Inelastic Response of 3D RC Frames of a Urban Viaduct Considering the Interaction Soil-Structure Effect by Numerical Simulation

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SUMMARY:
Bridges and viaducts are subjected to considerable forces during strong ground motions that might cause major structural failures, damage and, in some cases they can lead to the collapse of the structure; for example the Loma Prieta, Northridge, Kobe and Chile earthquakes. Therefore, the performance of the viaducts has a particular interest of the structural behaviour, response, bridge design and retrofit. Then this research is focused to study the non-linear behaviour and structural response of a viaducts type frame of concrete placed in the soft soil of the Mexico Valley. To determine the response, the seismic simulation was done by numerical modelling with 3D non-linear analysis taking into account: the geometrical characteristics of the structure and non-linear behaviour of the material, as well as the soil-structure interaction effect is incorporated, using real and synthetic acceleration records. Finally, these numerical results show a behaviour and response expected according to the Mexican codes.

Keywords: bridges, non-linear behaviour of frames, inelastic response, structural behaviour, numerical modelling, interaction soil- structure

1. INTRODUCTION

The vehicular transportation has been increased in the Mexico City in the last two decades. Then additional transportation lines have been created how one alternative to solve the traffic problem in the city. As result of this solution the bridges and viaducts were adopted, these types of structures represent complex structural systems. Those systems choose for the structural characteristics are constituted by concrete rigid frames of two levels. The structures are placed in high or moderate seismic areas, because they restrict the lateral displacement.

1.1. Problem statement

The San Antonio Viaduct is part of elevated roadway systems linking Miguel Alemán Viaduct with the south Periférico. It is located in south-west of the Mexico City. The double-deck viaduct carried four lanes of southbound traffic on the lower level and three lanes of southbound traffic on the upper level see Figures 1.1 to 1.4. The upper and lower level box girder roadway deck were supported by structure system of reinforced concrete. This viaduct has three different configurations such as, a. isolated column, b. high frame and c. double frame. In this study, only c. the double frame is considered to be studied in this work. The frames studied are structure with circle columns for both levels and different box beams at each level.

1.2. Objective and scope

This study investigates the potential damage that they could suffer during strong motions. Then to known the inelastic damage of the structure the ANSYS program was used for this study. The structures were analyzed by step-by-step methods. The requirements of RCDF and NTC-2004 were used for the analysis.

1.3. Structure studied

The upper and lower level box girder roadway deck were supported by structure system of reinforced concrete. Type double frame consisted essentially by two portal frames, one on top of the other linking by rigid joints and other parallel frame with longitudinal rigid beams. The frames studied are structures with circle columns for both levels and different box beams at each level. Typical dimensions of the frames are shown in Figures 1.1 to 1.6.

Figure 1.1. Viewing of the San Antonio Viaduct

Figure 1.2. Viewing of the double frame structure

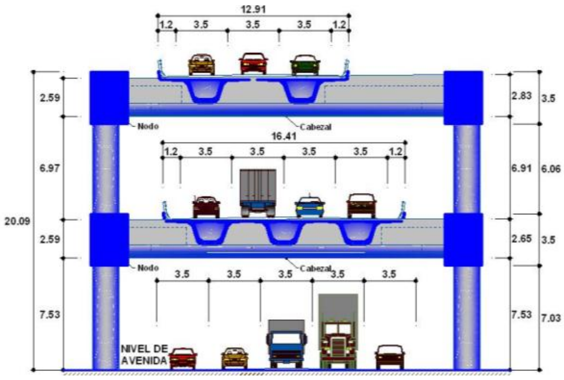


Figure 1.3. Configuration of the main frame with two levels

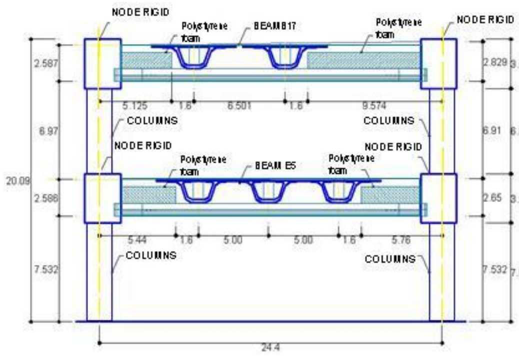
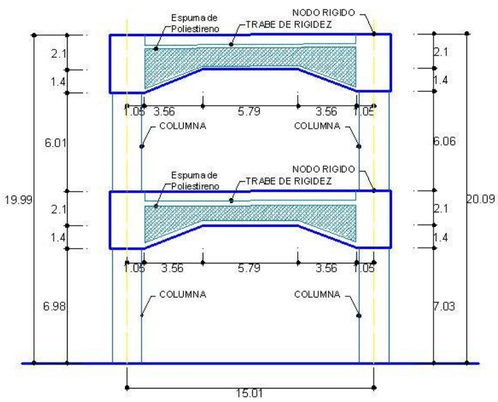


Figure 1.4. Typical geometrical characteristics of the frame studied



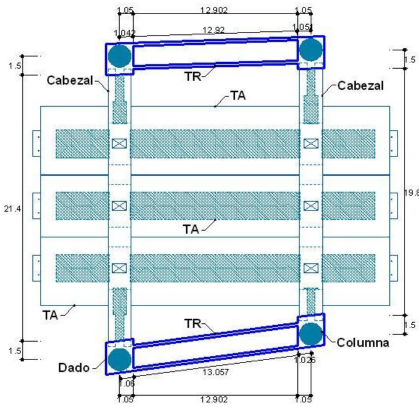
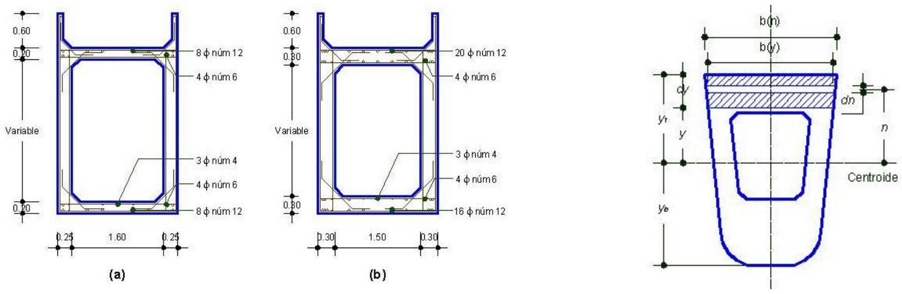


Figure 1.5. Rigid transversal frame **Figure 1.6.** Typical geometrical characteristics of the structure studied

The columns are a circle form of constant section (diameter 1.80 m) for both levels, and the transverse box-girder (supporting longitudinal pre-cast beams); these beams are box sections with different dimensions. The connection between the first-level and the upper level columns is considered a rigid node. The sections of the box-girder are shown in Figure 1.7. The dimensions and mechanical characteristics of the materials of the columns, box girders, tapered beams and girders of the foundation were taken of Cruz, (2008).



Tapered box girders (a) second level (b) first level (c) box girders of the main frame
Figure 1.7. Typical transversal section of the box girders

2. NUMERICAL MODEL OF THE STRUCTURE

In this part is described a modelling procedure for the analysis by FEM to simulate the nonlinear behaviour and its response of the 3D RC frames with beam elements. The constitutive model of the materials and the real geometry were included and for the soil-structure interaction case the characteristic of the soil is modelled together. Seismic record of seduction originated in the Mexican Pacific Ocean has been used by a time history analysis. Finally, the results obtained of the real structures attempt to be considered to determine new design, as well in the regulations environment in our country.

2.1. Modelling of the RC frames with couple columns

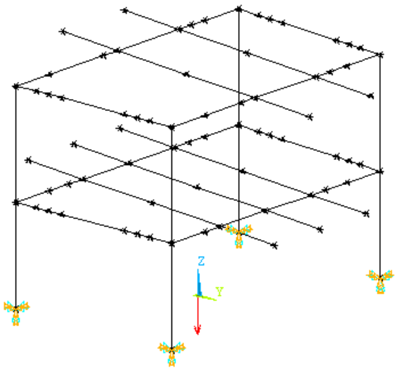


Figure 2.1. Numerical models in **3D** with rigid foundation

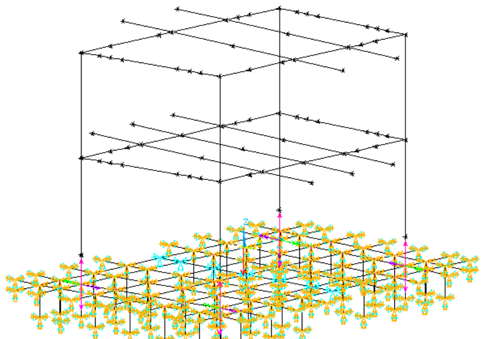


Figure 2.2. Numerical models in **3D** with flexible foundation (soft soil)

The concrete frames were modeled as an assemblage of prismatic and tapered members as shown in Figures 2.1 and 2.2, the columns were modeled as vertical elements and box-girder as the transverse elements, and all joints were assumed to be rigid. The FE model of the frames was based by kinematic displacement which was a structural idealization of subsystems of the all structure to represent the geometry of these frames. Then the frames were composing by beam elements interlinks with nodes to determine the kinematic displacement produced for static and dynamic excitation. The discretization of the frames was presented in Figures 2.1 and 2.2. The concrete elements as columns and prismatic box girders and tapered beams were modelled as beam elements with Beam 188 element see Figure

2.3. This type of beam element has a linear or quadratic node, with six degrees of freedom at each node; these include translations in the x, y, and z directions and rotations about the x, y, and z directions. Special features of the Beam 188 element are the elasticity and isotropic hardening plasticity models. Although this beam element is well-suited for linear, large rotation, and/or large strain nonlinear applications, includes stress stiffness terms. The stress stiffness terms provided enable the elements to analyze flexural, lateral and torsional stability problems.

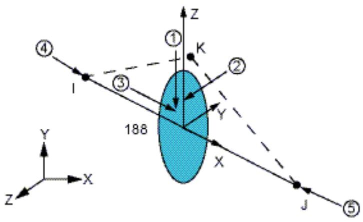


Figure 2.3. Beam188 element

2.2. Meshing of the structure and boundary conditions

Table 2.1 shows the characteristics of the numerical models and the meshing built with Beam 188 element are employed for determining the displacement for two boundary conditions cases, rigid foundation and interaction soil-structure.

Table 2.1. Meshing with beam 188 element and boundary conditions

Case of study	Boundary conditions	Nodes	Elements	Finite Element
Rigid foundation	Built-in	226	363	Beam 188
I S-S	Springs	88	70	Beam 188

2.3. Constitutive model

The simplified stress-strain curve for beam model is constructed from four points connected by straight line (MISO material model). The first branch 1-2 start with an initial slope of the curve represented by the elastic modulus of the concrete material $\mathbf{E_c}$, the second branch (curve 2-3) start at point 2 at $0.5f_c'$ and the slope is calculated by the stress and strain relationship of the concrete, point 3 represent the maximal stress of the concrete (yield stress 40 MPa) which is obtained for the maximum strain $\epsilon_{cu} = 2f_c'/E_c$, and finally the point 4 in this work is considered as a perfectly plastic behaviour, see Figure 2.4, Park et al (1983) and Scott et al (1982).

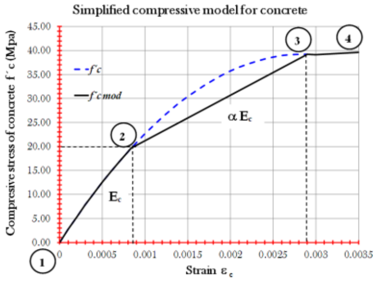


Figure 2.4. Stress strain relationships curve of concrete

2.4. Determination of the capacity of structural members

The computing of the beams and columns capacity was obtained through interactions diagrams and yield bending moment. The procedure involves the nominal resistant of the concrete and steel percentage of the sections the box-girders and columns (Figure 1.7). The nominal resistant (M, P) of the columns and beams were determined to obtain the maximal capacity. The Figure 2.5 shows the scheme of the yield surfaces (nominal moment of box-girders and interaction diagram of column).

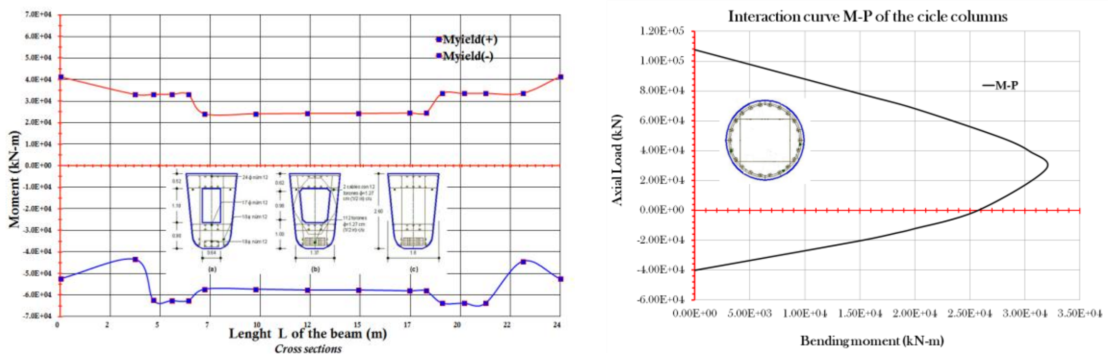
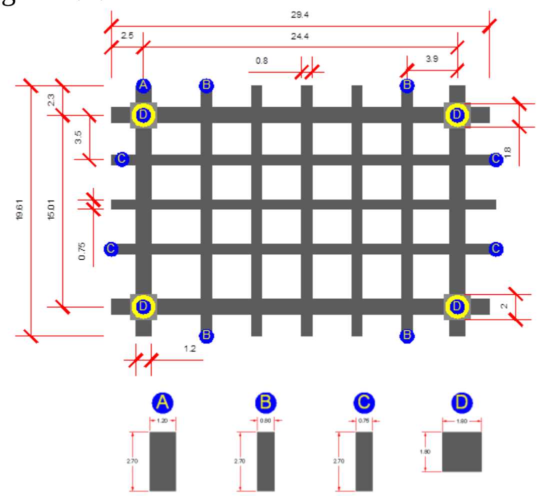


Figure 2.5. Capacity of the structural members, box girders (a) and circle columns (b)

2.5. Characteristics of the foundation and soft soil

The structure studied is placed in the geotechnical zone II (transition soil), NTC-DS, (2004), the foundation is composed by a grid of RC girders of foundation as show in Figure 2.6 and the grid is supported by piles. The characteristic of the soft soil was taken of the Santoyo, (1996) geotechnical report, place near to the bridge, with a general stratigraphy and velocity profiles of the 90m of deeper see Figure 2.7.



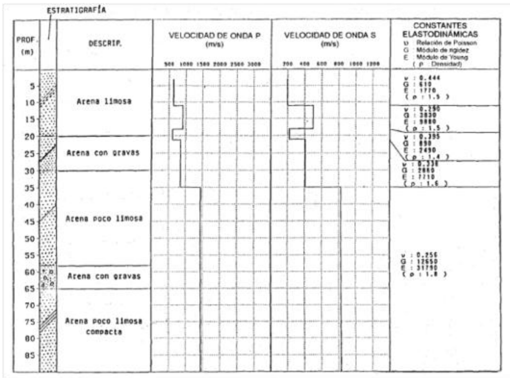


Figure 2.6. Foundation of the structure. **Figure 2.7.** Stratigraphy and velocity profiles well no.14

2.5.1 Equivalent system with soil-structure effect

The structure was modeled as a 1DOF oscillator with rigid foundation and flexible base. The structure is considered supported by a superficial foundation infinitely rigid with 2DOF, one of translation and other of rotation or rocking respectively. The foundation is supposed to be placed on a deposit of soil horizontally stratified, responding as 1DOF oscillator and the deposit of soil has a homogenous stratum, so that the soil structure system is replaced for an equivalent system, see Figure 2.8. So, this model is represented as a unimodal approach and the mass M_e, stiffness and damping effectives ζe can be replaced by fundamental mode parameters and H_e as a height to centroide of the inertial loads.

Then, the effectives, period $\tilde{T}_e$ and damping $\tilde{\zeta}_e$ are computed using equations 2.1 to 2.5 to obtain by iterative solution the dynamical impedances of the soil, to carry out this process is considered the initial conditions such as static stiffness of the soil, following the simplified methodology recommend by CFE Manual, (2008), where the geotechnical characteristics of the soft soil such: the period of the soil for translation and rocking are taking into a count as well as the dynamic characteristics of the structure with rigid foundation to obtain the effective soil-structure system. Therefore, the inertial effects are the enlargement of the fundamental period of vibration and effective damping of the system concerning to rigid foundation case.

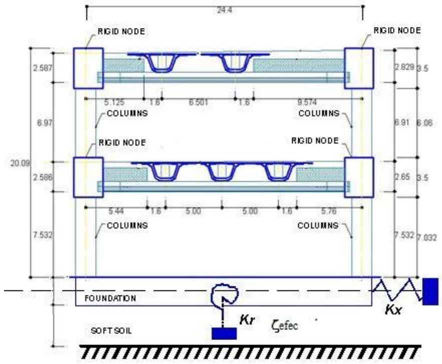


Figure 2.8. Equivalent system with soil-structure effect

$$\tilde{T}_e = (T_e^2 + T_h^2 + T_r^2)^{\frac{1}{2}} \tag{2.1}$$

$$T_h = 2\pi \left(\frac{M_e}{K_h}\right)^{\frac{1}{2}} \tag{2.2}$$

$$T_r = 2\pi \left(\frac{M_e(H_e+D)^2}{K_r}\right)^{\frac{1}{2}} \tag{2.3}$$

where: $\boldsymbol{T_e}$ is a natural period of vibration of the structure with rigid foundation, $\boldsymbol{T_h}$ is a natural period of vibration of the structure with rigid foundation if the base could to move, $\boldsymbol{T_r}$ is a natural period of vibration of the structure with rigid foundation if the base could to rotate, $\boldsymbol{K_h}$ is a lateral stiffness of the foundation and $\boldsymbol{K_r}$ is a rotational stiffness of the foundation

Effective damping

$$\tilde{\zeta}_e = \zeta_e \left(\frac{T_e}{\tilde{T}_e}\right)^3 + \frac{\zeta_h}{1+2\zeta_h^2}\left(\frac{T_h}{\tilde{T}_e}\right)^2 + \frac{\zeta_r}{1+2\zeta_r^2}\left(\frac{T_r}{\tilde{T}_e}\right)^2 \tag{2.4}$$

where:

$$\zeta_h = \frac{\tilde{\omega}_e C_h}{2K_h};\ \zeta_r = \frac{\tilde{\omega}_e C_r}{2K_r} \tag{2.5}$$

$\boldsymbol{\zeta_e}$ is a structural damping of the bridge with rigid foundation, $\boldsymbol{\zeta_h}$ is a structural damping of the bridge with rigid foundation if the base could to move, $\boldsymbol{\zeta_r}$ is a structural damping of the bridge with rigid foundation if the base could to rotate

2.6. Seismic Records

The seismic non-linear analysis step-by-step were carried out employing a seismic record at Mexico City, obtained of the earthquake originated at the Subduction zone of Mexican Pacific Coast in 1985 (see Figure 2.9) and a synthetic seismic record. In this research it chooses one of the six synthetic accelerations records generated for the analyses, this signal is **UC44-W089,** the signal and the elastic response spectra with 5% of critical damping is shows in Figure 2.10, derivate of real earthquakes. The synthetic records were obtained from UC IMSS station located near to structure. The accelerations records were simulated by stochastic process modulated in amplitude and frequency depending of the characteristics of seismic sources (magnitude and distance), Sánchez et al (2006). The real and synthetic seismic records were applied at the base of the models

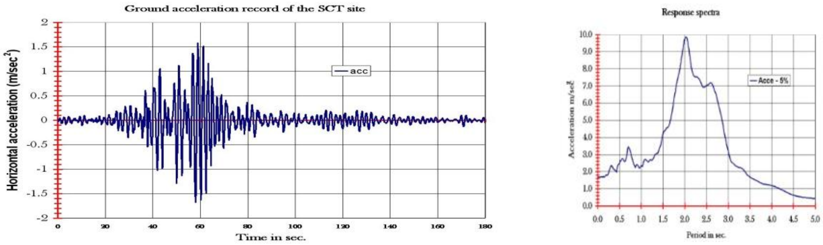


Figure 2.9. Horizontal ground acceleration record SCT-85 and response spectra, Mexico City

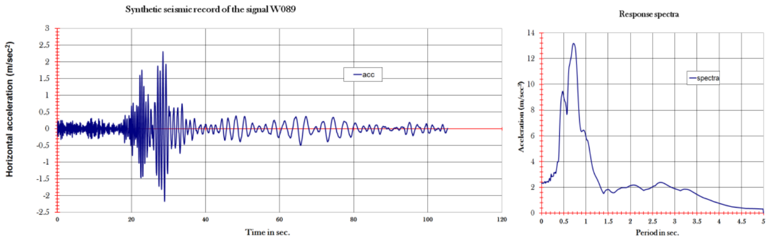


Figure 2.10. Synthetic accelerations record **UC44-w089** at the UC44 station and response spectra

3. NUMERICAL RESULTS

3.1. Modal analysis

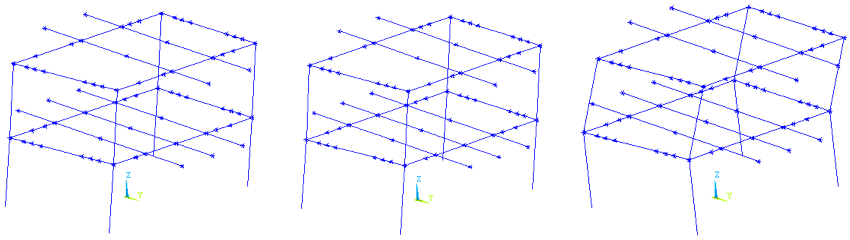
Dynamic modal analysis was carried out for the two cases, rigid and flexible foundation, with the aim to validate the 3D numerical models and know the dynamic parameters.

3.1.1. Numerical results in 3D with rigid foundation models.

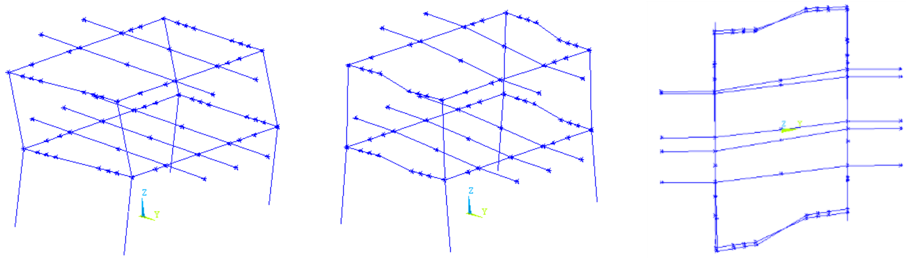
Table 3.1 shows the modal dynamic numerical results for the rigid foundation case; the Figure 3.1 describes the first five modes.

Table 3.1. Numerical results of the dynamic analysis

	Mode 1 T_1 (sec)	Mode 2 T_2 (sec)	Mode 3 T_3 (sec)	Mode 4 T_4 (sec)	Mode 5 T_5 (sec)
DRAIN (2D)	0.5182	-	-	-	-
SAP-2000	0.6808	0.6859	0.2965	0.1967	0.1604
ANSYS	0.6836	0.6834	0.2924	0.1035	0.0978



(a) Mode 1 translation dir y (b) Mode 2 translation dir x (c) Mode 3 translation dir y



(d) Mode 4 translation dir x (e) Mode 5 torsion dir z (f) Mode 5 torsion dir z (in plane)

Figure 3.1. Modal configurations

3.1.2. Numerical results in 3D with flexible foundation (soft soil)

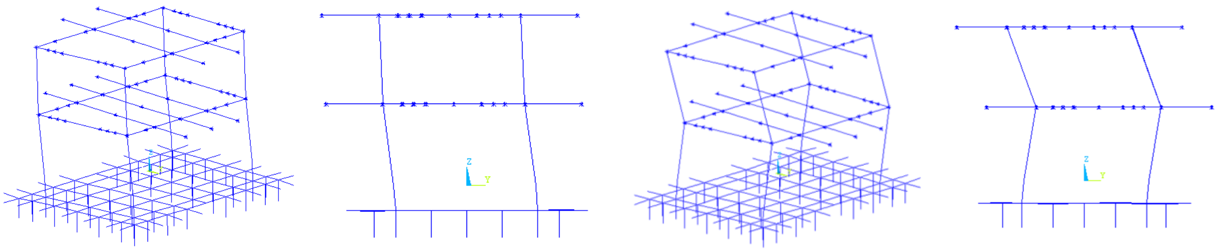
Table 3.2 shows the analytical results of the periods and effective damping of the s-s interaction systems obtained from CFE-Manual, (2008) and the Table 3.3 present the comparison between numerical results obtained with ANSYS program and analytical results with CFE simplified procedure and finally in Figure 3.2 it is presented the first two modes of vibration.

Table 3.2. Periods and effective damping of the s-s interaction systems

Site	Rigid foundation	Soil – Structure Interaction		Equivalent dynamic stiffness of the system		
	T_{e1} (sec); ζ_e	T_{soil} (sec)	T_{efec} (sec)	ζ_{efec}	K_r (kN-m)	K_x (kN/m)
Pozo 14	0.70; 0.05	0.90	0.96	0.060	1.37E+08	1.24E+06
A. U.		0.86	0.86	0.061	2.27E+08	1.91E+06
E. O.		0.42	0.72	0.051	1.05E+09	8.02E+06

Table 3.3. Numerical results of the dynamical analysis in 3D of the structure placed in soft soil of Mexico City

Site	Rigid Foundation T_{e1} (sec); ζ_e	Period of soil T_s (sec)	Soil – Structure Interaction			
			CFE-Manual 2008 T_{efec} (sec)	ANSYS T_{efec} (sec)	T_{efec} / T_{e1}	Effective damping ζ_e =
Pozo 14	0.6836; 0.05	0.90	0.96	1.0061	1.47	0.06
A. U.		0.86	0.86	0.8464	1.23	0.061
E. O.		0.42	0.72	0.6613	1.03	0.051



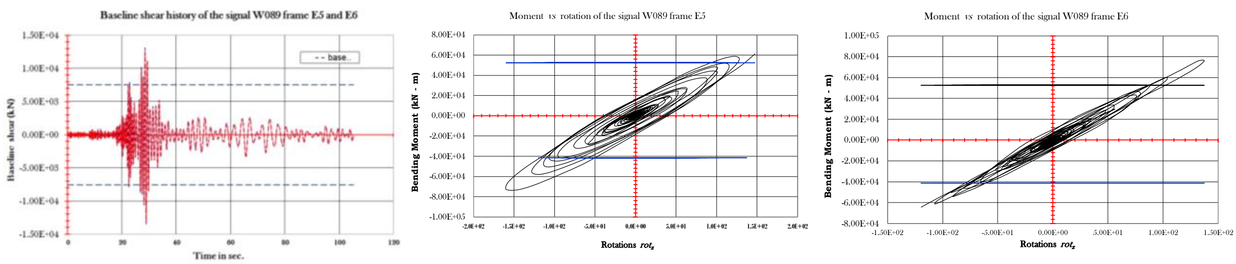
(a) Mode 1 translation dir y (b) Mode 1 translation dir y (c) Mode 2 translation dir y (c) Mode 2 translation dir y

Figure 3.2. Modal configurations

3.2. Time-History analysis

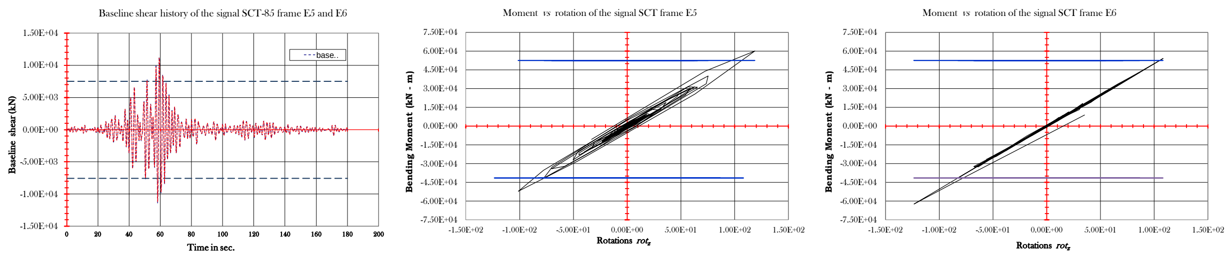
In this section are presents the numerical time-history analysis using the synthetic record **UC44-W089** and the real ground acceleration record SCT-85.

3.2.1. Numerical results response in 3D with rigid foundation



(a) baseline shear V_B (b) beam element frame E5 (c) beam element frame E6

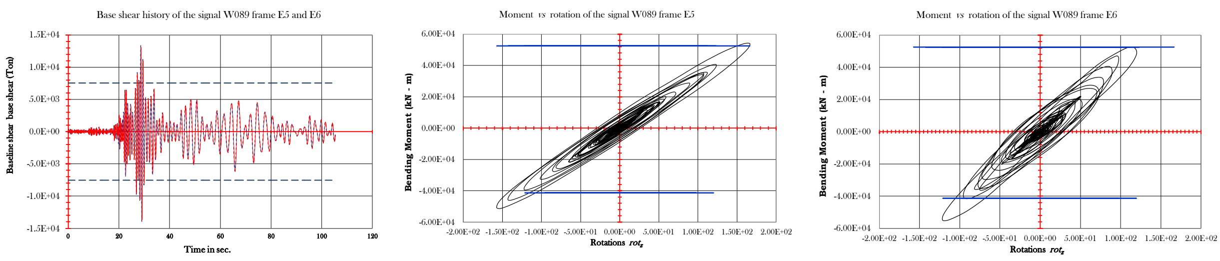
Figure 3.3. Response history V_B and the hysteretic curves moment M *vs* rot_z with signal W089



(a) baseline shear $\mathbf{V_B}$ (b) beam element frame E5 (c) beam element frame E6

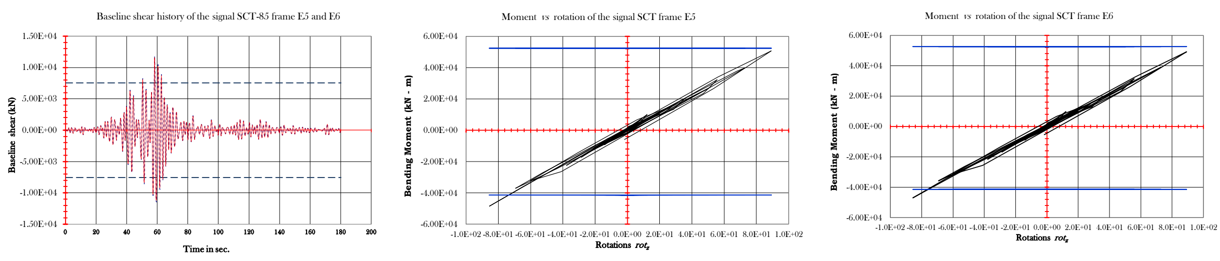
Figure 3.4. Response history $\mathbf{V_B}$ and the hysteretic curves moment M *vs* rot$_z$ with signal SCT-85

3.2.2. Numerical results response in 3D with soft soil foundation



(a) baseline shear $\mathbf{V_B}$ (b) beam element frame E5 (c) beam element frame E6

Figure 3.5. Response history $\mathbf{V_B}$ and the hysteretic curves moment M *vs* rot$_z$ with signal W089



(a) baseline shear $\mathbf{V_B}$ (b) beam element frame E5 (c) beam element frame E6

Figure 3.6. Response history $\mathbf{V_B}$ and the hysteretic curves moment M *vs* rot$_z$ with signal SCT-85

Figures 3.3.a, to 3.4.c show the comparison of the numerical results of the response history of the baseline shear $\mathbf{V_B}$ between the two frames E5 an E6 and the hysteretic curves bending moment M *vs* rot$_z$ for frame E5 and E6 related to the rigid foundation case, obtained with synthetic signal W089 and real ground acceleration record SCT-85. Finally in Figures 3.5.a, to 3.6.c present the soil-structure interaction results obtained with synthetic signal W089 and SCT-85 real acceleration record, the comparison of the response history of the baseline shear $\mathbf{V_B}$ between two frames E5 an E6 is showed in Figures 3.5.a and 3.6.a, the hysteretic curves bending moment M *vs* rot$_z$ for frame E5 and E6 are displayed in Figures 3.5.b, 3.5.c, 3.6.b and 3.6.c.

4. CONCLUSIONS

This study investigates the damage that could suffer during strong motions the RC frames of the Viaduct. Then to known the inelastic damage the structures were analyzed by step-by-step using real and synthetic records with ANSYS program, the 3D modelling include Beam 188 element and non-linear behaviour of the concrete material by constitutive model with a bilinear stress-strain relationship curve representative of the RC behaviour of beams and columns. The requirements NTC-2004 and CFE-2008 were used for the modelling of the soil-structures interaction system.

The most important comparison observed between the numerical results of the time history analysis for the rigid foundation and soil-structure interaction cases shows an increase of the dynamical parameters such as, the period of vibration $\widetilde{T_e}/T_e = 1.47$ and the damping $\widetilde{\zeta_e}/\zeta_e = 1.2$ effectives of the system, and an amplification of the maximal horizontal displacement of the soil-structure system obtain with seismic record **SCT-85** and **UC44-W089**, but a reduction of the drifts of the structure. Respect to the mechanical elements response comparison it is observe a reduction of the bending moment for the soft soil foundation case respect to rigid foundation about of 15.6% and for **UC44-W089** the bending moment about of 11.79%. These numerical results show a behaviour and response expected according to the Mexican codes, yet if the deposits of soft soil at the bottom of the Urban Viaduct are not very deeper. Finally it recommend to refine de meshing of the numerical model including solid elements and more complete and sophisticated non-linear behaviour of the material.

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