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EVALUATION OF A FOUNDATION WITH IMPORTANT PROBLEMS CAUSED BY REGIONAL SETTLEMENT

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Abstract

Terminal 2 of the International Airport of Mexico City was built to provide better services and spaces to passengers after the cancelation of the original project of the New International Airport of Mexico City in 2001. This expansion is composed of a Terminal Building founded on end bearing piers and two waiting rooms, the North and South structures, founded on friction piles. The regional subsidence phenomenon, caused by aquifer exploitation, has led to an accumulation of serious differential settlement between the structures that compose Terminal 2. Given the importance of ensuring the operation of this transport terminal in the case of an urban emergency and considering its functionality during the remainder of its service life, a revision of the current condition of the foundation system of each one of the edification elements was performed based on Mexico City's Building Code. In addition, finite difference numerical models were developed in the FLAC 3D platform to analyze the dynamic soil-structure interaction in the case of an important seismic event. The results obtained are expected to facilitate decision making for minimizing the differential settlements accumulated in the structures.

Keywords: Piers, piles, foundation slabs, airport, regional subsidence, dynamic soil-structure interaction.

1. Introduction

Terminal 2 of the International Airport in Mexico City came into operation in November 2007 to increase the airport's capacity to transport passengers. Its construction and design were completed according to the Mexico City Building Code of 2004, which is still in place. This edification is composed of three main structures: the Terminal Building and two waiting rooms, the North and South Structures. The airport is located in the lacustrine geotechnical zone (Zone III) of Mexico City, where the peak ground acceleration (PGA) is approximately 0.10 g.

Aquifer exploitation for the supply of drinking water has caused the pore pressure to decrease, generating serious settlement in the zone where Terminal 2 is located. In addition to the regional subsidence phenomenon, other factors, such as the high compressibility and low shear strength of lacustrine soils, have caused an accumulation of differential settlement, which will eventually cause the functionality conditions for the users and personnel to become unacceptable.

The accumulation of differential settlement in the structure connections mainly occurs because the waiting rooms are founded on friction piles, whereas the Terminal Building presents an apparent emersion because its design includes bearing piers founded on Deep Deposits.

In this work, the safety of each foundation system of the buildings that compose Terminal 2 was verified considering the Mexico City Building Code and the Complementary Norms, which are still in place [1]. In addition, finite difference numerical models [2] were developed to analyze the soil-structure dynamic interaction and to facilitate the decision making for minimizing the accumulated differential settlement resulting from the latent regional subsidence in the area.

1.1 Characteristics of the soils of Mexico City

The majority of Mexico City is situated on lacustrine soils in which large amounts of volcanic ash and other pyroclastic material were deposited. With time, the chemical weathering of such materials resulted in highly compressible clays and a high water content, both of which are characteristic of the Valley of Mexico.

The urban area of Mexico City can be divided into three geotechnical zones [3]: the Hill zone (Zone I), the Transition zone (Zone II) and the Lake zone (Zone III). The Hill zone is composed of firm and compact volcanic soils with high amounts of gravel and well-cemented pumice tuffs. The Transition zone presents highly marked stratigraphic variations between sandy and silty-sand strata intercalated with layers of lacustrine clay. Finally, the Lake zone presents deposits of highly compressed clay and low load capacity, separated by sandy layers of firm consistency and variable thickness.

1.2 Regional subsidence

In 1947, based on the Mechanics of Soils, Nabor Carrillo explained that the regional subsidence phenomenon is due to the consolidation process that soft soils undergo when there is an increase in the effective stresses due to the decreased pore pressure in the soil. The exploitation of aquifers to supply of drinking water and water losses through subway lines, collectors and tunnels has allowed this induced consolidation process to take place, causing serious damage in the foundation systems and in the water supplying and drainage facilities.

When a structure is founded on the Hard Layer or on the Deep Deposits in an area where the regional subsidence is significant, it presents an apparent emersion effect with respect to the surrounding ground (Fig. 1). This leads to the presence of significant differential settlement with respect to the surrounding structures, loss of the foundation surface confinement, loss of verticality of the edifications and a separation between the substructure slab and supporting soil.

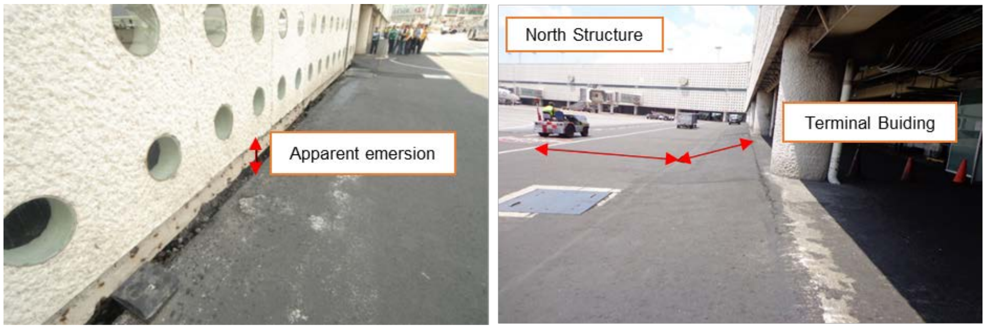


Fig. 1 – Apparent emersion and differential settlement in the Mexico City International Airport, Terminal 2

1.3 Site effects

The site effects correspond to the modification of the amplitude, frequency and duration of the seismic waves due to the topographic and geotechnical conditions of the area. Such modification mainly consists of the amplification of the seismic signal by several orders of magnitude because the clay deposits present a linear elastic behavior for large shear deformations (1%) and extremely low dampings (3-5%) for this degree of deformation [4].

Studies performed by Romo, Magaña and Bárcena (1990) show that the nonlinear response of a clay deposit significantly depends on the G/G_{max} vs. γ and ξ vs. γ curves of the clays of which it is composed.

In contrast, Romo and Jaime (1986) and Romo and Seed (1986) demonstrated that the main aspects of the seismic motions can be reproduced by a simple shear-wave vertical propagation model. Such a model allows the free-field motions to be predicted via unidimensional models within the Valley of Mexico with sufficiently high precision for practical applications.

2. Problem

Terminal 2 of the Mexico City International Airport is currently experiencing differential settlement of approximately 80 cm, which has been exacerbated by the regional subsidence phenomenon (Fig. 2). The Terminal Building experiences an apparent emersion with respect to the surface of the surrounding ground, whereas the North and South structures follow the behavior of the soil subjected to the induced consolidation process. Such settlement can be attributed to the following three main factors:

- The high compressibility and low shear strength resistance of the lacustrine deposits on which the edification is constructed.
- Incompatibility in the behavior of the foundation systems.
- The regional subsidence phenomenon.

(a) (b) (c)

Fig. 2 – Current conditions of Terminal 2: (a) Terminal Building-South Structure connection at the ground-floor level; (b) Access aisle at the ground-floor level, South Structure; (c) Terminal Building-North Structure connection at the ground-floor level.

2.1 Characteristics of Terminal 2

Terminal 2 is composed of three main structures: the Terminal Building and the waiting rooms, the North and the South Structures (Fig. 3).The Terminal Building has four mezzanine levels, yielding a total height of 30.8 m. The foundation of this structure is reinforced concrete and it is composed of a ground floor slab with grade beams and 87 piers with a circular cross-section (56 of ϕ=1.2 m and 31 of ϕ=1.5 m) grounded at a depth of 57 m and embedded 3 m in the Deep Deposits.

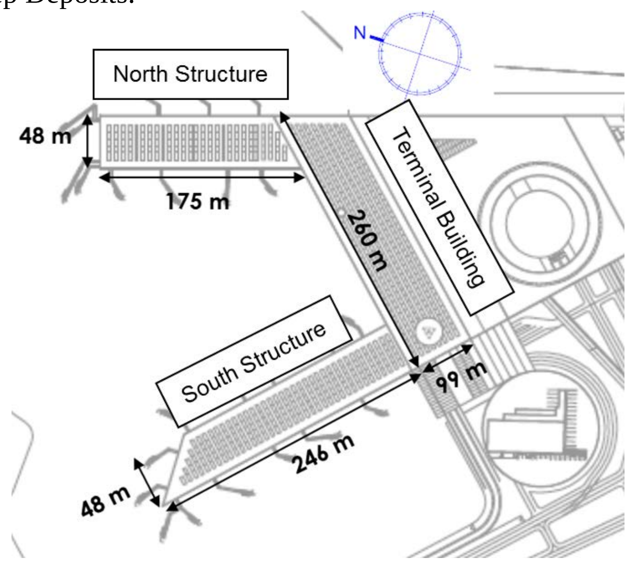


Fig. 3 –Plant localization of the main buildings of Terminal 2

The North and South Structures are composed of moment-resisting steel frames; they have two mezzanine levels and a maximum height of 15 m. The foundation system of both edifications is reinforced concrete composed of a ground floor slab with grade beams without a bottom slab. In addition, these structures are supported on square friction piles with dimensions of 0.4 m × 0.4 m and a length of 30 m. The North structure has 400 piles, whereas the South structure has 528 piles.

3. Stability analysis of the foundations

3.1 Limit state of failure and service

Compliance with the inequalities described in the Mexico City Building Code and the current Complementary Technical Norms [1], it was considered to revise the limit state of failure and service of the foundation systems of the edifications that compose Terminal 2. This verification was performed for both static and dynamic conditions (Fig. 4).

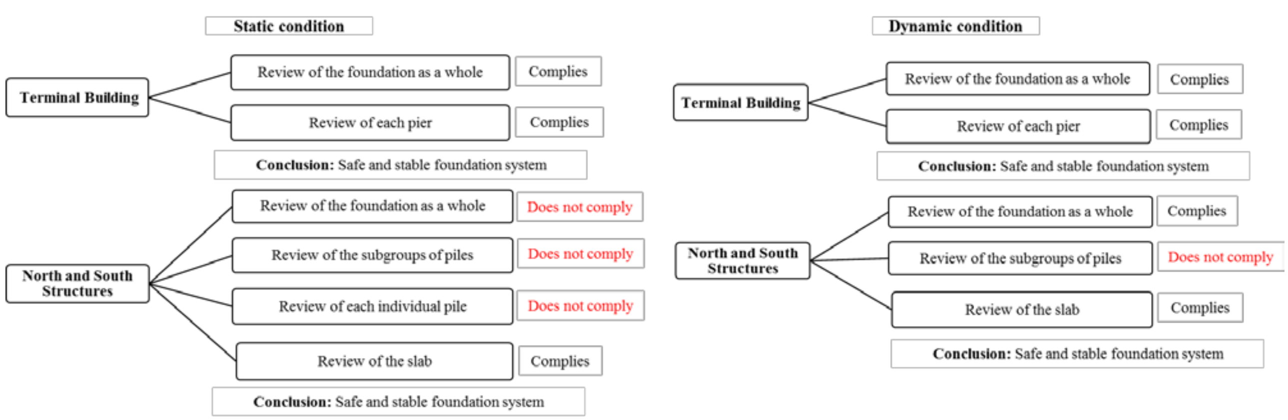


Fig. 4 – Verification of the foundation systems for static and dynamic conditions

To analytically perform the revision of Terminal 2 under dynamic conditions, an equivalent horizontal force was estimated to represent the seismic action. To determine this horizontal force, the design spectrum of Terminal 2 was built considering the fundamental vibration period of the soil (3.2 s < T_s <3.3 s) and the specific characteristics of the structure.

Fig. 4 illustrates that Terminal 2 does not have load capacity problems and that the foundation systems of the structures composing the terminal, provide a stable and safe behavior. However, when the limit state of service was revised by the calculation of the immediate and deferred settlements and the apparent emersion, none of the structures was found to comply with the maximum limits defined in the Complementary Technical Norms for Design and Construction of Foundations [1].

In conclusion, when reviewing the foundation systems for the limit states of failure and service of the North and South Structures, the foundation slabs are the elements that provide stability to the structures, and the friction piles contribute by reducing the amount of settlement. The piers of the Terminal Building are responsible for safeguarding the structure against static and dynamic conditions.

3.2 Regional subsidence

The elasto-viscoplastic (EVP) model by Yin and Graham (1994, 1996) [5] was used to evaluate the future effects of the regional subsidence and the rate at which it will increase over the next 5, 10 and 15 years in the zone where Terminal 2 is located. In addition, the IINCON program developed by Ossa (2004) [6] was used to solve the differential equations of this model considering the available piezometric and topographic information.

The points of analysis in which the regional subsidence effects were evaluated were precisely where the greatest differential settlements problems currently occur, i.e., at the Terminal Building-North Structure and Terminal Building-South Structure connections. According to the topographic leveling performed in April 2014, such differential settlement is approximately 80 cm.

The pore pressure distributions obtained for the points of analysis, North and South, after validating the results of the numerical model with the topographic leveling performed at the site between August 2010 and November 2012, are shown in Fig. 5.

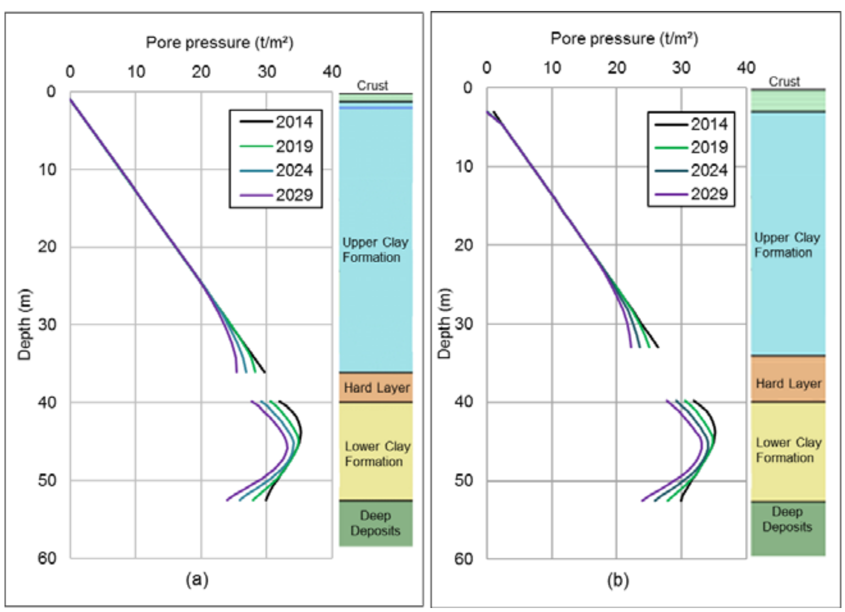


Fig. 5 –Evolution of the pore pressure in the North Point (a) and South Point (b) due to the regional subsidence phenomenon

Once the pore pressure distributions were obtained for each of the points of analysis, the settlements were calculated for the next 15 years of the structures' service life starting from 2014 (Table 1).

Table 1 – Accumulated settlements calculated using the EVP model

Point of analysis	Settlement (m)		
	2019	2024	2029
North	1.09	1.43	1.71
South	0.98	1.29	1.53

Table 1 illustrates that the differential settlements will continue to accumulate over the next 15 years if the rate of pore pressure decline in the permeable boundaries remains constant. This increased accumulation must be considered when making decisions regarding the rehabilitation and maintenance process at these critical points.

4. Numerical analysis

Finite difference 3D models were developed in the FLAC 3D program to evaluate the current dynamic behavior of the foundation system of Terminal 2 [2]. Only the analysis of soil-structure dynamic interactions of the North Structure are presented in this work given the similarities in the geotechnical conditions and structural characteristics of the North and South Structures.

4.1 Characterization of the seismic environment

To define the seismic environment of the zone where Terminal 2 is located, some of the seismic events recorded in the nearby stations were used to consider the variability of the characteristics of the seismic waves and to obtain a response spectrum that is realistic and consistent with what has been measured *in situ*. The Hangares accelerograph station was selected to define the seismic environment of the zone because of the quality of its records and its proximity to the study site (450 m).

The envelope of the response spectrum of the seismic events was obtained by scaling a normalized spectral shape using peak ground acceleration (PGA) as a factor to remove the motion intensity factor recorded in rock at the site [7]. Fig. 6 shows the normalized and scaled response spectrum for the North-South (NS) and East-West (EW) components with 5% damping. All of the response spectrum were scaled at a PGA of 0.15 g, which corresponds to the design spectrum of Zone III for a Type A structure [1].

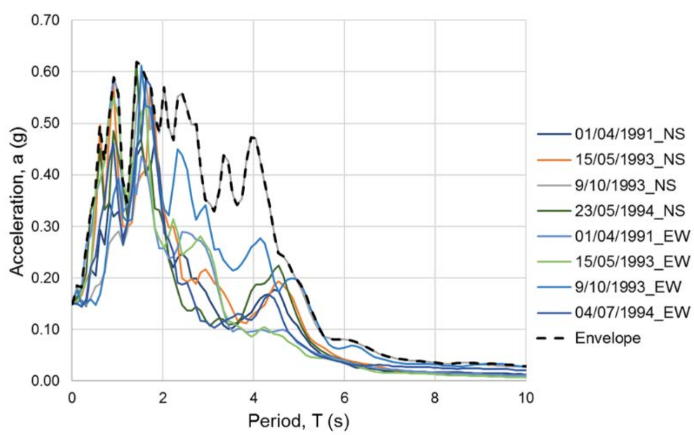


Fig. 6 –Design envelope for the NS and EW components

4.2 Input motion

The RSPTMatch99 program [8] by Abrahamson (1993) was used to obtain a time history with a response spectrum that would approximately fit the envelope of the response spectrum considered. This program implements the algorithm described by Lilhanand and Tseng (1987, 1988) to modify an acceleration history seed in the time domain and make it compatible with the specified reference spectrum. The acceleration history seed used corresponds to that of the NS component of the seismic event of October 1993.

The decreasing cosine function of Abrahamson (1993) was used to perform a first fit, and then, the function of Lilhanand and Tseng (1987, 1988) was used to refine it. The synthetic accelerogram corresponding to the best-fit response spectrum is shown in Fig. 7.

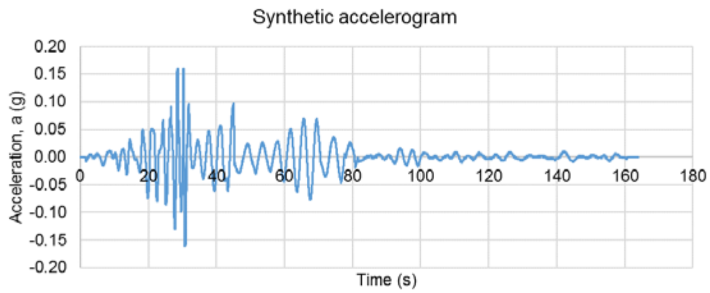


Fig. 7 – Synthetic accelerogram

An accelerogram typically contains the entire acceleration history of a seismic event. However, for engineering purposes, only the most intense part of the motion is of interest. Therefore, the criterion of Arias (1970) was used to define the synthetic accelerogram interval in which most of the seismic event energy is contained. Such a criterion considers that the time interval is limited by the points at which 5% and 95% of the recorded total energy take place. In this manner, the 164 s duration of the original synthetic record is reduced to 54 s, representing a 67% decrease.

The dynamic behavior and soil-structure interaction of the North Structure was analyzed using the FLAC^{3D} program [1], where the input motion should be applied at the base of the 3D model. Thus, the SHAKE 91 code [9] is used to take the synthetic acceleration history, obtained from a surface signal, to the base of the model. The curves of degradation of the shear modulus, *G*, and of damping ratio , ξ, of Vucetic & Dobry (1991) [10] for the clay materials and of Seed & Idriss (1970) [11] for the sandy materials were referenced in the program to consider the non-linearity of the soil. These curves are presented in Fig. 8.

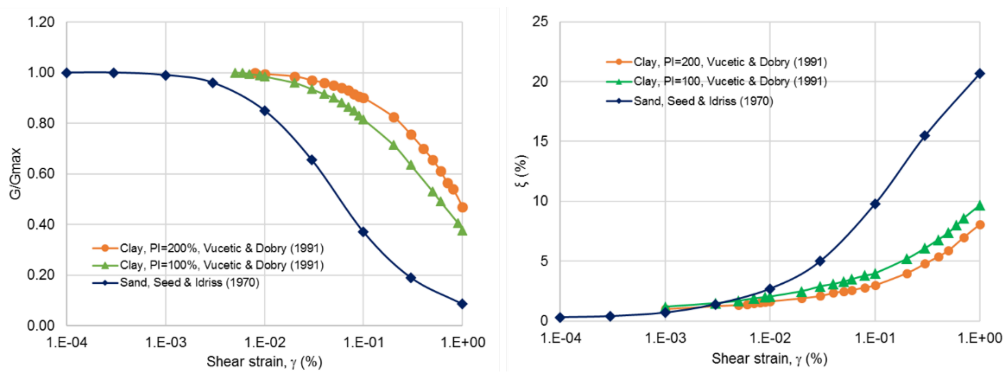


Fig. 8 – Degradation curves of the shear modulus G and damping ξ

The maximum shear modulus, G_{max}, to characterize each substrata was obtained from the shear wave velocity, V_s, as presented in Eq. (1).

$$G_{\max} = V_s^{\,2} \rho \tag{1}$$

where ρ is the soil density.

To properly represent the soil elements in the FLAC^{3D} program [2], the parameters compatible with the degree of deformation that induces seismic motion in the deconvolution process in the SHAKE 91 code [9] were considered. Table 2 provides the values of the shear modulus and bulk modulus, *K*, equivalent for each soil stratum compatible with the degree of deformation induced by seismic motion in the deconvolution process in the SHAKE 91 code [9]

To evaluate the bulk modulus, *K*, Eq. (2) was considered using a Poisson's ratio, ν, of 0.30 for granular materials and of 0.45 for clay materials of the considered soil deposit.

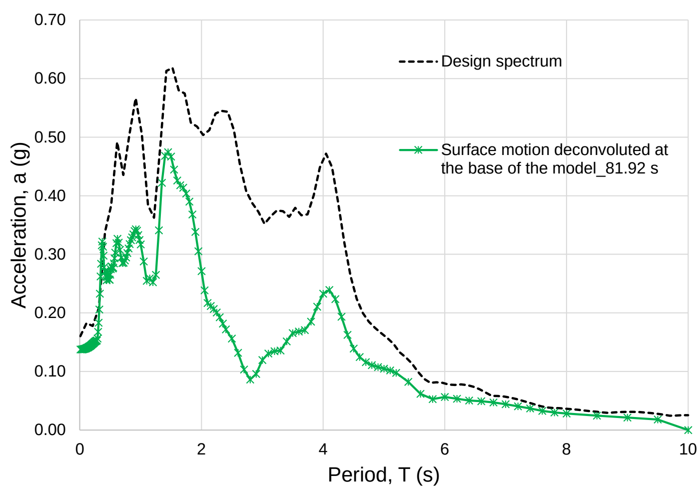
$$K = \frac{2G(1+\nu)}{3(1-2\nu)} \tag{2}$$

where G is the shear modulus.

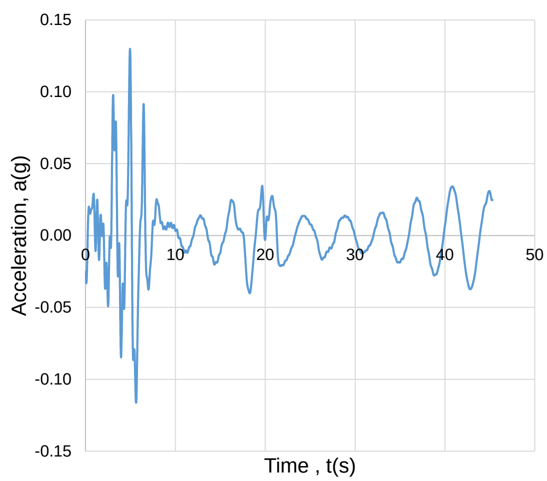
Table 2 – Resultant parameters of the wave propagation analyses

Stratum	Shear Modulus, G (kPa)	Bulk Modulus, K (kPa)
1_Crust	14196	23661
2_Upper clay formation	4713	42329
3_Hard layer	84520	220416
4_Lower clay formation	12958	125264
5_Sand lens	6001	15651
6_Lower clay formation	12645	12223
7_Deep deposits	839738	1819432

According to the Arias intensity criterion, only the intense phase of the deconvoluted motion at the base of the profile was considered, in order to decrease the calculation time requirement. Therefore, a window of 45.28 s of the total duration of the deconvoluted motion was applied at the base of the 3D model in FLAC^{3D} [2] (See Fig.9).



(a)



(b)

Fig. 9 – Input motion (a) response spectrum and (b) acceleration history

4.3 Modelling in FLAC^{3D}

The finite difference FLAC^{3D} program [2] allows the dynamic interaction between the structure and soil to be directly evaluated. The proper representation of the dynamic systems and the validity of the results obtained through the program depend largely on the boundary conditions, the selection and application of the dynamic actions, the constitutive models and the damping of the material of the soil elements.

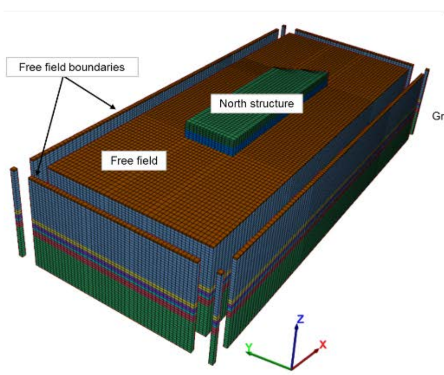
The model dimensions were defined according to the geometry of the North Structure and the distance require for reducing the potential refractions waves in the half-space because of the boundaries. Soil elements were modeled with a linear elastic constitutive model but its stiffness parameters were obtained from the deconvolution process. The parameters are consistent with the strain level induced by the seismic motion according to the given stiffness degradation and damping relationships [12].

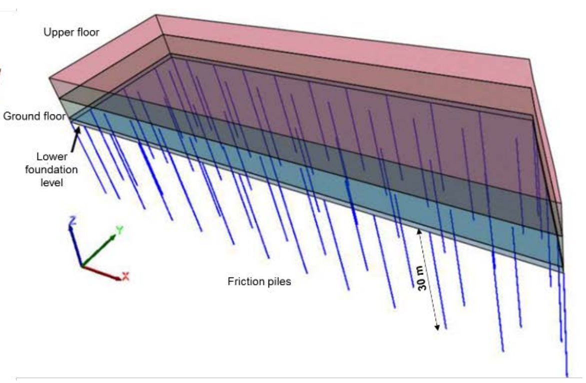
A local damping was considered for the soil elements to perform the soil-structure dynamic analyses. Such damping is specified by the local damping coefficient, α_L, which is related to the critical damping by the following equation:

$$\alpha_L = \pi\xi \tag{3}$$

In the generation of the initial stress state, the displacement components were constrained at the base and at the lateral sides. The Then, once the initial stress state was generated, the constraints on the displacements were released, and the dynamic boundary conditions were assigned, as presented in Fig. 10 (a). Fig. 10 (b) presents the structure of the North Structure constituted by regular solid elements to which weight and stiffness properties, equivalent to those of the real edification, were associated. That construction has two mezzanine levels, for a maximum height of 15 m, and a lower foundation level at -1.5 m. Finally, the beam elements implemented in the program were used to model the 400 friction piles, which are also part of the foundation system of the North Structure. The settlement obtained due to the building construction was 11 cm.

In this finite difference numerical model, it was analyzed the dynamic soil-structure interaction only for the current condition of the foundation, structure and soil deposit system under the action of an important seismic event. Therefore, consolidation analysis were not performed in FLAC 3D due to the purpose of this analyses is determinate the response at present day of the soil foundation system.





(a) (b)

Fig. 10 – Complete 3D model (a) dynamic boundary conditions and (b) North Structure

A series of preliminary analyses only considering the soil deposit and applying the acceleration history in the X direction (100 Sx) were performed to properly reproduce the seismic motion in the finite difference 3D model. That model was adjusted until the free-field response was consistent with the one obtained by the SHAKE 91 code [9]. The surface response spectrum obtained from the application of the seismic motion at the base of the finite difference model and at the base of the stratigraphic profile of the SHAKE 91 code [9] are shown in Fig. 11. The responses of both FLAC^{3D} [2] and SHAKE 91 [9] do not cover the entire frequency content of the reference envelope because of the window of only 18 s of the seismic event deconvoluted at the rocky outcrop that was considered. The most significant differences between the spectral ordinates of the numerical model and the unidimensional shear wave propagation code are between 0 and 0.5 s. This can be associated with the local damping assigned to the soil elements of the finite difference model. In general, an acceptable fit was achieved between the three spectrums.

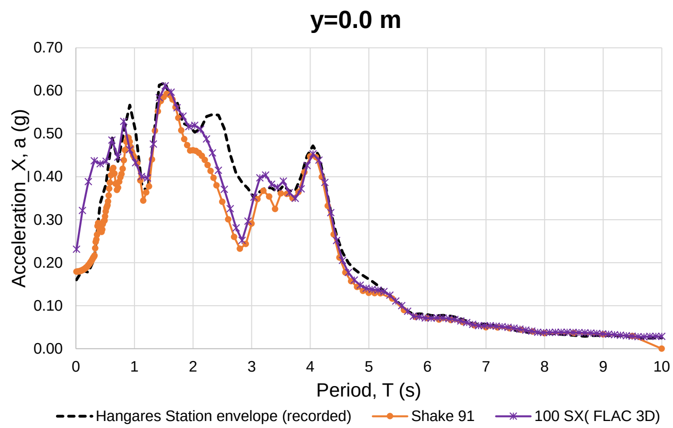


Fig. 11 – Surface response spectrum of the numerical and SHAKE 91 models for a seismic motion of 45.28 s

4.4 Soil-structure dynamic interaction

To understand the behavior of the North Structure in the case of a seismic event and its interaction with the environment on which is founded, the following combinations were considered:

- SX + 0.3 SY (100% of the seismic motion intensity in the X direction and 30% in the Y direction)
- SY + 0.3 SX (100% of the seismic motion intensity in the Y direction and 30% in the X direction)

The structure fundamental period of vibration, *T*, is equal to 0.6 s. This value was determined by field measurements of the environmental seismic noise at diverse points of the building [13]. In contrast, the fundamental period of the soil ranges from 3.2 to 3.3 s according to the results obtained from the suspended probe [13]. In this case, the pore pressure excess do not produce a critical condition in the soil structure system.

To establish the magnitude of the soil-structure interaction and determine the influence of the edification and its foundation on the soil response, free-field response spectrum and spectrum from inside the structure were obtained. Fig. 12 presents the surface response spectrum, and Fig. 13 presents the spectrum at the foundation level (-1.5 m) for each of the aforementioned combinations. The reference envelope of the seismic motion recorded at the Hangares station is also presented.

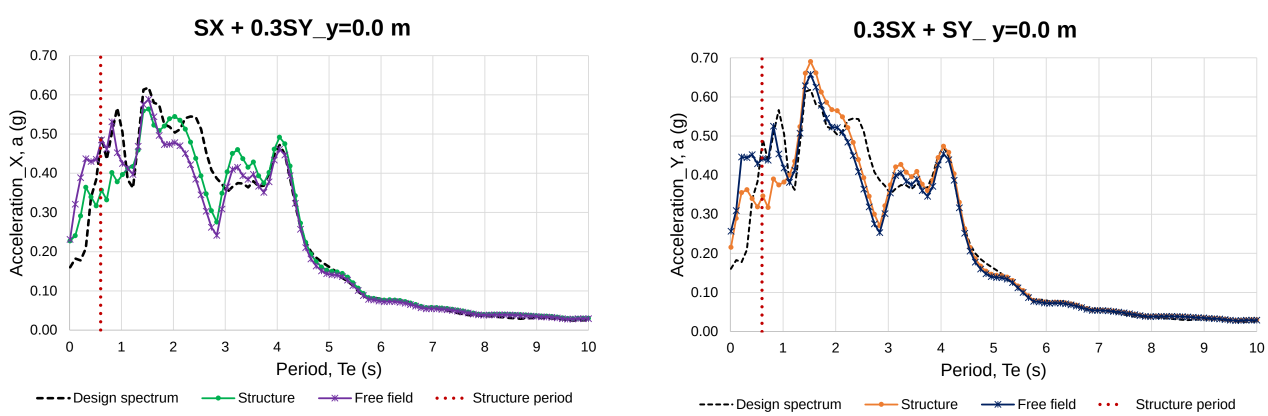


Fig. 12 – Response spectrum at a depth of y=0.0 m

Between 0.0 and 1.0 s, the amplitude of the spectral accelerations decreases by 20%, possibly due to the stiffness contrast between the soil and structure. Differences mainly occur between 2.0 and 3.0 s because it was applied the intense phase of a 45.28 s motion of the deconvoluted seismic motion lasting 81.92 s. However, between 3.0 and 10.0 s, the response spectrum obtained from the numerical model for both combinations has a similar shape and magnitude for the spectral ordinates as the reference response spectrum.

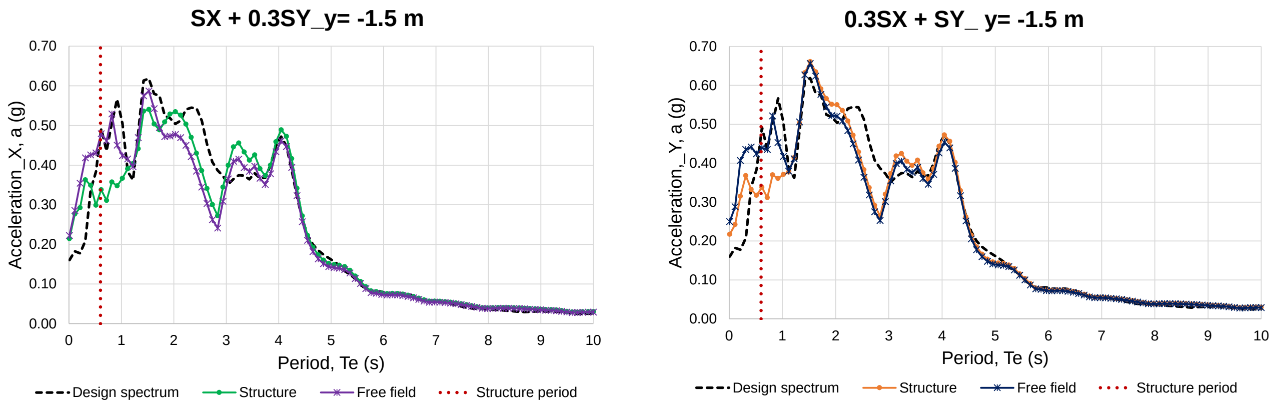


Fig. 13 – Response spectrum at a depth of y=-1.5 m

When the soil-structure interaction was considered for the SX + 0.3SY combination, the peak accelerations increased from 0.54 g to 0.60 g at 1.5 s. However, in the 0.3SY + SX combination, the peak acceleration did not shift and is approximately 0.67 g for both depths.

When applying the seismic motion in the Y direction, the response spectrum presented spectral ordinates 9% higher than those obtained by applying the seismic motion in the X direction between 0.82 and 2.53 s. This discrepancy is associated with the particular extension and geometry of the structure, which makes it more vulnerable to the application of the seismic motion in the Y direction.

The design spectrum considered exhibits a ground acceleration (T=0.0 s) of 0.16 g. The ground acceleration is between 0.22 and 0.26 g for the response spectrum obtained from the finite difference model for both combinations. This difference in the ground acceleration is associated with the type of damping assigned to the soil elements of the model.

In conclusion, the North Structure is a low height structure with a short fundamental period of vibration (less than 1 s) that will present little damage if an intense seismic event takes place because it is a highly rigid structure founded on a highly compressible soil deposit whose period ranges from 3.0 to 4.0 s ($T << T_s$).

5. Conclusions

In this work, the current condition of the Terminal 2 foundation system was evaluated according to the Mexico City Building Code. In addition, the dynamic behavior was evaluated against a representative seismic motion of the site through a finite difference numerical model.

The structures that compose Terminal 2 of the Mexico City International Airport do not present load capacity problems under static and dynamic conditions. However, they present important differential settlement problems due to incompatibility in the behavior of the foundation systems with the latent regional subsidence phenomenon in the area. Therefore, the current connection of the buildings should be modified to avoid the accumulation of such differential settlements over the next 15 years, thereby guaranteeing the functionality of Terminal 2 in the event of an urban emergency.

In the area where Terminal 2 is located, the site effects are important, mainly due to the material properties and the thickness of the clay deposits. This could be observed in the frequency content of the envelope of the response spectrum for seismic motions recorded in the field.

The North Structure is a rigid structure that will present little damage if an intense seismic event takes place during its operational life because its fundamental vibration period is significantly lower than the period of the lacustrine deposit on which it is founded.

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