Seismic analysis of buildings with basements

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Abstract. This research work analyzes how seismic waves affect buildings with basements that are usually studied independently: In surface structures the inertial forces originated by the movement of their base and in subway structures the deformations due to ground motion are considered. The building located in the city of Quito with twelve elevated floors and five subfloors is studied using nonlinear finite element modeling, considering that the subsoil depends on seismic excitations in a dynamic soil-basement-structure set, for which it is modeled together with its geotechnical components. The superstructure is integrated to the infrastructure and a time-history analysis is included with the most representative earthquakes can affect the building. The present investigation has three basic components: seismic, geotechnical, and structural, which are merged by means of specialized 3D structural engineering programs. In the seismic component, local and regional information about tectonic faults within Ecuador is collected; the building is then located on a seismic risk map and the magnitudes, types, and other parameters of the site where the building is located are determined. Using the results of the Earthquake Risk Assessment Project for the city of Quito (TREQ) and the Pacific Earthquake Engineering Research Center (PEER) database, three relevant seismic events are chosen for the time-history analysis that meet the seismic risk data for the building. For the geotechnical soil component, the geotechnical parameters of the original design of the building foundation, stratigraphy, and characteristics of soil solids, among others, are used. The structural component considers the type of material, geometry, dimensions, joints between elements and others of the original design for both basements and superstructure. The connection of these three components is carried out using Midas Gen and Midas GTS NX. The results of deformations as a function of time are compared with similar standards, studies, and investigations.

1 Introduction

Although structures such as basements and tunnels are less affected by relatively severe earthquakes than surface structures, they should be made safe because there is a risk of loss of life, vehicles, utilities, equipment, machinery, and so on, as well as their own structural damage [1].

The comprehensive analysis of buildings with basements is complex and multidisciplinary because it includes concepts of both geotechnical engineering and structural engineering. According to J. Pinto et al. [2] the engineering assumption of omitting

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the soil around the basement walls of buildings by ignoring the nonlinear behavior of the soil and omitting the analysis of the Soil-Basement-Structure Interaction (SBSI) are not realistic conditions of seismic response. Pinto recommends that new procedures contribute more reliable estimates of the seismic response of tall buildings and additional research to explore other soil conditions and geometries using multidimensional effects (3D models).

This paper considers the recommendations of the research [2] and proposes the analysis of a twelve-story building located in Quito - Ecuador, with SBSI interaction using a 3D tool for both the Nonlinear Dynamic Structural Analysis of finite elements and the Time-History Seismic Analysis.

2 Methodology

The building is located on Av. 6 de diciembre and Av. Shyris, in the center-north of Quito. It was designed in February 2010 and built at the coordinates (Lat. -0.18901, Long. -78.48183). According to data from the Metropolitan District of Quito (MDQ), the structure is in an area of medium population density and high commercial value. The building is made of reinforced concrete and its walls are made of block. It has five basements for parking vehicles that occupy the entire available area, a ground floor with a common room and reception area, twelve upper floors of similar architecture for offices, and an inaccessible roof.

To perform the seismic analysis of the building, the geological faults that affect the city of Quito and their incidence on the building were investigated. With the results of the Training and Communication for Earthquake Risk Assessment for the city of Quito (TREQ) [3], the seismic risk of this building is found. Using the database and applications of the Pacific Earthquake Engineering Research Center (PEER) [6], three representative seismic events like the most unfavorable for the building, according to TREQ, are chosen. For the analysis of the geotechnical component, the soil report of the original study is used. For the structural analysis, three models are analyzed.

Model 1 (M1) consists of the elevated floors with supports restricted at the base to displacement and rotation. Model 2 (1) adding the 5 subfloors, perimeter walls and bottom slab, and Model 3 (M3) which includes the 3D floor that is linked to the structure formed by raised floors and basements. Structured in this way, the results of the 3 models can be comparable. See Fig. 1.

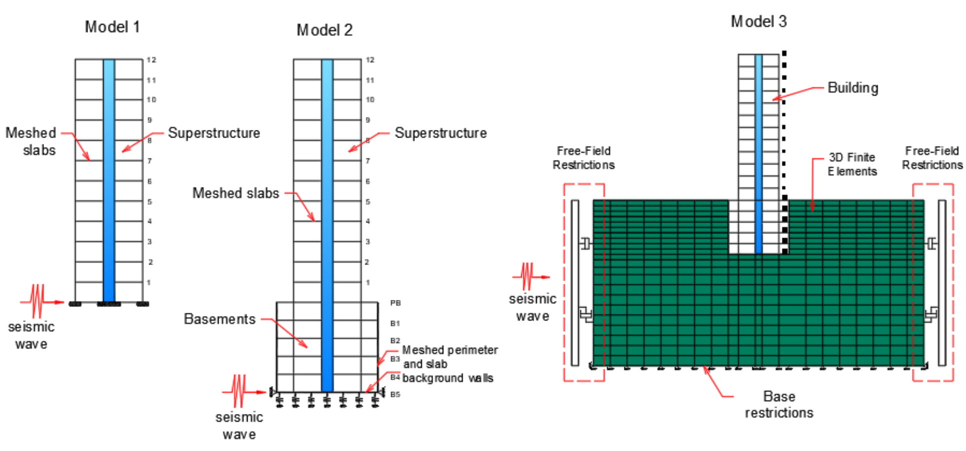


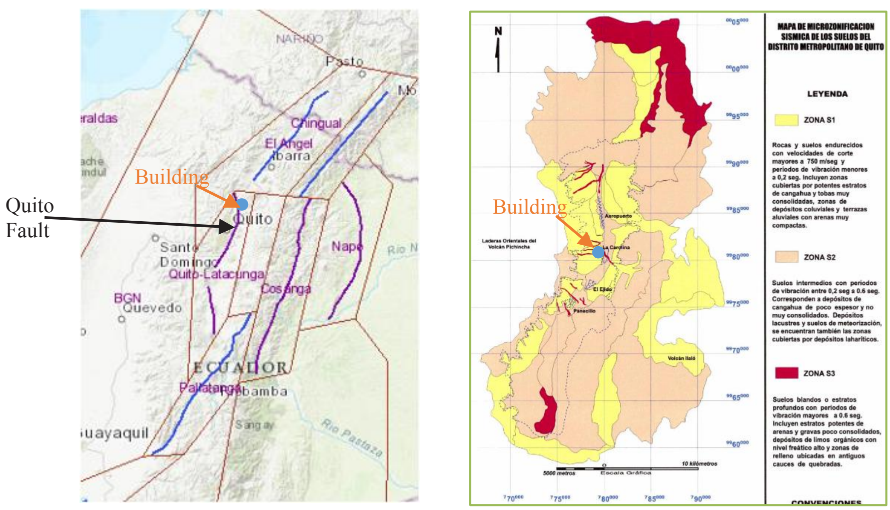
Fig. 1. Building Models.

For Model 1 and Model 2, MIDAS Gen tool is used, where the Eigenvalue Analysis is performed to identify the periods of the structures, the Linear and Non-Linear Static Analysis with the loads of the buildings and others recommended by the codes, and the Response Spectrum Analysis using the NEC 15 code. For static earthquakes, the Colombian NSR-10 code adjusted to NEC 15 is considered, since MIDAS does not currently have NEC15 in its software.

For Model 3, Midas GTS NX is used to perform the Non-Linear Dynamic Analysis (DNL) of the complete structure also considering a Time-History Analysis of the three previously selected seismic records. Licenses and technical support for Midas Gen and Midas GTS NX were provided by Midas Latin America [4].

The modeling results are presented in a wide range of tables and graphs with their respective interpretations and comments. The results of the comprehensive modeling are compared and discussed with similar studies in high seismic risk countries and with NEC 15 standards.

2.1 Seismic component



(a) (b)

Fig. 2. (a) Fault map of Quito and its surroundings. (b) Seismic micro zonation map of the MDQ.

To analyze the seismic component, the most recent studies on the faults of the city and its surroundings information on micro zonation of the MDQ, state of the art of seismic vulnerability, among others, are compiled. Fig. 2 indicates that the building is located less than 4 km from normal faults and within an S2 seismic micro zonation of intermediate soils [5].

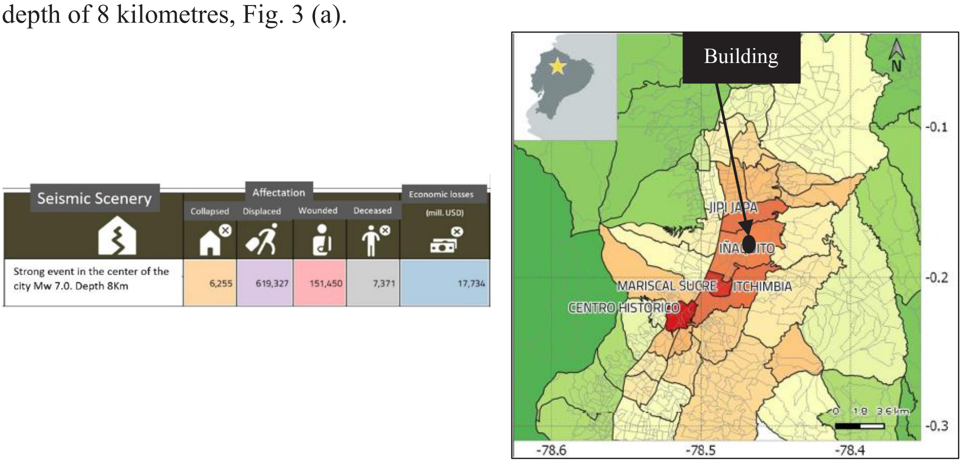
Report D261 Seismic Risk Assessment Report for the MDQ presents the results of the urban risk assessment obtained by the TREQ Project [3], financed by the United States Humanitarian Aid Office. The project proposes five hypothetical seismic events considering Quito's faults, the most catastrophic seismic event that the country has had in 1906 and the last representative earthquake that occurred in the province of Manabí. See Table 1.

Table 1. Seismic events in Quito and scenarios included in the TREQ Project [3].

Event	Description	Magnitude	Depth
1	Quito Fault. Mw6.5 event in the center of the city.	6.5	8
2	Quito Fault. Mw6.5 event in the north of the city.	6.5	8
3	**Quito Fault. Mw7.0 event in the city center.**	**7.0**	**8**
4	Quito Fault. Mw6.5 event in the south of the city.	6.5	8
5	Quito Fault. Event Mw6.5 to the west of the city.	6.5	8
6	Nazca. Earthquake of 1906. Subduction of the Nazca Plate.	8.8	20
7	Manabí. Muisne Earthquake 2016.	7.8	20

To define the seismic hazard model, TREQ describes the sources of seismicity in the region and the ground motion models that allow evaluating the intensity of seismic shaking. The seismic hazard model used is the one proposed by Beauval et al. (2018). To define the exposure model TREQ describes the inventory of urban buildings exposed to seismic hazard.

Regarding the evaluation of seismic risk in Quito, TREQ presents two approaches: The deterministic risk is estimated due to the occurrence of a single seismic event of defined characteristics. From the events analyzed, TREQ concludes that an event such as the Nazca Plate subduction earthquake of 1906, with a magnitude of 8.8 and a depth of 20 kilometres, would have a medium impact on the city. The highest impact scenario is a hypothetical surface earthquake, occurring in the center of the city, with a magnitude of Mw 7.0 and a depth of 8 kilometres, Fig. 3 (a).



(a) (b)

Fig. 3. (a) Results of the critical scenario of the TREQ Project. (b) Map of results of the hypothetical seismic scenario with the greatest effect on the building under study (Magnitude 7.0 - 8 km depth). Modified based on [3].

The results of the relevant case for the present research are shown in Fig. 3 (a) and Fig. 3 (b). In Fig. 3(a), the first column shows the seven seismic scenarios proposed by TREQ, arranged in descending order according to the degree of damage. For the Iñaquito sector, which is the closest to the building under study, there is a high collapse rate in the first scenario. Fig. 3(b) shows the location of the building within the seismic risk map for Quito.

As for event 7, the magnitude of the Muisne 2016 earthquake, because of the displacement of Nazca and South American tectonic plates was 7.8 Mw, however the magnitude with which this event reaches the city of Quito is 4.8 Mw [3], which shows that earthquakes coming from the Ecuadorian coast are less critical than those produced due to local faults.

The following are the results of Event 3: Quito Fault. Mw7.0 event in the city center at 8 km depth, obtained from the TREQ data tables, which corresponds to the location of the building under study. View table 2.

Table 2. Taken from Table Quito TREQ Project [3].

Registration	Length	Latitude	vs_{30}	z1pt0	z1pt5	$vs_{30measured}$
5173	-78.4874	-0.1856	349.43	413.74	1.48	1

where:
vs_{30} - Mean velocity of the shear wave.
z1pt0 - Vertical distance from the earth's surface to the layer where seismic waves begin to propagate with a speed greater than 1.0 km/sec.
z1pt5 - Vertical distance from the earth's surface to the layer where seismic waves begin to propagate with a speed greater than 2.5 km/sec.

2.2 Seismic events to be considered for Time-History Analysis

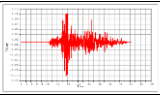
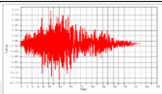
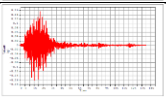
To choose the seismic events to be used for the nonlinear time-history analysis, the PEER applications are used, entering as search criteria the data provided by the TREQ simulation. The search generates several records, from which the ones shown in Table 3 are chosen.

2.3 Geotechnical component

The geotechnical study of the building indicates that a direct exploration was performed, based on two borings, standard penetration tests and sample recovery, application of soil mechanics theories for interpretation, definition of the type of foundation, foundation elevations, admissible load capacity, among others [7].

See results in Table 4. According to the geotechnical study the water table was detected at a depth of 7.50 m. In Model 2, Young's modulus obtained through correlations is used to simulate the vertical coefficient (Kv) at a depth of 16m where the bottom slab is located.

Table 3. Selected Seismic Records.

Event	Year	Fault Mechanism	Station	Magnitude	Rjb (km)	Rup (km)	Vs_{30} (m/s)	
Duzce Turkey	1999	Glide	Lamont 1061	7.14	413.74	1.48	481.0	
Landers	1992	Glide	Yermo Fire Station	7.28	23.62	23.62	353.63	
Darfield New Zeeland	2010	Glide	Heathcote Valley Primary School	7.0	24.36	24.47	422.0	

where:
Rjb - Distance from the place of energy release to the study place
Rup - Closest distance to the seismic rupture plane
Vs_{30} - Average shear wave velocity

Table 4. Geotechnical Data [7].

Stratum	Description	Depth (m)	SPT Avg	γ kNm^{-3}	ν	k_0	c kNm^{-2}	φ [°]	E kNm^{-2}
1	Low compressibility silt of medium consistency	0 - 4	7	15.1	0.35	0.54	42	0	2000
2	Low compressibility silt, very firm consistency	4 - 10	35	17.4	0.35	0.54	180	0	10000
3	Compact fine sands and gravels	10 - 17.5	42	18.3	0.30	0.38	0	38	26600
4	Hard low compressibility silt	17.5 – 22	39	18.1	0.35	0.54	270	0	16400
5	Sands with gravels, very compact	> 22	R>70	18.9	0.30	0.36	0	40	30000

where:
γ - volume weight of the soil, ν - Poisson's ratio, k_0 - factor,
c - cohesion, φ - angle of internal friction,
E - modulus of elasticity of soil

In Model 3, the constitutive model used to represent the linear physical-mechanical and elastic-plastic properties of the soil is the Hardening Soil model, since it is a versatile model for soft and stiff soils [4]. The deformations are a function of soil stiffness and applied loads. It is an acceptable model for the soil types found under the building, which are sands, gravels, and silts, and it is also acceptable for undrained soils applicable to the present case, since in the time-history analysis the action and response times of earthquakes are short [10].

2.4 Structural component

2.4.1 Model 1 and Model 2

The building was built from reinforced concrete f'c 270, designed with circular columns of 800, 700 and 600 mm diameter at the front, main columns of 800x600, 700x500, 600x400 mm, and edge columns around the elevator box of 400x400 and 250x250 mm. The beams are 400x700, 400x600 and 300x600, and 250x250 mm edge beams are placed around each slab; the sections of the columns and beams are adjusted every certain height. The original slabs are Waffle type with 200 mm thickness for the roof, 250 mm in the intermediate floors and 300 mm in the subfloors, which are equivalent to a solid slab thickness of 115, 150 and 170 mm respectively. The bottom slab is 30cm and is in contact with the ground. The shear elements in the elevators are 200 mm thick and the walls are 250 mm thick.

Model 1 is made in Midas Gen and consists of 12 high floors embedded in the base of the columns. The columns and beams are Beam type, the slabs are Plate type, the elevator walls and perimeter walls are Wall type. The second floor is generated from the coordinates of the original plan, then the columns, beams and shear walls are placed. The 11 elevated floors are auto generated and the dimensions of the columns and beams are adjusted, in the same way the roof is generated with the respective dimensions.

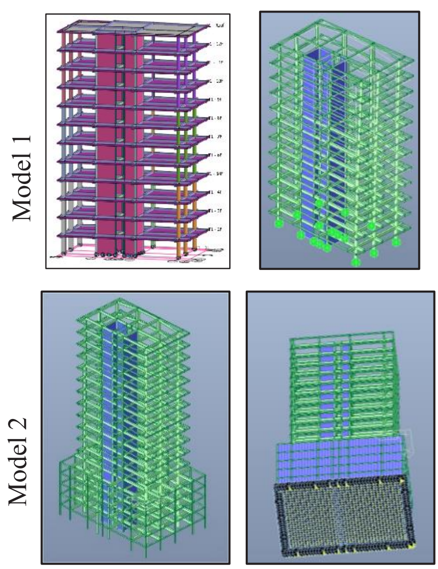
Model 2 is also executed in Midas Gen, here basements are added with their columns, beams, slabs and elevator walls. From the foundation slab, a box of perimeter walls for the subfloors is extruded (Extrude). Finally, Surface Spring is placed on the bottom slab and displacements are limited only to DZ by placing Point Springs. See Fig. 4 (a).

For Model 1, self-weight loads are considered, which include the weight of the floor slabs and roof, dead loads (DL) of finishes, masonry and installations. The loads for office use are obtained from the Ecuadorian Building Code - Non-seismic loads. NEC15-SE-CG [8], likewise the loads for Model 2 are entered, which includes as self-weight the parking lot slabs, bottom slab and perimeter walls. The loads are taken as -3.5 kN/m2 as indicated by Pazmiño and Ponce in their thesis on the analysis and design of the parking lot building of the Ecuadorian Armed Forces [9]. See the complete load table in Fig. 4(b).

For wind loading, the Wind lived on Structure type and the NSR-10 code are used for both X and Y directions, considering that the elevated floors are totally affected (dimensions: 22.2 x 32 x 43.2 m) and have an exposure to wind classified as category C of approximately 60 km per hour. Wind eccentricity is not considered. For the EQS static seismic loading for both X and Y directions, the NSR-10 code adjusted to NEC15-SE-DS is used considering the coefficient 0.145W. For the Response Spectrum function, the T vs Sa data set processed as design spectrum with NEC15-SE-DS standards is taken with the most relevant site parameters for Quito which are: Fa 1.2, Fd 1.19, Fs 1.28, Z 0.4, T0 0.1269, soil type D, I=1.0 and R=8. The resulting spectral data is imported into Midas Gen for both Model 1 and Model 2. The mass factors are 1.0 for self-weight, 1.0 for dead load and 0.25 for live load.

The most important loads for this paper are the Time History seismic acceleration records: Duzce which has increments of 0.01 s and 45 s duration (4500 stages), Landers 0.02 s and 45 s (2250 stages) and Darfield 0.005 s and 140 s (28000 stages). These records are entered in Time History Functions in Midas Gen.

Once the geometries, loads and system parameters are entered, the Eigenvalue Analysis Type: Rits Vectors of Model 1 is processed, which indicates a natural period of 1.47 s and 1.71 s for Model 2. The seismic load (EQD) allows performing a pseudo-dynamic analysis based on displacements where the maximum response of the structure is determined through the superposition of the effects of the vibration modes obtained as a function of the number of floors. The philosophy of this article is to compare the displacements that occur when the above-mentioned earthquakes and other loads typical of building design act.



Loads

Name	Load Type	Value/CODE (kN/m)	Description
DEAD	Dead Load		
DLC	Dead Load	-2,40	roof
DLS	Dead Load	-3,87	superestructure
DLPB	Dead Load	-3,87	PB
DLSub	Dead Load	-1,00	basements
LLC	Live Load	-0,70	roof
LLS	Live Load	-2,40	superestructure
LLSub	Live Load	-3,50	basements
LLE	Live Load	-4,80	elevator
LLPB	Live Load	-4,80	PB
WLX	Wind load on Structure	NSR-10	x direction
WLY			y direction
EQSX	Earthquake	NSR-10	Static seismic load X
EQSY	Earthquake		Static seismic load Y
EQDX	Earthquake	NEC15 (Ecuador)	Dynamic seismic load X
EQDY	Earthquake		Dynamic seismic load X
Landers	Ground Motion	PEER data	TH Spectrum
Duzce	Ground Motion	PEER data	TH Spectrum
Darfield	Ground Motion	PEER data	TH Spectrum

(a) (b)

Fig. 4. (a) Model 1 and Model 2 in MIDAS Gen. (b) Loads and values.

Table 5 shows the absolute maximum deformations of all load cases. Fig. 5 shows the relative horizontal displacements of floors for M2 practically start on the ground floor (GF) due to the great transversal rigidity caused by the perimeter walls.

Table 5. Absolute Maximum Displacement in (cm) for the Model 1 and Model 2.

	Model 1		Model 2	
Load Case	X	Y	X	Y
WLX	0.67	0.06	2.78	0.17
WLY	0.07	0.29	0.19	1.14
EQSX	13.63	2.00	38.51	4.25
EQSY	3.00	9.25	6.24	27.14
EQDX	7.15	2.51	8.29	1.85
EQDY	4.28	5.17	3.53	6.72
DuzceX	8.65	3.02	8.12	2.09
DuzceY	4.43	5.69	3.95	7.82
LandersX	13.97	5.31	8.18	2.09
LandersY	5.94	8.81	3.95	7.82
DarfieldX	25.05	7.57	24.04	6.07
DarfieldY	12.30	14.17	12.27	23.67

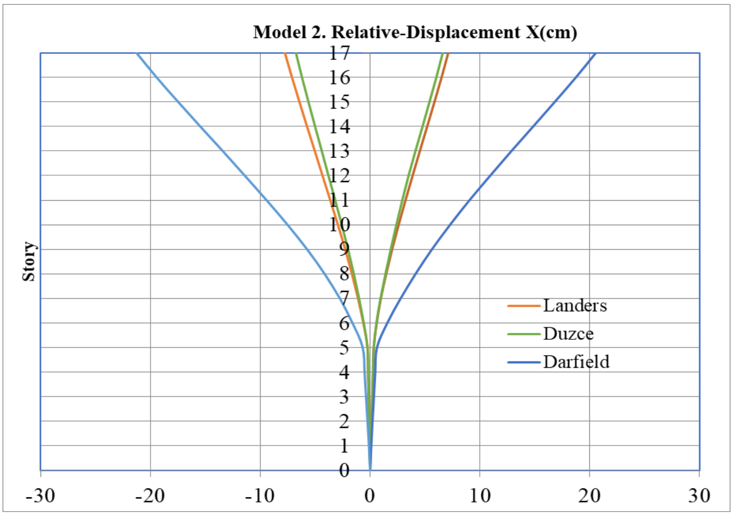


Fig. 5. Maximum and minimum relative displacements per floor - Model 2.

The development of Model 3 requires the strengths of a finite element software that considers the seismic, geotechnical, and structural components, so in the following section, Midas GTS NX is used.

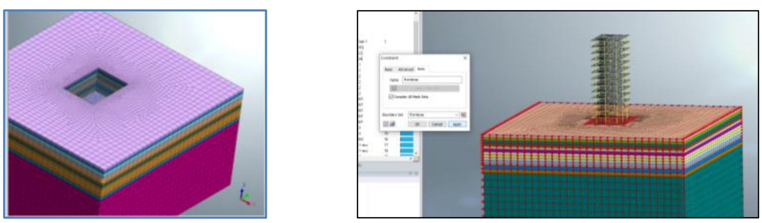
2.4.2 Model 3

It is the fusion of Model 2 with the soil simulation using Finite Element Analysis (FEA). Each discretized element is a solid with linear physical-mechanical characteristics from Table 4 which are then internally transformed by Midas GTS NX to nonlinear. To avoid the bouncing of seismic waves on the building walls and floor edges, the maximum size of the finite elements was found by Kuhlemeyer and Lysmer (1973) [10] by using the frequency range with the largest amplitude in the Fourier series such that the mesh size is smaller than $\lambda/8$ or $\lambda/10$ (where λ is the wavelength), see table 6.

Table 6. Finite Element Dimensions for Duzce.

Element	**Shear wave velocity Vs (m/s)**	**Wavelength λ (m)**	**Max. size (m)**	**Design size (m)**
Bottom slab and walls	320.0	26.66	2.66	2.4
Soil boundaries	Free field	N/A	N/A	6.40
Lower base of solids	520.0	65.00	6.50	6.40

Model 2 without the bottom slab and without perimeter walls, named M2', is exported to an mxt file. Independently in Midas GTS NX, the soil stratigraphy's are generated with dimensions: 118.2m, 128.0m, 49.0m. This soil structure is exported to CAD. Then, in a new Midas GTS NX project, first import the building (.mxt) and then the previous stratigraphy. See Fig. 6 (a). Once the Model 2' boundary conditions imported from Midas Gen are removed, and for the building to be anchored to the ground, we proceed to print the nodes that are placed at the lower ends of each column. Midas GTS NX allows the creation of the wall around the basement and the creation of the bottom slab, both shell type, as well as the boundary conditions and the interface between the wall and the ground. See Fig. 6 (b).



(a) (b)

Fig. 6. (a) Soil Stratigraphy (b) Boundary Conditions. Boundary Constraint.

Midas GTS NX supports Time-History Non-Linear (TH NL) analysis which includes material nonlinearity and is based on implicit time integration. The dynamic equilibrium equation in nonlinear time-history analysis uses the HHT-α method as implicit time integration, as for linear time-history analysis, and uses the following modified equilibrium equation. [4]

$$\frac{\partial}{\partial t}(Mv^{n+1}) + (1+\alpha_{H})[Cv^{n+1} + f^{int,\,n+1} - f^{ext,\,n+1}] - \alpha_{H}[Cv^{n} + f^{int,\,n} - f^{ext,\,n}] = 0 \quad (1)$$

To adjust the damping of the spectra, Midas GTS NX uses:

$$\text{Primary wave } C_p = \rho * A * \sqrt{\frac{\lambda+2G}{\rho}} = W*A*\sqrt{\frac{\lambda+2G}{W*9.81}} = cp * A \quad (2)$$

$$\text{Secondary wave } C_z = \rho * A * \sqrt{\frac{G}{\rho}} = W*A*\sqrt{\frac{G}{W*9.81}} = cz * A \quad (3)$$

where, $\lambda = \frac{v*E}{(1+v)(1-2v)}$ $\quad G = \frac{E}{2(1+v)}$

λ: volumetric modulus (tf/m^2)
G: shear modulus (tf/m^2)
E: modulus of elasticity (tf/m^2)

v: Poisson´s ratio
A: Element face area (m^2)

In practice, the effective damping in Midas is achieved by placing the Real Eigenvalues of the two modes of vibration where there is the largest mass participation. In Model 3, a Rayleigh-type damping ratio of 0.05 is used to control the deformations of the building in the elastic range. See Y-damping in Fig. 7.

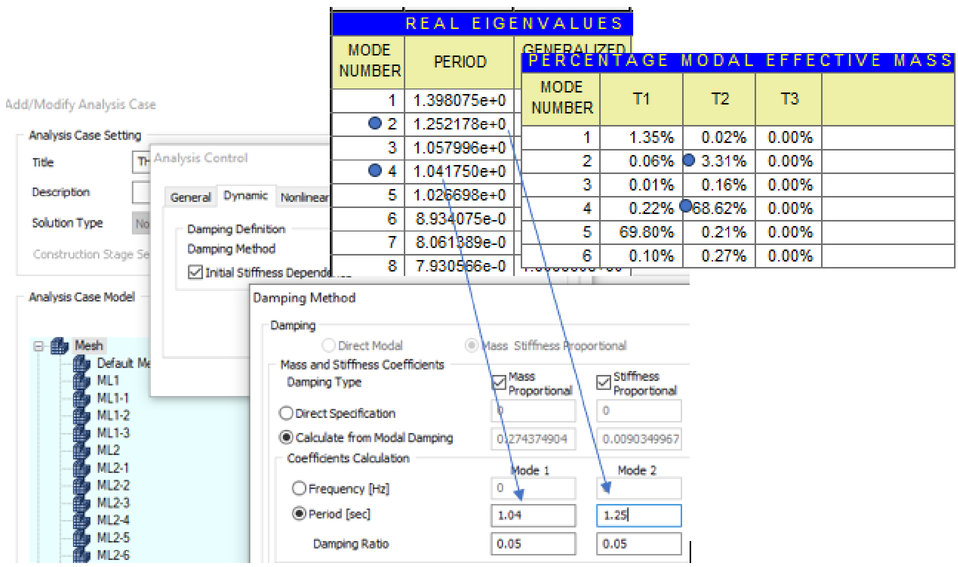


Fig. 7. Y-shaped damping, depending on the vibration modes. Model 3.

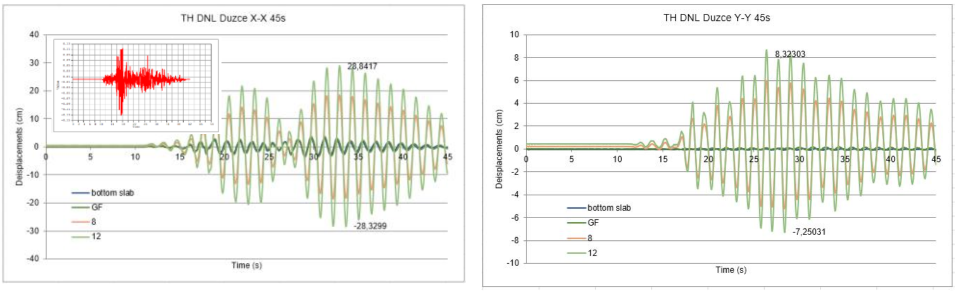
As for the boundaries, when rigid boundaries are used, the displacements of the dynamic analysis can become excessive. Model 3 includes Free Field boundaries that simulate the infinite extension of the strata by means of a system of damping springs that allow having an infinite ground condition (100 000 propagations of the strata), where it is simulated that the seismic waves continue propagating towards infinity and do not rebound within the model, additionally an Advanced Constraint is created that includes the nodes of the bottom slab and those of the base of the seismic boundaries, see Fig. 1. Finally, for the HT analysis, the three seismic records of Table 3 are included, and the analysis cases of the assembly are created. Prior to running each of the analysis cases, 17 Probe nodes are entered, one node in each floor, whose behaviour is plotted in Fig. 8.

3 Results

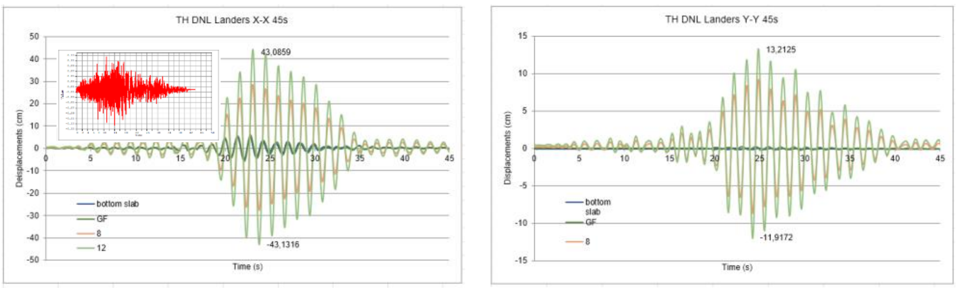
The deformation plots of Model 3 by Duzce, Landers and Dafield action in both X and Y are presented below. The natural period of vibration in Model 3 is 1.389 s. For the Darfield analysis, only 40 s are considered instead of the 140 s of its total duration because the highest seismic intensity is concentrated in the first 40 s and because the computational processing is quite demanding.

Fig. 8. Maximum Displacements per floor TH DNL Analysis with SBSI. (a) Duzce (b) Landers (c) Darfield.

Table 7. Displacements TH DNL SBSI (cm). Model 3.



(a)



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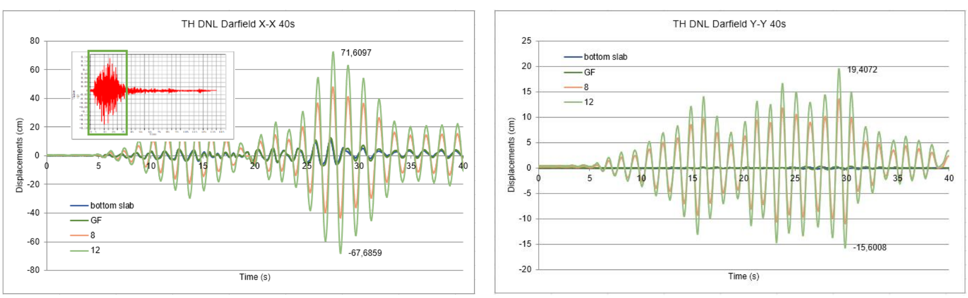

(c)

Seismic Logging	X-X Seism		Y-Y Seism	
	X-X Max	X-X Min	Y-Y Max	Y-Y Min
Duzce	28.84	-28.32	8.30	-7.25
Landers	43.08	-43.13	13.21	-12.91
Darfield	71.60	-67.68	19.40	-15.60

4 Conclusions and recommendations

In this research, three components were considered to analyze the seismic response of the 12-story building with 5 basements located in the city of Quito.

Seismic component: It is found that the earthquakes that would most affect the building are surface earthquakes occurring in the center of the city because of the fault located in the center of Quito, with a magnitude of Mw 7.0 and a depth of 8 kilometers, simulation that was considered as a hypothetical case in the TREQ Project. The results of this simulation offer the parameters that are introduced in the PEER database to choose the three most relevant for the present study. Duzce 1999, with an increment of 0.01s and 45s, Landers 1992, with 0.02s and duration of 45s and Darfield 2010, with 0.005s and 140s respectively.

Geotechnical component: It is found that the building is founded on an intermediate soil that starts with low compressibility silts of medium to firm consistency up to 10 m and then compact to very compact fine sands and gravels. From the geotechnical study, the depth, γ, μ, k0, c_u, φ_u and E of each stratum are obtained.

Structural component: Three models are considered for the structural analysis of the building. Model 1 with only the raised floors and embedded in the base of the columns. Model 2 adding the subfloors enclosed in a caisson formed by walls and bottom slab, and Model 3 including the soil characteristics that are linked to the structure formed by raised floors and subfloors.

Model 1 and Model 2 are represented in Midas Gen and analyzed with eigenvalue analysis, static nonlinear analysis and pseudo dynamic nonlinear analysis. It is found that the natural period of Model 1 is 1.47 and of Model 2 1.71s. The lateral displacements of Model 1 and Model 2 due to the action of the Duzce, Landers, and Darfield earthquakes are relatively similar, due to the lateral stiffness of the basements of the building because of the perimeter wall caisson, intermediate slabs, and bottom slab.

Model 3 is represented in Midas GTS NX and considers a Non-Linear Dynamic Analysis of finite elements. This model is built based on Model 2, with its respective static loads and anchored to the soil meshes with its strata and geotechnical characteristics. In this type of studies, it is important to define the boundary conditions, which in this case are Free Field. The displacements per floor of Model 3 compared to Model 1 or Model 2 are much higher.

The results validate the assertion that subway structures are significantly more resistant to seismic waves than their surface counterparts [11], showing that from the bottom slab to the first floor there is no deformation beyond 10 mm. However, from the first floor onwards, the displacement is greater by 3.55 times for Duzce, 5.27 times for Lander and 2.98 times for Darfield in relation to Model 2. An earthquake with Darfield characteristics would eventually affect the structure the most. See Fig. 9. The -49 m level is considered as the base point of deformations, where solids are restricted to displacement.

The building was analyzed with the most realistic conditions possible in each of the models, to compare their results and draw conclusions on the type of analysis used. Model 1 and Model 2 offer similar displacement values between them, considering static and pseudo dynamic seismic loads. The Non-Linear Dynamic Time-History analysis of Model 3 considering SBSI, on the other hand, is the most realistic analysis of a building with basements to date.

Finally, based on this research, it is recommended that buildings with basements in Quito use Nonlinear Dynamic Time-History Analysis with SBSI to analyze and design more reliable solutions in our environment that is constantly threatened by seismic events.

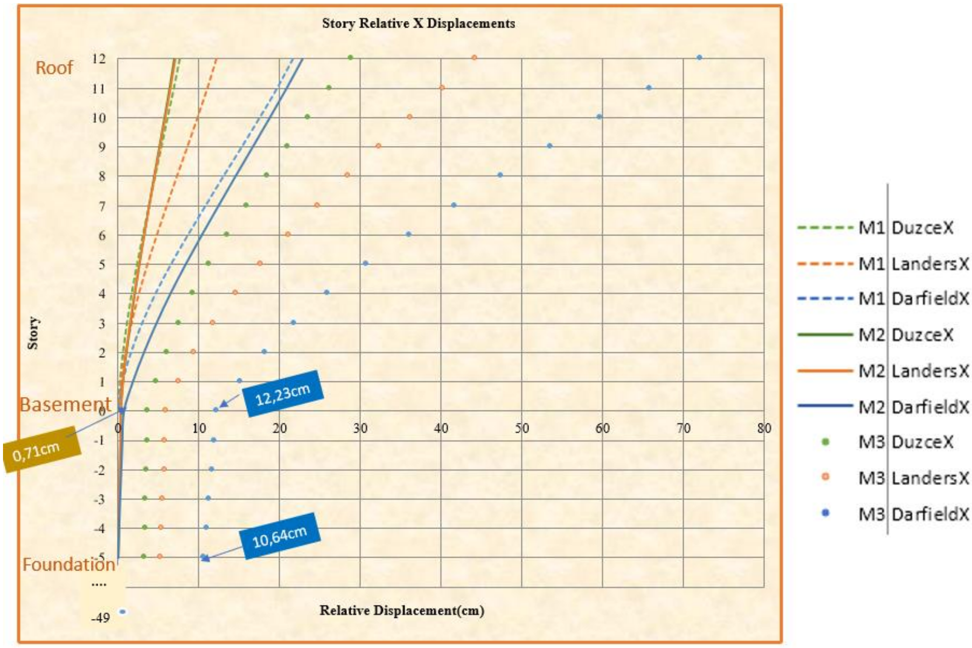

Fig. 9. Comparison of Displacements Model 1, Model 2, and Model 3.

5 Discussion

The study [12] is an experimental numerical approach to better understand the seismic response of tall buildings with basement levels, considering explicit SBSI, uses NL finite element analysis together with the results of centrifuge experiments. Their results show how seismic response parameters and modal characteristics such as story drifts, shear force, bending moment, natural frequencies, and damping ratios change when SBSI is properly incorporated. In the present case, tried to simulate the continuity of the earthquake in the soil solids with SBSI - Free Field and with Nonlinear Dynamic Time-History analysis, obtaining larger displacements in X and Y in the three earthquakes for Model 3 and likewise in the shear force, bending moments and natural periods of vibration.

In reference [13] Appendix A: Sherman Oaks Building Model Development, an SBSI modelling of a 12-story building with 1 basement and pile foundations is performed. It uses OpenSees (Open System for Earthquake Engineering Simulation) as the analysis tool and Landers Earthquake 1992 as one of the earthquakes for time-history. The study provides as a result the lateral displacement histories of the deck, 8th floor, GF and bottom slab. Since the soil conditions, the structure and even the earthquake used as reference are similar, one would have expected similar displacements produced by the Landers influence of: 12.5 cm in deck, 10 cm in 8th floor, 2 cm in GF, 2 cm in bottom slab, however in the present study, X values of 50.78, 32.98 cm, 6.59 and 5.6 cm respectively and Y values of 12.97, 9.13, 0.12 and 0.09 cm are found. The variation could be attributed to the finite element modelling used in this study.

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