EVALUATION OF EXISTING MEXICAN HIGHWAY BRIDGES UNDER MAINSHOCK-AFTERSHOCK SEISMIC SEQUENCES

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ABSTRACT :

This paper presents the results of an analytical investigation aimed at evaluating the inelastic response of typical Mexican reinforced concrete highway bridges under main shock-aftershock seismic sequences. For that purpose, a suite of 28 mainshock-aftershock seismic sequences compiled from recording stations located near the subduction zone of the Mexican Pacific coast were considered in this study. A family of 9 two-dimensional bridge model taken into account typical configuration of low-height highway bridges in Mexico was modeled and subjected to the selected seismic sequences. The results indicate that the Mexican highway bridges considered in this investigation do not experience significant lateral peak and residual drift demands under as-recorded mainshocks due to their high inherent structural overstrength and their relatively small low-to-medium earthquake intensity. However, the studied bridges might increase their lateral drift demands when aftershocks are considered. Furthermore, when seismic sequences are scaled to represent stronger ground motion intensities, the level of peak and residual drift demands tends to increase as a consequence of aftershocks. The level of increment of peak and residual drift demands depends on the type of hysteretic behavior considered in the columns and the level of ground motion intensity.

KEYWORDS: highway bridges, seismic sequence, aftershocks, residual drift

1. MOTIVATION

Nowadays, there is a consensus among the earthquake engineering community that the structural damage is a consequence of lateral deformation demands imposed to the structures during earthquake ground shaking. Thus, modern seismic assessment procedures are based on the evaluation of lateral peak inelastic displacement demands due to main earthquake shocks. However, man-made civil engineering structures are exposed to a sequence of foreshocks, main shock, and aftershocks during earthquake events. As a consequence, post-earthquake field reconnaissance has shown that damaged or even undamaged structures due to main shock attack might increase their damage state due to the presence of aftershocks. For instance, several reinforced concrete piers of the interstate I-5/I-605 separator, a nine-span freeway overpass bridge, damaged after the October 1, 1987 Whittier Narrows earthquake (M=5.9) increased their structural damage after the main aftershock (M=5.3) occurred three days later (Priestley, 1988). Thus, modern seismic assessment procedures should take into account the effect of main shock-aftershock seismic sequences. In particular, post-earthquake field reconnaissance have evidenced that residual (permanent) lateral displacement demands after earthquake excitation (e.g. residual roof drift ratio or maximum residual inter-story drift ratio) should also play an important role in the evaluation of structural performance in addition to maximum (transient) lateral displacement demands and floor acceleration. For example, many RC bridge piers were demolished in Kobe after the 1995 Hyogo-Ken-Nambu earthquake for the elevated cost that would be required to repair piers with large permanent drifts in excess of 1.5% (Kawashima, 2000). In addition, residual drift demands have been identified as one of the most important response parameters in the evaluation of the residual capacity of damaged structures to sustain aftershocks (Mackie and Stojadinovic, 2000).

The objective of the investigation reported in this paper consisted on evaluating the response of typical Mexican highway reinforced concrete bridges when subjected to mainshock-aftershock seismic sequences recorded on accelerographic stations placed on firm sites located near the subduction zone of the Mexican Pacific coast. Special emphasis is given to evaluate the level of residual (permament) drift demands at the end of the seismic excitation.

2. STUDY CASES OF MEXICAN HIGHWAY BRIDGES

2.1. Selected study cases

From an inventory of 76 Mexican highway bridges, typical of the Mexican highway bridge system, it was revealed that about 45.3% of the bridges were built in the period from 1960 to 1980. Among them, around 53.7% have a substructure height (i.e. bridge pier including beam cap) shorter than 5m, while 31.7% and 12.2% have substructure height between 5-10m and 10-15 m, respectively. In addition, 13.2% included reinforced concrete single, two-column, or multi-column bents in the transverse direction (Figure 1).

Figure 1 Typical transverse configuration of R.C. highway bridges built in Mexico

For the purpose of evaluating the seismic response of typical Mexican highway bridges under a mainshock-aftershock scenario, bridge models representative of single-, two-, and three-column bents in the transverse direction having 5m column-height were developed. Circular column cross- section diameter of 1.5m and 1.2m corresponding to single-column and two- or three-column was assumed for bridge models. In addition, for each circular column, it was considered three cases aimed at representing the amount of longitudinal, ρ_l , and transverse, ρ_t , steel reinforcement ratio assigned during the design phase of bridges before 1972 (case I), from 1972 to1992 (case II), and after 1992 (case III). The amount of ρ_l and ρ_t corresponding to each study case is reported in Table 2.1. Thus, a family of 9 bridge models was considered in this investigation. For identification purposes, the nomenclature B**X**C-**Y** was employed to identify a bridge model with **X** number of columns and corresponding to the case **Y**.

Table 2.1 Amount of steel reinforcement considered in this investigation

Case	ρ_l [%]	ρ_t [%]
I	0.75	0.15
II	1.50	0.50
III	2.00	0.70

2.2. Bridge modeling

Each bridge configuration was modeled as a two-dimensional centerline frame using the computer software RUAUMOKO (Carr 2004). Figure 2 shows a three-column bridge bent model. The columns were modeled using frame elements were inelastic deformation was restricted to plastic hinges at both ends. The well-known Takeda (TK) model, including moderate and sever strength-degradation, was considered for modeling the nonlinear moment-curvature relationship at plastic hinges. The moment flexural capacity of each column was obtained from fiber-based moment-curvature analysis using the software XTRACT (ISS, 2004). In all cases, while unconfined and confined concrete compressive strength, $f_c^{'}$, was taken as 250 kg/cm^2 and 289 kg/cm^2, respectively, reinforcing steel yield strength, f_y, was considered as 4218 kg/cm^2. An axial load ratio of 0.1 was assumed for all moment-curvature analyses. In addition, while the lead rubber bearings were modeled using vertical spring elements having bilinear hysteretic behavior, the beam cap was modeled as an elastic frame element. All bridge models were assumed fixed with negligible soil-structure interaction.

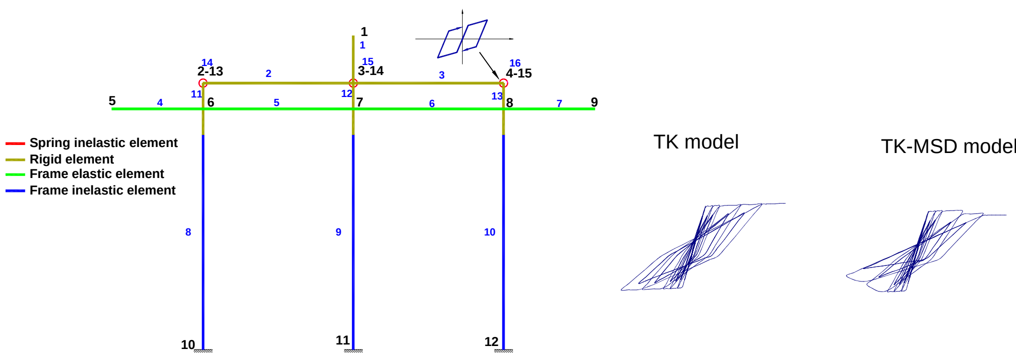

Figure 2 Example of bridge's transverse configuration modeling considered in this study

2.3. Lateral strength and deformation capacity

Once each bridge configuration was modeled, a nonlinear pushover analysis was performed in order to determine the lateral strength capacity (i.e. normalized base shear strength with respect to its weight) and deformation capacity (i.e. yield displacement at top pier) of each bridge study case using the computer software RUAUMOKO (Carr, 2004). A comparison of the design elastic and inelastic (i.e. assuming a response modification factor of two to account for nonlinear behavior) normalized lateral strength spectrum established for a firm soil site in a region of high seismicity in Mexico with respect to the yield strength capacity of each bridge model is shown in Figure 3. It should be noted that lateral yield strength capacity obtained for some of the bridge models reflex the structural over-strength inherent in Mexican bridge design practice.

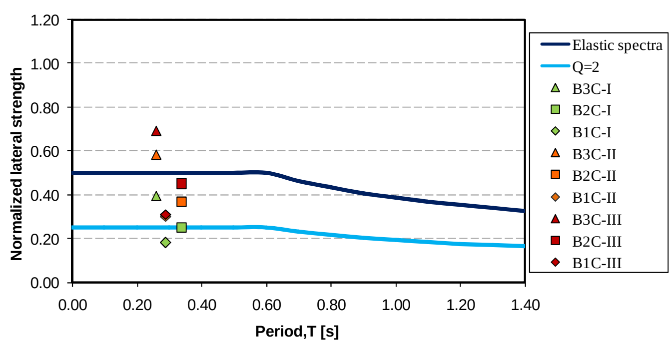

Figure 3 Comparison of normalized lateral strength capacity of bridge models with respect to elastic and inelastic design spectra for a firm site in a region of high seismicity in Mexico

3. MAINSHOCK-AFTERSHOCK GROUND MOTION ENSAMBLE AND INTENSITY MEASURE

In order to study the behavior of typical existing highway Mexican bridges under mainshock-aftershock sequences representatives of the seismic hazard in the subduction zone of the Pacific coast, an ensemble of ground motion seismic sequences was assembled from the Mexican Database of Strong Motions (SMIS, 1999). For this purpose, seismic sequences including the mainshock and at least one aftershock recorded from 1960 to 1999 were identified from the database. Thus, around 500 seismic sequences were first identified. Next, seismic sequences were selected according with the following criteria: a) mainshock magnitude equal to or greater than 5.5 and afterhock magnitude equal to or greater than 4.0; b) available information about the soil condition; c) acceleration time histories recoded on stations placed on free field or low-height buildings were soil-structure interaction effects were negligible; and d) seismic sequences having peak ground acceleration of one of the mainshock horizontal component greater than 100 cm/s^2. Under these criteria, 28 seismic sequences were selected for this investigation. For illustration purposes, Figure 4 shows 5 seismic sequences included in the database. It should be noted that in some seismic sequences the intensity of the aftershock (measured by the peak ground acceleration) was greater than that of the corresponding mainshock, although the magnitude of the mainshock was greater than that of the main aftershock. As an example, seismic sequence PAPN850919 recorded in station PAPN, placed in Papanoa town located near the epicenter area, during the mainshock (September 19, 1985, Ms=8.1) and the main aftershock (September 20, 1985, Ms=7.6) is shown in Figure 4.

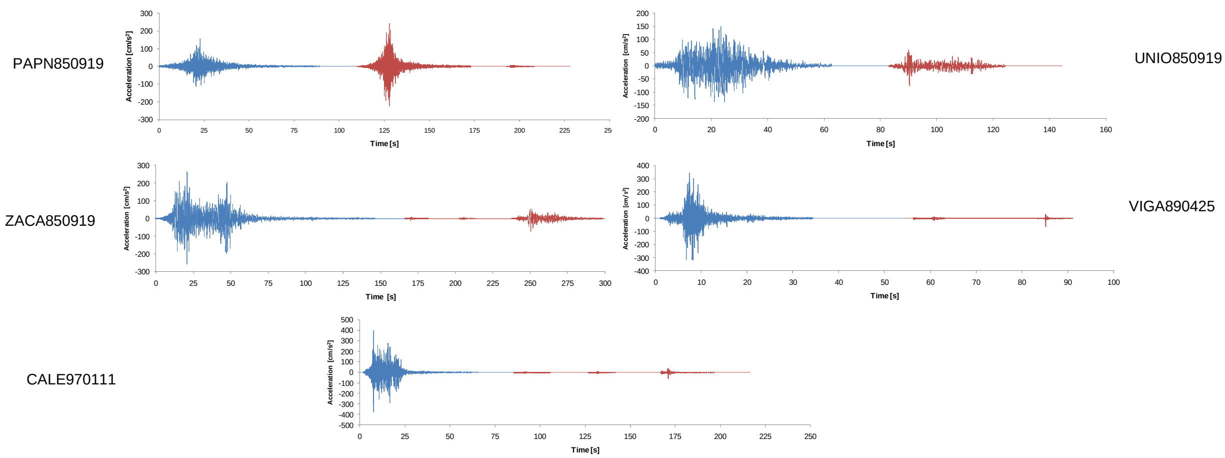

Figure 4 Examples of seismic sequences recorded near the subduction zone of the Mexican Pacific coast

Of particular interest to this investigation was the estimation of drift demands in the bridge models under a set of mainshock-aftershocks sequences. Since most of the bridge models reflex the structural overstrength involved in their design phase, it is expected that they do not exhibit nonlinear behavior under some seismic sequences. Thus, in order to induce the nonlinear response, it was decided to scale up each seismic sequence to represent different levels of intensity. This procedure is commonly known as Incremental Dynamic Analysis (Vamvatsikos and Cornell 2002). Therefore, for a specific bridge model, all acceleration time histories were scaled to reach the same maximum inelastic displacement demand, corresponding to multiples of the bridge´s yield displacement Δ_y obtained from pushover analysis, in an equivalent elastoplastic single-degree-of-freedom (ESDOF) system with the same fundamental period of vibration (i.e. same initial lateral stiffness) of the bridge model, T_1. This scaling process guarantees that each bridge model will behave nonlinearly under each seismic sequence. In addition, maximum inelastic displacement can be related to exceedance probability, or return period, for a specific structure using the procedure suggested by Ruiz-Garcia and Miranda (2007). Thus, all nonlinear dynamic analysis were performed using the computer software RUAUMOKO (Carr, 2004) taking into account P-Delta effects due to large displacements.

4. RESULTS

4.1. Response under individual seismic sequences

Due to space limitations, only few relevant observations are presented as follows. Figure 5b shows the response of B1C-II model (T_1=0.43s, c_y=0.30, Δ_y=1.44 cm) subjected to the seismic sequences shown in Figure 4 when scaled to an intensity level of two (i.e. each seismic sequence was scaled to reach two times Δ_y in an ESDOF). For comparison purposes, Figure 5a shows similar response of the bridge model under the unscaled seismic sequences. It can be seen that residual drift demands after mainshock, θ_r, does not significantly increase as a consequence of the aftershocks even when the seismic sequences are scaled. This issue could be explained since the ratio of peak ground acceleration (PGA) of the main aftershock and the PGA of the mainshock was kept constant.

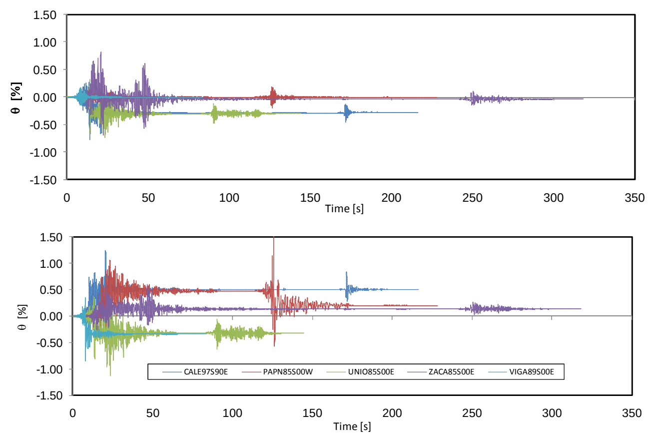

Figure 5 Influence of the type of seismic sequence on the response of B1C-II model under unscaled and scaled sequences shown in Figure 4

Figure 6 show a comparison of the response of B3C-I (T=0.26s, c_y=0.40, Δ_y=0.69 cm), B3C-II (T=0.29s, c_y=0.58, Δ_y=1.07cm), and B3C-III (T=0.26s, c_y=0.69, Δ_y=1.22cm) bridge models when subjected to the as-recorded seismic sequence PAPN850919 (left-side) and the sequence scaled to an intensity level of two (right-side). For these study cases, it can be seen that the amount of reinforcement does not have a significant influence on the response of the bridge models and, furthermore, that the aftershock does not increase lateral drift demands. However, when the bridge models are subjected to the scaled seismic sequence, it can be observed that the amount of steel reinforcement has significant effect on constraining residual drift demands at the end of both the mainshock and aftershock.

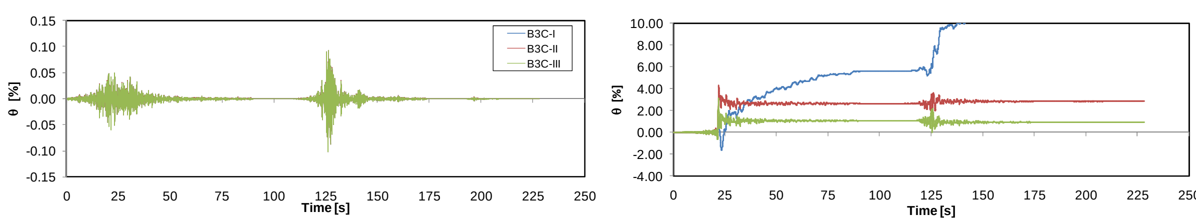

Figure 6 Influence of the amount of steel reinforcement on the response of B3C models under scaled PAPN sequence

During this investigation, it was observed that the effect of strength deterioration included in the hysteretic modeling of bridge columns might constraint the level of residual drift demands after mainshock, which means that the bridge model tends to re-center to its original position. This re-centering capability could be explained since stiffness-and-strength degrading models leads to smaller hysteresis loops after large inelastic excursions. For example, while left-side of Figure 7 shows the response of the B3C-II model (T=0.29s, c_y=0.58, Δ_y=1.07cm) subjected to the seismic sequence ZACA scaled to reach the bridge´s yield displacement and assuming Takeda hysteretic behavior without strength degradation (TK, blue line), moderate (TK-MSD, red line) and severe (TK-SSD, light green line) strength degradation, right-side shows the bottom-end moment-curvature response the middle column. This re-centering capability that constraint residual deformations was previously observed by Ruiz-Garcia and Miranda (2006).

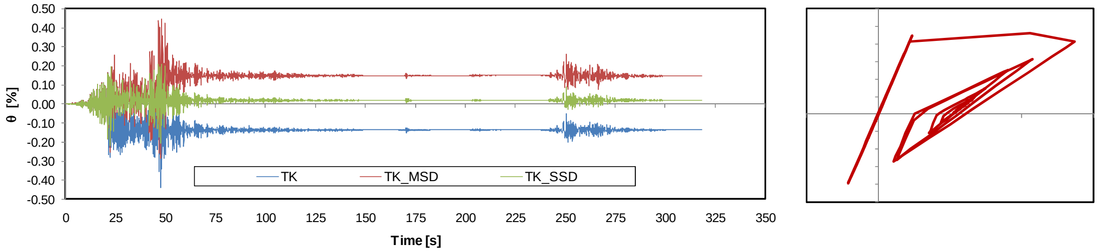

Figure 7 Influence of hysteretic behavior on the response of B3C-II model under scaled ZACA sequence

4.2. Response under all 28 seismic sequences

Statistical studies employing all 28 seismic sequences, scaled and unscaled, were also developed as part of this investigation. Due to space limitations, only few relevant observations are given. For example, while the variation of median maximum drift demand, $\theta_{\max}$, and median θ_r (blue line when including moderate TK-MSD model and red-line when employing TK model) with changes in the intensity level for B1C-I model (T =0.43s, c_y =0.18, Δ_y =0.85 cm) under all 28 main shocks is shown in the left-side of Figure 9, a similar representation taking into account all 28 mainshock-aftershock sequences is illustrated in the right-side. For reference purposes, median $\theta_{\max}$ and median θ_r (dash-green line) obtained from unscaled seismic sequences using TK hysteretic model is also shown in each figure. It should be noted that for an intensity level of two (i.e. all seismic sequences were scaled to reach twice the bridge pier's yield displacement), bridge models including strength-and-stiffness degrading hysteretic features could lead to smaller $\theta_{\max}$ and θ_r demands than bridge models that include only stiffness-degrading hysteretic features. However, as the seismic intensity level increases the level of $\theta_{\max}$ and θ_r significantly increases when using TK-MSD hysteretic model. It can also be seen that the level of $\theta_{\max}$ and θ_r can increase when aftershocks and strength-and-stiffness degrading hysteretic behavior are considered. For example, for intensity level of two, median $\theta_{\max}$ and θ_r could increase about 25% and 4% as a consequence of aftershocks with respect of median $\theta_{\max}$ recorded due to the mainshocks. An important observation for seismic evaluation is that the level of record-to-record variability increases as the ground motion increases while estimating both central tendency of $\theta_{\max}$ and θ_r .

Next, Figure 10 shows the evolution of $\theta_{\max}$ and θ_r obtained for the B3C-I model (T =0.26s, c_y =0.40, Δ_y =0.69 cm) when subjected to the mainshock and the mainshock-aftershock sequence. Under as-recorded (unscaled) seismic sequences, the bridge model increases the level of $\theta_{\max}$ in approximately 15% as a consequence of the aftershocks, while the level of θ_r is negligible. The low levels of median lateral drift demands and negligible residual drift demands reflex the overstrength inherent in this type f bridges. It can also be seen that considering TK-MSD hysteretic behavior in the columns leads to larger $\theta_{\max}$ and θ_r demands than when TK model. In addition, it can be seen that the aftershocks increases the level of $\theta_{\max}$ and θ_r demands as the ground motion intensity increases either using TK-MSD and TK hysteretic models. For example, when TK hysteretic model is considered, while the level of $\theta_{\max}$ increases about 8% and 9% as a consequence of aftershocks when seismic sequences are scaled to reach two and three times the bridge pier´s yield displacement, the level of θ_r increases approximately 2% and 28% for the same intensity levels, respectively. For this study cases, record-to-record variability was increased as the level of ground motion intensity increased.

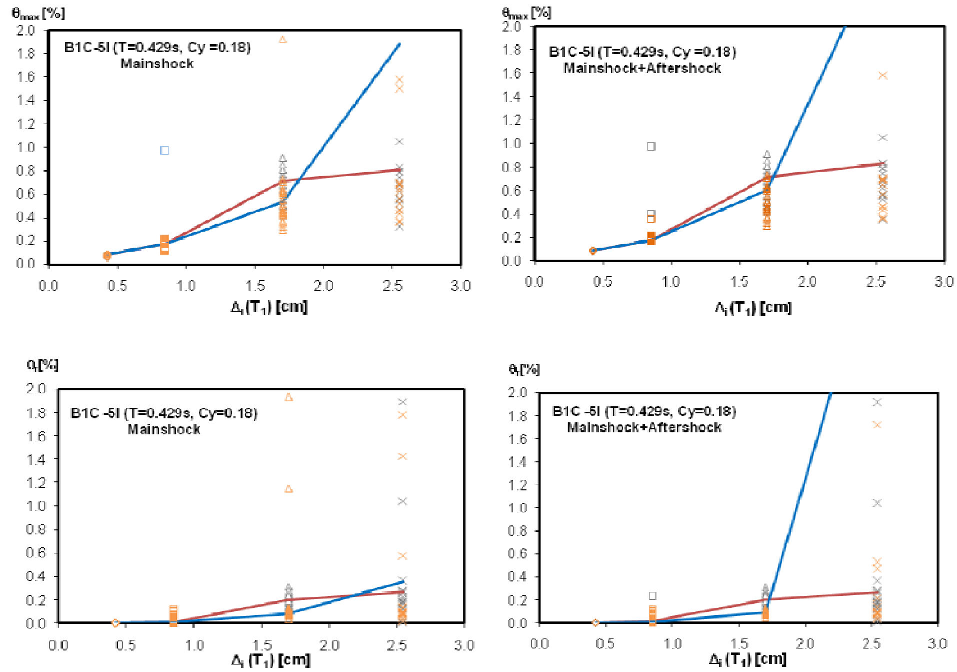

Figura 9 Evolution of $\theta_{\max}$ and θ_r as the level of seismic intensity increases for B1C-I model subjected to 28 mainshock and corresponding 28 mainshock-aftershock sequences

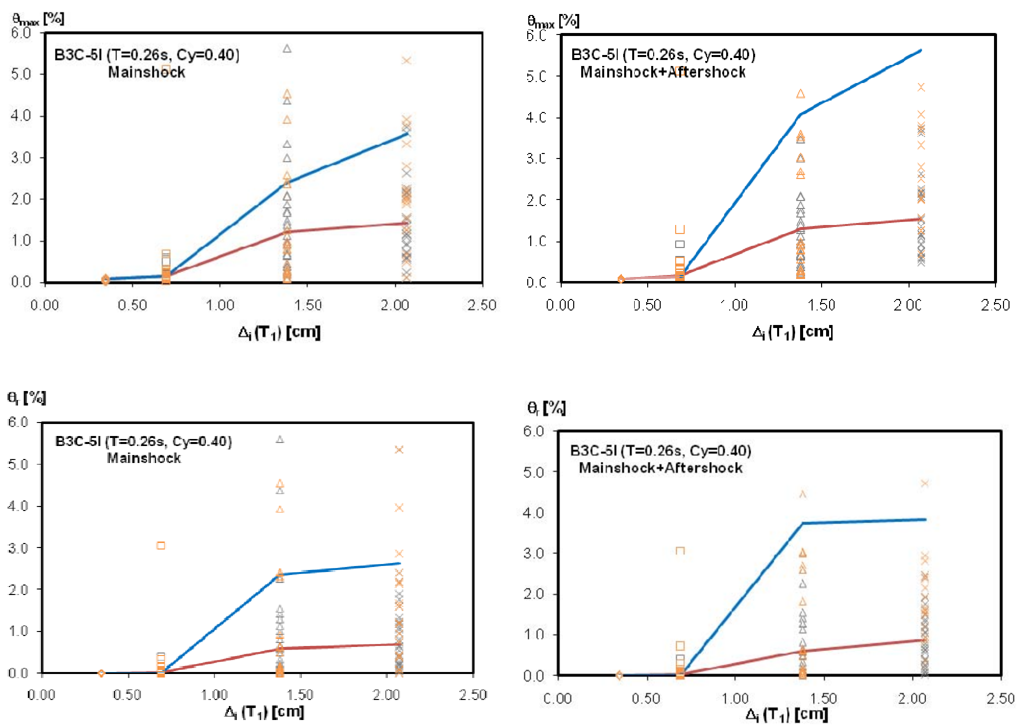

Figura 10 Evolution of $\theta_{\max}$ and θ_r as the level of seismic intensity increases for B3C-I model subjected to 28 mainshock and corresponding 28 mainshock-aftershock sequences

5. CONCLUSIONS

This paper presented relevant results related to the evaluation of two-dimensional bridge models representative of typical existing Mexican highway reinforced concrete bridges subjected to 28 mainshock-aftershock seismic sequences recorded on accelerographic stations located near the subduction zone of the Mexican Pacific coast. From the results obtained in this investigation, the following conclusions are drawn:

- Aftershocks in as-recorded seismic sequences do not significantly increase large lateral drift demands since drift demands under mainshock are relatively low. This could be explained due to the inherent overstrength in the low-height highway bridges and that the as-recorded seismic sequences have low-to-medium earthquake intensity.
- Under specific seismic sequences, the effect of strength degradation tends to diminish residual drift demands due to the re-centering capability in the hysteretic response of bridge columns.
- Under scaled seismic sequences, the effect of aftershocks tends to increase both median $\theta_{\max}$ and median θ_r when the bridge models behave nonlinearly during the mainshock. The increment depends on the level of the ground motion intensity and the hysteretic behavior considered in the columns.
- The record-to-record variability in the estimation of central tendency of $\theta_{\max}$ and θ_r of bridge models under scaled seismic sequences increased as the ground motion intensity increased. In particular, the variability in the estimation of θ_r was higher than that of θ_r.

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