ISSN 2303-4521
Original Research
Periodicals of Engineering and Natural Sciences
Vol. 11, No. 4, July 2023, pp.94-104

A multi-story reinforced concrete structure's physical and geometric nonlinear evaluation based on open sees seismic analysis in architectural engineering building design

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ABSTRACT

Research Purpose: The purpose of this research is to analyze a G+20 reinforced concrete building in Amman, specifically focusing on its behavior under gravitational loading and wind, Applied Research Method: The research employs four types of static analysis: linear elastic analysis, geometric nonlinear analysis, physical nonlinear analysis, and physical and geometric nonlinear analysis. The OpenSees open-source program, developed at the University of California, Berkeley using C++ language, is used as the platform for the study, Principal Results: The research reveals significant differences in the results obtained from the various types of analyses performed. The linear elastic analysis, geometric nonlinear analysis, physical nonlinear analysis, and physical and geometric nonlinear analysis produce varying results. The effort (forces and stresses) experienced by a specific beam on different floors also exhibits reasonable variation, Major Conclusions: Based on the analysis results, it can be concluded that considering geometric and physical nonlinearity is crucial in accurately modeling the behavior of the reinforced concrete building. The use of the OpenSees program, along with the corotational formulation to account for large rotations and the P-Delta method for result validation, proves effective in capturing the complex behavior of the structure. The study highlights the importance of considering concrete cracking and reinforcement plasticity with precision to better understand the structural response.

Keywords: Open Sees, Seismic analysis, Multi-storeys building, static analysis, comparative study

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1. Introduction

Studying the realistic behavior of reinforced concrete structures subjected to accurate loads of use is still an advanced topic in the technical-scientific environment [1]. The effects of physical and geometric non-linearity, including reinforcement yielding and concrete cracking along the bar sections, and the presence of second-order effects stand out as some of the difficulties encountered in the development of the research [2]. Experimental studies of the behavior of large reinforced concrete structures using reduced prototypes often need to be more feasible, as they demand a large laboratory structure, financial resources and specialized technical staff. Thus, the computational simulation of the behavior of these structures becomes a more viable alternative [3]. Advances in technology, new materials and more elaborate computational methods in the analysis and design of buildings are a new reality. In some cities with a high population density, such as in some Asian countries, tall buildings may be one of the only viable solutions to the housing problem [4]. The evolution of hardware and software made possible static and dynamic analyses of very sophisticated spatial structures, enabling the use of refined models that better represent the real behavior of the structures. This becomes especially important in regions with the presence of intense seismic activity and other dynamic requests, such as some wind-loading situations. The extensive and frequent use of ultimate limit state design and non-linear structural analysis should advance structural design procedures [5].

Therefore, the study was developed by creating computational models very close to reality. This research aimed to simulate the behavior of an actual structure (a tall building in reinforced concrete) with the help of computer programs. For this, some very realistic finite element numerical models were developed. After the development of these models, several linear and non-linear analyzes were carried out, simulating different load cases. For this article, the analyzed results were the displacement of a node at the top of the building, the redistribution of the normal efforts of the columns, and the comparison of the bending moment diagrams in the beams on different floors.

2 Building description

The residential building under study is being finished (April 2013) in the city of Amman. Compliance with the Structural Safety regulations of the International Association for Earthquake Engineering (IAEE) [6], in regard to structural performance due to displacement, requires compliance with not exceeding the maximum permissible drift. To achieve this, they must have coherent and adequate form, each of the elements that make up the structural system and has high adaptability to its architecture as far as possible, often achieving this is very complicated due to the limitations that architecture allows by restricting the height of beams to very small values, the problem is even more critical if you want to use a beam that connects or couples two structural walls cut into concrete cores, which generally serve as support for the installation of elevators and emergency stairs. Therefore, it is adequately justified to couple the walls to improve the structural performance of the structural system [7]

Concrete of different resistance was considered, which varies with the height of the building depending on the compressive capacity that the elements of the system resistant to lateral forces will have, with a compressive resistance (f'c) of 280 kg/cm^2, 350kg/cm^2, 420g/cm^2, 490kg/cm^2, and reinforcing steel with a yield stress of 4200 kg/cm2. In the structural analysis models of the buildings, the contribution of the slab in both rigidity and resistance was taken into account when applying a rigid diaphragm, which condenses the degrees of freedom to three per floor; that is, there is freedom of translational movement. in two directions of analysis orthogonal to each other and rotational in the Plan, in each of the levels of the tower, rectangular sections were used in the perimeter and main beams of the lateral resistance system and under the assumption of the use of the rigid diaphragm, this is allowed. The thickness of the slabs was 20 cm, and it was conceived as a slab solid in two directions, which were designed for vertical loads and to satisfy deformation and service limit states. The structural configuration of the building is as follows and is presented in Figure 1: Number of levels = 20 floors + rooftop area for social events; Area = 1,296.98 square meters/ Dimensions = 43.75 x 26.25 m to axes.

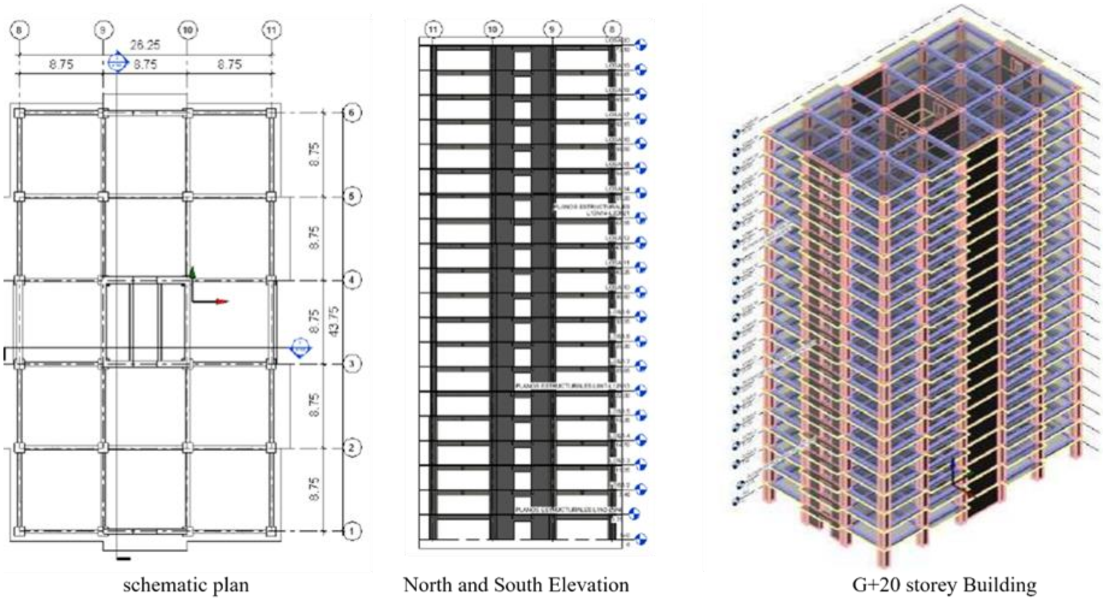


Figure 1. G+20-story building model, Plan, north and south elevation, colour-coded 3D schematic of structural components. [Color code: Lateral Resistance System = Red+blue (Columns, walls, coupled walls and beams) Mezzanine System = Yellow (Solid slabs) Vertical load resistant system (Columns, walls and coupled walls)]

3 Models developed

The non-linear analysis of complex structures through computational methods using microcomputers demands a high processing time. The solution to this problem is the modelling of simpler structures. For this, the study was divided into two phases:

The first phase was developed in the SAP2000 computer program (SAP2000 Manual) [8]. In this phase, a finite element mesh of the typical pavement was elaborated, to obtain the loads distributed in the column beams resulting from the loads applied to the slabs. The loads distributed on the beams can be reached by obtaining the shear stress diagrams.

The second phase was developed in the program The Open System for Earthquake Engineering Simulation (OpenSees) [9-10]. The model developed was a three-dimensional frame with non-linear elements of bars representing columns and beams. The slabs were disregarded and replaced by the loads distributed on the beams obtained in the study's first phase. The rigid diaphragm model was used to represent the great rigidity provided by the floor slab in its own plane.

The soil-structure interaction was disregarded in this analysis, assuming a perfectly rigid foundation without settlements

3.1 Modelling of the pavement type by finite elements in SAP2000

The mesh is generated by the computational program SAP2000. It was determined that bar elements (frame type) represent the beams, while shell elements (shell type) represent the slabs. In the model, the dimensions of the columns with rigid connections (riding links) at the ends of the beams were considered—the deformed configuration of the pavement model under permanent and incidental loading.

3.2 Brief description of the opensees program

The Opensees program was developed at the University of California, Berkeley, originating from a doctoral thesis [10]. This computational program is constantly improved by the academic community through a communication system called Concurrent Versions System (CVS) through the internet.

Currently, many researchers are engaged in this technical-scientific project, implementing new models or even improving existing models to make the computational analysis as realistic as possible.

This program had as its initial objective the simulation of the behavior of structures subjected to earthquakes due to the fact that the region where the program was developed presents a high level of seismic intensity (West Coast of the United States). Because it is not a commercial program, Opensees is sponsored by the National Science Foundation, the science development foundation in the United States.

Currently, many researchers are engaged in this technical-scientific project, implementing new models or even improving existing models to make the computational analysis as realistic as possible.

3.3 Modelling of the building structure in the computational program open sees

The structure was analyzed considering the two effects of non-linearity:

a) Effect of material or physical non-linearity

b) Geometric nonlinearity effect

The finite element used for modelling the bars of the three-dimensional frame is based on the force method in such a way that only one element is needed to represent an entire structural member (column and beam) [11]. The beam-column element employed considers the distribution of inelasticity along the length and along the sections of the bar, through the use of a fiber model, in such a way that it is possible to consider the cracking of the concrete and the plasticity of the reinforcement with great accuracy (figure 2).

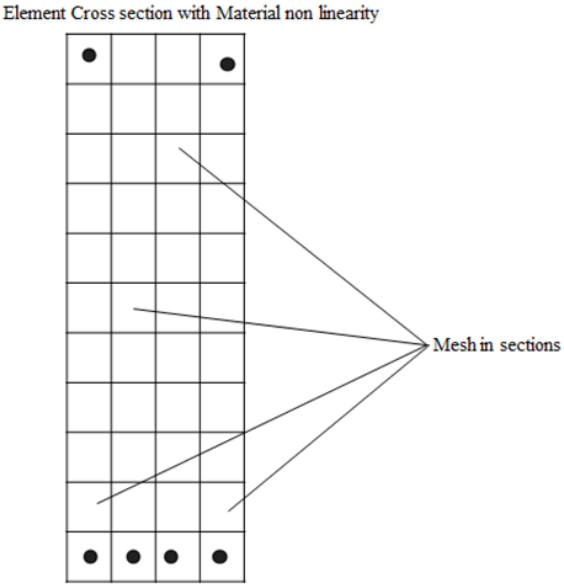


Figure 2. Mesh in the cross-section of beams and columns

The reinforcement arrangement along the beams was represented in the model with the variation in five points (Gauss-Lobatto points) [12] (figure 3).

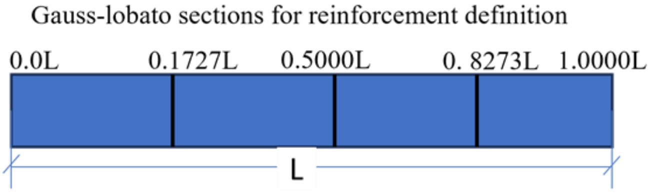


Figure 3. Reinforcement variation along the beam

The formulations used to consider second-order effects were the Correlational formulation [13] and the P-Delta method. The algorithm used for convergence in the analysis was the Newton-Raphson method.

The loads considered in the analysis were permanent loads, accidental loads and loads from the action of the wind, in compliance with the standards in force (Eurocode 2 and EN 1998-1-Eurocode 8)[14]. In the analyses in which the action of the wind was admitted, this load was considered to be acting in the direction of less rigidity of the structure (figure 4).

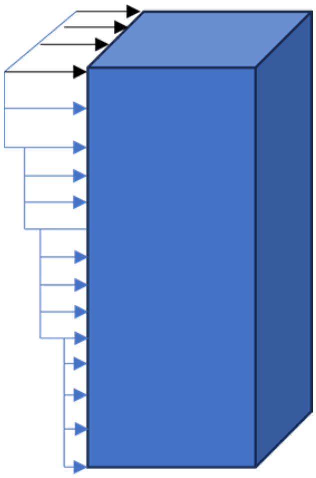
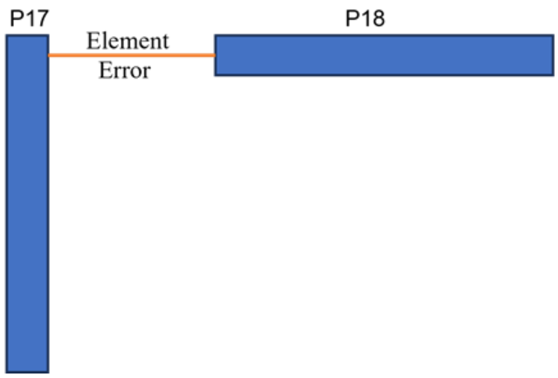


Figure 4. Wind loads – variation along the building height

In the 3D modelling of the structure, a node was defined at each of the centers of the columns and at the beam-beam intersections. Then, the initial and final nodes for the beam segments were specified. However, to consider the dimensions of the column sections at the meeting points (nodes) of the structural elements, special measures were taken to correct the simplifications present in the models of frames with bar elements. Figure 5 shows the correct way the structure was designed. The dimensions of the pillars are shown in blue, and the axis of the beam segment is in brown. And in Figure 6, there is how the elements were defined in the computational model.



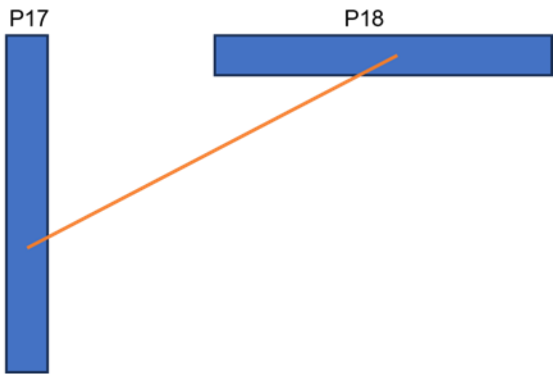


Figure 5. Column/Beam correct. Figure 6. The computational model defined the elements.

To correct the positions of the initial and final nodes of the beam segments, the rigid regions (joint offsets) at the ends of the beams were taken into account to represent the dimensions of the columns. The Open Sees program has a command that facilitates this assignment. With only the initial node and the final node in the centers of the columns and the distances that these nodes are from the correct positions of the nodes, rigid connections are created at the ends of the beams (figure 7).

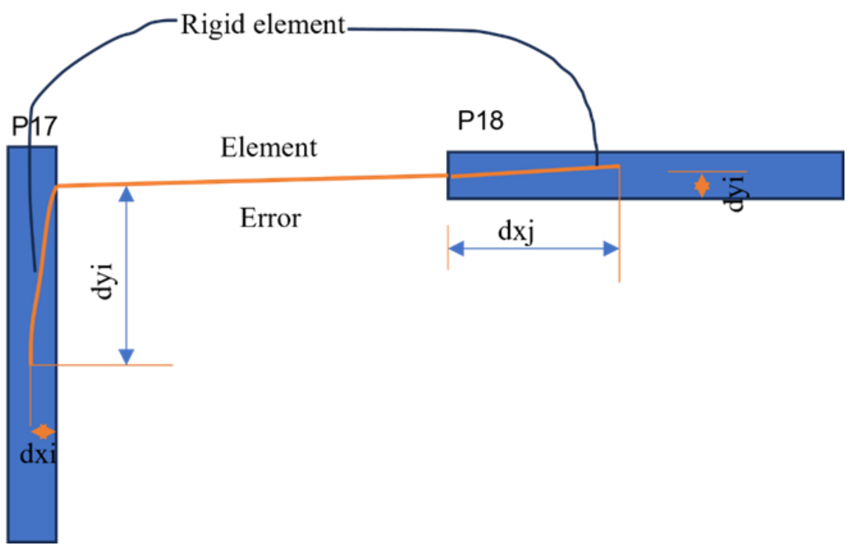


Figure 7. Mode positions, rigid connections

Having defined all the centers of the columns, all the segments of the beams and all the necessary rigid connections at the ends of the beams, the types of analysis and loads to which the structure is subjected were defined. Four types of computational models were developed:

- 1st Model: Linear-Elastic Behavior.
- 2nd Model: Behavior with Geometric Nonlinearity.
- 3rd Model: Behavior with Material Non-Linearity.
- 4th Model: Behavior with Geometric and Material Non-Linearity.

For the developed model, two load cases were considered:
- Case 1: Permanent and accidental loading.
- Case 2: Permanent, accidental and wind loading in the direction of least rigidity.

4 Results

4.1 Displacement at the top of the Structure in the direction of less rigidity

Table 1 presents the horizontal displacement (in the direction of lower stiffness of the frame) of the central node at the top of the building, corresponding to a load factor equal to 1.0 for the various models and load cases considered.

Table 1. Horizontal displacement (in the direction of less rigidity)

	Model-1 (cm)	Model-2 (cm)	Model-3 (cm)	Model-4 (cm)
Case -1	1.76	1.82	2.50	2.67
Case -2	8.77	9.07	11.29	12.25

Table 2 presents the load factors corresponding to the highest load value applied to the structure (horizontal section of the load-displacement diagram).

Table 2. Collapse Factor

	Model-1	Model-2	Model-3	Model-4
Case -1	..	30.6	3.6618	3.315
Case -2	..	21.93	2.805	1.9992

Figures 8, 9, 10 and 11 illustrate the curves obtained from the analysis of the models

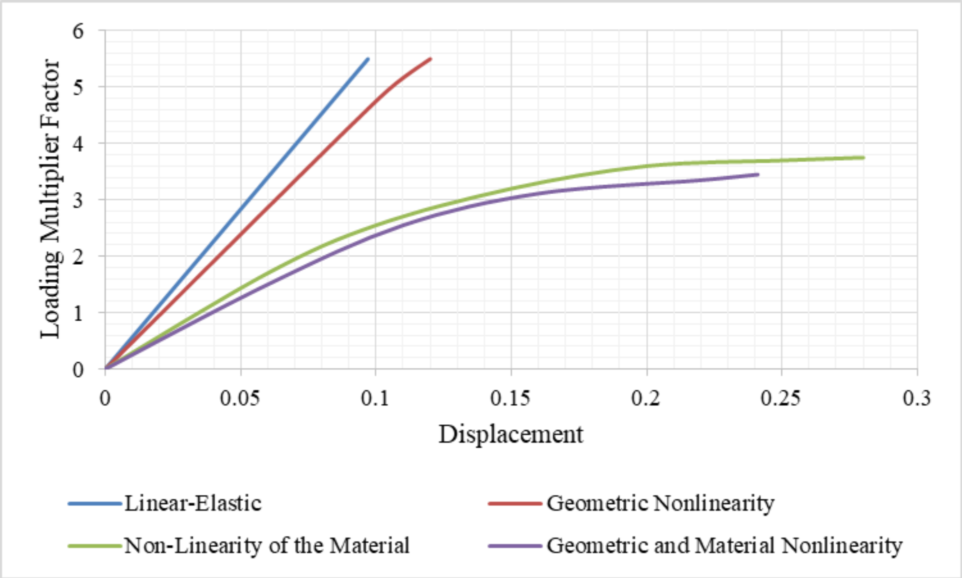


Figure 9. Four Models without wind (Structure Behavior)

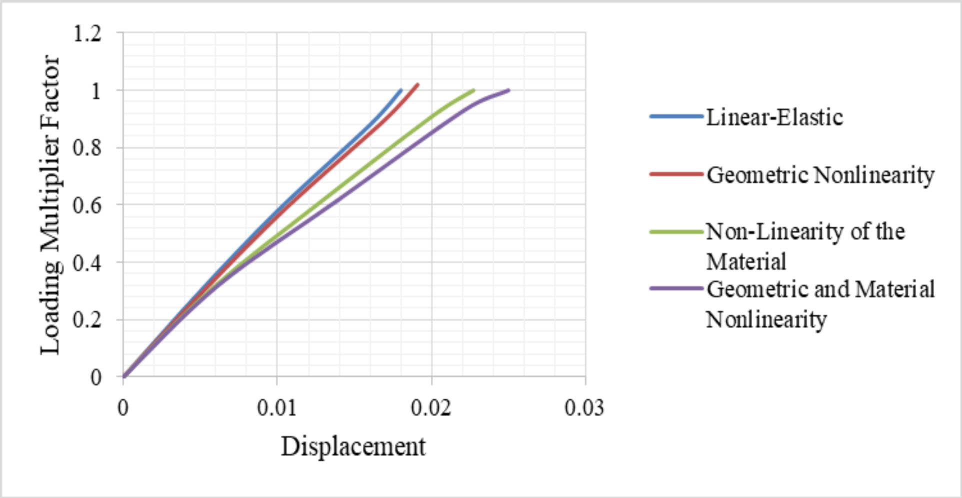


Figure 10. detail of Figure 9 (Structure Behavior)

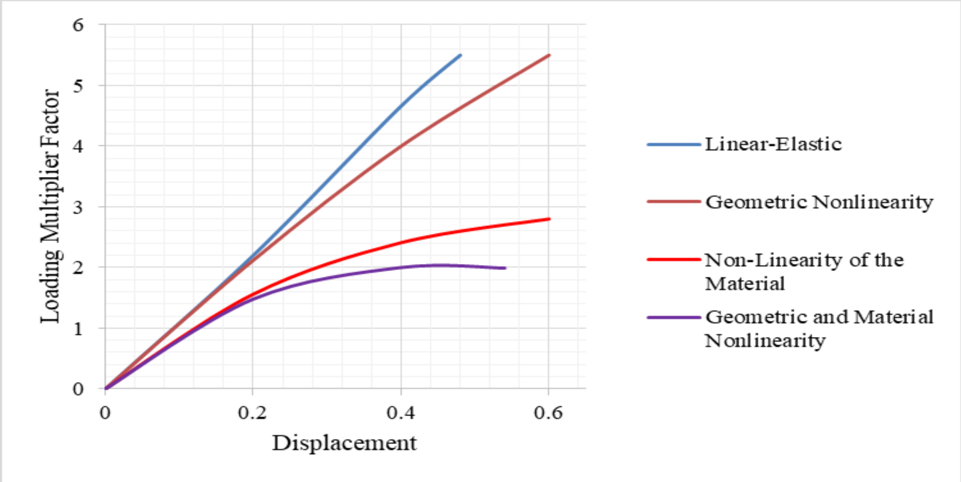


Figure 11. Four models with wind. (Structure Behavior)

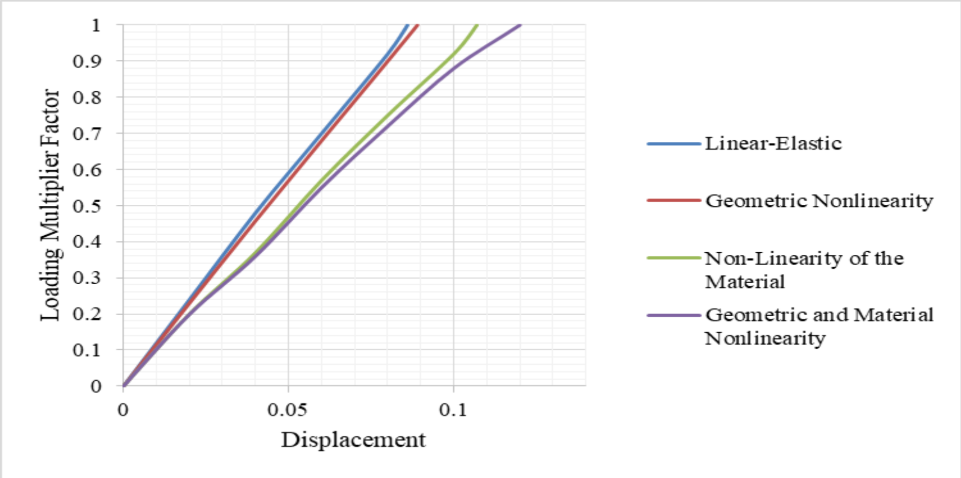


Figure 12. Detail of Figure 11 (Structure Behavior)

4.2 Comparison of bending moment diagrams

In this work, the variation in the behavior of the beams in relation to the floor level was also analyzed through the bending moment diagram. For this, two floors were chosen in the building: the first and the last typical floor.

The efforts of beam V4 (figure 13) were monitored to plot the bending moment diagrams. This beam was selected because it is a central beam of large dimensions.

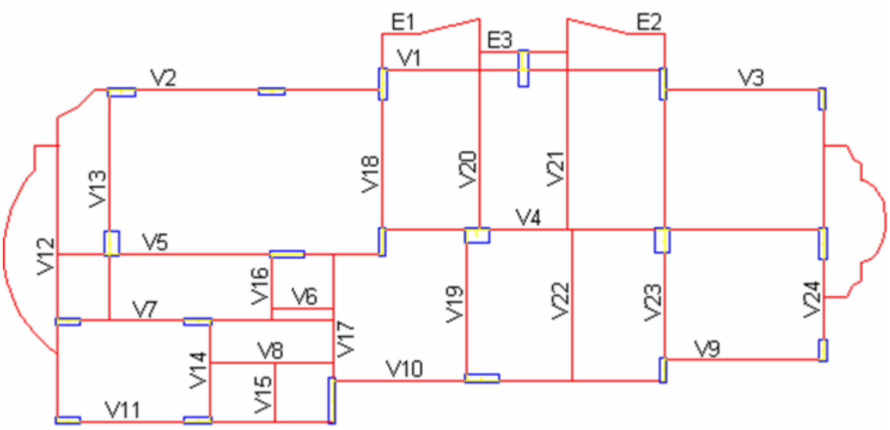


Figure 13. Numbering of beams

4.3 Comparison of effects on pillar bases

Table 3 shows the normal efforts and the bending moments in the setting of each column at the base of the building (disregarding the flexibility of the foundation). The linear-elastic model was compared with the geometric and physical non-linear models. The numbering of the pillars in the structure is shown in Figure 14.

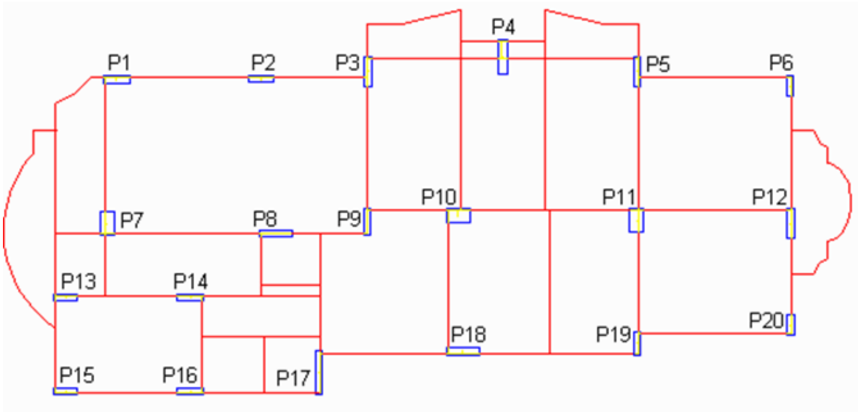


figure 14. Numbering of pillar

Table 3. With Wind Load in the direction of less rigidity

With Wind Load in the direction of less stiffness									
	Linear-Elastic			Nonlinear Geometric and Physical			Comparison of the two models		
PILLER	Charge(C1)	MY1	MZ1	Charge(C2)	MY2	MZ2	$\left(\frac{C_1-C_2}{C_1}\times 100\right)$ (%)	$\left(\frac{MY_1-MY_2}{MY_1}\times 100\right)$ (%)	$\left(\frac{MZ_1-MZ_2}{MZ_1}\times 100\right)$ (%)
P-1	2592900	-19585	16198	2442150	-47090	-78030	5.8	-141.1	584.6
P-2	1648200	-15540	14373	1437150	-9553	20456	12.9	38.7	-42.5
P-3	2492400	-1067	287456	2241150	-673	315411	10.1	37.1	-9.8
P-4	3899400	-6924	596032	3738600	-6986	671966	4.1	-0.9	-12.8
P-5	2733600	-1537	287438	2542650	-1464	307528	7.0	4.7	-7.0
P-6	1628100	2396	98591	1427100	2530	80702	12.4	-5.6	18.2
P-7	3366750	-9508	303128	3628050	-14025	358470	-7.8	-47.7	-18.4
P-8	1889400	-34060	10854	1899450	-29432	15320	-0.5	13.7	-41.3
P-9	1487400	-492	179834	1618050	-367	228344	-8.8	25.5	-27.1
P-10	3366750	-26169	109126	3376800	-25490	158194	-0.3	2.6	-45.2
P-11	3869250	-8888	303128	4009950	-9002	352115	-3.7	-1.3	-16.2
P-12	2160750	1254	308357	2160750	2105	332145	0.0	-68.1	-7.7
P-13	1015050	-4392	17000	880380	-1836	17393	13.3	58.5	-2.3
P-14	1216050	-24713	19125	1346700	-22547	32527	-10.8	8.8	-70.4
P-15	390892	-4392	7413	338526	-1991	8755	13.5	54.9	-18.2
P-16	424946	-24713	8340	452889	-24179	16028	-6.6	2.2	-92.7
P-17	779206	-1196	781849	739200	-710	441770	5.2	40.9	43.7
P-18	1407000	-35432	19908	1648200	-33117	32164	-17.2	6.6	-61.9
P-19	1045200	-1256	148262	1236150	-1312	176877	-18.4	-4.5	-19.4
P-20	330481	878	99324	411165	1083	88725	-24.5	100.0	10.7

4.4 Redistribution of loads on columns

Table 4 presents the results of the normal efforts of the columns in the various analyses when subjected to permanent and accidental loads. Referring to a load factor of 1.0 [15].

Table 4. Structure Submitted to Permanent and Accidental Loading

PILLER	Structure Submitted to Permanent and Accidental Loading			
	C1	C2	C3	C4
	Linear-Elastic	Geometric Nonlinearity	Material Non-Linearity	Geometric and Material Nonlinearity
P-1	1949300	1959400	1989700	1999800
P-2	1424100	1424100	1313000	1313000
P-3	1999800	1999800	1898800	1908900
P-4	3151200	3161300	3161300	3171400
P-5	2373500	2383600	2312900	2323000
P-6	1282700	1292800	1181700	1191800
P-7	3070400	3070400	3242100	3252200
P-8	1767500	1767500	1727100	1737200
P-9	1474600	1474600	1565500	1565500
P-10	3322900	3322900	3343100	3343100
P-11	3716800	3716800	3888500	3888500
P-12	1919000	1919000	1949300	1949300
P-13	1111000	1111000	1020100	1010000
P-14	1222100	1222100	1302900	1302900
P-15	734000	730861	634205	628896
P-16	872163	868305	887422	880334
P-17	1656400	1646300	1454400	1444300
P-18	2121000	2110900	2171500	2161400
P-19	1706900	1706900	1737200	1727100
P-20	1050400	1040300	956680	948021

Table 5. The results of the comparisons in the different analyses

PILLER	Comparison1 $\frac{C_1 - C_2}{C_1}$	Comparison2 $\frac{C_1 - C_3}{C_1}$	Comparison3 $\frac{C_1 - C_4}{C_1}$	Comparison4 $\frac{C_2 - C_3}{C_2}$	Comparison5 $\frac{C_2 - C_4}{C_2}$	Comparison6 $\frac{C_3 - C_4}{C_3}$
	(%)	(%)	(%)	(%)	(%)	(%)
P-1	-0.57	-2.28	-2.85	-1.71	-2.27	-0.56
P-2	0	8.58	8.58	8.58	8.58	0
P-3	0	5.56	5.01	5.56	5.01	-0.58
P-4	-0.35	-0.35	-0.7	0	-0.35	-0.35
P-5	-0.47	2.81	2.34	3.27	2.79	-0.48
P-6	-0.87	8.66	7.8	9.45	8.59	-0.94
P-7	0	-6.15	-6.51	-6.15	-6.51	-0.34
P-8	0	2.52	1.88	2.52	1.88	-0.64
P-9	0	-6.78	-6.78	-6.78	-6.78	0
P-10	0	-0.67	-0.67	-0.67	-0.67	0
P-11	0	-5.08	-5.08	-5.08	-5.08	0
P-12	0	-1.74	-1.74	-1.74	-1.74	0

PILLER	Comparison1 $\frac{C_1 - C_2}{C_1}$	Comparison2 $\frac{C_1 - C_3}{C_1}$	Comparison3 $\frac{C_1 - C_4}{C_1}$	Comparison4 $\frac{C_2 - C_3}{C_2}$	Comparison5 $\frac{C_2 - C_4}{C_2}$	Comparison6 $\frac{C_3 - C_4}{C_3}$
	(%)	(%)	(%)	(%)	(%)	(%)
P-13	0	9	10	9	10	1.09
P-14	0	-7.27	-7.27	-7.27	-7.27	0
P-15	0.47	14.96	15.75	14.54	15.35	0.92
P-16	0.48	-1.93	-1.03	-2.42	-1.53	0.88
P-17	0.67	13.42	14.08	12.83	13.5	0.76
P-18	0.53	-2.62	-2.09	-3.16	-2.63	0.52
P-19	0	-1.96	-1.3	-1.96	-1.3	0.64
P-20	1.06	9.81	10.73	8.84	9.76	1

Variations in column loads in the different analyses are shown in Figure 15.

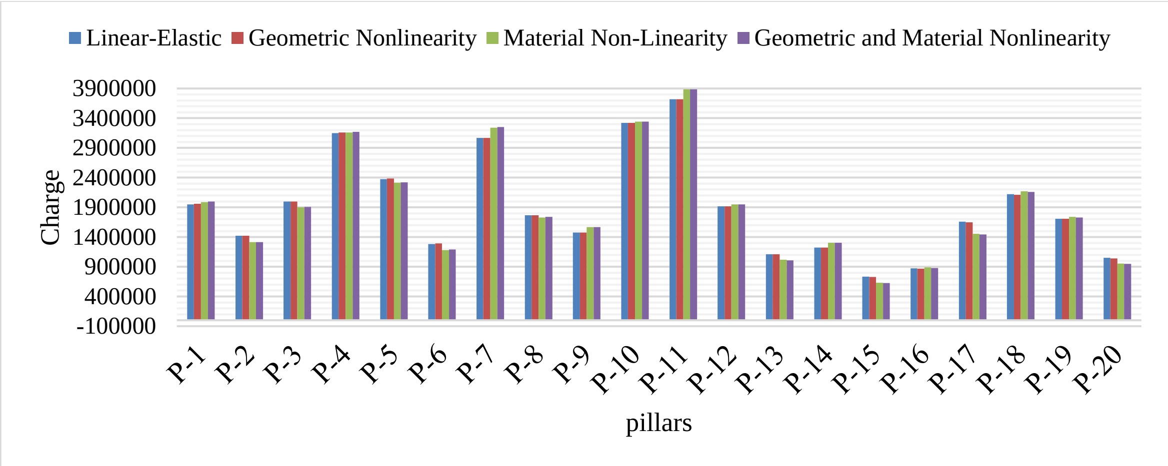


Figure 15. Variations in column loads for various analyses

5. Conclusion

The displacement obtained for the top of the building, assuming the effects of geometric and physical non-linearity, subjected to permanent, accidental and wind loading in the direction of less rigidity, was 12.01 cm, referring to a load factor of 1.0. Compared with the result obtained in the linear-elastic model that displaced 8.60 cm, the displacements increased by 39.66%. It is concluded that the non-linearity effects are significant and must be taken into account in the dimensioning phase. The variation of efforts in the beams for different floors was representative when the results from the linear-elastic analysis were compared with the physical and geometric non-linear analysis. It is concluded that in the dimensioning of the beams, the variation of efforts in the function of the floor level in the building must be considered.

The current International/European concrete standard Eurocode 2 provides for considering the portion of second-order effects when these were greater than 10% of the primary effects. Comparing the redistributions of axial loads in the models, linear elastic with geometric and physical non-linear were representative when the structure was subjected to permanent, accidental and wind loading in the direction of less rigidity. There were many pillars with a difference greater than 10% (P2 – 12.90%; P3 – 10.1%; P6 – 12.40%; P13 – 13.30%; P14 – 10.8%; P15 - 13.50%; P18 - 17.2%; P19– 18.4% and P20 – 24.501%). When the structure was subjected only to permanent and accidental loading, only two pillars showed a difference greater than 12% (P15 – 14.96% and P17 – 13.42%), referring to a load factor of 1.0. It is concluded that the non-linear effects can be representative and must be verified in the sizing.

Funding information

No funding was received from any financial organization to conduct this research.

Declaration of competing interest

The authors declare that they have no known financial or non-financial competing interests in any material discussed in this paper.

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