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A PERFORMANCE ANALYSIS PROCEDURE BASED ON CORRECTED DISPLACEMENTS TO EVALUATE THE SEISMIC RESPONSE OF STEEL 2D FRAMES.

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Abstract. In the proposed methodology, a load pattern is applied in a non-adaptive fashion to obtain the seismic response of two-dimensional steel moment resisting frames. The proposed methodology is based on the structural dynamics theory and consists of a single run nonlinear analysis. This invariant load pattern is formulated by considering higher mode effects with the use of an effective modal mass contribution factor. Also, part of the proposed procedure, a corrective factor is employed to adjust the displacements obtained from the nonlinear analysis ensuring that the drift values obtained from the corrected displacements are adequate. The procedure allows the analysis of the structural response, i.e, story displacement and story drifts. To evaluate the methodology a nine-story steel moment frame is analyzed. Material and geometric non linearities are considered for all the cases. The results are compared with the ones obtained by the Nonlinear time history analysis.

1 INTRODUCTION

To estimate structural demands imposed by seismic activity, one must consider the inelastic behaviour of structures, i.e., geometric non linearities as well as material plasticity. To this goal, some international codes as [1, 2] propose performance methods based on nonlinear static analysis (NLSA). Seismic NLSA consist of applying a monotonically increasing lateral force or displacement vector on a nonlinear structural model that produces large displacements and plasticity. This lateral force vector represents the inertial forces or displacements expected during a seismic event. The force vector is applied incrementally until a predetermined target displacement is reached. The target displacement represents the displacement demand that the seismic ground motions would impose on the structure. Once the target displacement is achieved, the demand parameters (e.g., base shears, story drifts, story displacements, etc.) for the structure are compared with the respective acceptance criteria for the desired performance state. It is a well-known fact that NLSA can produce acceptable results if the structures are dominated by the first mode, i.e., short, and symmetric buildings. In the case of structures that are taller, slender, or present irregularities, where multiple vibration modes affect the structural behavior, NLSA becomes less suitable [3,4,5]. Research have been made to overcome the limitations of traditional NLSA. Some methodologies are based on invariant load distributions which are formulated on structural dynamics theory [6,7,8,9,10,11,12] Others are the ones known as adaptive pushover procedures which name derives from the fact that they attempt to change the load pattern in every step of the analysis, i.e., variable loading vector [13,14,15].

The most rigorous and robust procedure to evaluate seismic demands on structures responding in their nonlinear range is the nonlinear time history analysis (NLTHA). Contrary to the NLS procedure, NLTHA, when implemented, provides a more accurate calculation of the structural response to a strong ground acceleration as it incorporates inelastic member behavior under cyclic seismic ground motions. The NLTHA procedure explicitly simulates hysteretic energy dissipation in the nonlinear range. The dynamic response is calculated for input earthquake ground motions, resulting in response history data on the relevant demand parameters. Due to the variability in seismic records, several analyses for multiple ground motion accelerations are necessary to calculate statistically robust values of the demand parameters for different scenarios. For the reasons stated before, there is still room and need for simpler analysis tools. It is necessary for researchers to continue the development of NLSA methods, so that these analyses can become more reliable and applicable for irregular structures. The performance-based procedure proposed in this article employs a NLSA procedure in which a single lateral force distribution pushes the structure to a certain target according to the N2 method. The single lateral force distribution is based on a modal analysis in which an effective mass factor represents the contribution of each mode. Subsequently the displacements obtained from pushing the structure to the target displacement are corrected to allow the adequate calculation of drifts. Several examples were considered, and the results obtained are compared with those obtained using the NLTHA and the adaptive force-based procedure.

2 DETERMINATION OF TARGET DISPLACEMENT AND N2 METHOD

According to the Eurocode 8 [2], Target displacement, is defined as the seismic demand derived from the elastic response spectrum in terms of the displacement of an equivalent single degree of freedom system. This parameter is of extreme importance because it dictates the accuracy of the NLSA. We consider the N2 method [16,17], present in the Eurocode, to obtain the target displacement. This method is a graphical procedure which compares a structures capacity with the demands produced by seismic forces. The capacity of the structure is represented by a capacity curve (Base shear – displacement curve) obtained by a NLSA. The values of the capacity curve are converted to spectral accelerations and spectral displacements of an equivalent SDOF, respectively. The converted spectral values determine the capacity spectrum. The earthquake demands are defined by damped elastic spectra in an acceleration-displacement response spectrum format. The intersection of both spectrums provides an approximation of the inelastic acceleration and displacement demand.

3 GROUND MOTIONS AND RESPONSE SPECTRUMS.

The ground motion for "El Centro" earthquake was chosen to develop the proposal and for the evaluation of the method. The ground motion was scaled to varying intensity levels. The intensity factors considered are 0.5, 1.0, 1.5, 2.0,2.5 and 3.0 x El Centro ground motion. This was done to ensure that the structures are excited in several behavioral scenarios ranging from elastic behavior to highly inelastic behavior. For the NLTHA, a numerical direct integration scheme must be employed to solve the system of equations of motion. For the present study the Hilber-Hughes-Taylor implicit algorithm has been chosen. The factor employed are α=-0.1, β=0.3025, γ=0.6 and ΔT= 0.0067 seconds (4500 Steps). Damping is proportional to the stiffness. The nonlinear static analysis was made with the help of equivalent response spectrums that match each scaled time history record of the El Centro earthquake. Each response spectrum was generated according to Eurocode 8 elastic response. The resulting spectrums based on a seismicity zone type 1, Ground type B, damping at 5% and 0.15,0.3,0.45,0.6,0.75 and 0.9 PGA.

4 MODELING CONSIDERATIONS

Plastic behavior (material non-linearity) for structural steel is considered by means of a uniaxial bilinear stress-strain model with kinematic strain hardening. Inelastic behavior of the elements is considered by means of a fiber model that contemplates distributed plasticity. The stress-strain state of the section of the beam-column elements is obtained through the integration of the non-linear uniaxial stress-strain response of the individual fibers. In this case 150 per section and where each element has been divided in 5 sections. The interpolation functions for the elements have been considered based on forces. Geometric nonlinearity has been considered by using a total co-rotational formulation. Each element is modeled as a plane frame element that has two nodes each node with 3 degrees of freedom. Seismostruct v2022 is employed in this research to develop the NLSA and the NLTHA.

5 PROPOSED LOAD VECTOR.

The proposal consists of a single non adaptive load vector. This vector allows the consideration of higher mode effects. It is based on the displacement load pattern produced by relevant vibration modes. To determine this load pattern an eigenvalue analysis must be performed to determine the eigenvectors/mode shapes. Subsequently these eigenvectors are multiplied by the effective modal mass factor. The load pattern is as follows:

$$Fv_n = \sum_{n=1}^{N} [EMM_n \Phi_n] \tag{1}$$

Fv_n is an applied load vector. Φ_n is the mode shape vector of mode n and EMM_n is the effective modal mass factor. EMM_n is relevant because vibration modes with high percentage values are likely to contribute significantly to structural response. The modal excitation factor L_n which depends on the mass m and the influence factor l which represents the direction of the excitation factor and the effective modal mass factor EMM_n are expressed as:

$$L_n = \Phi_n{}^T ml \tag{2}$$

$$EMM_n = \frac{L_n{}^2}{\Phi_n{}^T m \Phi_n} \tag{3}$$

Fv_n is based on the deformed shape represented by each relevant vibration mode represented by each individual effective mass factor. The objective of employing just one load pattern is to maintain simplicity. Also, it is important to mention that Fv_n must be normalized to a value of 1 on the roof. With respect to the loading/solution scheme that must be employed in this procedure, the scheme must be a “Response Control” rather than a “load control”. In a response control scheme, the structural response, i.e., displacements and or rotations are directly incremented/controlled, and depending on these increments, the load factor corresponding to such deformation level can be calculated. Therefore, the variation of the load factor, is not prescribed by the user as in a load control scheme but is instead calculated so that the applied loading vector at a particular increment corresponds to the attainment of the target response displacement/rotation at the control node. The shape of Fv_n and not the magnitude is what matters. The reason of this is (as stated before) because the purpose of the procedure is to obtain a certain response (displacement) of a particular node in the structure, i.e., a target displacement.

6 PROPOSED CORRECTIVE DISPLACEMENTS.

A corrective factor is proposed. The factor is applied to the displacements obtained from pushing the structure with the proposed lateral load vector to the target displacement obtained from the N2 method. To develop this corrective factor 6 frames where considered. The steel frames have varying heights with 3,8,12,20,25 and 30 stories. For every steel frame, eigenvalue, NLTHA and NLS analysis were performed. This was done considering all the seismic intensities previously mentioned as well as the matching spectrums. The data obtained from all the analysis was vibrating modes, effective modal mass factor, ductility, and target displacements.

With respect to the NLTHA one observes that maximum story displacement and maximum story drift may not be coincidental and if one employs the maximum story displacement obtained from the NLTHA or from the NLSA directly to calculate the story drifts the margin of error may be high.

This phenomenon is explained in the following figures obtained from the NLTHA performed on the 12-story frame. Figure 1a represents the roof displacements obtained from the 2.5 x El Centro. In this figure, the maximum displacement at the roof is 48.4 cm. In Figure 1b we can observe the story drifts (displacement at top floor minus displacements at the floor below). From comparing both graphs, one can observe that the maximum drift is obtained at 2.5 seconds and the maximum displacement at 27 seconds. One concludes that maximum drift can happen at different displacements values rather than the maximum displacement.

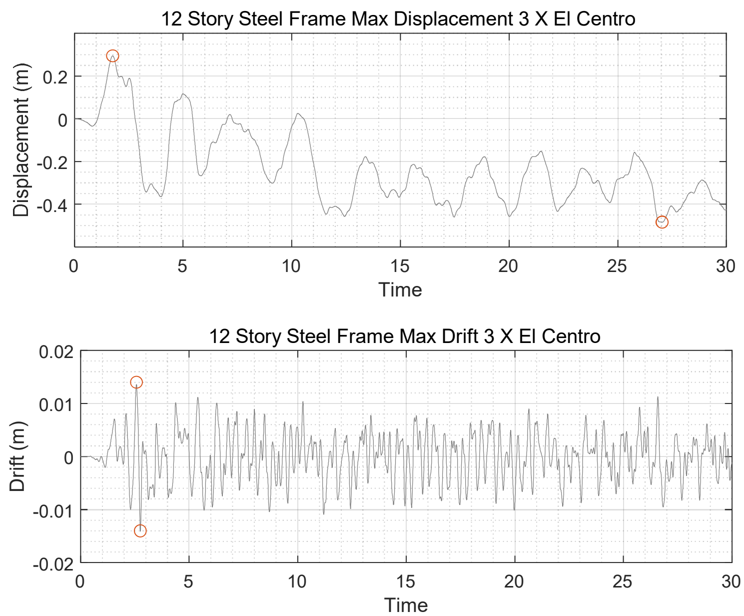
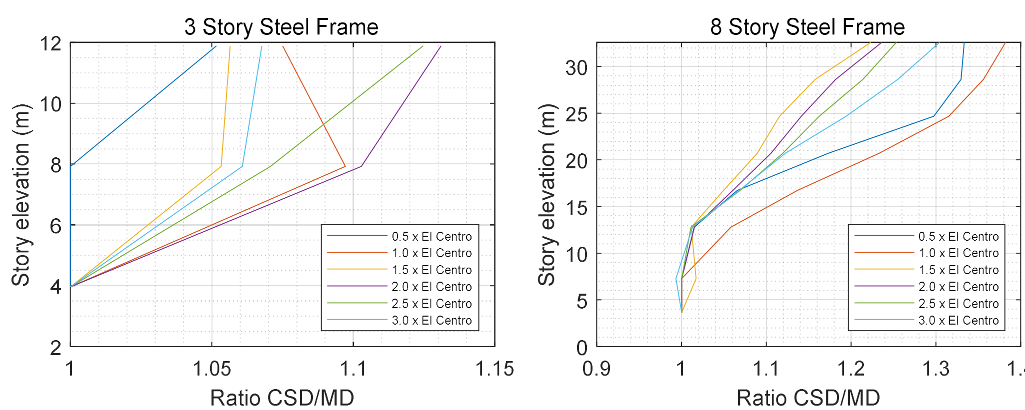


Figure 1: Max displacement and max drift for 12 story frame

The ratio of displacement values of maximum story drift in a cumulative fashion or cumulative story drifts “CSD” and maximum displacements “MD” was obtained for each frame for each seismic intensity from the El Centro record.



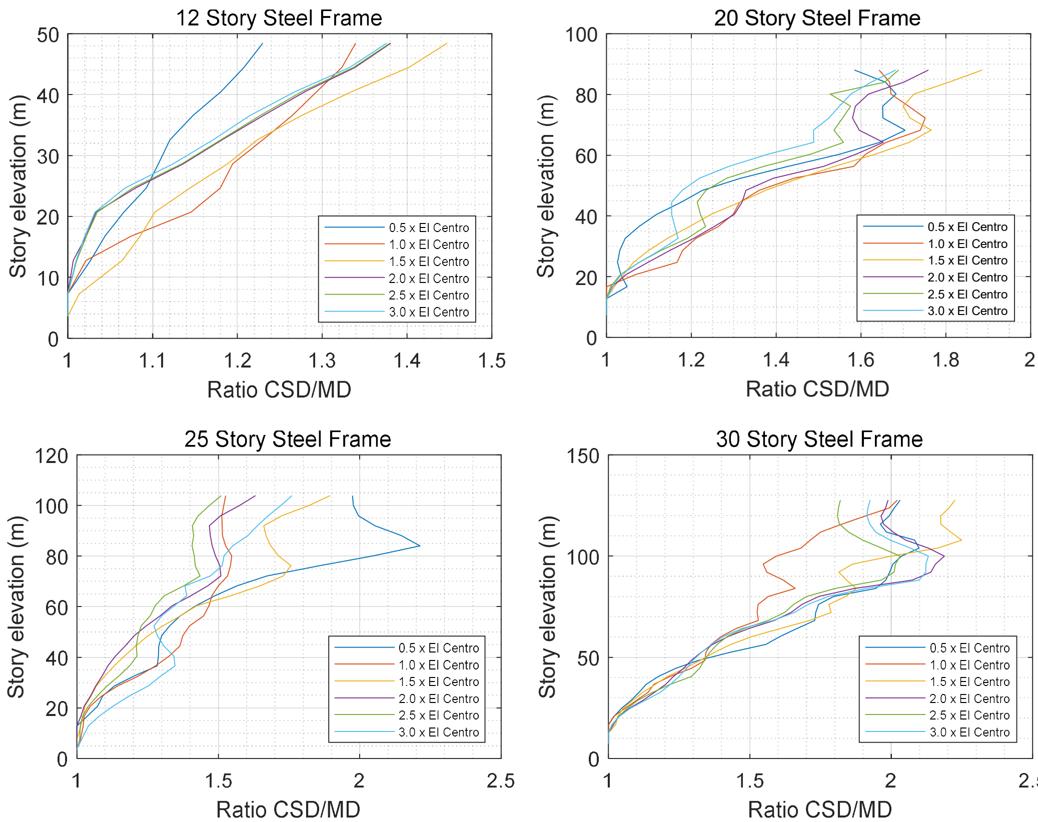


Figure 2: Ratio CSD/MD for the six frames considered.

The proposed corrective factor is expressed in equation (4) TD_{corr} represents the adjusted displacement from which one calculates story drifts. D_{target} is the vector displacement obtained from pushing the structure to the target roof displacement using the classical N2 method.

$$TD_{corr}=D_{target}\times \psi_c \tag{4}$$

The corrective factor ψ_c depends on the extent of the inelasticity, the natural frequencies of the building and the height of the building. This factor is an amplification factor so it must at least have a value of 1. The equation proposed to calculate ψ_c is:

$$\Psi_c=(\psi_h)(\psi_{dr})>1 \tag{5}$$

The first term in equation (5) represents a modification factor that depends on the story elevation with respect to the total height of the building. This factor, which is a ratio that ranges from 0.4 to 1, was obtained from the results of the NLTHA and can be observed in equation (6) and graphically in Figure 3.

$$\Psi_h=0.556*e^{\left((0.607)\left(\frac{hs}{ht}\right)\right)} \tag{6}$$

The values of the CSD versus maximum displacements MD were normalized for each frame for every scaled ground motion considered. Also, the story level height with respect to the roof were normalized. With both sets of data, the scatter 2D plot in Figure 3a was generated.

Employing the curve fitter app from MATLAB an exponential trend curve was obtained. This curve is represented in Figure 3, where hs and ht represent the height or elevation of the story and the total height of the building respectively.

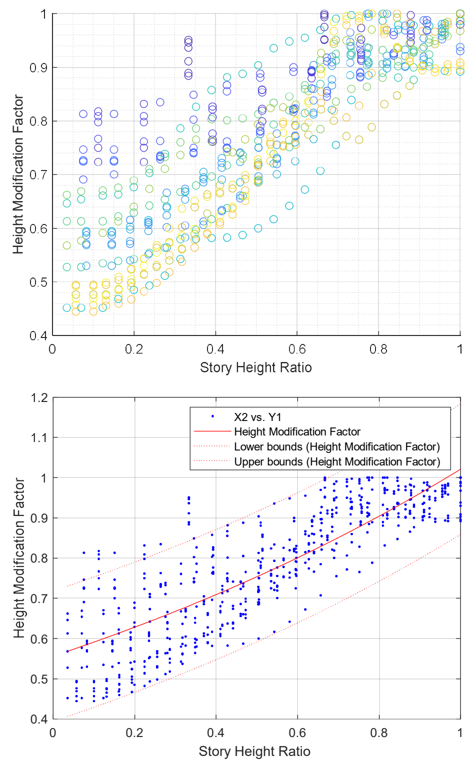
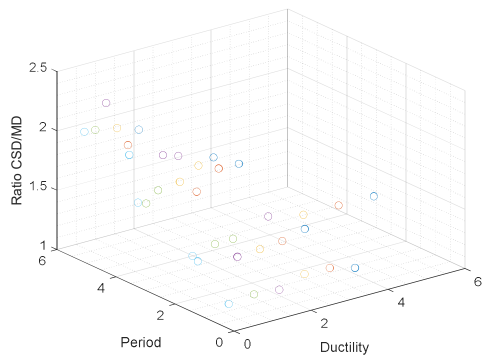


Figure 3: Height modification factor. Scattered plot with normalized data and trend curve

The term ψ_{dr} represents the modification factor for the roof. It dictates into what extent is the structure behaving inelastically. Ψ_{dr} depends on two parameters. The first parameter, ductility μ, which represents the relationship between target displacement and yield displacement of the idealized bilinear curve. The second parameter, fundamental natural frequency, which represents the contribution of the stiffness and is represented by the fundamental natural period T of the idealized structure in seconds. Each of the six frames were analyzed by the NLS procedure employing the corresponding response spectrum to obtain the target displacement. The prosed lateral load vector was included in this analysis.

$$\Psi_{dr}=0.7349+0.0464*\mu+0.2703*T-0.01868*\mu*T-0.004746*T^2 \quad (7)$$

Equation (7) is best explained in the Figure 4a. This figure shows the ratio of CSD versus maximum displacements MD (obtained from the NLTHA) for the roof. This data was matched to each degree of inelasticity μ and structural period T obtained from each NLS generated from the equivalent response spectrums. From the scatter 3D plot and employing the curve fitter app from MATLAB R2024a a polynomial, second degree trend surface was obtained. This surface has an R-square value of 0.9. This curve is the basis for the equation (7) and the result can be observed in Figure 4b.



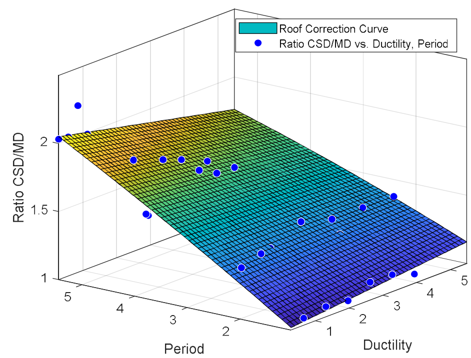


Figure 4: Roof correction factor. Scattered plot and 3d Trend.

7 PROCEDURE VALIDATION

The accuracy and effectiveness of the proposed nonlinear static procedure is assessed by comparing results obtained in displacements and drifts with the results of displacements and drifts obtained from the NLTHA. The structure employed in the verification is a nine-story steel frame present in [18]. This nine-story steel structure is part of a building that has 45.73 m by 45.73 m in plan, and 37.19 m in elevation. The bays are 9.15 m (30 ft) on center, in both directions, with five bays each in the north-south (N-S) and east-west (E-W) directions. The building's lateral load-resisting system is comprised of steel perimeter moment-resisting frames (MRFs) with simple framing on the furthest south E-W frame. The interior bays of the structure contain simple framing with composite floors. The columns are 345 MPa (50 ksi) steel. The columns of the MRF are wide flange. The levels of the 9-story building are numbered with respect to the ground level. The ninth level is the roof. The building has a basement level denoted B-1. Typical floor-to-floor heights (for analysis purposes measured from center-of-beam to center-of-beam) are 3.96 m. The floor-to-floor height of the basement level is 3.65 m ad for the first floor is 5.49 m. The column bases are modeled as pinned and secured to the ground (at the B-1 level). Concrete foundation walls and surrounding soil are assumed to restrain the structure at the ground level from horizontal displacement. The floor system is comprised of 248 MPa (36 ksi) steel wide-flange beams acting compositely with the floor slab as in the 3-story building. Like the 3-story building, each frame resists one half of the seismic mass associated with the entire structure. The seismic mass of the structure is due to various components of the structure, including the steel framing, floor slabs, ceiling/flooring, mechanical/electrical, partitions, roofing and a penthouse located on the roof. The seismic mass of the ground level is 9.65×10^5 kg (66.0 kipssec2 /ft), for the first level is 1.01×10^6 kg (69.0 kips-sec2/ft), for the second through eighth levels is 9.89×10^5 kg (67.7 kips-sec2/ft) and for the ninth level is 1.07×10^6 kg (73.2 kips-sec2/ft). The seismic mass of the above ground levels of the entire structure is 9.00×10^6 kg (616 kips-sec2/ft). The 9-story N-S MRF is depicted in Figure 5.

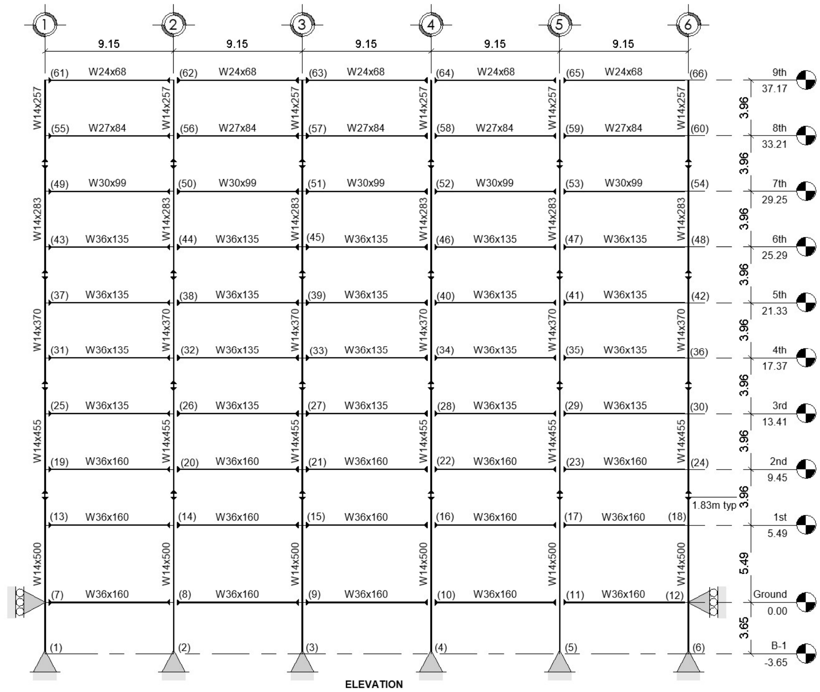


Figure 5: 9 story moment resisting frame.

For the NLTHA evaluation, the 1.0 and 1.5 x El Centro ground motion is employed. For the NLS evaluation the matching response spectrums based on a seismicity zone type 1, Ground type B, damping at 5% will have 0.3 and 0.45 PGA respectively. This proposal is based two parts. The first part is a single, non-adaptive, lateral load vector present in equations (1) to (3) from which one obtains the displacements. These displacements are not the corrected displacements. The corrected displacements in equations (4) to (7) are employed only for drift calculation purposes. After obtaining the results the graphs in the following figures were developed. These graphs contain the displacements obtained from the NLTHA and the NLS evaluation employing the proposed lateral load vector, which is the first part of the proposal. The displacements for each story level are in meters. As the graphs show the results from the proposed load vector are in agreement with the results from the NLTHA.

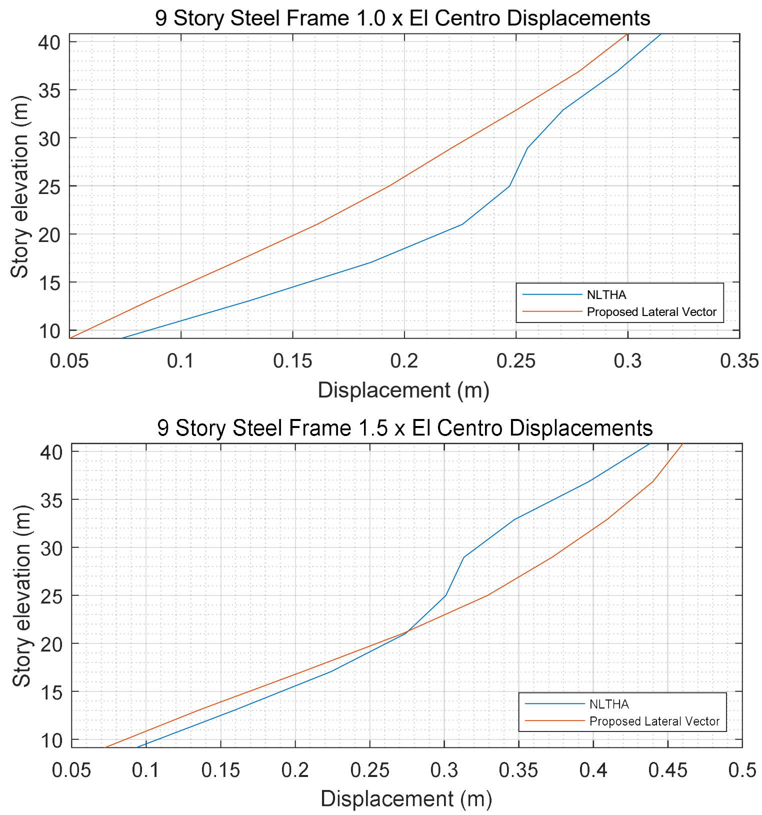
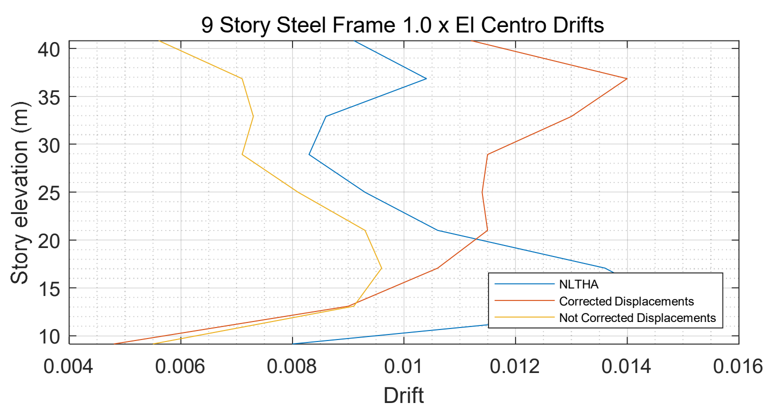


Figure 6: Displacements from NLTHA and NLS for 1.0 and 1.5 x El Centro ground motion

The drifts obtained from the corrected displacements (second part of the proposal) for the nine-story structure are presented below. The drifts are displayed for each story level. The corrected displacements, as shown in the graphs, allow the calculation of drifts that have similar magnitudes as the ones from NLTHA in the upper levels.



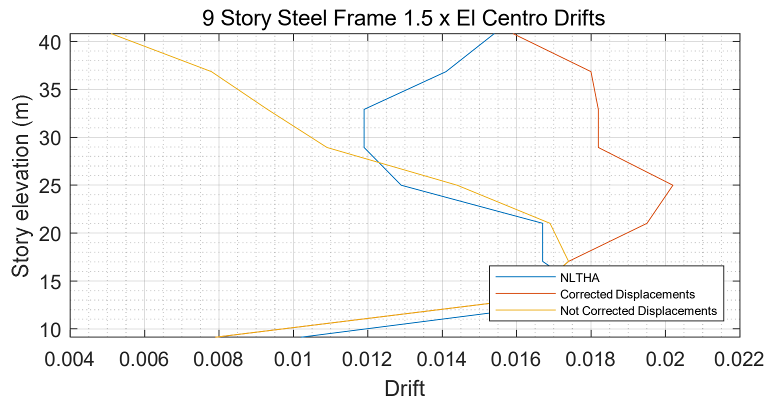


Figure 7: Drifts from NLTHA and NLS for 1.0 and 1.5 x El Centro ground motion

8 CONCLUSIONS

- A performance analysis procedure based on a unique single load vector which depends on the effective mass factor proposed. This load pattern in combination with the N2 method allows the calculation of an accurate target displacement for steel 2D frames.
- The Modal analysis allows only a certain degree of precision in the inelastic range. As was shown in this study, the proposed lateral load pattern allows in a simple manner the inclusion of several vibrating modes in a single run static analysis.
- Story drift calculated from the NLTHA has a unique characteristic in the way that maximum displacement in each story may not represent maximum story drift. This phenomenon tends to become more relevant as structures become more sensitive to higher mode effects.
- Story height has a large effect on the drifts. The higher the story the larger the ratio of the maximum drift displacement to maximum displacement. This was concluded after observing the graphs obtained from the NLTHA.
- The level of ductility, as was shown on the analysis, has a small level of incidence in the corrective factors. On the other hand, fundamental period is of great relevance because it amplifies greatly the corrective factor. The greater the period the more flexible a structural system tends to become. So, it is expected that less rigid systems present higher ratios from CSD versus maximum displacements.
- This procedure considers a corrective factor that combines story height percentage, ductility, and fundamental period. The comparison to the NLTHA proves that the proposed procedure gives acceptable predictions for displacements and drift values for a NLSA.

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