastic mechanism base-shear strength corresponding to Mechanism 3, it is necessary to first identify the weak story and then calculate the strengths of the columns and walls in the weak story. The lateral strength of an individual column in the weak story is the smaller of the column shear strength and the shear associated with development of the column moment strength, that is:

$$V_{ncx} = \min\left[V_{nc}, \sum M_{nc} / l_u\right] \tag{1.52}$$

where:

V_{ncx} = lateral strength of column in weak story x

V_{nc} = shear strength of column in weak story x calculated in accordance with Eq. (1.41)

M_{nc} = flexural strength of column, where the summation applies to the flexural strengths at the top and bottom of the weak story x

l_u = clear height of the columns in weak story x

The shear strength of the walls V_{nw1} in the weak story is calculated in accordance with American regulations as described before, but not exceeding the shear corresponding to development of wall flexural strengths. The plastic mechanism shear strength at story x corresponding to Mechanism 3 is:

$$V_{px} = \sum V_{ncx} + \sum V_{nwx} \tag{1.53}$$

where the summation applies to all the columns and walls in the weak story x.

Given the shear V_{px} at story x, the corresponding plastic mechanism base-shear strength V_{p1} is equal to:

$$V_{p1} = \frac{V_{px}}{\sum_{i=x}^{n} C_{vi}} \tag{1.54}$$

where:

V_{p1} = plastic mechanism base shear strength at story 1

V_{px} = plastic mechanism shear strength at story x

C_{vx} = vertical distribution factor for story forces as described in Eq. (1.46)

4th Mechanism

Similar to Mechanism 3, Mechanism 4 applies only to buildings with an obvious strength irregularity. In this case, the strength irregularity is associated with a significant reduction in the moment strength of columns, walls, or both. Note that Mechanism 4 is similar to Mechanism 2, except that the sidesway mechanism occurs in the upper stories of the building. Therefore, the steps to determine base-shear strength for Mechanism 2 and Mechanism 4 are similar (Figure 1.5).

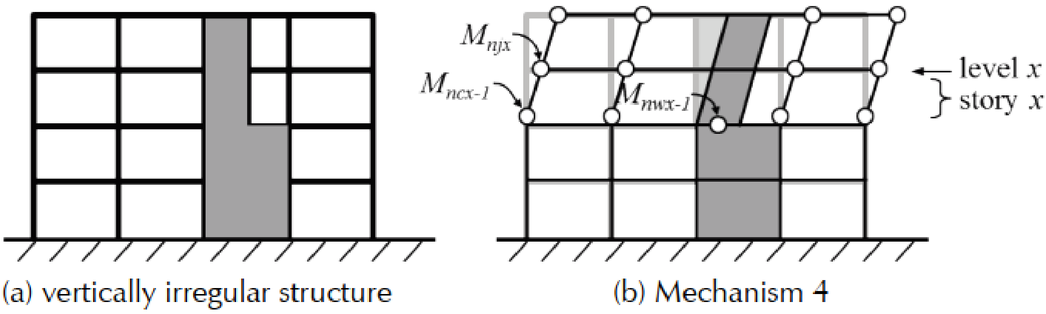


Figure 1.5 Mechanism 4 for calculation of plastic mechanism base-shear strength

To determine the plastic mechanism base-shear strength corresponding to Mechanism 4, it is necessary to calculate the framing strengths of columns and walls at the level of the strength irregularity, as well as framing strengths resolved at beam-column (and slab-column) and beam-wall (slab-wall) connections. The procedures are analogous to those for Mechanism 2 and are not repeated here. The plastic mechanism shear strength at story x corresponding to Mechanism 4 is:

$$V_{px} = \frac{\sum M_{ncx-1} + \sum M_{nj} + \sum M_{nwx-1}}{h_{eff}} \tag{1.55}$$

where:

M_{ncx-1} = flexural strength of column at level x–1, that is, bottom of story x, but not exceeding the moment corresponding to column shear failure

M_{nj} = values of moment that can be resisted at each joint, based on the limiting strength of beams, columns and joints, similar to Mechanism 2

M_{nwx-1} = flexural strength of wall at level x–1, that is, bottom of story x, calculated in accordance with the previous

h_{eff} = height of centroid of lateral forces acting above level x–1, which is the same as the centroid of the vertical distribution factors C_{vx} from level x to roof n

Given the shear V_{px} at story x, the corresponding plastic mechanism base-shear strength V_{p1} is equal to:

$$V_{p1} = \frac{V_{px}}{\sum_{i=x}^{n} C_{vi}} \tag{1.56}$$

where:

V_{p1} = plastic mechanism base shear strength at story 1

V_{px} = plastic mechanism shear strength at story x

C_{vx} = vertical distribution factor for story forces as described in Eq. (1.46)

- **Determination of Critical Plastic Mechanism and Critical Story**

Calculation of the plastic mechanism base-shear strength for the building as the minimum value of V_{p1} for all four (4) applicable mechanisms, as given by Equations (1.49), (1.51), (1.54), (1.56) and use this value as the effective yield strength V_y.

Where Mechanism 1 or 2 controls the strength along a principal framing direction, the critical story in that direction is the first story above the base. In cases where Mechanism 2 controls for determination of effective yield strength, if the calculated plastic mechanism base-shear strength for Mechanism 2 is equal to or greater than three-quarters (3/4) of the calculated plastic mechanism base-shear strength for Mechanism 1, Mechanism 1 should be taken as the controlling mechanism for the purposes of identifying critical stories and calculating drift demands.

- **Calculation of Effective Fundamental Period**

The effective fundamental periods in each of two principal horizontal directions is used to estimate the pseudo-acceleration spectral demands and spectral displacements in a building. The first thing is to determine the base shear ratio V_y/W, where V_y is the effective yield strength and W is the total effective seismic weight of a structure. Figure 1.6 illustrates the effective stiffness K_e intended for determination of fundamental period in this methodology.

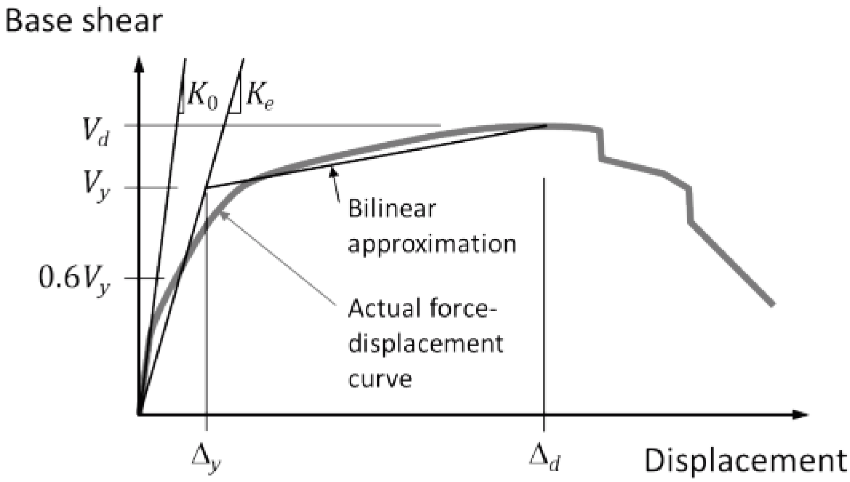


Figure 1.6 Force-displacement curve, showing the definition of effective stiffness K_e for calculation of the effective fundamental period (adapted from ASCE/SEI 41-17).

The effective fundamental period in this methodology is conceptually similar to the effective fundamental period defined in the Nonlinear Static Procedure of ASCE/SEI 41-17[5], which corresponds to an effective initial stiffness that accounts for concrete cracking and failure of more brittle elements prior to overall effective yielding of the structural system.

For frame systems, the effective fundamental period T_e maybe taken as:

$$T_e = 0.07\left(h_n\right)^{0.5}\left(\frac{V_y}{W}\right)^{-0.5} \tag{1.57}$$

For frame-wall systems other than pier-spandrel systems and for bearing wall systems, T_e maybe taken as:

$$T_e = 0.0026\frac{h_n}{\sqrt{C_w}} \leq T_e\left(\text{frame systems}\right) \tag{1.58}$$

where:

$$C_w = \frac{100}{A_{base}}\sum_{i=1}^{j}\frac{A_{wi}}{1+0.83\left(\frac{h_{wi}}{l_{wi}}\right)^2} \tag{1.59}$$

where:

h_n = height from the base of the building to the highest level of the seismic force-resisting system (ft)

A_{base} = area of the base of the structure (ft^2)

A_{wi} = area of the web of wall i (ft^2)

h_{wi} = height of the wall i, (ft)

l_{wi} = length of the wall i, (ft)

j = number of walls in the building effective in resisting lateral forces in the direction under consideration

- **Calculation of Global Demand-to-Capacity Ratio**

The global demand-to-capacity ratio $\mu_{strength}$ for a given direction of earthquake loading is calculated as:

$$\mu_{strength} = \frac{S_a}{V_y / W} C_m \tag{1.60}$$

where:

S_a = is the spectral acceleration at the effective fundamental period T_e

V_y/W = base shear ratio

C_m = is the effective mass factor determined in accordance with ASCE/SEI 41-17[5], as provided in Table 1.7.

Table 1.7 Values for effective mass factor C_m

Number of Stories	Frame System	Wall or Frame-Wall System	Pier-Spandrel System	Infill Wall System
1-2	1.0	1.0	1.0	1.0
≥ 3	0.9	0.8	0.8	1.0

Note that C_m shall be taken as 1.0 if the fundamental period T_e in the direction under consideration is greater than 1.0 sec.

- **Identification of Seismic Risk of the Buildings**

Some buildings can be classified as lower seismic risk buildings without further evaluation based on the global demand-to-capacity ratio $\mu_{strength}$. A building is considered to be lower seismic risk if $\mu_{strength}$ meets one of the following criteria in each of the two principal horizontal directions:

- $\mu_{strength} \leq 0.75$ in the case of frame systems with average column shear strength ratio in the critical story $V_p/V_n > 0.6$ or
- $\mu_{strength} \leq 1.50$ in all other cases

On the other hand, some buildings can be classified as exceptionally high seismic risk buildings without further evaluation based on the presence of certain seismic deficiencies. A building is considered to be lower seismic risk if $\mu_{strength}$ meets one of the following criteria in each of the two principal horizontal directions:

- Frame systems with $\mu_{strength} > 2.0$ and average column shear strength ratio in the critical story of $V_p/V_n > 1.5$ or
- Frame systems with $\mu_{strength} > 5.5$ and average column shear strength ratio in the critical story of $V_p/V_n < 0.6$ or

In frame buildings with a combination of shear-controlled and flexure-controlled columns in the critical story, the threshold value of $\mu_{strength}$ for exceptionally weak buildings can be linearly interpolated between the values of 2.0 and 5.5, based on the average value of shear strength ratio V_p/V_n in the critical story, ranging from 1.5 to 0.6.

To conclude, if a building is classified as lower seismic risk according to the above criteria, no further action is required, otherwise the following steps must be taken to determine the "Building Rating (BR)".

- **Calculation of Global Seismic Drift Demand**

Using the controlling plastic mechanism calculated as described above in each of the two principal framing directions, critical stories are defined. It is possible for different plastic mechanisms to control along each of the two principal framing directions. In such cases, the critical story in one principal direction may differ from the critical story in the other principal direction. The next step is the identification of critical components.

Critical components are those components deemed to be most vulnerable to damage and loss of vertical load-carrying ability. Columns in critical stories and columns below discontinuous walls are designated as critical components. Columns in other stories are not designated as critical, except where changes in column geometry or detailing create an increased vulnerability for column failure. In such cases, columns at other levels should also be designated as critical.

Vertical wall segments in critical stories are designated as critical components. Vertical wall segments in stories other than critical stories, as well as horizontal wall segments, are not designated as critical. Critical wall segments are evaluated based on drift demands and axial demands. Seismic overturning forces shall be estimated and added to gravity loads to determine $P/A_g f_c'$ on vertical wall segments with $h_w/l_w \geq 2$ and $l_w/b_w \leq 6$ occurring at the ends of critical walls, where h_w is the clear height, l_w is the horizontal length and b_w is the thickness of the wall segment. In other cases, tributary gravity loads can be used without seismic overturning forces.

In beam-column frames, beam-column corner connections at the top of columns in critical stories are designated as critical components where both conditions (a) and (b) apply:

a) The beam-column joint lacks transverse reinforcement
b) The joint shear strength calculated as described above is less than the joint shear generated by the controlling mechanism

Additionally, beam-column connections satisfying both (a) and (b) should be designated as critical components at other levels where unusual conditions create an increased vulnerability for joint shear failure.

The global seismic drift demand δ_{eff} for an equivalent single-degree-of-freedom (SDOF) system is equal to:

$$\delta_{eff} = C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g \tag{1.61}$$

where:

C_1 = modification factor to relate expected maximum inelastic displacement to displacement calculated for linear elastic response equal to:

$$C_1 = \begin{cases} \min\left(1+\dfrac{\mu_{strength}-1}{0.04a}, 1+\dfrac{\mu_{strength}-1}{aT_e^2}\right), & \mathrm{T}_e < 0.2 \\ 1+\dfrac{\mu_{strength}-1}{aT_e^2}, & 0.2 \le \mathrm{T}_e < 1.0 \\ 1, & \mathrm{T}_e \ge 1.0 \end{cases} \tag{1.62}$$

where:

α = site class factor equal to: 130 (Site Class A,B), 90 (Site Class C), 60 (Site Class D,E)

C_2 = modification factor to represent the effect of pinched hysteresis shape, cyclic stiffness degradation and strength deterioration on maximum displacement response equal to:

$$C_2 = \begin{cases} 1+\dfrac{1}{800}\left(\dfrac{\mu_{strength}-1}{T_e}\right)^2, & \mathrm{T}_e \le 0.7 \\ 1, & \mathrm{T}_e > 0.7 \end{cases} \tag{1.63}$$

T_e = effective fundamental period as described above

S_a = spectral acceleration at period, T_e

g = acceleration of gravity

- **Calculation of Story Seismic Drift Demand**

The story drift demand δ_x of story x is given by the equation:

$$\delta_x = \alpha_x h_{sx} \left(\frac{\delta_{eff}}{h_{eff}} \right) \leq \delta_{eff} \tag{1.64}$$

where:

α_x = coefficient to modify story drifts at story x for building configuration and strength characteristics (Table 1.8)

h_{sx} = height of story x

δ_{eff} = global drift demand of the equivalent SDOF system Eq. (1.61)

h_{eff} = effective height of the building, defined as the height from the base to the centroid of lateral forces (same as the effective height of an equivalent single-degree-of-freedom system, which may be taken as $0.7h_n$ in multistory buildings having uniform distribution of effective weight over the building height and h_n in single-story buildings)

h_n = height from the base of the building to the highest level of the seismic force-resisting system

Table 1.8 Values of the α_x coefficient for frame-wall systems

Number of Stories	**Yield Mechanism**	**Values of α_x coefficient**	
		Critical Stories	**Other Stories**
1	any	1.0	(n/a)
2	1	1.4	0.5
	2	1.2	1.0
	3, 4	1.5	1.0
≥ 3	1	$0.8h_{eff}/h_{sx}$	0.5
	2	1.2	1.0
	3	$0.8h_{eff}/h_{sx}$	0.5
	4	1.5	0.5 (stories below critical story) 1.0 (stories above critical story)

- **Calculation of Drift Demand on Critical Components**

For each critical component, the adjusted drift demand Δ_D is given by the equation:

$$\Delta_D = A_T \gamma \delta_x \tag{1.65}$$

where:

A_T = torsional amplification factor

γ = drift factor representing the fraction of story drift affecting the critical component

δ_x = the story drift demand

The torsional amplification A_T varies linearly in plan between a value of 1.0 at the center of strength and a value of $A_{T,max}$ at the edge of the building furthest from the center of strength (i.e., the weak or flexible side of the building). For all components located on the strong or stiff side of the building (between the center of strength and the edge of the building closest to the center of strength), $A_T = 1.0$.

The maximum torsional amplification factor $A_{T,max}$ is calculated in accordance with Eq (1.66), but should not be taken less than 1.0.

$$A_{T,\max} = 2.75\left(TR\right) + 0.5 \tag{1.66}$$

where:

TR = is the torsional ratio

The maximum torsional amplification factor $A_{T,max}$ may be taken as 1.0 if the wall index WI < 0.0004, or if the torsional ratio TR < 0.25.

TR shall be taken as the value of torsional ratio TR_x at the critical story, unless there is a significant torsional irregularity in another story. TR_x is the torsional ratio for story x calculated as:

$$TR_x = \frac{T_{Dx}}{T_{Cx}} \tag{1.67}$$

where:

T_{Dx} = torsion demand on story x

T_{Cx} = torsion capacity (strength) of story x

Torsion demand T_{Dx} is directional and must be calculated for each direction of earthquake loading according to equation:

$$T_{Dx} = V_{px} e \tag{1.68}$$

where:

V_{px} = plastic shear capacity of the critical story

e = eccentricity between the center of mass and the center of strength in the direction perpendicular to the direction of earthquake loading, e may not be taken as less than 5% of L (i.e., 0.05L), where L is the overall plan dimension perpendicular to the direction of earthquake loading

The coordinates of the center of strength $(\bar{x},\ \bar{y})$ are calculated from:

$$\bar{x} = \frac{\sum_{i=1}^{n_f} x_i V_{pfi}}{\sum_{i=1}^{n_f} V_{pfi}}, \quad \bar{y} = \frac{\sum_{i=1}^{n_f} y_i V_{pfi}}{\sum_{i=1}^{n_f} V_{pfi}} \tag{1.69}$$

where:

$(\bar{x},\ \bar{y})$ = are the orthogonal distances from between the column or wall line of interest and an established reference point

V_{pfi} = plastic capacity of frame or wall line i

n_f = number of frame or wall lines in story x, considering all frame or wall lines that resist torsion

Torsion capacity T_C is calculated considering the capacity of all frame or wall lines in all orientations and is given by the equation:

$$T_{Cx} = \sum_{i=1}^{n_f} \left|R_{fi}\right| \left|V_{pfi}\right| \tag{1.70}$$

where:

R_{fi} = orthogonal distance between frame or wall line i and the center of strength

The drift factor γ defines the fraction of story drift demand δ_x affecting critical components. Values of drift factor γ for critical components are defined below. Drift factors for columns are provided in Table 1.9. Values are required in critical stories for each column in each direction of earthquake loading. In Table 1.9, the drift factor for columns depends on the ratio of the strengths of columns to the strengths of horizontal members framing into the column. This ratio is calculated for each column at the beam-column or slab-column connection at the top of the critical story. For beam-column framing, this is represented by $\Sigma M_c/\Sigma M_b$, where ΣM_c is the sum of column strengths above and below the beam-column joint and ΣM_b is the sum of the strengths of beams framing into the joint in the direction under consideration.

Table 1.9 Drift factor γ for columns

Ratio of Column Strengths to Beam Strengths	**Column Drift Factor γ**
≤ 0.6	0.85
1	0.70
≥ 2.4	0.30

For critical vertical wall segments, the drift factor γ is taken as 1.0. For slab-column connections and beam-column corner connections, the drift factor γ is taken also as 1.0.

- **<u>Calculation of Drift Capacity on Critical Components</u>**

The drift capacity of critical columns is given by the following equation:

$$\Delta_C = l_u\left(\theta_c + 0.01\right) \tag{1.71}$$

where:

l_u = is clear height of the column

θ_c = column plastic rotation capacity

The column plastic rotation capacity θ_c is calculated in accordance with Table 1.10 for tied columns and Table 1.11 for spiral-reinforced columns. Column plastic rotation capacity θ_c is based on the column shear strength ratio V_p/V_n, axial load ratio $P/A_g\, f_{ce}'$ and shear reinforcement ratio ρ_t. Except for corner columns, the axial load ratio is based on the gravity load P_g. For corner columns, the axial load ratio is based on the total column axial load $P = P_g + P_{eq}$, where P is positive in compression and P_g and P_{eq} are determined according to equations (1.39) and (1.40). Note that V_p/V_n should not be taken less than 0.2 and ρ_t should not be taken greater than 0.0175 in any case, nor greater than 0.0075 when ties are not adequately anchored in the core. Table 1.10 and Table 1.11 are divided between flexure-critical columns and other columns (flexure-shear or shear-critical columns). Flexure-critical columns are columns with $V_p/V_n \leq 0.6$, $\rho_t > 0.002$ and $s/d < 0.5$. Flexure-shear or shear-critical columns are defined as columns not classified as flexure critical.

Table 1.10 Plastic rotation capacities for tied columns

Flexure-Critical Columns ($V_p/V_n \le 0.6$, $\rho_t > 0.002$, and $s/d < 0.5$)	
For $\left(\dfrac{P}{A_g f'_{ce}}\right) \ge 0.1$	$\theta_c = 11.4\rho_t + 0.034 - \left(\dfrac{P}{A_g f'_{ce}}\right)(14\rho_t + 0.036) \ge 0.0$
For $\left(\dfrac{P}{A_g f'_{ce}}\right) < 0.1$	$\theta_c = 10\rho_t + 0.03 \ge 0.0$
Flexure-Shear and Shear-Critical Columns (i.e., Columns not classified as Flexure-Critical Columns)	
For $\left(\dfrac{P}{A_g f'_{ce}}\right) \le 0.5$	$\theta_c = \dfrac{0.5}{5 + \dfrac{P}{0.8A_g f'_{ce}} \dfrac{1}{\rho_t} \dfrac{f'_{ce}}{f_{ye}}} - 0.01 \ge \theta_{c,\min}$ $P / A_g f'_{ce}$ should not be taken smaller than 0.1
θ_c should be reduced linearly for $\left(\dfrac{P}{A_g f'_{ce}}\right) > 0.5$ from its value at $\left(\dfrac{P}{A_g f'_{ce}}\right) = 0.5$ to zero at $\left(\dfrac{P}{A_g f'_{ce}}\right) = 0.7$	
$\theta_{c,\min} = 0.042 - 0.023\left(\dfrac{P}{A_g f'_{ce}}\right) + 0.63\rho_t - 0.023\left(\dfrac{V_p}{V_n}\right) \ge 0.0$ $P / A_g f'_{ce}$ should not be taken smaller than 0.1	

Table 1.11 Plastic rotation capacities for spiral-reinforced columns

Flexure-Critical Columns ($V_p/V_n \le 0.6$, $\rho_t > 0.002$, and $s/d < 0.5$)	
For $\left(\dfrac{P}{A_g f'_{ce}}\right) \ge 0.1$	$\theta_c = 1.15\left[11.4\rho_t + 0.034 - \left(\dfrac{P}{A_g f'_{ce}}\right)(14\rho_t + 0.036)\right] \ge 0.0$
For $\left(\dfrac{P}{A_g f'_{ce}}\right) < 0.1$	$\theta_c = 1.15[10\rho_t + 0.03] \ge 0.0$
Flexure-Shear and Shear-Critical Columns (i.e., Columns not classified as Flexure-Critical Columns)	
For $\left(\dfrac{P}{A_g f'_{ce}}\right) \le 0.5$	$\theta_c = \dfrac{0.65}{5 + \dfrac{P}{0.8A_g f'_{ce}} \dfrac{1}{\rho_t} \dfrac{f'_{ce}}{f_{ye}}} - 0.01 \ge \theta_{c,\min}$ $P / A_g f'_{ce}$ should not be taken smaller than 0.1
θ_c should be reduced linearly for $\left(\dfrac{P}{A_g f'_{ce}}\right) > 0.5$ from its value at $\left(\dfrac{P}{A_g f'_{ce}}\right) = 0.5$ to zero at $\left(\dfrac{P}{A_g f'_{ce}}\right) = 0.7$	
$\theta_{c,min} = 0.06 - 0.06\left(\dfrac{P}{A_g f'_{ce}}\right) + 1.3\rho_t - 0.037\left(\dfrac{V_p}{V_n}\right) \ge 0.0$ $P / A_g f'_{ce}$ should not be taken smaller than 0.1	

The drift capacity Δ_C of critical walls and vertical wall segments is calculated in accordance with Table 1.12, for flexure-critical walls and Table 1.13, for shear-critical walls. If Mechanism 2 or 4 controls, walls and vertical wall segments can be assumed to be flexure-critical. If Mechanism 1 or 3 controls, walls and vertical wall segments are generally shear-critical, except when the calculation of wall shear strength V_{nwx} is controlled by the shear corresponding to the development of the wall flexural strength. In Table 1.12, l_w is the horizontal length, b_w is the thickness and c is the neutral axis depth of the vertical wall segment. The neutral axis depth c can be computed from moment curvature analysis or approximated by:

$$\frac{c}{l_w} = \frac{a}{100} + b\frac{P}{A_g f_{ce}'} \tag{1.72}$$

where:

a, b = coefficients provided in Table 1.14

Table 1.12 Drift capacity of flexure-critical walls or vertical wall segments (%)

	c/l_w (2)											
l_w/b_w (2)	0.05	0.1	0.15	0.2	0.25	0.3	0.35	0.4	0.45	0.5	0.55	0.6
≤6	3.50	3.50	3.50	3.50	3.25	3.00	2.67	2.62	1.85	1.80	1.76	1.71
9	3.50	3.50	3.42	3.28	3.00	2.50	2.20	2.00	1.34	1.24	1.14	1.04
12	3.50	3.35	3.09	2.84	2.59	2.00	1.54	1.32	1.00	0.75	0.75	0.75
15	3.46	3.06	2.67	2.28	1.88	1.75	1.50	1.25	1.00	0.75	0.75	0.75
18	3.28	2.72	2.15	1.75	1.75	1.50	1.25	1.00	0.75	0.75	0.75	0.75
21	3.08	2.31	1.75	1.75	1.50	1.25	1.25	1.00	0.75	0.75	0.75	0.75
24	2.84	1.83	1.75	1.75	1.50	1.25	1.25	1.00	0.75	0.75	0.75	0.75
27	2.57	1.75	1.75	1.75	1.50	1.25	1.25	1.00	0.75	0.75	0.75	0.75
30	2.28	1.75	1.75	1.75	1.50	1.25	1.25	1.00	0.75	0.75	0.75	0.75
>35	1.75	1.75	1.75	1.75	1.50	1.25	1.25	1.00	0.75	0.75	0.75	0.75

Table 1.13 Drift capacity of shear-critical walls or vertical wall segments

$P/A_g f'_c$ (2)	Drift Capacity (%)
0.0	4.00
0.005	3.50
0.01	3.00
0.03	2.30
0.05	2.00
0.10	1.50
0.15	1.25
0.20	1.00
0.30	0.75
0.40	0.60
0.50	0.45

Table 1.14 Coefficients for calculation of neutral axis depth c

Cross-Section	*a*	*b*
Rectangular	10	1.2
I-shaped and Barbell	3	1.4
T-shaped, L-shaped and Half-Barbell (web in compression)	30	0.7
T-shaped, L-shaped and Half-Barbell (web in tension)	20	2.0

Where the ends of walls and vertical wall segments are confined by spirally reinforced columns or confined boundaries as defined in Table 1.15, the drift capacity may be increased by 25% for flexure-critical walls and by 50% for shear-critical walls.

Where integral concrete columns are located within the length of a wall or vertical wall segment, the drift capacity shall be determined as follows:

- For walls that are not more than 15 feet in length, with an axial load ratio of 0.30 or less, with an integral concrete column on forming to the confinement requirements in Table 1.15, located within the middle third of the wall length, the drift capacity may be taken as 4%.
- For other cases of walls with integral concrete columns, the drift capacity may be taken as the larger of:
    - the drift capacity determined for the wall or vertical wall segment, excluding consideration of the integral concrete column, or
    - the drift capacity determined for a segment of wall on either side of the column, treated as a half-barbell section, with the column assumed to be located at the compression end of the segment.

Table 1.15 Minimum transverse reinforcement in integral columns or boundary elements required to be classified as confined

Transverse Reinforcement	Applicable Expression
$\frac{A_{sh}}{sb_c}$ for rectilinear hoops[1]	Greater of: $0.2\left(\frac{A_g}{A_{ch}}-1\right)\frac{f'_c}{f_{yt}}$ and $0.06\frac{f'_c}{f_{yt}}$
ρ_s for spiral or circular hoops	Greater of: $0.3\left(\frac{A_g}{A_{ch}}-1\right)\frac{f'_c}{f_{yt}}$ and $0.08\frac{f'_c}{f_{yt}}$

The drift capacity Δ_C of critical beam-column corner connections is calculated according to equation:

$$\Delta_C = \left(0.1 - 0.33\frac{P}{A_g f_{ce}'}\right)h_{sx} \tag{1.73}$$

Story drift capacity Δ_C need not be taken less than $0.025h_{sx}$. Note that story drift capacity refers to story x while the drift capacity ratio is calculated for the beam-column connection at level x, that is, the level at the top of story x.

Column axial load P is calculated considering combined gravity and earthquake loading in both orthogonal directions (X and Y) as:

$$P = P_g + P_{eq,X} + P_{eq,Y} \tag{1.74}$$

where:

P_g = axial load due to gravity, determined in accordance with (1.39)

P_{eq} = axial load due to earthquake overturing effects, determined in accordance with Eq. (1.40) in both directions

- **<u>Determination of Column and Wall Ratings</u>**

Column ratings CR for critical columns, slab-column connections and corner beam-column connections and wall ratings WR for critical walls and vertical wall segments, are determined as the ratio of drift demand to drift capacity Δ_D/Δ_C in accordance with Table 1.16, for earthquake loading in each direction.

Table 1.16 Column rating (CR) and Wall rating (WR)

Drift Demand to Drift Capacity Ratio Δ_D/Δ_C	Column Rating, *CR* Wall Rating, *WR*
$\Delta_D/\Delta_C \leq 0.25$	0.0
$0.4 \geq \Delta_D/\Delta_C > 0.25$	0.1
$0.5 \geq \Delta_D/\Delta_C > 0.4$	0.2
$0.7 \geq \Delta_D/\Delta_C > 0.5$	0.3
$0.9 \geq \Delta_D/\Delta_C > 0.7$	0.4
$1.1 \geq \Delta_D/\Delta_C > 0.9$	0.5
$1.4 \geq \Delta_D/\Delta_C > 1.1$	0.6
$1.8 \geq \Delta_D/\Delta_C > 1.4$	0.7
$2.5 \geq \Delta_D/\Delta_C > 1.8$	0.8
$3.0 \geq \Delta_D/\Delta_C > 2.5$	0.9
$\Delta_D/\Delta_C > 3.0$	0.93

- **Determination of "Story Rating (SR)"**

The story rating is a number representing the relative likelihood that an individual story will lose its ability to support vertical loads under the assumed earthquake loading. The story rating SR is given by the following equation:

$$SR = 1.5R_{adj} - 0.1 \tag{1.75}$$

where:

R_{adj} = the adjusted average of column and wall ratings in the story, defined by equation:

$$R_{adj} = R_{avg} + 0.625R_{avg}\left(COV - 0.4\right) \tag{1.76}$$

where:

COV = the standard deviation of all the column and wall ratings at a story divided by the weighted average rating, R_{avg}, at that story

R_{avg} = the weighted average rating for all columns, walls and vertical wall segments in the story, in which the values are weighted by the gravity load taken by each column, wall, or wall segment, given by the equation:

$$R_{avg} = \sum_{i=1}^{n_{col}} f_{col,i}CR_i + \sum_{i=1}^{n_{wall}} f_{wall,i}WR_i \tag{1.77}$$

where:

$f_{col,i}$, $f_{wall,i}$ = fraction of gravity loads supported by column i and wall i, respectively

n_{col}, n_{wall} = number of columns and walls in a story

R_{adj} shall not be taken less than R_{avg}, nor greater than $1.25R_{avg}$. Moreover, the story rating, SR, shall not be taken less than 0.1 nor greater than 0.9. Low values of story rating, SR, indicate a low likelihood of failure, while high values indicate a high likelihood of failure.

- **Determination of "Building Rating (BR)"**

The building rating BR is taken as the maximum story rating SR determined in either direction for critical stories over the height of the building.

1.4. Third Level Pre-Earthquake Assessment Method

1.4.1. Greek Seismic Code of Interventions (KANEPE)

The Greek Code of Interventions (KANEPE)[3] is the main method for evaluating and redesigning structures in Greece. Below are described the basic theoretical and practical characteristics of the method used in this work. The evaluation is usually carried out using the inelastic static analysis (pushover analysis), which is performed under the action of constant gravity loads ($G+\psi_2 \cdot Q$) and periodically increased seismic loads.

The seismic (horizontal) loads applied to the building under study follow different types of distributions such as:

- Triangular
- Uniform
- Modal

During the application of the gradual increase in seismic action, some reduction in the stiffness of the structure occurs due to the inelastic behavior of its members. Thus, for each step of the analysis, by calculating the base shear and the displacement of the top of the structure, the capacity curve is obtained (Figure 1.7).

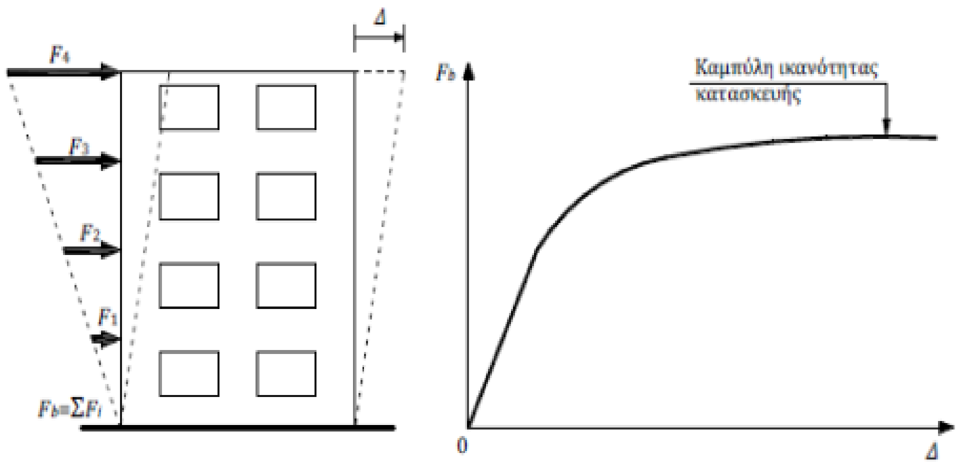


Figure 1.7 Indicative capacity curve of a multi-story building

The seismic capacity of each structure is determined by the Code of Interventions through the combination of certain seismic action levels and the three basic performance levels, i.e., the acceptable level of damage. These levels are:

A) **"Damage Limitation"**: The Damage Limitation after the earthquake (A) according to KANEPE[3] is a condition in which it is expected that no building operation is interrupted during and after the design earthquake, with the possible exception of minor importance functions. A few hairline crack may occur in the structure.

B) **“Significant Damage”**: The Significant Damage (B) according to KANEPE[3] is a condition in which repairable damage to the structure is expected to occur during the design earthquake, without causing loss or serious injury of people and without substantial damage to personal property or materials that are stored in the building.

C) **“Near Collapse”**: The Near Collapse (C) according to KANEPE[3] is a condition in which extensive and serious or severe (non-repairable, in general) damage to the structure is expected during the design earthquake, however, the structure retains its ability to bear the prescribed vertical loads (during and for a period after the earthquake), in any case without other substantial safety factor against total or partial collapse.

In order to choose the admissible type of analysis and the appropriate confidence factor values, the following three data reliability levels (DRL) are defined:

- Tolerable DRL
- Sufficient DRL
- Hight DRL

The factors determining the obtained data reliability level are the (i) geometry, which is the geometrical properties of the structural system and of non-structural elements, e.g. masonry infill panels, that may affect structural response, (ii) details, which include the amount and detailing of reinforcement in reinforced concrete sections, the connection of floor diaphragms to lateral resisting structure, the bond and mortar jointing of masonry and the nature of any reinforcing elements in masonry and finally (iii) materials, that is the mechanical properties of the constituent materials. In this work, high DRL used because it is a newly constructed building.

The target displacement δ_t (§ 5.7.4.2 of KANEPE[3]) shall be calculated taking into account all the relevant factors affecting the displacement of a building that responds inelastically. It is permitted to consider the displacement of an elastic single degree of freedom system with a fundamental period equal to the fundamental period of the building that is subjected to the seismic actions for which the verification is made. An appropriate correction is needed in order to derive the corresponding displacement of the building assumed to be responding as an elastic-perfectly plastic system.

If a more accurate method is not used, the target displacement δ_t can be calculated using the following equation and be corrected (where necessary) according to §5.7.4.2 of KANEPE[3] as follows:

$$\delta_t = C_o C_1 C_2 C_3 \left(\frac{T_e^2}{4\pi^2} \right) S_e \left(T \right) \tag{1.78}$$

where:

S_e = elastic spectral acceleration according to Eurocode 8 – Part1[6]

T_e = equivalent fundamental period of the structure

C_o, C_1, C_2, C_3 = correction factors

With regard to the coefficients, the following applies:

a) *C_o coefficient*

This coefficient relates the spectral displacement of the equivalent elastic system of stiffness K_e [S_d = [$T_e^2/4\pi^2$]·S_e(T)], with the actual displacement δ_t of the top of the structure, which is assumed to be responding as an elasto-plastic system (§ 5.7.3.4 of KANEPE[3]). The values of this coefficient can be taken equal to 1.0, 1.2, 1.3, 1.4, 1.5, for a number of stories equal to 1, 2, 3, 5 and ≥ 10, respectively.

b) *C_1 coefficient*

The ratio $C_1 = \delta_{inel.}/\delta_{el}$ the maximum inelastic displacement of a building to the corresponding elastic displacement may be obtained from the following relationship:

$$C_1 = \left\{ \begin{array}{ll} 1.0, & \text{if } T_e \geq T_c \\ \dfrac{1 + \left(R - 1 \right) \dfrac{T_c}{T_e}}{R}, & \text{if } T_e < T_c \end{array} \right\} \tag{1.79}$$

where:

T_c = the corner period initiating the descending branch of the response spectrum (EC8[6])

T_e = equivalent fundamental period of the structure

R = V_e/V_y, the ratio of the elastic demand over the yield strength of the structure. This ratio can be estimated from the relationship:

$$R = \frac{S_e / g}{V_y / W} C_m \tag{1.80}$$

where:

V_y = the yield strength calculated by appropriate bilinearization of the base shear vs. top displacement relationship of the building, as defined in §5.7.3.4 of KANEPE[3]. For simplicity, (and conservatively), the ratio V_y/W in equation can be taken equal to 0.15 for buildings with a dual structural system and 0.10 for buildings with a pure frame system

C_m = coefficient of active mass (to take account of higher eigenmodes), which may be taken to be equal to 0.85

c) C_2 coefficient

This coefficient takes into account the influence of the shape of the hysteresis loop at the maximum displacement. Its values may be obtained from Table 1.17

Table 1.17 Values of the C_2 coefficient (KANEPE)

Performance Level	T = 0.1 sec		T ≥ T_c	
	Structural Type 1	Structural Type 2	Structural Type 1	Structural Type 2
Immediate Occupancy	1.0	1.0	1.0	1.0
Life Safety	1.3	1.0	1.1	1.0
Collapse Prevention	1.5	1.0	1.2	1.0

As structural systems of Type 1 are denoted structures with low ductility (e.g. buildings constructed prior to 1985 or buildings whose capacity curve is characterized by an available displacement ductility which is lower than 2), that are expected to have inferior hysteretic behavior than structures with high ductility which are characterized as Type 2 systems, e.g. buildings constructed after 1985, or buildings whose capacity curve is characterized by an available displacement ductility which is higher than 2. Given the fact that the influence of hysteretic behavior is greater for higher levels of post-elastic structural response, the values of the C_2 coefficient are conditioned to the performance level.

d) C_3 coefficient

This coefficient takes into account the increase of displacements due to second order (P-D) effects. It can be taken equal to $1 + 5(\theta\text{-}0.1)/T_e$, where θ is the interstory drift sensitivity coefficient (see EC8-Part1[6]). In the common case (for RC and masonry buildings) where $\theta < 0.1$, the coefficient is taken equal to $C_3 = 1.0$.

The nonlinear force-displacement relationship that relates the base shear with the displacement of the control node shall be replaced by an idealized curve for the determination of the equivalent lateral stiffness K_e and the corresponding yield strength V_y of the building as mentioned above.

It is recommended that the idealized capacity curve (force-displacement relationship) is bilinear, with a slope of the first branch equal to K_e and a slope of the second branch equal $a \cdot K_e$. The two lines that compose the bilinear curve can be defined graphically, on the criterion of approximately equal areas of the sections defined above and below the intersection of the actual and the idealized curves (Figure 1.8).

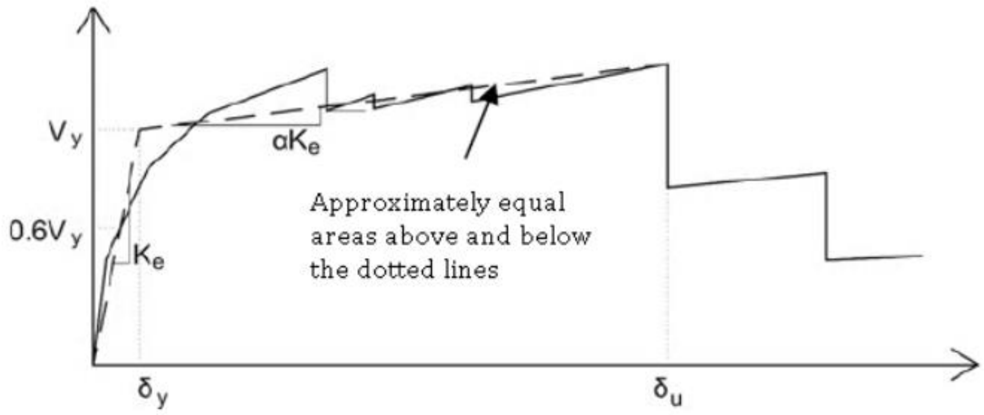


Figure 1.8 Idealization of a (indicative) capacity curve with a bilinear curve (KANEPE)

The equivalent lateral stiffness K_e is determined as the secant stiffness that corresponds to a force equal to the 60% of the yielding force V_y, the latter defined by the intersection of the lines above. The normalized inclination (a) of the second branch is determined by a straight line passing through the point of the (actual) nonlinear capacity curve that corresponds to the ultimate displacement (δ_u), beyond which a significant drop of the strength of the structure is observed (Figure 1.8). In any case, the derived value of a must be positive (or zero), but not larger than 0.10 (in order to be compatible with the other assumptions made by the method for estimating the target displacement δ_t, such as the C_1 coefficient). The recommended fraction of the resistance reduction is 15%, provided that no primary vertical member has reached failure at this level (in such a case, the bilinearization of the curve shall be made for the displacement that corresponds to this failure). The equivalent fundamental period in the direction examined shall be estimated based on the idealized capacity curve.

The value T_e of the equivalent fundamental period is derived by the following expression:

$$T_e = T\sqrt{\frac{K_o}{K_e}} \tag{1.81}$$

where:

T = elastic fundamental period in the direction under examination and is derived by eigenvalue analysis

K_o = corresponding elastic lateral stiffness

K_e = equivalent lateral stiffness

All the member checks (chord rotation capacity and shear capacity) should be carried out for all the elements of every story, according to sections 7.2.2, 7.2.4 and Appendix 7C of KANEPE[3] (for the equations see Dritsos S. Method), considering the members as primary or secondary seismic elements, designated in accordance with the definitions of section 2.4.3.4 of KANEPE[3]. Moreover, beam-column joints checks can be employed in order to check (i) the joint's diagonal tension and (ii) the joint's diagonal compression.

The Nonlinear Static Procedures are frequently used in regular engineering applications to avoid the inherent complexity and additional computational effort required by the incremental nonlinear time-history analyses. This method was developed for regular structures where the influence of higher eigenmodes is not important. In order to check this condition, an initial dynamic elastic analysis is required, taking into account the eigenmodes that contribute at least 90% of the total mass. Then a second dynamic elastic analysis based only on the first eigenmode (in each direction) is performed. The influence of the higher eigenmodes can be considered significant when the shear force on just one story resulting from the first analysis exceeds 130 % that from the second analysis. Where the influence of the higher eigenmodes is significant, the static inelastic analysis can be applied provided it is used in conjunction with a complementary dynamic elastic analysis. The procedure used to overcome this problem is the Extended N2-Method[22] as proposed by Peter Fajfar.

The basic assumption used in pushover-based methods is that the structure vibrates predominantly in a single mode. This assumption is not always fulfilled, especially in the case of high-rise buildings and/or torsionally flexible plan-asymmetric buildings. If higher modes of vibration are important, either in plan or in elevation, some corrections have to be applied to the basic procedure. The extension of the N2 method to plan-asymmetric buildings, where torsional influences are important, was made by assuming that the torsional influences in the inelastic range are the same as in the elastic range. The torsional influences are determined by the standard elastic modal analysis. They are applied in terms of correction factors, which are used for the adjustment of results obtained by the usual pushover analysis.

In order to predict the structural response for a building with a non-negligible effect of higher modes along the elevation, the following procedure can be applied:

- Perform the basic N2 analysis (EC8-Part 3[4]) or KANEPE[3] procedure. The basic N2 analysis or KANEPE[3] method consists of a pushover analysis of an MDOF structural model, a bilinear idealization of the pushover curve and the transformation to an equivalent SDOF model, the determination of the displacement demand of the SDOF system by using inelastic response spectrum and the transformation of the displacement demand from the SDOF to the MDOF system. It is assumed that the effects of higher modes on the displacement demand (target roof displacement) are negligible. Seismic demand for all relevant quantities is represented by the results of the pushover analysis at the target roof displacement.
- Perform the standard elastic modal analysis of the MDOF model considering all relevant modes. Determine story drifts for each story. Normalize the results in such a way that the top displacement is equal to the target top displacement.
- Determine the envelope of the results obtained in Steps1 and 2.
    - (3a) For each story, determine the correction factors c_{HM}, which are defined as the ratio between the results obtained by elastic modal analysis (Step2) and the results obtained by pushover analysis (Step1). If the ratio is larger than 1.0, the correction factor c_{HM} is equal to this ratio, otherwise it amounts to 1.0. Note that the correction factors for displacement are small and can be neglected in most practical applications. The correction factors for story drifts are important.
    - (3b) The resulting story drifts (and displacements, if necessary) are obtained by multiplying the results determined in Step 1 with the corresponding correction factors c_{HM}.
- Determine other local quantities. The resulting correction factors for story drifts c_{HM} apply to all local deformation quantities (e.g. rotations). Correction factors c_{HM} for story drifts also apply to internal member forces, provided that the resulting internal forces do not exceed the load-bearing capacity of the structural member.

1.4.2. Eurocode 8 – Part 3

The most common method for assessment of existing buildings is the nonlinear static analysis. It is based on pushover analyses carried out under constant gravity loads and increasing lateral forces, applied at the location of the masses to simulate the inertia forces induced by the seismic action. As the model may account for both geometrical and mechanical nonlinearity, this method can describe the evolution of the expected plastic mechanisms and structural damage.

Each pushover analysis leads to a capacity curve, which is a relationship between the total base shear and the horizontal displacement of a representative point of the structure, termed "control node". The demand at the considered Limit State–Near Collapse, Significant Damage or Damage Limitation-is determined by the appropriate comparison between the capacity determined by the pushover curve and the demand established as the damped Linear Response Spectrum. To do so, the "control node" displacements are defined in terms of spectral quantities relative to an equivalent single-degree-of-freedom (SDOF) system which is derived from the multi-degree-of-freedom (MDOF) response estimated according to Annex B of EN1998-1:2004[6].

The structural demand associated with the acquired target displacement shall fulfil the verification criteria defined in Eurocode 8–Part 3[4]. Accordingly, element's demand for brittle (shear) and ductile (chord rotation deformation) actions are deemed to comply with limits that take into account: section mechanical properties, element's bending, shear and axial force interaction and strength/stiffness degradation associated with the ductility demand and cyclic hysteretic response of reinforced concrete elements, through appropriate material nonlinearity consideration.

According to EN1998-3[4] section 2.1, performance requirements refer to the state of damage in the structure defined through three limit states, namely Near Collapse (NC), Significant Damage (SD) and Damage Limitation (DL). More specifically:

A) **"Damage Limitation"**: The limit state of Damage Limitation (DL) may be selected, according to EN 1998-3[4], where the target state of damage in the structure is insignificant and does not need any repair measures. The structure is only lightly damaged, with structural elements prevented from significant yielding and retaining their strength and stiffness properties. Non-structural components, such as partitions and infills may show distributed cracking, but the damage could be economically repaired. Permanent drifts are negligible. The appropriate level of protection is achieved by choosing a seismic action with a return period of 225 years corresponding to a probability of exceedance of 20% in 50 years.

B) **“Significant Damage”**: The limit state of Significant Damage (SD) may be selected, according to EN 1998-3[4], where the target state of damage in the structure is significant and can sustain after-shocks of moderate intensity, although it is likely to be uneconomic to repair. Some residual lateral strength and stiffness and vertical elements are capable of sustaining vertical loads. Non-structural components are damaged, although partitions and infills have not failed out-of-plane. Moderate permanent drifts are present. The appropriate level of protection is achieved by choosing a seismic action with a return period of 475years corresponding to a probability of exceedance of 10% in 50 years.

C) **“Near Collapse”**: The limit state of Near Collapse (NC) may be selected, according to EN 1998-3[4], where the target state of damage in the structure is near collapse and would probably not survive another earthquake, even of moderate intensity. The structure is heavily damaged with low residual lateral strength and stiffness, although vertical elements are still capable of sustaining vertical loads. Most non-structural components have collapsed and large permanent drifts are present. The appropriate level of protection is achieved by choosing a seismic action with a return period of 2.475 years corresponding to a probability of exceedance of 2% in 50 years.

In order to choose the admissible type of analysis and the appropriate confidence factor values, the following three knowledge levels are defined:

- KL1: Limited Knowledge
- KL2: Normal Knowledge
- KL3: Full Knowledge

The factors determining the obtained knowledge level are (i) geometry, i.e. the geometrical properties of the structural system and the non-structural elements, e.g. masonry infill panels, that may affect structural response, (ii) details, which include the amount and detailing of reinforcement in reinforced concrete sections, the connection of floor diaphragms to lateral resisting structure, the bond and mortar jointing of masonry and the nature of any reinforcing elements in masonry and finally (iii) materials, that is the mechanical properties of the constituent materials.

The target displacement is defined as the seismic demand derived from the elastic response spectrum in terms of displacement of an equivalent single-degree-of-freedom system. To define the target displacement of a MDOF system a number of steps have to be followed according to Annex B of EN 1998-1[6].

The following relation between normalized lateral forces F_i and normalized displacements Φ_i is assumed:

$$F_i = m_i \Phi_i \tag{1.82}$$

where:

m_i = mass in the i-th story

Φ_i = normalized displacements, in such a way that $\Phi_n = 1$, where n is the control node, consequently $F_n = m_n$

The mass of an equivalent SDOF system m^* is determined as:

$$m^* = \sum m_i \Phi_i = \sum F_i \tag{1.83}$$

The transformation factor is given by:

$$\Gamma = \frac{m^*}{\sum m_i \Phi_i^2} = \frac{\sum F_i}{\sum \left(\frac{F_i^2}{m_i} \right)} \tag{1.84}$$

The force F^* and displacement d^* of the equivalent SDOF system are computed as:

$$F^* = \frac{F_b}{\Gamma}, \qquad d^* = \frac{d_n}{\Gamma} \tag{1.85}$$

where:

F_b and d_n are, respectively, the base shear force and the control node displacement of the Multi Degree of Freedom (MDOF) system.

The yield force F_y^*, which represents also the ultimate strength of the idealized SDOF system, is equal to the base shear force at the formation of the plastic mechanism. The initial stiffness of the idealized system is determined in such a way that the areas under the actual and the idealized force-deformation curves are equal, as shown in the figure below.

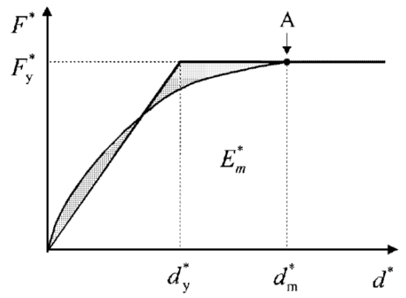


Figure 1.9 Determination of the idealized elasto-perfectly plastic force-displacement relationship (EN 1998-3)

Based on this assumption, the yield displacement on the idealized SDOF system d_y^* is given by:

$$d_y^* = 2\left(d_m^* - \frac{E_m^*}{F_y^*}\right) \tag{1.86}$$

where:

E_m = actual deformation energy up to the formation of the plastic mechanism

The period T^* of the idealized equivalent SDOF system is determined by:

$$T^* = 2\pi\sqrt{\frac{m^* d_y^*}{F_y^*}} \tag{1.87}$$

The target displacement of the structure with period T^* and unlimited elastic behavior is given by:

$$d_{et}^* = S_e\left(T^*\right)\left[\frac{T^*}{2\pi}\right]^2 \tag{1.88}$$

where:

$S_e(T^*)$ = elastic acceleration response spectrum at the period T^*

For the determination of the target displacement d_t^* for structures in the sort-period range and for structures in the medium and long-period ranges different expressions should be used as indicated below. The corner period between the short-and medium-period range is T_C.

➔ For $T^* < T_c$

$$d_t^* = \begin{cases} d_{et}^*, & \text{if } F_y^*/m^* \geq S_e\left(T^*\right) \\ \dfrac{d_{et}^*}{q_u}\left(1+\left(q_u - 1\right)\dfrac{T_c}{T^*}\right) \geq d_{et}^*, & \text{if } F_y^*/m^* < S_e\left(T^*\right) \end{cases} \tag{1.89}$$

where:

q_u = ratio between the acceleration in the structure with unlimited elastic behavior $S_e(T^*)$ and the structure with limited strength F_y^*/m^*.

$$q_u = \frac{S_e\left(T^*\right)m^*}{F_y^*} \tag{1.90}$$

➔ For $T^* \geq T_c$

$$d_t^* = d_{et}^* \tag{1.91}$$

The target displacement of the MDOF system is given by:

$$d_t = \Gamma d_t^* \tag{1.92}$$

The target displacement corresponds to the displacement of the control node. All the member checks (chord rotation capacity and shear capacity) should be carried out for all the elements of every story, according to Annex A of EN 1998-3[4], considering the members as primary or secondary seismic elements, designated in accordance with the definitions in EN 1998-1[6]. Moreover, beam-column joints checks maybe employed in order to check (i) the horizontal shear forces acting on the core of the joints, (ii) the joint's horizontal hoops area and (iii) whether adequate vertical reinforcement is provided to the column passing through the joint. The equations used for the calculation of chord rotation capacity and shear capacity are similar to KANEPE[3].

If the influence of higher eigenmodes is important, then the Extended N2-Method[22] is used to take into account the torsion effects as described above.

1.4.3. ASCE/SEI 41-17

Current practice in USA is regulated by the ASCE 41-17[5]: Seismic Evaluation and Retrofit of Existing Buildings in combination with ACI 318[21]: Building Code Requirements for Structural Concrete.

According to ASCE 41-17[5], the seismic actions effects in combination with the effects of the permanent and variable loads are evaluated using one of the following methods:

- Linear Static Procedure (LSP) in accordance with section 7.4.1 of ASCE 41-17[5]
- Linear Dynamic Procedure (LDP) in accordance with section 7.4.2 of ASCE 41-17[5]
- Nonlinear Static Procedure (NSP) according to section 7.4.3 of ASCE 41-17[5]
- Nonlinear Dynamic Procedure (NDP) according to section 7.4.4 of ASCE 41-17[5].

The most common method in assessment practice of existing buildings is the nonlinear static analysis. It is based on pushover analyses carried out under constant gravity loads and increasing lateral forces, applied at the location of the masses to simulate the inertia forces induced by the seismic action.

Each pushover analysis leads to a capacity curve, which is a relationship between the total base shear and the horizontal displacement of a representative point of the structure, termed "control node". The demand at the considered Performance Level – Operational Level, Immediate Occupancy, Life Safety or Collapse Prevention – is determined by the appropriate comparison between the capacity determined by the pushover curve and the demand established as the damped Linear Response Spectrum. The structural demand associated with the acquired target displacement shall fulfil the verification criteria defined in ASCE 41-17[5].

According to ASCE 41-17[5] section 2.2, the objectives of the assessment or redesign (Table 1.18) consist of combinations of both a performance level and a seismic action, given an "acceptable probability of exceedance within the life cycle of the building" (design earthquake), as shown in Table 1.18 of ASCE 41-17[5] below.

The target building performance levels refer to the state of damage in the structure defined through four limit states, namely Operational Level (1-A), Immediate Occupancy (1-B), Life Safety (3-C) and Collapse Prevention (5-D).

Table 1.18 Performance objectives (ASCE 41-17)

Seismic Hazard Level	**Target Building Performance Levels**			
	Operational Performance Level (1-A)	**Immediate Occupancy Performance Level (1-B)**	**Life Safety Performance Level (3-C)**	**Collapse Prevention Performance Level (5-D)**
50%/50 years	a	b	c	d
BSE-1E (20%/50 years)	e	f	g	h
BSE-2E (5%/50 years)	i	j	k	l
BSE-2N (2%/50 years)	m	n	o	p

✓ Performance Level of Operational Level (1-A)

The Operational Level (1-A), according to ASCE 41-17[5], is a condition in which it is expected that damage is insignificant and structure does not need any repair measures. Structural elements are prevented from significant yielding and retaining their strength and stiffness properties. All systems important to normal operation are functional. Non-structural components, such as partitions and infills should not be damaged.

✓ Performance Level of Immediate Occupancy (1-B)

The Immediate Occupancy after the earthquake (1-B), according to ASCE 41-17[5], is a condition in which it is expected that no building operation is interrupted during and after the design earthquake, with the possible exception of minor importance functions. Structural elements are retaining their strength and stiffness properties. A few hairline cracks may occur in the structure.

✓ Performance Level of Life Safety (3-C)

The Life Safety (3-C), according to ASCE 41-17[5], is a condition in which moderate damage to the structure is expected to occur during the design earthquake, although it is likely to be uneconomic to repair. Structural elements are retaining some residual strength and stiffness. Non-structural components are damaged, although partitions and infills have not failed out-of-plane. Moderate permanent drifts are present.

✓ Performance Level of Collapse Prevention (5-D)

The Collapse Prevention (5-D), according to ASCE 41-17[5], is a condition in which severe (non-repairable, in general) damage to the structure is expected during the design earthquake and would probably not survive another earthquake. The structure is heavily damaged with low residual lateral strength and stiffness, although vertical elements are still capable of sustaining vertical loads. Most non-structural components have collapsed and large permanent drifts are present.

The criteria for the selection of the Performance Objectives may be found in ASCE 41-17[5].

In order to choose the admissible type of analysis and the appropriate confidence factor values, the following three knowledge levels are defined:

- Minimum Knowledge
- Usual Knowledge
- Comprehensive Knowledge

The factors determining the obtained data reliability level are the (i) geometry, which is the geometrical properties of the structural system and of non-structural elements, e.g. masonry infill panels, that may affect structural response, (ii) details, which include the amount and detailing of reinforcement in reinforced concrete sections, the connection of floor diaphragms to lateral resisting structure, the bond and mortar jointing of masonry and the nature of any reinforcing elements in masonry and finally (iii) materials, that is the mechanical properties of the constituent materials.

The target displacement δ_t (§ 7.4.3.3 of ASCE 41-17[5]) shall be calculated taking into account all the relevant factors affecting the displacement of a building that responds inelastically. It is permitted to consider the displacement of an elastic single degree of freedom system with a fundamental period equal to the fundamental period of the building that is subjected to the seismic actions, for which the verification is made. An appropriate correction is needed in order to derive the corresponding displacement of the building assumed to be responding as an elastic-perfectly plastic system.

For buildings with rigid diaphragms at each floor level, the target displacement shall be calculated in accordance with equation (7-28) of ASCE 41-17[5] or by an approved procedure that accounts for the nonlinear response of building.

The equations are the same as described with FEMA P-2018 method above [Equations (1.57) to (1.63)]

1.5. Non-Structural Elements

According to § 4.3.5.2 of EN 1998-1[6], non-structural elements (appendages) of buildings (e.g. parapets, gables, antennae, mechanical appendages and equipment, curtain walls, partitions, railings) that might, in case of failure, cause risks to persons or affect the main structure of the building or services of critical facilities, shall, together with their supports, be verified to resist the design seismic action.

In this work, there is a water tank on the roof of the building that need to be examined because of the acceleration is greatly amplified. The effects of the seismic action may be determined by applying to the non-structural element a horizontal force F_a which is defined as follows:

$$F_a = \frac{S_a W_a \gamma_a}{q_a} \tag{1.93}$$

where:

W_a = weight of the non-structural element

γ_a = importance factor taken as 1.0

q_a = behavior factor of the element taken as 2.0, because we have tank on legs acting as unbraced cantilevers along less than one half of their total height

S_a = seismic coefficient applicable to non-structural elements equal to:

$$S_a = aS\left[\frac{3\left(1 + z/H\right)}{1 + \left(1 - T_a/T_1\right)^2} - 0.5\right] \tag{1.94}$$

where:

α = ratio of the design ground acceleration on type A ground a_g to the acceleration of gravity g

S = soil factor

T_a = fundamental vibration period of the non-structural element

T_1 = fundamental vibration period of the building in the relevant direction

z = height of the non-structural element above the level of application of the seismic action

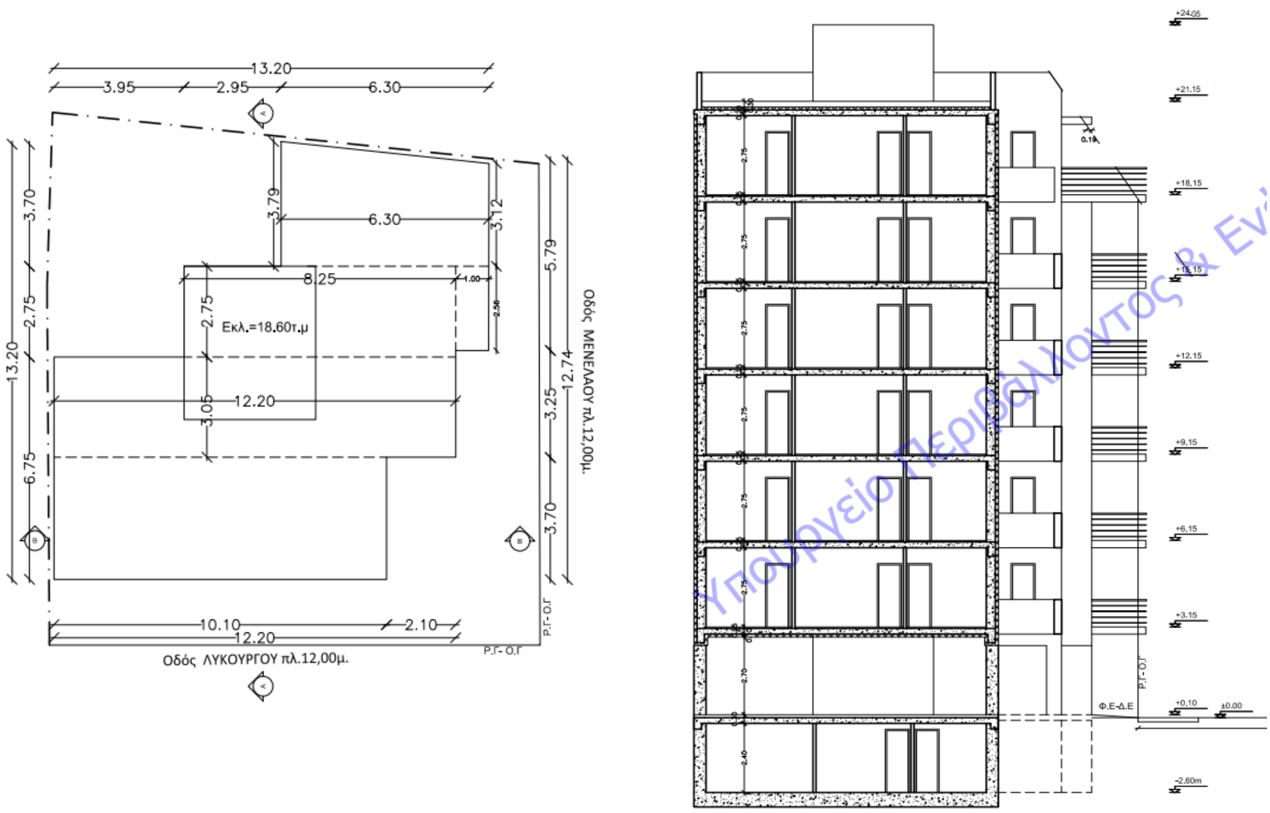
H = total height of the building

Chapter 2

Building Description

2.1. General Characteristics of the Building

It is an existing 7-story reinforced concrete building with basement, built in 2022 in Kallithea Attica, an urban region in the South Sector of Athens. The total plot area is 229.39 m^2, the floor plan area is 121.59 m^2 and the penthouse area is 18.60 m^2 (Figure 2.1). The height of the ground floor (pilotis) is 3.15 m, while in the others it is 3.00 m, except for the basement and the penthouse where the height is limited to 2.60 m and 2.90 m, respectively (Figure 2.1). As described in the figure below, there is regularity in elevation and irregularity in plan.



(a) (b)

Figure 2.1 (a) Floor plan of the building, (b) Typical section of the building

Furthermore, a water tank was placed on the roof of the building, although it was not on the plans because we wanted to investigate the seismic behavior of this structure while it was high above the ground. In general, these tanks are used to supply settlements developed in areas with very low slopes and are combined with the operation of pumping stations, such as in Cyprus, where there are a lot of these structures (Figure 2.2).

Figure 2.2 Typical water tank on the roof of a building in Cyprus[23]

The water tank provides the necessary pressure in the water distribution network. In this work, a horizontal water tank with a capacity of 2000 L was selected by Top Roto LTD[24] (Figure 2.3).

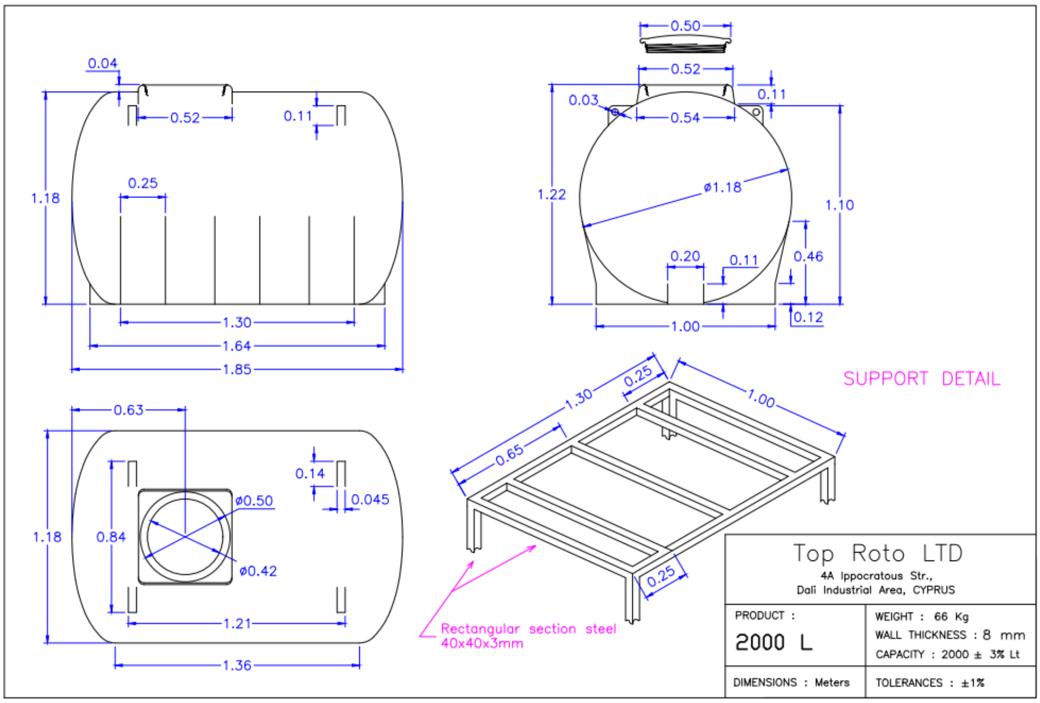


Figure 2.3 Dimensions of the selected water tank and support

The position of the water tank on the roof of the building is shown in the figure below (Figure 2.4).

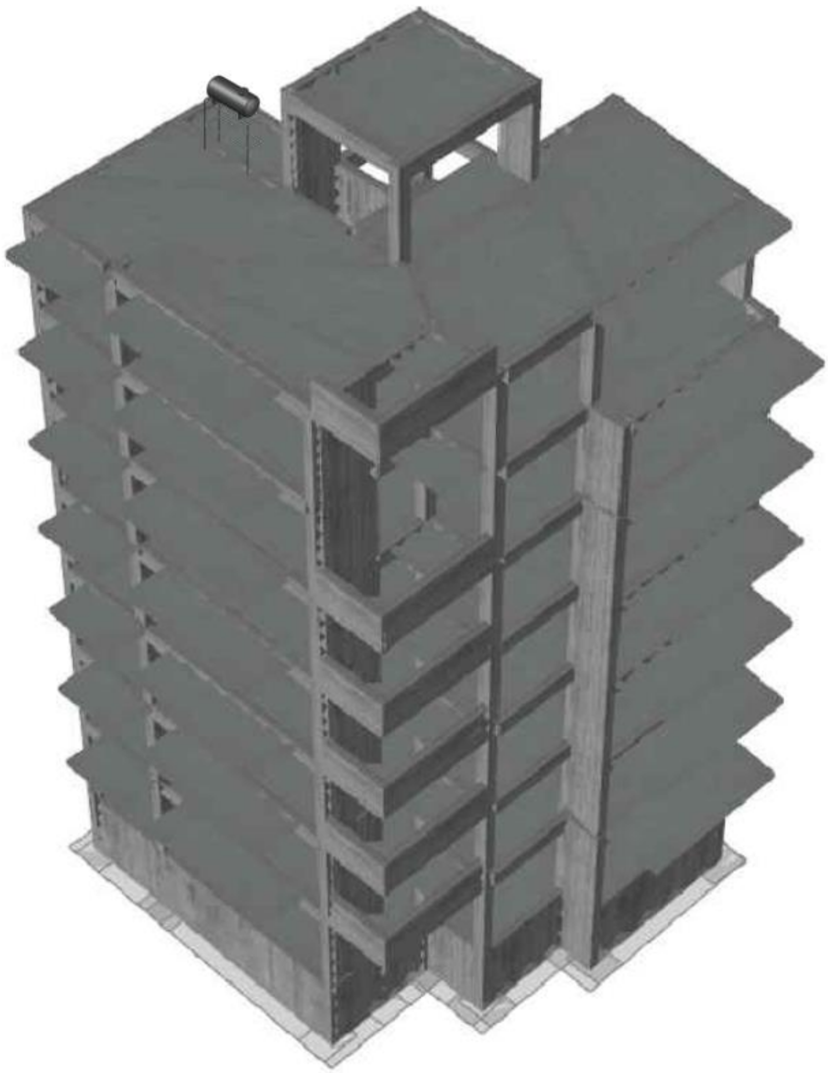


Figure 2.4 Position of the water tank on the roof of the building

It is important to study the water tank behavior, because if it collapses in a future earthquake, the water will run all over the roof and the tank could fall off the building causing serious damage.

According to the technical report, the load-bearing structure is made of reinforced concrete of C30/37 quality according to CEB and Concrete Technology Regulation and B500c class steel. The walls, columns and beams create spatial frames that safely support all the loads acting on the building. Full details are shown on the plans. The loads are finally transferred to the ground by means of a 60 cm thick raft foundation.

Class B soil (deposits of very dense sand, gravel or very hard clay, at least several tens of meters thick, characterized by a gradual improvement of the mechanical properties with depth), according to EN 1998-1[6], is taken. The bearing capacity of the soil for ultimate limit state is taken as 1.40 x σ_o = 1.40 x 200 = 280 kN/m^2.

As the building under study has importance class II and soil class B, no technical soil test is required in accordance with § 3.1, 3.2 of EN 1998-1[6] and it is allowed to assess the soil class and bearing capacity based on existing experience from adjacent structures founded on similar ground formations which have not shown significant subsidence and have shown good behavior in previous significant seismic actions.

Regarding the loads acting on the building, according to the technical report of the study, the following applies:

➔ **Permanent Loads**
- Self weight of reinforced concrete: 25 kN/m^3
- Exterior infills: 3.60 kN/m^2
- Interior infills: 2.10 kN/m^2
- Additional permanent load on the floor: 1.20 kN/m^2
- Additional permanent load on the roof of the building: 2.50 kN/m^2
- Self weight of water tank on the roof of the building: 2 tn

➔ **Live Loads**
- Live load on the floor: 2.00 kN/m^2
- Live load on the roof of the building: 2.00 kN/m^2
- Live load on the stairwell: 3.50 kN/m^2
- Live load on the balcony: 5.00 kN/m^2
- Vehicle live load (pilotis): 5.00 kN/m^2

➔ **Seismic Loads**
- Seismic zone: Z1 (ag = 0.16g)
- Importance class: II
- Soil class: B
- Behavior factor q: 3.5

It should be noted that the infill load has to be multiplied by the clear height of the story. For instance, if h_{clear} = 2.50 m, then the exterior infill load should be q = 3.60 x 2.50 = 9 kN/m. This load is applied to the supporting beams. There are no infills at the ground floor (pilotis), so no load is used there. In addition, there is a concrete barrier around the perimeter of the roof, 1.20 m in height and 25 cm in width, creating an additional distributed load on the beams equal to 7.50 kN/m. Finally, the self-weight of the tank (2 tn) is applied as linear load to the supporting beam.

The following standards were taken into account in the preparation of the study:

- ✓ Eurocode 1[25]
- ✓ Eurocode 2[1]
- ✓ Eurocode 8[6]

As already mentioned, C30/37 concrete and B500c steel were used. The characteristic values are f_{ck} = 30 MPa and f_{yk} = 500 MPa. However, according to KANEPE[3], mean values should be used for the calculation of chord rotation, strains, etc. So, according to Eurocode 2[1], the mean value of concrete is taken as f_{cm} = 38MPa and the mean value of steel as f_{ym} = 555.56 MPa (Table 2.1).

Table 2.1 Material properties

Material	**Name**	**Strength (MPa)**	
		Characteristic	**Mean**
Concrete	C30/37	30	38
Longitudinal Reinforcement	B500c	500	555.56
Transverse Reinforcement	B500c	500	555.56

Moreover, FEDRA[26] software was used for the calculation of compressive strength of infills. The typical brick dimensions are 90x90x190 mm with compressive strength f_b = 4.90 MPa. The compressive strength of infill according to Eurocode 6[27] is equal to:

$$f_k = K f_b^{0.7} f_m^{0.3} \tag{2.1}$$

where:

f_b = compressive strength of brick (4.90 MPa)

f_m = compressive strength of mortar (M5 category used, so 5.00 MPa)

K = coefficient equal to 0.50

So, the final result for the infills is taken as f_k = 2.46 MPa.

The building formwork drawings and reinforcement details are analytically presented in Appendix B.

2.2. Building Modelling

The modelling of the building was carried out with the Seismobuild program. Seismobuild can simulate the behavior of 3D reinforced concrete models under static and dynamic loads, as well as model with accuracy the sectional gradual insertion of structural elements in plasticity. It is capable of taking into consideration geometric and material nonlinearities due to its large amount of 3D elements, sections, concrete and steel materials, as well as FRP jackets.

SeismoBuild can model static loads including forces and displacements, permanent and incremental. The supported analysis types are nonlinear dynamic analysis, static pushover analysis and eigenvalue analysis.

Large displacements/rotations and large independent deformations relative to the frame element's chord (also known as P-Delta effects) are taken into account in Seismobuild, through the employment of a total co-rotational formulation. This formulation is based on an exact description of the kinematic transformations associated with large displacements and three-dimensional rotations of the beam-column member. This leads to the correct definition of the element's independent deformations and forces, as well as to the natural definition of the effects of geometrical nonlinearities on the stiffness matrix.

In Seismobuild, use is made of the so-called fiber approach to represent the cross-section behavior, where each fiber is associated with a uniaxial stress-strain relationship, the sectional stress-strain state of beam-column elements is then obtained through the integration of the nonlinear uniaxial stress-strain response of the individual fibers in which the section has been subdivided. The ideal number of section fibers, sufficient to guarantee an adequate reproduction of the stress-strain distribution across the element's cross-section, varies with the shape and material characteristics of the latter, depending also on the degree of inelasticity to which the element will be forced to, normally 150 fibers or more are employed (the discretization of a typical reinforced concrete cross-section is depicted, in the figure below).

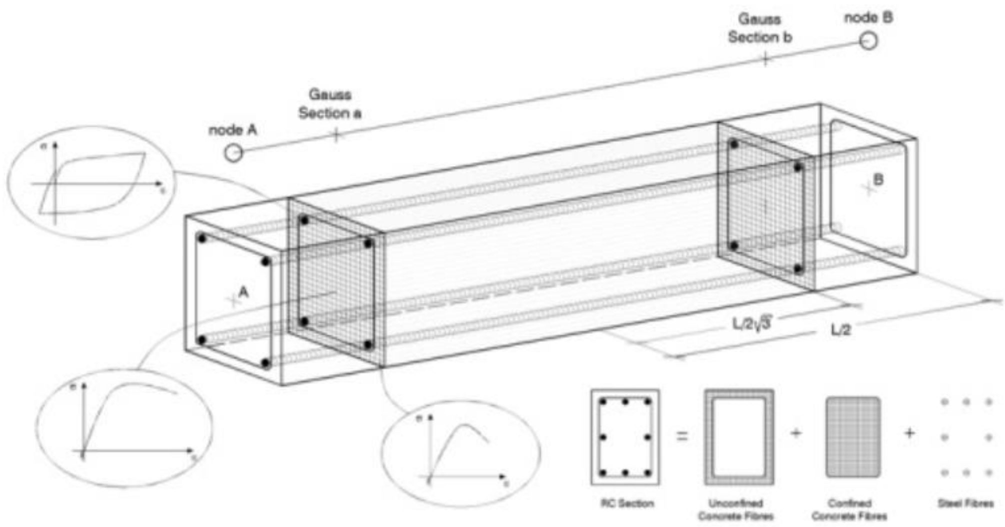


Figure 2.5 Discretization of a typical reinforced concrete cross-section in Seismobuild

The foundation is modelled through rigid supports, which implies the absence of significant differential displacements.

The columns and beams were simulated with “inelastic plastic-hinge force-based frame elements”. In contrast, walls were simulated with “inelastic force-based frame elements” The staircase was also simulated with “elastic frame elements”, in order to take into account its influence on the overall behavior of the structure. As for the materials, “Mander et al.” nonlinear concrete model and “Menegotto-Pinto” steel model were used. Finally, a model that included exterior infills as elements also utilized, using the “Crisafulli et al.2000”[28] method that concerns the simulation of a diagonal rod which is activated only during compression (Figure 2.6).

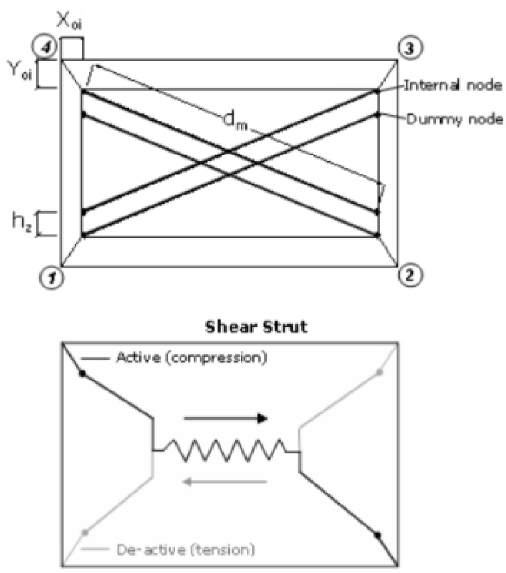
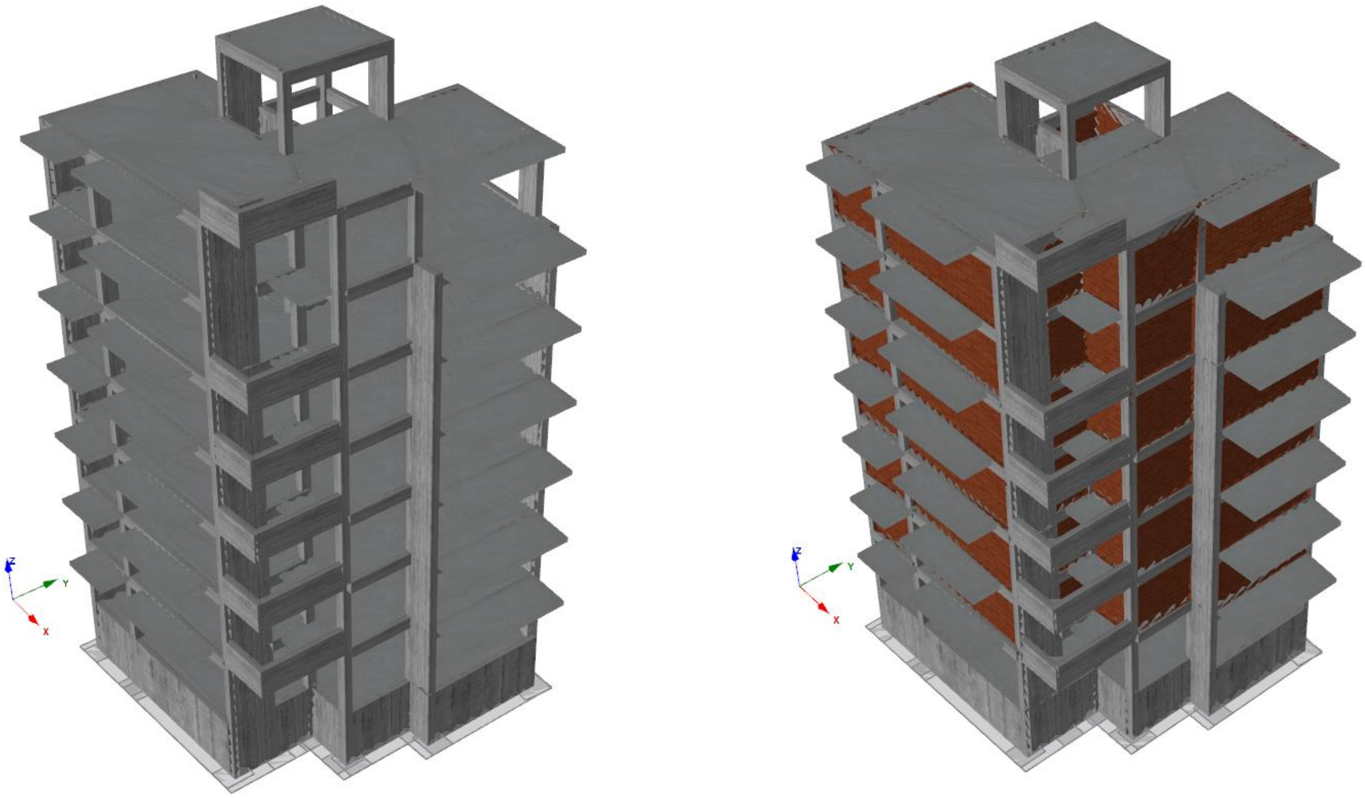


Figure 2.6 Implemented infill panel model

The user needs only to specify the effective depth of infill (t_{eff}), the compressive strength of brick and mortar and finally the specific weight. In this case we don't use the permanent load of 3.60 kN/m^2 as described in the first model, but the software calculates it automatically, so we must be careful to give the right data because the total weight of structure must be the same for both models. In this work, an 18 cm thick "infill element" was used (2 bricks of 90 mm width) with specific weight 15 kN/m^3. A 3D view and a plan view of the structure for both models are presented below.



(a) (b)

Figure 2.7 (a) 3D view of the building in Seismobuild (Without Infills), (b) 3D view of the building in Seismobuild (With Infills)

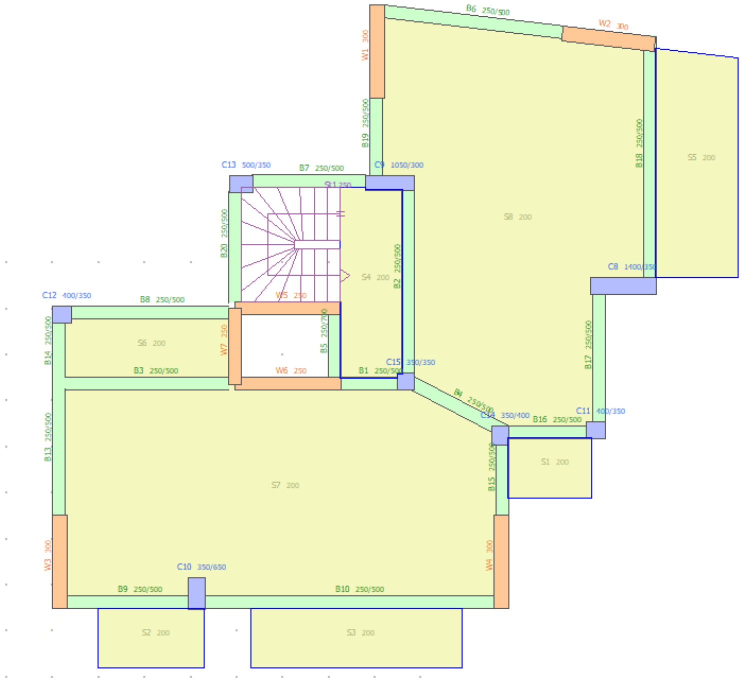


Figure 2.8 Typical floor plan of the building in Seismobuild [Without Infills] (Orange color: Walls, Blue color: Columns, Green color: Beams, Yellow color: Slabs)

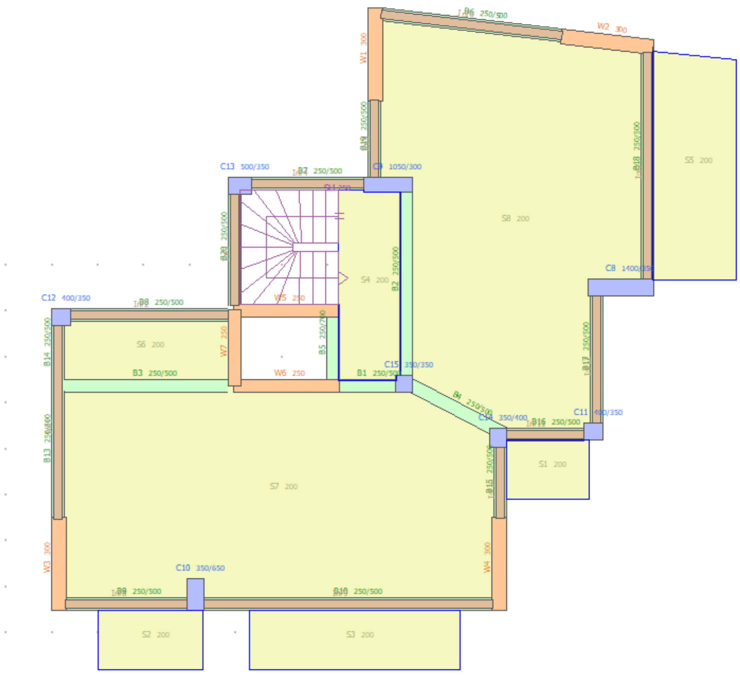


Figure 2.9 Typical floor plan of the building in Seismobuild [With Infills] (Orange color: Walls, Blue color: Columns, Green color: Beams, Yellow color: Slabs, Brown color: Infills)

With regard to the nonlinear parameters used in this work to evaluate the building, the following applies:

- Pushover analysis steps: 200
- Maximum interstory drift: 2%
- Maximum number of iterations of each step: 200
- Divergence Iteration: 200
- Maximum Tolerance: 1e+22
- Number of Eigenvalues: 70
- Convergence type: Displacement/Rotation
- Criterion of convergence: 1e-05

Frame Element Class (columns-beams)	FBPH
Frame Element Class (walls)	FB [User-Defined]
Include Geometric Nonlinearities (Frame Elements)	Yes
Rigid Links / Diaphragms Type	Penalty Functions
Rigid Links: Penalty Fuctions Exponent	1e17 [User-Defined]
Rigid Diaphragms: Penalty Fuctions Exponent	1e14 [User-Defined]
Convergence Type	Displacement/Rotation-based
Dspl. Based criterion	1,0000000E-005 [User-Defined]
Rotation based criterion	1,0000000E-005 [User-Defined]
Force Reference Value	-
Moment Reference Value	-
Maximum Number of Iterations:	200 [User-Defined]
Number of Stiffness Updates:	150 [User-Defined]
Divergence Iteration:	200 [User-Defined]
Maximum Tolerance:	1,0000E+022 [User-Defined]
Maximum Step Reduction:	0,0001 [User-Defined]
Minimum Number of Iterations:	1
Number of Eigenvalues	70 [User-Defined]
Concrete Material Model	con_ma
Steel Material Model	stl_mp
Maximum Total Interstorey Drift (%)	2,00
Pushover Analysis Steps	200 [User-Defined]

Figure 2.10 Nonlinear parameters used for the modelling of the building in Seismobuild

The software SAP2000[29] was used to model the water tank. The structure consists of two moment-restrained frames in both directions. The plan dimensions are 1.30x1.00 m with a total height of 1.50 m and regarding to the cross-sections, SHS 40x40x3 was used for all members with S275 grade steel (Figure 2.3). Moreover, pin support was used so as not to burden the beam of the building. Finally, an extra node was added, 0.60 m above the steel structure (center of the tank) to apply the weight of the water and also the acting seismic force. To be realistic, body constraints were used to connect the nodes of the steel structure with this extra node. (Figure 2.11)

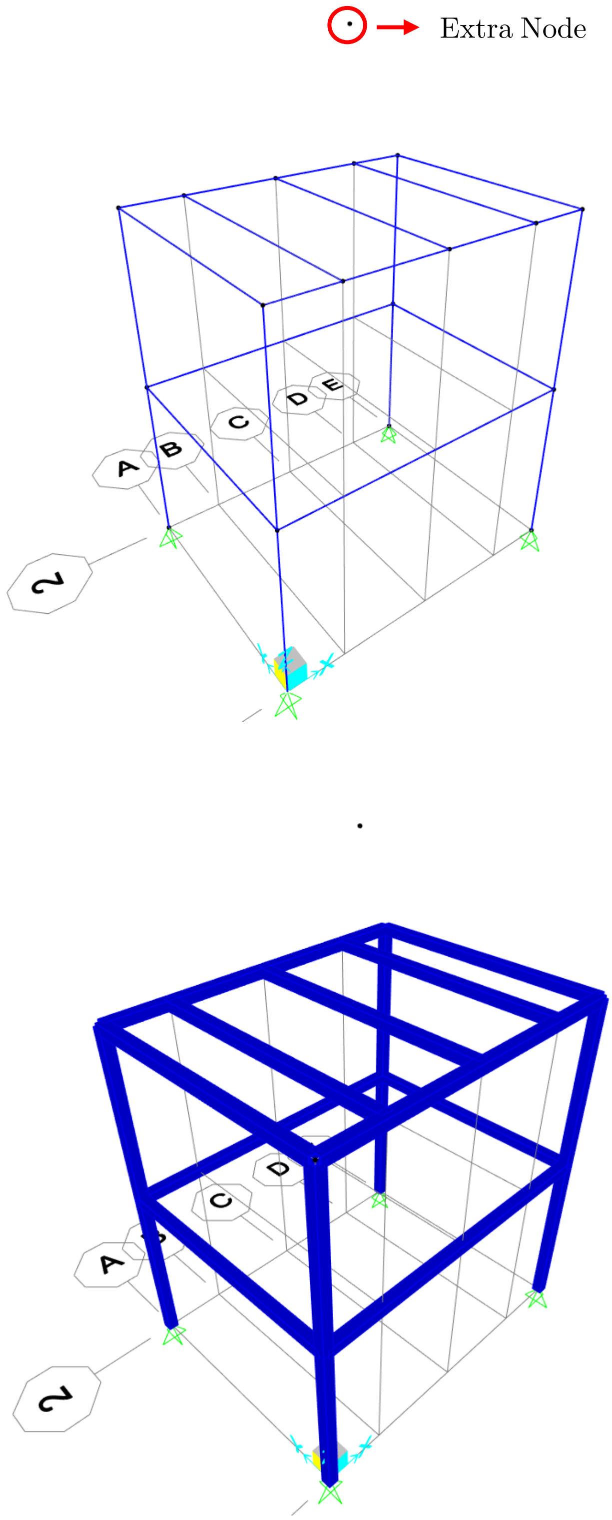


Figure 2.11 Model of the water tank on SAP2000 (Standard View – Extruded View)

Chapter 3

Application of Seismic Assessment Methods

3.1. Application of First Level Pre-Earthquake Assessment Method

As already mentioned in Chapter 1 (§ 1.2), the Inspection Report must first be determined. Thus, a structural type RCc category is derived due to the construction date associated with the corresponding design code for reinforced concrete buildings (E.A.K 2000[13], E.K.O.S 2000[17]). Table 3.1 summarizes all criteria affecting the "final structural rating of the building", where according to Table 1.3 the building has an overall score F.R. = 5.5, classified as "Low Priority".

Table 3.1 Inspection report (First Level Pre-Earthquake Assessment Method)

Parameter	**RCc**
Basic rating, depending on the Structural Type	8.0
Seismic Zone I	-0.5
Ground Type B (for A: -0.1)	-0.3
Without Anti-Seismic Regulation	-
Previous seismic loads, problems	-
Bad condition	-
Impact with adjacent buildings	-
Pilotis and/or Short Columns	-
Irregular infill arrangement in plan	-
High height	-0.5
Irregularity in elevation	-
Irregularity in plan	-0.5
Torsion (intense)	-0.5
Operating intensity	0.2
Number of users 10 - 99	-0.4

3.2. Application of Second Level Pre-Earthquake Assessment Method

3.2.1. Application of Dritsos S. Method

In the initial phase of this methodology[18], the elements of the structure that have a decisive influence on the seismic behavior of the building were examined, summarized in thirteen (13) criteria. The criteria are graded with a whole number β_i from one (1) to five (5) [except the first three (3) criteria, where it begins with zero (0)], with one (1) being the highest level of influence equivalent to a reduction in the building's seismic performance and five (5) being the lowest.

- **1st Criterion (Static failure damage)**

There are no damages to the columns and joints of the building, therefore the grade of criterion 1 is equal to $\beta_1 = 5$.

- **2nd Criterion (Reinforcement oxidation)**

As this is a newly constructed building, no rebar oxidation has occurred, hence the grade $\beta_2 = 5$.

- **3rd Criterion (Relative axial force on ground floor)**

First, the relative axial force v^i_d needs to be calculated for all ground floor members i. The following applies:

- $\beta_i = 1$, if $0.65 \leq v^i_d \leq 0.75$
- $\beta_i = 2$, if $0.50 \leq v^i_d < 0.65$
- $\beta_i = 3$, if $0.40 \leq v^i_d < 0.50$
- $\beta_i = 4$, if $0.30 \leq v^i_d < 0.40$
- $\beta_i = 5$, if $v^i_d < 0.30$

Then, the average relative axial force v_d need to be calculated. In this case, the classification is as follows:

- $\beta = 1$, if $0.45 \leq v_d \leq 0.55$
- $\beta = 2$, if $0.35 \leq v_d < 0.45$
- $\beta = 3$, if $0.25 \leq v_d < 0.35$
- $\beta = 4$, if $0.15 \leq v_d < 0.25$
- $\beta = 5$, if $v_d < 0.15$

The final grade is the lower value between the two criteria above. Table 3.2 shows the results obtained for this building.

Table 3.2 Relative axial force of the ground floor members

Member	**A_i (m^2)**	**Axial Force (kN)**	**Relative Axial Force**	**Grade**
wall W1	0.600	916.80	0.059	5
wall W2	0.600	1234.62	0.079	5
wall W3	0.600	1062.54	0.068	5
wall W4	0.600	1639.37	0.105	5
wall W5	0.569	997.04	0.067	5
wall W6	0.569	1042.29	0.070	5
wall W7	0.413	727.07	0.068	5
column C8	0.490	1252.49	0.098	5
column C9	0.315	830.33	0.101	5
column C10	0.228	1045.78	0.176	5
column C11	0.140	406.65	0.111	5
column C12	0.140	438.17	0.120	5
column C13	0.175	347.61	0.076	5
column C14	0.140	416.76	0.114	5
column C15	0.123	282.96	0.089	5

The average relative axial force is equal to $v_d = 0.093$, so $\beta_3 = 5$.

4^{th} Criterion (Regularity in plan)

For buildings with an oblong (rectangular) plan, the side lengths, L_{max} and L_{min}, are measured in both directions and the ratio $\lambda = L_{max}/L_{min}$ is calculated.

- If $\lambda < 4.0 \rightarrow \beta_4 = 5$
- If $\lambda \geq 8.0 \rightarrow \beta_4 = 1$
- Intermediate grades shall be selected at the discretion of the engineer

In buildings with a complex floor plan, these things need to be determined:

- the cumulative area of the ΣA_E of the recesses
- the area of the largest recess, $A_{E,max}$, and the area of the floor plan, A_{tot}.

In this case, the classification is as follows:

- If $\Sigma A_E < 0.25\ A_{tot}$ or $A_{E,max} < 0.15\ A_{tot} \rightarrow \beta_4 = 5$
- If $\Sigma A_E \geq 0.40\ A_{tot}$ or $A_{E,max} \geq 0.25\ A_{tot} \rightarrow \beta_4 = 1$
- Intermediate grades shall be selected at the discretion of the engineer

So, for this building, according to Figure 3.1, the total floor plan area is equal to A_{tot} = 121.59 m^2, the total area of all the recesses is ΣA_E = 16.74 m^2 (13.77% A_{tot}) and the largest recess area is $A_{E,max}$ = 5.63 m^2 (4.63% A_{tot}). Moreover, the ratio $\lambda = L_{max}/L_{min}$ is equal to 1.94 (12/6.2) in the x-direction and 2.00 (13.09/6.55) in the y-direction. Therefore, the final grade is β_4 = 5 in both directions.

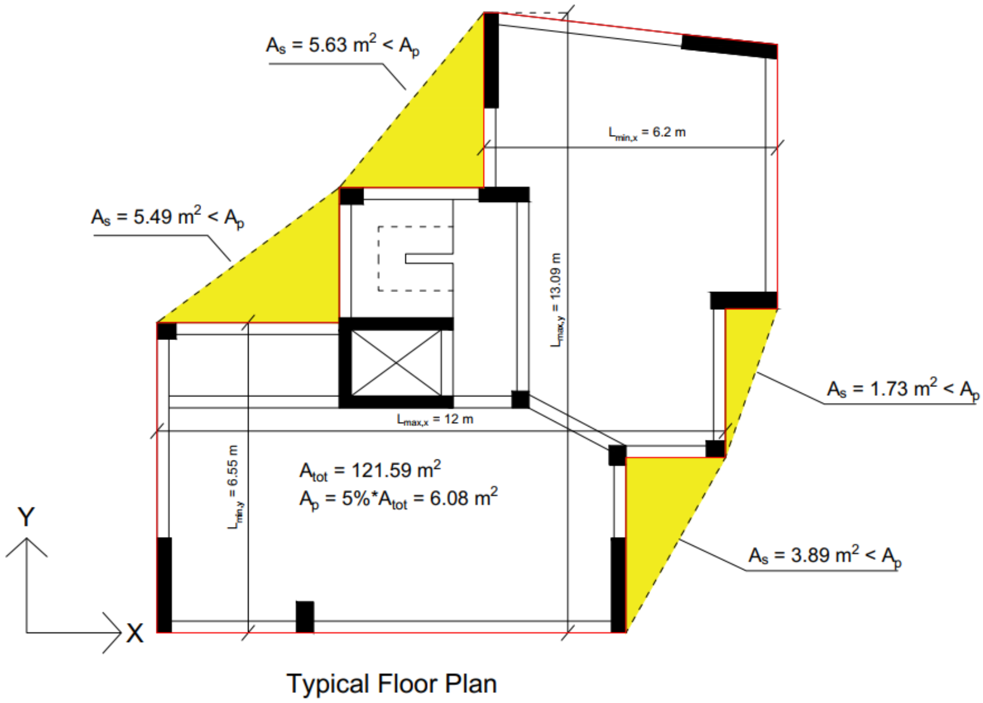


Figure 3.1 Typical floor plan of the building for the calculation of the area recesses

5th Criterion (Stiffness distribution in plan - Torsion)

The asymmetric distribution of the elements that contribute to the lateral stiffness of the structure and the absorption of seismic actions usually leads to an uneven distribution of the seismic intensity with the direct consequence of the occurrence of significant damage to the most affected structural elements. This problem is mainly found in buildings with non-symmetrical perimeter walls or with eccentrically placed strong staircase/elevator cores. The seismic response of these buildings is often torsional (usually a combination of translational and torsional oscillations). Torsion is generally considered to be an undesirable vibration mechanism, as it results in particular increased “loading” on the structural elements (columns) at the perimeter of the building, which may not have been foreseen in the design. Due to the torsion of the diaphragms, these elements develop large displacements and are likely to fail due to the increased (compared to design) seismic intensity in them or due to second-order phenomena.

The steps to determine the grade for this criterion are the following:

- Calculation of the center of mass (CM) on the ground floor according to the equation:

$$x_M = \frac{\sum N_{Sd,i} x_{C,i}}{\sum N_{Sd,i}}, \quad y_M = \frac{\sum N_{Sd,i} y_{C,i}}{\sum N_{Sd,i}} \tag{3.1}$$

where:

N_{Sd} = axial force of the ground floor members

$x_{C,i}$, $y_{C,i}$ = coordinates of the vertical members (reference point: edge of wall W3 (Figure 2.8))

- Calculation of the vertical elements stiffness, $K_{Cx,i}$, $K_{Cy,i}$ (per direction), at ground level according to the equation:

$$K_C = \frac{12\left(EI\right)_{eff}}{H^3\left(1+n\right)} a_k \tag{3.2}$$

where:

$(EI)_{eff}$ = effective stiffness of the element (§ 4.4.1.4 KANEPE[3])

H = story height

n = coefficient equal to:

$$n = \begin{cases} \dfrac{30\left(EI\right)_{eff}}{E_C A_w H^2}, & \text{walls} \\ 0, & \text{columns} \end{cases} \tag{3.3}$$

where:

A_w = cross-section area

E_C = Young's Modulus of concrete

α_k = coefficient equal to:

$$a_k = \frac{k_c + \sum k_{ba}}{4k_c + \sum k_{ba}} \tag{3.4}$$

where:

$k_c = (EI)_{eff}/H$, stiffness index of the vertical member (column or wall)

Σk_{ba} = the sum of the stiffness index $(EI)_{eff}/L_b$ of the relevant beams at the head of the vertical member in the tested direction (§ 4.4.1.4 KANEPE[3])

In addition, it is necessary to calculate the stiffness of the infills, but in this case there are no infills, since the base floor has parking areas.

- Calculation of the center of stiffness or center of rigidity (CS or CR) on the ground floor by the following equation:

$$x_{CR} = \frac{\sum K_{Cy,i} x_{C,i}}{\sum K_{Cy,i}}, \qquad y_{CR} = \frac{\sum K_{Cx,i} y_{C,i}}{\sum K_{Cx,i}} \tag{3.5}$$

where:

$K_{Cx,i}$, $K_{Cy,i}$ = vertical members stiffness according to Eq. (3.2)

x_C, y_C = coordinates of the elements (reference point: edge of wall W3 (Figure 2.8))

It is important to understand that in order to calculate the x-coordinate of CR, you need to take the stiffness of vertical elements in the y-direction (K_{Cy}) and vice versa.

- Calculation of the eccentricities per direction, e_x and e_y, and their relative values, ε_x and ε_y, as follows:

$$\begin{aligned} e_x &= \left| x_{CR} - x_{CM} \right| \rightarrow \varepsilon_x = e_x / L_x \\ e_y &= \left| y_{CR} - y_{CM} \right| \rightarrow \varepsilon_y = e_y / L_y \end{aligned} \tag{3.6}$$

where:

L_x, L_y = the maximum side length of the building per direction as described in Figure 3.1

The classification is as follows:

- If $\varepsilon < 0.05 \rightarrow \beta_5 = 5$
- If $\varepsilon \geq 0.30 \rightarrow \beta_5 = 1$
- Intermediate grades shall be selected at the discretion of the engineer

So, taking into account the axial forces from Table 3.2 and Eq. (3.1), the coordinates of the center of mass on the ground floor are:

$$\boxed{x_{CM} = 6.98 \text{ m}, \; y_{CM} = 5.71 \text{ m}}$$

With regard to the coordinates of the center of rigidity, the stiffness of vertical elements must first be determined. The results for this building are shown in the table below.

Table 3.3 Stiffness of vertical elements

Member	$\mathbf{x_C}$ **(m)**	$\mathbf{y_C}$ **(m)**	$\mathbf{\Sigma K_{ba,x}}$	$\mathbf{\Sigma K_{ba,y}}$	$\mathbf{K_{Cx}}$ **(kN/m)**	$\mathbf{K_{Cy}}$ **(kN/m)**
wall W1	7.050	12.090	11379.59	27944.90	8387.27	187870.30
wall W2	12.090	12.377	11379.59	10436.40	183779.22	11858.29
wall W3	0.150	1.000	16970.23	15642.12	9300.04	186037.22
wall W4	9.750	1.000	7138.27	29013.61	7618.63	188028.96
wall W5	5.113	6.525	13035.10	106881.31	206883.43	12144.06
wall W6	5.113	4.875	48468.85	88143.36	211681.15	11604.80
wall W7	3.975	5.700	25527.07	18737.96	6424.68	99383.22
column C8	12.400	7.015	0.00	25220.29	132655.49	12940.76
column C9	7.325	9.250	18355.55	37852.89	52014.43	8875.49
column C10	3.125	0.325	24108.50	0.00	7559.72	13276.43
column C11	11.800	3.875	26453.59	16118.67	6737.76	4782.49
column C12	0.200	6.375	13035.10	33940.45	5335.89	6063.67
column C13	4.100	9.225	18355.55	18737.96	9428.05	5835.26
column C14	9.725	3.750	46205.61	39427.67	6603.30	7648.04
column C15	7.675	4.925	55728.91	21656.84	7770.01	5847.75

It is clear that there is a large difference in stiffness in the walls depending on the direction, which is normal. More specifically, in the x-direction there are three (3) walls (W2, W5, W6) and in y-direction there are four (4) walls (W1, W3, W4, W7). So, according to Eq. (3.5), the coordinates are:

$$\boxed{x_{CR} = 5.66 \text{ m}, \ y_{CR} = 7.46 \text{ m}}$$

So, the eccentricities, e_x and e_y, and their relative values, ε_x and ε_y, are:

$$\boxed{e_x = 1.32 \text{ m}, \ e_y = 1.76 \text{ m}}$$

$$\boxed{\varepsilon_x = 0.110 \ , \ \varepsilon_y = 0.134}$$

Taking all this into account, it is obvious that critical is an earthquake in the x-direction because $\varepsilon_y > \varepsilon_x$ (more torsion), so the final grade is equal to $\beta_5 = 3$ in the x-direction and $\beta_5 = 4$ in the y-direction, because the escalation for the grade β_5 is 0.05, 0.13, 0.21 and 0.30.

As shown in Figure 3.2, the center of mass and the center of stiffness is inside the stairwell and there are not very significant torsional effects because the distance between these two (2) points is small.

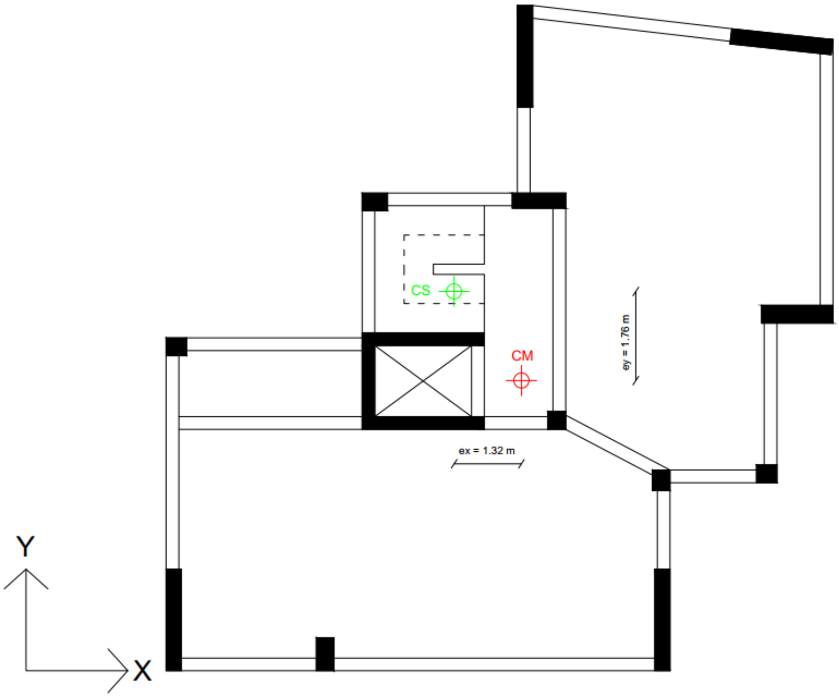


Figure 3.2 Center of mass and center of stiffness of the building

6th Criterion (Regularity in elevation)

The area of the first six (6) stories is equal to A_{tot} = 121.59 m^2, where in the last story A_{tot} = 115.87 m^2, which is not much difference, so $\beta_6 = 5$.

7th Criterion (Stiffness distribution over the height of the building)

As it is known, the stiffness of the elements involved in the absorption of seismic actions should remain unchanged from story to story, in order to ensure the best possible height distribution of seismic displacements. The problem is identified when the stiffness of a story is significantly less than that of its neighbors (below and above). Such rapid decreases in stiffness are mostly caused by special demands on the use of the story (e.g. ground floor with parking space in a residential building) and in Greece they occur with the particularly widespread “pilotis” type ground floor configuration.

For the evaluation of the criterion, the difference in stiffness between adjacent stories is taken into account. So, the first step is the calculation of vertical elements stiffness, $K_{Cx,i}$, $K_{Cy,i}$ (per direction), of all the stories (except penthouse) according to Eq. (3.2). The only difference is the α_k coefficient, where Eq. (3.4) applies only to the ground floor. On the other stories, is given by the following equation:

$$a_k = \frac{\sum k_{ba} + \sum k_{bb}}{4k_c + \sum k_{ba} + \sum k_{bb}} \tag{3.7}$$

where:

Σk_{ba} = the sum of the stiffness index $(EI)_{eff}/L_b$ of the relevant beams at the head of the vertical member in the tested direction

Σk_{bb} = the sum of the stiffness index $(EI)_{eff}/L_b$ of the relevant beams at the foot of the vertical member in the tested direction

Furthermore, in the other stories, the stiffness of the infills need to be calculated according to the equation:

$$K_{\inf} = \frac{E_{\inf} A_{\inf}}{L_d} \cos^2(a) \tag{3.8}$$

where:

E_{inf} = Young's Modulus of the infill (2.46 GPa)

L_d = length of the equivalent rod (§ 7.4.1 KANEPE[3]) (Figure 2.6)

A_{inf} = cross-section area of the equivalent rod

α = angle of inclination of the rod to the horizontal

According to § 7.4.1 of KANEPE[3], the cross-section area of the equivalent rod is equal to:

$$A_{\inf} = bt_{eff} \tag{3.9}$$

where:

t_{eff} = infill thickness (in our case two bricks of 90 mm, so 180 mm)

b = width of the infill, equal to $0.15L_d$

The next step is the calculation of the story stiffness K_{tot} by adding the stiffness of the vertical elements and infills and the last step is the calculation of the percentage stiffness difference ΔK_{tot} between stories in each direction using the equation:

$$\Delta K_{tot}(\%) = \frac{K_{tot,up} - K_{tot,down}}{K_{tot,down}} \cdot 100 \tag{3.10}$$

The total stiffness of the stories and the percentage stiffness difference are presented in the Table 3.4. and in Figure 3.3.

Table 3.4 Percentage stiffness difference between stories

Floor	$K_{tot,x}$ (kN/m)	$K_{tot,y}$ (kN/m)	$\Delta K_{tot,x}$ (%)	$\Delta K_{tot,y}$ (%)
1	862179.06	762196.72	-	-
2	349871.75	347598.37	59.42	54.40
3	349871.75	347598.37	0.00	0.00
4	349871.75	347598.37	0.00	0.00
5	349871.75	347598.37	0.00	0.00
6	349871.75	347598.37	0.00	0.00
7	352295.23	341685.24	0.69	1.70

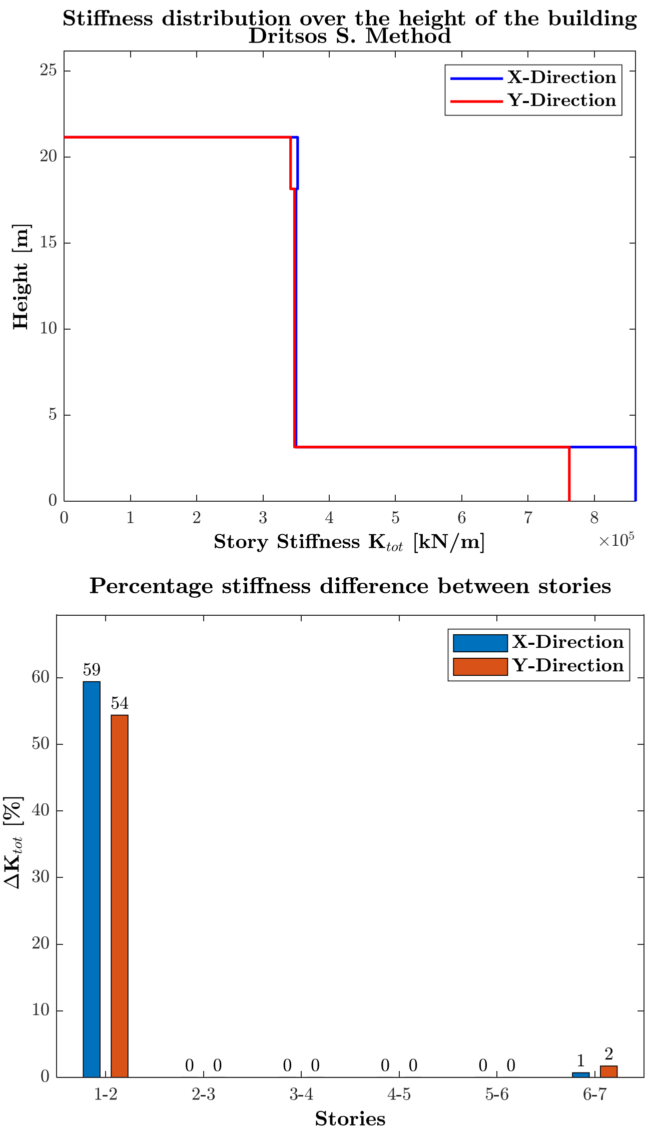


Figure 3.3 Stiffness distribution over the height of the building

As shown in the figure below, the maximum stiffness difference occurs between the first and second story and is almost 60% in both directions. This is not normal but according to the sub-assemblage method[30] used by Dritsos, it may not be valid for all buildings. More specifically, it is applicable for MRF with first story column fixed at its base and not for braced MRF and MRF with structural walls. So, for this building is not the right method because it has a lot of walls, but the engineer has to use it because of the legislation. Therefore, the final grade is equal to $\beta_7 = 1$ in both directions, because $\Delta K_{tot} > 50$ %.

The reason for this problem is the α_k coefficient which is different between the first story and the others (Eq. (3.4), Eq. (3.7)). So in the case of the walls, this coefficient is much smaller on the other stories than on the ground floor resulting in lower stiffness according to Eq. (3.2).

The best method is the lateral force-deformation method[30], where results of structural analysis of building subjected to design earthquake loads are used to estimate story stiffness as the ratio of cumulative story shear force to the interstory lateral displacement. (Figure 3.4)

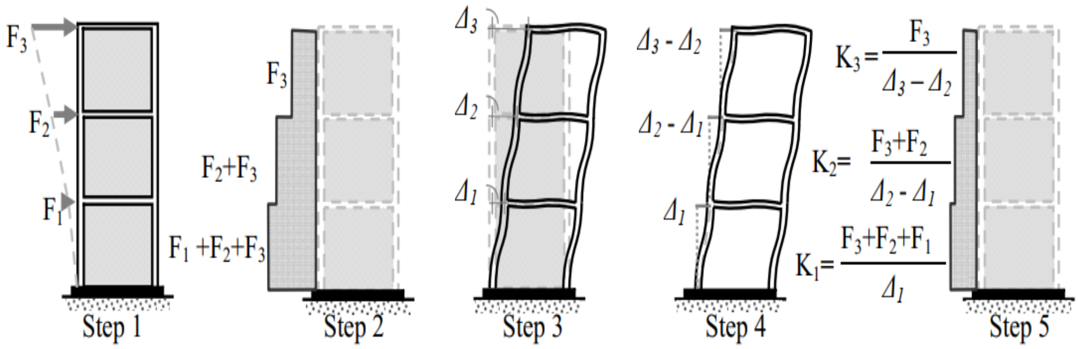


Figure 3.4 Method to estimate the lateral story stiffness

The only drawback of this method is that it needs a software in order to calculate the story stiffness, but it is applicable in all buildings.

8th Criterion (Mass distribution over the height of the building)

Conventional seismic design is based on the assumption of a practically uniform mass distribution in height. The mass difference of the adjacent stories is required to be evaluated as part of the secondary seismic control procedure. The classification is as follows:

- If $\Delta M_{tot} < 20$ % $\rightarrow \beta_8 = 5$
- If $\Delta M_{tot} > 50$ % $\rightarrow \beta_8 = 1$

According to Table 3.5, $\Delta M_{tot} = 4.26$ % → $\beta_8 = 5$. The penthouse is not considered as a story. The total mass of the building is W = 12640.48 kN (1288.97 tn).

Table 3.5 Percentage mass difference between stories

Floor	**Mass (tn)**	**ΔM_{tot} (%)**
1	182.92	-
2	181.17	0.96
3	181.14	0.02
4	181.13	0.01
5	181.13	0.00
6	176.74	2.43
7	184.26	4.26

9th Criterion (Short columns)

The degree of influence of short columns shall be calculated at each level and per direction. The degree of influence of short columns shall be determined as the centrobaric mean value of the degrees β_l assigned to each column according to the l/h ratio.

More specifically, for each column, a grade β_l is assigned depending on its relative length. The contribution to the structure shall be taken into account by means of corresponding weight factors, w.f., as follows:

- For $l/h \leq 2 \rightarrow \beta_l = 1$ and w.f. = 5
- For $2 < l/h \leq 3 \rightarrow \beta_l = 2$ and w.f. = 4
- For $3 < l/h \leq 4 \rightarrow \beta_l = 3$ and w.f. = 3
- For $4 < l/h \leq 5 \rightarrow \beta_l = 4$ and w.f. = 2
- For $l/h > 5 \rightarrow \beta_l = 5$ and w.f. = 1

If n is the number of columns at the story under consideration and:

- n_1 the number of columns with $\beta_l = 1$
- n_2 the number of columns with $\beta_l = 2$
- n_3 the number of columns with $\beta_l = 3$
- n_4 the number of columns with $\beta_l = 4$
- n_5 the number of columns with $\beta_l = 5$

For structures without walls, the classification of the criterion per story i shall be determined as follows:

$$\overline{\beta^i} = \frac{n_1 \cdot 1 \cdot 5 + n_2 \cdot 2 \cdot 4 + n_3 \cdot 3 \cdot 3 + n_4 \cdot 4 \cdot 2 + n_5 \cdot 5 \cdot 1}{n = 5n_1 + 4n_2 + 3n_3 + 2n_4 + 1n_5} \tag{3.11}$$

For structures with walls, the grade per story i is a function of the degree a_T (Eq. (1.5)). In this case, the classification is as follows:

- If $a_T < 0.10 \rightarrow \beta_9{}^i = \overline{\beta}$
- If $a_T \geq 0.50 \rightarrow \beta_9{}^i = 5$
- If $0.10 \leq a_T < 0.50$, then the grade is calculated according to equation:

$$\beta_9{}^i = \overline{\beta^i} + \frac{a_T\left(5 - \overline{\beta^i}\right)}{0.6} \leq 5 \tag{3.12}$$

The final grade for this criterion, β_9, per direction, is the lowest value between the stories, which means:

$$\beta_9 = \min(\beta_9{}^i),\ i = 1, 2, 3, 4, 5, 6, 7$$

In this building, the degree $a_T = 0.25$ in the x-direction and $a_T = 0.37$ in the y-direction (for the V_{Ri} see Table 3.8), so the final grade for this criterion is equal to:

$$\boxed{\beta_{9,x} = 3.76,\ \beta_{9,y} = 4.77}$$

- **10th Criterion (Vertical discontinuities)**

The existence of severe vertical discontinuities in columns and walls shall be checked. In this structure, there is an eccentricity (e_x, e_y) along the axis of two vertical elements in the 7th floor (wall W2, column C8) (Figure 3.5).

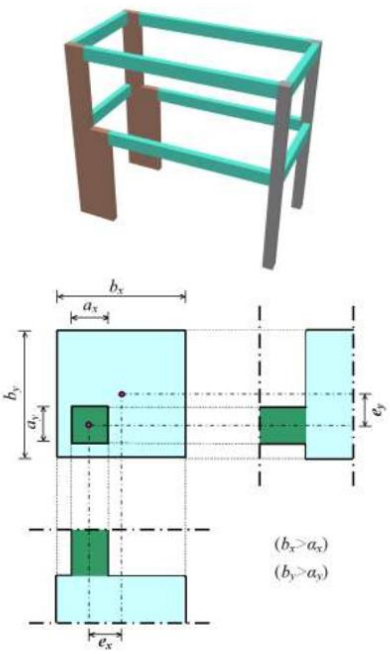


Figure 3.5 Illustration of the vertical discontinuity according to this method

The criterion is classified according to the table below depending on the eccentricity e between vertical elements.

Table 3.6 Classification of the 10th criterion depending on the eccentricity between vertical elements

Grade	Eccentricity
1	$e_{x,y} > 0.35b_{x,y}$
2	$0.25b_{x,y} < e_{x,y} \leq 0.35b_{x,y}$
3	$0.15b_{x,y} < e_{x,y} \leq 0.25b_{x,y}$
4	$0.05b_{x,y} < e_{x,y} \leq 0.15b_{x,y}$
5	$e_{x,y} \leq 0.05b_{x,y}$

In the x-direction, the eccentricity in column C8 is $e = 0.525$ m $> 0.35 \cdot 1.40 = 0.49$, so the final grade is $\beta_{10} = 1$.

In the y-direction, the eccentricity in wall W2 is $e = 0.053$ m $< 0.25 \cdot 0.30 = 0.075$, so the final grade is $\beta_{10} = 3$.

11th Criterion (Path and Transfer of forces in the building)

A complete system for absorbing horizontal forces, which forms an integrated force path between the foundation, the diaphragms and the remaining elements of the load-bearing structure, is an extremely important asset for the seismic behavior of a building. If there is a discontinuity in the force path, then the building is disadvantaged in terms of its overall seismic capacity, regardless of the strength of its individual elements.

There are two criteria affecting the classification of this criterion. The first one evaluates the interaction between the walls and the diaphragm (grade B_a) and the second one evaluates the existence of clear frame planes (grade B_b).

As for the first grade, B_a, the walls are fully connected with the diaphragm with beams on all sides, so $B_a = 5$. On the other hand, there are not clear frame planes, so $B_b = 3$.

The final grade is determined according to degree a_T (Eq. (1.5)) as follows:

- If $a_T > 0.80 \rightarrow \beta_{11} = B_a$
- If $a_T < 0.20 \rightarrow \beta_{11} = B_b$
- For intermediate values we have:
    - If $a_T = 0.60 \rightarrow \beta_{11} = 2/3\ B_a + 1/3\ B_b$
    - If $a_T = 0.40 \rightarrow \beta_{11} = 1/3\ B_a + 2/3\ B_b$

- For different values of a_T there is linear interpolation between the above limits

As mentioned above, $a_T = 0.25$ in the x-direction and $a_T = 0.37$ in the y-direction, so the final grade for this criterion is equal to:

$$\boxed{\beta_{11,x} = 3.17,\ \beta_{11,y} = 3.55}$$

12th Criterion (Adjacent Buildings)

The criterion considers the adverse interaction of adjacent buildings during an earthquake, mainly based on the possibility of an adverse collision of the building with its neighbors, due to their out-of-phase movement.

In this structure, there is a seismic joint 15 cm thick in both directions (Figure B.2), so $\beta_{12} = 5$.

13th Criterion (Malfunctions - Injuries)

It is a newly constructed building, so there are not any problems. As a result, $\beta_{13} = 5$.

Table 3.7 Final grade of the criteria that affect the load-bearing capacity of the building

Order Number	Criterion	Grade β_i		Weight Factor σ_i
		X-Direction	Y-Direction	
1	Static failure damage	5.00	5.00	0.10
2	Reinforcement oxidation	5.00	5.00	0.10
3	Relative axial force on ground floor	5.00	5.00	0.05
4	Regularity in plan	5.00	5.00	0.05
5	Stiffness distribution in plan - Torsion	3.00	4.00	0.10
6	Regularity in elevation	5.00	5.00	0.05
7	Stiffness distribution over the height of the building	1.00	1.00	0.15
8	Mass distribution over the height of the building	5.00	5.00	0.05
9	Short columns	3.76	4.77	0.15
10	Vertical discontinuities	1.00	3.00	0.05
11	Path and Transfer of forces in the building	3.17	3.55	0.05
12	Adjacent Buildings	5.00	5.00	0.05
13	Malfunctions - Injuries	5.00	5.00	0.05

For the calculation of the reduction factor β that has a decisive influence on the seismic behavior of the building, Eq. (1.3) is used, resulting in:

$$\boxed{\beta_x = 0.75,\ \beta_y = 0.82}$$

Table 3.8 shows the shear strength of each vertical element, calculated using the relationship (1.6) for each direction of the building.

Table 3.8 Shear strength of the vertical elements on the ground floor

Member	**X-Direction**				**Y-Direction**			
	V_{Rd}(kN)	**M_y (kNm)**	**V_{Ri} (kN)**	**Check**	**V_{Rd}(kN)**	**M_y (kNm)**	**V_{Ri} (kN)**	**Check**
wall W1	560.44	291.94	220.33	Flexure	1225.08	2562.64	242.33	Flexure
wall W2	1246.12	2584.27	284.77	Flexure	571.60	341.45	257.70	Flexure
wall W3	536.70	295.61	223.11	Flexure	858.49	2479.27	234.45	Flexure
wall W4	753.17	347.13	261.98	Flexure	1696.46	2959.93	279.90	Flexure
wall W5	1902.92	3300.11	312.07	Flexure	663.20	291.60	220.08	Flexure
wall W6	1336.69	3087.97	292.01	Flexure	658.77	277.28	209.27	Flexure
wall W7	737.98	250.55	189.09	Flexure	1382.51	2158.96	179.54	Flexure
column C8	1358.71	2865.37	1358.71	Shear	681.70	516.02	389.45	Flexure
column C9	665.77	1413.38	665.77	Shear	488.66	277.32	209.30	Flexure
column C10	468.29	297.01	224.16	Flexure	801.29	688.17	519.38	Flexure
column C11	345.10	206.36	155.74	Flexure	290.48	171.54	129.47	Flexure
column C12	361.48	211.33	159.49	Flexure	303.81	175.63	132.55	Flexure
column C13	375.25	332.44	250.90	Flexure	324.78	210.69	159.01	Flexure
column C14	283.97	171.43	129.38	Flexure	346.03	206.22	155.64	Flexure
column C15	282.31	174.32	131.56	Flexure	272.56	174.32	131.56	Flexure

Then, the total shear strength V_R at the base of the building was calculated, obtained by relations (1.2), (1.4) and using Table 1.5, where for the presence of columns and walls, the coefficients are $\alpha_1 = 0.7$ and $\alpha_2 = 0.85$. Note that the walls are treated as columns on their weak axis (three (3) walls in the x-direction [W2, W5, W6] and four (4) in the y-direction [W1, W3, W4, W7]). So, the total shear strength is equal to:

$$\boxed{V_{R0,x} = 3534.67\text{ kN},\ V_{R0,y} = 2555.16\text{ kN},\ V_{Rx} = 2631.70\text{ kN},\ V_{Ry} = 2091.50\text{ kN}}$$

For the determination of the seismic demand V_{req}, the relation (1.1) was used according to Appendix A. The coefficient of behavior is equal to q = 2.3 according to Table A.3 for unfavorable presence of walls and without damage to primary elements.

The eigenperiod values for the two directions were obtained from the Modal Analysis using cracked sections and are equal to $T_x = 0.93$ sec and $T_y = 0.90$ sec, while the empirical is determined according to Appendix A and is equal to $T_{exp} = 0.81$ sec, for $h = 21.15$ m. The total mass of the building is $M = 1288.97$ tn. The seismic demand is calculated as follows:

- $V_{req,x} = V_{req,y} = 1627.84$ kN → Empirical
- $V_{req,x} = 1418.77$ kN, $V_{req,y} = 1466.06$ kN → Modal Analysis

The empirical method was chosen because it is the critical one and is also described by Dritsos as the most appropriate. From the equations (1.34) and (1.35), the "Priority Index λ" was determined as follows:

$$\boxed{\lambda_x = 0.65,\ \lambda_y = 0.74 \rightarrow \lambda = 74}$$

The final step of this method is to determine the seismic category (K) of the building, which is practically the maximum assessment target that the building can achieve for performance level "Significant Damage". According to Eq. (1.36), the coefficient δ for this building is equal to:

$$\boxed{\delta = 1.361}$$

So, taking into account Table 1.6, the seismic category of this building is $K1^+$, corresponding to an earthquake with 5% probability of exceedance (poe) in 50 years, which is a great result for a newly constructed building.

3.2.2. Application of Vougioukas E. Method

As described in Chapter 1 (§ 1.3.2), this method is a rapid assessment of the seismic capacity of existing buildings. The seismic capacity of the building per direction is obtained by adding the corresponding shear strength of all the elements together. So, taking the shear strength V_{Ri} of each element from Table 3.8, the final results are:

$$\boxed{V_{Rx} = 4859.06 \text{ kN},\ V_{Ry} = 3449.61 \text{ kN}}$$

These results don't show anything but only the total base shear strength of the building. This method is generally used for comparison between buildings to find which one is more vulnerable. So, the first thing is to divide the shear strength by the total mass of the building to find the acceleration that causes that force (V_y/W, where $W = 12640.48$ kN). This is equal to:

$$\boxed{S_{a,c,x} = V_{Rx}/W = 0.38g,\ S_{a,c,y} = V_{Ry}/W = 0.27g}$$

It is clear that this is a large acceleration but even that is not enough to draw firm conclusions. So, the spectral acceleration in the fundamental period (T_{exp} = 0.81 sec), which is equal to $S_{a,d}$ = 0.13g, has to be divided by this acceleration. According to this:

$$\boxed{a_x = S_{a,d}/S_{a,c,x} = 0.34,\ a_y = S_{a,d}/S_{a,c,y} = 0.48}$$

Therefore, there is no problem with this building, since this coefficient has a small number (<1.0) in both directions.

So, the purpose of this method is to classify the buildings by this coefficient in order to see which one will have problems in the future.

3.2.3. Application of FEMA P-2018 Method

As described in Chapter 1 (§ 1.3.3), the first step is the calculation of seismic spectrum. The recommended seismic hazard level for evaluation is the ASCE/SEI 41-17[5] Basic Safety Earthquake BSE-2E, which corresponds to a 5% probability of exceedance in a 50-year period (Figure 3.6).

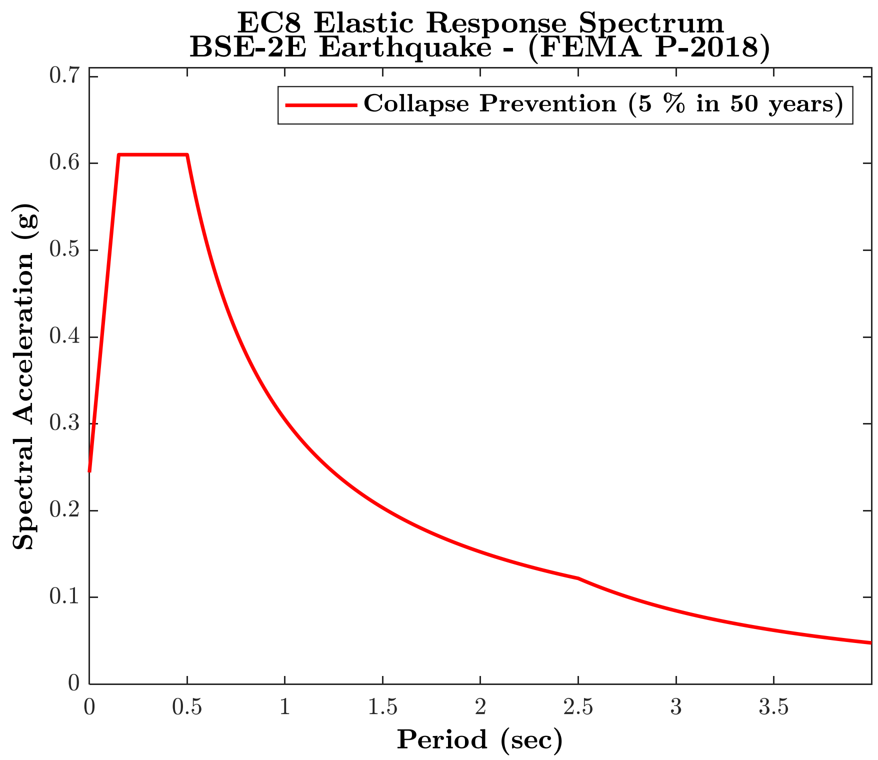


Figure 3.6 Seismic Spectrum – FEMA P-2018 Method

The next step is the calculation of components strength (flexural and shear) for all floors according to ASCE/SEI 41-17[5]. The results for all elements are presented in Appendix C.

In order to define the effective yield strength of the building V_y, the four (4) plastic mechanisms must be examined to find the critical one. Table 3.9 shows the base shear strength for all mechanisms. It is clear that the second mechanism is the critical with the base shear being equal:

$$\boxed{V_{px} = 5826.19 \text{ kN},\ V_{py} = 5421.75 \text{ kN}}$$

Table 3.9 Base shear for each plastic mechanism – FEMA P-2018

Plastic Mechanism	Critical Story	V_{px} (kN)	V_{py} (kN)
1st Mechanism	1	10406.55	9572.67
2nd Mechanism	1	5826.19	5421.75
3rd Mechanism	1	10406.55	9572.67
	2	11372.89	9669.91
	3	13343.06	11357.84
	4	16434.89	14234.23
	5	23497.69	20457.92
	6	36896.21	34480.69
	7	57901.16	64923.24
4th Mechanism	7	40531.23	46278.59

For a newly constructed building, the 2nd mechanism is the most appropriate because damages are not concentrated on one story, but they are distributed all over the building. This is the capacity design according to EN 1998-1[6].

Mechanism 2 controls the strength of the building in both directions, so the first story above the base is the critical one. In addition, the calculated plastic mechanism base-shear strength for Mechanism 2 is less than three-quarters (3/4) of the calculated plastic mechanism base-shear strength for Mechanism 1, so Mechanism 1 is not used as the controlling mechanism.

This building is classified as a "Frame-Wall Structural System", because the columns carry more than 25% of the gravity load according to Table 3.10. More specifically, considering the axial loads at the base of the building, 40% are borne by the columns and 60% by the walls. The total weight of the building is equal to W = 12531.21 kN. Furthermore, as shown in Table C.1 (Appendix C), the base shear strength of walls is less than 65% of the total shear resistance of the building, so it is a dual system according to EN 1998-1[6].

Table 3.10 Axial load of vertical elements on the ground floor – FEMA P-2018

Member	**Axial Load (kN)**
wall W1	910.6596
wall W2	1226.735
wall W3	1068.103
wall W4	1620.878
wall W5	982.744
wall W6	1015.741
wall W7	728.561
column C8	1237.213
column C9	829.4135
column C10	1032.396
column C11	401.9529
column C12	434.7435
column C13	344.6467
column C14	417.8405
column C15	279.5797

After having determined the critical mechanism and the structural type of the building, the effective fundamental period has to be estimated based on the shear ratio V_y/W according to equations (1.57) and (1.58). The final results are:

Direction X

$$\boxed{V_y/W = 46.49\ \%,\ A_{base} = 1308.78\ ft^2,\ C_w = 0.013,\ h_n = 69.39\ ft \rightarrow T_e = 0.86\ sec}$$

Direction Y

$$\boxed{V_y/W = 43.27\ \%,\ A_{base} = 1308.78\ ft^2,\ C_w = 0.010,\ h_n = 69.39\ ft \rightarrow T_e = 0.89\ sec}$$

The two fundament periods are almost the same in both directions because there is no difference in the ratio V_y/W. Moreover, the eigenperiods are close to the ones that have been calculated from modal analysis using cracked stiffness (T = 0.93 sec and 0.90 sec, respectively).

Figure 3.7 shows the spectral acceleration S_a for these fundamental periods based on an earthquake which corresponds to a 5% probability of exceedance in a 50-year.

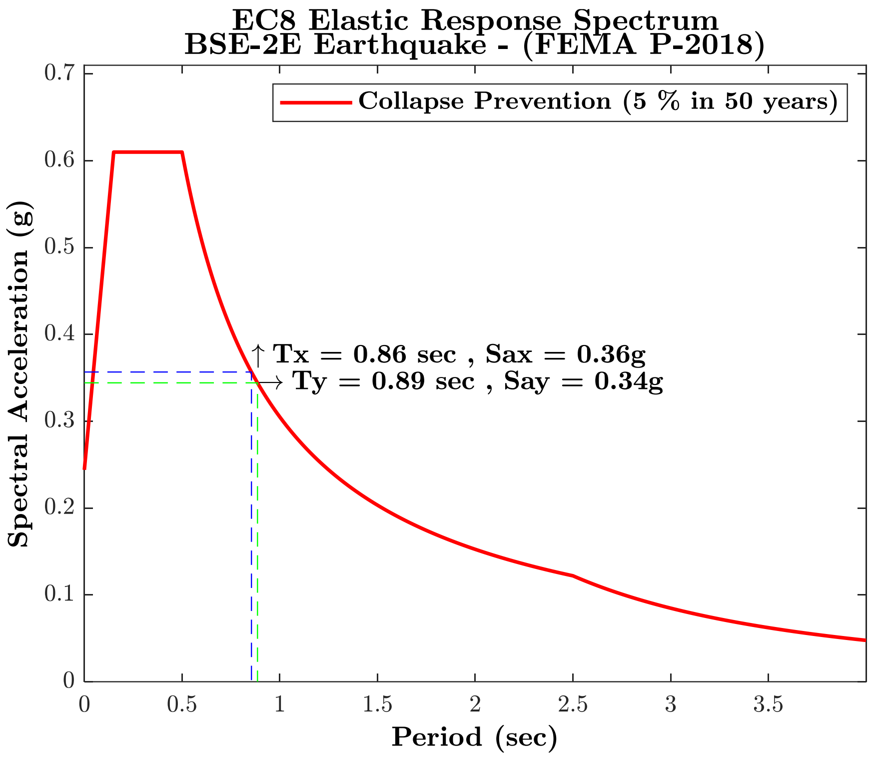


Figure 3.7 Spectral acceleration in the fundamental period – FEMA P-2018

As described in the Vougioukas Method (§ 3.2.2), the ratio of the seismic demand to capacity must be determined in order to draw firm conclusions. According to Eq. (1.60) with $C_m = 0.80$, we have:

$$\boxed{\mu_{strength,x} = 0.61,\ \mu_{strength,y} = 0.64}$$

Therefore, this building is classified as lower seismic risk, because $\mu_{strength} \leq 1.50$, so no further action required.

However, for the sake of clarity, all the steps up to the determination of the "Building Rating (BR)" are shown.

The next step is the calculation of the global seismic drift demand δ_{eff} for an equivalent single-degree-of-freedom (SDOF) system. According to Eq. (1.61) with $\alpha = 130$ (soil class B) we have:

Direction X

$$\boxed{T_e = 0.86 \text{ sec},\ C_1 = 0.996,\ C_2 = 1.0,\ S_a = 0.36g \rightarrow \delta_{eff} = 0.065 \text{ m}}$$

Direction Y

$$\boxed{T_e = 0.89 \text{ sec},\ C_1 = 0.996,\ C_2 = 1.0,\ S_a = 0.34g \rightarrow \delta_{eff} = 0.067 \text{ m}}$$

Regarding the story drift in critical story (Eq. (1.64)), which in this case is the first due to the 2nd plastic mechanism and taking the α_x coefficient equal to 1.2 (Table 1.8), the final results are:

Direction X

$\boxed{h_{sx} = 3.15 \text{ m},\ h_{clear} = 2.65 \text{ m},\ \rightarrow \delta_x = 0.0165 \text{ m} \rightarrow \text{Drift} = 0.52\ \%}$

Direction Y

$\boxed{h_{sx} = 3.15 \text{ m},\ h_{clear} = 2.65 \text{ m},\ \rightarrow \delta_x = 0.0171 \text{ m} \rightarrow \text{Drift} = 0.54\ \%}$

For frame systems, to account for the P-delta effect of gravity loads acting through lateral displacements, story drift demand δ_x calculated using Eq. (1.64) shall be increased according to the equation:

$$\delta_{x1} = \delta_x \left[\frac{1}{1 - \dfrac{W_x \delta_x}{V_{px} h_x}} \right] \tag{3.13}$$

where:

δ_{x1} = story drift demand of story x amplified for P-delta effects

δ_x = story drift demand according to Eq. (1.64)

V_{px} = plastic mechanism shear strength at story x

W_x = gravity load, approximated as the seismic weight of the stories above level x

Although it is not required to amplify the story drift demand because it is a frame-wall structural system, it is calculated and the results are:

Direction X

$\boxed{\delta_{x,final} = 0.0167 \text{ m} \rightarrow \text{Drift} = 0.53\ \%}$

Direction Y

$\boxed{\delta_{x,final} = 0.0173 \text{ m} \rightarrow \text{Drift} = 0.55\ \%}$

Having defined the story drift, the next step is the determination of drift demand of structural vertical elements and joints according to Eq. (1.65). To do this, the torsional ratio TR must first be calculated. The coordinates of the center of mass and the center of stiffness based on this method are:

$\boxed{x_{CM} = 6.98 \text{ m},\ y_{CM} = 5.71 \text{ m}}$

$\boxed{x_{CR} = 6.94 \text{ m},\ y_{CR} = 5.97 \text{ m}}$

For the calculation of the center of rigidity, the plastic shear capacity V_{pfi} of each element in both directions is used (Table C.1).

The final eccentricity is taken as: → $e_x = 0.60$ m, $e_y = 0.655$ m. The critical plastic mechanism as described above is the second (2^{nd}), hence the shear strength is $V_{px} = 5826.19$ kN and $V_{py} = 5421.75$ kN. Therefore, the torsional ratio TR and the maximum amplification factor $A_{T,max}$ are equal to:

Direction X

$$\boxed{T_{Dx} = 3813.24 \text{ kNm},\ T_{Cx} = 59806.92 \text{ kNm},\ TR_x = 0.064 \rightarrow A_{T,max} = 1.00}$$

Direction Y

$$\boxed{T_{Dy} = 3253.05 \text{ kNm},\ T_{Cy} = 59806.92 \text{ kNm},\ TR_y = 0.054 \rightarrow A_{T,max} = 1.00}$$

So, the amplification factor A_T is equal to 1.00 and regarding the drift factor γ, for the walls it is also 1.00 and for the columns is calculated according to Table 1.9. Therefore, taking the results from Table C.2, the drift demand of vertical elements is presented in the table below.

Table 3.11 Drift demand of vertical elements – FEMA P-2018

Member	X-Direction			Y-Direction		
	A_T	γ	Δ_D (%)	A_T	γ	Δ_D (%)
wall W1	1.000	1.00	0.529	1.000	1.00	0.550
wall W2	1.000	1.00	0.529	1.000	1.00	0.550
wall W3	1.000	1.00	0.529	1.000	1.00	0.550
wall W4	1.000	1.00	0.529	1.000	1.00	0.550
wall W5	1.000	1.00	0.529	1.000	1.00	0.550
wall W6	1.000	1.00	0.529	1.000	1.00	0.550
wall W7	1.000	1.00	0.529	1.000	1.00	0.550
column C8	1.000	0.30	0.159	1.000	0.30	0.165
column C9	1.000	0.30	0.159	1.000	0.30	0.165
column C10	1.000	0.58	0.306	1.000	0.30	0.165
column C11	1.000	0.30	0.159	1.000	0.35	0.193
column C12	1.000	0.39	0.205	1.000	0.30	0.165
column C13	1.000	0.30	0.159	1.000	0.30	0.165
column C14	1.000	0.48	0.252	1.000	0.30	0.165
column C15	1.000	0.32	0.168	1.000	0.57	0.311

The drift demand of the corner beam-column connections must also be determined. In this case, the torsional coefficient A_T is equal to 1.00 and also the drift factor γ is 1.00. The results are presented in the table below.

Table 3.12 Drift demand of corner beam-column connections – FEMA P-2018

X-Direction				Y-Direction			
Node	$\mathbf{A_T}$	$\boldsymbol{\gamma}$	$\boldsymbol{\Delta_D}$ **(%)**	**Node**	$\mathbf{A_T}$	$\boldsymbol{\gamma}$	$\boldsymbol{\Delta_D}$ **(%)**
9	1.000	1.00	0.529	8	1.000	1.00	0.550
10	1.000	1.00	0.529	9	1.000	1.00	0.550
11	1.000	1.00	0.529	11	1.000	1.00	0.550
12	1.000	1.00	0.529	12	1.000	1.00	0.550
13	1.000	1.00	0.529	13	1.000	1.00	0.550
14	1.000	1.00	0.529	14	1.000	1.00	0.550

The calculation of the drift capacity of vertical members is based on Eq. (1.71) and for corner beam-column connections based on Eq. (1.73). The clear height is equal to $l_u = 2.65$ m. More precisely, the calculation of the plastic rotation capacity θ_c for columns is based on Table 1.10 or Table 1.11, depending on the type of stirrups and the ratio V_p/V_n (plastic shear/total shear). This ratio shows if the element has shear or flexural behavior. In this work, after calculating the resistance of all members according to ASCE/SEI 41-17[5], it follows that $V_p/V_n = 1.00$. With regard to the axial ratio $P/A_g f_{ce}'$, the axial load P is determined taking into account the earthquake according to Eq. (1.40) Table 3.13 shows the results for columns.

Table 3.13 Drift capacity of columns – FEMA P-2018

Member	X-Direction				Y-Direction			
	$\boldsymbol{\rho_t}$	$\mathbf{P/A_g f_{ce}}$	$\boldsymbol{\theta_c}$	$\boldsymbol{\Delta_C}$ **(%)**	$\boldsymbol{\rho_t}$	$\mathbf{P/A_g f_{ce}}$	$\boldsymbol{\theta_c}$	$\boldsymbol{\Delta_C}$ **(%)**
column C8	0.0056	0.149	0.0616	6.021	0.0037	0.126	0.0564	5.582
column C9	0.0079	0.171	0.0653	6.338	0.0045	0.130	0.0597	5.862
column C10	0.0059	0.151	0.0622	6.075	0.0082	0.151	0.0684	6.592
column C11	0.0067	0.161	0.0636	6.192	0.0059	0.153	0.0619	6.049
column C12	0.0067	0.174	0.0621	6.062	0.0059	0.164	0.0606	5.940
column C13	0.0042	0.126	0.0591	5.814	0.0039	0.123	0.0582	5.738
column C14	0.0059	0.159	0.0612	5.994	0.0067	0.163	0.0633	6.168
column C15	0.0067	0.076	0.0718	6.880	0.0067	0.076	0.0718	6.880

For the walls, the calculation of drift capacity is based on Table 1.12 or Table 1.13. In this building, there are only flexural walls, so the ratio c/l_w (Eq. (1.72)) need to be determined. The coefficient a is equal to 10 and the coefficient b is equal to 1.2 (Table 1.14). The axial ratio $P/A_g f_{ce}'$ is calculated in the same way as the columns. The final results for the walls are presented in the table below.

Table 3.14 Drift capacity of walls – FEMA P-2018

Member	**X-Direction**				**Y-Direction**			
	l_w/b_w	c/l_w	$P/A_g f_{ce}$	Δ_C (%)	l_w/b_w	c/l_w	$P/A_g f_{ce}$	Δ_C (%)
wall W1	6.667	0.181	0.067	3.463	6.667	0.183	0.069	3.462
wall W2	6.667	0.209	0.091	3.396	6.667	0.205	0.088	3.419
wall W3	6.667	0.192	0.077	3.456	6.667	0.193	0.078	3.455
wall W4	6.667	0.234	0.112	3.237	6.667	0.236	0.113	3.228
wall W5	9.100	0.203	0.085	3.251	9.100	0.206	0.088	3.233
wall W6	9.100	0.203	0.086	3.250	9.100	0.206	0.088	3.230
wall W7	6.600	0.215	0.096	3.365	6.600	0.200	0.083	3.455

According to this method, we could increase the drift capacity of walls because there are confined columns on both edges, but this is not used for safety reasons.

Finally, the drift capacity of corner beam-column connections is calculated. Table 3.15 shows the final results. Note that the axial load P is calculated on both orthogonal directions (X and Y).

Table 3.15 Drift capacity of corner beam-column connections – FEMA P-2018

X-Direction			**Y-Direction**		
Node	$P/A_g f_{ce}$	Δ_C (%)	**Node**	$P/A_g f_{ce}$	Δ_C (%)
9	0.2139	2.942	8	0.1908	3.704
10	0.2187	2.782	9	0.2139	2.942
11	0.2340	2.500	11	0.2187	2.782
12	0.1828	3.969	12	0.2340	2.500
13	0.2223	2.663	13	0.1828	3.969
14	0.2139	2.942	14	0.2223	2.663

After determining the drift demand and capacity of all vertical members, the final step is the calculation of the column/wall rating (Table 1.16) and hence the building rating.

For columns, the ratio Δ_D/Δ_C is taken as the largest between the results obtained by the columns drift demands/capacities and beam-column corner connections. The final results are presented in the table below.

Table 3.16 Final ratings of the vertical elements – FEMA P-2018

Member	X-Direction		Y-Direction	
	Δ_D/Δ_C	CR or WR	Δ_D/Δ_C	CR or WR
wall W1	0.1528	0.00	0.1588	0.00
wall W2	0.1559	0.00	0.1608	0.00
wall W3	0.1531	0.00	0.1591	0.00
wall W4	0.1635	0.00	0.1704	0.00
wall W5	0.1628	0.00	0.1701	0.00
wall W6	0.1629	0.00	0.1702	0.00
wall W7	0.1573	0.00	0.1591	0.00
column C8	0.0264	0.00	0.1484	0.00
column C9	0.1799	0.00	0.1869	0.00
column C10	0.0503	0.00	0.0250	0.00
column C11	0.1902	0.00	0.1976	0.00
column C12	0.2117	0.00	0.2199	0.00
column C13	0.1334	0.00	0.1385	0.00
column C14	0.1987	0.00	0.2065	0.00
column C15	0.0244	0.00	0.0452	0.00

So, the weighted average rating for all columns, R_{avg}, is equal to 0.0 as well as the adjusted average of column, R_{adj}. Therefore, the story rating (SR) takes the lowest value according to this method which is equal to 0.10.

After examining all the results, the final building rating as expected is equal to BR = 0.10, which means that the building is safe.

Note that the BR did not need to be calculated as long as the $\mu_{strength}$ coefficient was less than 1.50.

3.3. Application of Third Level Pre-Earthquake Assessment Method

After the second level pre-assessment checks, the inelastic static analysis was performed using the SeismoBuild software for the modal and uniform distribution of the horizontal forces for each loading direction. As described above, three (3) codes (KANEPE[3], EN 1998-3[4], ASCE/SEI 41-17[5]) were used to evaluate the building and find the differences. Each code has its own performance levels with a specific intensity of seismic action. Analytically, the performance levels can be found in § 1.4. In addition, the data reliability (DRL) was high for all the methods. The infills were taken into account for the extra weight and for the rigidity of the structure (two models created - Figure 2.7). Furthermore, sections with cracked and uncracked stiffness were used in the models. For the final evaluation of the structure, all members were checked for the chord rotation capacity and the shear strength in each performance level.

3.3.1. Eigenvalue Analysis

First, the eigenvalue analysis was performed to see the response of the building. For each direction of the earthquake and a given eigenmode, a percentage of the total mass is activated which is the effective eigenmode mass. Eigenvalue analysis is used to analyze the inherent dynamic properties of the ground/structure and this can be used to obtain the natural mode (mode shape), natural period (natural frequency), modal participation factor etc. of the ground/structure. These properties are determined by the mass and stiffness of the structure. In other words, if a structure is determined, the natural frequency and vibration mode (natural mode) are also determined and the number of properties are the same as the degree of freedom of the structure. For real cases, the structure does not vibrate at a single mode shape and multiple modes overlap to display a complex vibration shape. The analysis was carried out for both models (with – without infills) using sections with cracked and uncracked stiffness. For general structure, considering only vibration modes with a mass participation factor sum of around 90% is still regarded as a sufficiently accurate analysis. In this work, this percentage was achieved using seventy (70) eigenmodes. For the sake of simplicity, the first three (3) modes are presented in the next figures for each model in order to draw conclusions. The analytical results of the percentage effective mass for each eigenmode are presented in Appendix D.

Building without infills – Uncracked sections

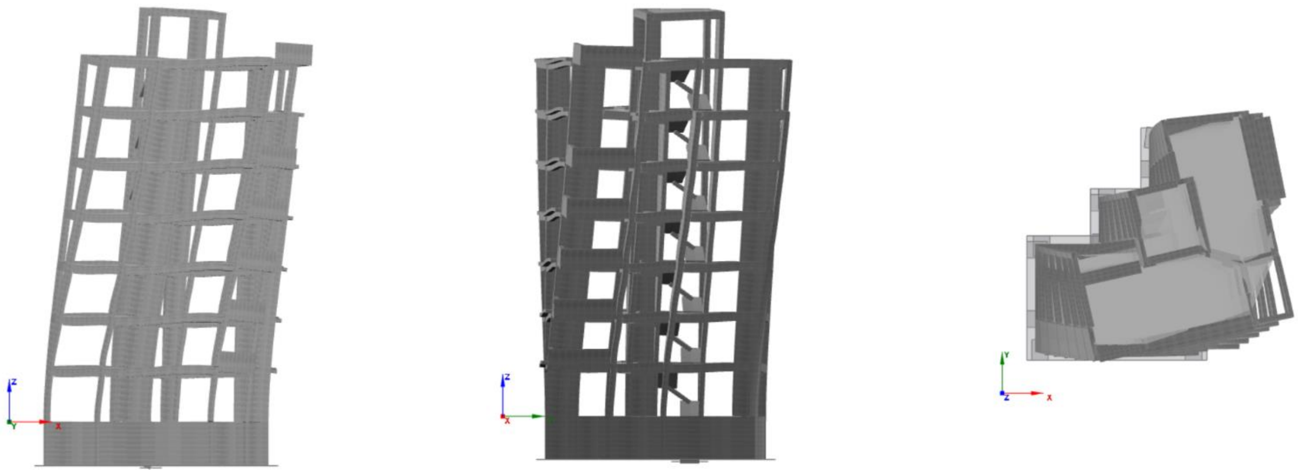


Figure 3.8 1st Eigenmode (T_1 = 0.53 sec), Transitional X, Rotational Z (Table D.1)

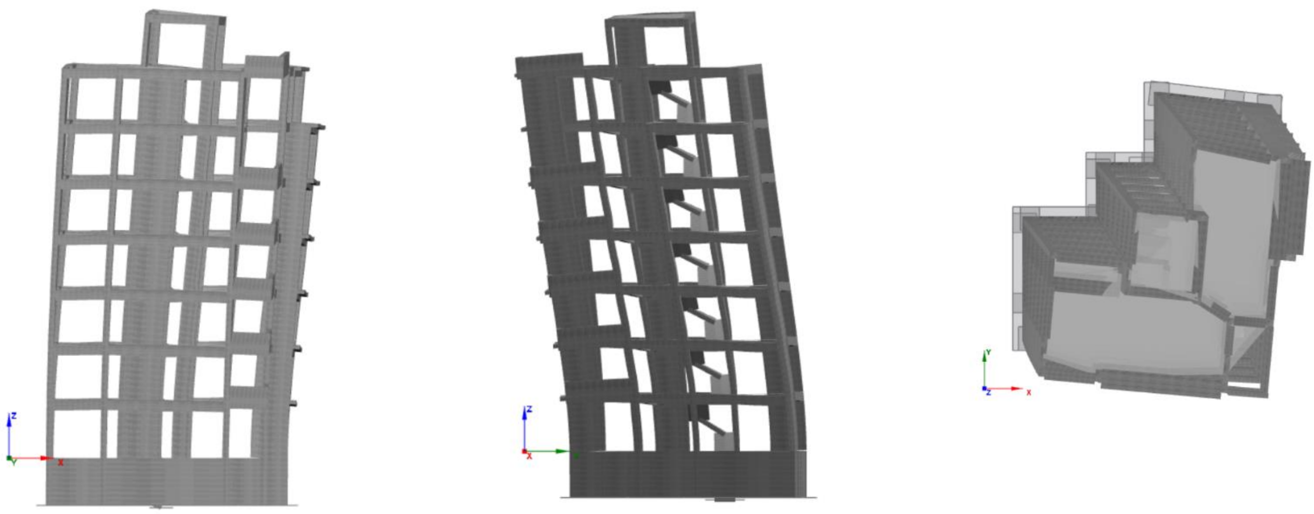


Figure 3.9 2nd Eigenmode – T_2 = 0.51 sec, Transitional Y (Table D.1)

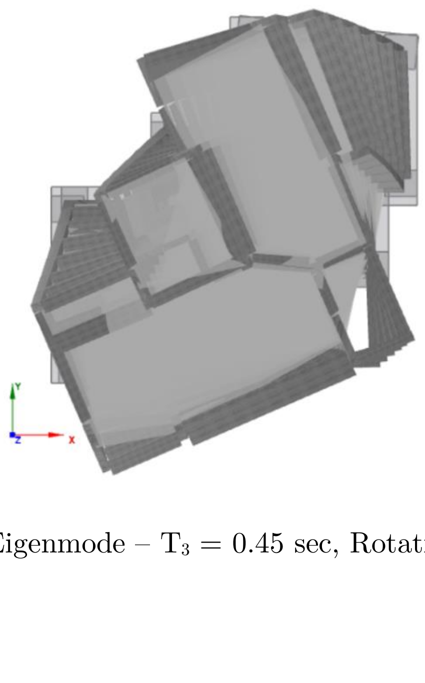


Figure 3.10 3rd Eigenmode – T_3 = 0.45 sec, Rotational Z (Table D.1)

Building without infills – Cracked sections

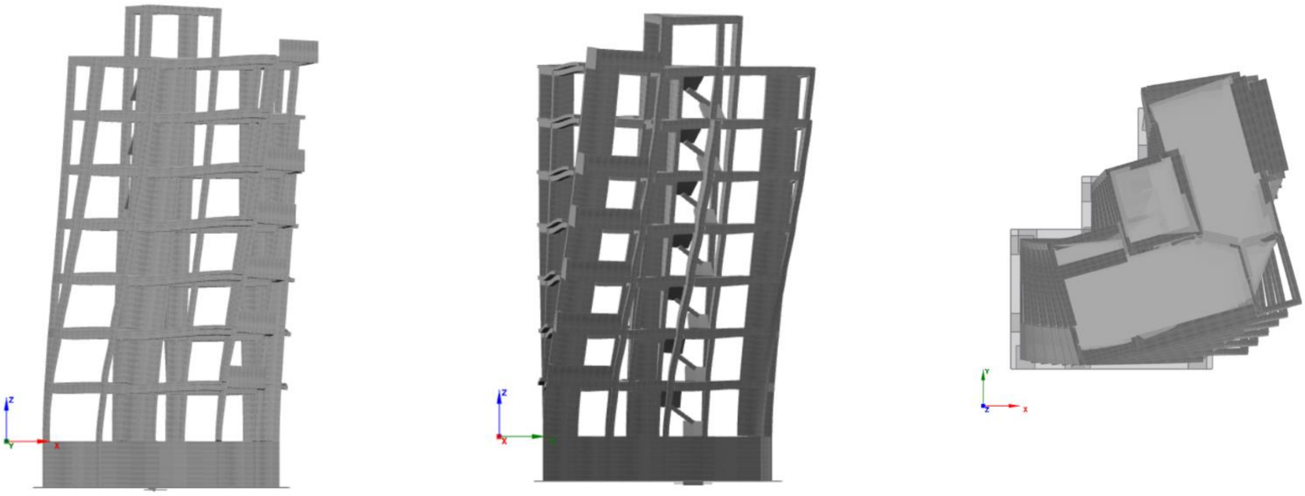


Figure 3.11 1^{st} Eigenmode ($T_1 = 0.93$ sec), Transitional X/Y, Rotational Z (Table D.2)

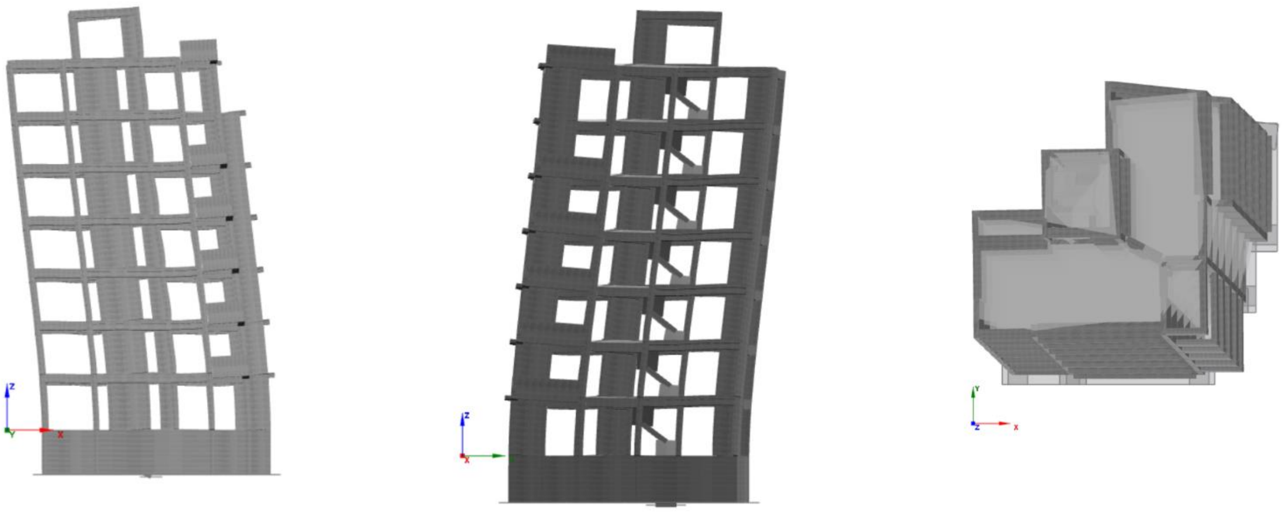


Figure 3.12 2^{nd} Eigenmode – $T_2 = 0.90$ sec, Transitional X/Y (Table D.2)

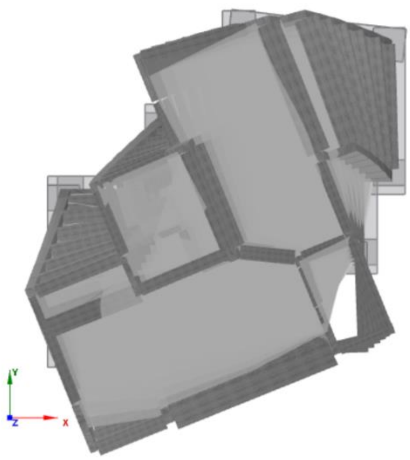


Figure 3.13 3^{rd} Eigenmode – $T_3 = 0.79$ sec, Rotational Z (Table D.2)

Building with infills – Uncracked sections

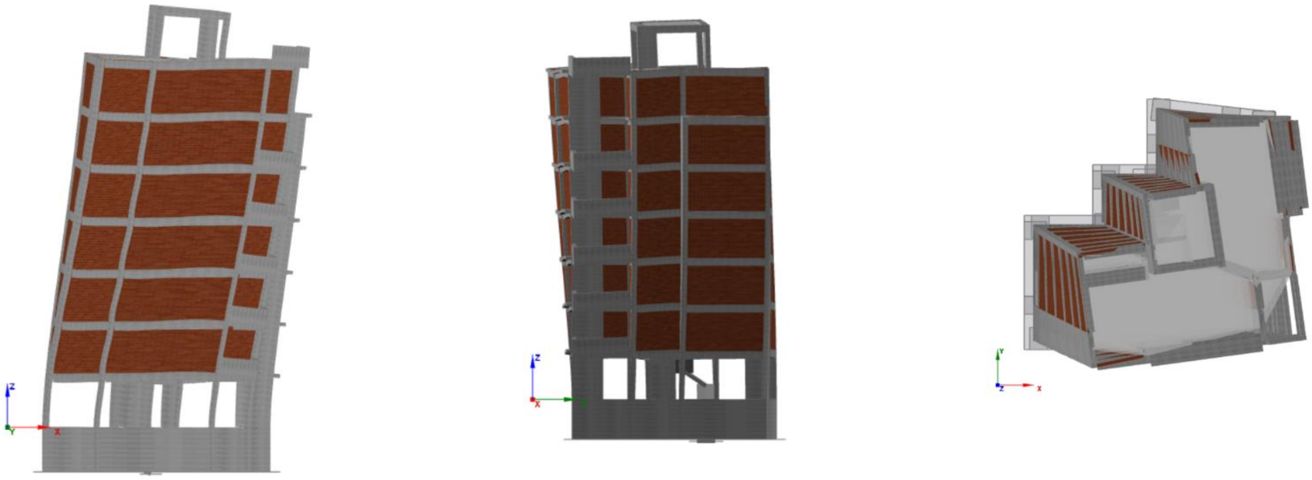


Figure 3.14 1st Eigenmode (T_1 = 0.44 sec), Transitional X (Table D.3)

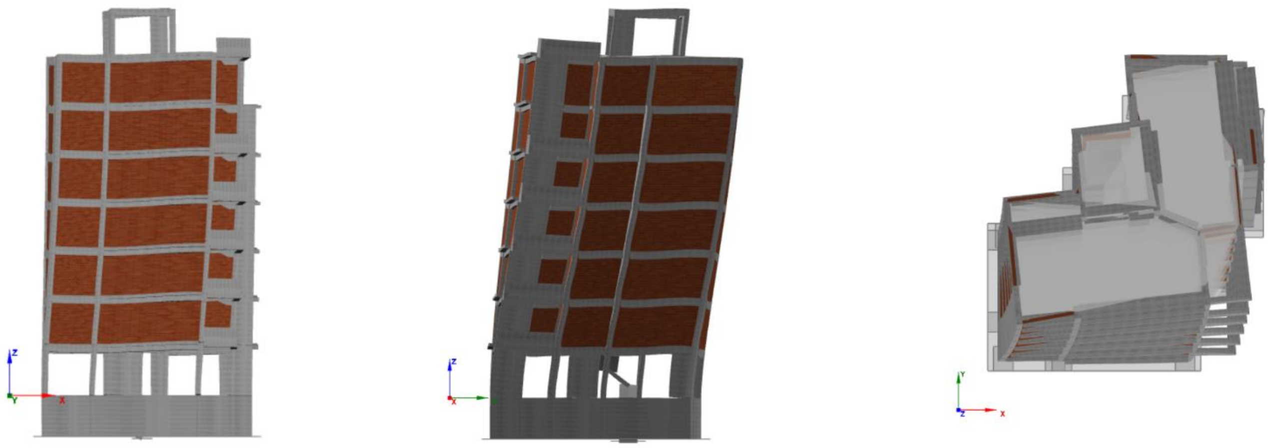


Figure 3.15 2nd Eigenmode – T_2 = 0.42 sec, Transitional Y (Table D.3)

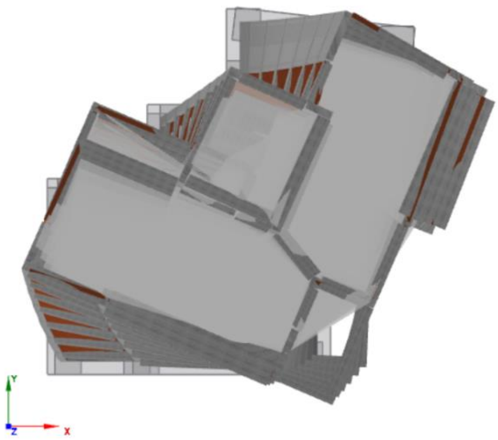


Figure 3.16 3rd Eigenmode – T_3 = 0.38 sec, Rotational Z (Table D.3)

Building with infills – Cracked sections

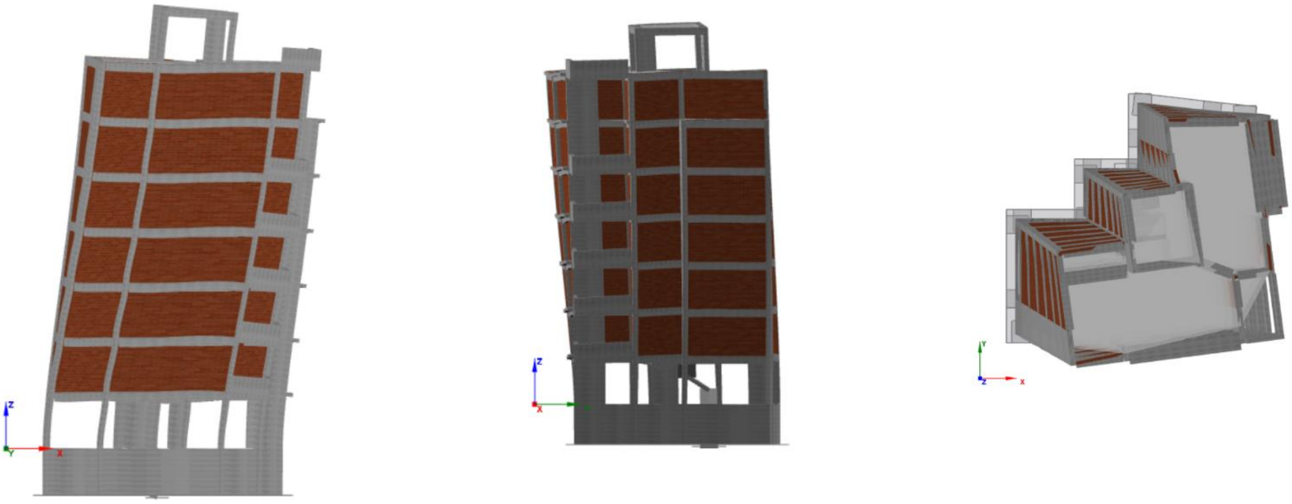


Figure 3.17 1st Eigenmode (T_1 = 0.75 sec), Transitional X (Table D.4)

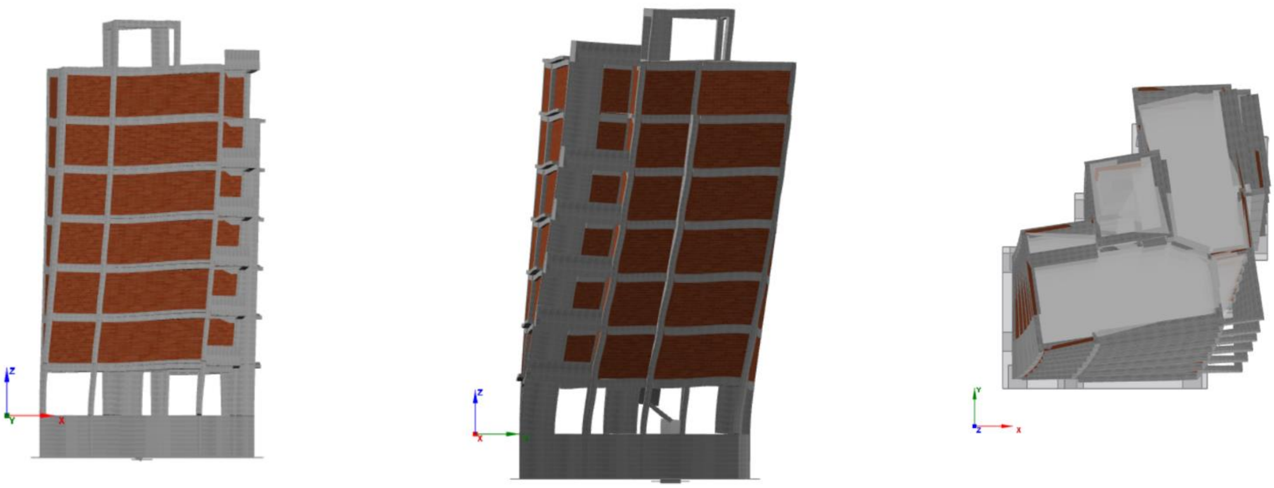


Figure 3.18 2nd Eigenmode – T_2 = 0.72 sec, Transitional Y (Table D.4)

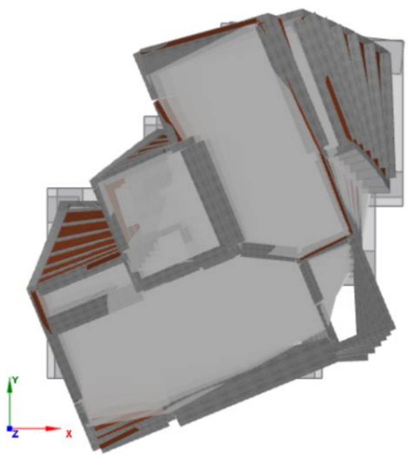


Figure 3.19 3rd Eigenmode – T_3 = 0.67 sec, Rotational Z (Table D.4)

Table 3.17 Comparison of the first three (3) eigenperiods of the building

Model	T_1 (sec)	T_2 (sec)	T_3 (sec)
Without Infills – Uncracked Sections	0.53	0.51	0.45
Without Infills – Cracked Sections	0.93	0.90	0.79
With Infills – Uncracked Sections	0.44	0.42	0.38
With Infills – Cracked Sections	0.75	0.72	0.67

Table 3.18 Comparison of the effective modal mass (1st Eigenperiod)

Model	Ux	Uy	Rz
Without Infills – Uncracked Sections	44.73%	8.46%	14.98%
Without Infills – Cracked Sections	23.47%	23.24%	23.14%
With Infills – Uncracked Sections	59.91%	3.16%	6.72%
With Infills – Cracked Sections	57.07%	7.28%	6.55%

Table 3.19 Comparison of the effective modal mass (2nd Eigenperiod)

Model	Ux	Uy	Rz
Without Infills – Uncracked Sections	14.35%	49.75%	2.40%
Without Infills – Cracked Sections	34.13%	32.68%	0.03%
With Infills – Uncracked Sections	1.64%	64.62%	4.82%
With Infills – Cracked Sections	3.78%	60.06%	8.89%

Table 3.20 Comparison of the effective modal mass (3rd Eigenperiod)

Model	Ux	Uy	Rz
Without Infills – Uncracked Sections	6.81%	8.35%	52.34%
Without Infills – Cracked Sections	9.40%	11.42%	47.43%
With Infills – Uncracked Sections	7.56%	2.11%	61.75%
With Infills – Cracked Sections	9.56%	3.68%	59.13%

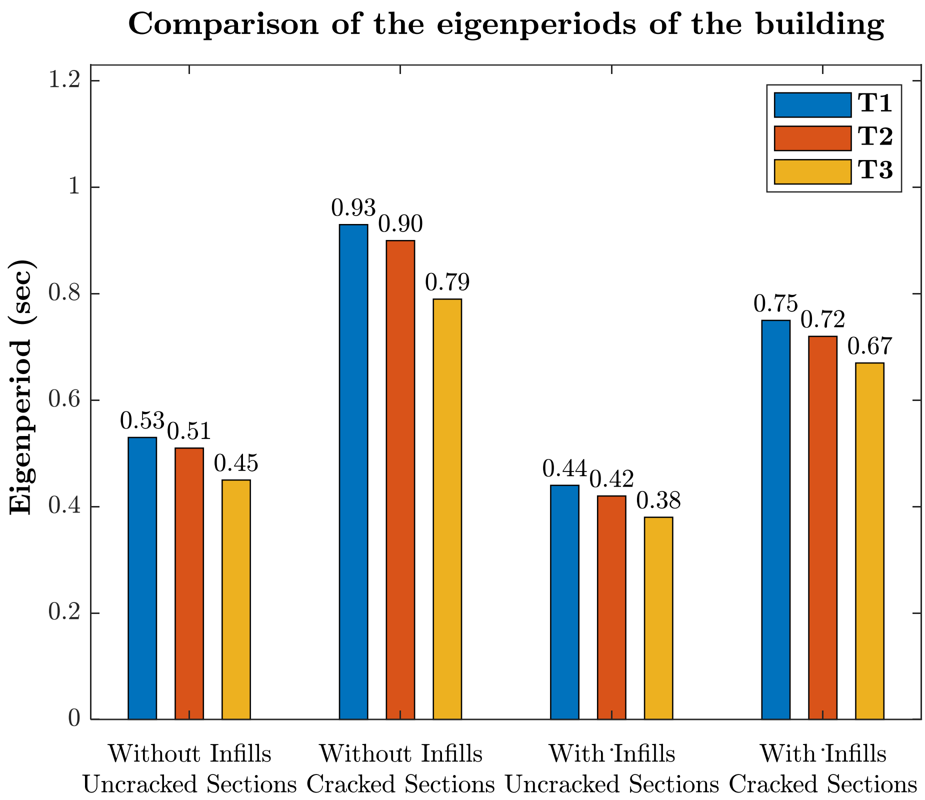


Figure 3.20 Comparison of the first three (3) eigenperiods

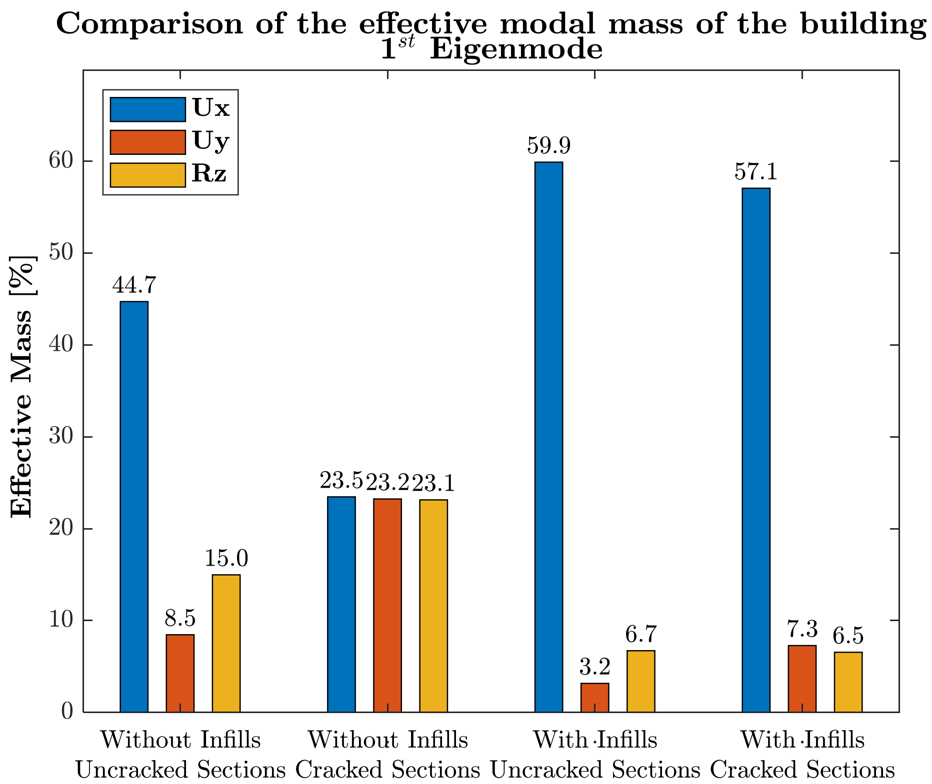


Figure 3.21 Comparison of the effective modal mass (1^{st} Eigenperiod)

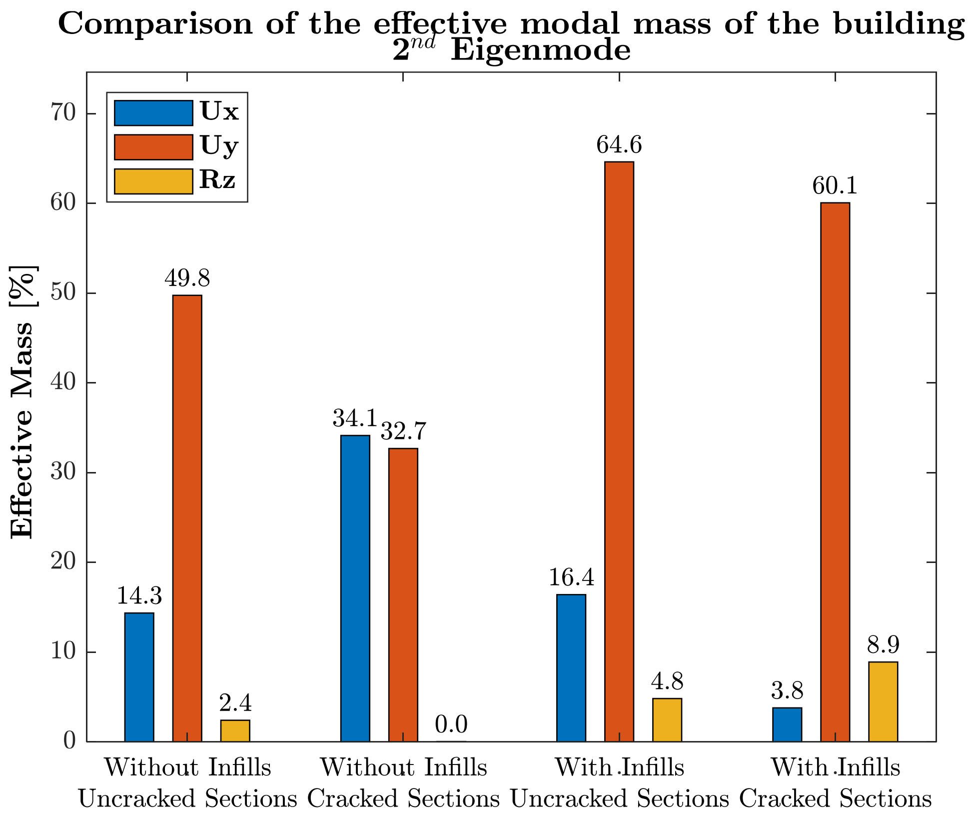


Figure 3.22 Comparison of the effective modal mass (2nd Eigenperiod)

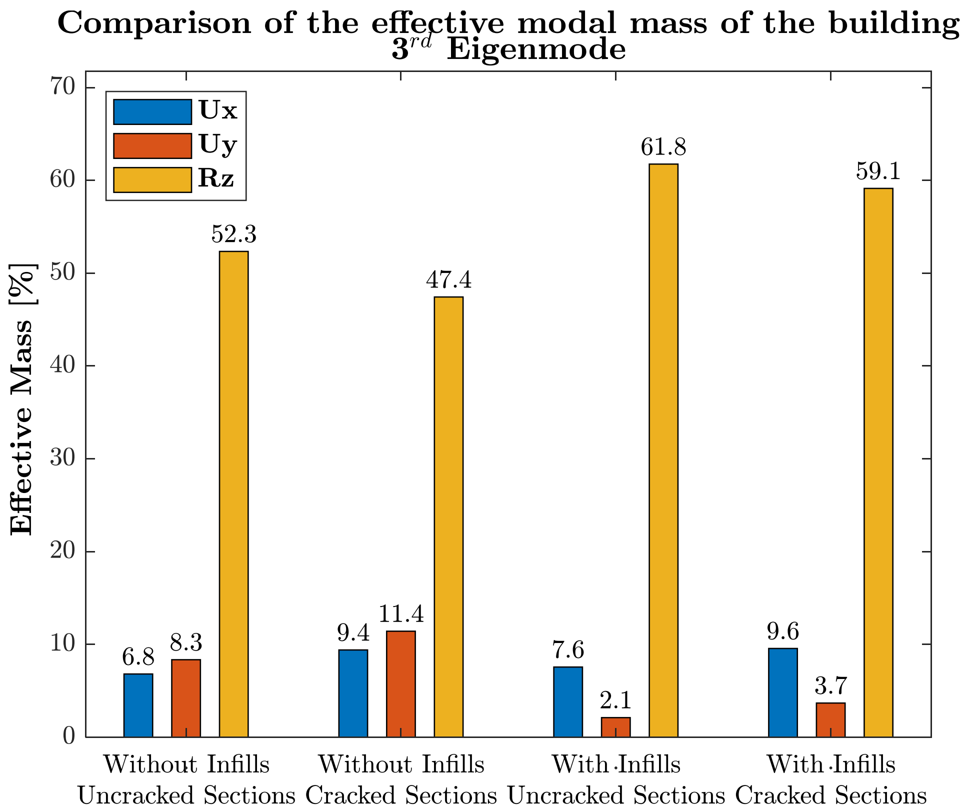


Figure 3.23 Comparison of the effective modal mass (3rd Eigenperiod)

After performing the eigenvalue analysis for these four (4) models, the final conclusions are:

- The eigenperiods of the first two (2) eigenmodes are very close to each other for all models
- The difference in eigenperiods between cracked and uncracked sections is 70 – 75 % whether there are infills or not. The structure becomes more flexible with cracked sections as expected
- The difference in eigenperiods between "with and without infills" is 20 – 25 %. The use of infills makes the structure stiffer, but not very much due to the presence of many walls
- In the 1^{st} eigenmode, the highest effective modal mass is in the x-direction except for the second model (Without Infills – Cracked Sections) where it is present in all directions, both transitional and rotational (Figure 3.21)
- The same applies in the 2^{nd} eigenmode with the difference that the highest effective modal mass is in the y-direction except for the second model (Figure 3.22)
- In the 3^{rd} eigenmode, the movement of the building is rotational along the z-axis for all models (Figure 3.23)
- Torsional effects are stronger in the x-direction than in the y-direction. If we compare Figure 3.21 with Figure 3.22, it can be seen that the effective modal mass along the z-axis (rotational) is larger in the first picture than in the second
- Finally, although the 1^{st} and 2^{nd} eigenmodes relate to the x and y directions, respectively, the models without infills have a lower participation factor than the models with infills (Figure 3.21 and Figure 3.22)

3.3.2. Application of Greek Code of Interventions (KANEPE)

The seismic capacity of each structure is determined by the Code of Interventions through the combination of certain seismic action levels and the three basic performance levels, i.e., the acceptable level of damage. These levels are:

- Damage Limitation
- Significant Damage
- Near Collapse

The next step is to assign a specific intensity of seismic action to each performance level to evaluate the structure. Figure 3.24 shows the performance objectives according to KANEPE[3].

$a_g / a_{g,ref}$	Στάθμη Επιτελεστικότητας Φέροντος Οργανισμού		
	Α «Περιορισμένες Βλάβες»	Β «Σημαντικές Βλάβες»	Γ «Οιονεί Κατάρρευση»
1.80	**A0**	**B0**	**Γ0**
1.30	**A1+**	**B1+**	**Γ1+**
1.00	**A1**	**B1**	**Γ1**
0.75	**A2+**	**B2+**	**Γ2+**
0.60	**A2**	**B2**	**Γ2**
0.45	**A3+**	**B3+**	**Γ3+**
0.35	**A3**	**B3**	**Γ3**
0.25	**A4+**	**B4+**	**Γ4+**
<0.25	**A4**	**B4**	**Γ4**

Figure 3.24 Performance objectives for assessment of buildings according to KANEPE

For existing structures, the seismic performance should be verified at least for the Near Collapse (NC) limit state. However, in this work all three (3) limit states were used to be more precise.

According to KANEPE[3], the minimum acceptable objectives for assessment and retrofit of existing buildings are given in the following table for different importance categories.

Table 3.21 Minimum acceptable objectives for assessment of existing buildings (KANEPE)

Importance Category	**Minimum performance objectives for assessment of buildings**
I	Γ2
II	Γ1
III	B1
IV	B1 and A2

These performance objectives are different from those used for new structures. E.g., for buildings of standard importance (category II), the assessment of existing buildings is performed for objective Γ1, while the design of new buildings is performed for objective B1.

In the new version of KANEPE[3], new performance objectives added for assessment of existing buildings. Table 3.22 shows the minimum seismic class that a building must achieve for performance level “Significant Damage” according to its construction date.

Table 3.22 Minimum basic seismic classes of existing buildings of importance I and II

Construction date	Minimum basic seismic building class
...<1985	B3
1985≤...<1995	$B3^+$
1995≤...	$B2^+$

In this work, the performance objectives used for the evaluation of the building are presented in the table below.

Table 3.23 Performance objectives for the assessment of the examined building (KANEPE)

Performance Level	Performance Objective	Probability of Exceedance in 50 years	Return Period (years)
Damage Limitation	A2	30 %	141
Significant Damage	B1	10 %	475
Near Collapse	Γ0	2 %	2475

As this is a newly constructed structure, it is designed for an earthquake with 10% probability of exceedance in 50 years. That's why, the seismic class B1 is used for the performance level "Significant Damage". However, if this performance objective is not achieved, the minimum seismic class according to Table 3.22 is $B2^+$. To perform the analysis, the design PGA, which is equal to 0.16g, is entered into the software and the seismic spectrum for each performance level is defined.

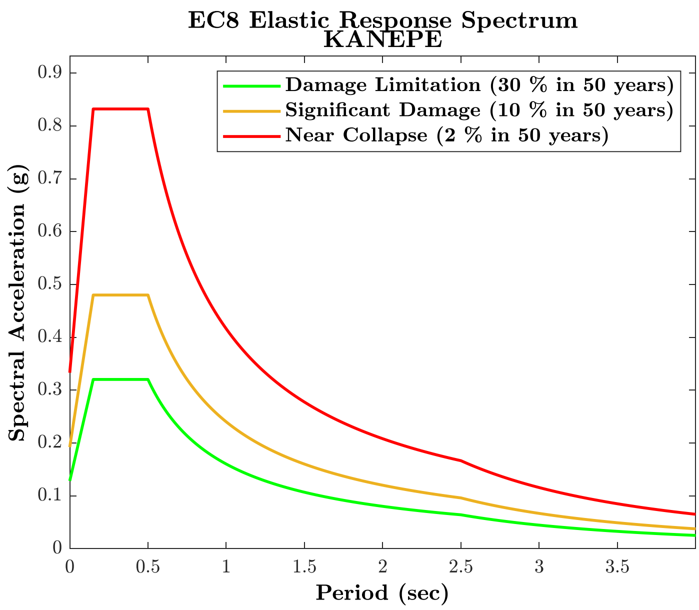


Figure 3.25 Seismic spectrums used for the assessment of the building (KANEPE)

According to KANEPE[3], sixty-four (64) seismic combinations used for the assessment of the existing building. In addition, as described in § 1.4.1, the influence of the higher eigenmodes has to be examined to find out if it is significant or not. This requires an initial dynamic elastic analysis taking into account the eigenmodes that contribute at least 90% of the total mass. A second dynamic elastic analysis based only on the first eigenmode (in each direction) then needs to be performed. If the shear force on just one floor resulting from the first analysis exceeds 130 % of that from the second analysis, the higher eigenmodes are essential.

Table 3.24 Seismic combinations for the assessment of the building according to KANEPE

Number	**Analysis**	**Direction**	**Number**	**Analysis**	**Direction**
1	Uniform + X + 0.3Y + eccX	X	33	Modal + X + 0.3Y + eccX	X
2	Uniform + X + 0.3Y - eccX	X	34	Modal + X + 0.3Y - eccX	X
3	Uniform + X - 0.3Y + eccX	X	35	Modal + X - 0.3Y + eccX	X
4	Uniform + X - 0.3Y - eccX	X	36	Modal + X - 0.3Y - eccX	X
5	Uniform - X + 0.3Y + eccX	X	37	Modal - X + 0.3Y + eccX	X
6	Uniform - X + 0.3Y - eccX	X	38	Modal - X + 0.3Y - eccX	X
7	Uniform - X - 0.3Y + eccX	X	39	Modal - X - 0.3Y + eccX	X
8	Uniform - X - 0.3Y - eccX	X	40	Modal - X - 0.3Y - eccX	X
9	Uniform + 0.3X + Y + eccX	Y	41	Modal + 0.3X + Y + eccX	Y
10	Uniform + 0.3X + Y - eccX	Y	42	Modal + 0.3X + Y - eccX	Y
11	Uniform + 0.3X - Y + eccX	Y	43	Modal + 0.3X - Y + eccX	Y
12	Uniform + 0.3X - Y - eccX	Y	44	Modal + 0.3X - Y - eccX	Y
13	Uniform - 0.3X + Y + eccX	Y	45	Modal - 0.3X + Y + eccX	Y
14	Uniform - 0.3X + Y - eccX	Y	46	Modal - 0.3X + Y - eccX	Y
15	Uniform - 0.3X - Y + eccX	Y	47	Modal - 0.3X - Y + eccX	Y
16	Uniform - 0.3X - Y - eccX	Y	48	Modal - 0.3X - Y - eccX	Y
17	Uniform + X + eccY + 0.3Y	X	49	Modal + X + eccY + 0.3Y	X
18	Uniform + X - eccY + 0.3Y	X	50	Modal + X - eccY + 0.3Y	X
19	Uniform + X + eccY - 0.3Y	X	51	Modal + X + eccY - 0.3Y	X
20	Uniform + X - eccY - 0.3Y	X	52	Modal + X - eccY - 0.3Y	X
21	Uniform - X + eccY + 0.3Y	X	53	Modal - X + eccY + 0.3Y	X
22	Uniform - X - eccY + 0.3Y	X	54	Modal - X - eccY + 0.3Y	X
23	Uniform - X + eccY - 0.3Y	X	55	Modal - X + eccY - 0.3Y	X
24	Uniform - X - eccY - 0.3Y	X	56	Modal - X - eccY - 0.3Y	X
25	Uniform + 0.3X + eccY + Y	Y	57	Modal + 0.3X + eccY + Y	Y
26	Uniform + 0.3X - eccY + Y	Y	58	Modal + 0.3X - eccY + Y	Y
27	Uniform + 0.3X + eccY - Y	Y	59	Modal + 0.3X + eccY - Y	Y
28	Uniform + 0.3X - eccY - Y	Y	60	Modal + 0.3X - eccY - Y	Y
29	Uniform - 0.3X + eccY + Y	Y	61	Modal - 0.3X + eccY + Y	Y
30	Uniform - 0.3X - eccY + Y	Y	62	Modal - 0.3X - eccY + Y	Y
31	Uniform - 0.3X + eccY - Y	Y	63	Modal - 0.3X + eccY - Y	Y
32	Uniform - 0.3X - eccY - Y	Y	64	Modal - 0.3X - eccY - Y	Y

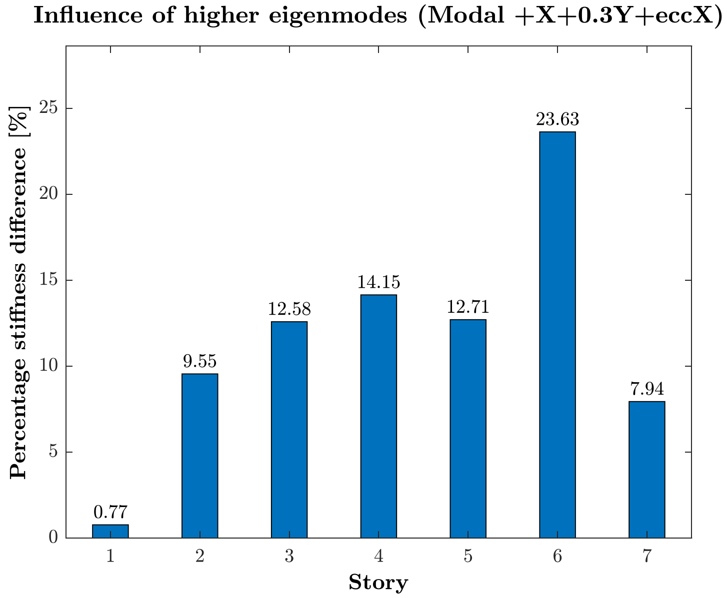


Figure 3.26 Influence of higher modes (KANEPE)

As shown in the figure, the influence of higher modes is not significant in this building since the maximum story shear difference is equal to 24% < 30%. Furthermore, critical is the x-direction, which is logical since there is more torsion in that direction, as described in the second level methods. After completing the analysis, the capacity curves for the two main directions are obtained. Finally, all members are checked for the chord rotation capacity and the shear strength in each performance level. FiguresFigure 3.27 - Figure 3.30 show the capacity curve and checks in the x-direction and Figures Figure 3.31 - Figure 3.33 in the y-direction for the model without infills.

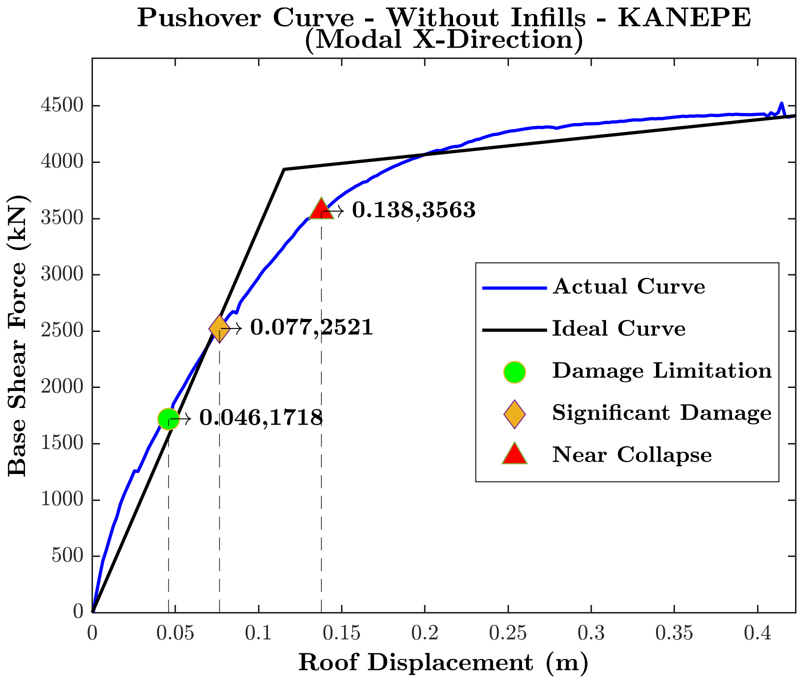
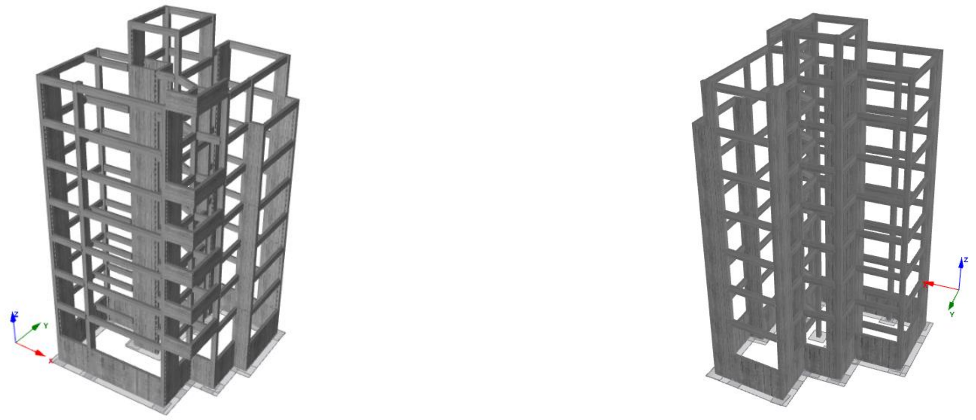
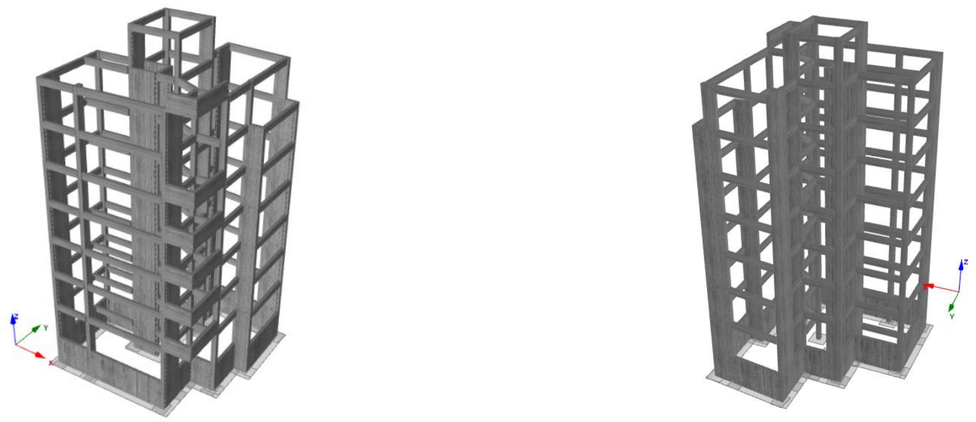


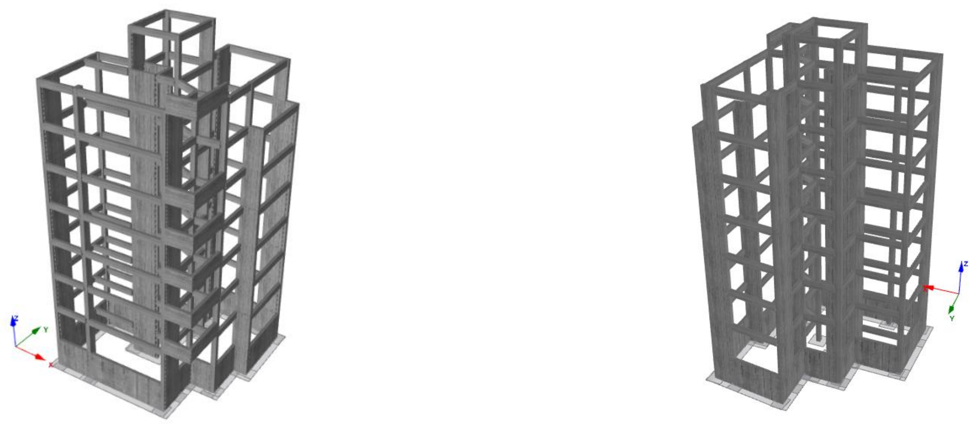
Figure 3.27 Pushover curve in the x-direction – Without Infills (KANEPE)



(a)



(b)



(c)

Figure 3.28 Chord rotation check in the x-direction – Without Infills (KANEPE)
(a) Damage Limitation, (b) Significant Damage, (c) Near Collapse – **No Failure**

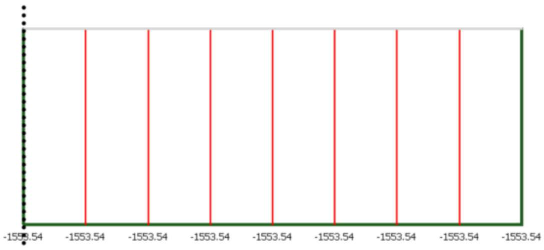
(a)

(b)

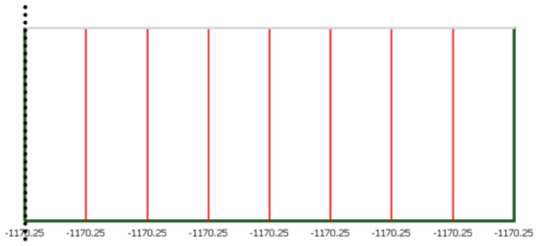
(c)

Figure 3.29 Shear strength check in the x-direction – Without Infills (KANEPE)
(a) Damage Limitation, (b) Significant Damage, (c) Near Collapse - **Failure**

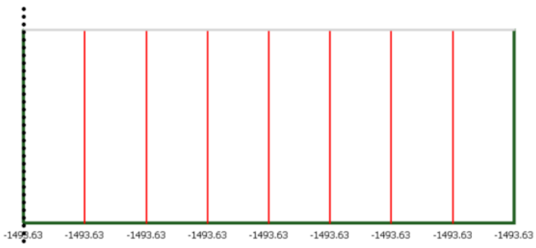
As shown in the figures above, there is no problem in the chord rotation check (no color means no failure), unlike the shear strength, where the performance objective is not achieved in any limit state, resulting in some members exhibiting brittle behavior. More specifically, this failure in the beams is not "real" since the software uses fiber elements. This can be seen from the fact that the axial load of the failed beams is really high, which is not correct, so it is a software error resulting from the simulation of the building using fiber elements. Only four (4) beams have problems (B1, B5, B15, B19 - (Figure 2.8)). Figure 3.30 shows the axial load of these beams at the performance level "Significant Damage".



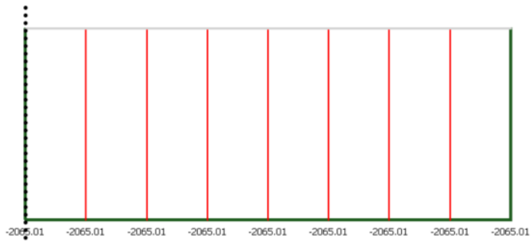
Beam B1 (Figure 2.8)



Beam B5 (Figure 2.8)



Beam B15 (Figure 2.8)



Beam B19 (Figure 2.8)

Figure 3.30 Axial load of the failed beams at the performance level "Significant Damage" (KANEPE)

It is obvious that this failure occurs due to this high axial compression load, which is common for all methods (Greek, European, and American), so the shear failure of these beams is neglected and it will not be presented next.

Considering the vertical members, it can be seen that only the walls have failure in all limit states. More specifically:

- "Damage Limitation" → 1 failed member
  - Wall W6 – floors: -1 → Performance Ratio: 1.40
- "Significant Damage" → 9 failed members
  - Wall W2 – floors: -1 → Performance Ratio: 1.05
  - Wall W5 – floors: -1, 1, 2 → Performance Ratio: 1.42 – 1.01
  - Wall W6 – floors: -1, 1, 2, 3 → Performance Ratio: 1.59 – 1.07
  - Wall W7 – floors: -1 → Performance Ratio: 1.05

- "Near Collapse" → 14 failed members
    - Wall W2 – floors: -1, 1, 3 → Performance Ratio: 1.43 – 1.02
    - Wall W4 – floors: -1 → Performance Ratio: 1.02
    - Wall W5 – floors: -1, 1, 2 → Performance Ratio: 1.86 – 1.06
    - Wall W6 – floors: -1, 1, 2, 3, 4, 5, 6 → Performance Ratio: 2.12 – 1.03

It is apparent that the failure concerns mainly the walls of the stairwell as these are stiffer and therefore absorb the highest shear. This is not a good result considering the fact that it is a newly constructed building, however it must be noted that KANEPE[3] gives rather conservative results for the calculation of the shear strength in the wall members, because it is mainly for columns.

In addition, the software classifies the failure using the following colors:

- Orange (Performance Ratio < 1.20)
- Red (1.20 ≤ Performance Ratio < 2.00)
- Deep Red (Performance Ratio ≥ 2.00)

In the first case (orange color) the failure can be neglected because it may be due to the analysis performed by the software. As already mentioned, it is a nonlinear analysis with P-Delta effects using fiber elements, so the accuracy is not perfect and there are some errors that can lead to incorrect results.

So, for the x-direction, the building failed the shear strength check for all performance levels. After that, the y-direction examined to see if there are any differences.

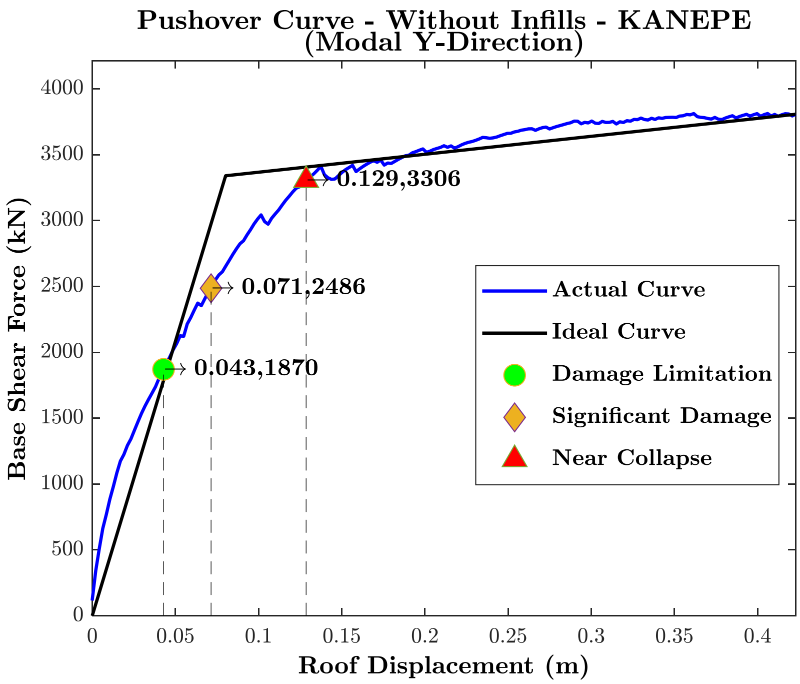
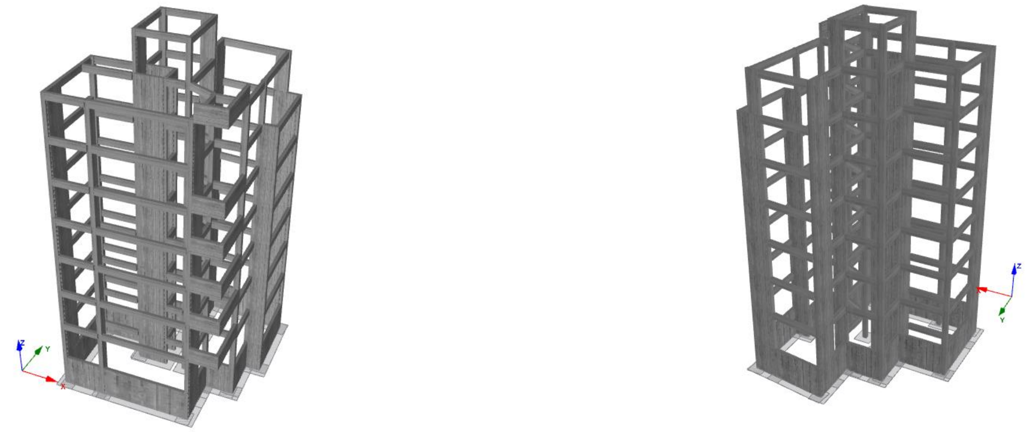
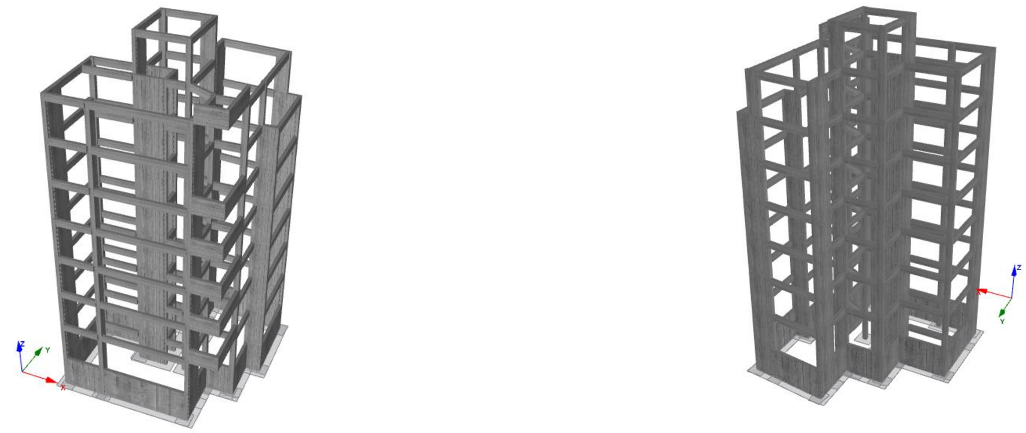


Figure 3.31 Pushover curve in the y-direction – Without Infills (KANEPE)



(a)



(b)

(c)

Figure 3.32 Chord rotation check in the y-direction – Without Infills (KANEPE)
(a) Damage Limitation, (b) Significant Damage, (c) Near Collapse – **No Failure**

(a)

(b)

(c)

Figure 3.33 Shear strength check in the y-direction – Without Infills (KANEPE)
(a) Damage Limitation, (b) Significant Damage, (c) Near Collapse – **Failure**

Again, the chord rotation checks passed for all members, but the same problem occurs regarding the shear strength. As mentioned before, these beams that have shear failure are neglected. Furthermore, only the walls are having failure as previous. More specifically:

- "Damage Limitation" → 2 failed members
  - Wall W4 – floors: -1, 2 → Performance Ratio: 1.09 – 1.06
- "Significant Damage" → 9 failed members
  - Wall W1 – floors: -1, 2 → Performance Ratio: 1.75 – 1.49
  - Wall W3 – floors: 2 → Performance Ratio: 1.01
  - Wall W4 – floors: -1, 2 → Performance Ratio: 2.04 – 1.66
  - Wall W5 – floors: -1 → Performance Ratio: 1.17
  - Wall W6 – floors: -1 → Performance Ratio: 1.03
  - Wall W7 – floors: -1, 2 → Performance Ratio: 1.31 – 1.19
- "Near Collapse" → 15 failed members
  - Wall W1 – floors: -1, 1, 2, 3 → Performance Ratio: 2.21 – 1.38
  - Wall W2 – floors: 1 → Performance Ratio: 1.60
  - Wall W3 – floors: -1 → Performance Ratio: 1.23
  - Wall W4 – floors: 1, 2 → Performance Ratio: 1.85 – 1.26
  - Wall W5 – floors: -1 → Performance Ratio: 1.46
  - Wall W6 – floors: -1, 1, 2 → Performance Ratio: 1.88 – 1.04
  - Wall W7 – floors: -1, 1, 2 → Performance Ratio: 2.52 – 1.06

By comparing the results from both directions, the number of failed wall members is almost identical, which is normal since the building has almost the same number of walls in both directions. The next step is to examine the model with infills to see if there are any differences. Figures Figure 3.34 - Figure 3.39 show the results.

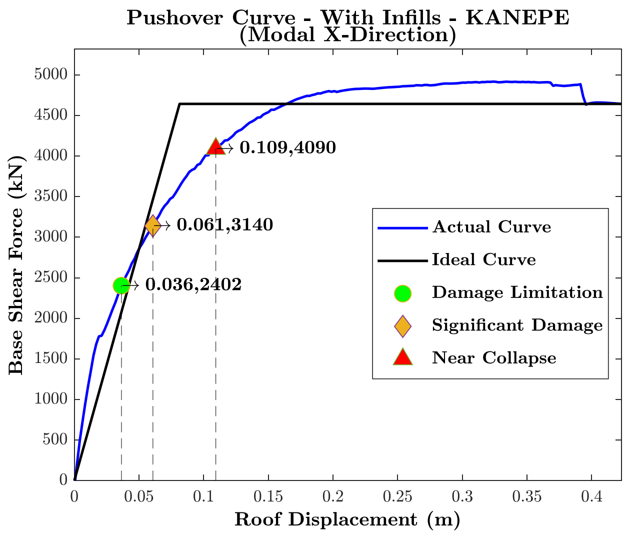
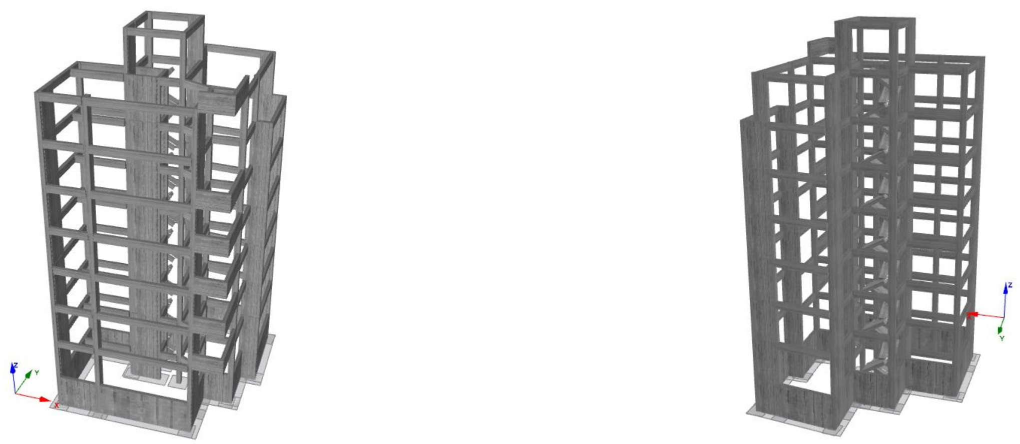


Figure 3.34 Pushover curve in the x-direction – With Infills (KANEPE)

(a)



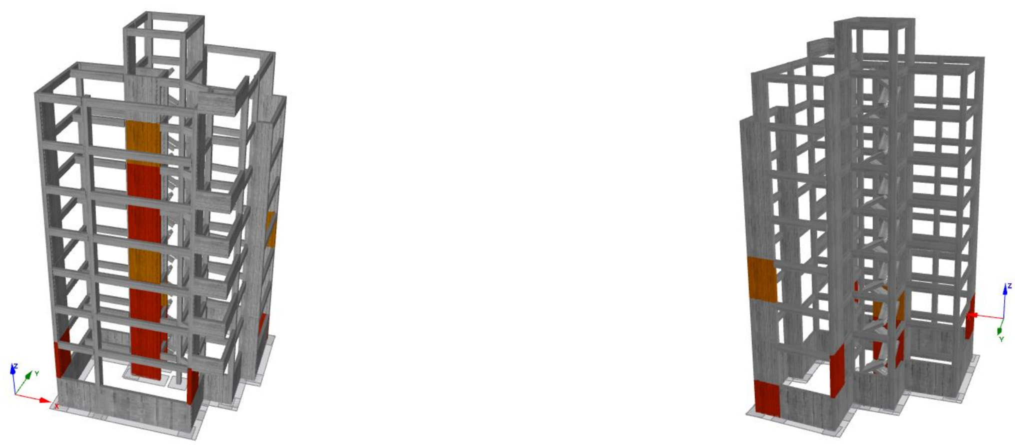
(b)

(c)

Figure 3.35 Chord rotation check in the x-direction – With Infills (KANEPE) (a) Damage Limitation, (b) Significant Damage, (c) Near Collapse – **No Failure**

(a)

(b)



(c)

Figure 3.36 Shear strength check in the x-direction – With Infills (KANEPE)
(a) Damage Limitation, (b) Significant Damage, (c) Near Collapse – **Failure**

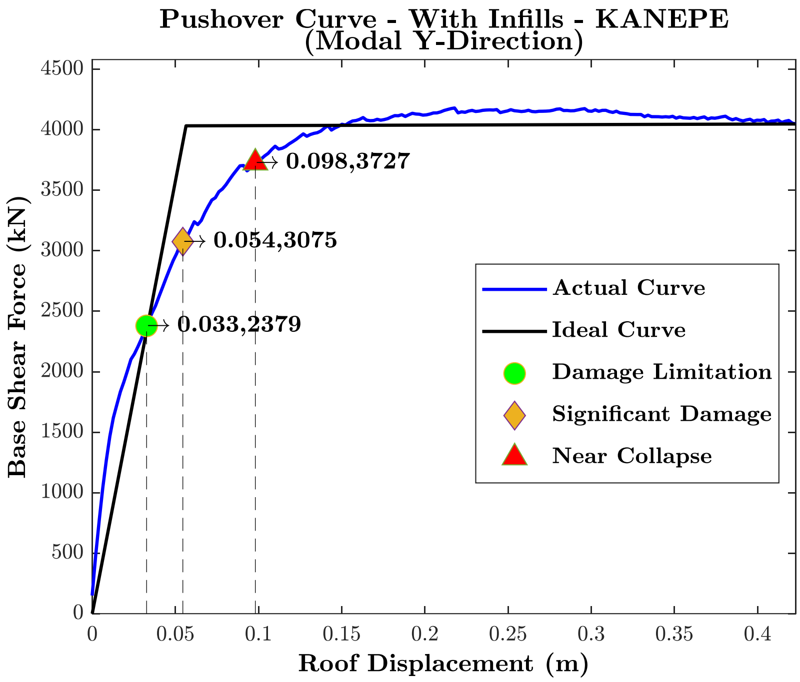
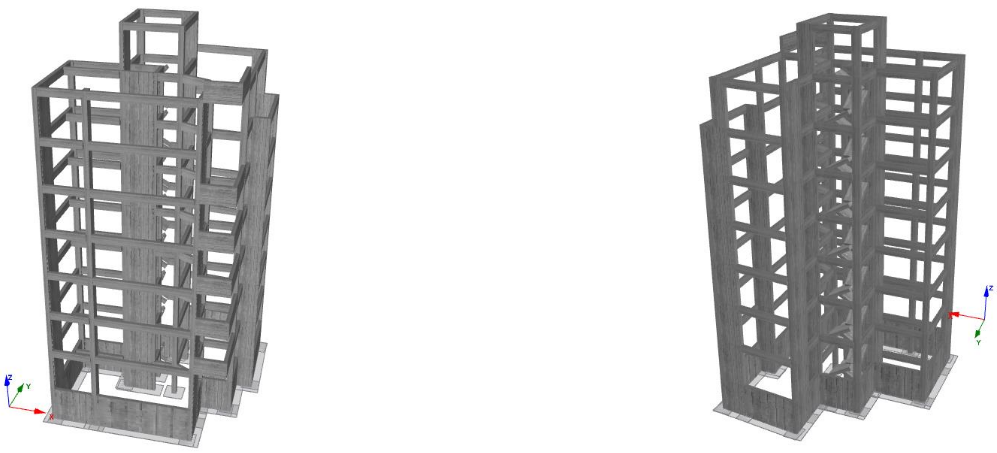
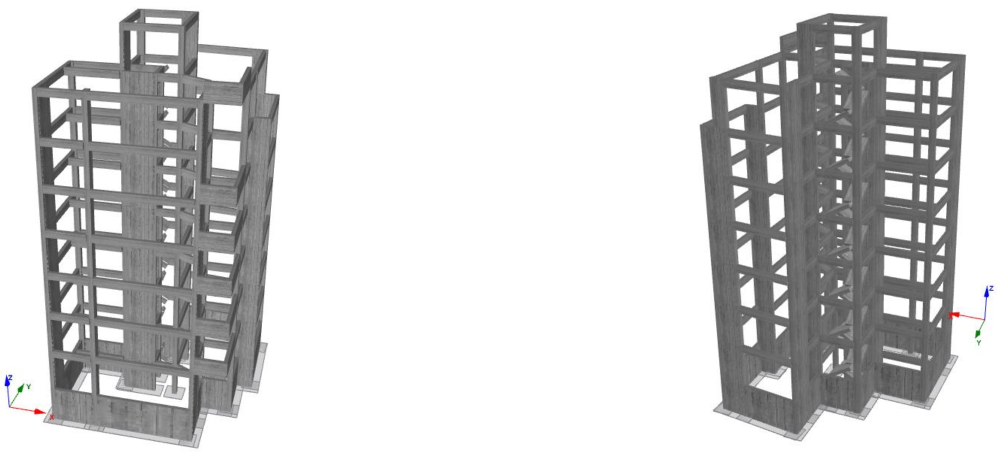


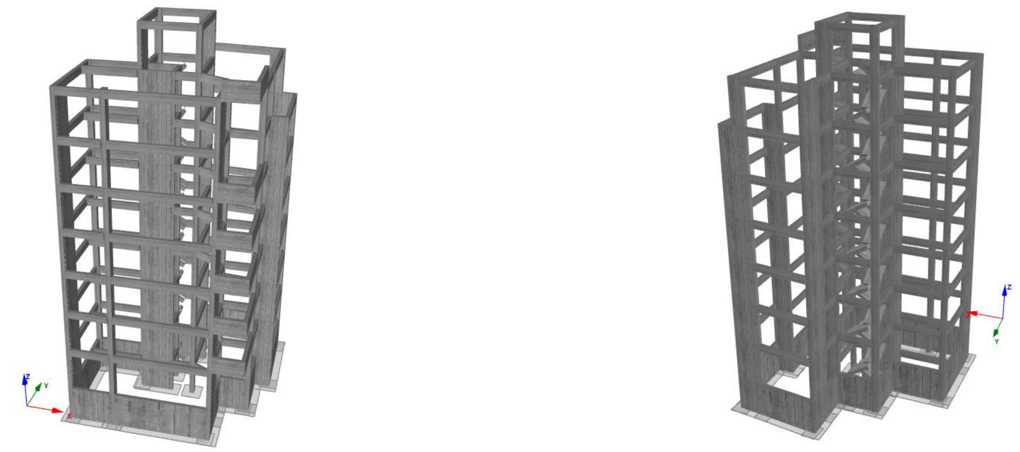
Figure 3.37 Pushover curve in the y-direction – With Infills (KANEPE)



(a)



(b)



(c)

Figure 3.38 Chord rotation check in the y-direction – With Infills (KANEPE)
(a) Damage Limitation, (b) Significant Damage, (c) Near Collapse – **No Failure**

(a)

(b)

(c)

Figure 3.39 Shear strength check in the y-direction – With Infills (KANEPE)
(a) Damage Limitation, (b) Significant Damage, (c) Near Collapse – **Failure**

Comparing the results obtained from the two models (without/with infills), it is apparent that there is not a big difference. The chord capacity check achieved while the shear strength check not. Moreover, the failed members are the same meaning that the increase of the building stiffness due to the existence of the infills didn't change the results because the structure has many walls on both directions.

So, as long as the performance objectives we set were not met, the next step was to determine if the building could achieve the minimum seismic class for performance level "Significant Damage", which according to Table 3.22 is B2^{+} corresponding to an earthquake with probability of exceedance 20% in 50 years.

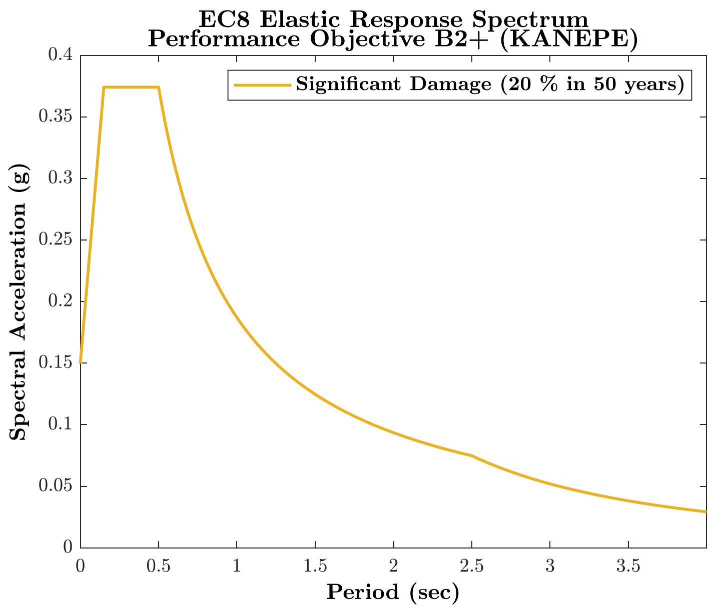
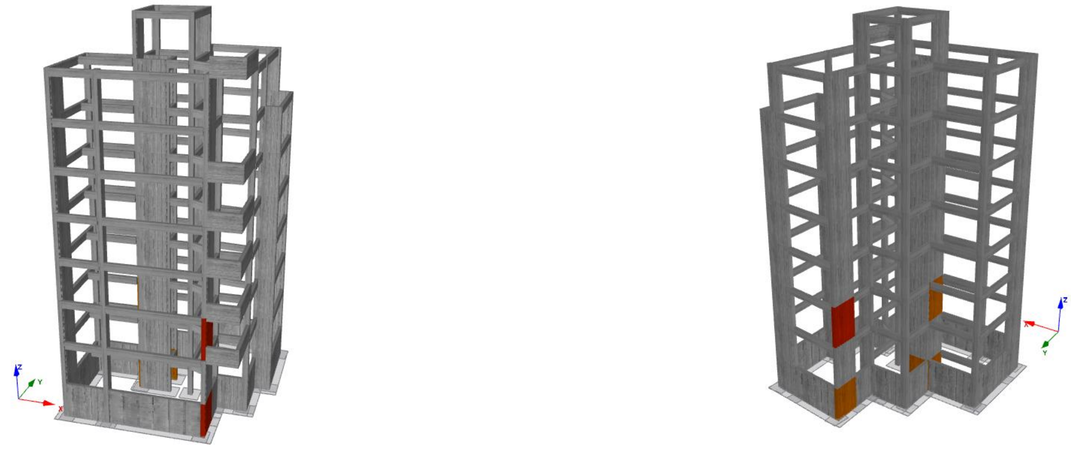


Figure 3.40 Seismic spectrum used for performance objective B2^{+} (KANEPE)

(a)



(b)

Figure 3.41 Shear strength check for performance objective B2^{+} – Without Infills (KANEPE) (a) X-Direction, (b) Y-Direction – **Failure**

First of all, only the first model (without infills) was examined, as there are no major differences. The building has not achieved even this minimum performance objective for the performance level "Significant Damage", although the failed members are less in both directions compared to the previous results (Figure 3.29 and Figure 3.33). More specifically:

- "Significant Damage" (X-Direction) → 4 failed members
    - Wall W5 – floors: -1 → Performance Ratio: 1.12
    - Wall W6 – floors: -1, 1, 2 → Performance Ratio: 1.36 – 1.22
- "Significant Damage" (Y-Direction) → 7 failed members
    - Wall W1 – floors: -1, 2 → Performance Ratio: 1.35 – 1.19
    - Wall W4 – floors: -1, 2 → Performance Ratio: 1.51 – 1.49
    - Wall W5 – floors: -1 → Performance Ratio: 1.01
    - Wall W7 – floors: -1, 2 → Performance Ratio: 1.07 – 1.04

So, as described in § 1.1, the uniform hazard spectrum from EFEHR[8] was used to compare the results with the EC8 spectrum. The seismic spectrum is shown in the figure below.

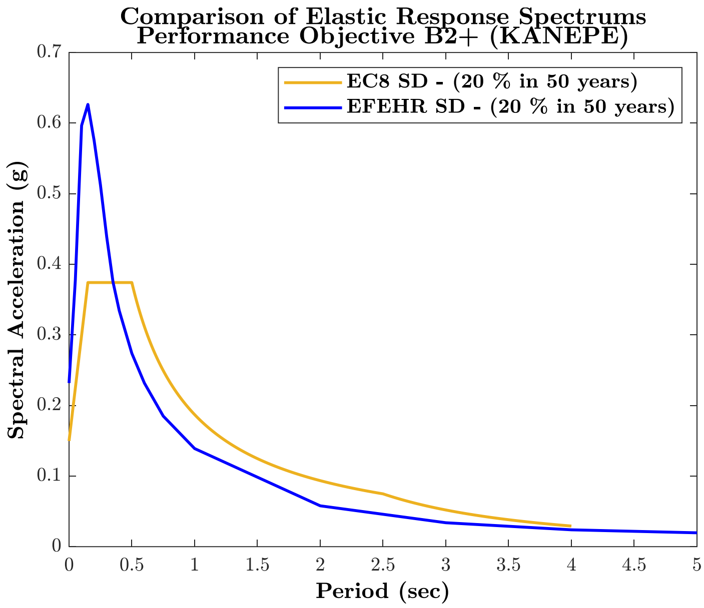


Figure 3.42 Seismic spectrum according to EFEHR for performance objective B2^{+} (KANEPE)

The EFEHR[8] spectrum uses the new ESHM20 model and as seen in the figure it gives lower values than the EC8 spectrum in higher periods, meaning that for this building with $T_{eff} \cong 0.80$ sec, the results would be better than before. Figure 3.43 show the shear strength check resulted from this spectrum.

(a)

(b)

Figure 3.43 Shear strength check for performance objective B2+ using the EFEHR spectrum – Without Infills (KANEPE) (a) X-Direction, (b) Y-Direction – **Small Failure**

Using this spectrum, it is obvious that the results are much better compared to the EC8 spectrum but again the performance objective has not been achieved. However, it should be noted that the color is orange for all failed members, meaning that it is a small failure that could theoretically be neglected because it may be due to the accuracy of analysis performed by the software. The maximum performance ratio is 1.02.

3.3.3. Application of Eurocode 8 – Part 3

The performance levels and spectrums used in EN 1998-3[4] are almost identical to those of KANEPE[3]. Figure 3.44 shows the spectrums and Table 3.25 the performance objectives used to evaluate the building.

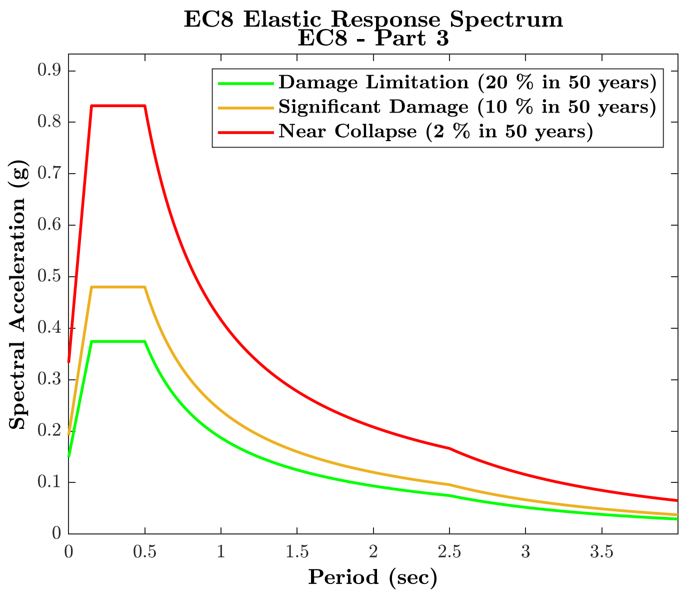


Figure 3.44 Seismic spectrums used for the assessment of the building (EC8)

Table 3.25 Performance objectives for the assessment of the examined building (EC8)

Performance Level	Probability of Exceedance in 50 years	Return Period (years)
Damage Limitation	20 %	225
Significant Damage	10 %	475
Near Collapse	2 %	2475

According to EN 1998-3[4], sixteen (16) combinations used for the assessment of the existing building. More specifically:

- Uniform ± X ± eccY
- Uniform ± Y ± eccX
- Modal ± X ± eccY
- Modal ± Y ± eccX

Figure 3.45 - Figure 3.47 show the capacity curve and checks in the x-direction and Figure 3.48 - Figure 3.50 in the y-direction for the model without infills. Correspondingly, Figure 3.51 - Figure 3.53 show the results in the x-direction and Figure 3.54 - Figure 3.56 in the y-direction for the model with infills.

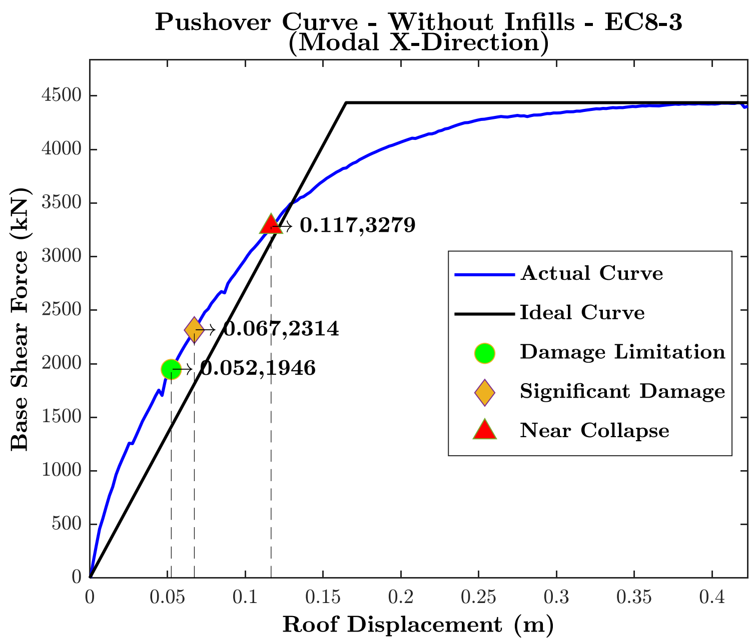


Figure 3.45 Pushover curve in the x-direction – Without Infills (EC8)

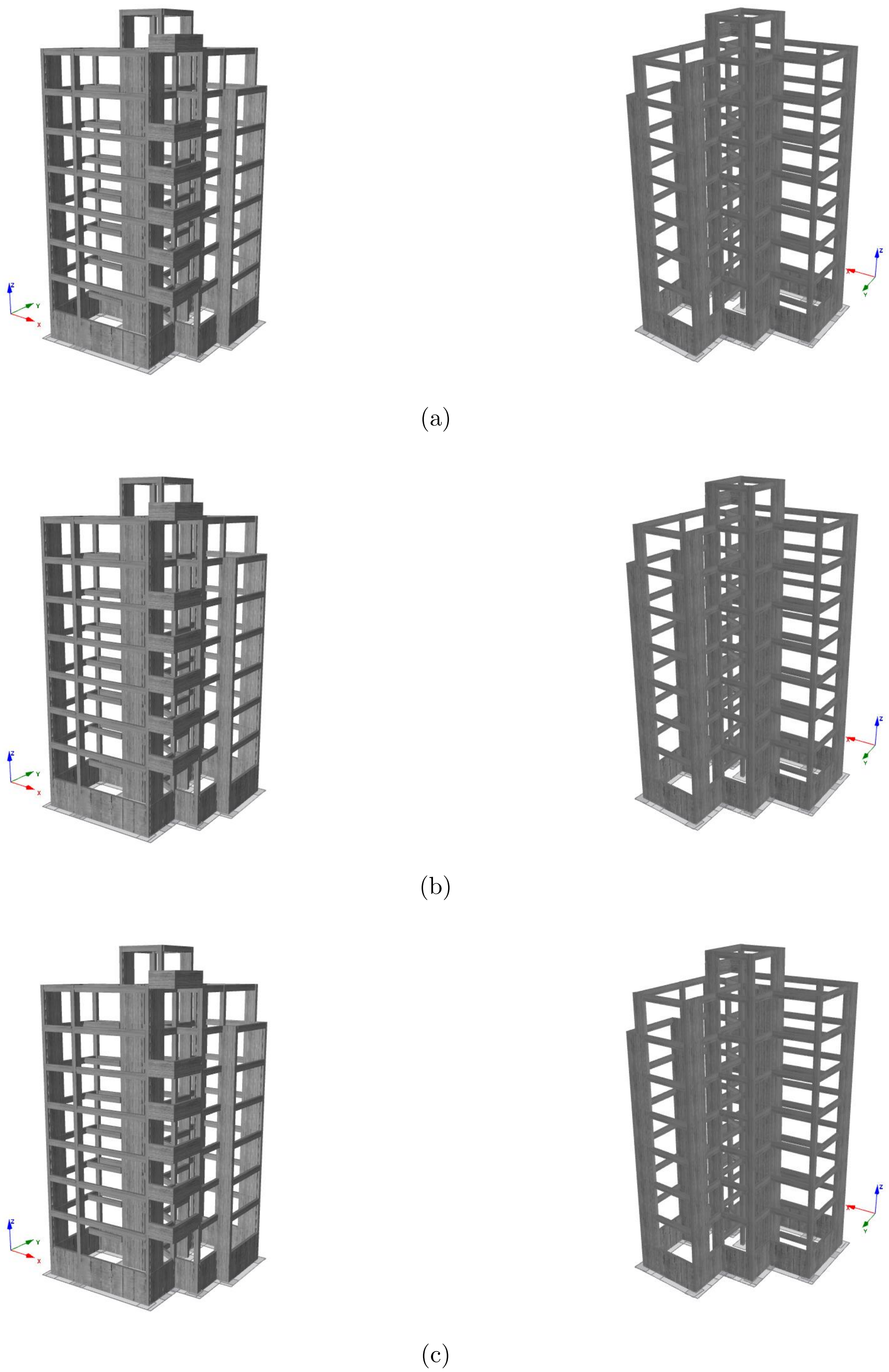


Figure 3.46 Chord rotation check in the x-direction – Without Infills (EC8)
(a) Damage Limitation, (b) Significant Damage, (c) Near Collapse – **No Failure**

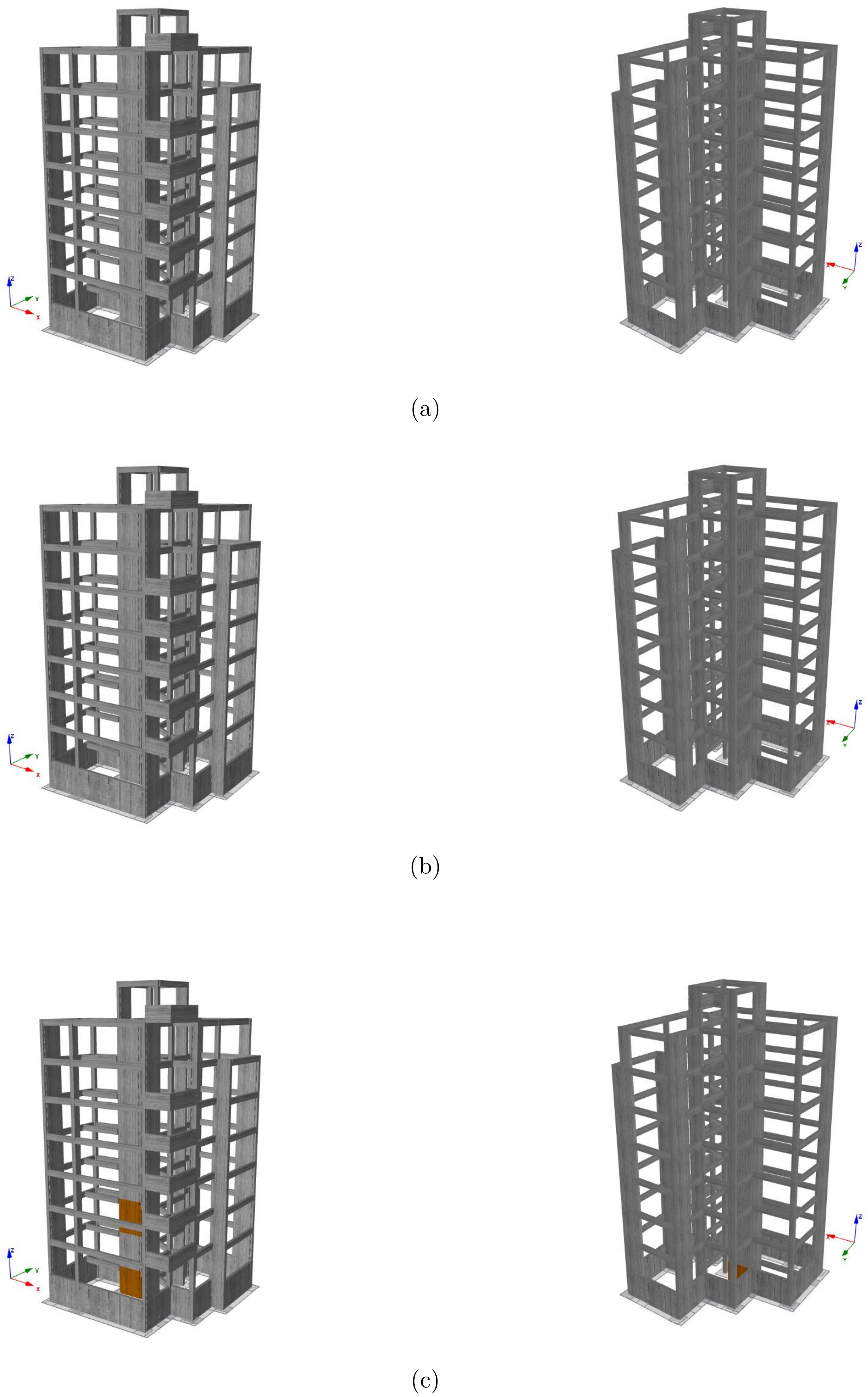


Figure 3.47 Shear strength check in the x-direction – Without Infills (EC8)
(a) Damage Limitation, (b) Significant Damage, (c) Near Collapse – **Small Failure**

As shown in the figures above, there is no problem in the chord rotation check and the shear strength, where the performance objective is achieved in all limit states, except the third limit state where there is a little failure (Performance Ratio 1.01) which is negligible.

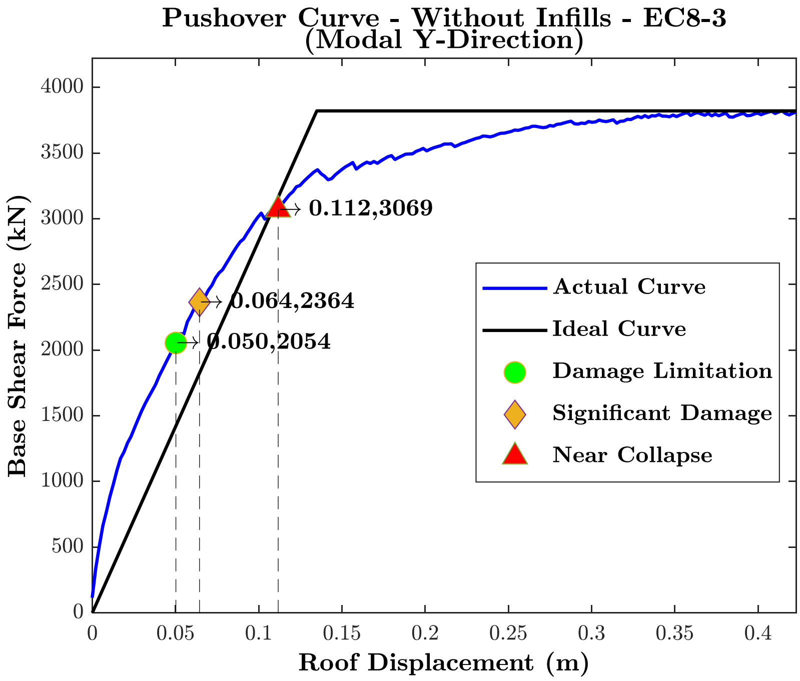
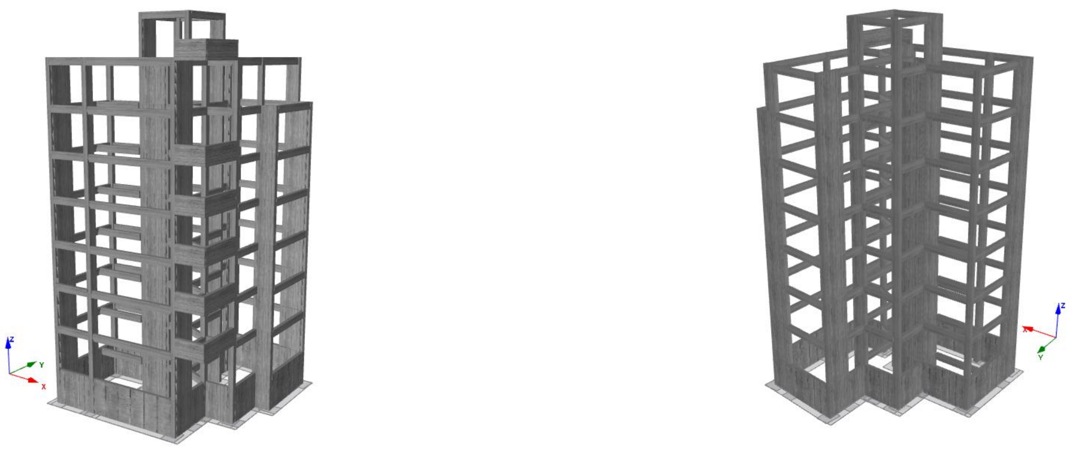
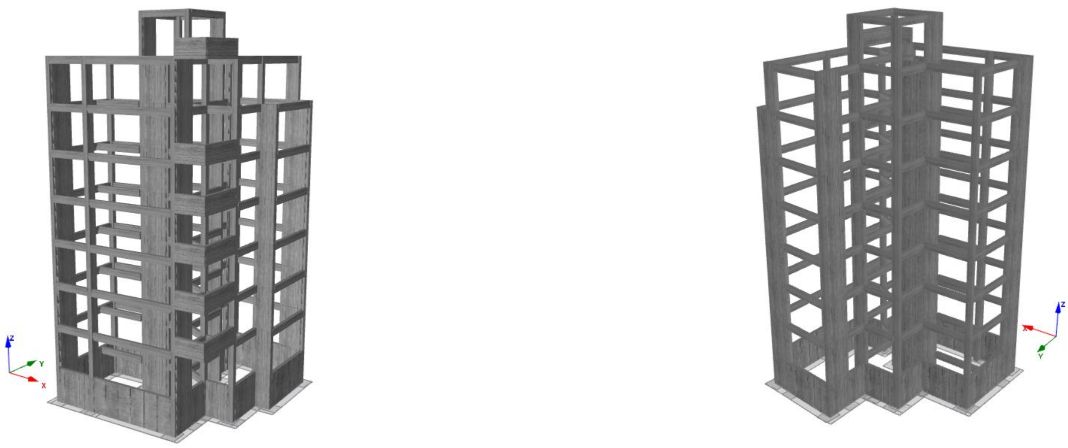


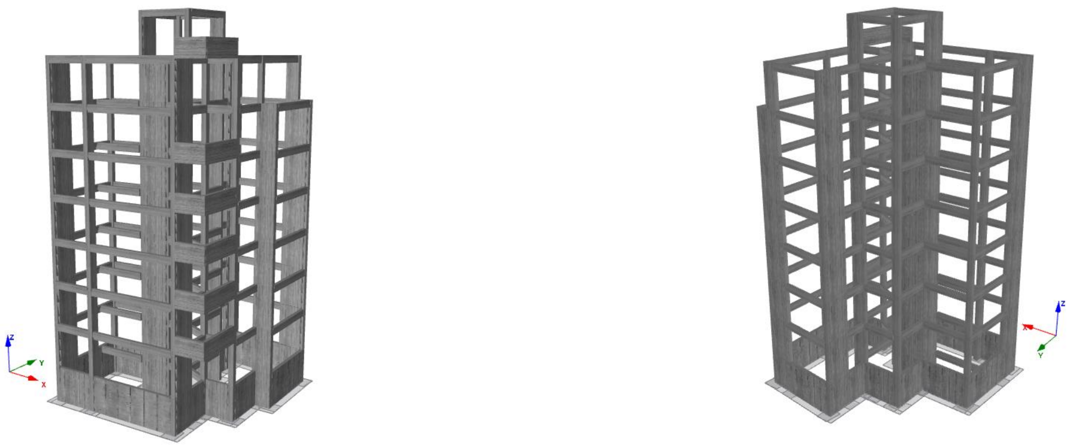
Figure 3.48 Pushover curve in the y-direction – Without Infills (EC8)



(a)

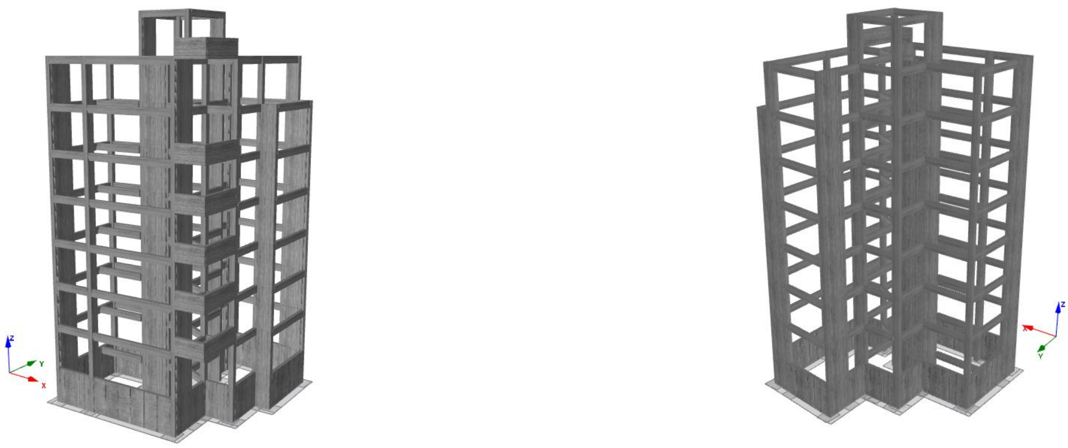


(b)

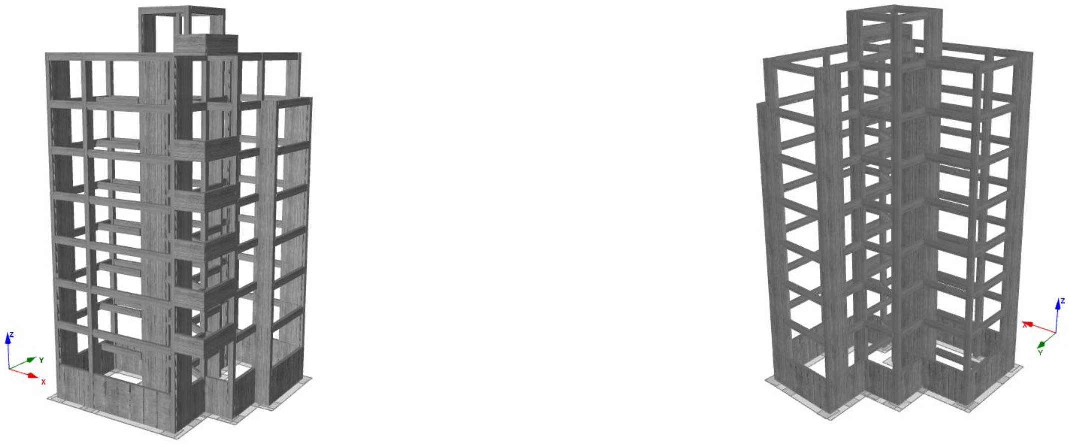


(c)

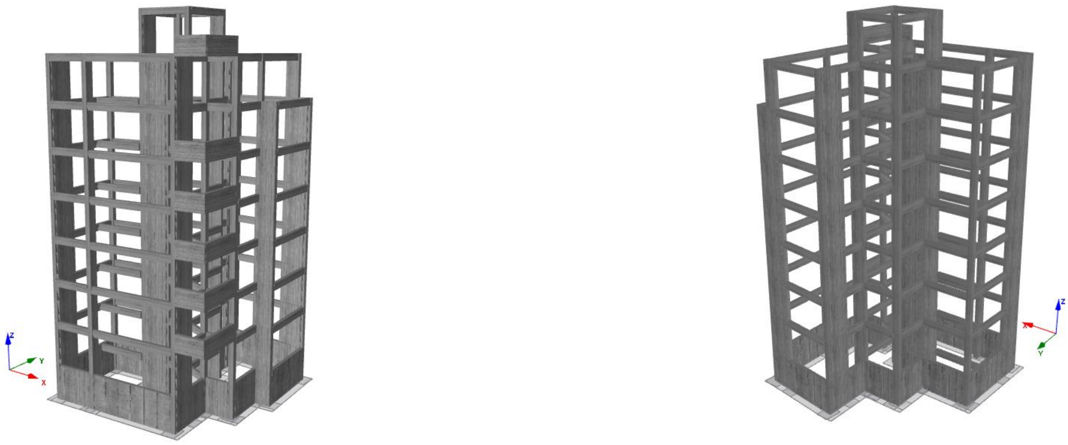
Figure 3.49 Chord rotation check in the y-direction – Without Infills (EC8)
(a) Damage Limitation, (b) Significant Damage, (c) Near Collapse – **No Failure**



(a)



(b)



(c)

Figure 3.50 Shear strength check in the y-direction – Without Infills (EC8)
(a) Damage Limitation, (b) Significant Damage, (c) Near Collapse – **No Failure**

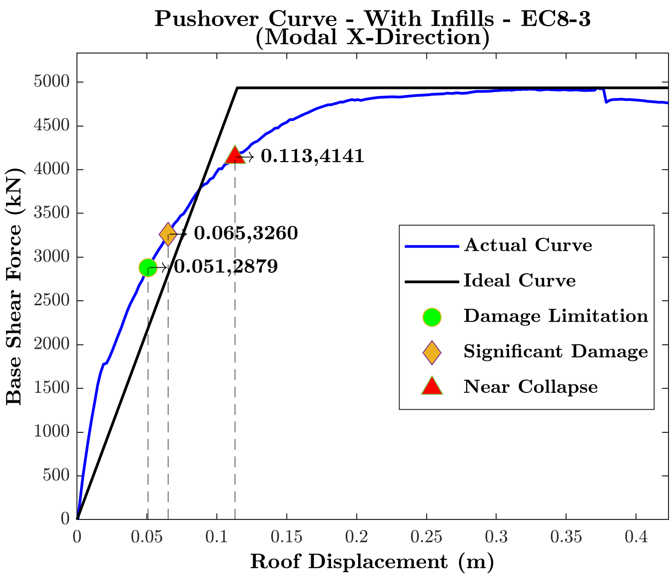
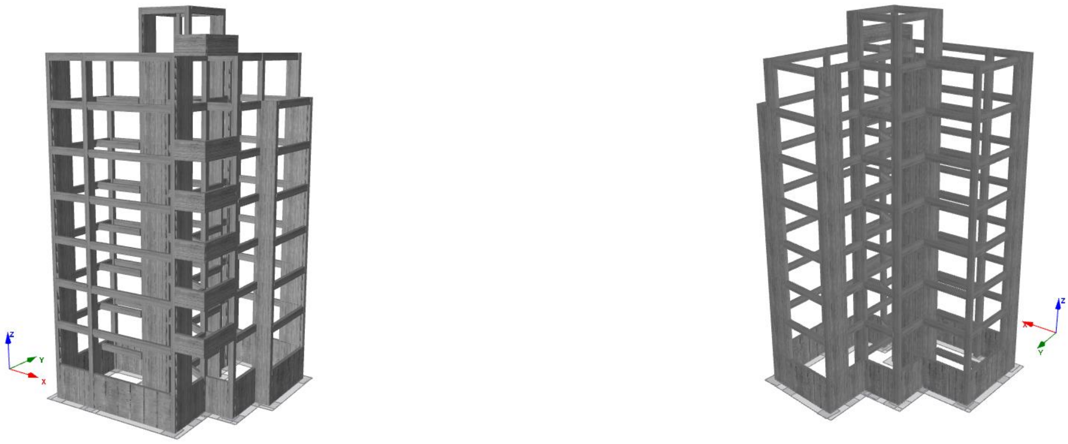
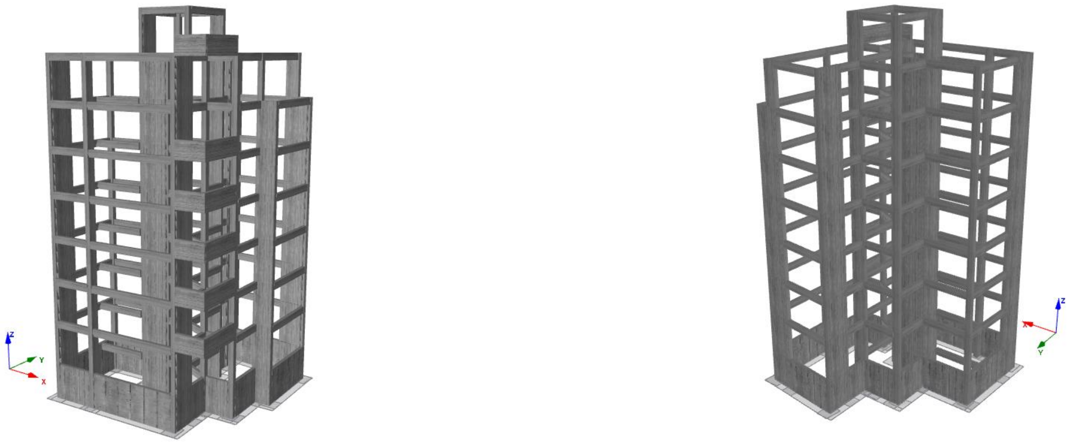


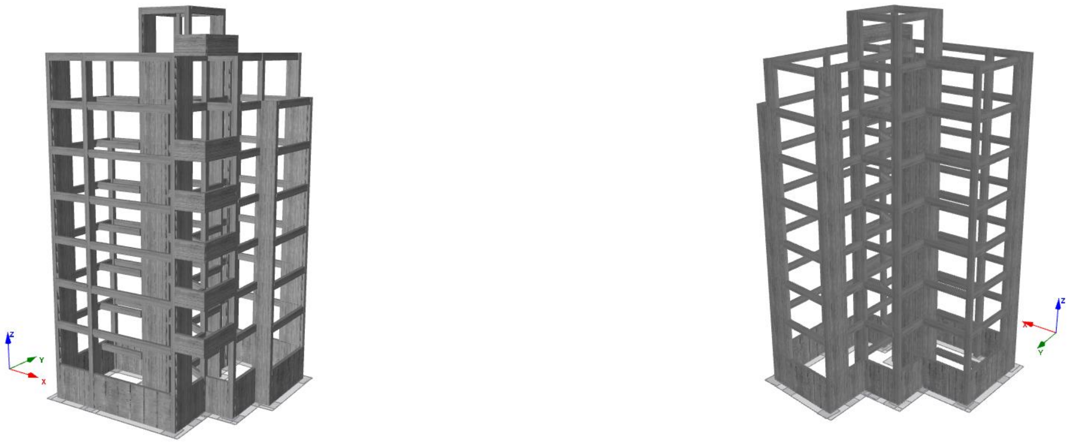
Figure 3.51 Pushover curve in the x-direction – With Infills (EC8)



(a)

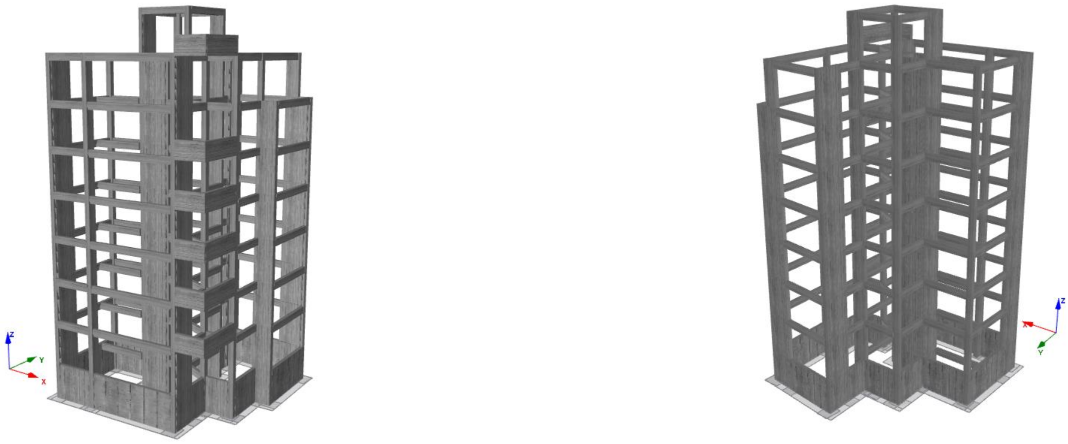


(b)

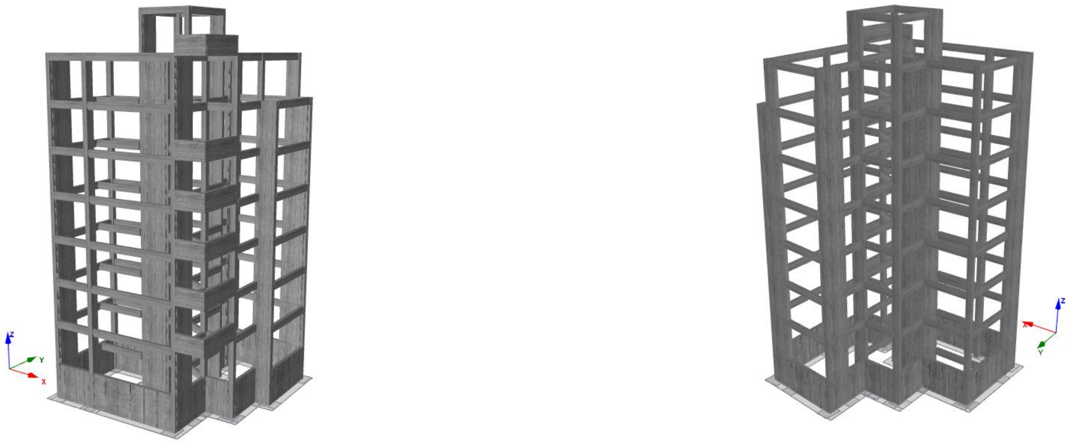


(c)

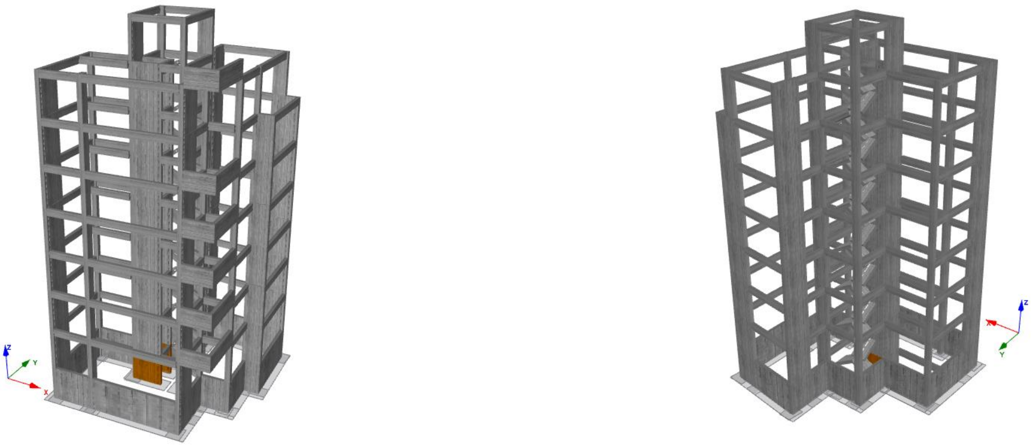
Figure 3.52 Chord rotation check in the x-direction – With Infills (EC8)
(a) Damage Limitation, (b) Significant Damage, (c) Near Collapse – **No Failure**



(a)



(b)



(c)

Figure 3.53 Shear strength check in the x-direction – With Infills (EC8)
(a) Damage Limitation, (b) Significant Damage, (c) Near Collapse – **Small Failure**

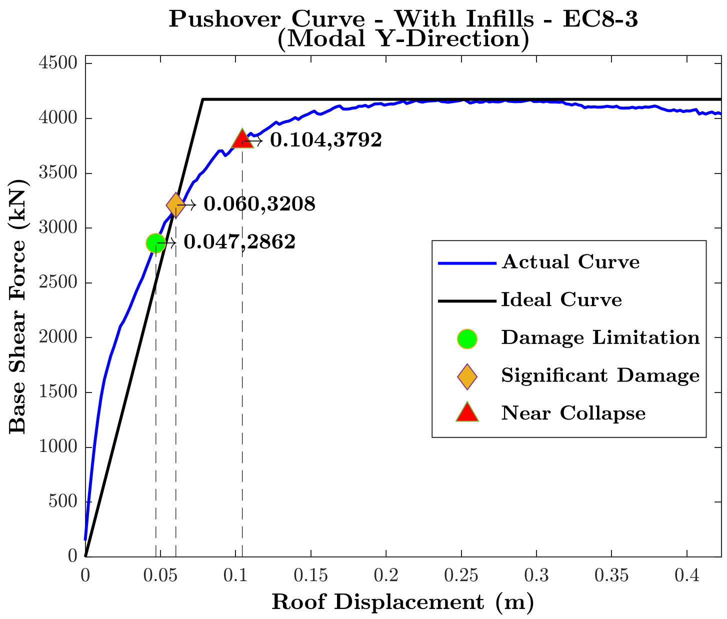
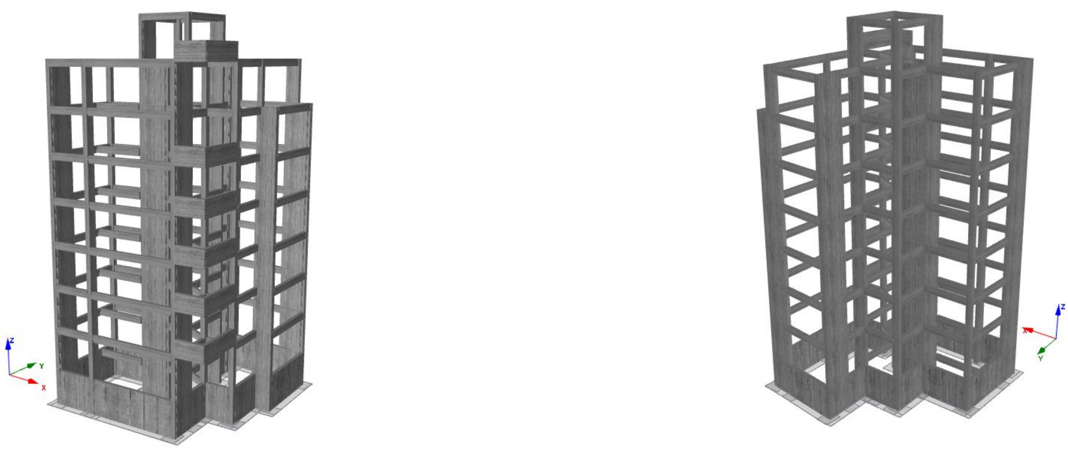
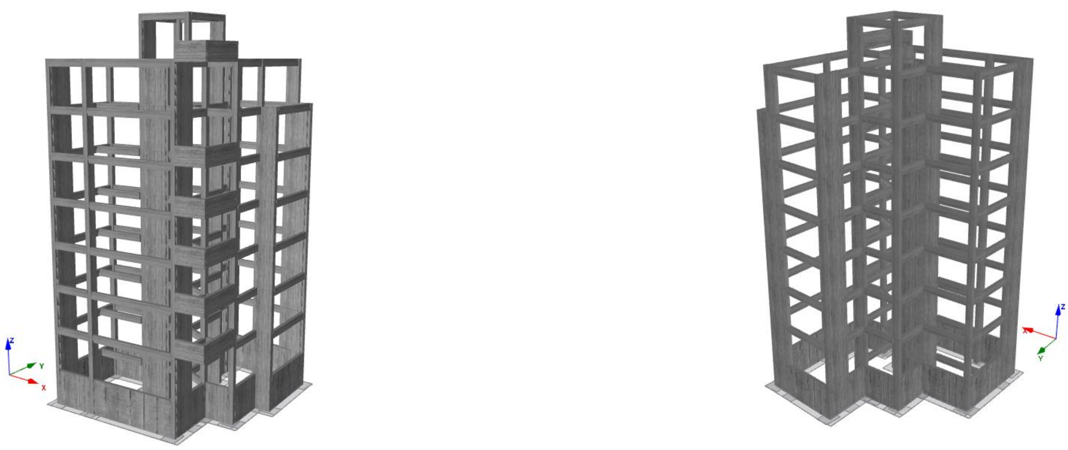


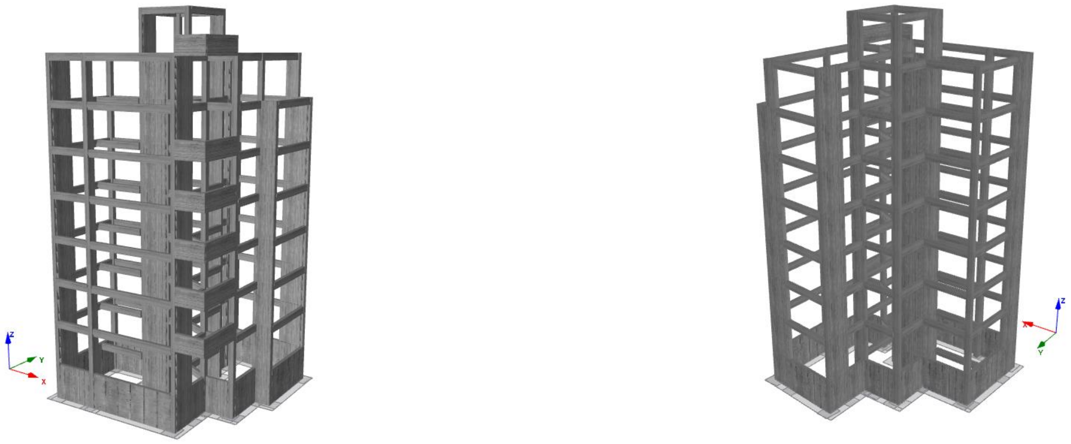
Figure 3.54 Pushover curve in the y-direction – With Infills (EC8)



(a)

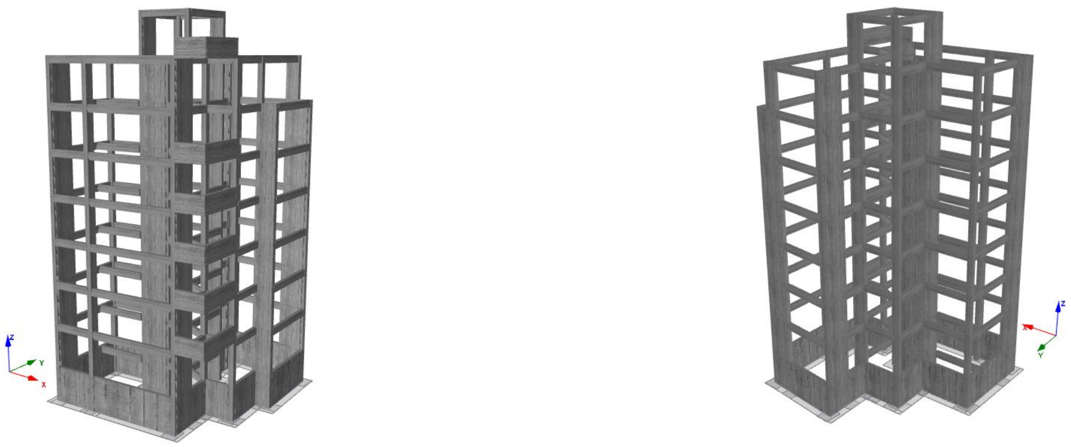


(b)

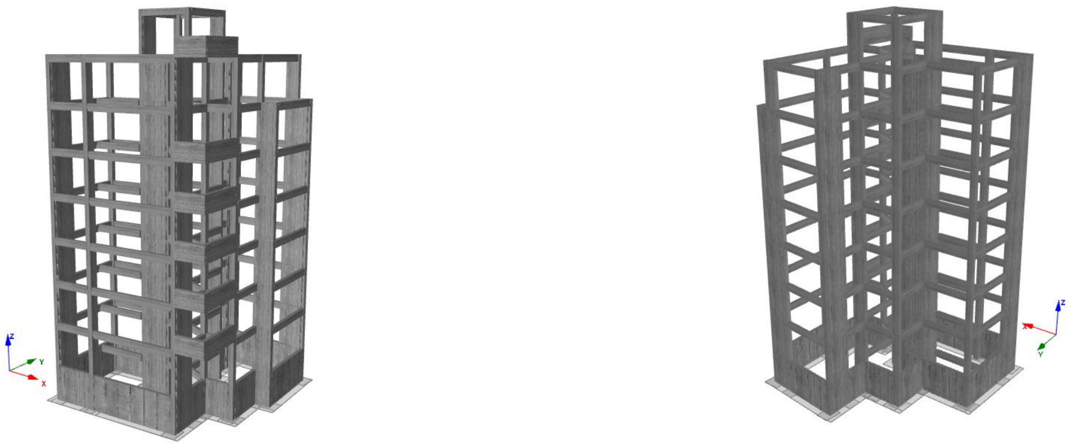


(c)

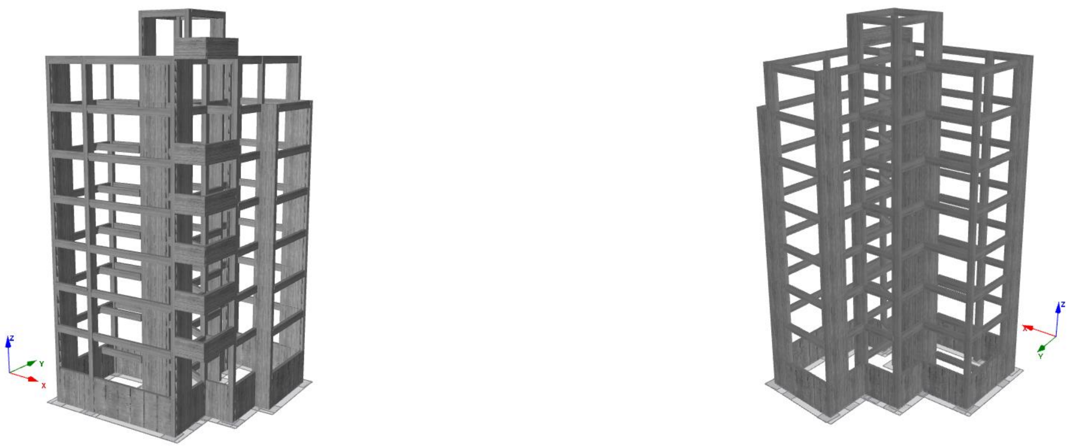
Figure 3.55 Chord rotation check in the y-direction – With Infills (EC8)
(a) Damage Limitation, (b) Significant Damage, (c) Near Collapse – **No Failure**



(a)



(b)



(c)

Figure 3.56 Shear strength check in the y-direction – With Infills (EC8)
(a) Damage Limitation, (b) Significant Damage, (c) Near Collapse – **No Failure**

Comparing the results obtained from the two models (without/with infills), it is apparent that there is not a big difference. The chord capacity check and shear strength check achieved for almost all limit states, with one exception in the "Near Collapse" performance level in the x-direction, where there is a small failure but it is negligible since the performance ratio is 1.03.

3.3.4. Application of ASCE/SEI 41-17

The performance levels and spectrums used in ASCE/SEI 41-17[5] are slightly different compared to those of KANEPE[3] and EN 1998-3[4], because the American code use an earthquake with a lower intensity of seismic action. Figure 3.57 shows the spectrums and Table 3.26 the performance objectives used to evaluate the building.

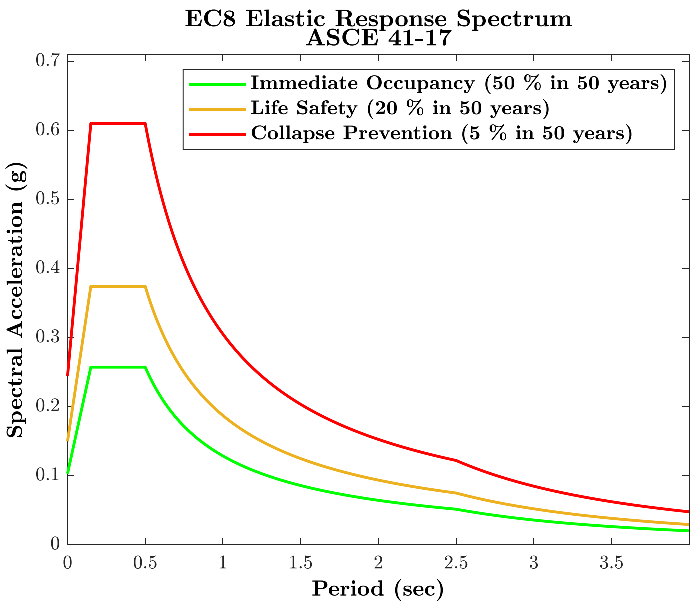


Figure 3.57 Seismic spectrums used for the assessment of the building (ASCE)

Table 3.26 Performance objectives for the assessment of the examined building (ASCE)

Performance Level	Probability of Exceedance in 50 years	Return Period (years)
Immediate Occupancy	50 %	73
Life Safety	20 %	225
Collapse Prevention	5 %	975

According to ASCE/SEI 41-17, eight (8) combinations used for the assessment of the existing building. More specifically:

- Modal ± X ± eccY
- Modal ± Y ± eccX

Figure 3.58 - Figure 3.63 show the capacity curve and checks for the model without infills, while Figure 3.64 - Figure 3.69 show the results for the model with infills.

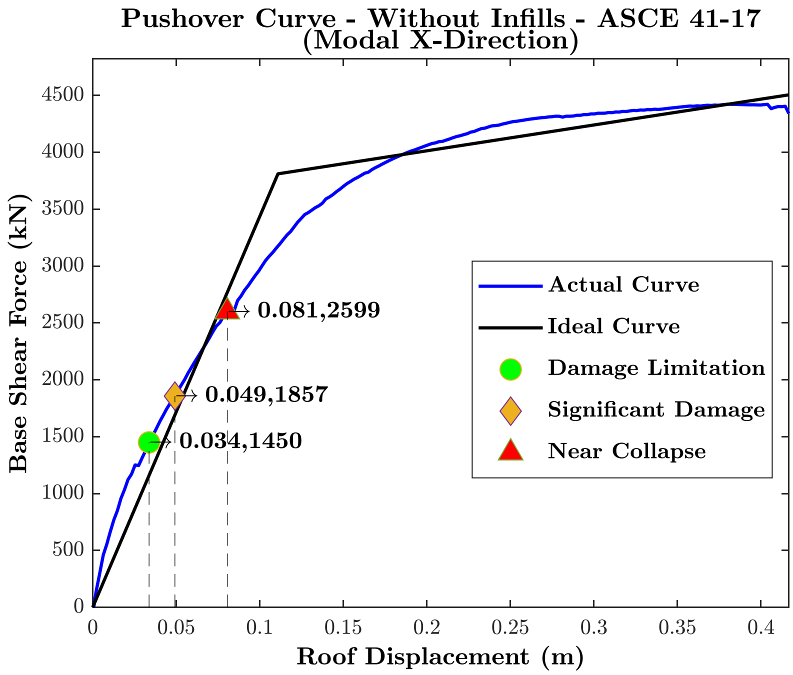
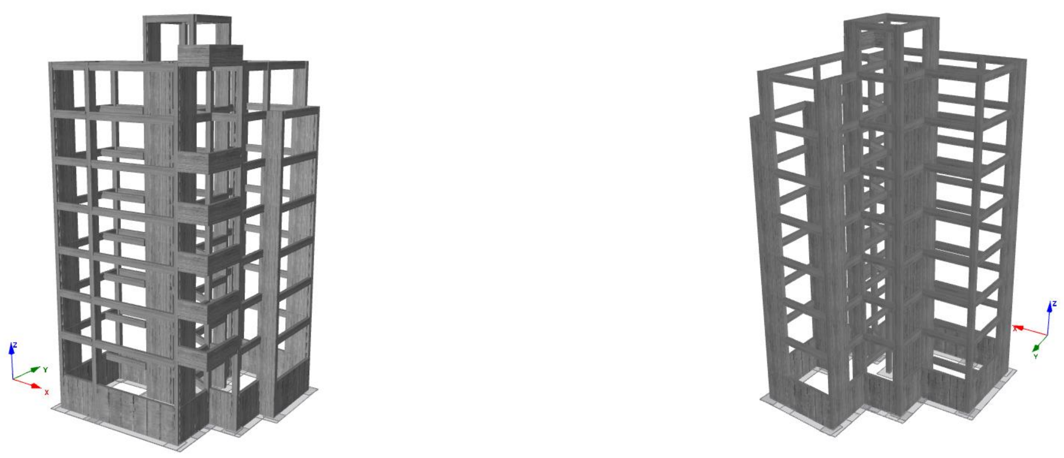


Figure 3.58 Pushover curve in the x-direction – Without Infills (ASCE)

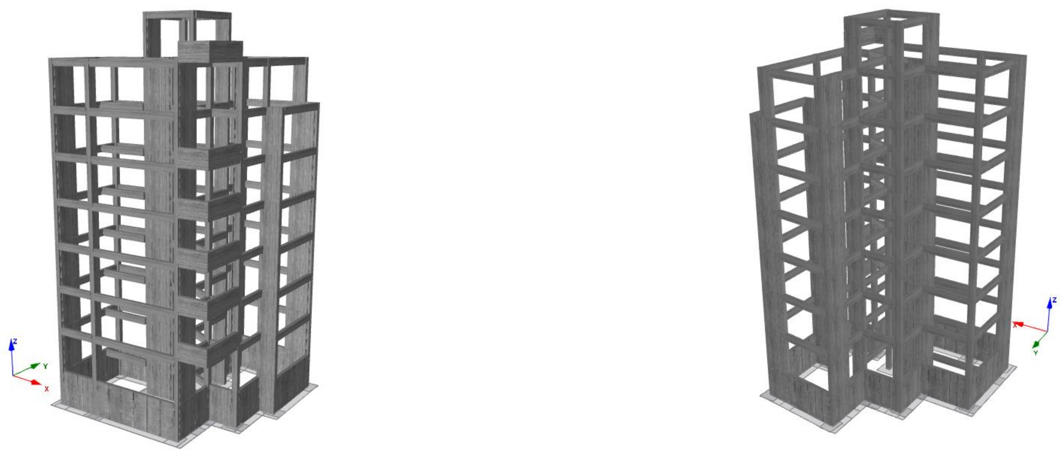
(a)

(b)



(c)

Figure 3.59 Chord rotation check in the x-direction – Without Infills (ASCE)
(a) Damage Limitation, (b) Significant Damage, (c) Near Collapse – **No Failure**



(a)

(b)

(c)

Figure 3.60 Shear strength check in the x-direction – Without Infills (ASCE)
(a) Damage Limitation, (b) Significant Damage, (c) Near Collapse – **Small Failure**

As shown in the figures above, there is no problem in the chord rotation check and the shear strength, where the performance objective is achieved in all limit states, except the third limit state where there is a little failure (Performance Ratio 1.02) which is negligible.

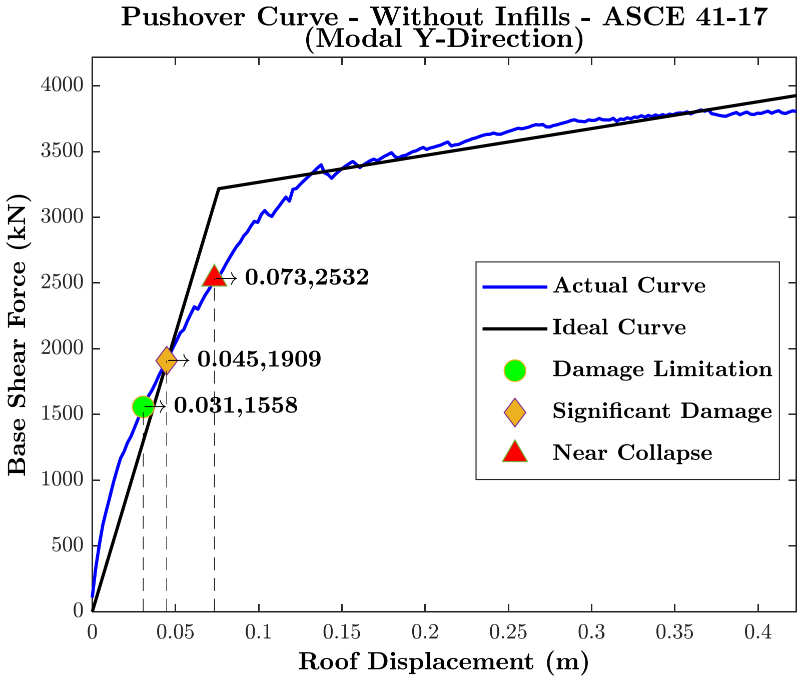
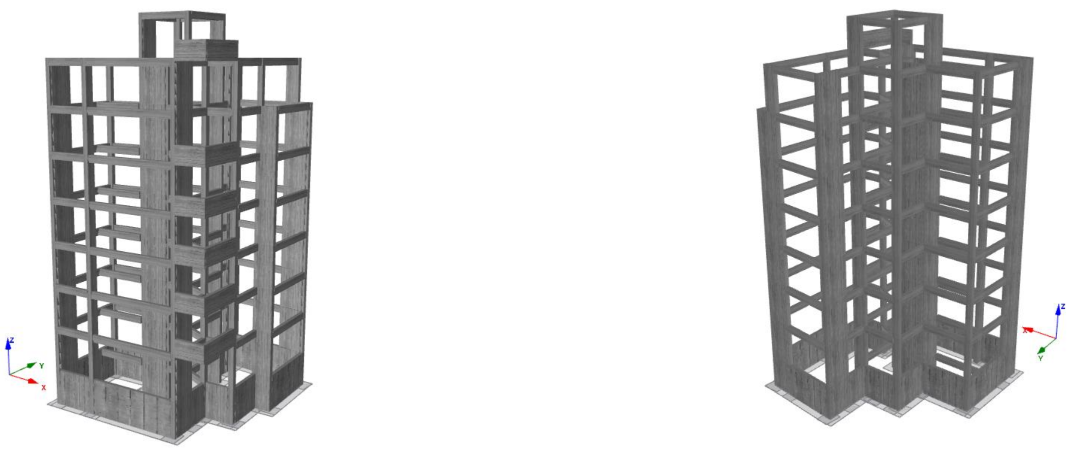
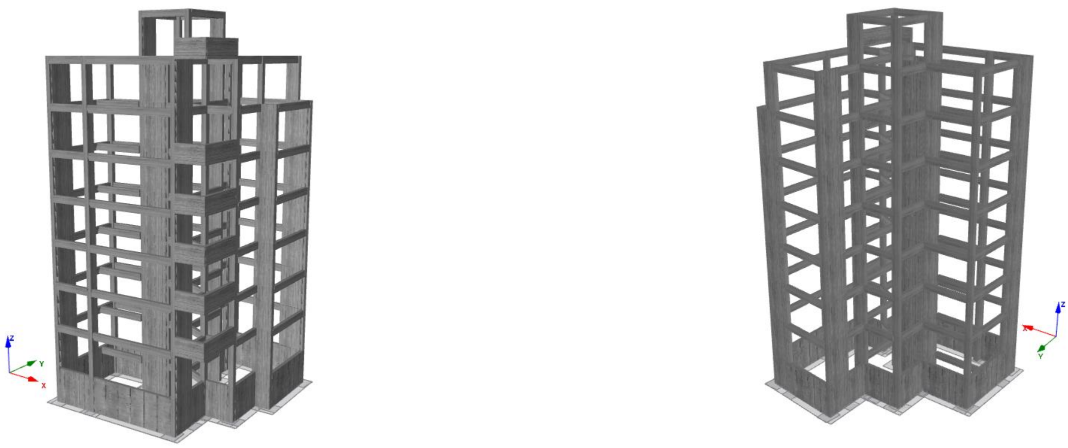


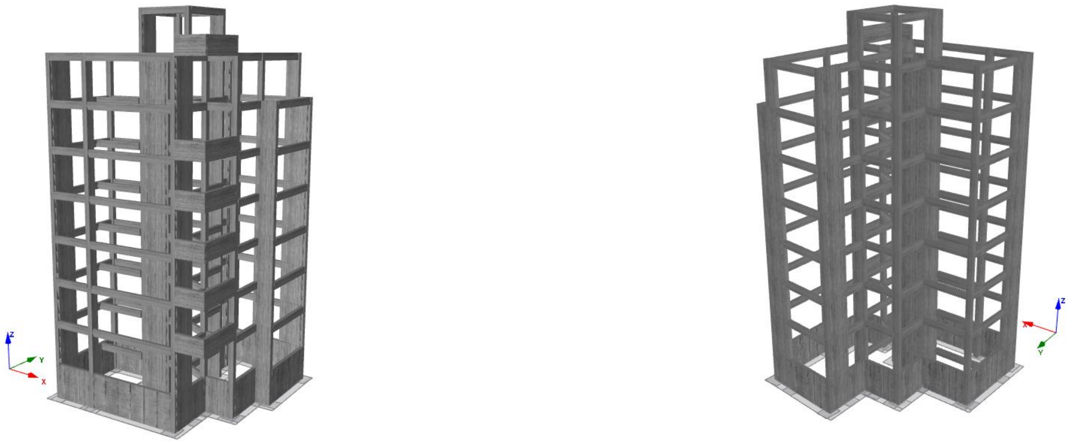
Figure 3.61 Pushover curve in the y-direction – Without Infills (ASCE)



(a)

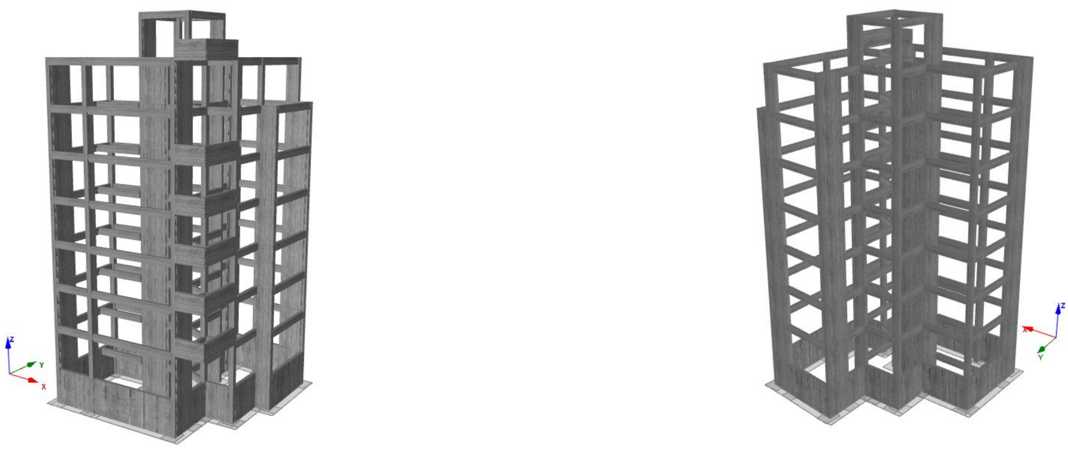


(b)

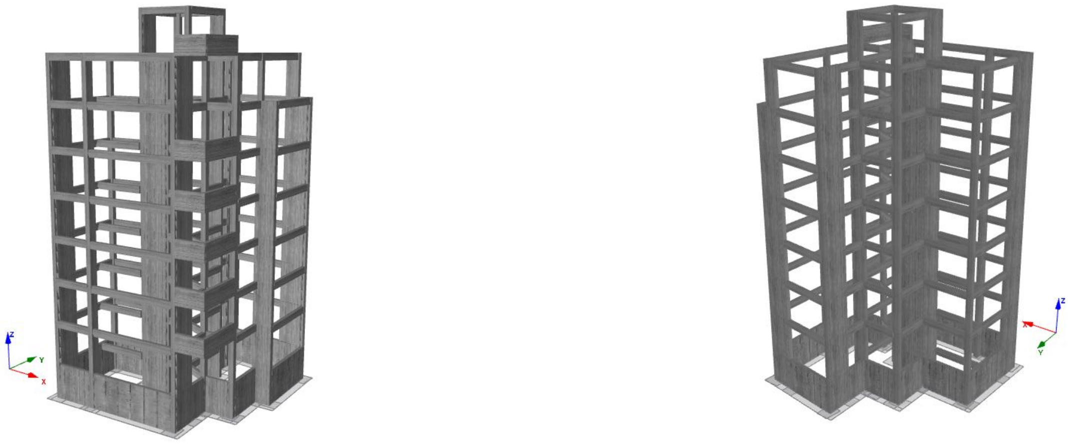


(c)

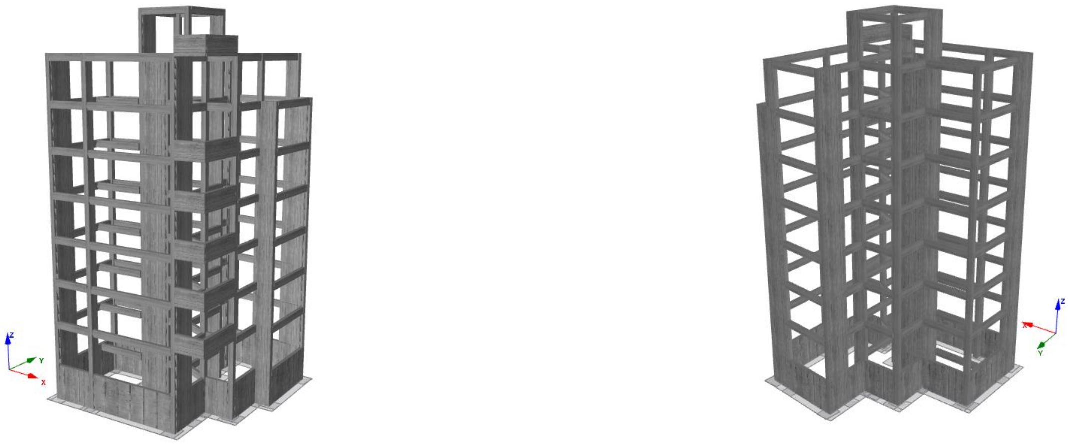
Figure 3.62 Chord rotation check in the y-direction – Without Infills (ASCE)
(a) Damage Limitation, (b) Significant Damage, (c) Near Collapse – **No Failure**



(a)



(b)



(c)

Figure 3.63 Shear strength check in the y-direction – Without Infills (ASCE)
(a) Damage Limitation, (b) Significant Damage, (c) Near Collapse – **No Failure**

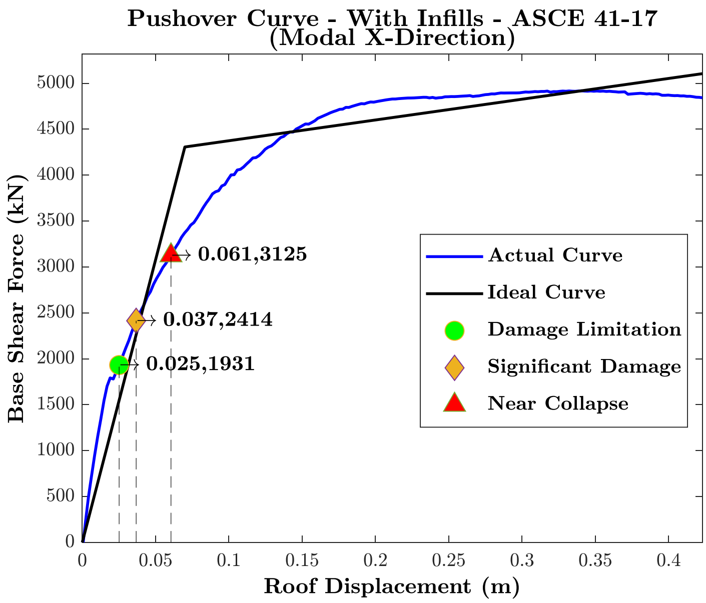
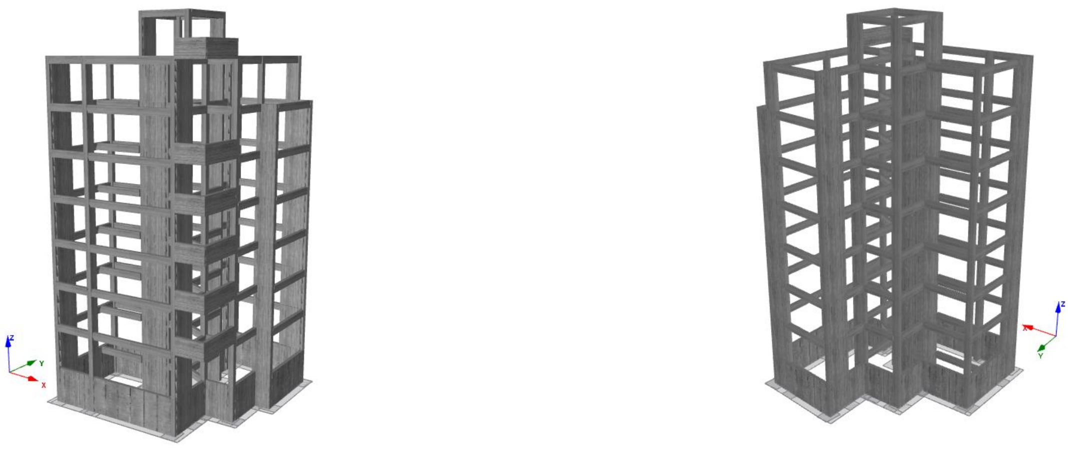
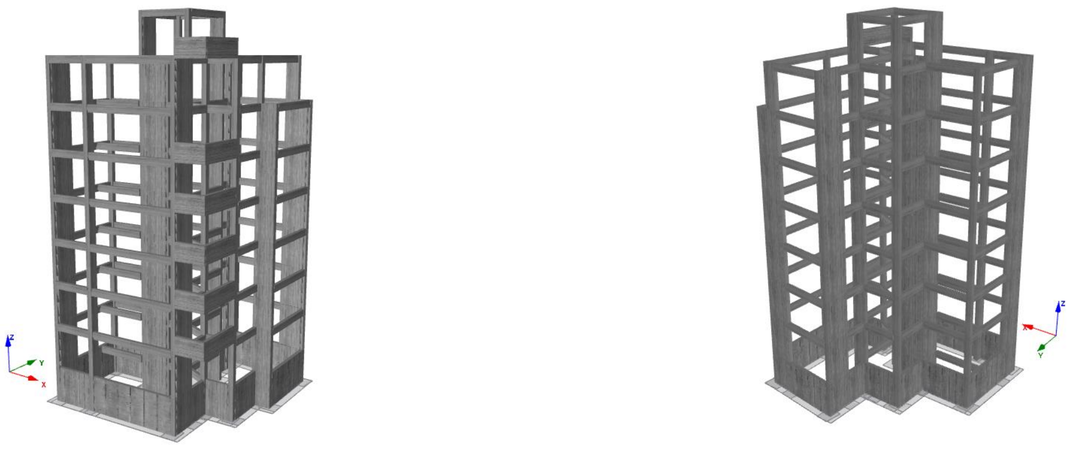


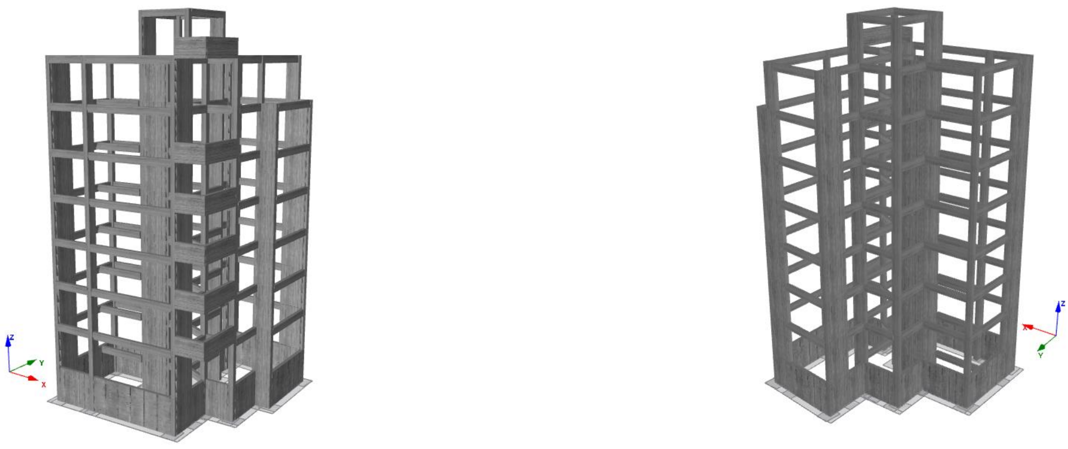
Figure 3.64 Pushover curve in the x-direction – With Infills (ASCE)



(a)

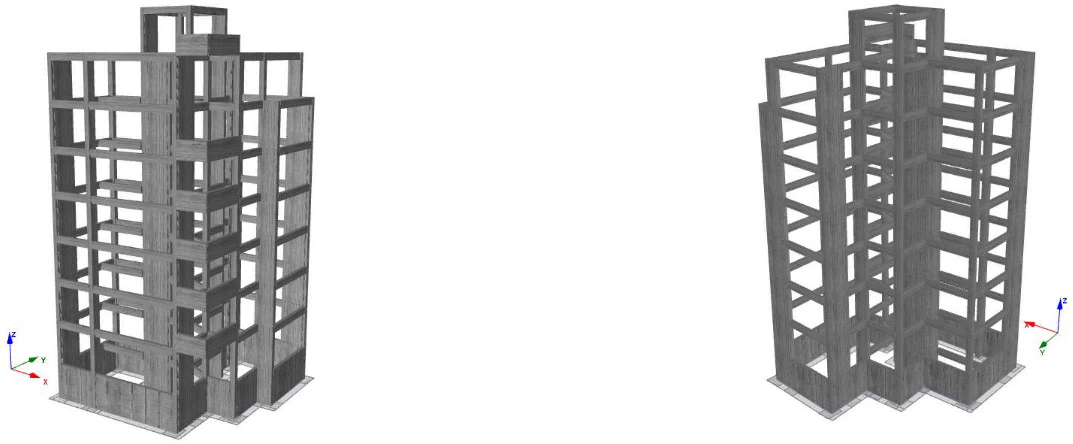


(b)

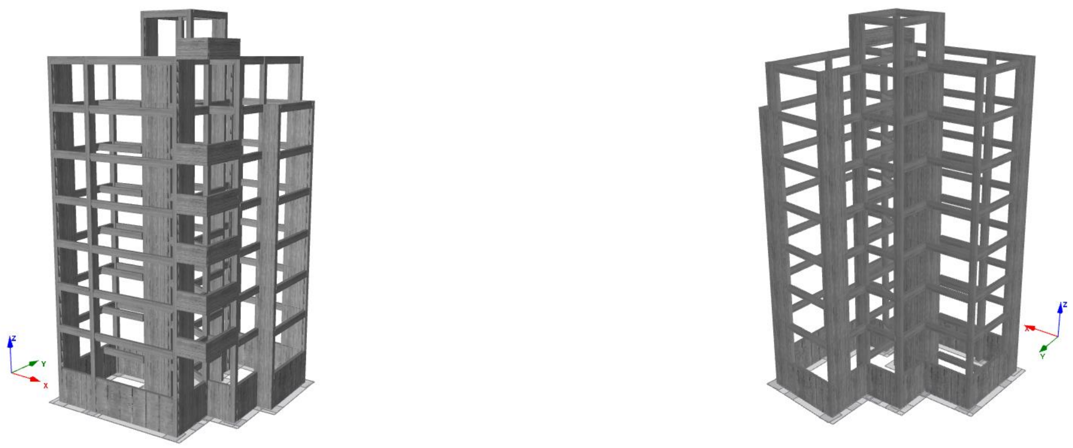


(c)

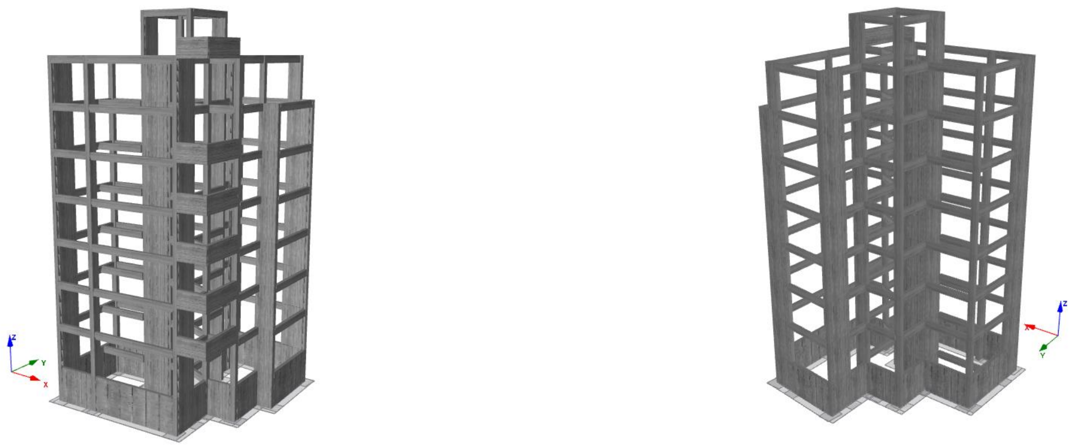
Figure 3.65 Chord rotation check in the x-direction – With Infills (ASCE)
(a) Damage Limitation, (b) Significant Damage, (c) Near Collapse – **No Failure**



(a)



(b)



(c)

Figure 3.66 Shear strength check in the x-direction – With Infills (ASCE)
(a) Damage Limitation, (b) Significant Damage, (c) Near Collapse – **No Failure**

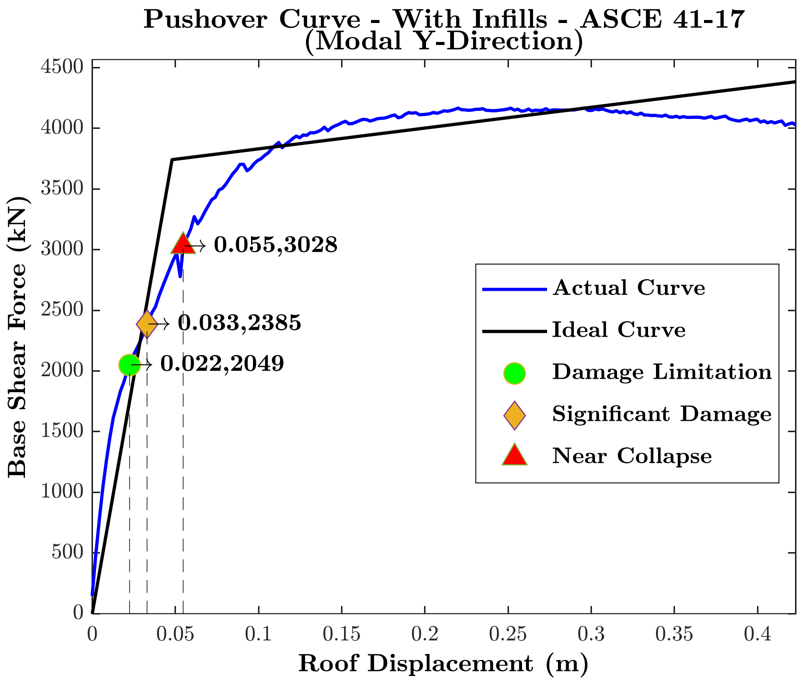
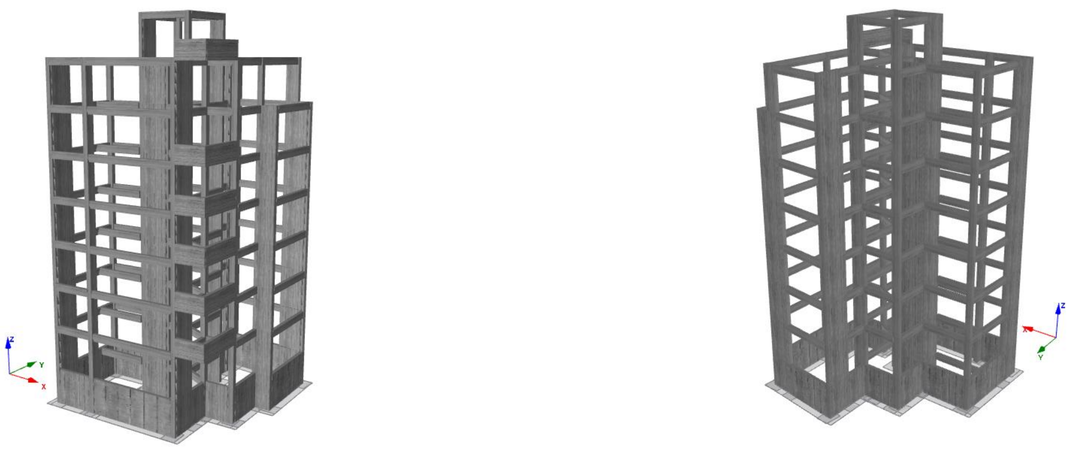
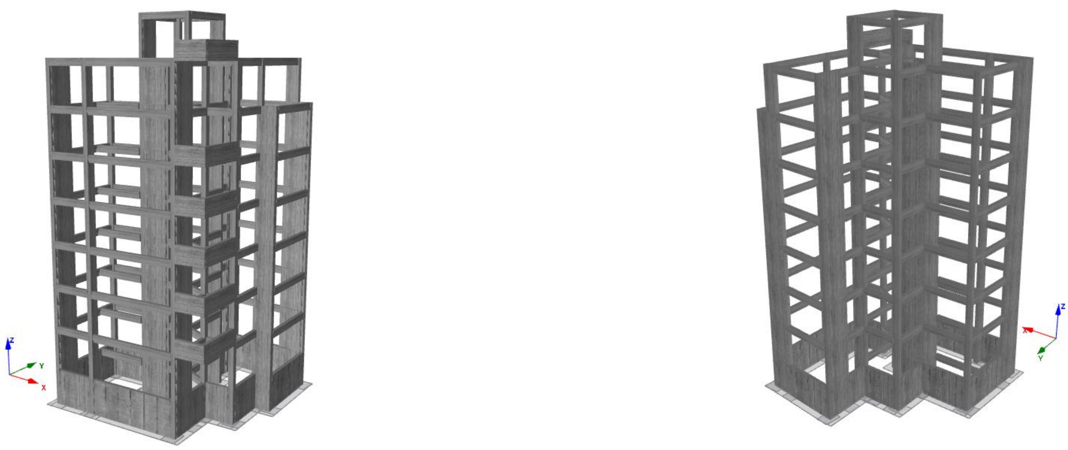


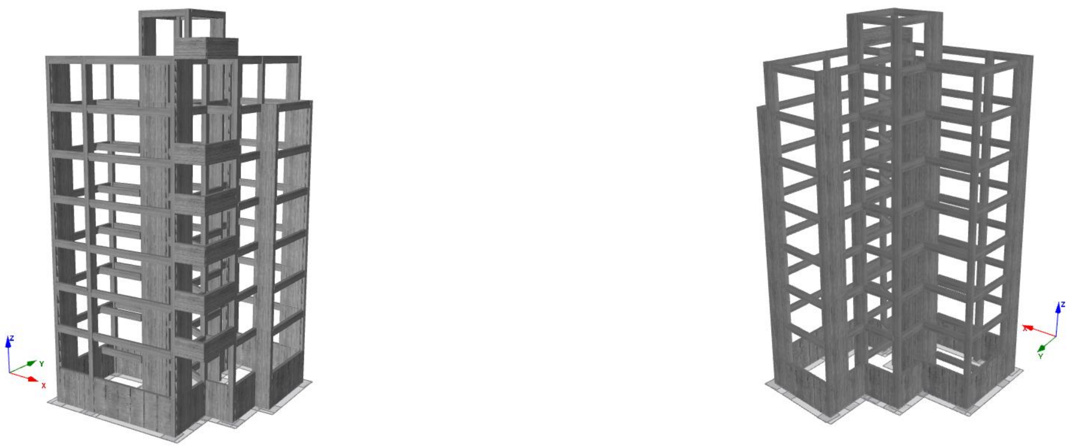
Figure 3.67 Pushover curve in the y-direction – With Infills (ASCE)



(a)

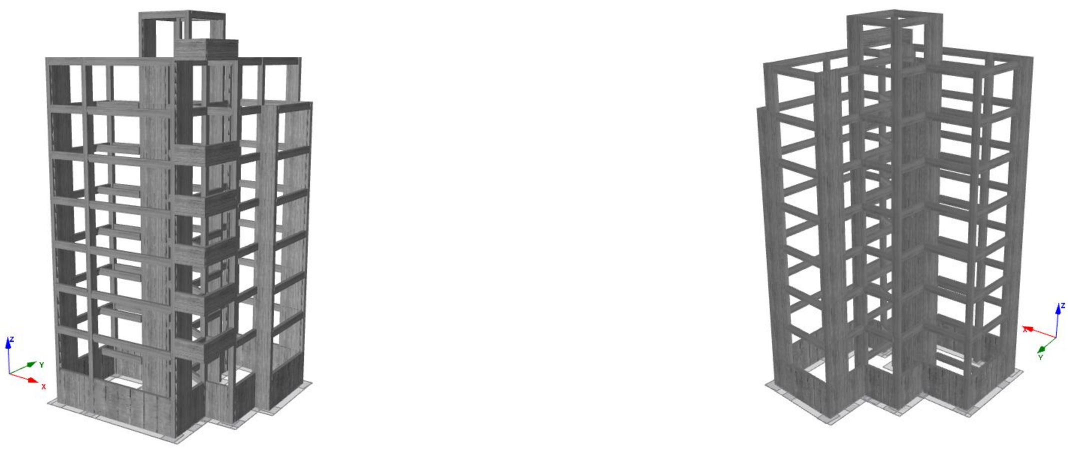


(b)

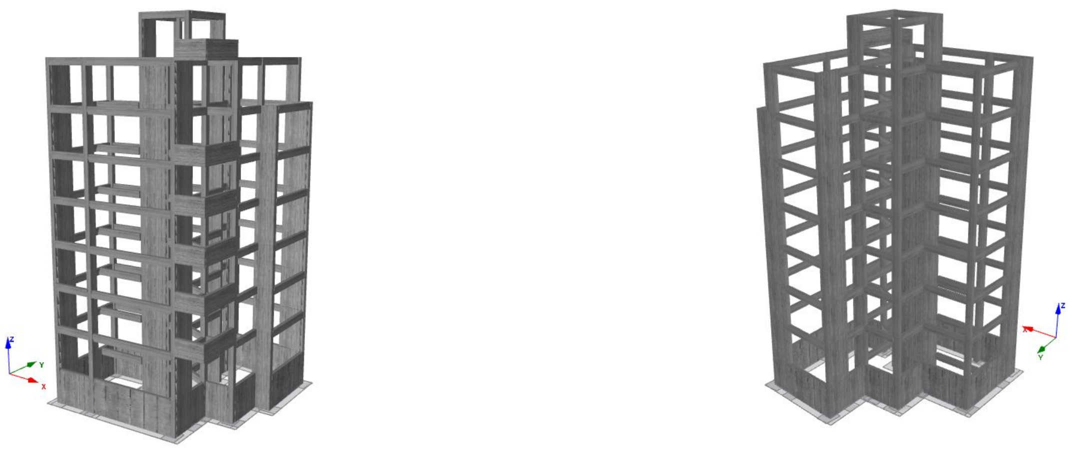


(c)

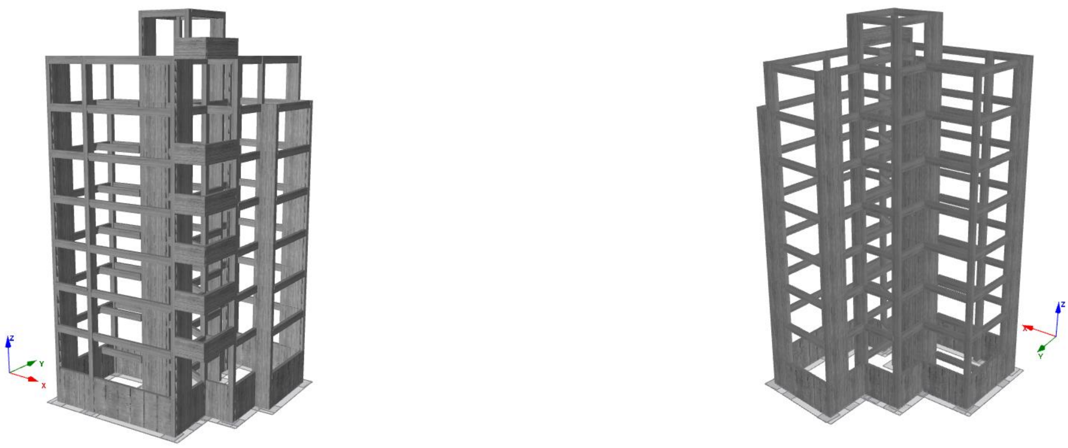
Figure 3.68 Chord rotation check in the y-direction – With Infills (ASCE)
(a) Damage Limitation, (b) Significant Damage, (c) Near Collapse – **No Failure**



(a)



(b)



(c)

Figure 3.69 Shear strength check in the y-direction – With Infills (ASCE)
(a) Damage Limitation, (b) Significant Damage, (c) Near Collapse – **No Failure**

3.4. Comparison of Results – Conclusions

If we see the results from KANEPE, it is necessary to carry out an investigation procedure in order to identify the acceleration at which all the checks are met. This procedure is carried out through tests gradually reducing ground acceleration while verifying the above checks, with the fundamental goal of finally being one step away from failure. This is also very important for the cost of the required repairs at different levels of damage (seismic loss assessment). It should be noted that the failure of the beams is not taken into account as described above.

This PGA value was determined using both standards (KANEPE and EC8) to see the differences. Also, we compared these results with those obtained with the second level pre-earthquake assessment methods (Dritsos S., Vougioukas E.).

Only the model without infills was used because, as we have already seen, there are not much differences due to the presence of walls. After all tests, it was found, for both standards (KANEPE and EC8), that in the x-direction the first failure occurs in the basement wall W6 and in the y-direction in the basement wall W4. As shown in Table 3.27 and Figure 3.70 - Figure 3.71, there is a large difference between KANEPE and EC8 in both directions since KANEPE's relationships for shear strength calculation are conservative. In addition, when comparing these results with those of the second level methods, we conclude that, firstly, the Vougioukas method gives very large values, since it does not use the seismic vulnerability criteria and also assumes that all vertical members exhaust all their strength simultaneously, and secondly, the shear capacity according to EC8 is similar to Dritsos S.'s method, which means that the actual value might be in this range.

Table 3.27 Comparison of the shear strength of the building

Method	Direction	PGA(g)	Shear Strength (kN)
KANEPE	X	0.070	1452
KANEPE	Y	0.055	1380
EC8	X	0.220	2860
EC8	Y	0.170	2420
Dritsos S.	X	0.208	2632
Dritsos S.	Y	0.165	2092
Vougioukas	X	0.384	4859
Vougioukas	Y	0.273	3450

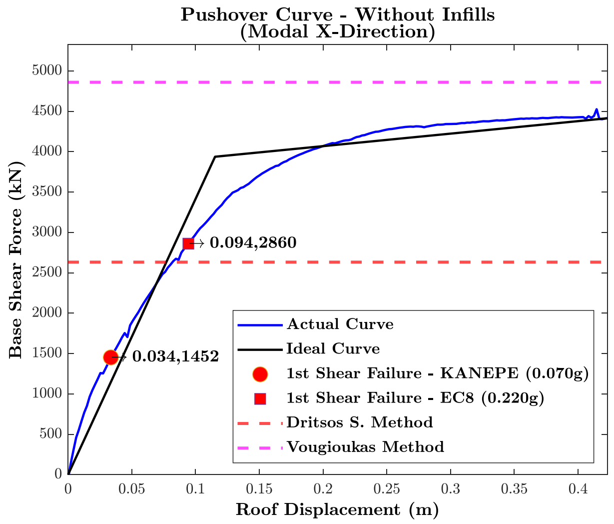


Figure 3.70 Comparison of the shear strength of the building – (X-Direction)

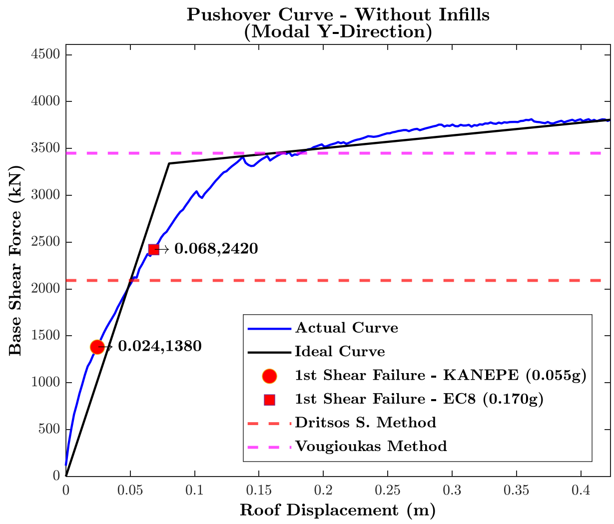


Figure 3.71 Comparison of the shear strength of the building – (Y-Direction)

After examining the building, taking into account all the methods (Greek, European, and American), the following conclusions are drawn:

- The KANEPE code is conservative and leads to many shear failures in walls compared to the other two standards (EC8 and ASCE/SEI 41-17), where all the checks passed in all limit states
- There is no difference between the two models (with/without infills) in all codes as the building has many walls in both directions
- According to Greek legislation, only the KANEPE code is approved. Therefore, although the building does not meet the performance objectives we set, it does not need retrofit because it achieves the minimum seismic class for performance level "Significant Damage", which is $B2^{+}$ taking the EFEHR spectrum
- As for the seismic loss assessment, the PGA value where we have the first member failure in both standards (KANEPE and EC8) is reached in the y-direction, but there are big differences between these values.

More specifically, as mentioned, the problem with KANEPE is that it gives lower element shear strength values compared to the EC8. Table 3.28 shows the results obtained from KANEPE and EC8 for the performance level "Significant Damage" in the x-direction for specific members, where there is a different of about 40 – 45 % showing that the relation of KANEPE (Eq. (1.9)) is conservative.

Table 3.28 Comparison of shear strength between KANEPE and EC8

Member	Story	KANEPE		EC8		Difference (%)
		Demand (kN)	Capacity (kN)	Demand (kN)	Capacity (kN)	
Wall W6	-1	1427.95	1102.14	1307.12	1582.55	43.59
Wall W6	2	1375.30	1074.92	1357.12	1572.39	46.28
Wall W6	3	1233.52	1152.65	1217.18	1620.94	40.63

Another thing that was examined was the performance levels of each code. First of all, KANEPE and EC8 use the same spectrums for the performance levels "Significant Damage" and "Near Collapse", while ASCE 41-17 use spectrums with lower seismic intensity, so there will be big differences. According to Figure 3.72 - Figure 3.73, it can be seen that for the model without infills, the performance levels of EC8 are lower than that of KANEPE in both directions, therefore the seismic demand is lower and there are no failures. Nevertheless, there is no much difference, so the failures are mainly due to the relations of KANEPE, which are conservative. In contrast, when we refer to the model with infills, the performance levels are almost the same in both directions (Figure

3.74 - Figure 3.75). Finally, the performance levels of the ASCE/SEI 41-17 are located very low on the pushover curve which is normal.

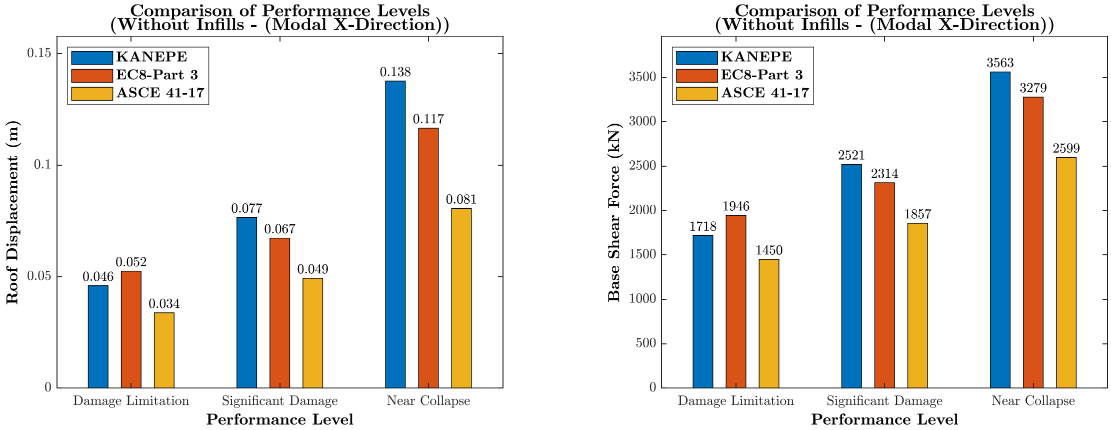


Figure 3.72 Comparison of the performance levels – Without Infills (X-Direction)

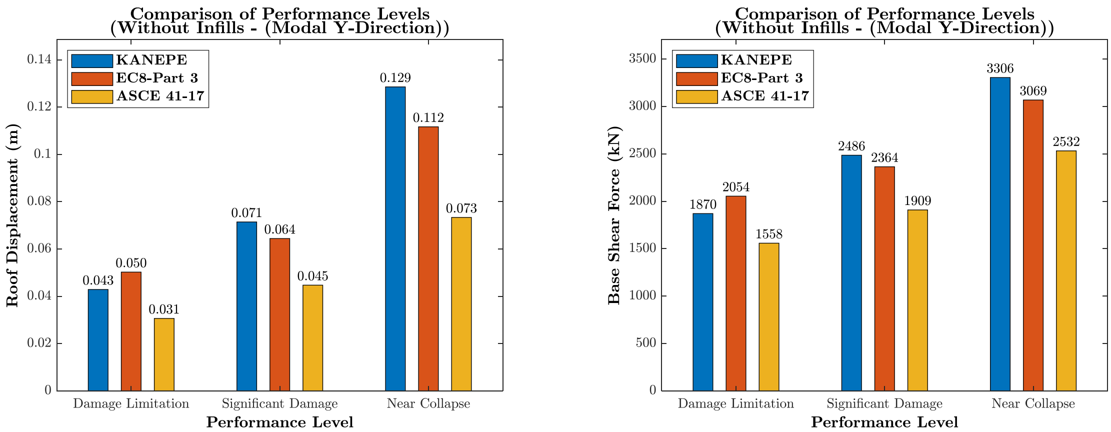


Figure 3.73 Comparison of the performance levels – Without Infills (Y-Direction)

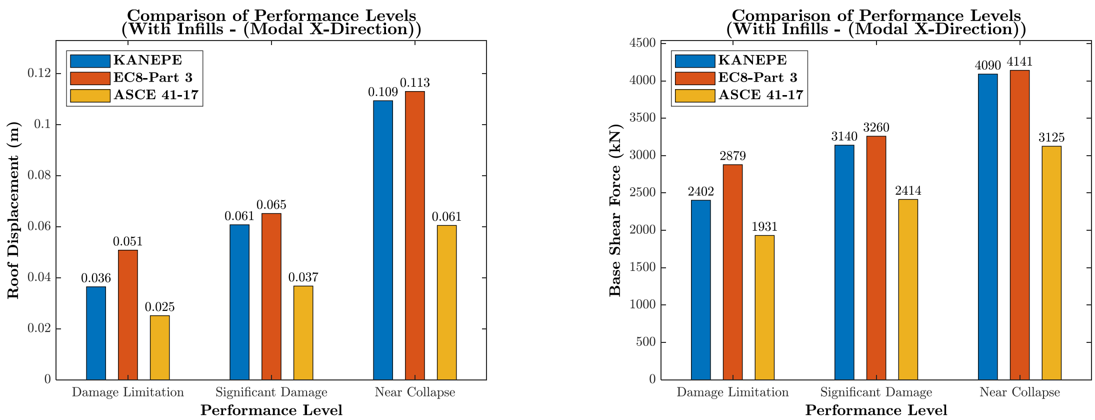


Figure 3.74 Comparison of the performance levels – With Infills (X-Direction)

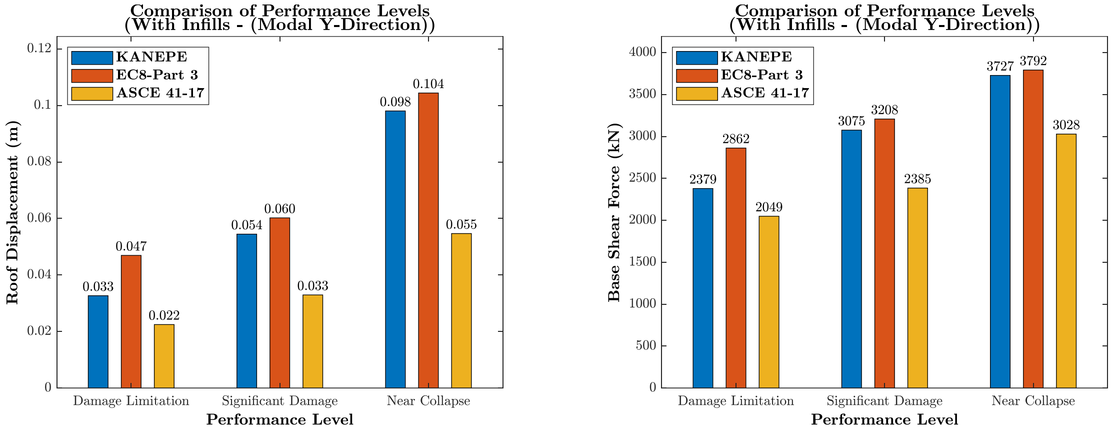


Figure 3.75 Comparison of the performance levels – With Infills (Y-Direction)

Then, we compared the pushover curves in terms of both load distribution and presence or not of infills in both directions. As presented in the figures below, there is a 35 – 40% difference regarding the load distribution in the x-direction and 30 – 33% in the y-direction, where as expected the uniform distribution gives a pushover curve with higher shear capacity. Regarding the infills, we can see that the difference is much smaller, almost 20% in both directions, which means that the structure is stiffer but not in a large scale due to the presence of many walls.

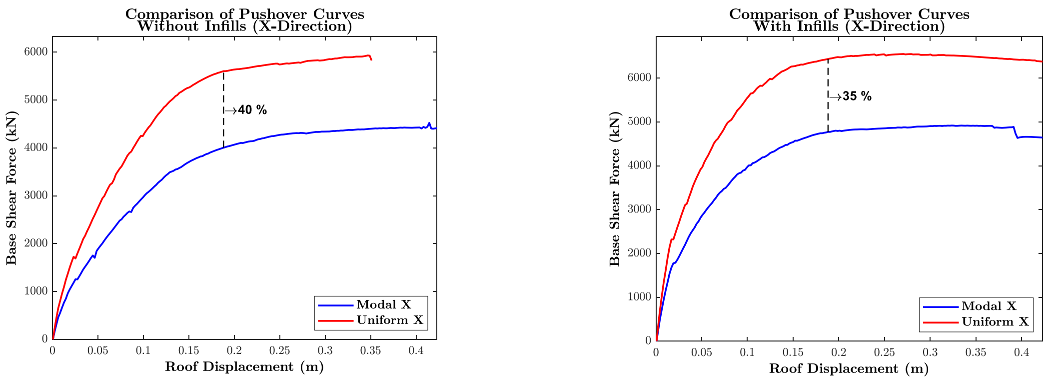


Figure 3.76 Comparison of pushover curves – (Load Distribution) - (X-Direction)
(a) Without Infills, (b) With Infills

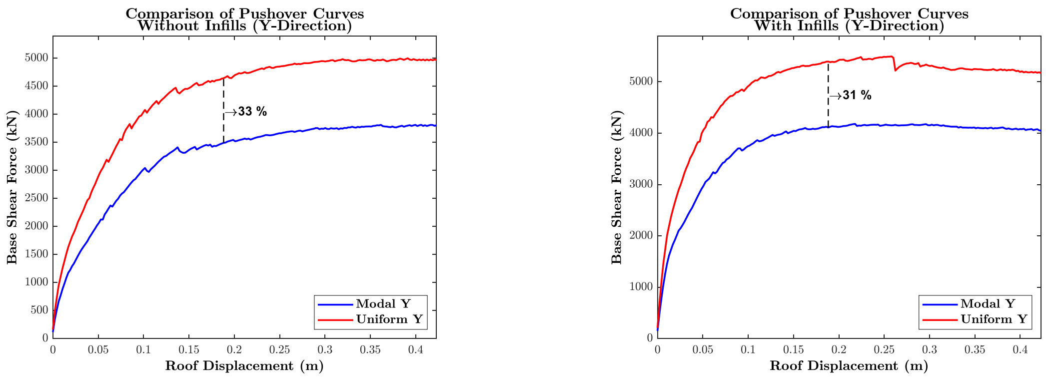


Figure 3.77 Comparison of pushover curves – (Load Distribution) - (Y-Direction)
(a) Without Infills, (b) With Infills

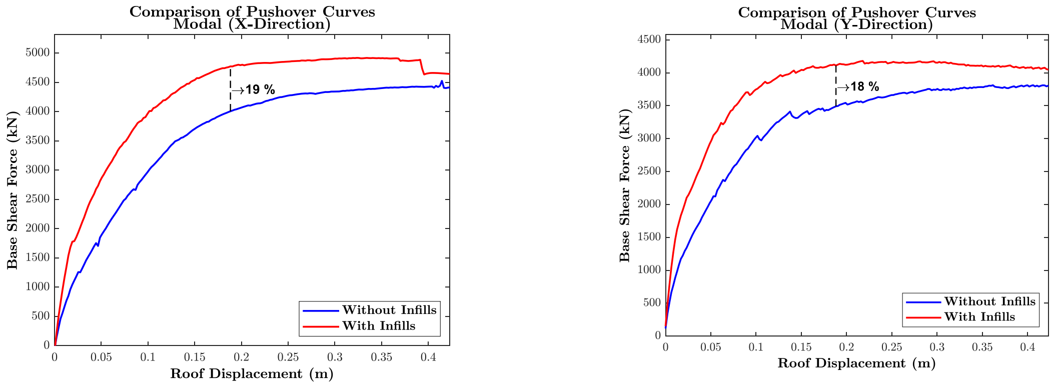


Figure 3.78 Comparison of pushover curves – (Presence of Infills)
(a) X-Direction, (b) Y-Direction

After that, the interstory drift ratio was examined to identify the critical story for all models. Figure 3.79 - Figure 3.80 show that there is a large difference between the performance levels "Damage Limitation" – "Near Collapse", because as we saw in the pushover curves there is a big distance between these points. More specifically, there is 60 – 70% difference between "DL – SD" limit states and 80 – 90% between the "SD – NC" limit states. Moreover, the third story is critical for all models, except for the first (x-direction/without infills), where it is the fourth. In addition, the maximum drift ratio is almost 0.70% for the model without infills and 0.60% for the model with infills, which is normal because the building is much stiffer in the second case. Finally, it can be seen that the distribution in the first two performance levels is smooth compared to the final limit state where there is a large reduction from the third story and above.

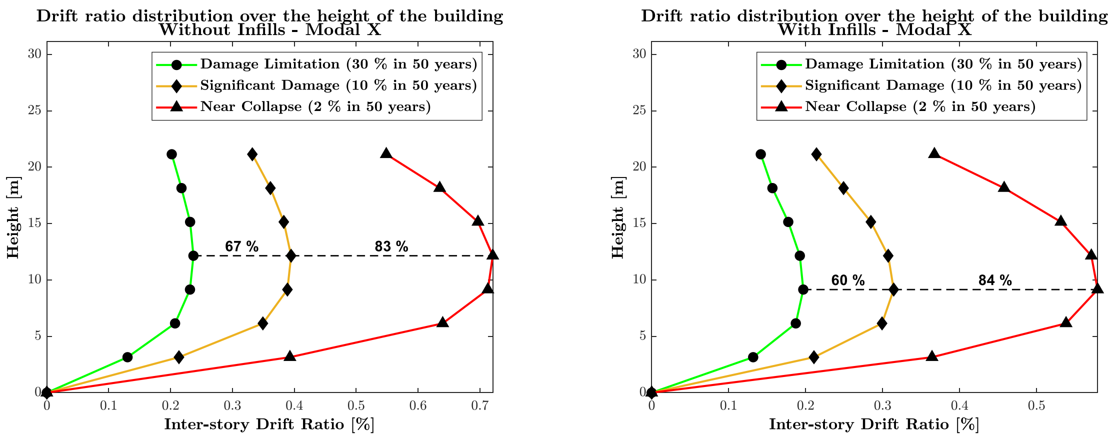


Figure 3.79 Inter-story drift ratio distribution – (X-Direction)
(a) Without Infills, (b) With Infills

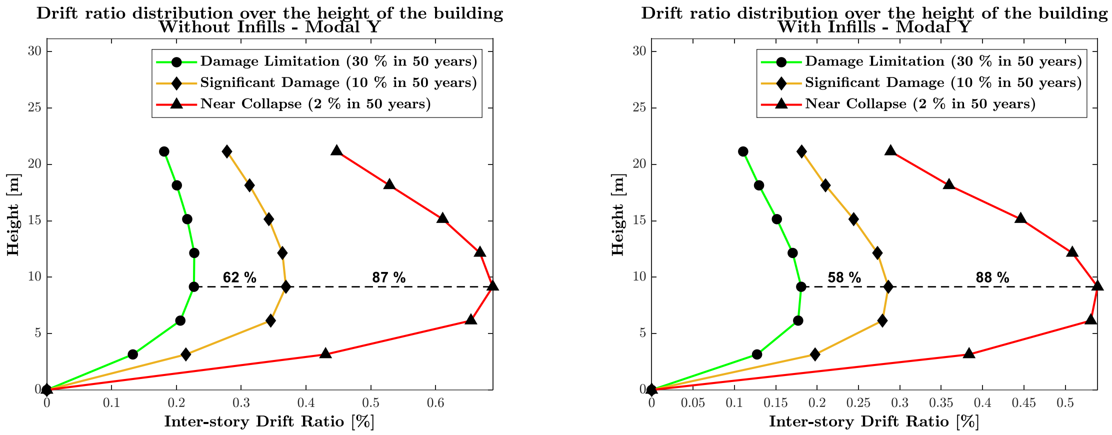


Figure 3.80 Inter-story drift ratio distribution – (Y-Direction)
(a) Without Infills, (b) With Infills

The next step was to compare the inter-story drift ratio with respect to the load distribution as happened with the pushover curves. According to Figure 3.81 - Figure 3.82, we conclude that the uniform distribution gives larger values up to the third - fourth story (maximum drift ratio), but after that point where the results coincide, the values decrease.

Also, a comparison of the inter-story drift was made with regard to the presence of infills. As shown in Figure 3.83 - Figure 3.84, the model with the infills has lower values because it is stiffer. However, it can be seen that the difference increases as we go up. More specifically, in the x-direction the largest difference is found for the performance level “Significant Damage”, while in the y-direction for the performance level “Damage Limitation”. On the third-fourth story, where we have the maximum drift, the percentage difference is about 30% in both directions.

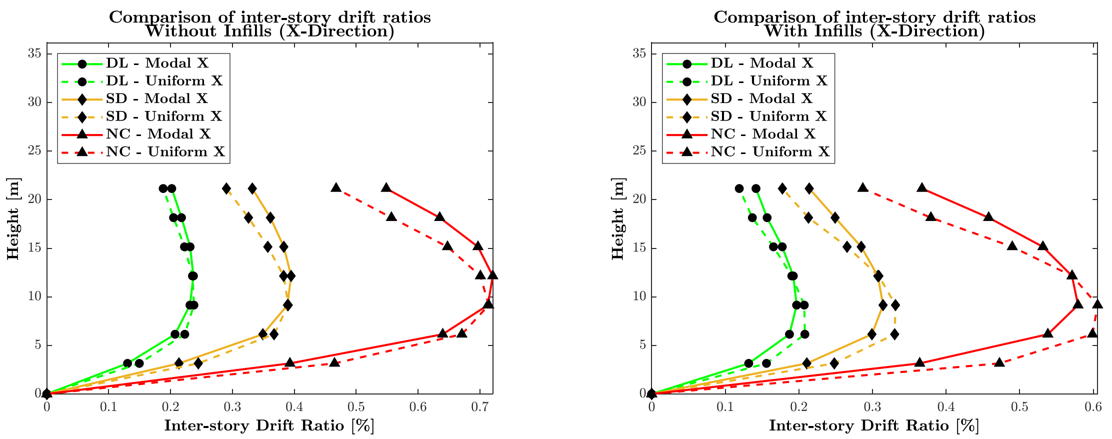


Figure 3.81 Comparison of inter-story drifts – (Load Distribution) - (X-Direction)
(a) Without Infills, (b) With Infills

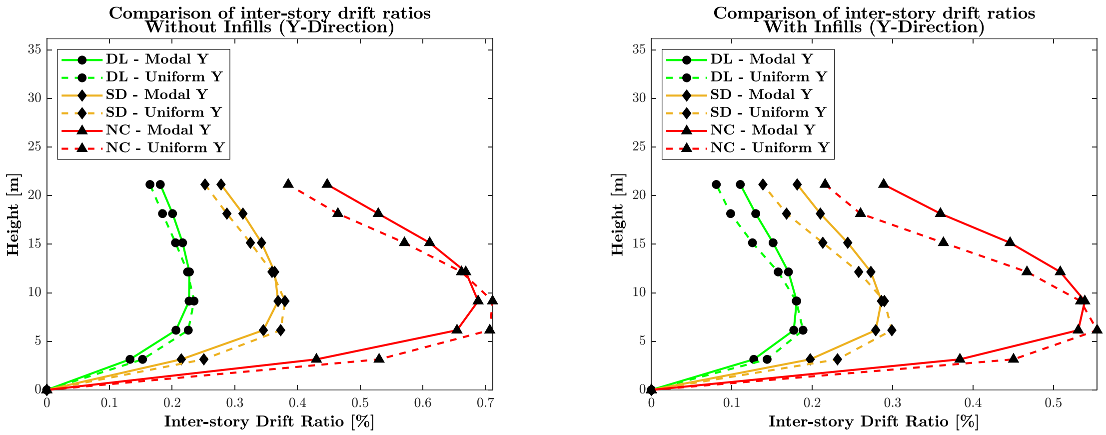


Figure 3.82 Comparison of inter-story drifts – (Load Distribution) - (Y-Direction)
(a) Without Infills, (b) With Infills

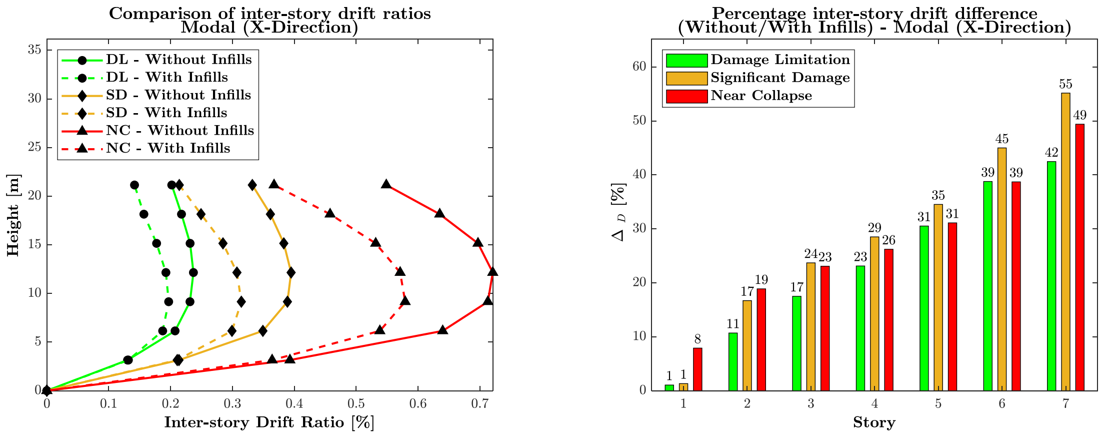


Figure 3.83 Comparison of inter-story drift ratio – (Presence of Infills) - (X-Direction)

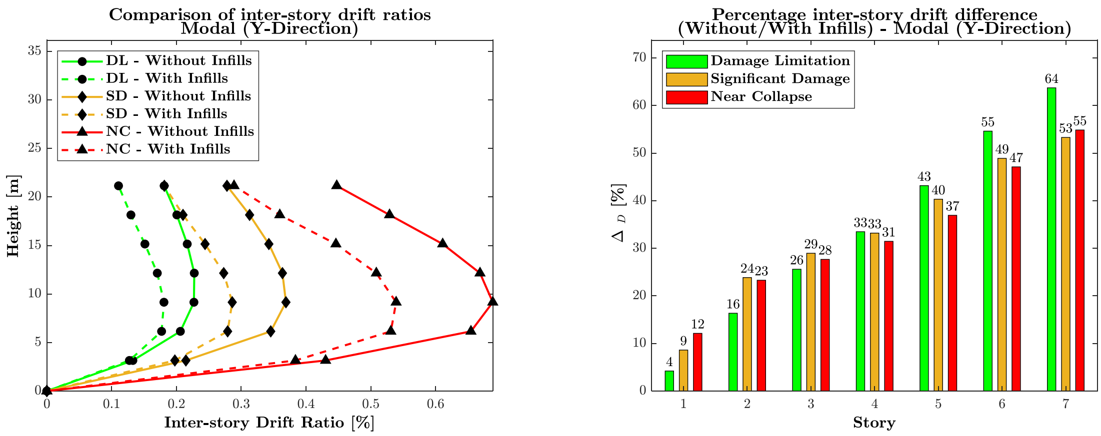


Figure 3.84 Comparison of inter-story drift ratio – (Presence of Infills) - (Y-Direction)

As described in § 3.2.1 (Figure 3.3 - Figure 3.4), the best method for evaluating the stiffness distribution of the structure is the lateral force-deformation method, where results of structural analysis of building subjected to design earthquake loads are used to estimate story stiffness as the ratio of cumulative story shear force to the interstory lateral displacement. So, in order to examine this, we applied this method using only the model without infills for the sake of simplicity, because as we saw before there are not big differences. Figure 3.85 - Figure 3.86 show that the total stiffness at the base of the building is almost the same in both directions. Moreover, the largest stiffness difference is between first-second and sixth-seventh story, where the percentage is almost 40%. This is normal because the first story is like it is fixed due to the rigid basement and on the final story, we have reduced the sections of some elements. If we compare the stiffness difference between the performance levels (Figure 3.87), we conclude that in the first stories the difference is almost constant and it reduces in the final two stories in both directions. In addition, the difference is greater between "SD – NC" performance level than "DL – SD" and it gives larger values in the y-direction.

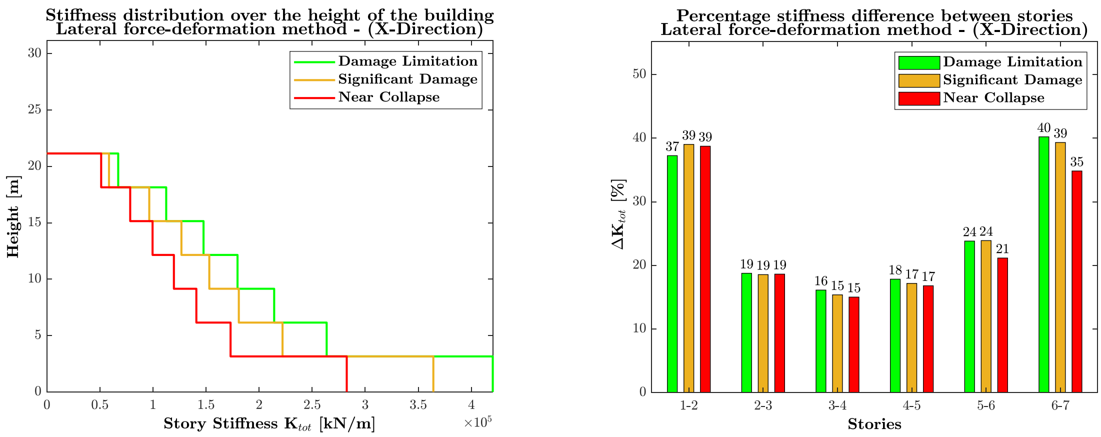


Figure 3.85 Stiffness distribution over the height of the building (Lateral force-deformation method) – (X-Direction)

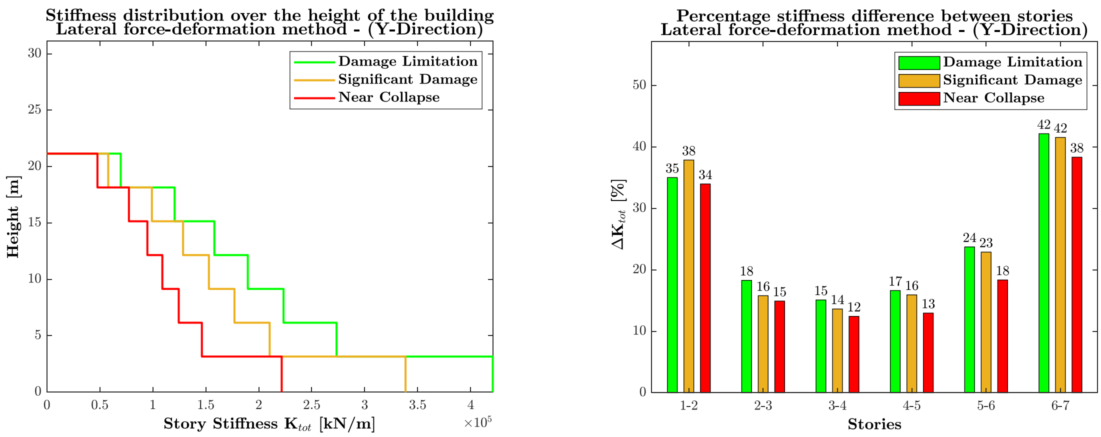


Figure 3.86 Stiffness distribution over the height of the building (Lateral force-deformation method) – (Y-Direction)

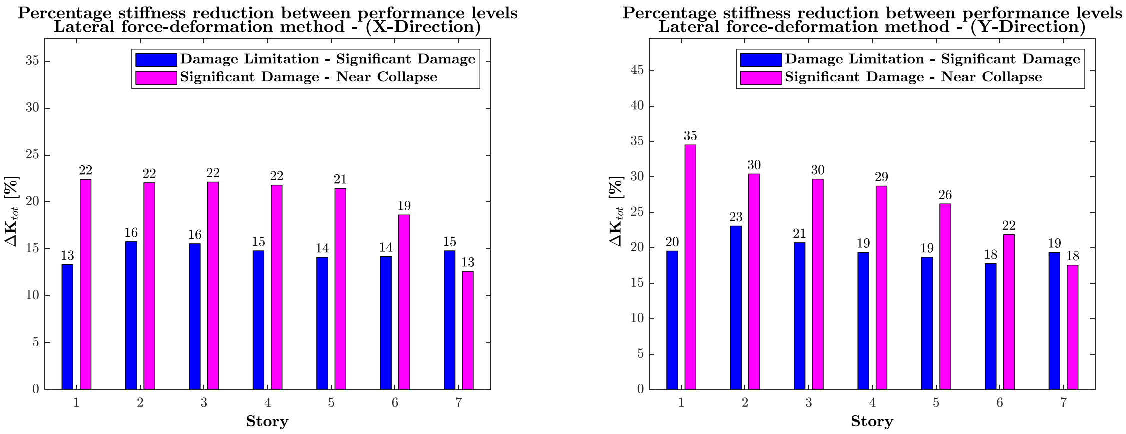


Figure 3.87 Stiffness difference between the performance levels
(a) X-Direction (b) Y-Direction

3.5. Roof Water Tank

As presented in § 1.5, roof water tanks are considered non-structural elements, where in order to examine them, we need to apply a horizontal force according to Eq. (1.93). First of all, a modal analysis performed to find the fundamental vibration period of this structure. Figure 3.88 shows that in the x-direction $T_x = 0.63$ sec and in the y-direction $T_y = 0.60$ sec. Furthermore, we need the first mode eigenperiod of the building in each direction, which in this case the uncracked model without infills is used because the water tank is expected to have damages at the beginning of an earthquake, where cracking has not started. Therefore, we have $T_{1x} = 0.53$ sec and $T_{1y} = 0.51$ sec (Figure 3.8, Figure 3.9). According to the design code, PGA = 0.16g for this region and W = 20.89 kN. So, the seismic coefficient S_a and the total force F_a are equal to:

Direction X

$$\boxed{S_{a,x} = 1.016g \rightarrow F_{a,x} = 10.61 \text{ kN}}$$

Direction Y

$$\boxed{S_{a,y} = 1.021g \rightarrow F_{a,y} = 10.66 \text{ kN}}$$

Figure 3.88 Modal analysis of the roof water tank – (a) $T_x = 0.63$ sec, $T_y = 0.60$ sec

This structure consists of two moment-restrained frames in both directions, so the buckling lengths of the columns must be defined, taking into account that there is pin support. The final results are:

$$\boxed{L_x = 3.090L,\ L_y = 2.875L\ (1^{st}\ \text{story}),\ L_x = 1.790L,\ L_y = 1.754L\ (2^{nd}\ \text{story})}$$

We performed nonlinear analysis using P-Delta effects to be more accurate. The total combinations used are the following:

- $G \pm F_{a,x} \pm 0.3F_{a,y}$
- $G \pm F_{a,y} \pm 0.3F_{a,x}$

Figure 3.89 shows the final results came from the envelope of these combinations, where the maximum performance ratio is 3.359 for the ground floor columns. This is due to the bi-axial bending of the square section SHS 40x40x3, which means that the structure is going to collapse in the design earthquake (10% poe in 50 years).

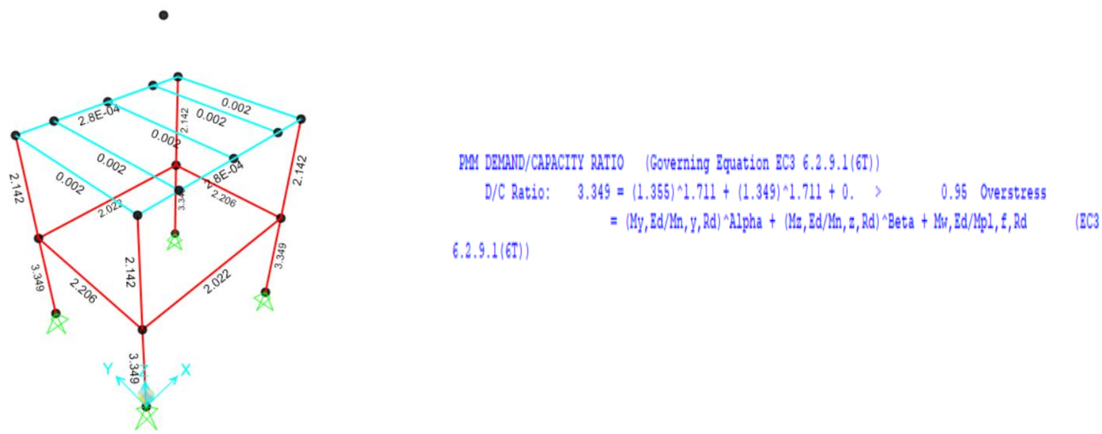


Figure 3.89 Final results for the roof water tank – PGA = 0.16g

The final step was to determine the PGA value where structure is safe (first failure), so as to perform in the final chapter the seismic loss assessment. After some tests, it was found that the maximum PGA value is 0.067g (40% of the design PGA) corresponding to an earthquake with 77% poe in 50 years (T_R = 35 years) (Figure 3.90), which is quite frequent event, meaning that the structure has a high probability of collapse during the lifetime of the building. The final value of the roof top acceleration is S_a = 0.430g and the total force is $F_a \cong 4.45$ kN. The results show that these structures must be designed as usual steel structures according to the standards and not using typical sections of the company. The amplified roof top acceleration results in high horizontal forces. As mentioned earlier, damage of these structures will cause the water run all over the roof, causing serious problems. So, we understand that these structures must be designed very carefully by structural engineers.

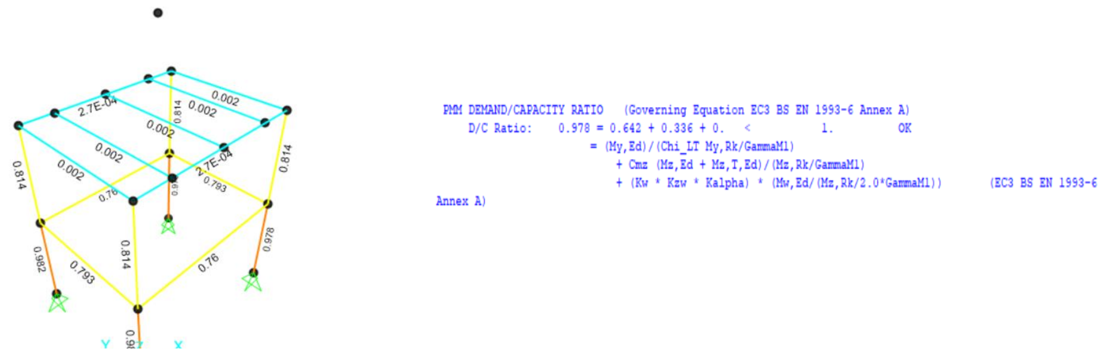


Figure 3.90 Final results for the roof water tank – PGA = 0.067g

Chapter 4

Seismic Loss Assessment

4.1. Fragility Curves – SPO2FRAG Software

Using nonlinear static analysis, where the capacity curve of the structure is estimated, the maximum ground acceleration at which the first failure of a vertical member occurs in each main direction was calculated. However, earthquakes are a probabilistic phenomenon and the calculation of fragility curves is a useful approach for additional information related to exceeding a building's performance level. Therefore, we want to transform the nonlinear static analysis (deterministic method) into a truly dynamic analysis. For this purpose, the software SPO2FRAG[9] was used, which calculates the fragility curves for different performance levels from the results of the nonlinear static analysis (pushover analysis). This calculation is achieved by considering an equivalent SDOF system and through the SPO2IDA[31] algorithm the curves of an incremental dynamic analysis (IDA) are generated, representing the spectral acceleration values ($S_a(T_1)$) as a function of the roof drift ratio (θ_{roof}) or the maximum inter-story drift ratio (θ_{max}) with a probability of exceeding 16%, 50% and 84% (Figure 4.1). Finally, these curves are converted for the MDOF system and the fragility curves are calculated for the defined performance levels.

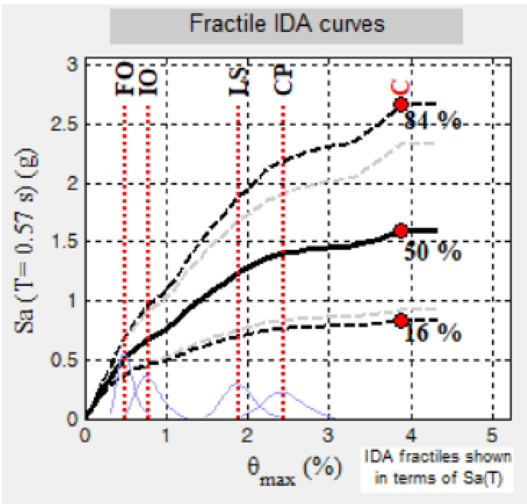


Figure 4.1 Indicative IDA curve estimated from SPO2FRAG

In this work, we used this software to determine the collapse fragilities and input them into the PACT software to estimate the seismic losses. So, we have identified six (6) fragility curves for the performance level "Near Collapse (Γ1)". More specifically:

- 1st shear failure of the wall in the x-direction according to KANEPE
- 1st shear failure of the wall in the x-direction according to EC8
- Shear failure of the infills in the x-direction
- 1st shear failure of the wall in the y-direction according to KANEPE
- 1st shear failure of the wall in the y-direction according to EC8
- Shear failure of the infills in the y-direction

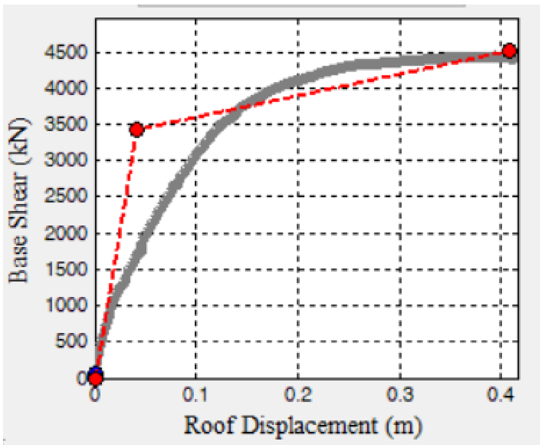
The presence of the infills, as we have seen before, provides great stiffness. We assume that the complete failure of the infills occurs at inter-story drift ratio 0.6%. Finally, the dispersion used was 30% for all cases.

In § 3.4 (Figure 3.70 - Figure 3.71), the ground acceleration of the first shear failure in a vertical member was calculated according to KANEPE and EC8. Table 4.1 presents the values of the roof drift ratio (θ_{roof}) and the maximum inter-story drift ratio (θ_{max}) for these PGA values for both standards (KANEPE and EC8).

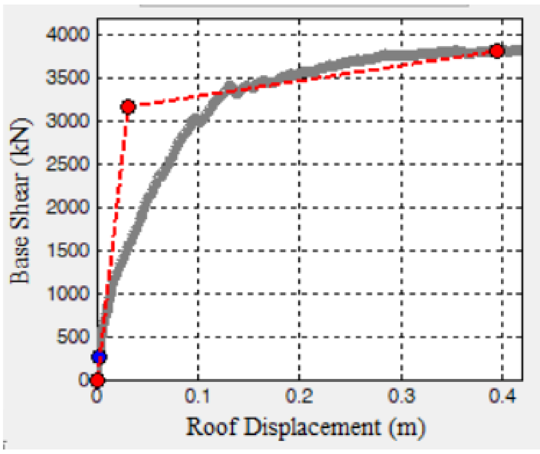
Table 4.1 Roof drift ratio (θ_{roof}) and maximum inter-story drift ratio (θ_{max}) according to KANEPE and EC8

Code/ Member	**Direction**	$\boldsymbol{\theta_{roof}}$ **(%)**	$\boldsymbol{\theta_{max}}$ **(%)**
Wall Shear Failure - KANEPE	X	0.159	0.179
Wall Shear Failure - KANEPE	Y	0.116	0.134
Wall Shear Failure – EC8	X	0.450	0.512
Wall Shear Failure – EC8	Y	0.323	0.372
Infills Shear Failure	X	-	0.600
Infills Shear Failure	Y	-	0.600

Before presenting the results, we will introduce the SPO2FRAG software. First, the capacity curve of the structure was entered in terms of base shear and roof displacement, as well as the displacements of each floor obtained by the Seismobuild software for each direction. Then, the idealized curve is calculated according to the software. There are many methods but, in this case, the bilinear curve was used. Figure 4.2 - Figure 4.3 show the results in both directions.

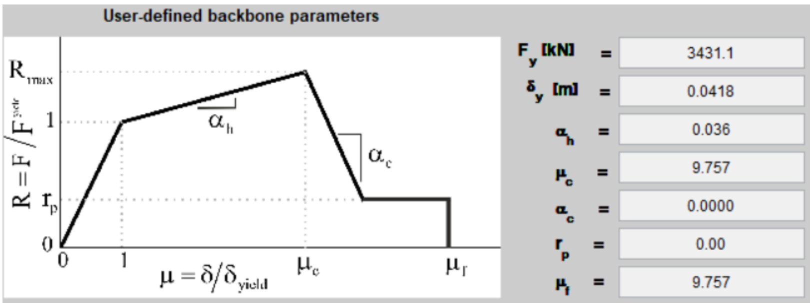


(a)

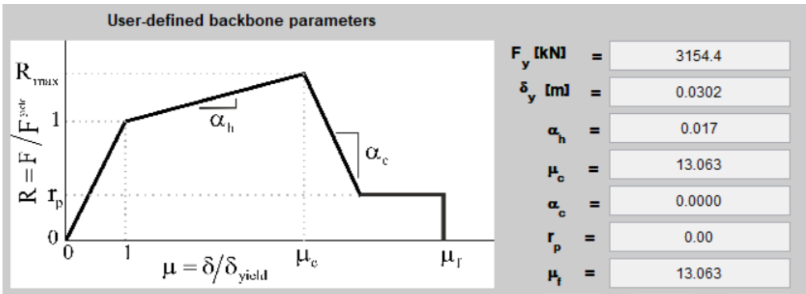


(b)

Figure 4.2 Idealized curves calculated with the SPO2FRAG software
(a) X-Direction, (b) Y-Direction



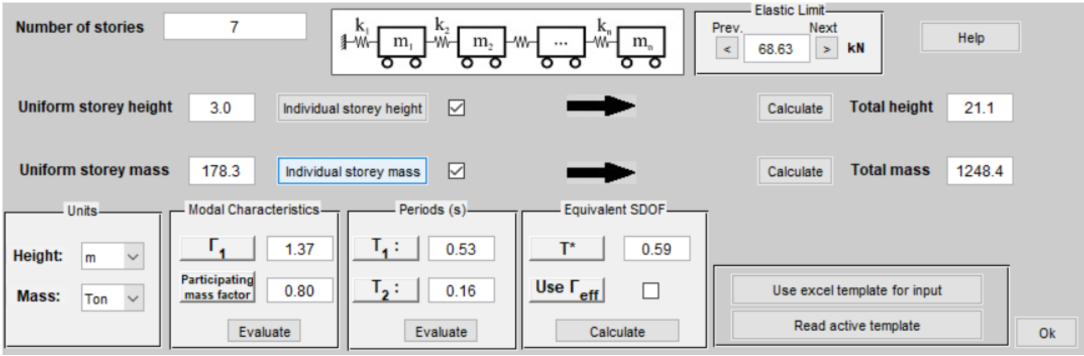
(a)



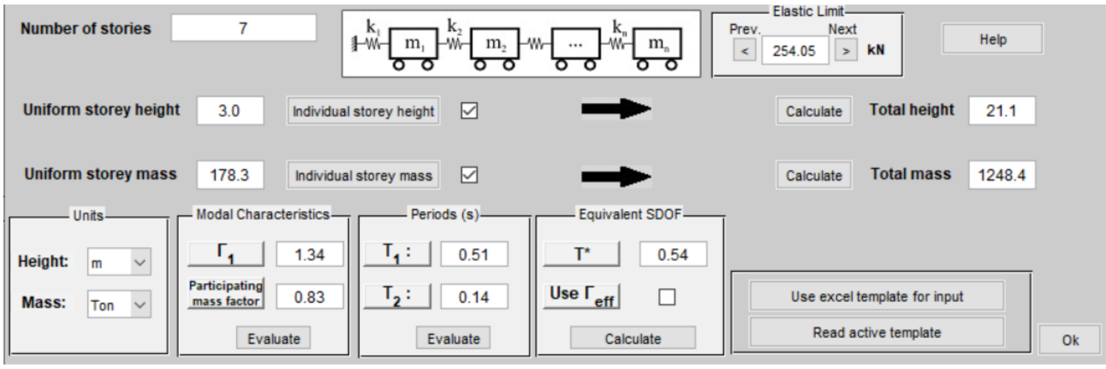
(b)

Figure 4.3 Backbone parameters defined from the SPO2FRAG software
(a) X-Direction, (b) Y-Direction

The dynamic characteristics of the structure were then integrated (Figure 4.4). The number of stories is 7, the height of each is 3.00 m, the total mass is 1248.45 tn and finally the total height of the building is 21.15 m. In this window, after the user enters the fundamental eigenperiod of the building in each direction, the software calculates the participation factor Γ1 and the period of the SDOF system.



(a)



(b)

Figure 4.4 Dynamic characteristics of the building (a) X-Direction, (b) Y-Direction

Afterwards, through the SPO2IDA algorithm the (IDA) curves were generated using as engineering demand parameter (EDP) the maximum inter-story drift ratio (θ_{max}). The demand parameter was set as the performance level "Collapse Prevention" using as θ_{max} the values obtained from Table 4.1 for all six (6) cases.

Table 4.2 shows the values of the mean spectral acceleration and their standard deviation, as calculated by the SPO2FRAG software. Figure 4.5 - Figure 4.6 show the fragility curves calculated by the SPO2FRAG for the performance level "Collapse Prevention". As shown in the figures, the spectral acceleration according to KANEPE has a much lower value compared to EC8 in both directions, which is normal. The EC8 hazard spectrum was used to calculate the PGA values corresponding to $S_a(T_1)$ for soil factor S = 1.20. It should be noted that $T_{1x} = 0.53$ sec and $T_{1y} = 0.51$ sec. Therefore, the y-direction is critical for both standards (KANEPE and EC8) and tloss assessment, since the wall shear failure is the crucial parameter.

Table 4.2 Mean spectral acceleration and standard deviation calculated with the SPO2FRAG software

Code/ Member	Direction	$S_a(T_1)$ (g)	Standard Deviation	PGA (g)
Wall Shear Failure - KANEPE	X	0.281	0.294	0.099
Wall Shear Failure - KANEPE	Y	0.255	0.295	0.087
Wall Shear Failure – EC8	X	0.761	0.335	0.269
Wall Shear Failure – EC8	Y	0.660	0.324	0.224
Infills Shear Failure	X	0.871	0.345	0.308
Infills Shear Failure	Y	0.969	0.352	0.329

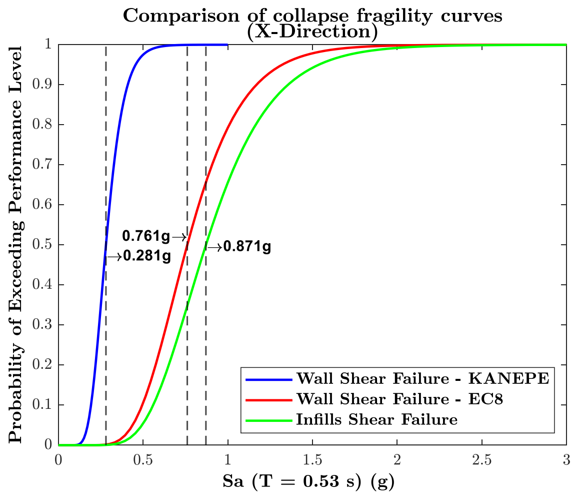


Figure 4.5 Collapse fragilities calculated with SPO2FRAG – (X-Direction)

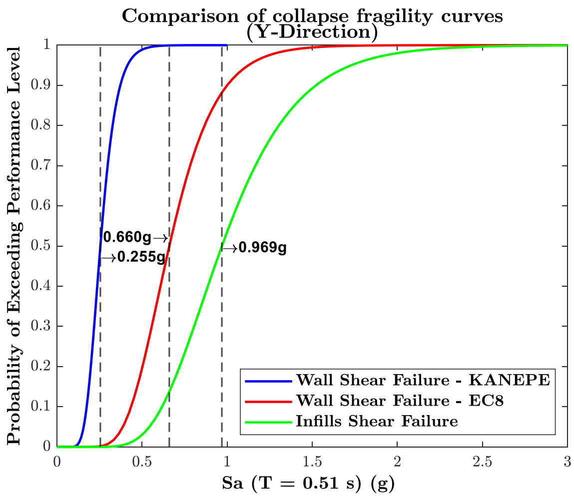


Figure 4.6 Collapse fragilities calculated with SPO2FRAG – (Y-Direction)

4.2. Estimation of Seismic Losses

Completing the analysis, the results from SPO2FRAG were used to estimate the financial losses that are likely to occur in the structure due to the seismic action. This analysis is carried out using the Performance Assessment Calculation Tool (PACT) software developed by FEMA P-58[11].

For each case, the basic characteristics of the building were entered into the software, such as the number of stories, total height in feet (ft) and total area in square feet (ft^2) (Figure 4.7). Moreover, the core and sell replacement cost was entered for the entire structure, which according to modern Greek data (before 2021) is estimated at $1100/m^2 multiplied by the total area of the building's floors gives a value of $996855. It should be noted that the cost for the basement was calculated with a coefficient 0.5, because this story will not show any damage in the future. After that, the total replacement cost was entered which is practically the core and sell replacement cost multiplied by 1.40 to account for the cost of the contents. This gives a total value of $1395597. The replacement time was estimated at 8 months or 240 days, while the total threshold beyond which the restoration is unprofitable was estimated at 60% of the total replacement cost. The maximum number of workers was set to be 1 worker/1000 ft^2.

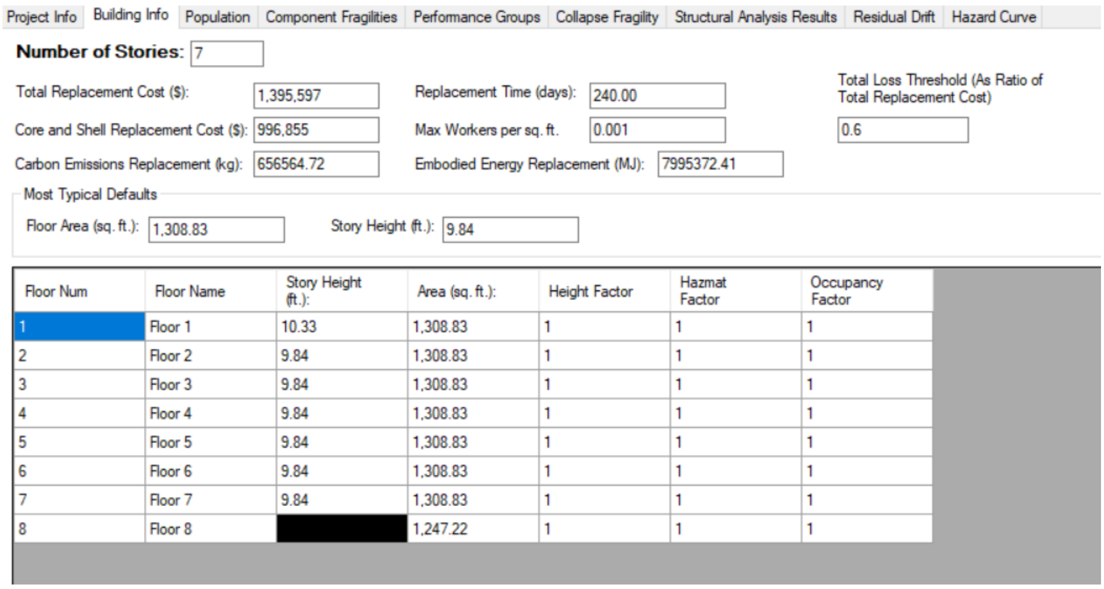


Figure 4.7 Data entered on PACT software for the calculation of seismic losses

The building consists of reinforced concrete elements, which are imported from the program's "libraries" that include for each level of damage of a structural element the fragility curve and the corresponding repair cost. As this is a US program, all the above libraries were modified in terms of repair cost to better match with the Greek data. The repair cost for each element was obtained from research carried out in the thesis of Vallianatou Dimitra[32]. Table 4.3 summarizes the results.

Table 4.3 Repair cost in each damage state for the structural elements of the building

Member	**Damage State**	**Repair Cost ($/m^2)**
Column	DS1	373
	DS2	275
	DS3	320
Beam	DS1	373
	DS2	200
	DS3	320
Wall	DS2	275
	DS3	320
Internal Masonry Infill	DS1	92
	DS2	156
	DS3	170
External Masonry Infill	DS1	110
	DS2	174
	DS3	217

It should be noted that the library of the walls in this software has only two damage states. Based on the values given in Table 4.3, the costs for a lower and a higher quantity of each category of the elements were calculated. The cost for the lowest quantity of each building element was set equal to the cost values in Table 4.3, while for the highest quantity a 20% discount was set in relation to the previous value. This software does not contain columns and beams separately, but joints. So, for this category, the beams were assumed to fail at a length equal to twice their static height, while the columns were assumed to fail at half the story height. This assumption for the columns was made because they are elements that are involved in both main directions and are therefore counted twice, so that the final repair cost is not taken for a length greater than the story height. Regarding the walls and infills, the damage was considered to occur in the form of diagonal cracks along their height and length. In order to estimate the seismic losses, the quantities of the structural and non-structural elements of the structure must be entered for both directions. The quantities of the structural and non-structural elements of the structure for both directions include the beam-column joints, walls, infills, hot and cold water pipes, electrical wiring, the elevator and finally the roof water tank. The library of the non-structural elements has not changed, except for the roof water tank, where we entered it into the software manually, because there was nothing like this.

More specifically, beam-column joints are measured in individual units, i.e., "each", while infills and walls are measured in units of HxH panels. For instance, for a building with a 36 foot long wall and 12 foot story height, the number of panels input into PACT would be three (36ft/12ft) in that direction. Table 4.4 shows the categories of structural elements used and their fragility data.

Table 4.4 Fragility curves of the structural elements used in the PACT software

Element ID	**Drift Ratio DS1**	**Dispersion DS1**	**Drift Ratio DS2**	**Dispersion DS2**	**Drift Ratio DS3**	**Dispersion DS3**
B1041.001c (35x35 – One)	0.02	0.40	0.0275	0.3	0.05	0.30
B1041.001d (35x35 – Both)	0.02	0.40	0.0275	0.3	0.05	0.30
B1041.001e (35x40 – One)	0.02	0.40	0.0275	0.3	0.05	0.30
B1041.001f (35x40 – Both)	0.02	0.40	0.0275	0.3	0.05	0.30
B1041.001g (35x50 - One)	0.02	0.40	0.0275	0.3	0.05	0.30
B1041.001j (35x60 – Both)	0.02	0.40	0.0275	0.3	0.05	0.30
B1041.001k (140x35 – One)	0.02	0.40	0.0275	0.3	0.05	0.30
B1041.001l (140x35 – Both)	0.02	0.40	0.0275	0.3	0.05	0.30
B1041.001m (105x30 – One)	0.02	0.40	0.0275	0.3	0.05	0.30
B1041.001n (105x30 – Both)	0.02	0.40	0.0275	0.3	0.05	0.30
B1041.001o (100x30 – One)	0.02	0.40	0.0275	0.3	0.05	0.30
B1044.071b	-	-	0.0033	0.35	0.0087	0.20
B1051.001a	0.0016	0.20	0.0025	0.25	0.0050	0.30
B1051.001b	0.0016	0.20	0.0025	0.25	0.0050	0.30

As we can see from the table, eleven (11) elements were used for the beam-column joints because we have different cross-sectional areas, so the repair cost is not the same. In addition, there is one (1) element for the walls (B1044.071b) and two (2) elements for the infills (B1051.001a → External Infills, B1051.001b → Internal Infills). The next step is the calculation of the upper and lower repair cost of these elements. For the beam-column joints, we considered both the columns and the beams at a specified length as described in the previous paragraph. For the walls, the repair cost is expressed for each 144 ft^2 panel, while for the infills is expressed for each 100 ft^2 panel. So, the values from Table 4.3 must be adjusted according to this assumption.

Table 4.5 Repair cost of the structural elements entered into the PACT software

Element ID	**DS1 (Low) [$]**	**DS2 (Low) [$]**	**DS3 (Low) [$]**	**DS1 Up [$]**	**DS2 Up [$]**	**DS3 Up [$]**
B1041.001c	1342.80	877.50	1152.00	1074.24	702.00	921.60
B1041.001d	1902.30	1177.50	1632.00	1521.84	942.00	1305.60
B1041.001e	1398.75	918.75	1200.00	1119.00	735.00	960.00
B1041.001f	1958.25	1218.75	1680.00	1566.60	975.00	1344.00
B1041.001g	1510.65	1001.25	1296.00	1208.52	801.00	1036.80
B1041.001j	2182.05	1383.75	1872.00	1745.64	1107.00	1497.60
B1041.001k	2517.75	1743.75	2160.00	2014.20	1395.00	1728.00
B1041.001l	3077.25	2043.75	2640.00	2461.80	1635.00	2112.00
B1041.001m	2070.15	1413.75	1776.00	1656.12	1131.00	1420.80
B1041.001n	2629.65	1713.75	2256.00	2103.72	1371.00	1804.80
B1041.001o	2014.20	1372.50	1728.00	1611.36	1098.00	1382.40
B1044.071b	-	3678.96	4280.97	-	2943.17	3424.78
B1051.001a	1021.93	1616.51	2016.00	817.55	1293.21	1612.80
B1051.001b	854.71	1449.29	1579.35	683.77	1159.43	1263.48

The roof water tank is entered manually into the software. The engineering demand parameter is the floor acceleration. As we have seen in Figure 3.90, the roof acceleration where the steel structure is safe is 0.430g. So, we used this value with a 40% dispersion. The repair cost used is $1000 per unit. Furthermore, we know the length of the exterior infills in each direction, but we do not have this information for the interior infills. For the calculation of the surface area of the internal masonry, it was assumed that there is 1.7 m^2 of interior infills per square meter of floor, evenly distributed in each direction. Finally, the area of the exterior infills was increased 30% to take into account the glazing and other things.

The calculation of the quantities of the non-structural elements of the structure was performed according to the Excel Normative Quantity Estimation Tool of FEMA P-58. Table 4.6 shows the quantities of the construction elements entered into the program based on the units of each category.

Table 4.6 Quantities of the structural and non-structural elements entered into the PACT software

Element ID	**X-Direction**			**Y-Direction**		
Story	**1st**	**2nd – 6th**	**7th**	**1st**	**2nd – 6th**	**7th**
B1041.001c	0	0	1	0	0	0
B1041.001d	1	1	1	1	1	2
B1041.001e	2	2	2	2	2	2
B1041.001f	1	1	1	1	1	1
B1041.001g	1	1	1	1	1	1
B1041.001j	1	1	1	0	0	0
B1041.001k	1	1	0	0	0	0
B1041.001l	0	0	0	1	1	0
B1041.001m	1	1	1	0	0	0
B1041.001n	0	0	0	1	1	1
B1041.001o	0	0	1	0	0	1
B1044.071b	2.08	2.18	1.52	2.43	2.55	2.55
B1051.001a	-	8.87	8.87	-	7.61	7.67
B1051.001b	-	11.48	10.94	-	11.48	10.94
D1014.011 (Elevator)	0.31	-	-	0.31	-	-
D2011.011a (Hot or Cold Potable)	0.14	0.14	0.14	0.14	0.14	0.14
D5012.031a (Distribution Panel)	1	1	1	1	1	1
F1045.001 (Roof water tank)	-	-	3	-	-	3

The non-structural elements (D1041.011 → F1045.001) are non-directional. In order to assess the damage of the building elements, the floor acceleration and the inter-story drift ratio must first be calculated. The inter-story drift ratio is estimated using the Seismobuild software after the corresponding PGA value is given. The floor acceleration is calculated via FEMA P-58[10] (§ 5.3.2 – Step 4).

The next step is to identify the hazard curve of the region using EFEHR[8]. This curve is obtained for the $S_a(T_{mean})$, where $T_{mean} = 0.5·(T_{1x} + T_{1y}) = 0.52$ sec. It should be noted that an appropriate adjustment of the data was made to the soil type, as the data obtained from the EFEHR refer to Class A soil, whereas the soil in our case is B. The generated hazard curve is shown in Figure 4.8.

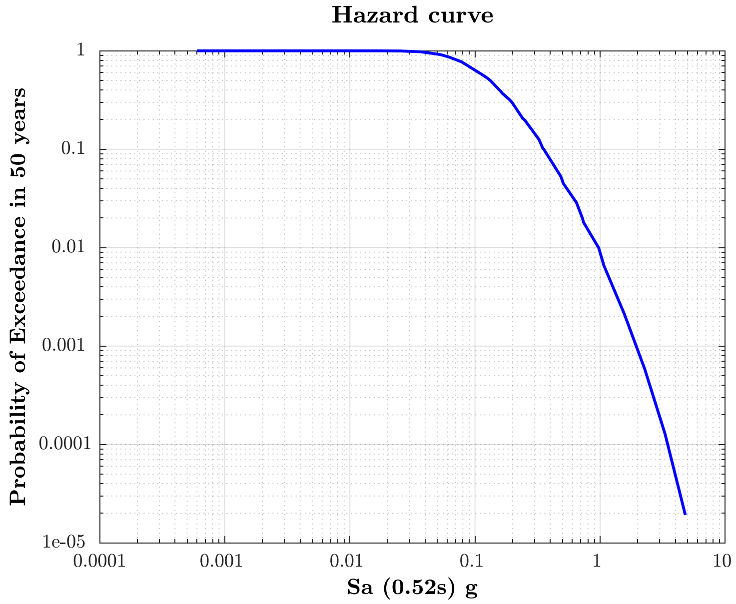


Figure 4.8 Hazard curve of the building for $T_{mean} = 0.52$ sec and soil class B

According to Table 4.2, the y-direction is the critical. For the estimation of the seismic losses, both codes (KANEPE and EC8) were used in order to compare the results. First of all, we adjusted the $S_a(T_1)$ to the $S_a(T_{mean})$. Six (6) intensity scenarios were used, including a 50%, 10% and 2% probability of exceedance at 50 years, as well as the first wall failure and two other values to reduce the gap. Table 4.7 and Table 4.8 show the final intensities with the spectral acceleration and the corresponding mean annual frequency. In addition, Table 4.9 - Table 4.12 show the inter-story drifts calculated with the Seismobuild software for these intensities for both standards.

Table 4.7 Intesity scenarios for the estimation of seismic losses - KANEPE

Scenario	$S_a(T_{mean})$ (g)	MAFE
Intensity 1 (86% in 50 years - 25% Sa(T))	0.063	0.0395
Intensity 2 (50% in 50 years - 53% Sa(T))	0.133	0.0139
Intensity 3 (32% in 50 years - 75% Sa(T))	0.188	0.0078
Intensity 4 (20% in 50 years - 100% Sa(T))	**0.250**	**0.0044**
Intensity 5 (10% in 50 years - 142% Sa(T))	0.354	0.0021
Intensity 6 (2% in 50 years - 288% Sa(T))	0.721	0.0004

Table 4.8 Intesity scenarios for the estimation of seismic losses – EC8

Scenario	$S_a(T_{mean})$ (g)	MAFE
Intensity 1 (50% in 50 years - 20% Sa(T))	0.133	0.0139
Intensity 2 (30% in 50 years - 30% Sa(T))	0.197	0.0071
Intensity 3 (10% in 50 years - 55% Sa(T))	0.354	0.0021
Intensity 4 (5% in 50 years - 75% Sa(T))	0.485	0.0011
Intensity 5 (3% in 50 years - 100% Sa(T))	**0.647**	**0.0006**
Intensity 6 (2% in 50 years - 111% Sa(T))	0.721	0.0004

Table 4.9 Inter-story drift ratios for the examined intensities – KANEPE (X-Direction)

Story	Intensity 1 (86%poe)	Intensity 2 (50%poe)	Intensity 3 (32%poe)	Intensity 4 (20%poe)	Intensity 5 (10%poe)	Intensity 6 (2%poe)
7	0.00051	0.00108	0.00146	0.00190	0.00264	0.00500
6	0.00055	0.00114	0.00155	0.00205	0.00287	0.00582
5	0.00058	0.00120	0.00163	0.00218	0.00306	0.00628
4	0.00059	0.00120	0.00166	0.00223	0.00315	0.00649
3	0.00055	0.00113	0.00161	0.00217	0.00309	0.00642
2	0.00045	0.00094	0.00143	0.00194	0.00277	0.00575
1	0.00025	0.00054	0.00089	0.00122	0.00171	0.00348

Table 4.10 Inter-story drift ratios for the examined intensities – KANEPE (Y-Direction)

Story	Intensity 1 (86%poe)	Intensity 2 (50%poe)	Intensity 3 (32%poe)	Intensity 4 (20%poe)	Intensity 5 (10%poe)	Intensity 6 (2%poe)
7	0.00041	0.00089	0.00124	0.00166	0.00230	0.00421
6	0.00047	0.00099	0.00138	0.00184	0.00258	0.00492
5	0.00053	0.00107	0.00150	0.00199	0.00280	0.00563
4	0.00056	0.00112	0.00157	0.00209	0.00295	0.00610
3	0.00055	0.00111	0.00156	0.00209	0.00297	0.00625
2	0.00047	0.00098	0.00140	0.00189	0.00274	0.00591
1	0.00025	0.00060	0.00089	0.00122	0.00172	0.00378

Table 4.11 Inter-story drift ratios for the examined intensities – EC8 (X-Direction)

Story	Intensity 1 (50%poe)	Intensity 2 (30%poe)	Intensity 3 (10%poe)	Intensity 4 (5%poe)	Intensity 5 (3%poe)	Intensity 6 (2%poe)
7	0.00094	0.00134	0.00234	0.00316	0.00406	0.00443
6	0.00100	0.00142	0.00252	0.00344	0.00457	0.00513
5	0.00105	0.00150	0.00268	0.00365	0.00490	0.00554
4	0.00106	0.00153	0.00275	0.00376	0.00505	0.00569
3	0.00099	0.00147	0.00269	0.00370	0.00500	0.00563
2	0.00082	0.00130	0.00242	0.00332	0.00447	0.00503
1	0.00047	0.00080	0.00152	0.00203	0.00264	0.00301

Table 4.12 Inter-story drift ratios for the examined intensities – EC8 (Y-Direction)

Story	Intensity 1 (50%poe)	Intensity 2 (30%poe)	Intensity 3 (10%poe)	Intensity 4 (5%poe)	Intensity 5 (3%poe)	Intensity 6 (2%poe)
7	0.00080	0.00117	0.00209	0.00276	0.00356	0.00387
6	0.00089	0.00131	0.00234	0.00311	0.00410	0.00450
5	0.00097	0.00141	0.00252	0.00341	0.00452	0.00505
4	0.00102	0.00148	0.00265	0.00362	0.00483	0.00542
3	0.00100	0.00147	0.00267	0.00367	0.00494	0.00554
2	0.00088	0.00133	0.00245	0.00344	0.00464	0.00523
1	0.00053	0.00083	0.00155	0.00213	0.00289	0.00328

Regarding the calculation of the floor acceleration, it was carried out on the basis of FEMA P-58 for the two main directions where the eigenperiods of the building are T_x = 0.53 sec and T_y = 0.51 sec. According to the FEMAP-58 methodology, at the base of the building the floor acceleration is equal to the ground acceleration (PGA), while for the upper floors the ground acceleration a_i* is calculated using the following equation:

$$\alpha_i^* = H_{ai}\left(S, T, h_i, H\right) \cdot PGA \tag{4.1}$$

where:

H_{ai} = acceleration correction factor for floor i ≥ 2 equal to:

$$\ln\left(H_{ai}\right) = a_o + a_1 T_1 + a_2 S + a_3 \frac{h_i}{H} + a_4 \left(\frac{h_i}{H}\right)^2 + a_5 \left(\frac{h_i}{H}\right)^3 \tag{4.2}$$

where:

T_1 = fundamental eigenperiod of the building in each direction

h_i = story height

H = total building height

a_0, a_1, a_2, a_3, a_4, a_5 = coefficients that have to do with the structural type of the building (Table 5-4 of FEMA P-58[10]). In our case it is a wall system, so $a_0 = 0.66$, $a_1 = -0.15$, $a_2 = -0.084$, $a_3 = -0.26$, $a_4 = 0.57$, $a_5 = 0.00$

S = strength ratio equal to:

$$S = \frac{S_a(T_1)W}{V_y} \tag{4.3}$$

where:

$S_a(T_1)$ = spectral acceleration derived from hazard curve

W = total weight of the building

V_y = estimated yield strength of the building as calculated in § 3.3.2, 3.3.3 for the model without infills

Table 4.13 - Table 4.16 show the results of the floor acceleration in the x and y directions for each seismic intensity for both standards (KANEPE, EC8). It should be noted that the PGA value has been multiplied by the soil factor S.

Table 4.13 Floor acceleration for the examined intensities – KANEPE (X-Direction)

Floor	**Intensity 1 (86%poe)**	**Intensity 2 (50%poe)**	**Intensity 3 (32%poe)**	**Intensity 4 (20%poe)**	**Intensity 5 (10%poe)**	**Intensity 6 (2%poe)**
8	0.058g	0.123g	0.175g	0.233g	0.327g	0.604g
7	0.052g	0.110g	0.156g	0.208g	0.292g	0.539g
6	0.048g	0.101g	0.143g	0.190g	0.267g	0.493g
5	0.044g	0.094g	0.133g	0.178g	0.249g	0.461g
4	0.042g	0.090g	0.127g	0.170g	0.238g	0.441g
3	0.042g	0.088g	0.125g	0.166g	0.233g	0.431g
2	0.042g	0.088g	0.125g	0.167g	0.234g	0.432g
1 (PGA)	0.026g	0.055g	0.078g	0.104g	0.147g	0.300g

Table 4.14 Floor acceleration for the examined intensities – KANEPE (Y-Direction)

Floor	**Intensity 1 (86%poe)**	**Intensity 2 (50%poe)**	**Intensity 3 (32%poe)**	**Intensity 4 (20%poe)**	**Intensity 5 (10%poe)**	**Intensity 6 (2%poe)**
8	0.058g	0.124g	0.175g	0.234g	0.321g	0.580g
7	0.052g	0.111g	0.157g	0.209g	0.287g	0.518g
6	0.048g	0.101g	0.143g	0.191g	0.262g	0.473g
5	0.045g	0.094g	0.134g	0.178g	0.245g	0.443g
4	0.043g	0.090g	0.128g	0.170g	0.234g	0.423g
3	0.042g	0.088g	0.125g	0.167g	0.229g	0.414g
2	0.042g	0.089g	0.125g	0.167g	0.230g	0.415g
1 (PGA)	0.026g	0.055g	0.078g	0.104g	0.147g	0.300g

Table 4.15 Floor acceleration for the examined intensities – EC8 (X-Direction)

Floor	**Intensity 1 (50%poe)**	**Intensity 2 (30%poe)**	**Intensity 3 (10%poe)**	**Intensity 4 (5%poe)**	**Intensity 5 (3%poe)**	**Intensity 6 (2%poe)**
8	0.123g	0.184g	0.330g	0.439g	0.564g	0.617g
7	0.110g	0.164g	0.295g	0.392g	0.503g	0.551g
6	0.101g	0.150g	0.269g	0.358g	0.460g	0.503g
5	0.094g	0.140g	0.252g	0.335g	0.430g	0.471g
4	0.090g	0.134g	0.241g	0.320g	0.411g	0.450g
3	0.088g	0.131g	0.236g	0.313g	0.402g	0.440g
2	0.088g	0.131g	0.236g	0.314g	0.403g	0.441g
1 (PGA)	0.055g	0.082g	0.147g	0.202g	0.269g	0.300g

Table 4.16 Floor acceleration for the examined intensities – EC8 (Y-Direction)

Floor	**Intensity 1 (50%poe)**	**Intensity 2 (30%poe)**	**Intensity 3 (10%poe)**	**Intensity 4 (5%poe)**	**Intensity 5 (3%poe)**	**Intensity 6 (2%poe)**
8	0.124g	0.185g	0.326g	0.430g	0.548g	0.598g
7	0.111g	0.165g	0.291g	0.384g	0.489g	0.534g
6	0.101g	0.150g	0.266g	0.351g	0.447g	0.488g
5	0.094g	0.141g	0.248g	0.328g	0.418g	0.456g
4	0.090g	0.135g	0.238g	0.314g	0.399g	0.436g
3	0.088g	0.132g	0.233g	0.307g	0.391g	0.427g
2	0.089g	0.132g	0.233g	0.307g	0.391g	0.427g
1 (PGA)	0.055g	0.082g	0.147g	0.202g	0.269g	0.300g

Figure 4.9 - Figure 4.13 show the repair cost of each element of the structure in detail for each intensity scenarios using the results from KANEPE.

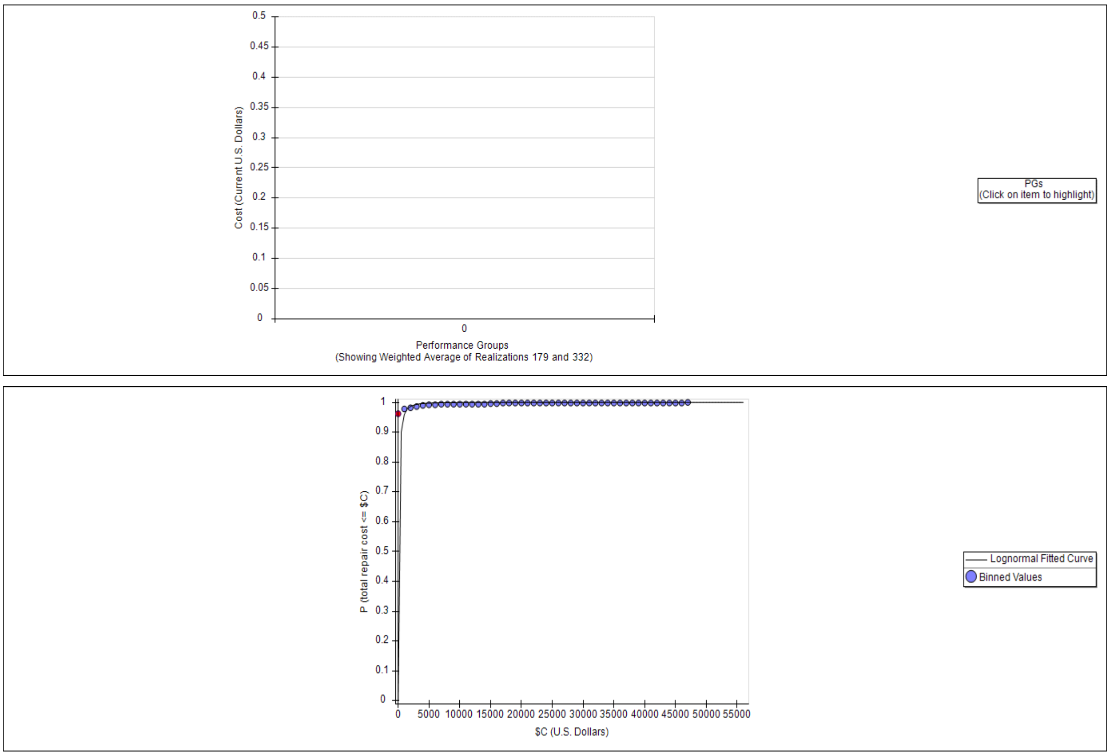


Figure 4.9 Distribution of repair cost of the building, seismic scenario 1 (KANEPE)

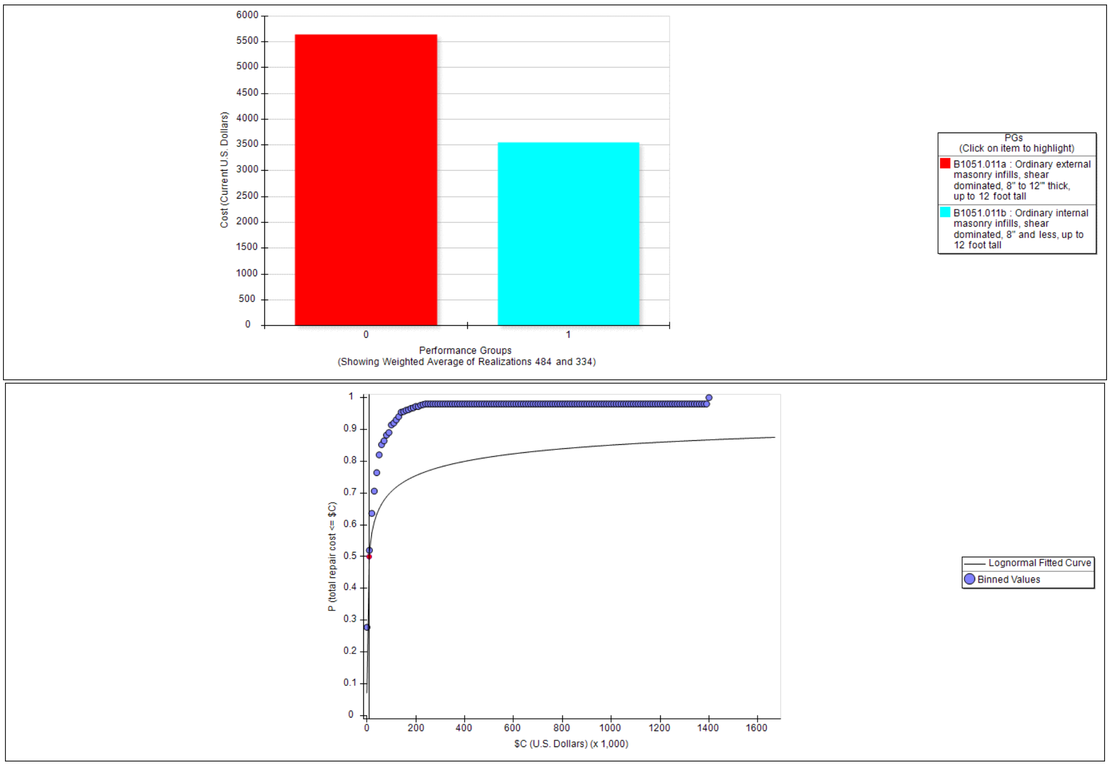


Figure 4.10 Distribution of repair cost of the building, seismic scenario 2 (KANEPE)

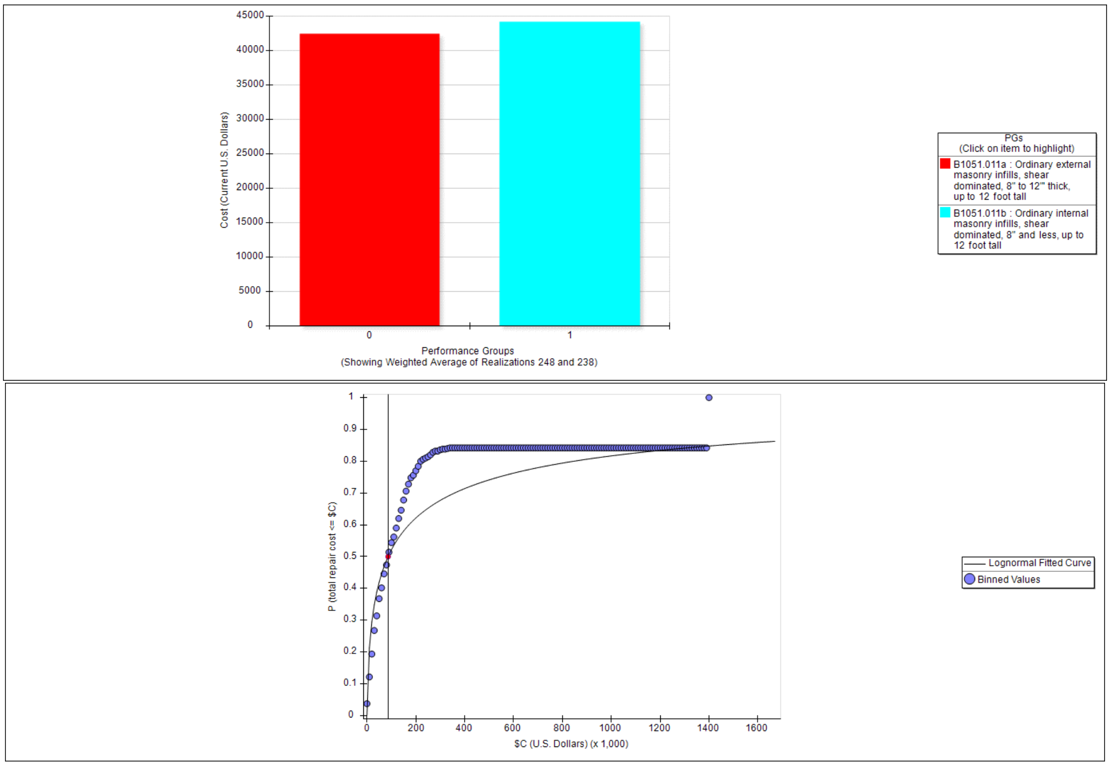


Figure 4.11 Distribution of repair cost of the building, seismic scenario 3 (KANEPE)

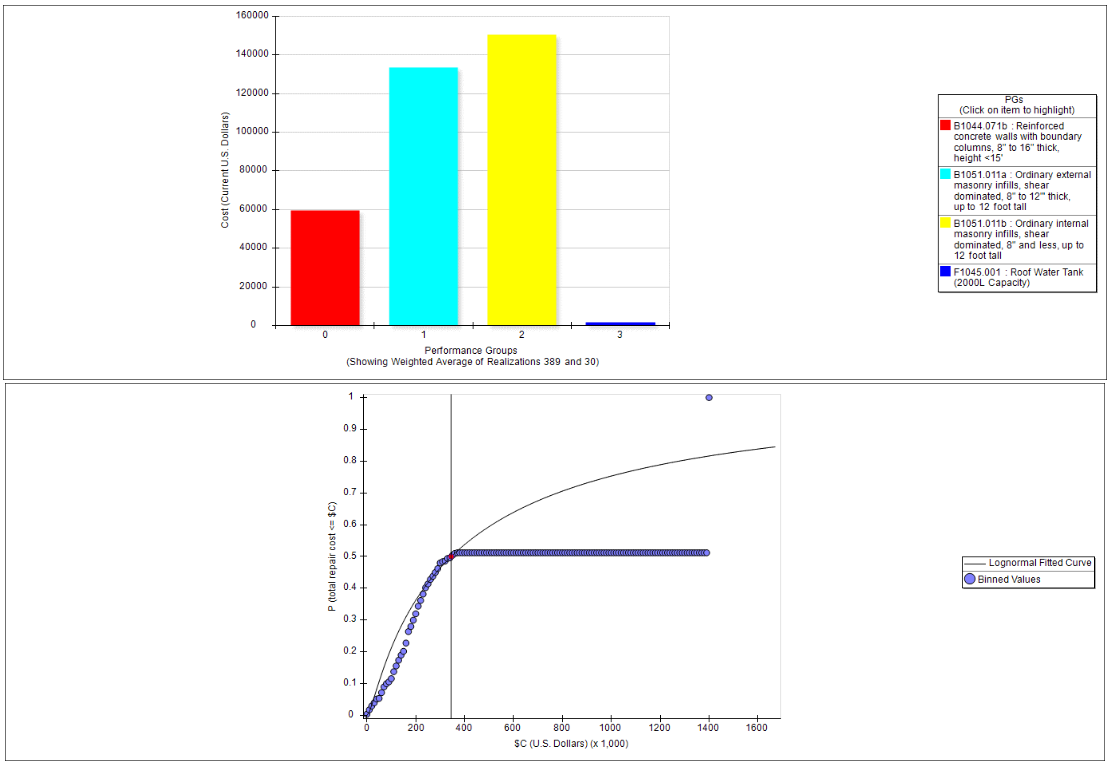


Figure 4.12 Distribution of repair cost of the building, seismic scenario 4 (KANEPE)

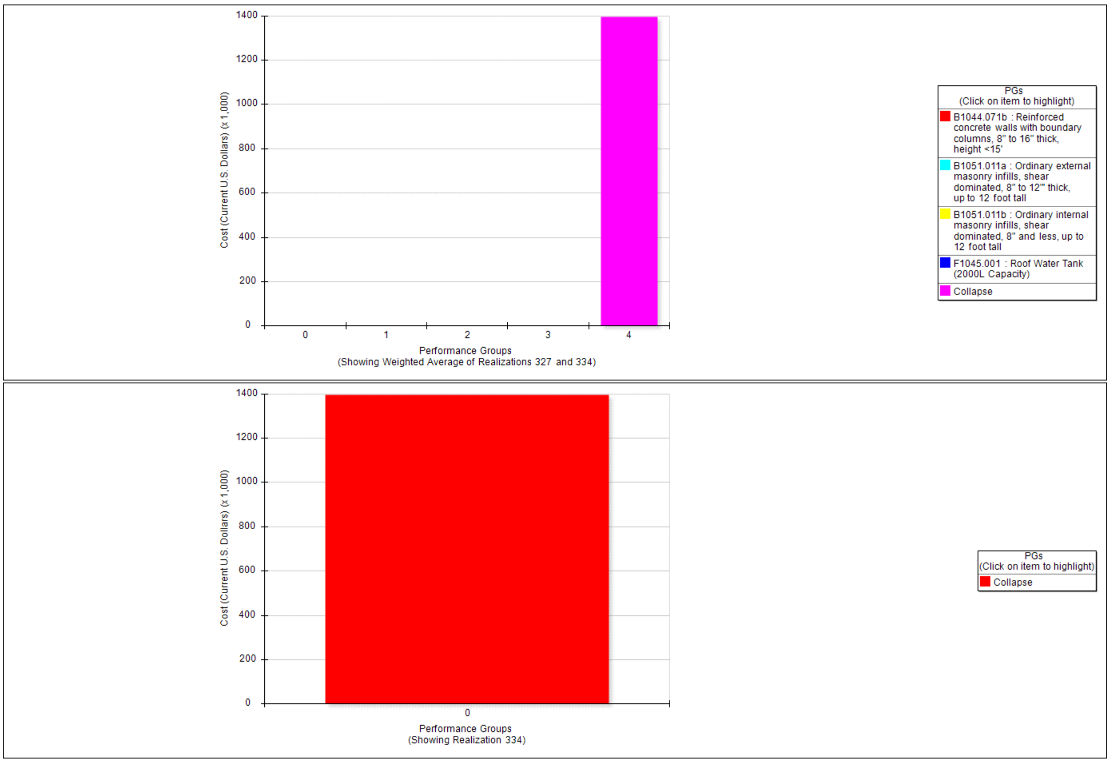


Figure 4.13 Distribution of repair cost of the building, seismic scenarios 5 - 6 (KANEPE)

As described in the figures above, in the case of scenario 1 (86% in 50 years – 25% $S_a(T)$), there is no damage. Regarding scenario 2 (50% in 50 years – 53% $S_a(T)$) and scenario 3 (32% in 50 years – 75% $S_a(T)$), there is damage to the infills (external and internal) and the repair cost is $9174 and 86500$, respectively. In the case of scenario 4

(20% in 50 years – 100% $S_a(T)$), which represents the first failure of the wall, there is damage to the infills, the walls and the roof water tank with the average repair cost being \$345000. Finally, in the case of scenario 5 (10% in 50 years – 142% $S_a(T)$) and scenario 6 (2% in 50 years – 288% $S_a(T)$), severe damage occurs leading to collapse which is normal since the spectral acceleration is higher than the collapse acceleration we entered into the software. Table 4.17 shows the final repair cost for each intensity and the total threshold as a ratio of the total replacement cost, where we can see that the repair cost for 100% of the spectral collapse acceleration is almost 25% of the total replacement cost which is a great result for a newly constructed building.

Table 4.17 Average building repair cost for each seismic scenario - KANEPE

Scenario	**$S_a(T_{mean})$ (g)**	**MAFE**	**Repair Cost (\$)**	**Threshold (%)**
Intensity 1 (86% in 50 years - 25% Sa(T))	0.063	0.0395	0	0
Intensity 2 (50% in 50 years - 53% Sa(T))	0.133	0.0139	9174	0.66
Intensity 3 (32% in 50 years - 75% Sa(T))	0.188	0.0078	86500	6.20
Intensity 4 (20% in 50 years - 100% Sa(T))	**0.250**	**0.0044**	**345000**	**24.72**
Intensity 5 (10% in 50 years - 142% Sa(T))	0.354	0.0021	1395597	100
Intensity 6 (2% in 50 years - 288% Sa(T))	0.721	0.0004	1395597	100

The same procedure was followed for the intensities of the EC8. Figure 4.14 - Figure 4.19 show the results obtained by the software.

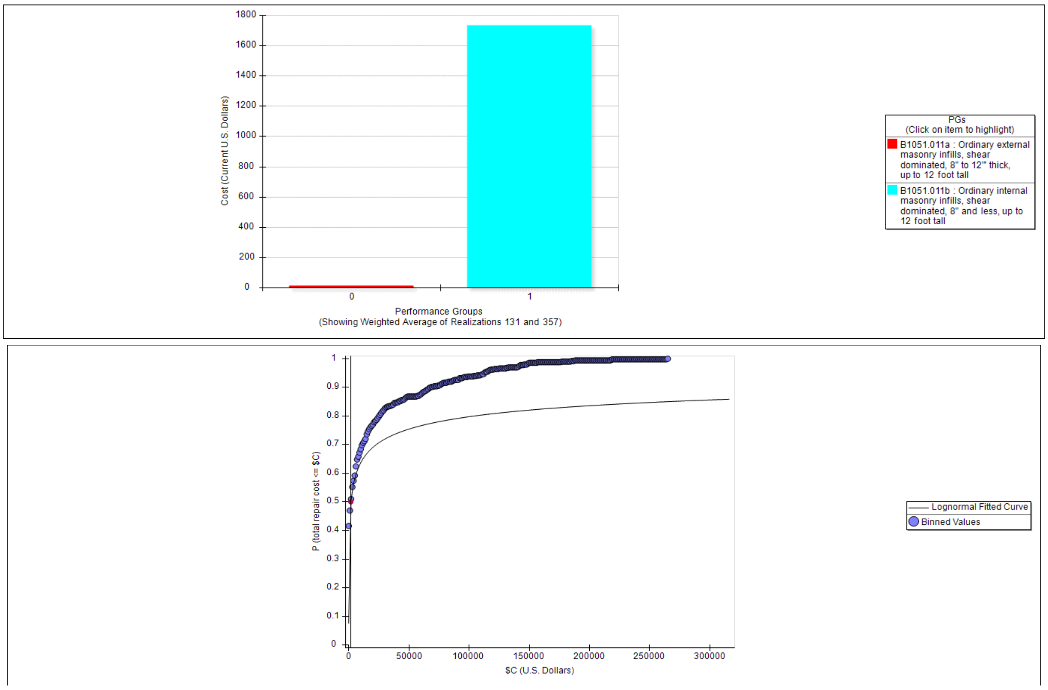


Figure 4.14 Distribution of repair cost of the building, seismic scenario 1 (EC8)

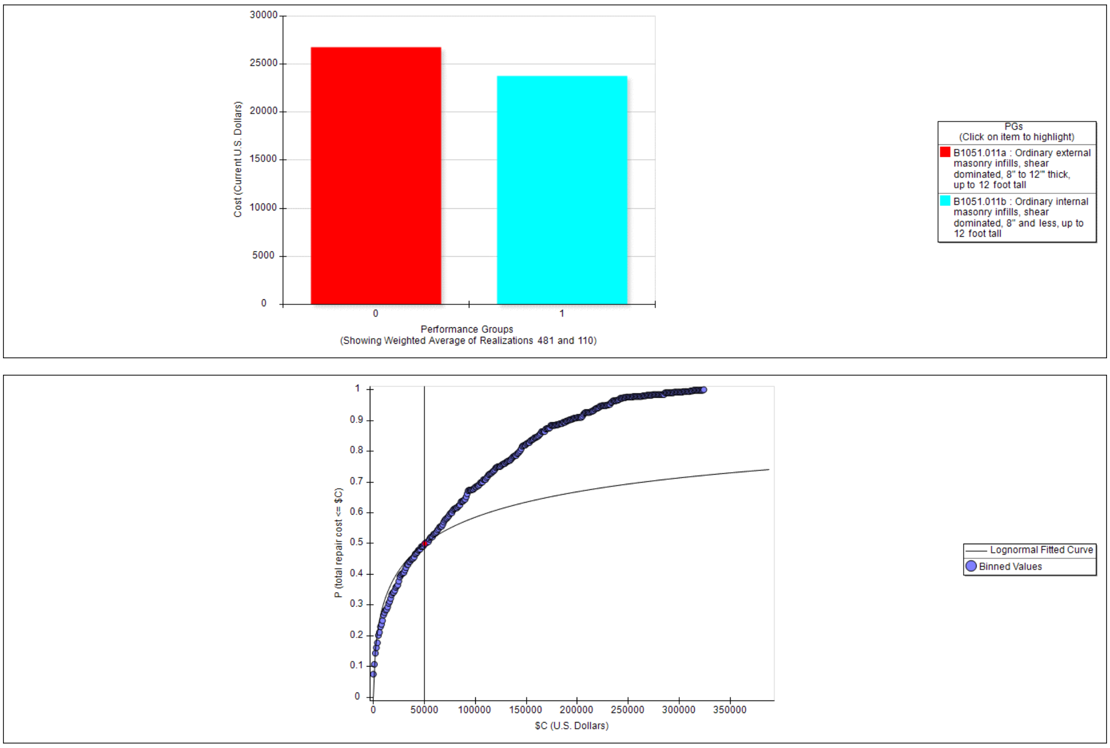


Figure 4.15 Distribution of repair cost of the building, seismic scenario 2 (EC8)

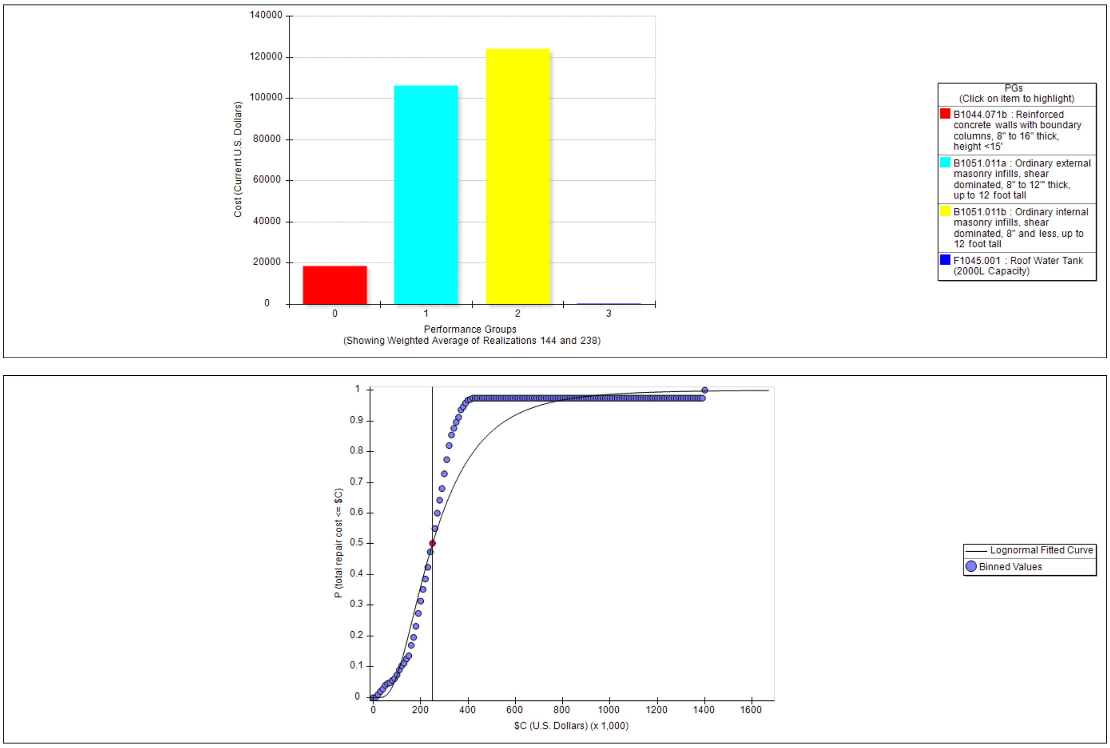


Figure 4.16 Distribution of repair cost of the building, seismic scenario 3 (EC8)

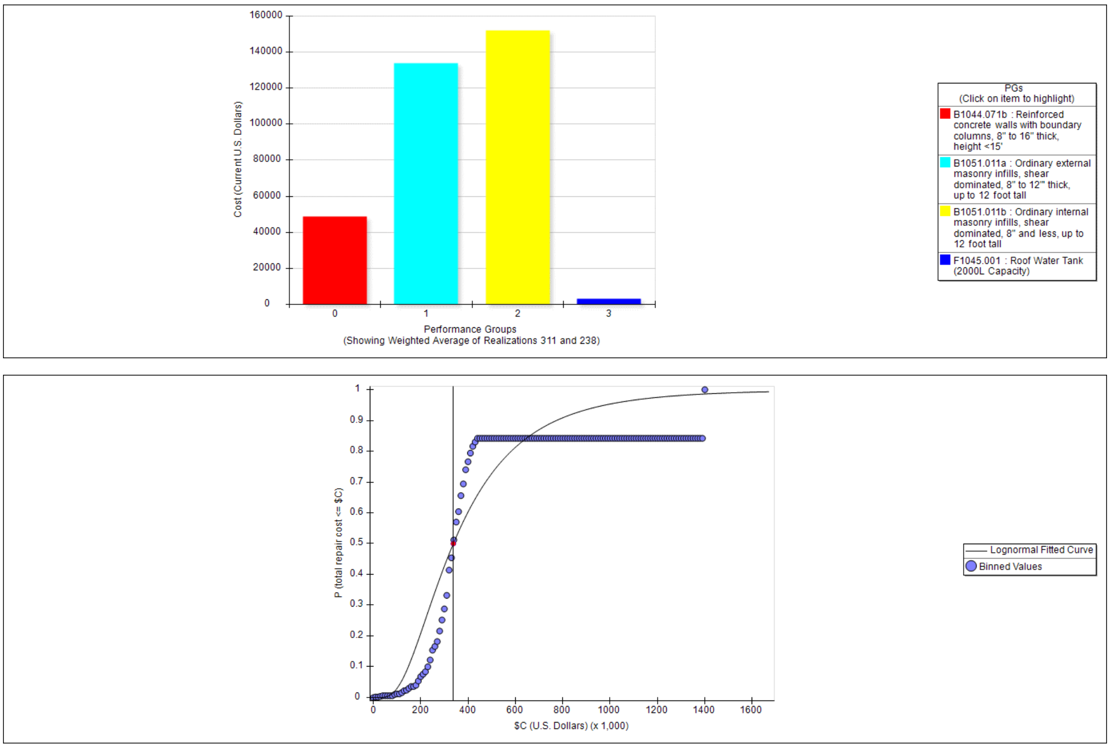


Figure 4.17 Distribution of repair cost of the building, seismic scenario 4 (EC8)

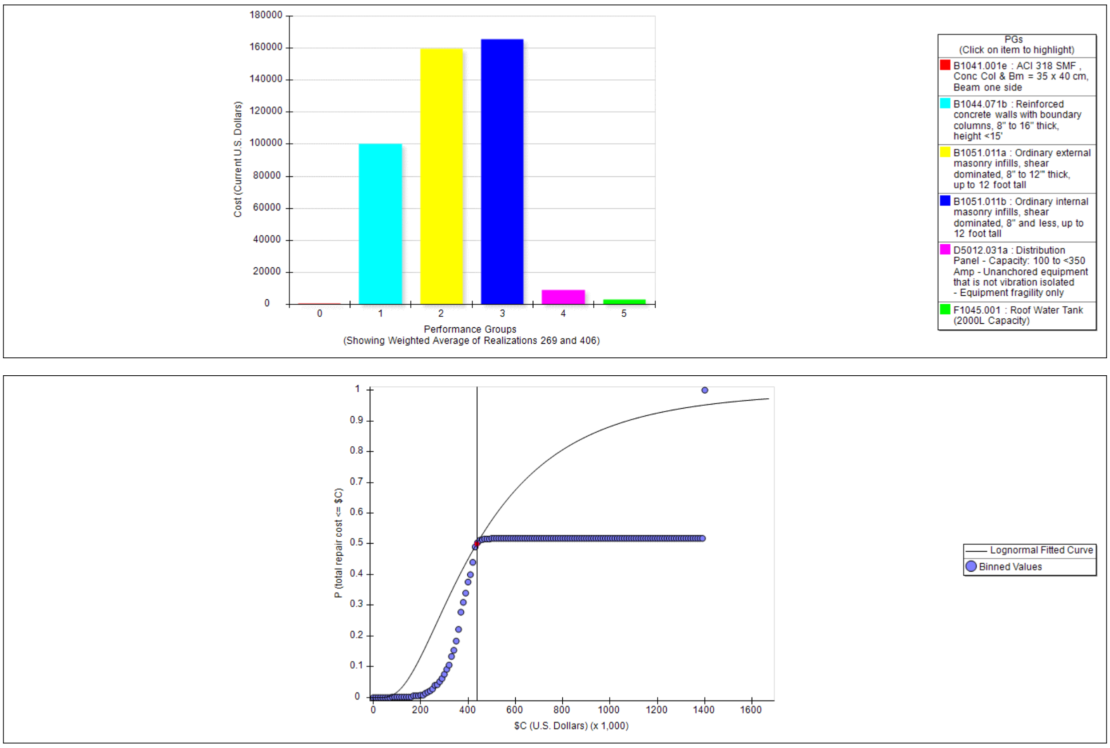


Figure 4.18 Distribution of repair cost of the building, seismic scenario 5 (EC8)

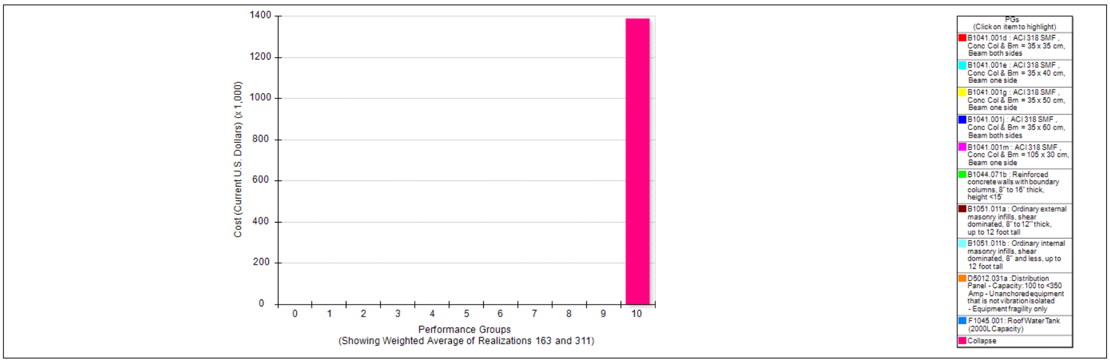


Figure 4.19 Distribution of repair cost of the building, seismic scenario 6 (EC8)

As described in the figures above, in the case of scenario 1 (50% in 50 years – 20% $S_a(T)$) and scenario 2 (30% in 50 years – 30% $S_a(T)$), there is damage only to the infills (external and internal) and the repair cost is \$1750 and 50500\$, respectively. In the case of scenario 3 (10% in 50 years – 55% $S_a(T)$) and scenario 4 (5% in 50 years – 75% $S_a(T)$) there is damage to the infills and a little bit on the walls and the average repair cost is \$249286 and \$337931, respectively. For scenario 5 (3% in 50 years – 100% $S_a(T)$), which represents the first failure of the wall, there is damage to the infills, the walls, the roof water tank and also the distribution panel and a column, with the average repair cost being \$437143. Finally, in the case of scenario 6 (2% in 50 years – 111% $S_a(T)$) as previous, severe damage occurs leading to collapse which is normal since the spectral acceleration is higher than the collapse acceleration we entered into the software for this code. Table 4.18 shows the final repair cost for each intensity and the total threshold as a ratio of the total replacement cost, where we can see that the repair cost for 100% of the spectral collapse acceleration is almost 31% of the total replacement cost which is an excellent result because it is a rare earthquake, with 3% poe in 50 years.

Table 4.18 Average building repair cost for each seismic scenario – EC8

Scenario	$S_a(T_{mean})$ (g)	MAFE	Repair Cost (\$)	Threshold (%)
Intensity 1 (50% in 50 years - 20% Sa(T))	0.133	0.0139	1750	0.13
Intensity 2 (30% in 50 years - 30% Sa(T))	0.197	0.0071	50500	3.62
Intensity 3 (10% in 50 years - 55% Sa(T))	0.354	0.0021	249286	17.86
Intensity 4 (5% in 50 years - 75% Sa(T))	0.485	0.0011	337931	24.21
Intensity 5 (3% in 50 years - 100% Sa(T))	**0.647**	**0.0006**	**437143**	**31.32**
Intensity 6 (2% in 50 years - 111% Sa(T))	0.721	0.0004	1395597	100

Figure 4.20 - Figure 4.21 show the average annual repair cost of the reinforced concrete building for both codes (KANEPE and EC8). In the case of KANEPE, the average annual repair cost is \$9277 corresponding to 0.66% of the total replacement cost, while on EC8

the annual repair cost is $2241 corresponding to 0.16% of the total replacement cost. Therefore, the average annual repair cost of the building is considered to be significantly lower than the replacement cost for both codes (KANEPE and EC8), which means that building will not have problems.

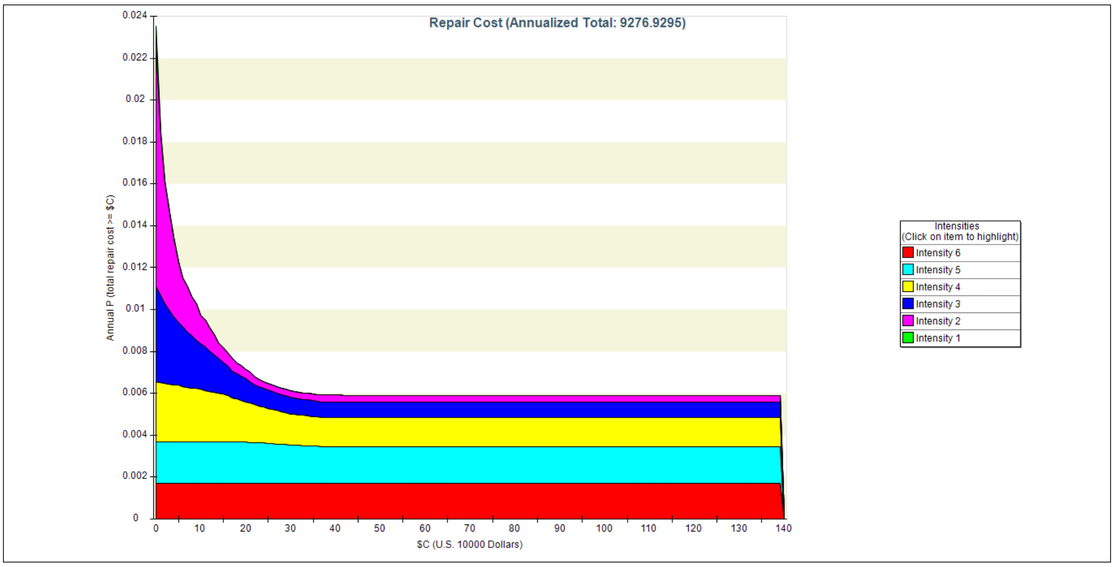


Figure 4.20 Annual probability of exceedance of repair cost of the building (KANEPE)

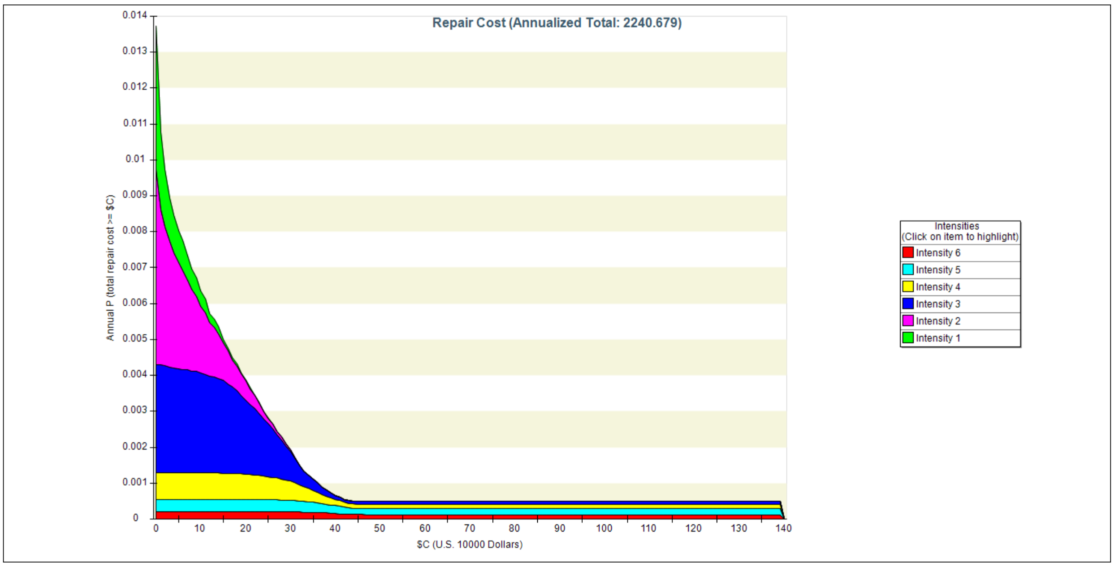


Figure 4.21 Annual probability of exceedance of repair cost of the building (EC8)

Chapter 5

Conclusions

In the present study, we investigated the seismic behavior of a newly constructed 7-story reinforced concrete building located in Kallithea, using some pre-earthquake assessment method. The Rapid Visual Inspection (first level) method was used in order to classify the building in a priority category for further checking. For the second level rapid seismic assessment, the methods proposed by Dritsos S., Vougioukas E. and the new FEMA P-2018 were applied. In addition, nonlinear static analyses (third level) were performed based on the Greek code (KANEPE), Eurocode 8-Part 3 and ASCE/SEI 41-17. Finally, we studied the seismic behavior of a roof water tank and estimated the total seismic losses using the SPO2FRAG and PACT software. Thus, after applying the above, certain conclusions were drawn, which are set out below.

- From the first level pre-earthquake assessment method the building is classified as “Low Priority”, so no further actions needed
- From the comparison of the results of the shear capacity obtained from the second level pre-earthquake assessment methods (Dritsos S., Vougioukas E.) and the nonlinear analyses (KANEPE, EC8), it was observed that KANEPE gives conservative values compared to the EC8 and Dritsos S. method, where their values are really close. On contrast, Vougioukas method gives really high values that are useful only for comparison between buildings to find which one is more vulnerable
- We have only shear failures on the walls according to KANEPE, but we achieved the performance objective $B2^{+}$, which is the minimum seismic class according to the code
- Roof water tank fails for the design PGA = 0.16g, because the roof top acceleration is highly amplified. After some checks, it was found that the PGA value where this structure is safe (first failure) is 0.067g corresponding to an earthquake with 35-year return period
- Regarding the seismic losses, the results showed that for both codes (KANEPE and EC8), the critical elements are generally the infills (external and internal), the walls and the roof water tank with small average annual repair cost (0.66% of the total replacement cost → KANEPE, 0.16% of the total replacement cost → EC8)

Appendix A

(Calculation of V_{req} – Dritsos S. Method)

Calculation of building fundamental eigenperiod

The first thing that must be examined is the fundamental eigenperiod of the structure in order to find the spectral acceleration. The equation is the following:

$$T = C_t \cdot h_n{}^{\beta}$$

where:

$C_t = 0.052$

h_n = total height of the building (m)

$\beta = 0.90$

Calculation of spectral acceleration

The seismic demand is calculated by:

$$0 < T \le T_B \rightarrow S_d(T) = a_g \cdot S \cdot \left[\frac{2}{3} + \frac{T}{T_B} \cdot \left(\frac{2.5}{q} - \frac{2}{3}\right)\right]$$

$$T_B < T \le T_C \rightarrow S_d(T) = a_g \cdot S \cdot \left(\frac{2.5}{q}\right)$$

$$T_C < T \le T_D \rightarrow S_d(T) = a_g \cdot S \cdot \left(\frac{2.5}{q}\right) \cdot \left(\frac{T_C}{T}\right) \ge \beta \cdot a_g$$

$$T_D < T \le 4 \rightarrow S_d(T) = a_g \cdot S \cdot \left(\frac{2.5}{q}\right) \cdot \left(\frac{T_C \cdot T_D}{T^2}\right) \ge \beta \cdot a_g$$

where:

a_g = ground acceleration according to seismic zone (Importance Factor $\gamma = 1.00$) (Table A.1)

S, T_B, T_C, T_D = parameters corresponding to the soil category (Table A.2)

$\beta = 0.20$

q = behavior factor for performance level "Significant Damage" according to building construction date (Table A.3)

Table A.1 Ground acceleration according to seismic zone

Seismic Zone	a_g
Zone 1	0.16g
Zone 2	0.24g
Zone 3	0.36g

Table A.2 Parameters corresponding to the soil category

Soil Category	T_B (sec)	T_C (sec)	T_D (sec)	S
A	0.15	0.40	2.50	0.85
B	0.15	0.50	2.50	1.00
C	0.20	0.60	2.50	1.00
D	0.20	0.80	2.50	1.15
E	0.15	0.50	2.50	1.25

Table A.3 Behavior factor for performance level “Significant Damage”

Design Codes	Favorable Presence or Absence of Infills		Unfavorable Presence of Infills	
	Substantial damage to primary elements		Substantial damage to primary elements	
	NO	YES	NO	YES
1995 ≤...	3.0	2.3	2.3	1.7
1985 ≤...≤ 1995	2.3	1.7	1.7	1.3
...≤ 1985	1.7	1.3	1.3	1.1

Soil classification is the same as described in EC8, taking into account the $V_{S,30}$.

Calculation of seismic demand (V_{req})

The seismic demand is given by the following equation:

$$V_{req} = MS_d(T)$$

where:

M = building total mass based on the static loads for the combination $G+\psi_2 \cdot Q$

S_d (T) = spectral acceleration as described above

The seismic load distribution over the height can be realized by the relationship:

$$F_i = V_{req} \frac{M_i Z_i}{\sum_{j=1}^{n} M_j Z_j}, \qquad i, j = 1, 2, \ldots.., n$$

where:

M_i = building mass on floor i

Z_i = distance of the floor i from the base

Appendix B

(Building formwork drawings)

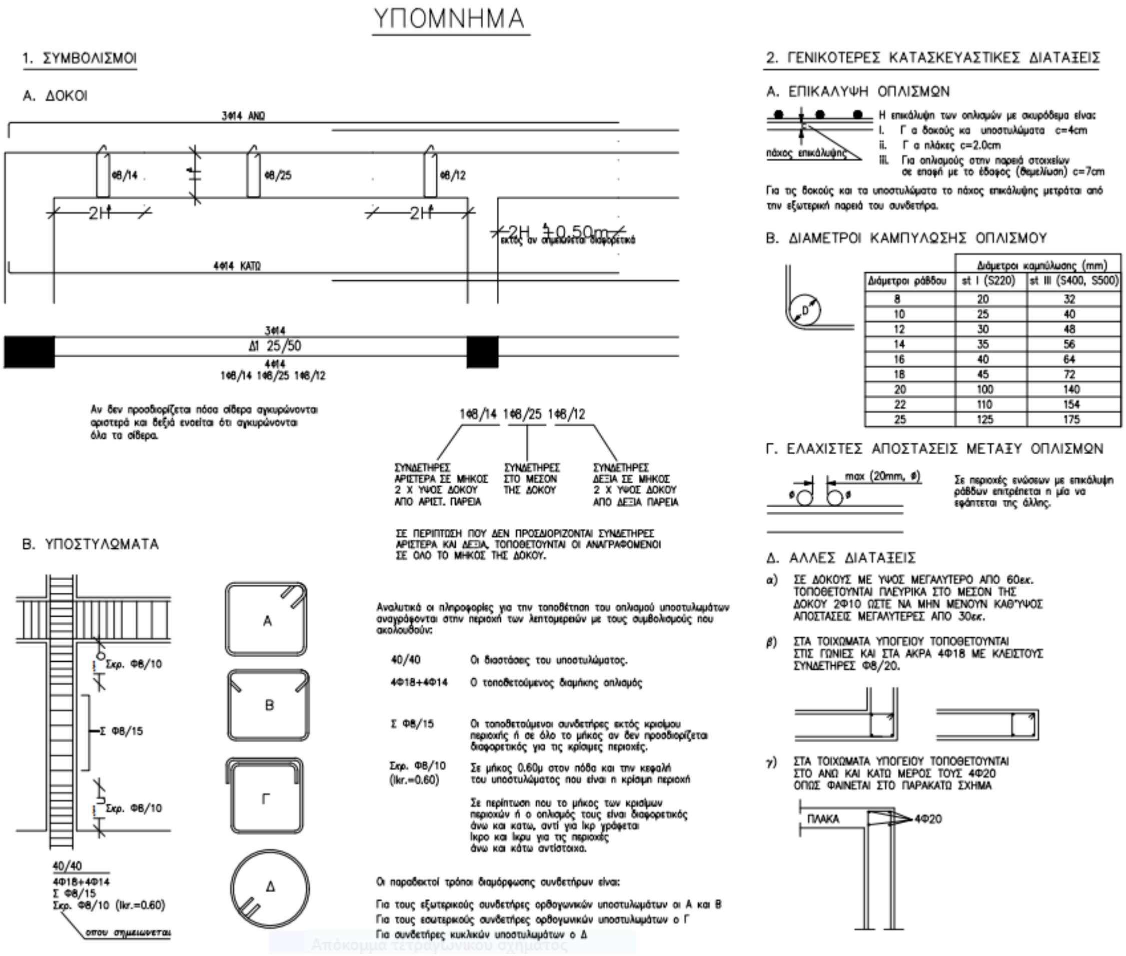


Figure B.1 General rules for reinforcement of elements

As described in the figure above, there are critical zones in beams and columns where dense stirrups are placed. The concrete cover is 40 mm for beams, walls, columns and 20 mm for slabs. Moreover, the lap length is adequate so that the concrete has a high bond.

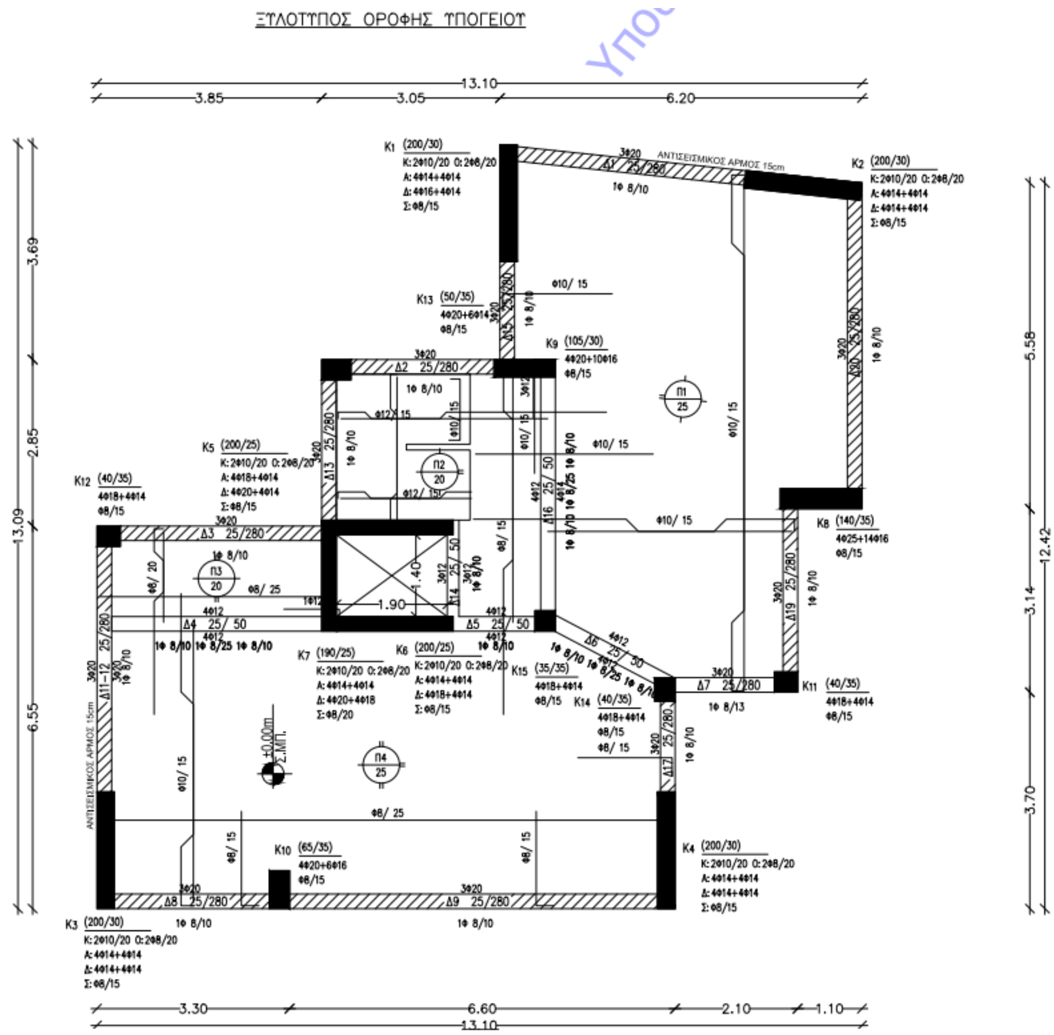


Figure B.2 Basement formwork

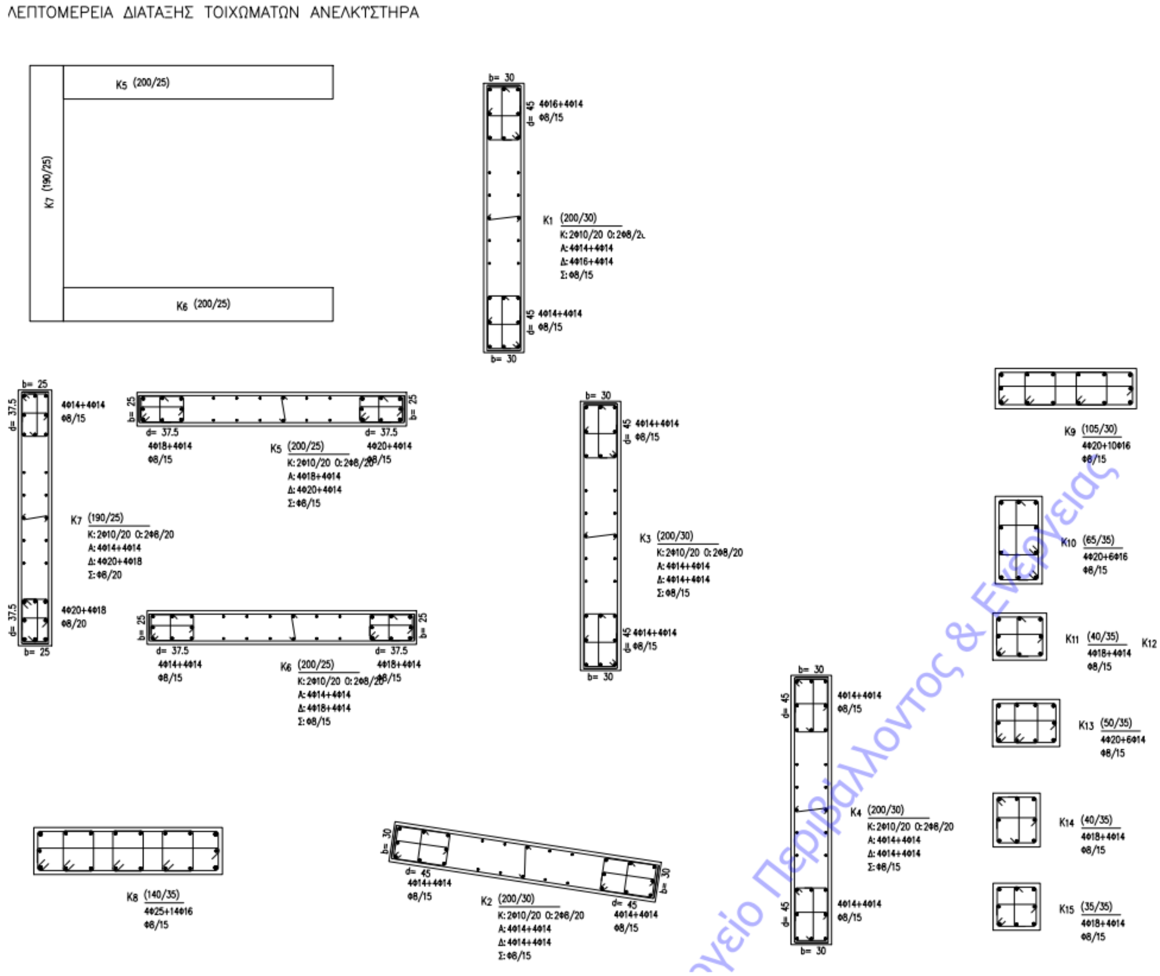


Figure B.3 Reinforcement detailing - Basement

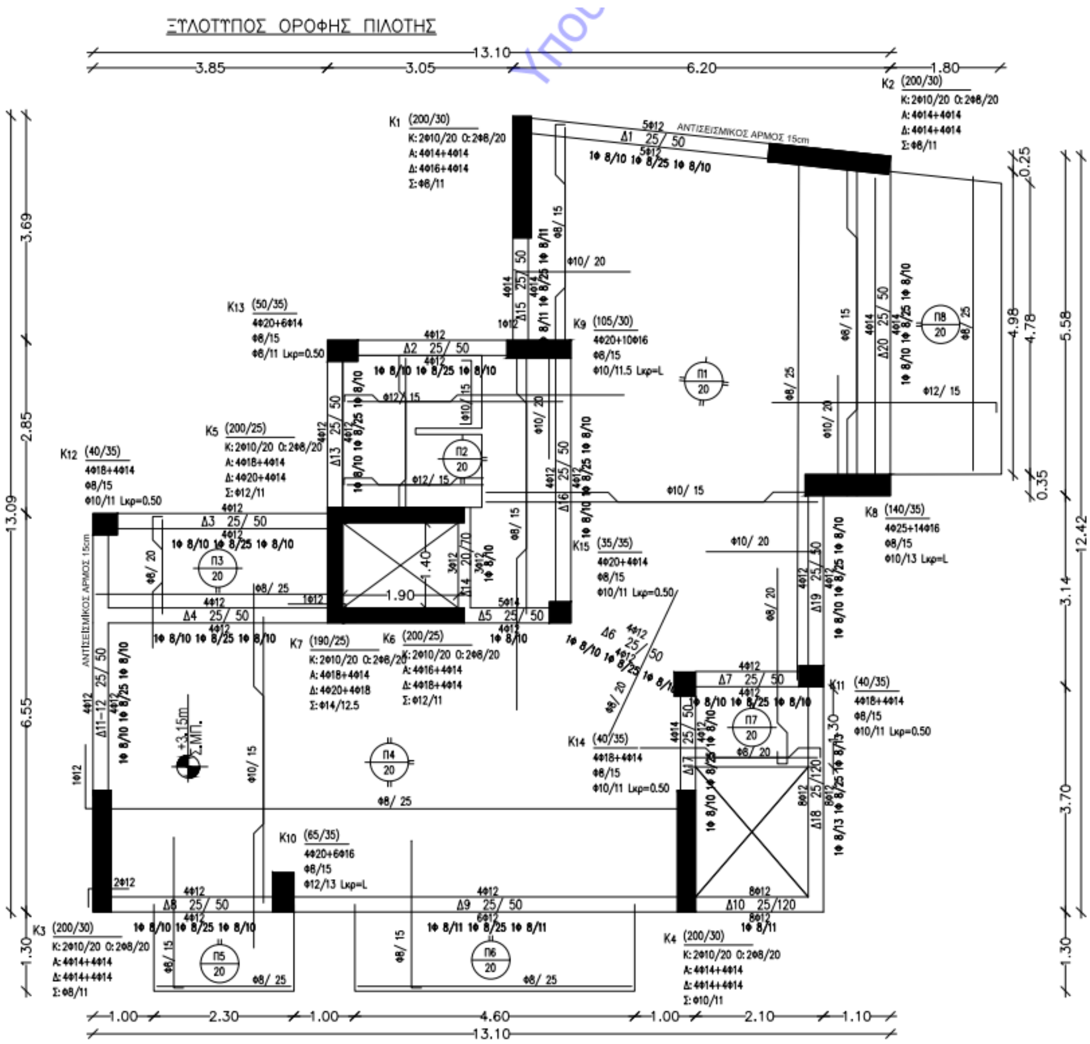


Figure B.4 Ground floor (pilotis) formwork

ΛΕΠΤΟΜΕΡΕΙΑ ΔΙΑΤΑΞΗΣ ΤΟΙΧΩΜΑΤΩΝ ΑΝΕΛΚΥΣΤΗΡΑ

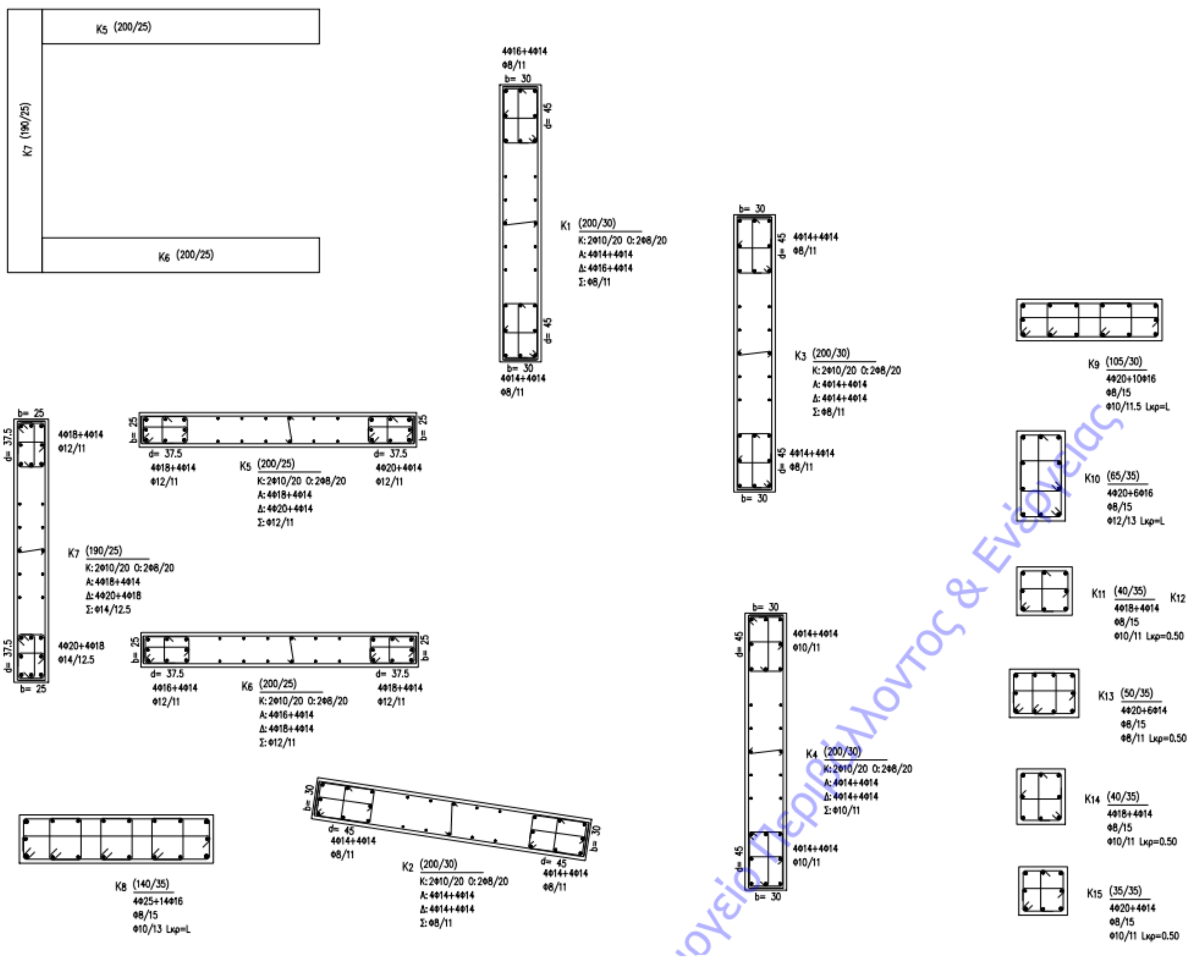


Figure B.5 Reinforcement detailing – Ground floor (Pilotis)

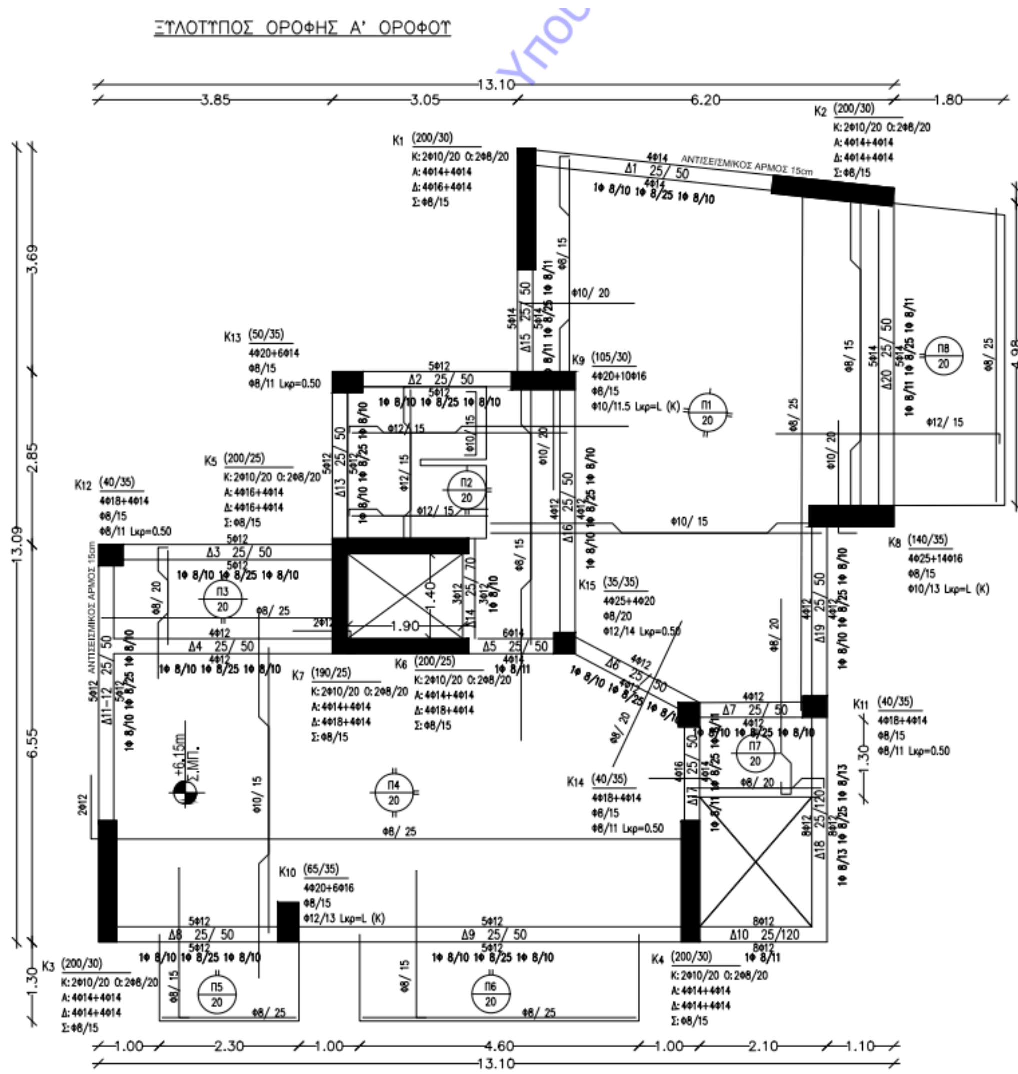


Figure B.6 Second floor formwork

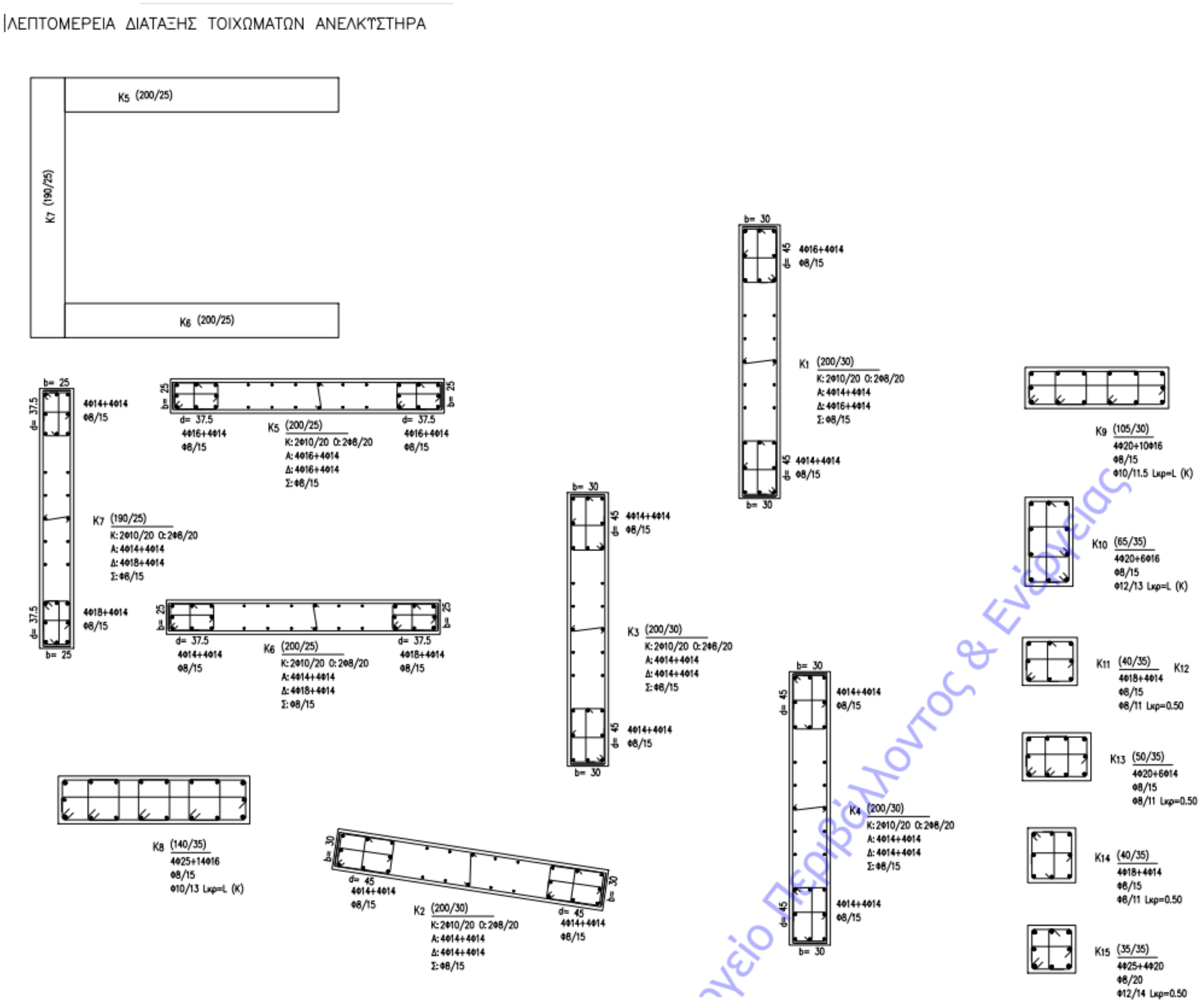


Figure B.7 Reinforcement detailing – Second floor

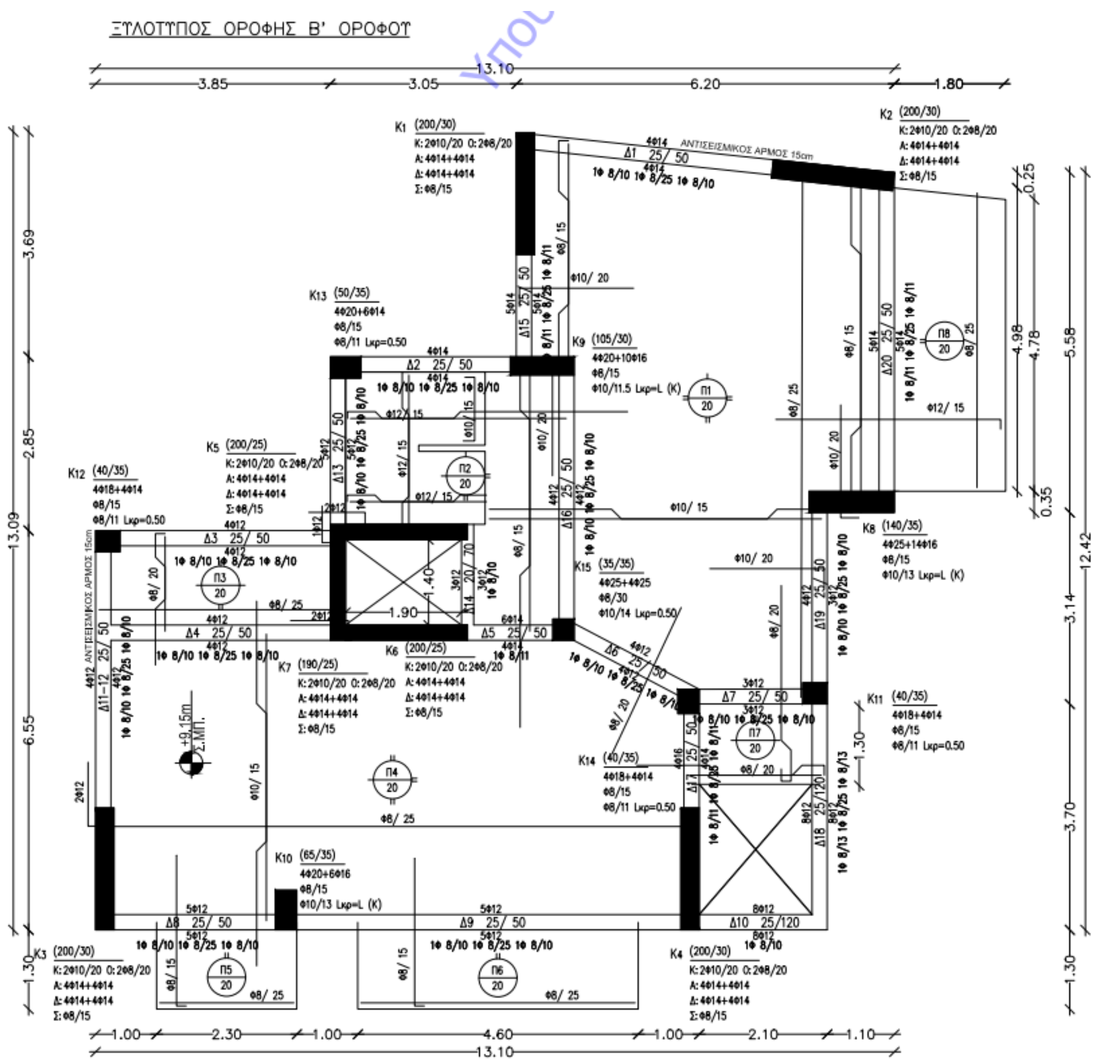


Figure B.8 Third floor formwork

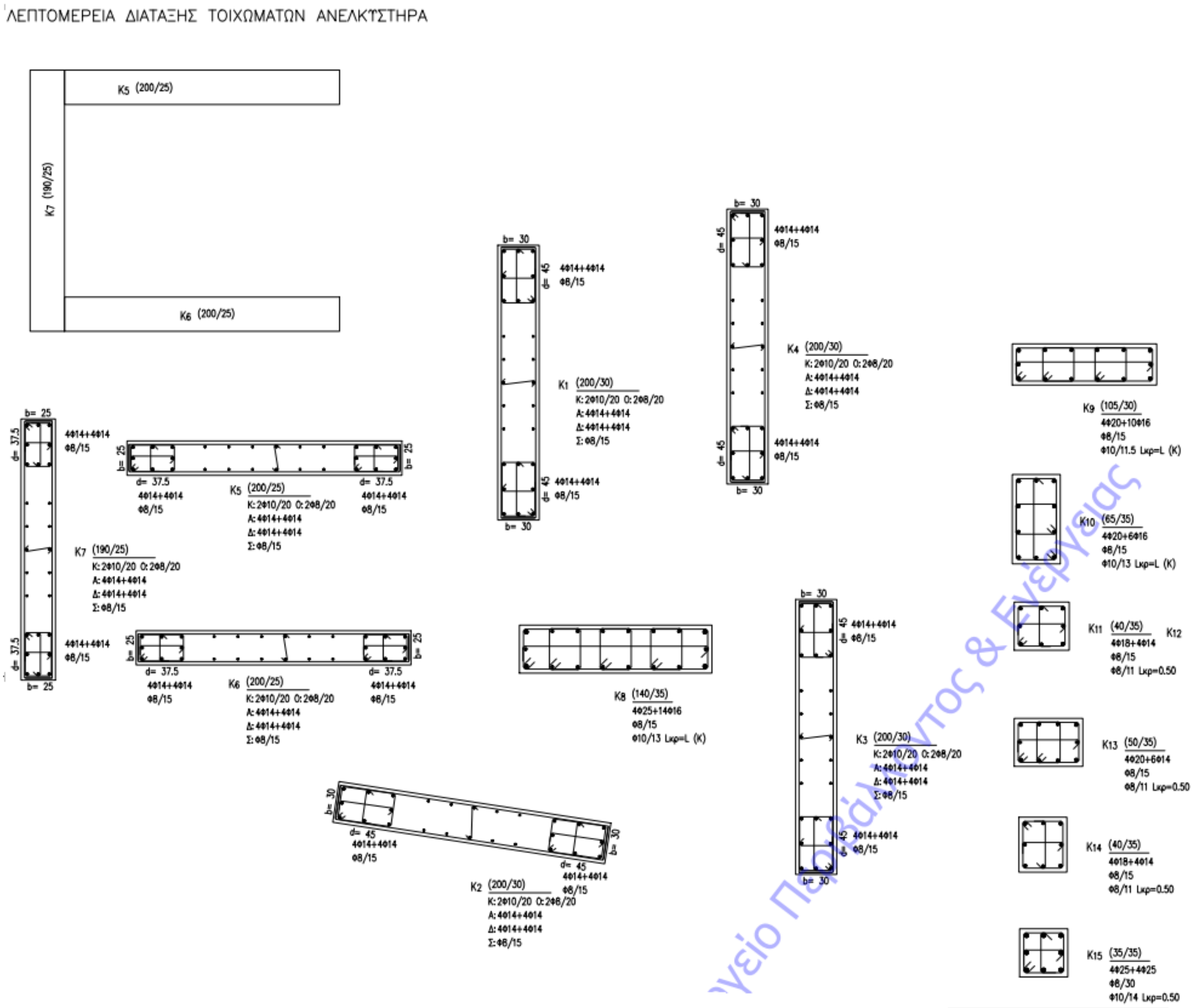


Figure B.9 Reinforcement detailing – Third floor

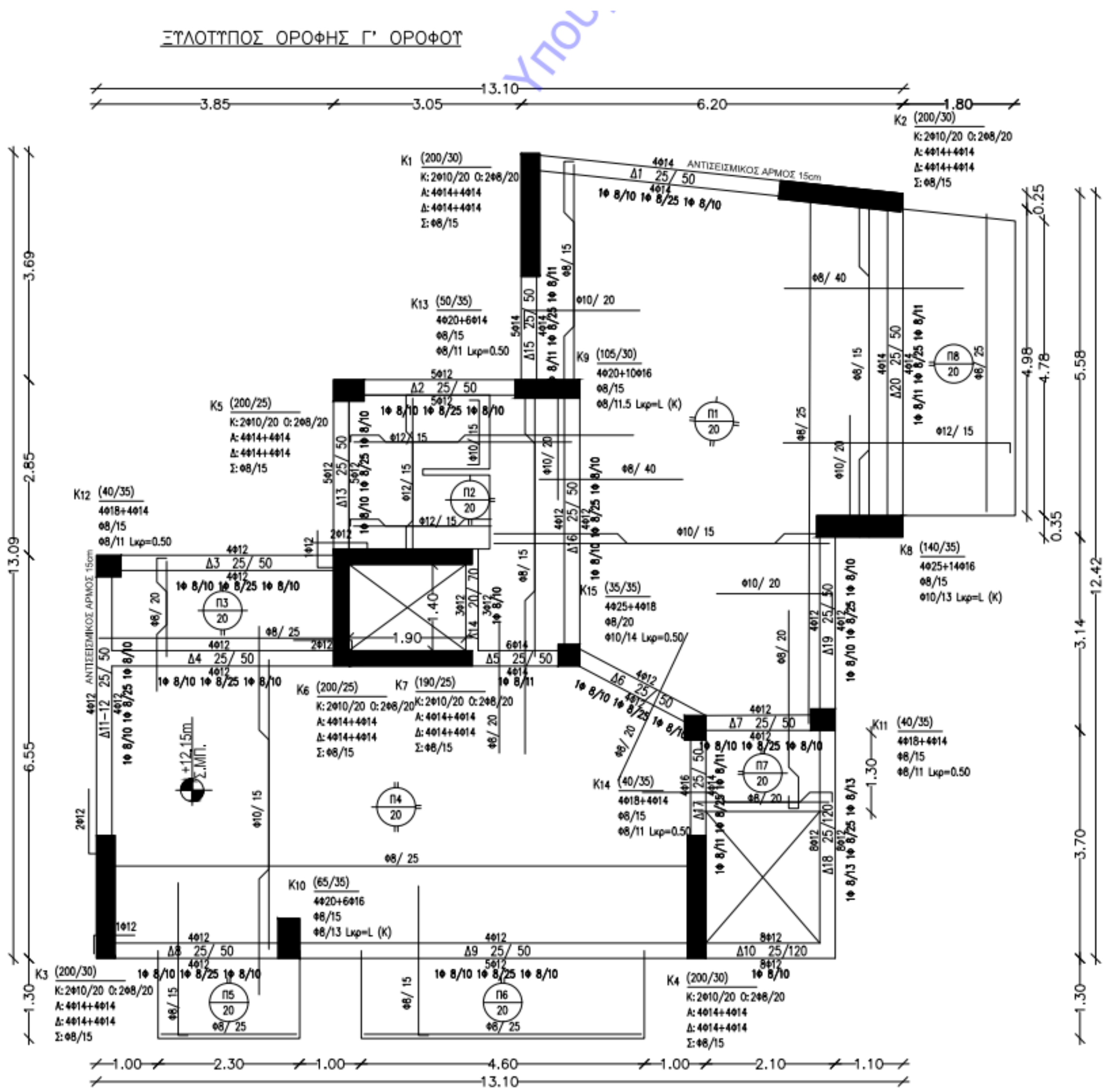


Figure B.10 Fourth floor formwork

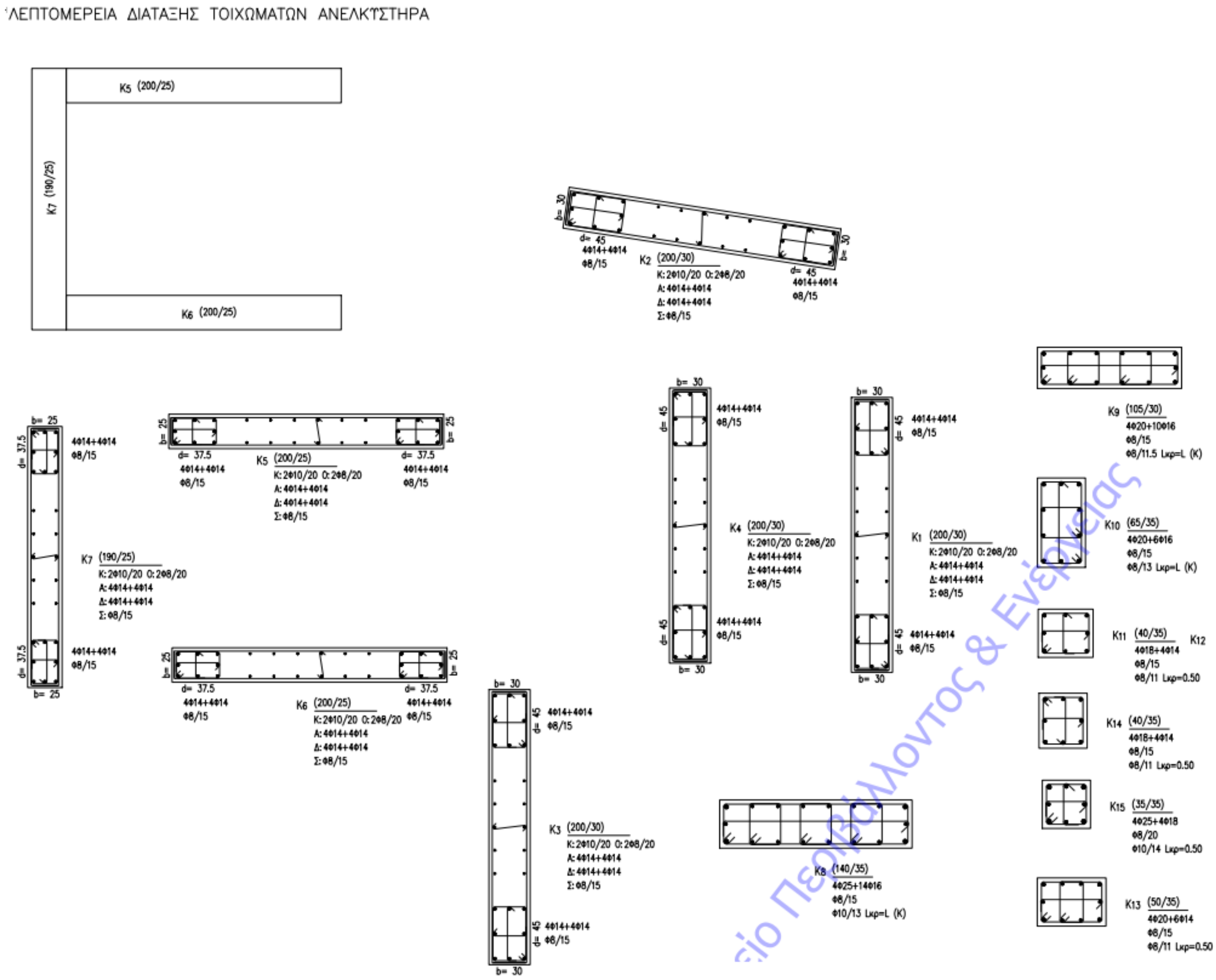


Figure B.11 Reinforcement detailing – Fourth floor

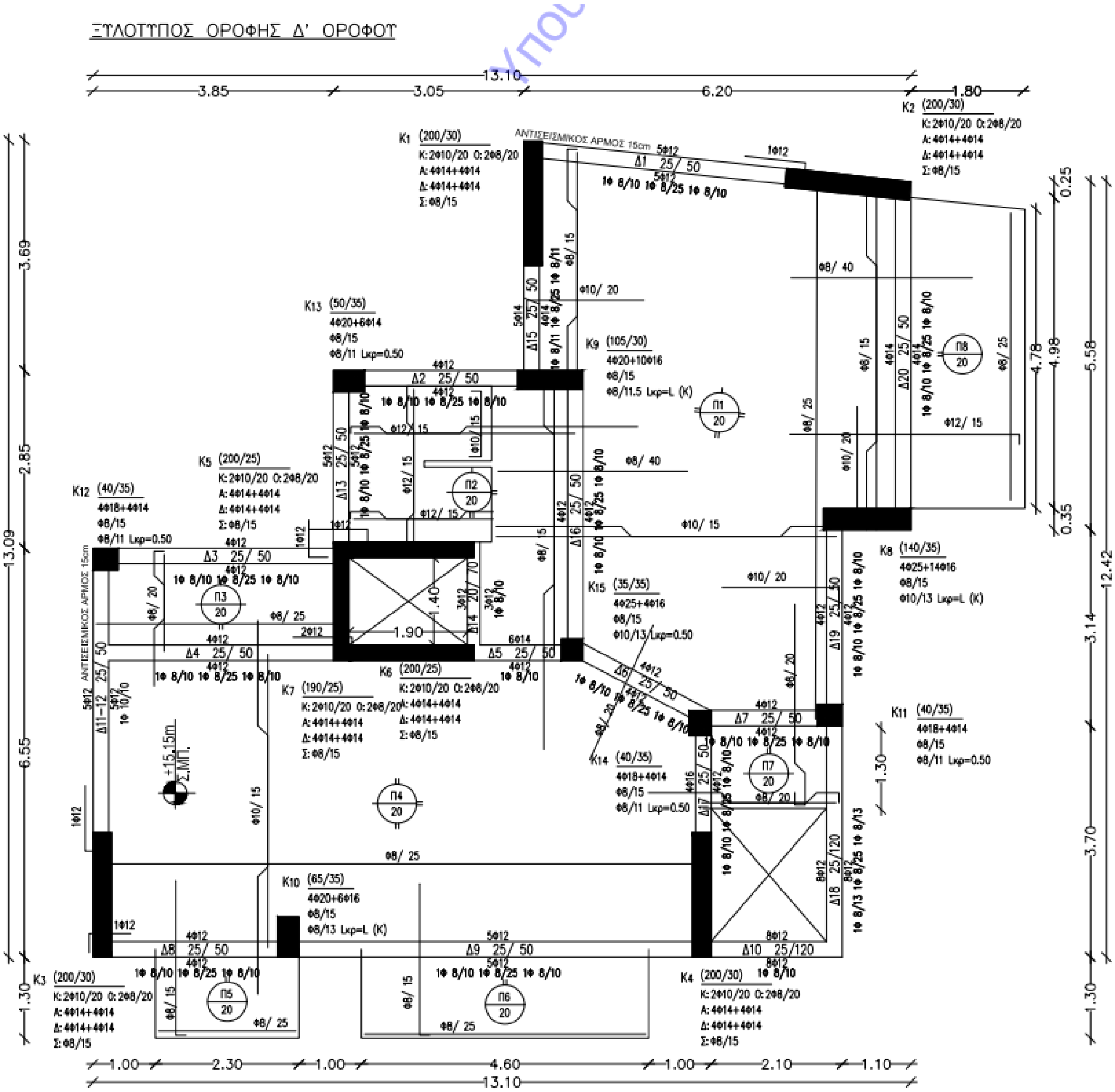


Figure B.12 Fifth floor formwork

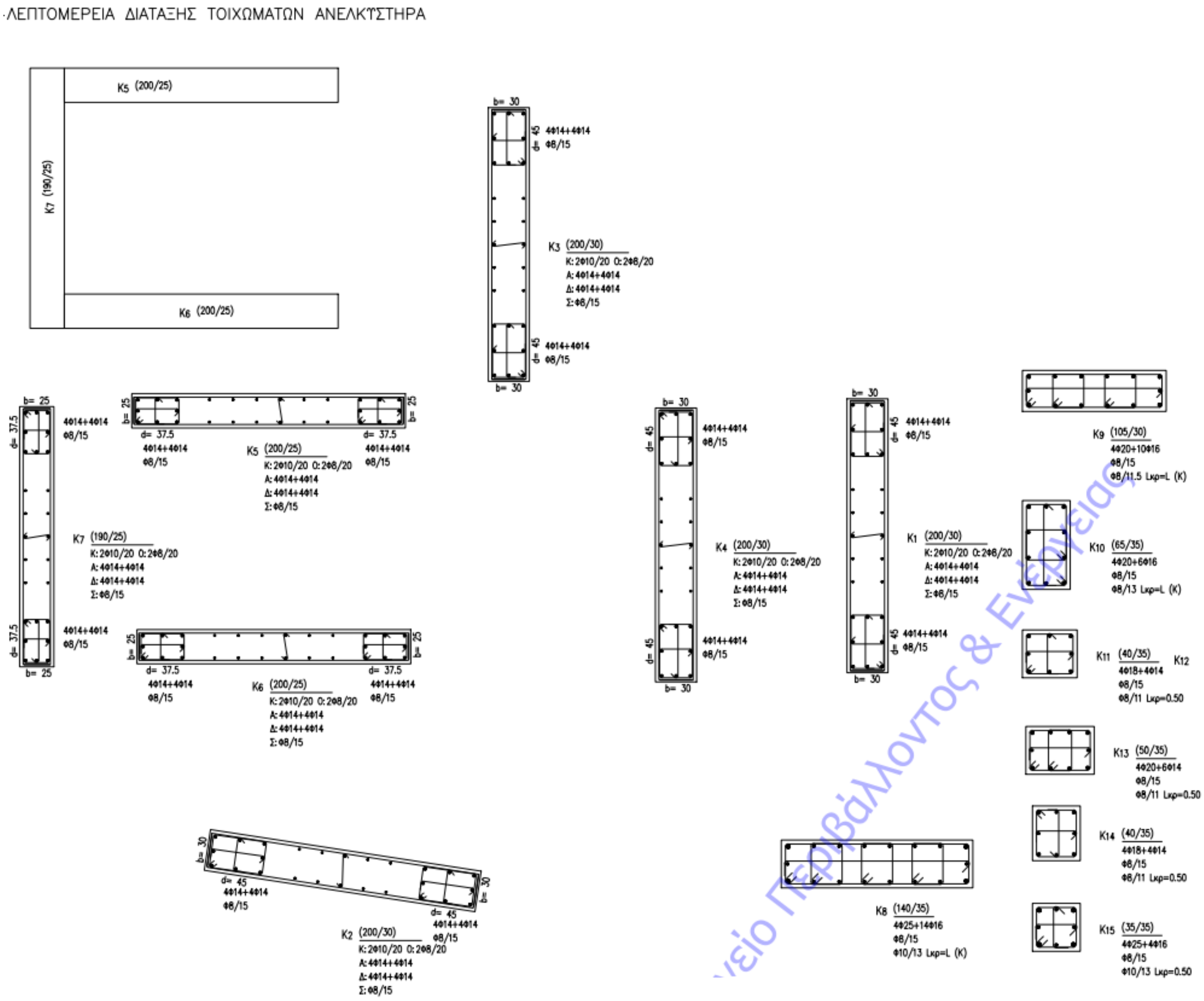


Figure B.13 Reinforcement detailing – Fifth floor

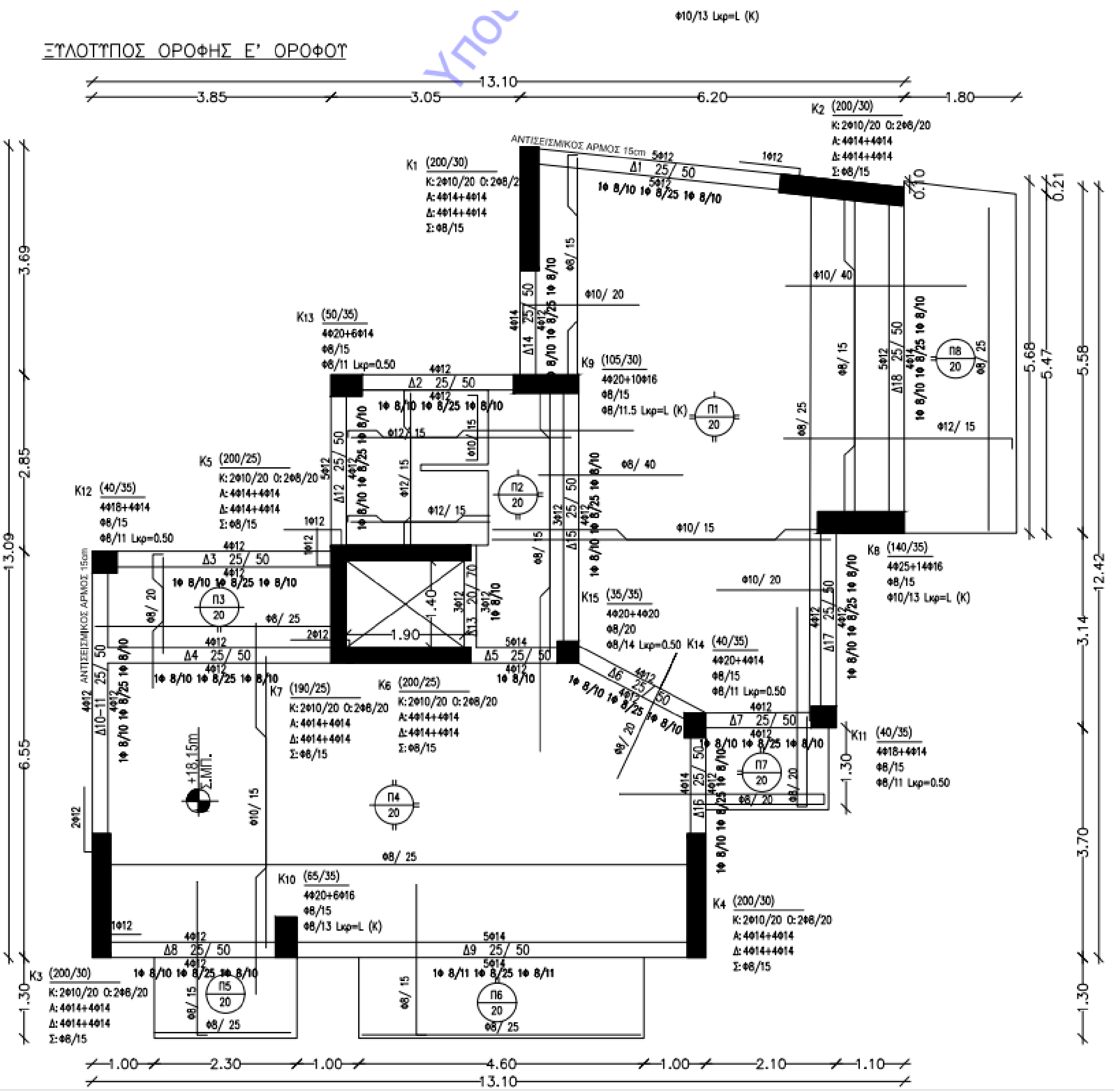


Figure B.14 Sixth floor formwork

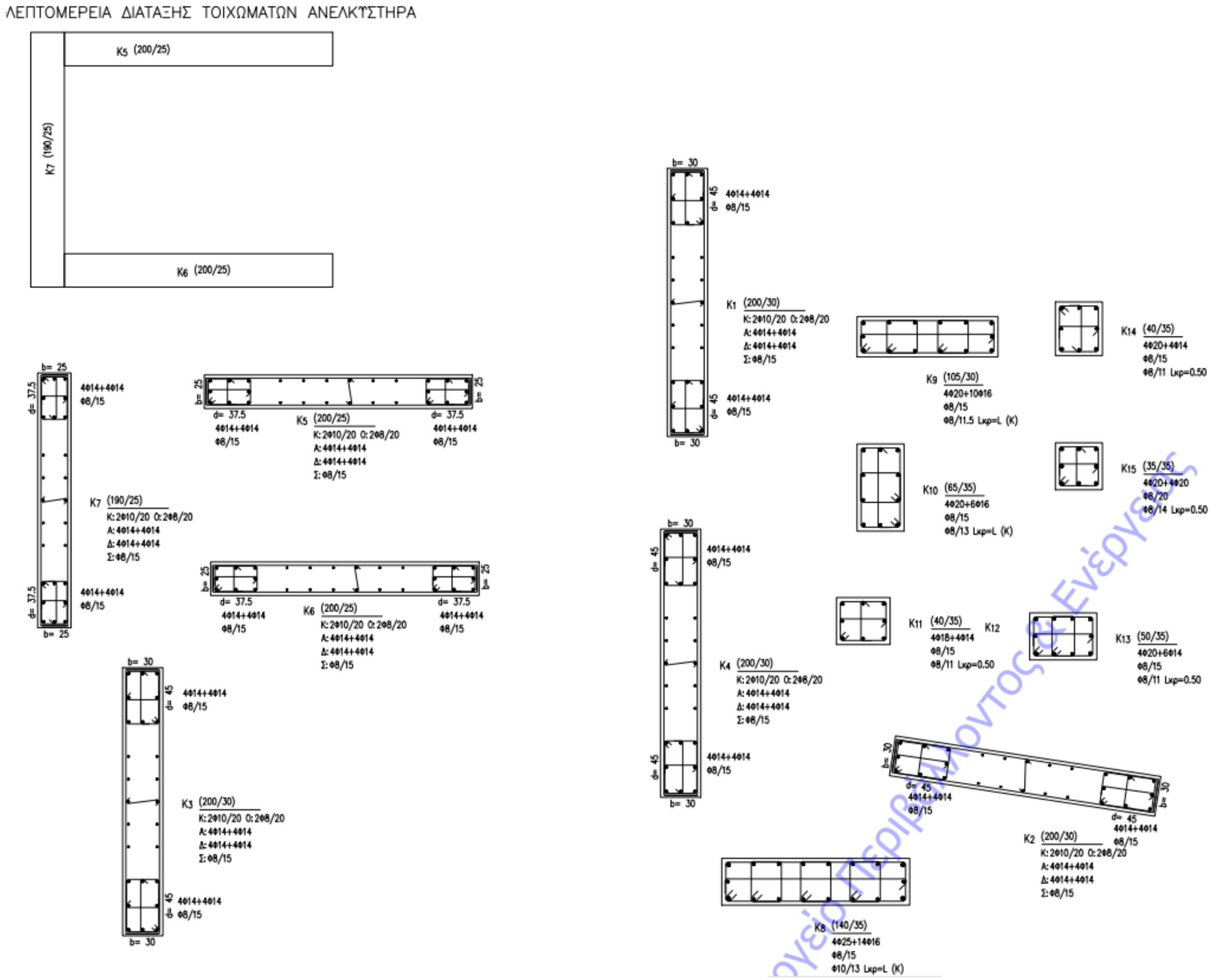


Figure B.15 Reinforcement detailing – Sixth floor

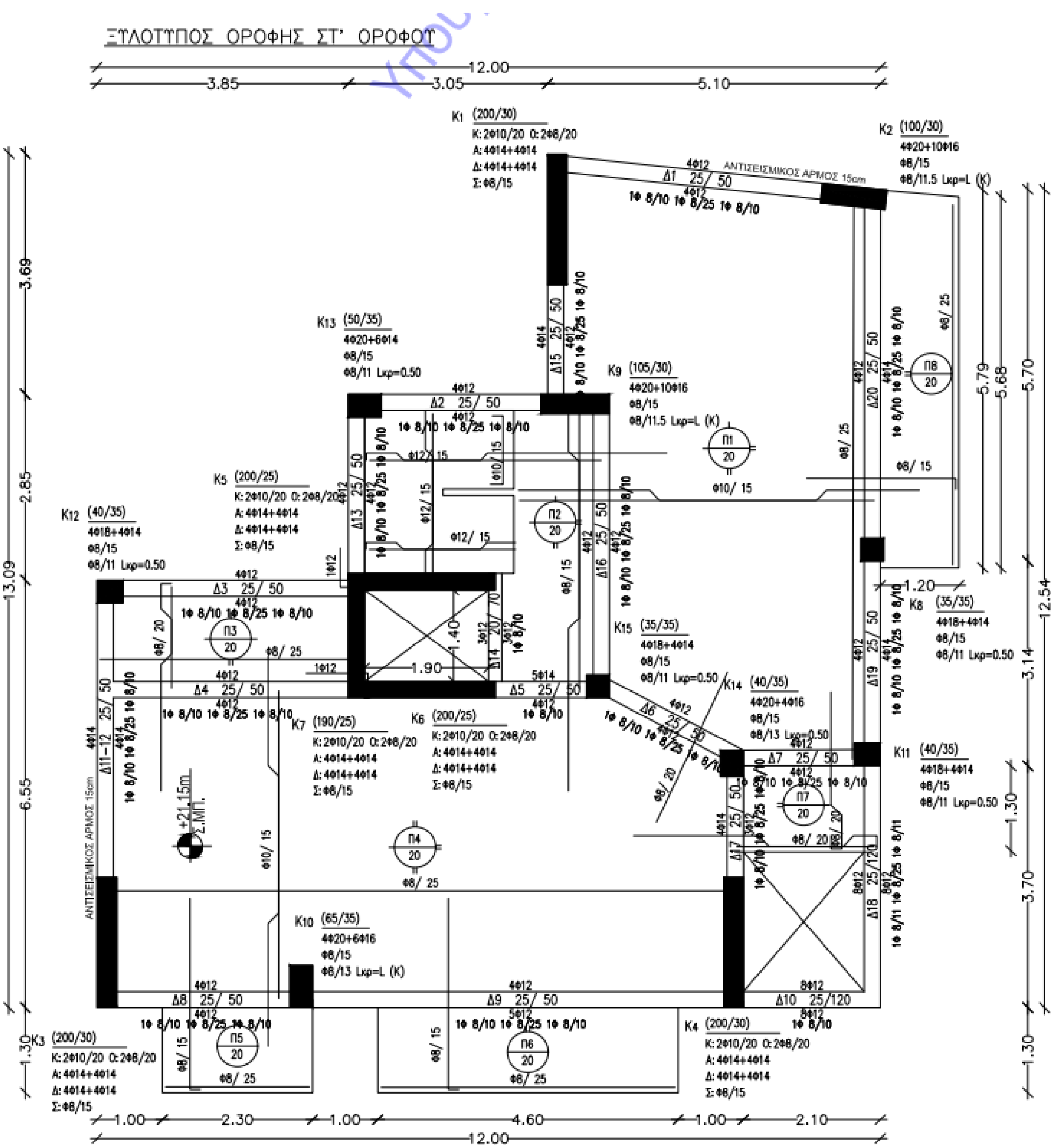


Figure B.16 Seventh floor formwork

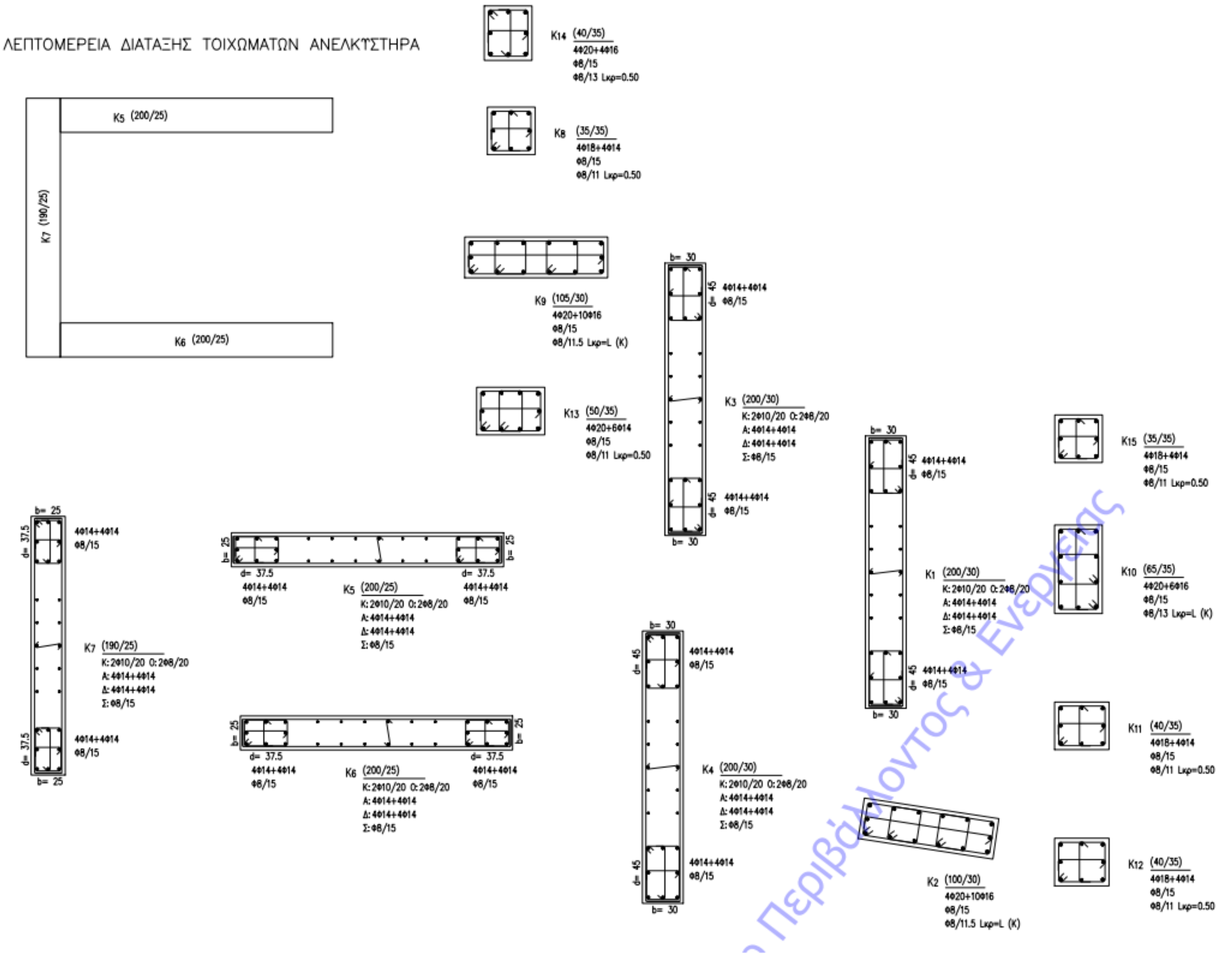


Figure B.17 Reinforcement detailing – Seventh floor

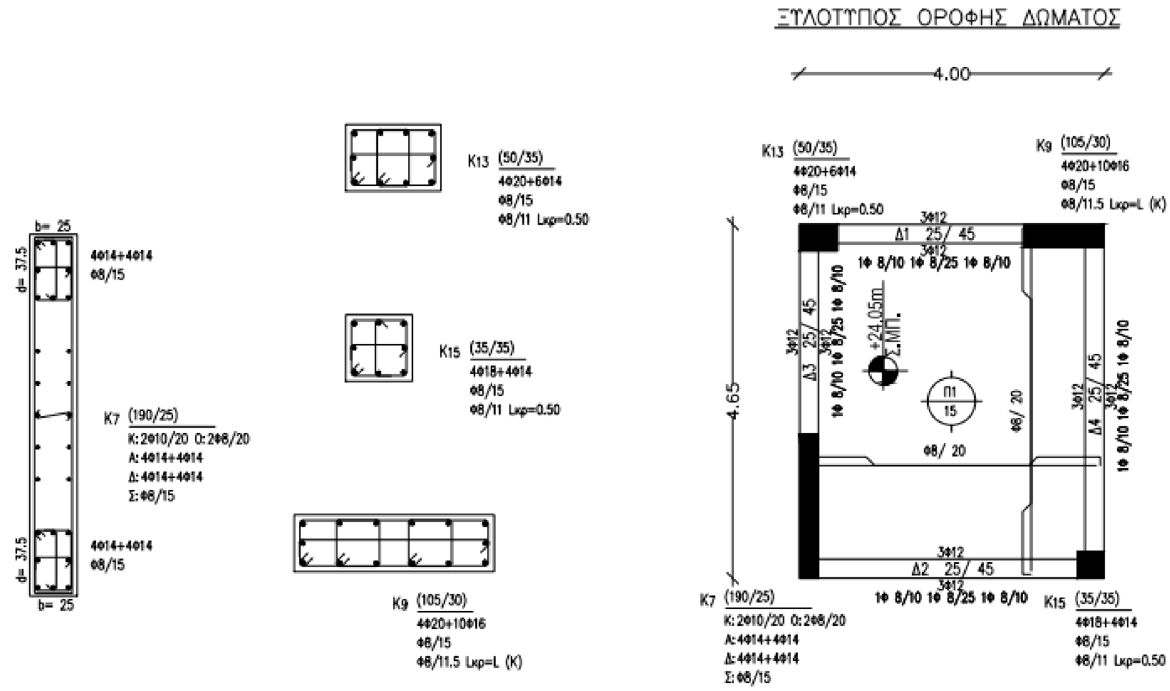


Figure B.18 Roof formwork and Reinforcement detailing

Appendix C

(Calculation of elements strength – ASCE/SEI 41-17)

Table C.1 Flexural and Shear strength of vertical elements – 1st floor

Member	X-Direction		Y-Direction	
	M_y (kNm)	**V_{Ri} (kN)**	**M_y (kNm)**	**V_{Ri} (kN)**
wall W1	663.26	500.57	5971.21	564.65
wall W2	6062.52	668.05	782.15	590.30
wall W3	674.63	509.16	5818.20	550.18
wall W4	860.83	649.69	7401.47	699.90
wall W5	8264.84	781.55	1154.52	871.33
wall W6	7729.34	730.91	1097.63	828.40
wall W7	993.12	749.52	5408.21	511.41
column C8	9245.01	1557.87	1680.84	1044.83
column C9	4412.92	1301.72	894.36	674.99
column C10	932.72	701.42	2100.94	1062.45
column C11	653.10	451.11	547.94	408.27
column C12	669.76	487.61	561.81	424.01
column C13	1097.95	520.66	688.73	502.38
column C14	547.15	408.11	652.15	450.95
column C15	551.36	388.60	551.36	388.60

The shear strength, V_{Ri}, does not exceed the shear corresponding to development of flexural strengths at the top and bottom of walls and columns.

In addition, the flexural strength of beam-column connections must be examined to evaluate all four (4) plastic mechanisms. To do this, the flexural strength of all beams must first be determined and then the moment at the joint according to Eq. (1.50) be calculated.

Table C.2 Flexural strength of joints – 1st floor

Node	X-Direction				Y-Direction			
	ΣM_{Rb} (kNm)	**ΣM_{Rc} (kNm)**	**M_{njx} (kNm)**	**Ratio**	**ΣM_{Rb} (kNm)**	**ΣM_{Rc} (kNm)**	**M_{njx} (kNm)**	**Ratio**
1	569.36	1301.61	569.36	2.29	303.36	11189.56	303.36	36.89
2	569.36	11742.97	569.36	20.62	775.76	1518.83	775.76	1.96
3	398.94	1319.28	398.94	3.31	423.15	11343.16	423.15	26.81
4	724.09	1609.39	724.09	2.22	230.94	13907.21	230.94	60.22
5	502.67	14966.16	502.67	29.77	641.28	1720.87	641.28	2.68
6	716.45	14030.26	716.45	19.58	288.29	1678.12	288.29	5.82
7	1031.47	1454.06	1031.47	1.41	352.99	9338.70	352.99	26.46
8	0.00	4220.25	0.00		1118.61	2673.89	1118.61	2.39
9	359.53	3336.45	359.53	9.28	657.78	1750.41	657.78	2.66
10	1242.67	1775.30	1242.67	1.43	0.00	2559.71	0.00	
11	249.95	1002.17	249.95	4.01	409.63	911.14	409.63	2.22
12	502.67	1052.74	502.67	2.09	195.07	934.19	195.07	4.79
13	359.53	1197.56	359.53	3.33	352.99	1135.11	352.99	3.22
14	509.39	909.51	509.39	1.79	367.73	1000.54	367.73	2.72
15	447.10	1047.18	447.10	2.34	712.93	1047.18	712.93	1.47

Table C.3 Flexural and Shear strength of vertical elements – 2nd floor

Member	X-Direction		Y-Direction	
	M_y (kNm)	V_{Ri} (kN)	M_y (kNm)	V_{Ri} (kN)
wall W1	638.35	510.68	5218.35	579.82
wall W2	5680.45	757.39	736.68	589.34
wall W3	644.65	515.72	5524.95	613.88
wall W4	748.55	598.84	6505.74	722.86
wall W5	6701.32	744.59	566.36	453.08
wall W6	6300.93	700.10	580.49	464.39
wall W7	460.94	368.75	3930.49	436.72
column C8	8985.80	1724.86	1617.85	1031.59
column C9	4262.33	1289.33	856.05	684.84
column C10	892.16	676.73	2049.56	921.57
column C11	657.90	323.56	551.09	296.14
column C12	668.55	325.32	559.95	297.90
column C13	1070.58	406.15	672.03	375.56
column C14	545.48	295.01	651.16	322.43
column C15	822.49	425.84	822.49	425.84

Table C.4 Flexural strength of joints – 2nd floor

Node	X-Direction				Y-Direction			
	ΣM_{Rb} (kNm)	**ΣM_{Rc} (kNm)**	**M_{njx} (kNm)**	**Ratio**	**ΣM_{Rb} (kNm)**	**ΣM_{Rc} (kNm)**	**M_{njx} (kNm)**	**Ratio**
1	570.15	1222.96	570.15	2.14	303.70	10667.37	303.70	35.13
2	570.15	10962.76	570.15	19.23	733.76	1427.21	733.76	1.95
3	398.05	1256.31	398.05	3.16	422.31	10727.72	422.31	25.40
4	898.84	1445.25	898.84	1.61	217.25	12533.71	217.25	57.69
5	504.21	12454.24	504.21	24.70	641.44	1073.09	641.44	1.67
6	704.71	12222.21	704.71	17.34	288.29	1098.11	288.29	3.81
7	1032.33	866.87	866.87	0.84	353.15	7681.98	353.15	21.75
8	0.00	4278.07	0.00		1076.01	2561.94	1076.01	2.38
9	452.09	3237.31	452.09	7.16	680.45	1677.78	680.45	2.47
10	1299.71	1492.91	1299.71	1.15	0.00	2080.15	0.00	
11	249.64	803.17	249.64	3.22	409.12	734.63	409.12	1.80
12	504.21	807.05	504.21	1.60	195.08	738.50	195.08	3.79
13	224.71	1012.66	224.71	4.51	353.15	934.36	353.15	2.65
14	508.99	731.89	508.99	1.44	353.99	800.43	353.99	2.26
15	435.95	931.73	435.95	2.14	580.37	931.73	580.37	1.61

Table C.5 Flexural and Shear strength of vertical elements – 3rd floor

Member	X-Direction		Y-Direction	
	M_y (kNm)	V_{Ri} (kN)	M_y (kNm)	V_{Ri} (kN)
wall W1	584.61	467.69	4938.93	658.52
wall W2	5282.31	880.38	690.53	552.42
wall W3	611.66	489.33	5202.77	693.70
wall W4	696.70	557.36	6027.97	803.73
wall W5	5752.92	767.06	506.74	405.39
wall W6	5921.28	789.50	517.62	414.09
wall W7	405.93	324.74	3751.50	500.20
column C8	8722.45	1697.60	1555.07	1017.96
column C9	4124.42	1300.51	821.74	657.39
column C10	874.87	517.59	2047.61	742.55
column C11	630.99	318.98	528.71	291.56
column C12	638.74	320.32	535.15	292.90
column C13	1043.64	403.98	655.61	371.93
column C14	523.63	290.50	624.88	317.92
column C15	1003.12	319.55	1003.12	319.55

Table C.6 Flexural strength of joints – 3rd floor

Node	X-Direction				Y-Direction			
	ΣM_{Rb} (kNm)	ΣM_{Rc} (kNm)	M_{njx} (kNm)	Ratio	ΣM_{Rb} (kNm)	ΣM_{Rc} (kNm)	M_{njx} (kNm)	Ratio
1	569.96	1139.37	569.96	2.00	304.10	9587.07	304.10	31.53
2	569.96	10163.46	569.96	17.83	733.26	1334.48	733.26	1.82
3	397.89	1188.19	397.89	2.99	422.13	10062.96	422.13	23.84
4	898.84	1339.61	898.84	1.49	217.52	11535.96	217.52	53.03
5	504.63	11160.64	504.63	22.12	641.39	991.05	641.39	1.55
6	704.23	11468.58	704.23	16.29	288.29	1010.95	288.29	3.51
7	1032.02	797.02	797.02	0.77	353.10	7338.81	353.10	20.78
8	0.00	4208.36	0.00		1075.27	2527.09	1075.27	2.35
9	360.29	2846.73	360.29	7.90	680.88	1494.73	680.88	2.20
10	1297.50	1128.89	1128.89	0.87	0.00	1609.01	0.00	
11	249.45	791.54	249.45	3.17	408.86	723.00	408.86	1.77
12	504.63	794.76	504.63	1.57	195.12	726.22	195.12	3.72
13	242.95	1001.32	242.95	4.12	353.10	925.29	353.10	2.62
14	508.73	720.89	508.73	1.42	354.22	789.44	354.22	2.23
15	436.12	795.21	436.12	1.82	580.33	795.21	580.33	1.37

Table C.7 Flexural and Shear strength of vertical elements – 4th floor

Member	X-Direction		Y-Direction	
	M_y (kNm)	V_{Ri} (kN)	M_y (kNm)	V_{Ri} (kN)
wall W1	554.76	443.81	4648.14	774.69
wall W2	4881.16	1084.70	643.95	515.16
wall W3	576.53	461.22	4860.19	810.03
wall W4	642.91	514.33	5507.99	918.00
wall W5	5407.72	901.29	484.31	387.45
wall W6	5547.30	924.55	493.33	394.67
wall W7	391.09	312.87	3587.31	597.89
column C8	8451.61	1669.09	1491.70	1003.71
column C9	4166.35	976.87	821.82	538.40
column C10	852.90	385.53	2028.22	544.66
column C11	604.28	314.25	506.49	286.83
column C12	611.19	315.49	512.24	288.08
column C13	1017.33	397.07	639.58	368.30
column C14	503.66	286.21	600.87	313.63
column C15	787.73	316.62	787.73	316.62

Table C.8 Flexural strength of joints – 4th floor

Node	X-Direction				Y-Direction			
	ΣM_{Rb} (kNm)	**ΣM_{Rc} (kNm)**	**M_{njx} (kNm)**	**Ratio**	**ΣM_{Rb} (kNm)**	**ΣM_{Rc} (kNm)**	**M_{njx} (kNm)**	**Ratio**
1	569.86	1077.96	569.86	1.89	304.39	8989.35	304.39	29.53
2	569.86	9286.16	569.86	16.30	733.30	1240.90	733.30	1.69
3	397.77	1116.12	397.77	2.81	422.08	9360.71	422.08	22.18
4	724.53	1230.51	724.53	1.70	217.71	10476.06	217.71	48.12
5	505.07	10450.36	505.07	20.69	641.38	944.89	641.38	1.47
6	704.17	10698.79	704.17	15.19	288.29	960.95	288.29	3.33
7	1032.01	768.28	768.28	0.74	353.08	7020.81	353.08	19.88
8	0.00	4135.42	0.00		1075.06	2490.62	1075.06	2.32
9	453.07	2442.40	453.07	5.39	810.56	1335.06	810.56	1.65
10	1301.11	950.81	950.81	0.73	0.00	1355.97	0.00	
11	249.35	779.46	249.35	3.13	408.60	710.91	408.60	1.74
12	505.07	782.79	505.07	1.55	195.15	714.24	195.15	3.66
13	224.71	984.43	224.71	4.38	353.08	916.13	353.08	2.59
14	508.44	710.20	508.44	1.40	354.32	778.75	354.32	2.20
15	436.32	810.53	436.32	1.86	580.23	810.53	580.23	1.40

Table C.9 Flexural and Shear strength of vertical elements – 5th floor

Member	X-Direction		Y-Direction	
	M_y (kNm)	**V_{Ri} (kN)**	**M_y (kNm)**	**V_{Ri} (kN)**
wall W1	523.20	418.56	4341.21	964.71
wall W2	4477.04	1468.34	596.95	477.56
wall W3	539.59	431.67	4500.51	1000.11
wall W4	587.60	470.08	4968.08	1104.02
wall W5	5042.63	1120.59	460.58	368.46
wall W6	5151.49	1144.78	467.61	374.09
wall W7	377.19	301.75	3433.50	763.00
column C8	8174.06	1639.24	1427.90	988.78
column C9	4040.28	977.05	791.49	529.65
column C10	802.37	375.13	1941.03	540.11
column C11	577.65	309.31	484.34	281.90
column C12	585.20	310.74	490.61	283.32
column C13	991.41	390.47	623.79	364.61
column C14	484.56	281.95	577.93	309.37
column C15	725.67	331.81	725.67	331.81

Table C.10 Flexural strength of joints – 5th floor

Node	X-Direction				Y-Direction			
	ΣM_{Rb} (kNm)	**ΣM_{Rc} (kNm)**	**M_{njx} (kNm)**	**Ratio**	**ΣM_{Rb} (kNm)**	**ΣM_{Rc} (kNm)**	**M_{njx} (kNm)**	**Ratio**
1	569.81	1013.29	569.81	1.78	304.70	8360.82	304.70	27.44
2	569.81	6607.51	569.81	11.60	778.86	1146.52	778.86	1.47
3	397.55	1040.61	397.55	2.62	584.68	8626.26	584.68	14.75
4	898.19	1119.00	898.19	1.25	231.95	9389.02	231.95	40.48
5	505.33	9569.92	505.33	18.94	641.35	898.41	641.35	1.40
6	716.20	9679.44	716.20	13.52	288.29	909.54	288.29	3.15
7	1032.15	737.30	737.30	0.71	353.06	6678.29	353.06	18.92
8	0.00	4059.17	0.00		1120.93	2452.49	1120.93	2.19
9	360.21	2421.48	360.21	6.72	809.88	1313.56	809.88	1.62
10	1295.75	924.50	924.50	0.71	0.00	1272.72	0.00	
11	249.28	766.42	249.28	3.07	408.90	697.87	408.90	1.71
12	505.33	770.80	505.33	1.53	270.11	702.26	270.11	2.60
13	360.21	972.88	360.21	2.70	353.06	906.73	353.06	2.57
14	508.03	699.37	508.03	1.38	368.38	767.91	368.38	2.08
15	448.13	697.81	448.13	1.56	580.05	697.81	580.05	1.20

Table C.11 Flexural and Shear strength of vertical elements – 6th floor

Member	X-Direction		Y-Direction	
	M_y (kNm)	V_{Ri} (kN)	M_y (kNm)	V_{Ri} (kN)
wall W1	490.09	392.07	4019.61	1339.87
wall W2	4070.31	1468.34	549.57	439.65
wall W3	501.02	400.82	4125.75	1375.25
wall W4	531.40	425.12	4420.94	1473.65
wall W5	4692.88	1509.10	437.83	350.27
wall W6	4756.45	1509.32	441.93	353.54
wall W7	360.11	288.09	3244.79	992.68
column C8	7891.80	1608.09	1364.13	973.21
column C9	3525.69	960.14	763.26	521.19
column C10	753.53	364.47	1853.92	478.07
column C11	549.50	303.82	460.91	276.40
column C12	560.01	305.91	469.65	278.49
column C13	965.32	387.83	607.92	360.78
column C14	526.65	277.54	628.65	304.96
column C15	672.15	226.44	672.15	226.44

Table C.12 Flexural strength of joints – 6th floor

Node	X-Direction				Y-Direction			
	ΣM$_{Rb}$ (kNm)	**ΣM$_{Rc}$ (kNm)**	**M$_{njx}$ (kNm)**	**Ratio**	**ΣM$_{Rb}$ (kNm)**	**ΣM$_{Rc}$ (kNm)**	**M$_{njx}$ (kNm)**	**Ratio**
1	569.12	945.40	569.12	1.66	320.27	6237.57	320.27	19.48
2	569.12	2202.50	569.12	3.87	775.69	1649.92	775.69	2.13
3	398.48	961.73	398.48	2.41	421.99	6343.71	421.99	15.03
4	603.48	1014.47	603.48	1.68	232.29	6638.90	232.29	28.58
5	506.13	6790.93	506.13	13.42	641.34	850.39	641.34	1.33
6	717.02	6791.93	717.02	9.47	288.29	857.40	288.29	2.97
7	1033.01	703.12	703.12	0.68	353.05	6173.21	353.05	17.49
8	0.00	2350.17	0.00		860.51	1556.57	860.51	1.81
9	360.30	2379.73	360.30	6.60	824.96	1292.68	824.96	1.57
10	1222.09	896.85	896.85	0.73	0.00	1283.79	0.00	
11	249.78	752.55	249.78	3.01	411.92	684.00	411.92	1.66
12	506.13	765.73	506.13	1.51	195.14	689.80	195.14	3.53
13	360.30	960.61	360.30	2.67	353.05	896.92	353.05	2.54
14	507.52	651.39	507.52	1.28	368.18	714.66	368.18	1.94
15	447.89	615.78	447.89	1.37	617.66	615.78	615.78	1.00

Table C.13 Flexural and Shear strength of vertical elements – 7th floor

Member	X-Direction		Y-Direction	
	M_y (kNm)	V_{Ri} (kN)	M_y (kNm)	V_{Ri} (kN)
wall W1	455.31	364.25	3682.36	1478.64
column C2	3197.16	880.28	769.95	510.97
wall W3	460.71	368.57	3734.65	1478.64
wall W4	483.07	386.45	3951.44	1478.64
wall W5	4304.43	1509.10	412.55	330.04
wall W6	4349.75	1509.32	415.47	332.37
wall W7	343.00	274.40	3055.88	992.68
column C8	478.07	272.05	478.07	272.05
column C9	3395.18	943.65	736.79	512.95
column C10	704.34	353.01	1525.83	548.97
column C11	522.48	298.22	438.43	270.80
column C12	534.58	306.68	448.49	273.35
column C13	938.85	380.66	591.82	356.76
column C14	555.16	243.56	664.79	266.76
column C15	453.01	266.18	453.01	266.18

Table C.14 Flexural strength of joints – 7th floor

Node	X-Direction				Y-Direction			
	ΣM_{Rb} (kNm)	**ΣM_{Rc} (kNm)**	**M_{njx} (kNm)**	**Ratio**	**ΣM_{Rb} (kNm)**	**ΣM_{Rc} (kNm)**	**M_{njx} (kNm)**	**Ratio**
1	566.43	455.31	455.31	0.80	321.48	2217.96	321.48	6.90
2	566.43	1100.36	566.43	1.94	1016.30	638.72	638.72	0.63
3	398.12	460.71	398.12	1.16	441.85	2217.96	441.85	5.02
4	724.53	483.07	483.07	0.67	234.30	2217.96	234.30	9.47
5	500.79	2263.64	500.79	4.52	641.28	412.55	412.55	0.64
6	718.37	2263.98	718.37	3.15	288.29	415.47	288.29	1.44
7	1026.37	671.17	671.17	0.65	352.98	4399.58	352.98	12.46
8	0.00	340.06	0.00		721.11	340.06	340.06	0.47
9	360.89	2258.97	360.89	6.26	698.67	1259.75	698.67	1.80
10	1301.46	441.26	441.26	0.34	0.00	686.21	0.00	
11	248.88	372.77	248.88	1.50	415.19	338.50	338.50	0.82
12	500.79	383.35	383.35	0.77	195.02	341.69	195.02	1.75
13	360.89	969.26	360.89	2.69	352.98	878.76	352.98	2.49
14	506.22	304.45	304.45	0.60	369.98	333.45	333.45	0.90
15	450.12	654.05	450.12	1.45	579.30	654.05	579.30	1.13

Appendix D

(Eigenvalue analysis of the building)

Table D.1 Eigenperiod and Effective Mass – Without Infills/Uncracked sections

Mode	Period (sec)	[Ux]	[Uy]	[Uz]	[Rx]	[Ry]	[Rz]
1	0.52924517	44.73%	8.46%	0.00%	3.54%	22.59%	14.98%
2	0.51029244	14.35%	49.75%	0.00%	24.88%	7.74%	2.40%
3	0.45295849	6.81%	8.35%	0.00%	5.29%	3.91%	52.34%
4	0.15541765	3.20%	3.46%	0.00%	4.81%	4.28%	5.22%
5	0.13582919	8.53%	5.52%	0.00%	6.70%	9.98%	0.05%
6	0.12456866	1.88%	4.59%	0.00%	5.42%	2.19%	6.29%
7	0.09363618	0.53%	0.39%	0.01%	0.23%	0.45%	0.00%
8	0.07617835	0.40%	1.03%	0.01%	0.94%	0.26%	2.95%
9	0.06958984	0.00%	0.00%	19.65%	3.95%	4.23%	0.00%
10	0.0652388	2.57%	1.11%	0.02%	0.61%	0.76%	0.00%
11	0.06268512	0.32%	0.00%	15.04%	0.06%	1.24%	0.01%
12	0.05984097	0.16%	0.00%	22.25%	0.59%	6.99%	0.00%
13	0.05782277	0.35%	2.71%	0.56%	1.34%	0.02%	1.36%
14	0.05473224	0.03%	0.10%	10.58%	5.34%	0.04%	0.03%
15	0.05222837	0.07%	0.01%	4.20%	6.10%	4.84%	0.03%
16	0.05000359	1.64%	0.06%	0.20%	0.42%	1.25%	0.30%
17	0.04906224	0.03%	0.03%	0.69%	0.09%	0.25%	0.04%
18	0.04789203	0.03%	0.13%	0.03%	0.12%	0.02%	0.01%
19	0.04782597	0.00%	0.01%	0.07%	0.03%	0.00%	0.00%
20	0.04769197	0.00%	0.01%	0.00%	0.02%	0.00%	0.00%
21	0.04766486	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%
22	0.04717258	0.04%	0.15%	0.00%	0.16%	0.03%	0.03%
23	0.04632706	0.24%	0.12%	0.17%	0.16%	0.07%	1.23%
24	0.04508217	0.03%	0.02%	1.65%	0.34%	0.20%	0.01%
25	0.04495211	0.04%	0.24%	0.00%	0.81%	0.18%	0.54%
26	0.04263392	0.01%	0.07%	2.73%	0.12%	0.12%	0.00%
27	0.04020442	0.00%	0.00%	0.07%	0.01%	0.04%	0.00%
28	0.03953347	0.00%	0.01%	0.20%	0.03%	0.12%	0.00%
29	0.03923931	0.00%	0.00%	0.44%	0.05%	0.22%	0.00%
30	0.03851019	0.01%	0.11%	0.13%	0.07%	0.10%	0.00%
31	0.03806886	0.02%	0.05%	0.66%	0.02%	0.75%	0.04%
32	0.03777384	0.01%	0.02%	1.30%	0.91%	0.80%	0.01%
33	0.03608863	0.00%	0.24%	0.09%	0.41%	0.01%	0.05%

34	0.03599893	0.00%	0.02%	0.41%	0.09%	0.03%	0.00%
35	0.03501124	0.18%	1.26%	0.00%	1.43%	0.25%	0.29%
36	0.03432181	0.00%	0.02%	0.18%	0.10%	0.02%	0.00%
37	0.03361511	1.81%	0.07%	0.04%	0.04%	1.93%	0.12%
38	0.03169968	0.27%	0.02%	0.00%	0.03%	0.59%	0.15%
39	0.03141433	0.03%	0.13%	0.02%	0.44%	0.04%	0.01%
40	0.03077817	0.02%	0.34%	0.01%	0.34%	0.04%	0.80%
41	0.03046149	0.00%	0.01%	0.00%	0.00%	0.01%	0.01%
42	0.02961797	0.00%	0.02%	0.00%	0.06%	0.00%	0.10%
43	0.02938954	0.00%	0.00%	0.02%	0.00%	0.00%	0.05%
44	0.02933336	0.00%	0.00%	0.01%	0.00%	0.00%	0.03%
45	0.02932125	0.00%	0.03%	0.02%	0.03%	0.00%	0.02%
46	0.02918065	0.00%	0.01%	0.01%	0.01%	0.00%	0.03%
47	0.02907466	0.00%	0.01%	0.03%	0.05%	0.00%	0.01%
48	0.02891036	0.00%	0.00%	0.00%	0.03%	0.02%	0.01%
49	0.02778007	0.02%	0.22%	0.01%	0.52%	0.01%	0.28%
50	0.02757229	0.06%	0.07%	1.10%	0.02%	0.06%	0.01%
51	0.0272293	0.06%	0.03%	0.35%	0.45%	0.00%	0.00%
52	0.02705475	0.11%	0.01%	0.57%	0.00%	0.36%	0.04%
53	0.02661959	0.03%	0.13%	0.05%	0.21%	0.05%	0.04%
54	0.02613473	0.01%	0.00%	0.75%	0.16%	0.09%	0.04%
55	0.02581983	0.00%	0.00%	0.12%	0.00%	0.11%	0.01%
56	0.02561839	0.00%	0.00%	0.07%	0.02%	0.00%	0.04%
57	0.02536532	0.06%	0.00%	0.00%	0.00%	0.13%	0.00%
58	0.02408551	0.02%	0.00%	0.13%	0.00%	0.00%	0.01%
59	0.02362436	0.50%	0.14%	0.01%	0.15%	0.71%	0.03%
60	0.02304104	0.22%	0.51%	0.01%	0.73%	0.30%	0.06%
61	0.02278819	0.00%	0.00%	0.16%	0.00%	0.01%	0.04%
62	0.02271678	0.01%	0.01%	0.71%	0.02%	0.14%	0.02%
63	0.02254669	0.06%	0.03%	0.00%	0.03%	0.11%	0.00%
64	0.02250226	0.01%	0.01%	0.06%	0.02%	0.12%	0.04%
65	0.02246396	0.02%	0.02%	0.49%	0.02%	0.21%	0.02%
66	0.0222953	0.01%	0.00%	0.02%	0.02%	0.06%	0.00%
67	0.0219	0.03%	0.01%	0.30%	0.00%	0.10%	0.02%
68	0.02174652	0.23%	0.41%	0.03%	0.50%	0.18%	0.02%
69	0.02069719	0.11%	0.25%	0.06%	0.74%	0.15%	0.44%
70	0.02040778	0.10%	0.02%	0.00%	0.07%	0.12%	0.15%
Results	**SUM**	**90.05%**	**90.56%**	**86.54%**	**79.92%**	**79.65%**	**90.88%**

Table D.2 Eigenperiod and Effective Mass – Without Infills/Cracked sections

Mode	**Period (sec)**	**[Ux]**	**[Uy]**	**[Uz]**	**[Rx]**	**[Ry]**	**[Rz]**
1	0.92854362	23.47%	23.24%	0.00%	10.33%	11.11%	23.14%
2	0.89586904	34.13%	32.68%	0.00%	16.42%	17.26%	0.03%
3	0.79131441	9.40%	11.42%	0.00%	6.66%	5.13%	47.43%
4	0.2812713	2.41%	3.87%	0.00%	5.79%	3.69%	5.31%
5	0.24610574	8.20%	5.36%	0.01%	7.08%	10.78%	0.00%
6	0.22665363	2.37%	3.85%	0.00%	5.51%	3.39%	5.78%
7	0.15945558	0.54%	0.52%	0.01%	0.35%	0.42%	0.07%
8	0.1408804	0.25%	0.81%	0.02%	0.67%	0.17%	2.79%
9	0.12006819	2.75%	1.30%	0.70%	0.84%	1.54%	0.01%
10	0.11912851	0.03%	0.01%	22.33%	4.11%	3.55%	0.01%
11	0.10877891	0.28%	1.67%	24.83%	0.18%	0.44%	0.69%
12	0.10743874	0.44%	0.69%	20.11%	0.72%	6.77%	0.49%
13	0.10250178	0.03%	0.13%	6.97%	3.76%	2.35%	0.25%
14	0.10090199	0.05%	0.07%	0.37%	8.33%	3.21%	0.00%
15	0.09578431	0.07%	0.00%	2.07%	0.23%	0.07%	0.00%
16	0.09267932	0.70%	0.01%	0.05%	0.00%	0.54%	0.02%
17	0.09134213	0.54%	0.22%	0.94%	1.07%	1.60%	0.39%
18	0.08704088	0.35%	0.30%	0.02%	0.17%	0.31%	1.45%
19	0.07833619	0.01%	0.13%	1.00%	0.34%	0.04%	0.00%
20	0.07479648	0.00%	0.05%	0.18%	0.13%	0.24%	0.00%
21	0.0702968	0.01%	0.06%	1.19%	1.03%	0.52%	0.01%
22	0.06837908	0.06%	1.47%	0.05%	1.09%	0.21%	0.32%
23	0.06642376	0.00%	0.00%	0.05%	0.00%	0.03%	0.00%
24	0.0649804	0.86%	0.06%	0.01%	0.03%	0.55%	0.07%
25	0.06460298	0.02%	0.00%	0.02%	0.00%	0.00%	0.05%
26	0.06411739	0.76%	0.00%	0.37%	0.01%	1.48%	0.02%
27	0.06306189	0.29%	0.01%	0.00%	0.10%	0.57%	0.00%
28	0.06298107	0.00%	0.01%	0.01%	0.00%	0.01%	0.13%
29	0.06180855	0.10%	0.01%	0.13%	0.03%	0.01%	0.00%
30	0.05968422	0.00%	0.50%	0.02%	0.74%	0.00%	0.67%
31	0.05808719	0.13%	0.09%	0.18%	0.29%	0.28%	0.24%
32	0.05704094	0.00%	0.00%	0.00%	0.01%	0.00%	0.00%
33	0.05685436	0.02%	0.01%	0.00%	0.12%	0.11%	0.22%
34	0.05483179	0.00%	0.01%	0.01%	0.00%	0.00%	0.06%
35	0.05427208	0.00%	0.00%	0.02%	0.04%	0.02%	0.02%
36	0.0537523	0.00%	0.04%	0.40%	0.00%	0.00%	0.01%
37	0.05357965	0.00%	0.03%	0.01%	0.06%	0.00%	0.02%
38	0.05325773	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%
39	0.05279405	0.06%	0.03%	0.00%	0.07%	0.07%	0.04%
40	0.05128656	0.07%	0.28%	0.22%	0.23%	0.08%	0.02%

41	0.05100148	0.06%	0.07%	0.01%	0.19%	0.08%	0.08%
42	0.05081259	0.07%	0.00%	0.22%	0.04%	0.11%	0.07%
43	0.04962905	0.00%	0.01%	0.24%	0.08%	0.01%	0.02%
44	0.04877374	0.00%	0.00%	0.14%	0.05%	0.09%	0.00%
45	0.04667948	0.00%	0.01%	0.01%	0.05%	0.01%	0.03%
46	0.0452708	0.65%	0.04%	0.01%	0.06%	0.84%	0.14%
47	0.04475634	0.01%	0.01%	1.07%	0.54%	0.08%	0.01%
48	0.04408247	0.27%	0.48%	0.00%	0.42%	0.39%	0.03%
49	0.04374514	0.02%	0.42%	0.03%	0.62%	0.03%	0.00%
50	0.04134937	0.00%	0.00%	0.08%	0.00%	0.09%	0.00%
51	0.0407814	0.36%	0.20%	0.03%	0.38%	0.85%	0.73%
52	0.03931719	0.00%	0.00%	1.97%	0.29%	0.04%	0.00%
53	0.03892446	0.05%	0.00%	0.52%	0.00%	0.15%	0.07%
54	0.0375558	0.11%	0.08%	0.03%	0.00%	0.01%	0.03%
55	0.03685019	0.00%	0.00%	0.09%	0.43%	0.14%	0.05%
56	0.03646733	0.03%	0.01%	0.77%	0.04%	0.04%	0.00%
57	0.03602725	0.00%	0.00%	0.73%	0.09%	0.02%	0.00%
58	0.03435737	0.00%	0.03%	0.16%	0.16%	0.02%	0.01%
59	0.03408431	0.13%	0.30%	0.01%	0.13%	0.07%	0.01%
60	0.03387276	0.06%	0.14%	0.42%	0.54%	0.34%	0.00%
61	0.03273348	0.00%	0.02%	0.08%	0.02%	0.00%	0.00%
62	0.03172847	0.00%	0.00%	0.02%	0.00%	0.05%	0.00%
63	0.03119391	0.39%	0.01%	0.01%	0.02%	0.69%	0.21%
64	0.03074692	0.01%	0.03%	0.01%	0.04%	0.01%	0.03%
65	0.0305107	0.01%	0.01%	0.14%	0.04%	0.01%	0.01%
66	0.03003794	0.00%	0.00%	0.00%	0.01%	0.00%	0.00%
67	0.02992704	0.01%	0.00%	0.01%	0.01%	0.01%	0.01%
68	0.02949239	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%
69	0.02895349	0.08%	0.13%	0.11%	0.21%	0.12%	0.13%
70	0.02829344	0.00%	0.00%	0.57%	0.33%	0.27%	0.00%
Results	**SUM**	**90.75%**	**90.95%**	**89.78%**	**81.33%**	**81.13%**	**91.48%**

Table D.3 Eigenperiod and Effective Mass – With Infills/Uncracked sections

Mode	**Period (sec)**	**[Ux]**	**[Uy]**	**[Uz]**	**[Rx]**	**[Ry]**	**[Rz]**
1	0.43734175	59.91%	3.16%	0.00%	1.85%	27.27%	6.72%
2	0.41779069	1.64%	64.62%	0.00%	27.68%	0.58%	4.82%
3	0.38364554	7.56%	2.11%	0.00%	1.58%	3.95%	61.75%
4	0.1318624	5.48%	1.36%	0.00%	3.24%	9.92%	3.62%
5	0.12222098	4.59%	7.63%	0.00%	12.73%	6.90%	0.26%
6	0.11391105	1.79%	2.90%	0.00%	4.48%	2.86%	6.19%
7	0.09273293	0.60%	0.42%	0.01%	0.44%	0.79%	0.03%
8	0.06980324	0.01%	0.01%	21.73%	3.88%	4.00%	0.00%
9	0.06859487	0.48%	0.61%	0.00%	0.85%	0.33%	2.24%
10	0.06314177	1.02%	0.89%	1.53%	0.30%	0.01%	0.05%
11	0.06218587	0.78%	0.07%	24.24%	0.12%	0.06%	0.01%
12	0.06028114	0.54%	0.04%	10.90%	0.75%	8.66%	0.00%
13	0.05593699	0.17%	1.87%	2.99%	0.01%	0.00%	0.93%
14	0.0546738	0.09%	0.58%	9.77%	6.95%	0.00%	0.20%
15	0.05282988	0.05%	0.05%	2.56%	5.74%	4.10%	0.05%
16	0.04939847	0.53%	0.00%	0.51%	0.20%	0.20%	0.07%
17	0.04900921	1.10%	0.06%	0.08%	0.03%	1.53%	0.23%
18	0.04766758	0.08%	0.06%	0.09%	0.04%	0.06%	0.00%
19	0.04764242	0.02%	0.03%	0.02%	0.05%	0.03%	0.00%
20	0.04751677	0.00%	0.01%	0.00%	0.02%	0.00%	0.00%
21	0.04748845	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%
22	0.04693303	0.02%	0.17%	0.00%	0.20%	0.02%	0.06%
23	0.04518345	0.04%	0.00%	1.03%	0.53%	0.10%	0.00%
24	0.04440704	0.06%	0.00%	0.83%	0.02%	0.01%	0.30%
25	0.0425215	0.16%	0.16%	0.38%	0.26%	0.14%	0.74%
26	0.04215003	0.02%	0.21%	1.79%	0.43%	0.26%	0.32%
27	0.03896025	0.00%	0.00%	2.59%	0.88%	1.72%	0.00%
28	0.03597357	0.00%	0.21%	0.03%	0.41%	0.00%	0.03%
29	0.03591037	0.00%	0.01%	0.18%	0.02%	0.07%	0.01%
30	0.03579677	0.00%	0.03%	0.40%	0.05%	0.03%	0.00%
31	0.03550903	0.00%	0.00%	0.08%	0.00%	0.07%	0.00%
32	0.03522197	0.01%	0.04%	0.04%	0.06%	0.00%	0.01%
33	0.03517887	0.04%	0.61%	0.01%	0.76%	0.17%	0.06%
34	0.03424599	0.02%	0.10%	0.07%	0.14%	0.13%	0.09%
35	0.03420806	0.00%	0.01%	0.14%	0.09%	0.01%	0.00%
36	0.0336182	0.01%	0.43%	0.04%	0.44%	0.02%	0.16%
37	0.03292699	1.28%	0.00%	0.04%	0.03%	1.43%	0.06%
38	0.03180739	0.44%	0.01%	0.00%	0.03%	0.81%	0.10%
39	0.03160259	0.07%	0.09%	0.02%	0.32%	0.08%	0.05%
40	0.03110182	0.00%	0.00%	0.00%	0.01%	0.00%	0.01%

41	0.02975344	0.06%	0.28%	0.00%	0.37%	0.10%	0.06%
42	0.02949107	0.01%	0.03%	0.00%	0.01%	0.01%	0.18%
43	0.02928331	0.00%	0.06%	0.01%	0.11%	0.00%	0.05%
44	0.02923384	0.01%	0.00%	0.01%	0.00%	0.02%	0.00%
45	0.02921889	0.00%	0.02%	0.00%	0.02%	0.00%	0.00%
46	0.02912057	0.03%	0.00%	0.05%	0.00%	0.04%	0.05%
47	0.0290231	0.02%	0.01%	0.02%	0.02%	0.02%	0.24%
48	0.02896595	0.00%	0.00%	0.01%	0.03%	0.00%	0.08%
49	0.02767097	0.00%	0.24%	0.01%	0.38%	0.00%	0.29%
50	0.02742665	0.03%	0.03%	0.35%	0.49%	0.03%	0.00%
51	0.02724533	0.16%	0.03%	0.01%	0.02%	0.37%	0.10%
52	0.02689115	0.02%	0.00%	1.81%	0.02%	0.01%	0.00%
53	0.0265348	0.03%	0.11%	0.26%	0.20%	0.04%	0.02%
54	0.02639492	0.00%	0.01%	0.01%	0.00%	0.04%	0.01%
55	0.0261289	0.00%	0.01%	0.20%	0.11%	0.12%	0.03%
56	0.02550031	0.01%	0.00%	0.10%	0.05%	0.00%	0.05%
57	0.02528252	0.06%	0.00%	0.00%	0.00%	0.12%	0.00%
58	0.02428665	0.01%	0.00%	0.10%	0.00%	0.01%	0.02%
59	0.02336167	0.27%	0.12%	0.22%	0.12%	0.74%	0.00%
60	0.02315704	0.01%	0.05%	0.83%	0.13%	0.19%	0.02%
61	0.02270054	0.01%	0.04%	0.00%	0.05%	0.00%	0.00%
62	0.02258117	0.04%	0.07%	0.07%	0.10%	0.02%	0.00%
63	0.02248234	0.05%	0.20%	0.01%	0.30%	0.10%	0.01%
64	0.02245042	0.13%	0.02%	0.02%	0.05%	0.20%	0.00%
65	0.02232985	0.18%	0.02%	0.00%	0.07%	0.32%	0.10%
66	0.02190071	0.00%	0.00%	0.45%	0.00%	0.00%	0.01%
67	0.02148208	0.19%	0.50%	0.00%	0.58%	0.21%	0.02%
68	0.02088007	0.04%	0.02%	0.40%	0.30%	0.06%	0.01%
69	0.02068746	0.06%	0.14%	0.08%	0.06%	0.09%	0.25%
70	0.02043232	0.17%	0.15%	0.02%	0.53%	0.28%	0.49%
Results	**SUM**	**90.22%**	**90.77%**	**87.17%**	**79.72%**	**79.48%**	**91.29%**

Table D.4 Eigenperiod and Effective Mass – With Infills/Cracked sections

Mode	**Period (sec)**	**[Ux]**	**[Uy]**	**[Uz]**	**[Rx]**	**[Ry]**	**[Rz]**
1	0.75021164	57.07%	7.28%	0.00%	3.72%	24.68%	6.55%
2	0.72405375	3.78%	60.06%	0.00%	24.46%	1.35%	8.89%
3	0.66508048	9.56%	3.68%	0.00%	2.31%	4.76%	59.13%
4	0.23126851	4.88%	1.49%	0.00%	3.99%	10.24%	3.57%
5	0.2181013	4.26%	7.63%	0.01%	14.42%	7.16%	0.11%
6	0.20379761	2.12%	2.25%	0.00%	4.35%	4.15%	5.86%
7	0.15938278	0.48%	0.37%	0.01%	0.45%	0.65%	0.01%
8	0.12563126	0.00%	0.01%	20.33%	3.90%	3.75%	0.00%
9	0.12361831	0.38%	0.56%	0.01%	0.48%	0.25%	2.11%
10	0.11481208	1.98%	0.85%	1.72%	0.95%	0.44%	0.00%
11	0.11016559	0.07%	0.12%	47.45%	0.16%	5.21%	0.00%
12	0.10558657	0.15%	0.59%	3.79%	1.18%	2.94%	0.17%
13	0.10264282	0.02%	0.35%	1.32%	0.47%	3.36%	0.40%
14	0.10138359	0.30%	1.40%	1.74%	10.58%	0.82%	0.59%
15	0.09702716	0.00%	0.01%	1.95%	0.09%	0.00%	0.01%
16	0.09295686	0.26%	0.00%	0.01%	0.15%	0.05%	0.01%
17	0.09069589	1.29%	0.15%	0.28%	0.58%	2.40%	0.24%
18	0.08025083	0.19%	0.04%	0.75%	0.02%	0.04%	0.74%
19	0.07835761	0.04%	0.26%	0.35%	0.42%	0.15%	0.41%
20	0.0746292	0.01%	0.05%	0.51%	0.32%	0.65%	0.01%
21	0.07287832	0.00%	0.01%	1.21%	0.67%	0.45%	0.00%
22	0.06675818	0.01%	1.16%	0.01%	1.03%	0.04%	0.27%
23	0.06493012	0.09%	0.05%	0.09%	0.19%	0.04%	0.03%
24	0.06241885	1.31%	0.05%	0.01%	0.18%	1.49%	0.08%
25	0.06143094	0.01%	0.00%	0.09%	0.00%	0.12%	0.00%
26	0.06021131	0.00%	0.01%	0.01%	0.01%	0.02%	0.02%
27	0.05997066	0.07%	0.00%	0.29%	0.00%	0.48%	0.00%
28	0.05988953	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%
29	0.05869261	0.05%	0.00%	0.07%	0.01%	0.13%	0.18%
30	0.05862327	0.13%	0.00%	0.03%	0.01%	0.24%	0.00%
31	0.05832629	0.06%	0.21%	0.14%	0.44%	0.00%	0.03%
32	0.05771926	0.02%	0.12%	0.01%	0.28%	0.13%	0.00%
33	0.05627446	0.05%	0.00%	0.00%	0.05%	0.10%	0.06%
34	0.0557822	0.01%	0.14%	0.00%	0.17%	0.00%	0.23%
35	0.05477316	0.03%	0.01%	0.00%	0.00%	0.04%	0.34%
36	0.05385344	0.00%	0.01%	0.34%	0.00%	0.00%	0.03%
37	0.05357866	0.00%	0.03%	0.00%	0.05%	0.00%	0.02%
38	0.0532662	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%
39	0.05289811	0.08%	0.00%	0.00%	0.00%	0.11%	0.07%
40	0.05243624	0.04%	0.25%	0.02%	0.36%	0.06%	0.00%

41	0.05174266	0.00%	0.00%	0.04%	0.01%	0.01%	0.01%
42	0.0510181	0.08%	0.11%	0.01%	0.19%	0.09%	0.19%
43	0.05033035	0.01%	0.01%	0.67%	0.02%	0.02%	0.00%
44	0.04979812	0.00%	0.01%	0.01%	0.03%	0.06%	0.01%
45	0.04808747	0.00%	0.00%	0.63%	0.21%	0.09%	0.00%
46	0.0474789	0.00%	0.00%	0.31%	0.23%	0.03%	0.01%
47	0.04470097	0.01%	0.00%	0.15%	0.00%	0.00%	0.04%
48	0.0441395	0.02%	0.00%	0.03%	0.00%	0.00%	0.00%
49	0.04348577	0.81%	0.00%	0.03%	0.00%	1.42%	0.05%
50	0.04272501	0.03%	0.87%	0.00%	1.20%	0.06%	0.01%
51	0.04071691	0.07%	0.10%	0.63%	0.35%	0.77%	0.33%
52	0.03992384	0.06%	0.00%	1.61%	0.18%	0.00%	0.06%
53	0.0395968	0.16%	0.04%	0.03%	0.15%	0.01%	0.33%
54	0.03817563	0.08%	0.03%	0.17%	0.17%	0.00%	0.02%
55	0.03655383	0.00%	0.02%	1.35%	0.01%	0.00%	0.01%
56	0.03621435	0.00%	0.02%	0.25%	0.28%	0.00%	0.01%
57	0.03604215	0.08%	0.02%	0.00%	0.03%	0.18%	0.08%
58	0.03446645	0.00%	0.00%	0.23%	0.11%	0.09%	0.00%
59	0.03424268	0.00%	0.03%	0.22%	0.13%	0.01%	0.01%
60	0.03354317	0.12%	0.28%	0.13%	0.36%	0.21%	0.01%
61	0.03308317	0.03%	0.17%	0.00%	0.26%	0.06%	0.01%
62	0.0326921	0.00%	0.00%	0.01%	0.00%	0.03%	0.00%
63	0.03099524	0.40%	0.00%	0.01%	0.01%	0.71%	0.18%
64	0.03061222	0.00%	0.03%	0.03%	0.02%	0.01%	0.02%
65	0.03046767	0.02%	0.02%	0.17%	0.05%	0.02%	0.03%
66	0.03007678	0.00%	0.00%	0.38%	0.18%	0.13%	0.00%
67	0.02997399	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%
68	0.02983277	0.01%	0.00%	0.00%	0.01%	0.02%	0.01%
69	0.02947978	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%
70	0.02907126	0.01%	0.01%	0.04%	0.02%	0.01%	0.00%
Results	**SUM**	**90.83%**	**90.98%**	**89.70%**	**80.70%**	**80.57%**	**91.58%**

References

[1] EN 1992-1-1, (2004), "Eurocode 2, Design of concrete structures - Part1: (Design of concrete: General rules and rules for buildings)".

[2] Seismosoft, (2022), "SeismoBuild 2022 - A computer program for seismic assessment of reinforced concrete framed structures". [Online] Available: https://seismosoft.com/support/seismobuild-support/

[3] Greek Earthquake Planning and Protection Organization (EPPO), (2022), "Greek Code of Interventions (KANEPE)", (OGG) 3197/B/22-6-2022, 3rd ed., Athens.

[4] EN 1998-3, (2004), "Eurocode 8, (Design of structures for earthquake resistance - Part 3: Assessment and Retrofitting of Buildings)", Brussels.

[5] ASCE/SEI 41-17, (2017), "Seismic Evaluation and Retrofit of Existing Buildings", American Society of Civil Engineers, Reston, Virginia.

[6] EN 1998-1, (2004), "Eurocode 8, (Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings)", Brussels.

[7] Baker J. W., Bradley B. A., and Stafford P. J., (2021), "Seismic Hazard and Risk Analysis", Cambridge University Press, Cambridge, England.

[8] Danciu L., Nandan S., Reyes C., Basili R., Weatherill G., Beauval C., Rovida A., Vilanova S., Sesetyan K., Bard P-Y., Cotton F., Wiemer S., and Giardini D., (2021), "The 2020 update of the European Seismic Hazard Model", *EFEHR Technical Report 001.* [Online]. Available: https://doi.org/10.12686/a15.

[9] Baltzopoulos G., Baraschino R., lervolino l., and Vamvatsikos D., (2017), "Software for seismic fragility assessment based on static pushover", *Bull Earthquake Eng,* vol. 15, pp. 4399–4425. [Online]. Available: http://wpage.unina.it/iuniervo/doc_en/SPO2FRAG.htm.

[10] Applied Technology Council, (2018), "FEMA P-58-1, Seismic Performance Assessment of Buildings,Volume 1–Methodology, Second Edition", Washington D.C.

[11] Applied Technology Council, (2018), "FEMA P-58-3, Seismic Performance Assessment of Buildings, Volume 3–Supporting Electronic Materials and Background Documentation", Washington D.C. [Online] Available: https://femap58.atcouncil.org/pact

[12] Greek Earthquake Planning and Protection Organization (EPPO), (2014), "First Level Pre-Earthquake Assessment Method", (OGG) 405/B'/05-09-2013, Athens.

[13] Greek Earthquake Planning and Protection Organization (EPPO), (2001), "Greek Code For Seismic Resistant Structures (EAK 2000)", (OGG) 2184 B'/ 20-12-1999, Athens.

[14] Official Goverment Gazette (OGG), (1959), "Greek Anti-Seismic Regualation", (OGG) 36/A/26-02-1959, Athens.

[15] Official Goverment Gazette (OGG), (1954), "Reinforced Concrete Regulation", (OGG) 160/A/26-07-1954, Athens.

[16] Official Goverment Gazette (OGG), (1984), "Modification of Anti-Seismic Regulation", (OGG) 239/B/16-04-1984, Athens.

[17] Greek Earthquake Planning and Protection Organization (EPPO), (2001), "Greek Regulation of Reinforced Concrete (EKOS 2000)", (OGG) 1329 B'/ 6-11-2000, Athens.

[18] Dritsos S. (EPPO), (2022), "Second Level Pre-Earthquake Assessment Method".

[19] ICBO, (1997), "Uniform Building Code", International Council of Building Officials, Whittier, CA.

[20] Applied Technology Council, (2018), "FEMA P-2018, Seismic Evaluation of Older Concrete Buildings for Collapse Potential", Washington D.C.

[21] ACI Committee 318, (2014), "Building Code Requirements for Structural Concrete and Commentary (ACI 318-14)", American Concrete Institute, Farmington Hills.

[22] Kreslin M. and Fajfar P., (2011), "The extended N2 method taking into account higher mode effects in elevation", *Earthquake Engineering and Structural Dynamics,* vol. 40, pp. 1571–1589.

[23] cyprus-life.info, (2021), "Utilities in Cyprus: water". [Online] Available: https://cyprus-life.info/cyprus_utilities/cyprus_utilities_water

[24] Top Roto LTD, (2023), "Storage Tanks". [Online] Available: http://www.top-roto.com.cy/tanks.html

[25] EN 1991-1-1, (2004), "Eurocode 1: Actions on structures - Part 1: (General actions: Densities, self-weight, imposed loads for buildings)".

[26] Runet, (2018), "Fedra, Design of masonry buildings according to EN 1996-1-1:2006". [Online] Available: https://www.runet.gr/fedra.htm

[27] EN 1996-1-1, (2004), "Eurocode 6, Design of masonsy structures - Part 1-1: (General rules for reinforced and unreinforced masonry structures)".

[28] Crisafulli F.J., Carr A.J., and Park R., (2000), "Analytical modelling of infilled frame structures–a general overview", *Bulletin of the New Zealand Society for Earthquake Engineering,* vol. 33.

[29] CSI, (2022), "SAP2000 Integrated Software for Structural Analysis and Design", Berkeley, California. [Online] Available: https://www.csiamerica.com/products/sap2000

[30] Vijayanarayanan A.R., Goswami R., and Murty C. V. R., (2017), "Estimation of storey stiffness in multi-storey buildings", *16th World Conference on Earthquake,* vol. 415.

[31] Vamvatsikos D. and Cornell C.A., (2002), "The Incremental Dynamic Analysis and its application to Performance-Based earthquake engineering", *12th European Conference on Earthquake Engineering,* Paper Reference 479.

[32] Vallianatou D., (2022), "Seismic performance, retrofit measures and loss assessment of a 4-storey reinforced concrete building, built in 1998 at Larissa", NTUA Master Thesis EMK ME 2022/08, Athens.