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SEISMIC VULNERABILITY ASSESSMENT OF SCHOOL BUILDINGS IN MEXICO

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Abstract

In this paper criteria for seismic vulnerability assessment of school buildings in Mexico are presented. The criteria consider the estimation of non-linear responses using structural models. Some dynamic properties of those models are calibrated from experimental analysis considering ambient vibration records. For describing the seismic vulnerability we use two different criteria, i) based on reliability functions that consider the Secant Stiffness Reduction Index (SSRI) and ii) based on reliabilty functions using SSRI and Simplified Reference Systems (SRS). These criteria were implemented in a school building that belongs to UPAEP University infraestructure.

To determine the SSRI it is necessary to carry out the pseudo-static non-linear response (pushover analysis) of the structural model that corresponds to a multiple degrees of freedom system. From this analysis we obtain a curve that relates the shear base, V_b, with the global lateral displacement at the roof of the system, X_N. Apart from these curves, the pushover analysis provides a series of lateral configurations of the displacements in different levels and response values at each instant of interest. Pushover analysis also provides the estimation of the deterministic deformation capacity, uF, of the structural system. In this paper the criterion for estimating the deformation capacity corresponds to a 20% reduction of the peak shear value. The pushover analysis is useful to obtain the mechanical properties of SRS.

The non-linear analysis also includes the response estimation obtained from step by step procedures of the same structural models described in above section; we also considered the uncertainties in the structural properties and for different seismic excitations. For this analysis a modified version of DRAIN-2D software is used. If we have a representative sample of the resulting responses, we can determine the probability distribution of the response; such distribution is characterized by the expected value of response and the corresponding interval in terms of seismic intensity. In order to estimate the structural reliability function, a probabilistic model of the mechanical properties and the gravitational loads acting on the system can be formulated. The reliability functions mentioned in criteria ii and iii) are in terms of Cornell´s reliability index, and here we use a normalized seismic intensity measure.

In order to illustrate the procedures a case was analysed that corresponds to reinforced concrete school building located in central Mexico. Finally, the seismic vulnerability assessment can be useful for estimating the seismic risk level and for the establishment of the rehabilitation strategies of those buildings.

Keywords: Seismic vulnerability assessment; school buildings; non-linear analysis; structural reliability index.

1. Introduction

The displacement-based practical criteria for earthquake resistant design are based on the concept that the event of reaching a given performance limit is associated with the condition that the structural distortion reaches a value equal to the corresponding deformation capacity. According to this, the estimation of the seismic reliability of complex nonlinear systems for given values of the ground-motion intensity is ordinarily based on a measure of the probability that, for an ensemble of earthquake excitations with a specified intensity, the ratio of the peak absolute value of the nonlinear displacement response demand of the system to the corresponding deformation capacity is greater than unity.

On the other hand, there are some studies related to vulnerability assessment and risk mitigation in school buildings around the world, as examples we can mention those developed by [1], [2], [3], [4], [5], [6], [7], [8], [9] and [10].

In this work the seismic vulnerability assessment is carried out using two different criteria, i) based on reliability functions that consider the Secant Stiffness Reduction Index (SSRI); ii) based on reliabilty functions using SSRI and Simplified Reference Systems (SRS).

To determine the SSRI it is necessary to carry out the pseudo-static non-linear response (pushover analysis) of the structural model that corresponds to a multiple degrees of freedom system (MDFS). From this analysis we obtain a curve that relates the shear base, V_b, with the global lateral displacement at the roof of the system, X_N. Apart from these curves, the pushover analysis provides a series of lateral configurations of the displacements in different levels and response values at each instant of interest. Pushover analysis also provides the estimation of the deterministic deformation capacity, u_F, of the structural system. In this paper the criterion for estimating the deformation capacity corresponds to a 20% reduction of the peak shear value. The pushover analysis is useful to obtain the mechanical properties of SRS. The non-linear analysis also includes the response estimation obtained from step by step procedures of the same structural models described in above section; we also considered the uncertainties in the structural properties and loads. For this analysis a modified version of DRAIN-2D software [11] is used.

If we have a representative sample of the resulting responses, we can determine the probability distribution of the response; such distribution is characterized by the expected value of response and the corresponding interval in terms of seismic intensity. In order to estimate the structural reliability function, a probabilistic model of the mechanical properties and the gravitational loads acting on the system can be formulated. The reliability functions mentioned in criteria i) and ii) are in terms of Cornell´s reliability index β [12], and here we use a normalized seismic intensity measure. Such procedures also need to be applied in practice conditions providing useful information to establish, in a simplified and quickly way, the level of seismic vulnerability. This information can be useful for making decisions regarding the maintenance, rehabilitation or strengthening the building of interest. An important part of this work is the estimation of the instrumental response and the measurement of vibrational periods through modal analysis on the structure. The modal analysis is carry out using techniques of environmental vibration and accelerometers Basalt of Kinemetrics™, that have been fundamental tools in the vulnerability assessment of school buildings [9]. Starting from the above the calibration of an analytical model with software SAP2000 v.14 will be presented using these results and later to perform a nonlinear analysis and estimate vulnerability.

In order to illustrate the procedure a four levels reinforced concrete school building was analyzed, this is located in Puebla City Mexico and was built in the early 80's. It is important to note that the structural configuration of the analyzed case is typical of school buildings (see Figure 3). Puebla and other cities located in Central Mexico region have been the scenarios of major seismic events over the last decades. Finally, the seismic vulnerability assessment can be useful for estimating the seismic risk level and for the establishment of the rehabilitation strategies of those buildings.

2. General procedure to obtain the structural models

2.1 Experimental test for obtaining modal parameters

Ambient vibration records in three points were obtained on the building, P01 and P02 which correspond to the geometric centroid and the corner at the roof of the building, respectively; P03 corresponds to the geometric centroid at the ground floor of the building. For each point four records were taken considering 15 minutes. A triaxial accelerometer (Kinemetrics Basalt Accelerometer) was used as instrument for obtaining the records.

Frequencies and periods were determined for the first three modes of vibration of the building: longitudinal mode (L), transversal mode (T) and rotational mode (R). For this, a computer program [13] was used in order to obtain the Amplitude Fourier Spectrum (AFS) for each record, in this way, the horizontal components of the movement (longitudinal and transversal) are only considered for computing the spectral ratios. The procedure for each of the mode is described below. Longitudinal mode (L), the numerator corresponds to the AFS in the longitudinal component obtained in the geometrical centroid at the roof level, and the denominator corresponds to the AFS in the longitudinal component obtained in the geometrical centroid at the ground floor. Transversal mode (T), the numerator corresponds to the AFS in the transversal component obtained in the geometrical centroid at the roof level, and the denominator corresponds to the AFS in the transversal component obtained in the geometrical centroid at the ground floor. Rotational mode (R), the numerator corresponds to the AFS in the transversal component obtained in the corner at the roof level, and the denominator corresponds to the AFS in the transversal component obtained in the geometrical centroid at the same level. In Table 1 the obtained results for frequencies and periods of the first three modes of vibration of building are shown.

Table 1 – Experimental values of frequencies and periods of vibration for the first three modes of building

Mode	**Building B**	
	f[Hz]	**T[s]**
Longitudinal	3.97	0.25
Transversal	4.76	0.21
Rotation	6.28	0.16

2.2 Generation and calibration of analytical models

In [8] is presented the estimation of the linear responses of the building analysed in this paper, they determined the linear responses from a structural model obtained with the computer program SAP2000 v.14.1. To assign and calibrate the properties of their models, they used ambient vibration records in order to obtain the vibration periods and frequencies of the first three modes of the building. In the following a description of the procedure is presented.

The structural model was projected in a way that structuring (supports, global geometry, and cross sections) and loads show faithfully the actual characteristics observed in the building under study. The building was considered built-in in its base. The floor system was modeled as a rigid diaphragm with dead load of 180 kg/m^2 and distribute to the crosswise beams (short span). To characterize the materials it was used a concrete with f’c=300 kg/cm^2 and modulus of elasticity E=242,487.11 kg/cm^2. The volumetric weight of concrete was 2,000 kg/m^3. Once determined the structural model it was proceeded to do the modal analysis to calculate the analytical values of vibration periods. Such values are shown in Table 2. In accordance to comparison of Tables

1 and 2 it was concluded that the periods obtained in an analytical and experimental way were achieved with good approximation.

Table 2 – Analytical values of frequencies and periods of the first three modes of the building under study, calculate with a modal analysis using SAP2000v14

Mode	**Building B**	
	f[Hz]	**T[s]**
Longitudinal	3.95	0.25
Transversal	4.58	0.22
Rotation	6.89	0.20

2.3 Generation of models for nonlinear analysis

In order to estimate the structural reliability, a probabilistic model of the mechanical properties and of the gravitational loads acting on the system was formulated, and a sample of possible realizations of the vector of those properties was generated by Monte Carlo simulation, as described by [14], using the statistical properties reported by [15] and [16]. The model proposed by [17] was adopted to represent the constitutive functions describing the bending behaviour of the critical sections at the ends of beams and columns. For each system in the sample, a nonlinear analysis was made of its response to a randomly chosen member of a sample of actual ground motion time histories representative of those expected at the site of interest. The nonlinear analysis was carried out in the longitudinal and transversal directions. With the goal of simplify the analysis, it was considered that the torsion effects due to irregularities were not significant and the 2D model represented with accuracy the general behavior of the building in each direction of study.

3. Nonlinear response analysis for 2D models

3.1 Pseudo-static pushover analysis

To carry out vulnerability assessment using SRS and SSRI criteria it is necessary to obtain the pseudo-static non-linear response (pushover analysis) of the structural model that corresponds to a multiple degrees of freedom system. In order to carry out the pushover analysis a two-dimensional structural model is subjected, at its base, to a monotonic acceleration that grows linearly with time. The growth rate of acceleration is taken low enough so as to avoid the occurrence of vibrations. As a result the structure is deformed by inertial effects alone. Uncertainties in the values of the gravitational loads and geometric properties are considered, and for the pushover analysis these parameters correspond to their expected values. The lateral configuration in the model is obtained by applying a mass distribution model, which corresponds to the first mode.

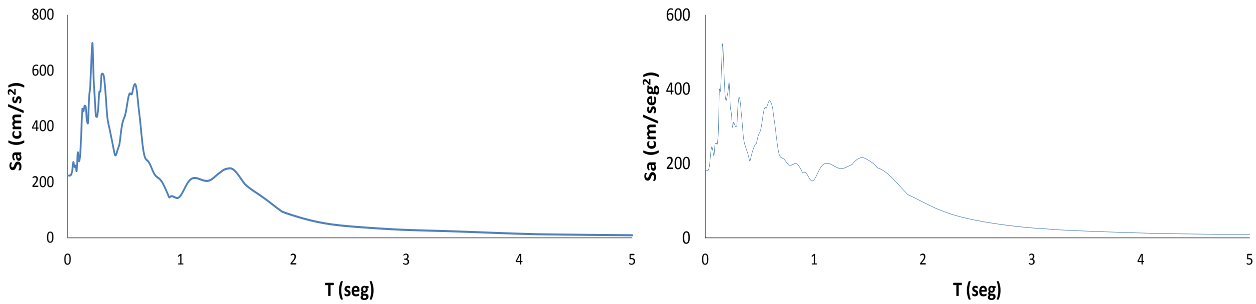
From this analysis we obtain a curve that relates the shear base, V_b, with the global lateral displacement at the roof of the system, X_N. Apart from these curves, the pushover analysis provides a series of lateral configurations of the displacements in different levels and response values at each instant of interest. Pushover analysis also provides the estimation of the deformation capacity, u_F, of the structural system. In this paper the criterion for estimating the deformation capacity corresponds to 20% reduction of the peak shear value.

3.1 Step by step nonlinear analysis

The estimation of nonlinear responses was performed step by step of structural systems in the longitudinal and transversal directions. In both, the uncertainties were considered in the structural properties for different seismic excitations. The DEIH software, which is a modified version of the DRAIN-2D [11] was used in this analysis. The hysteretic behavior of the system which describes the nonlinear response, the energy dissipation capacity and maximum deformation of the system was obtained from this analysis. For each case fifty simulations were performed; in each of these runs the structure was subjected to the action of the corresponding ground motion. In order to generate the nonlinear behavior in simulated systems it had to use an amplification factor for the accelerograms. Thus curves relating to base shear with roof displacement were obtained. From these and the maximum value of displacement it is possible to obtain the value of the secant stiffness.

3.2 Seismic excitation

For nonlinear analysis, seismic excitation that was used in this work corresponds to acceleration records of a simulated earthquake using the criteria given by [18]. Details about the procedure can be found in [18]. In Figure 1 the simulated earthquake Pseudo-acceleration response spectra used in this work are shown.



a) b)

Fig. 1 – Pseudo-acceleration response spectra for simulated earthquake
a) E-W component, b) N-S component

4. Structural vulnerability assessment

4.1 Based on the Secant Stiffness Reduction Index (SSRI)

This criterion is based on the adoption of a fault condition in terms of an index of damage (SSRI), which considers the reliability of the system referred to collapse. The SSRI value can be determined with the following equation:

$$I_{SSR} = \frac{(K_0 - K)}{K_0} \tag{1}$$

Where $K = V_b \, / \, X_N$ is defined as the value of the secant stiffness of a nonlinear system at the instant in which the global lateral displacement (X_N) reaches its maximum absolute value in response to a seismic excitation. V_b is the base shear at the same instant that the maximum response occurs. The K_0 variable is the value that acquires K when the system response is linear and in this work its value is obtained from the analysis of a pseudo-static lateral push (Push-over). The condition of survival of the system, at a given seismic intensity is achieved if I_{SSR} <1.0; if the condition I_{SSR} = 1.0 would mean the collapse of the structural system. The criteria presented below expresses that the capacity of the system is considered as the random value of the intensity that is required to produce failure [19], [20] and [21]. From the above it is possible to define the variable $Z_F = \ln Y_F$ where Y_F is the minimum value of the seismic intensity that produces the collapse condition. If Y_F is a random variable, it is possible to set the probability density function, cumulative distribution function and its statistical moments.

On the other hand, if for a given intensity y it is defined a safety margin $M_Z = \ln(\psi_c \, / \, \psi \, (y))$, where ψ_c is the lateral deformation capacity of the system and $\psi(y)$ is the demand for deformation intensity, it can be defined in the same way, the safety margin $M_Z = \ln(Y_F/y)$. From this way it can be defined the reliability index in accordance with the proposed by [22]; this definition is as follows:

$$\beta(y) = \frac{(E[Z_F] - \ln y)}{\sigma[Z_F]} \tag{2}$$

Thus, if there is a sample of pairs of random values of Z and I_{SSR}, it can be estimated the statistical moments, mean E[·] and standard deviation σ[·], $Z(u)$ = ln(Y), wherein Y corresponds to the value of $I_{SSR}=u$. If all the values that can acquire u are less than 1.0, functions and parameters describing the first two statistical moments of Z, as a function of I_{SSR}, are determined with a conventional regression analysis, for example, least squares. In this paper the following functions for the average E[Z] and variance var[Z] of Z, respectively, are used:

$$E[Z] = a + b(1-u) + c(1-u)^2 \tag{3}$$

and

$$\mathrm{var}[Z] = a_1 + b_1(u) \tag{4}$$

Where $b \leq 0$, $c \leq 0$ and $b_1 \geq 0$

4.2 Based on Simplified Reference System (SRS)

As part of the vulnerability analysis that is developed in this paper, it has that the number of variables that affect the maximum values of local responses of interest is broad, this implied to do studies of the dynamic response and damage functions in a large number of complex systems or multiple degrees of freedom (MDF). According to [23] a possible solution can be bypassed using the Simplified Reference Systems (SRS), which consist of systems with one degree of freedom with features shear-displacement similar to those that relate the base shear of the original system with the displacement at top (V_b, X_N). For a stablished system this function is obtained by an analysis of lateral push, which also provides a set of configurations of lateral responses that are required to estimate maximum values of local responses based on the maximum value of displacement on the roof.

The methodology that is based on the use of simple systems of one degree, seeks to represent the most significant properties of detailed systems. This system is defined by its initial stiffness (k_1), after yield stiffness (k_2), mass (m), damping (c) and, shear and displacement yielding, u_y and v_y, respectively. So it is possible to establish relationships between the most significant responses of detailed model with the estimated responses using simple systems. In Figure 2 it is shown schematically the model used for SRS. Relations between SRS and MDFS are given by [23]. The methodology used in the development of calculating the properties of SRS is also presented in [23].

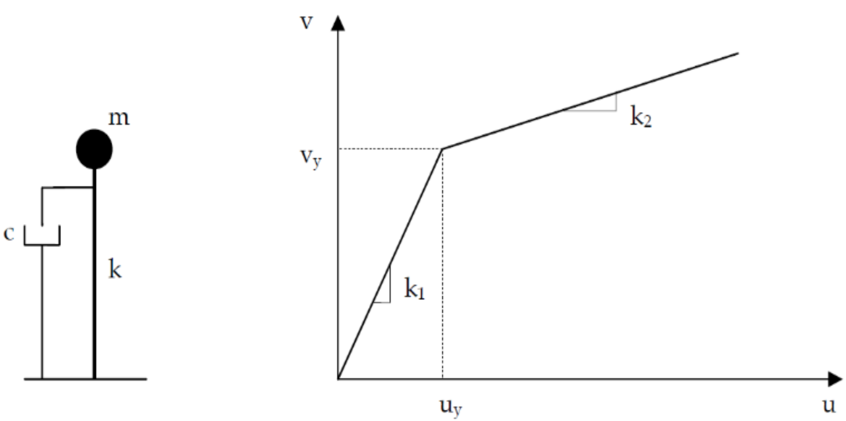


Fig. 2 – Idealization of simplified reference system SRS

5. Implementation and results

The case investigated in this work is the building B of the UPAEP University. The building has geometrical configuration typical of school buildings in Mexico. The structural system was solved using reinforced concrete frames. It is regular in plan and elevation, consists of 4 levels of 3.25 m each so the total height is 12.6 m. In the longitudinal direction it has eight bays 6 m each one, so the total length is 48 m. In the transversal direction it has one bay of 10.5 m. The structure consists of beams and columns. The beams in the transversal direction have square cross section at all levels. The beams on levels 1 and 4 of the longitudinal direction have variable prismatic cross section, while beams on levels 2 and 3 have rectangular cross section. The columns at all levels have variable hexagonal section. The floor system is prefabricated and it is supported in the longitudinal direction of the building. Figure 3 shows the building B.

Fig. 3 – Current view of building B

In Figure 4 push-over capacity curves in longitudinal and transversal directions are shown. With these graphs the values and parameters described in previous sections were obtained.

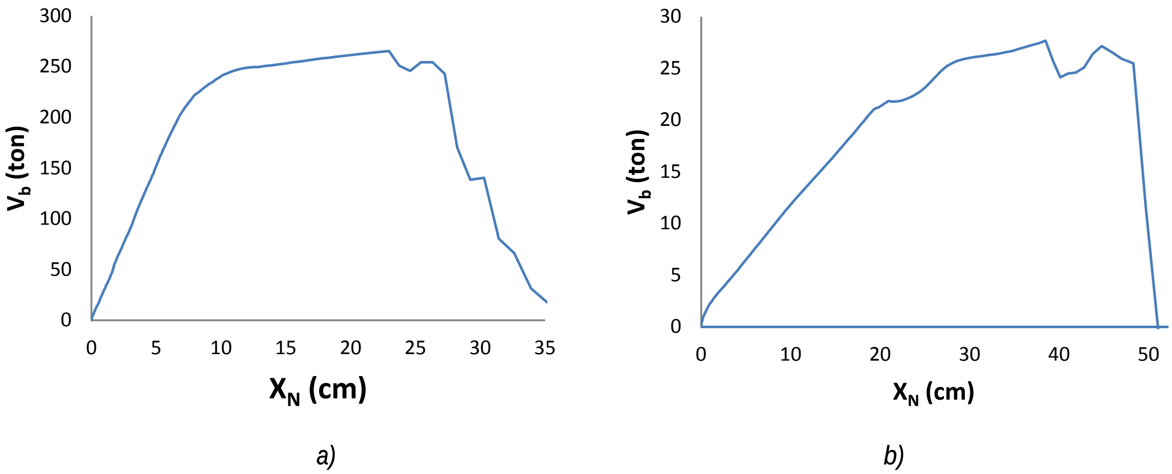
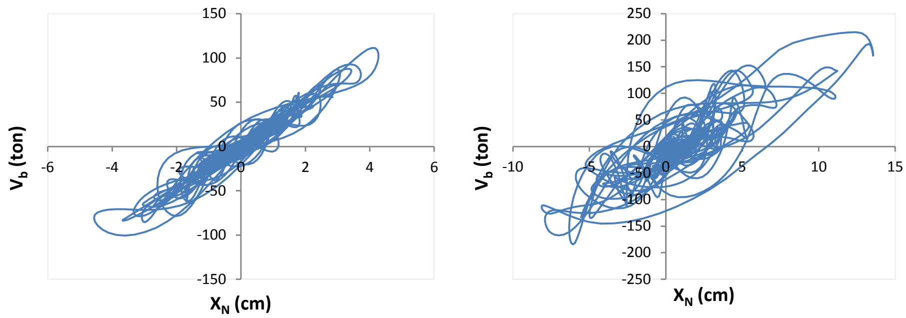


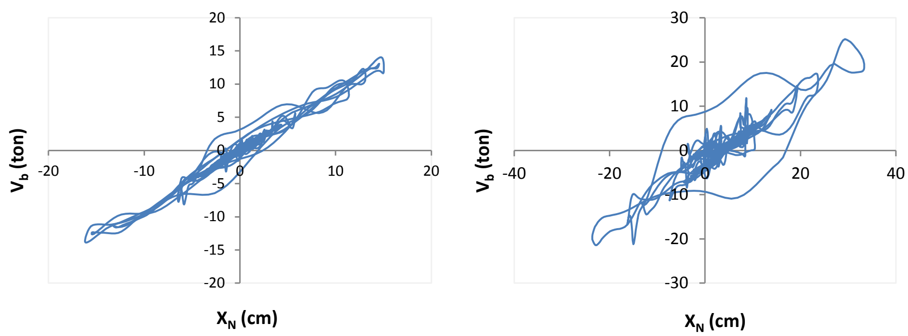
Fig. 4 – Curves obtained from the analysis of lateral push of Building B
a) Longitudinal direction, b) Transverse direction

Examples of the results for the step by step non-linear analysis in the longitudinal and transversal directions are shown in Figure 5 and Figure 6, respectively. With these results we could get the values of *K* for each framework described in calculating SSRI.



a) b)

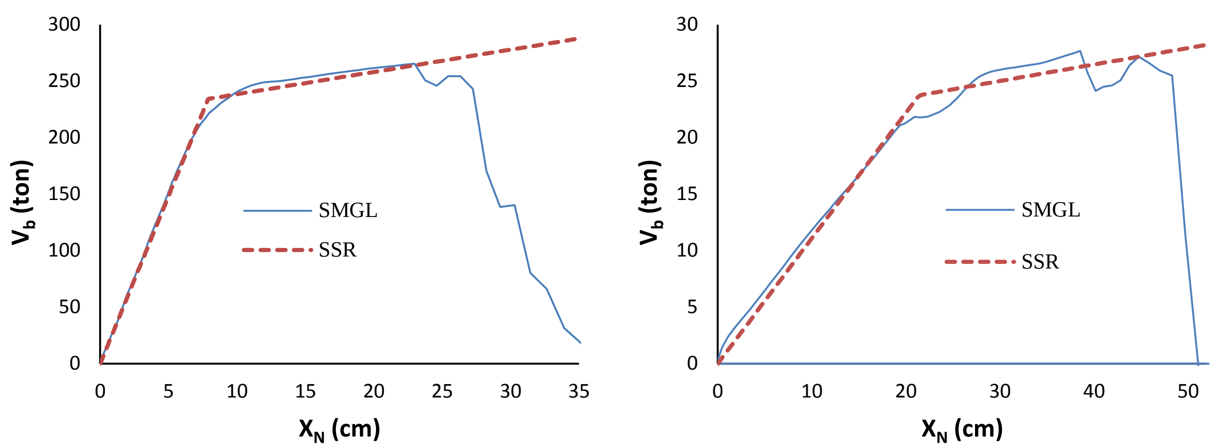
Fig. 5 – Seismic response calculated from step by step non-linear analysis, (longitudinal direction) for different amplification factors: a) 1, b) 3.5



a) b)

Fig. 6 – Seismic response calculated from step by step non-linear analysis, (transversal direction) for different amplification factors: a) 1, b) 2.5

In Figure 7 the results obtained for SRS in both directions are shown. Such reference systems were obtained from the methodology described above.

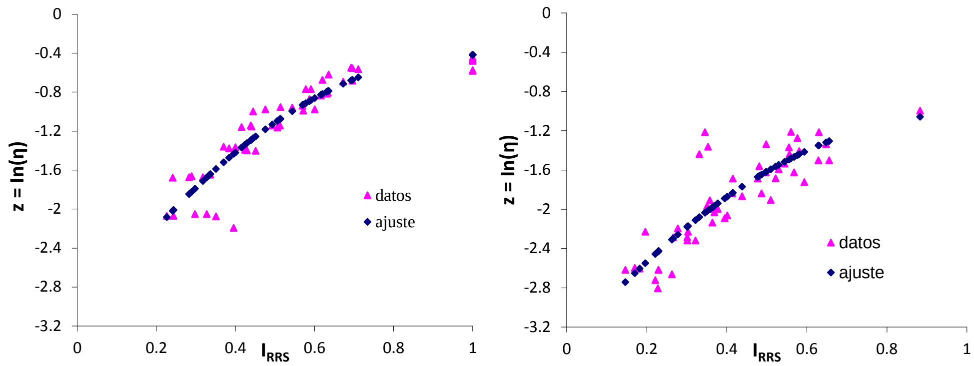


a) b)

Fig. 7 – Comparison between the curves obtained from non-linear seudo-static analysis MDFS (SMGL) and those obtained from the SRS (SSR); a) Longitudinal direction, b) Transversal direction

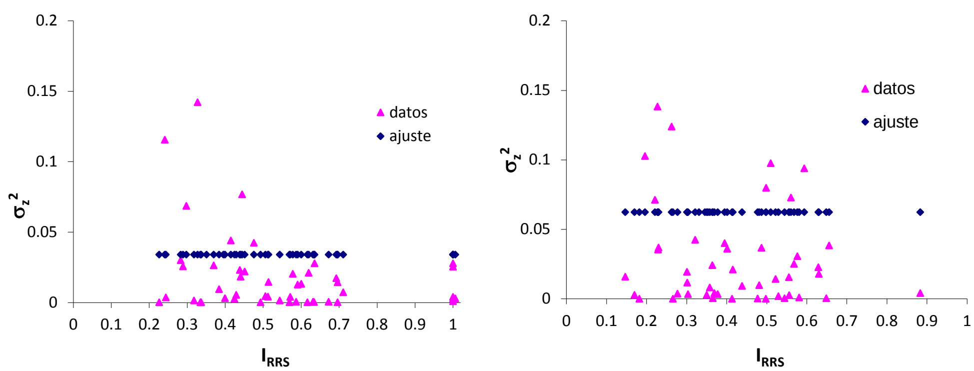
Thus, with the data obtained from nonlinear analysis, the normalized seismic intensity corresponding to $\eta=S_{dl}(T)/u_F$ was estimated. Here $S_{dl}(T)$ is the ordinate of the spectrum corresponding to the linear displacement of fundamental period, T, of structural models and u_F is the deformation capacity defined above and estimated from the curve of lateral push for MDFS.

In Figure 8 the values of E[Z] are presented in terms of u=I_{SSR}, obtained with Equation 3. To determine the parameters used in equation a regression least squares method was used. In Figure 9 the values of the variance of Z, var[Z], for which Equation 4 was used, are presented; in the same way, a regression method using least squares, was used to determine the values of the corresponding parameters. In the mentioned figures the results for the two directions considered are presented. Figure 8 shows a reasonable fit. Furthermore, in estimating the variance of Z a dispersion that can be considered acceptable is observed.



a) b)

Fig. 8 – Values of E[Z]: a) Longitudinal direction, b) Transversal direction;
blue marks correspond to fitted curve and pink marks correspond to data



a) b)

Fig. 9 – Values of the variance of Z, var[Z]: a) Longitudinal direction, b) Transversal direction;
blue marks correspond to curve and pink marks correspond to data

In order to estimate the reliability index β equation 2 was used, the index is presented in terms of the normalized seismic intensity. Figure 10 is referred to the reliability values obtained for the longitudinal frames (solid line) and transverse frames (dotted line). As can be seen, reliability levels are lower in the transversal direction.

In Figure 11 the distortion values of the SRS (ψ_0) are presented in terms of normalized intensity. Distortion values were obtained from a step by step analysis. Figure 11 represents data values called distortion obtained in terms of η, whereas values called adjustment represents the value of the estimated distortion also in terms of the normalized seismic intensity. This adjustment is obtained by a nonlinear regression as Equation 8, where a, b and c are the adjustment parameters.

$$\psi_0(\eta) = a\eta^m + b\eta + c \tag{8}$$

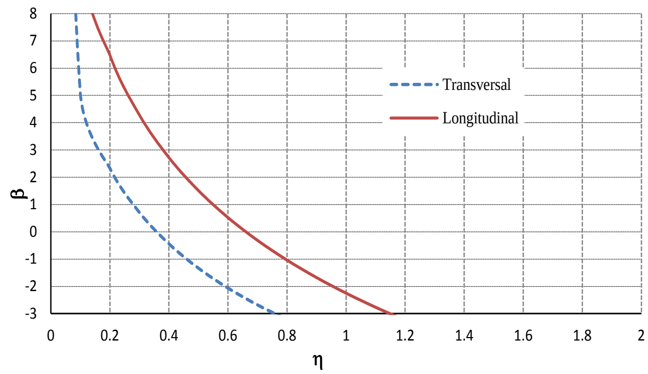
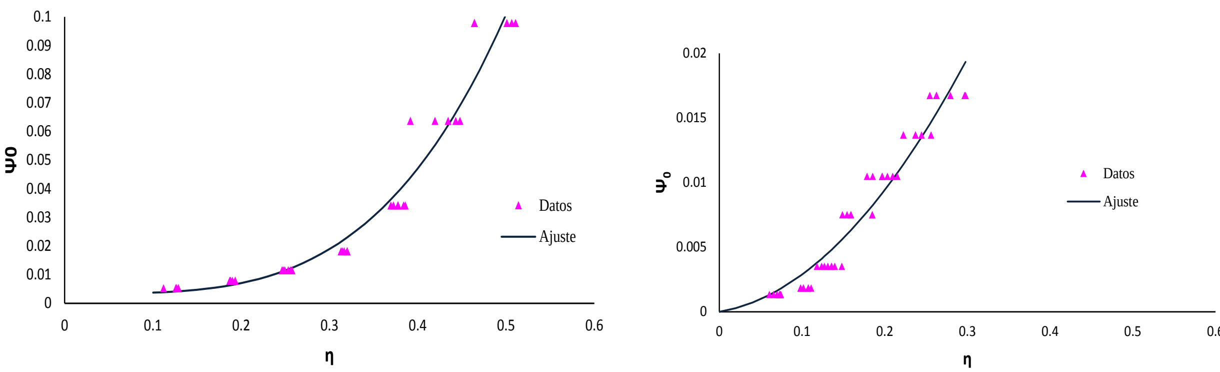


Fig. 10 – β reliability index in terms of the normalized seismic intensity (η)



a) b)

Fig. 11 – Drift estimation of SSR in terms of normalized seismic intensity; a) Longitudinal direction, b) Transversal direction; here continuous line represents fitted curve and pink marks

Assuming that studied building was designed with a criterion where the seismic performance factor Q is equal to 2, then it has an allowable interstory drift equal to 0.015 marked by the Mexican Code for seismic design; this distortion indicates that the system failed. In Figure 11, the distortion can be related with a normalized seismic intensity to the longitudinal and transversal direction. In Figure 10 can be observed the reliability indices β are about 4.5 and 1.5 for the longitudinal and transverse frames, respectively. From normalized intensity for permissible distortion condition or fault condition, the displacement intensity that leads to building this condition was obtained. These displacements are close to 7.5 cm and 12.5 cm for longitudinal and transverse frames, respectively.

6. Final comments

In this paper criteria for vulnerability assessment of school buildings in Mexico were presented. The procedures took into account the evaluation of some dynamic properties using environmental vibration records. These properties were useful to calibrate the analytical model of the building and to generate the corresponding SRS. Furthermore, a damage index that takes into account the reduction in secant stiffness of the structural system, when subjected to a seismic excitation with a given current was used. The non-linear responses of the structural system were estimated to calculate some properties of SRS. The rate of damage and the uncertainties related to the mechanical properties and gravitational loads were also considered.

The procedure was successfully applied to an existing school building, located in the city of Puebla. Seismic records used for step by step analysis, corresponded to simulated events that take into account the dynamic properties of soil as well as the site effect that appears in the foundation of the building were used. For the above, the 1999 earthquake event recorded in the seismic station Chila de las Flores was considered as seed.

This record corresponds to a normal faulting earthquake, which represents the greatest potential seismic hazard in the area where it is supported the building analyzed.

After analysis it was observed that the level of reliability of the building is low. It was noted that is even lower in the transversal direction. In this sense it is strongly recommended to university authorities carry out relevant activities to minimize this risk. It is clear that the results presented in this work were estimates of what might happen to a real seismic event, this because of the important sources of uncertainty that exist and were mentioned, so the results express probabilities of possible behavior that could have the structure under real conditions.

By last, the procedures presented here can be used for the analysis of the vulnerability of other existing buildings. Finally, the information obtained can be useful in mitigating the seismic risk of existing buildings such as schools and other.

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