PUSHOVER ANALYSIS OF A TWO-SPAN MULTICOLUMN BENT RC BRIDGE, EXPERIMENTALLY TESTED ON THREE SHAKE TABLES

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ABSTRACT :

A large scale model of two-span two-column bent RC bridge, representing one frame of a continuous bridge typical for the US practice was experimentally tested on three parallel shake tables at the UNR in the frame of the NEES project "Seismic Performance of Bridge Systems with Conventional and Innovative Materials", coordinated by Prof. M. Saiidi. Although the bridge seems to be slightly irregular, its response was quite complex. The intensity as well as the direction of the deck rotations was significantly varying depending on the seismic intensity.
At the ULJ, the investigated bridge was numerically analysed using the three typical pushover methods: a) the standard single mode N2 method, b) the non-adaptive multimode MPA method, and c) the adaptive multimode IRSA method. For lower seismic intensity levels, all methods estimated the displacement shape of the deck very well. For higher seismic intensity the single-mode method was less accurate. The estimation of the response made by the MPA was quantitatively relatively good, but the qualitative differences with the experiment were observed. Actual response was mostly influenced by one predominant mode, which varied the shape. In the MPA, the higher mode coincidentally compensated these changes. They were successfully identified by the IRSA method, where the estimation of the deflection shape of the superstructure was good. A poorer match with the experiment was observed only for the largest seismic intensity considered in the analytical studies. This discrepancy was mainly caused by the varying ratio of the bent stiffnesses after their yielding.

KEYWORDS: Pushover methods, Simplified nonlinear analysis, RC bridge, Experiment.

1 INTRODUCTION

Pushover methods have become a standard tool for the analysis of different types of structures. They are included into majority of the modern codes (e.g. EC8/2 - CEN 2005, FEMA 356 - ASCE 2000). However, due to the partial understanding of their limitations, these methods are frequently used indiscriminately, and sometimes even erroneously. Their indiscriminate use is particularly typical for bridges. The principles, rules and procedures which were originally developed for buildings have often been simply extrapolated to bridges, neglecting the major differences between these structural systems and their seismic response. The difference in the structural system is particularly important when the response of a bridge is analyzed in the transverse direction (Isaković & Fischinger, 2006).

The number of studies of the applicability of the pushover methods for the analysis of bridges is still small if compared to those performed on buildings. Previous investigations (Isaković & Fischinger, 2006), (Isaković et al., 2008) presented by the authors of this paper, where mostly related to the single-column bent viaducts. These and several other studies (Paraskeva et al., 2006), (Lupoi et al., 2007), (Pihno et al., 2007) were mostly analytical. Within the research, presented in the paper, the findings of the analytical studies were tested by means of the experimentally observed response of a typical RC bridge. Three typical methods investigated in the previous studies were employed: a) the N2 method (Fajfar & Fischinger, 1987, Fajfar et al, 1997) as a typical single mode pushover method, which is included into the EC8, b) the MPA method (Chopra & Goel, 2002), as a typical non-adaptive multimode pushover method, and c) the IRSA method (Aydinoglu, 2002) as a typical adaptive multimode method.

The experiment, which was used to examine the applicability of these pushover methods, was performed in the frame of the extensive NEES project "Seismic Performance of Bridge Systems with Conventional and Innovative Materials" (UNR, 2008). It has been coordinated by Prof. M. Saiidi from the University of Nevada, Reno (UNR). Several tests on three parallel shake tables have been performed in the frame of this project. One of the main purposes of the project has been to verify the computer simulations by means of experimental data in order to establish the reliability of the analytical studies. Only the so-called pre-NEES experiment is studied in this paper (see Figure 1). A two-span two-column bent bridge, typical for the US practice was analyzed. The main properties of this bridge are briefly described in Chapter 2. The related analytical model, used for the analytical studies is presented in Chapter 3. The main properties of the experimentally observed bridge response are described and analyzed in Chapter 4. The last, Chapter 5 is devoted to the comparison of the experimental results and analytical results.

Figure 1. The pre-NEES experiment performed at the University of Nevada, Reno
(principal investigator Prof. M. Saiidi)

2 THE SHORT SUMMARY OF THE EXPERIMENT

The analyzed bridge is moderately irregular structure, representing one frame of the continuous bridge, typical for the US practice. The 1:4 scale model is presented in Figure 2. This is a two span bridge, supported by three two-column bents. The length of the individual span was 9.14 m. The bridge model was supported by 6 columns of diameter 30.5 cm. The height of columns was 1.83m, 2.44m and 1.52m for the bents B1, B2 and B3, respectively. The geometry of the bent B1 is presented in Figure 2. Columns were reinforced by 16 bars of diameter ϕ9.5. The transverse reinforcement consisted of the spiral reinforcement of diameter ϕ4.9. The space between transverse bars was 32 mm.

Bridge was subjected to different seismic intensity levels. The main investigation was related to 7 high amplitude tests, noted as tests T12 – T19. These tests (except the last one) were included in the analytical studies. The intensity of the table accelerations were varied between 0,08g (T12) and 2,11g (T19). Due to the interaction between the structure and shake tables, the excitations at the tables where somewhat different. Typical acceleration and displacement spectra corresponding to the accelerations applied during the test T14 are presented in figure 3. More detailed description of the bridge and the applied load can be found in (Johnson et al. 2006).

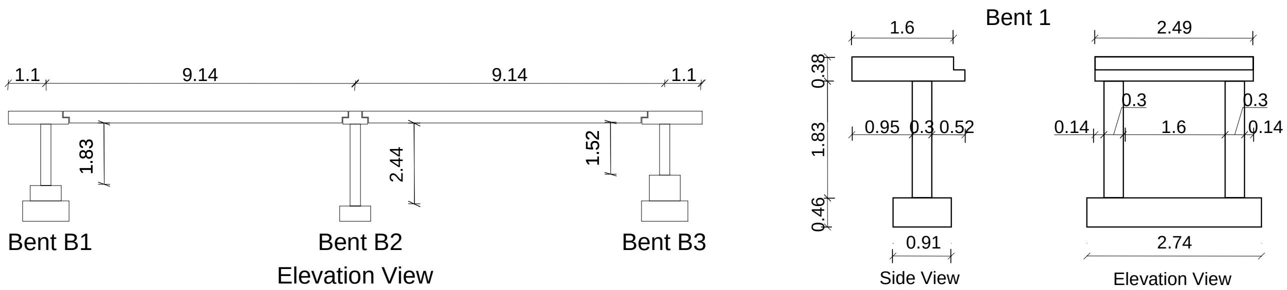


Figure 2. Scheme of the investigated bridge

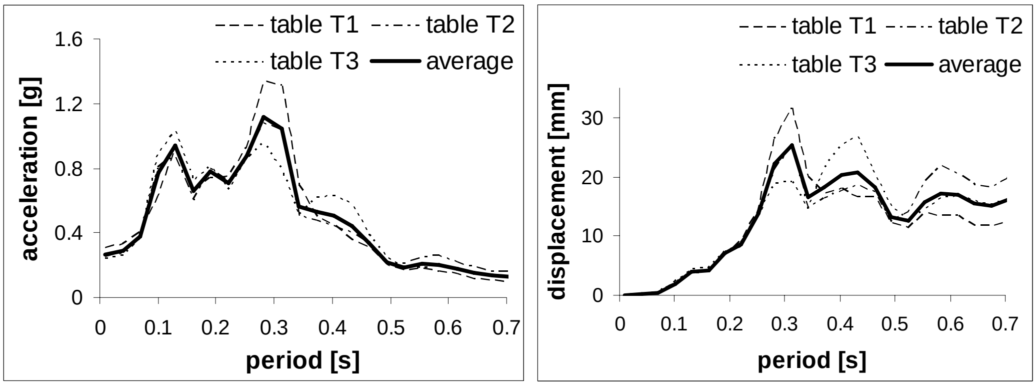


Figure 3. The typical acceleration and displacement spectra (test T14; average PGA = 0.31g)

3 ANALYTICAL MODEL

An analytical model of the bridge is schematically presented in Figure 4. Superstructure was modeled using elastic beam-column elements. Columns were modeled using the Nonlinear Beam Column Elements (Mazzoni et al., 2006). The cross-section was modeled using fiber model. Different properties for the core and the cover concrete were considered. Buckling of the longitudinal reinforcement was also taken into account.

To take into account the increased curvature over the plastic hinge length (taking into account the pull out of reinforcing bars, the influence of the shear cracks and the damage observed in the previous tests) the rotational springs were added at the top and at the bottom of each column. The detailed description of the analytical model could be found elsewhere (Isaković and Fischinger, 2007).

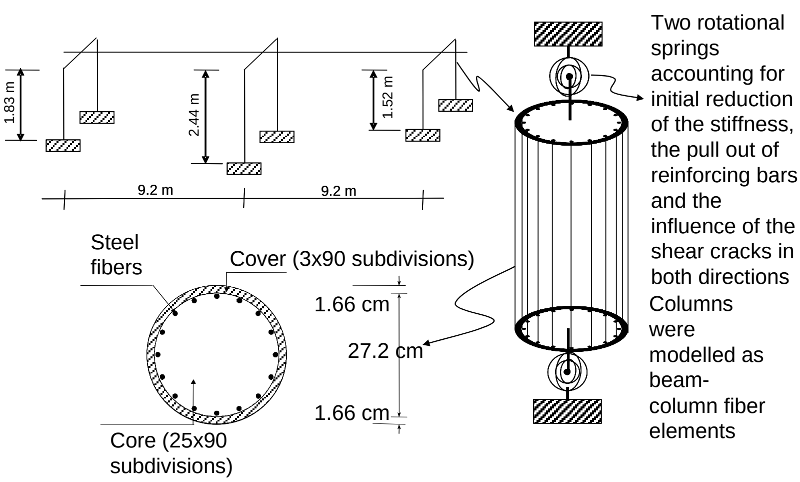


Figure 4. Analytical model of the bridge

4 EXPERIMENTALLY OBSERVED RESPONSE

The response of the analyzed bridge was predominantly influenced by the ratio of the stiffness of the side bents (B1 and B3). This ratio strongly influenced the mode shapes of the bridge. Therefore, a detailed study of the variations of the mode shapes, depending on the ratio of the stiffness of the side bents was performed. It is schematically presented in Figure 5.

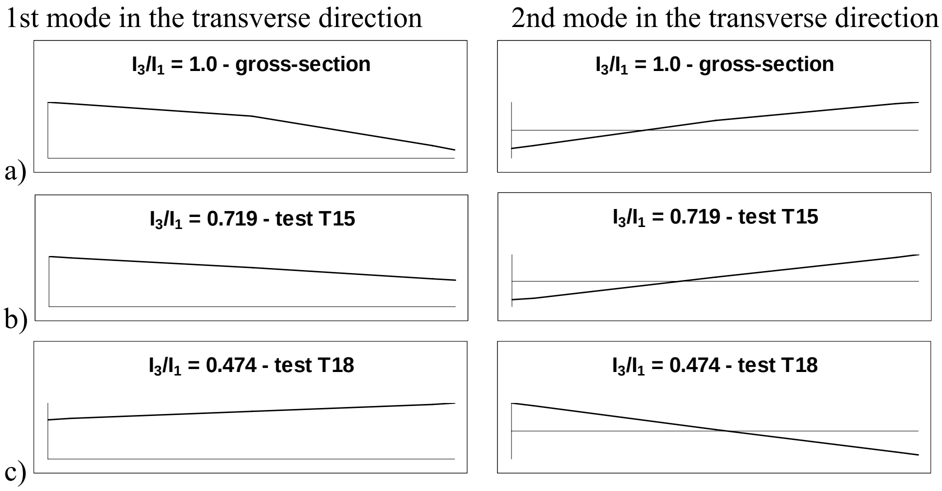


Figure 5. Changes of the mode shapes depending on the ratio of the stiffness of the bents B1 and B3

When the columns of the bents B1 and B3 were uncracked, the moments of inertia of side columns were the same ($I_3/I_1 = 1$) and the ratio of the bent stiffness k_3/k_1 (k_1 and k_3 is the stiffness of the left (B1) and the right bent (B3), respectively) was 1.75 (see Figure 5a). The center of the stiffness of the bridge was at the right half of the bridge. Consequently, the rotations of the bridge were clock-wise. The first mode was predominant. The corresponding effective mass was about 84% of the total mass of the bridge.

When the seismic intensity was increased, the clock-wise rotations of the deck were firstly increased, and consequently the damage of the bent B1 was proportionally increased. After the yielding of the bent B1, the bent B3 gradually become more exposed and the stiffness ratio of these side bents was changed. The stiffness of the bent B3 was decreased comparing to the bent B1. Consequently the center of stiffness was moved closer to the center of the mass and the rotations of the deck were decreased (see Figure. 5b). The importance of the higher modes was significantly decreased. The response of the bridge was influenced mostly by one predominant mode.

At the test T18 the bent B3 was more damaged than the bent B1 and the center of the stiffness was moved to the left part of the viaduct, changing the direction of the deck rotations (see Figure 5c). The response was influenced by one predominant mode. During this test all bents were heavily damaged. For example, in the bent B3 buckling of some bars was observed (see Johnson et al, 2006). Due to considerably reduced column stiffness, the response of the bridge was quite sensitive to relatively small variations of the columns' properties. The direction of the rotation was considerably changing depending on the ratio of the bents' stiffness. When the bent B3 was damaged more than bent B1 the rotations of the deck changed the direction causing the larger displacements at the side of the bent B3. Due to the large sensitivity of the response to small variations of the bent stiffness, the numerical modeling and the estimation of the response was quite demanding task.

Comparing to the spectra defined in the standards (e.g. EC8/2), the displacement spectra in the period range corresponding to the initial periods of the investigated bridge were smaller for the higher modes, with the shorter periods. Consequently the bent B3 was relatively more exposed than the bent B1 (see Figure 3). For example before the test T14 (during this test yielding of the bent B3 was registered) the periods around 0.4s and 0.3s corresponded to the first and the second mode in the transverse direction of the bridge, respectively. The spectral displacement corresponding to the second mode, which predominantly influenced the response of bent B3, was therefore larger than that, corresponding to the first mode, which predominately influenced the response of the bent B1. In the case of the spectra defined in the codes the ratio of these spectral displacements would be the opposite. Consequently, it can be expected that the difference between PGA corresponding to the yielding of the bents B1 and B3 would be larger than that registered in the experiment and the yielding of the bent B3 would occur at the higher seismic intensities.

5 COMPARISON OF ANALYTICAL AND EXPERIMENTAL RESULTS

5.1 Results of the single-mode pushover method

The results of the N2 method are compared with the experiment in Figure 6. Only the tests (T13, and T18), which illustrate the qualitative changes of the bridge response are presented. More details could be found elsewhere (Isaković & Fischinger, 2007).

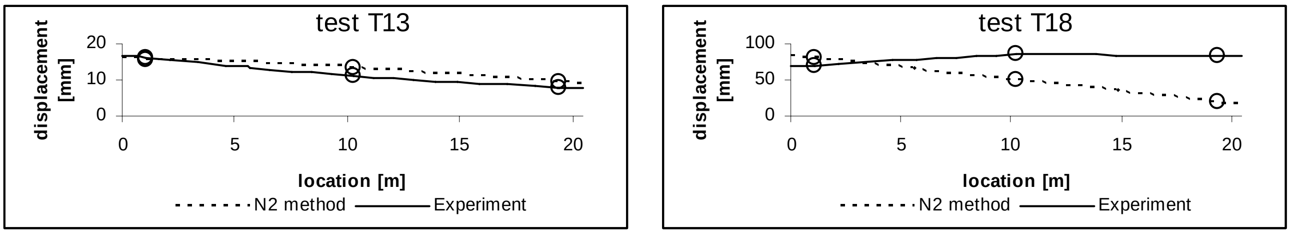


Figure 6. Displacements calculated by the N2 method and displacements measured during the experiment

The accuracy of the displacements shapes depend on the seismic intensity. For the lower seismic intensities (tests T12 - T14) the analytical displacement shapes correlated with the experimental data quite well (see test T13 in Figure 6). When the load level was increased, the match of the displacements shapes was worse, particularly during and after the test T15 when the qualitative changes of the bridge response were observed (see Chapter 4). Due to the changed ratio of the side bents, the rotations of the bridge changed their direction. Several reasons for these qualitative changes were identified. The significant damage of the side bents during the tests T17 and T18 makes the response quite sensitive to small variations of the bent properties. Since the bents were heavily damaged, their resistance was significantly reduced in both (transverse and longitudinal) direction. Consequently they were not capable to stabilize the rotations, unlike in buildings with undamaged elements perpendicular to the loading direction. The spectral characteristics of the applied seismic load caused that the bent B3 was relatively more exposed than the bent B1 (see an explanation in Section 4).

When the bridge was analyzed by the N2 method, the direction of the rotations was always the same and independent of the load intensity. Contrary to the experimentally observed response, the rotations were always clock-wise and more pronounced under the strong excitations. The N2 method was not able to identify the qualitative changes of the rotations of the superstructure. Consequently, the displacements of the right part of the viaduct (between bents B2 – B3) were considerably underestimated. The largest discrepancy was observed during the test T18.

5.2 Results of the multi-mode pushover methods

The multimode methods were applied considering two modes in the transverse direction. At the initial investigated seismic level the effective mass was 84% and 16% for the first and the second mode, respectively. The contributions of these two modes were combined using the SRSS rule.

The displacements of the superstructure obtained with the MPA and the IRSA method are compared with the experimental results in Figure 7. Only the results for tests T13, T15, T17 and T18 are presented, since they illustrate all basic characteristics of the response.

For the low seismic intensity levels correlation with the experiment was good for both multimode methods. The displacement shapes were estimated by the MPA method fairly well for the stronger earthquakes, too. Comparing to the N2 method, the displacements shapes were considerably improved. However, the detailed analysis of the response showed that the MPA has not identified the real cause of the change in the direction of the rotation. During the experiment the response of the bridge was influenced predominantly by the first mode (m_{eff}> 80%), which was considerably changing. The corresponding rotations changed the direction depending on the seismic level and during the test e.g. T15 and T18 the rotations of the deck were opposite to that in the tests with the lower seismic intensity. The non-adaptive MPA method could not take into account these considerable changes of the predominant mode shape. Similar to the N2 method, the rotations of the bridge, subjected to inertial forces proportional to the first mode, were always clock-wise and they increased with the load intensity (see Figure 8). However, the rotations corresponding to the inertial forces proportional to the second mode were in opposite direction (see Figure 8). These rotations compensated the difference between the experiment and the results of the analysis with the forces proportional to the first mode. Therefore, the total displacements match the experimental values reasonably well.

The IRSA method successfully identified changes of the important mode shapes as well as the variable importance of the higher modes (see Figure 7). The shape of the displacement line matched with the experimental data very well until the test T18. The absolute values of the displacements were, however, larger than those observed in the experiment. This difference was increased with the seismic intensity. Abrupt changes of the spectral displacements corresponding to the period range from 0.25 s – 0.5 s (see Figure 3) could be the reason for this discrepancy, since the small variation of the periods of the structure caused large variations of the spectral displacements. A possible reason for this discrepancy could be also the amount of the damping considered in the analytical studies (5%). For example, when the damping was increased (to 10%) for the tests with the larger seismic intensity, the absolute values of displacements were similar to those in the experiment (see Figure 9a).

The shape of the experimental and analytical deflection line determined by IRSA differ somewhat only for the test T18. It was explained in the Section 4 that during this test the significant damage of all bents was observed. Consequently, small variations of the ratio of the bent stiffnesses influenced the response qualitatively, changing the shape of the predominant mode. In general IRSA was not able to take into account these variations of the bent stiffnesses ratio once the yielding of all bents was achieved (at the test T15). If the ratio of the side bents assumed in IRSA was somewhat changed, the compared displacement shapes in the test T18 correlated better, however, in for this stiffness ratio of side bents match of analytical and experimental displacements in the test T16 and T17 was poorer. The second reason for the above mentioned discrepancy could be also the constant ratio of the spectral displacements of both important modes, which is assumed in the IRSA. This ratio in general changes when the structural system is changed due to the formation of the new plastic hinges.

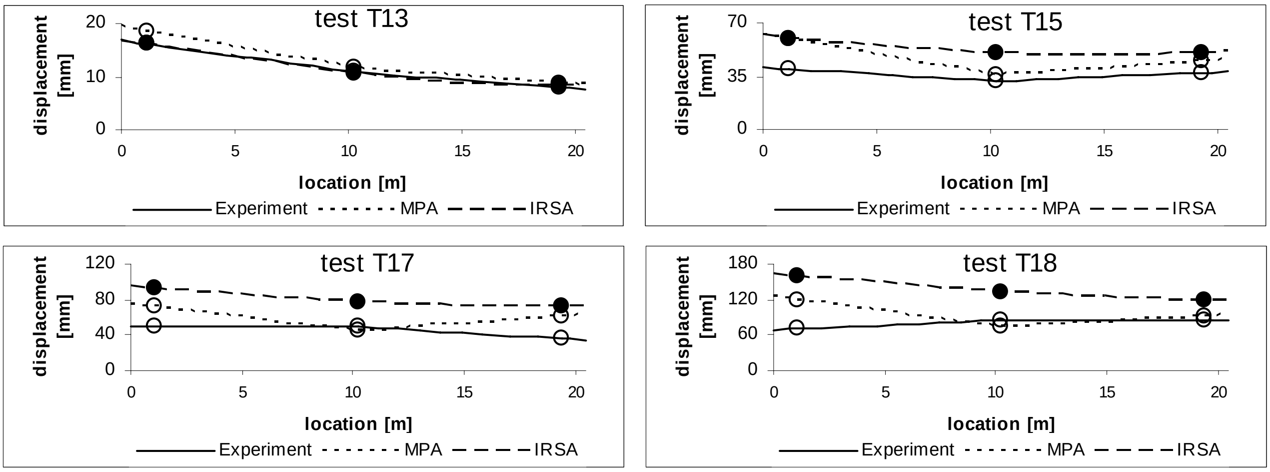


Figure 7. Comparison of maximum displacements obtained by the multimode methods and measured

displacements

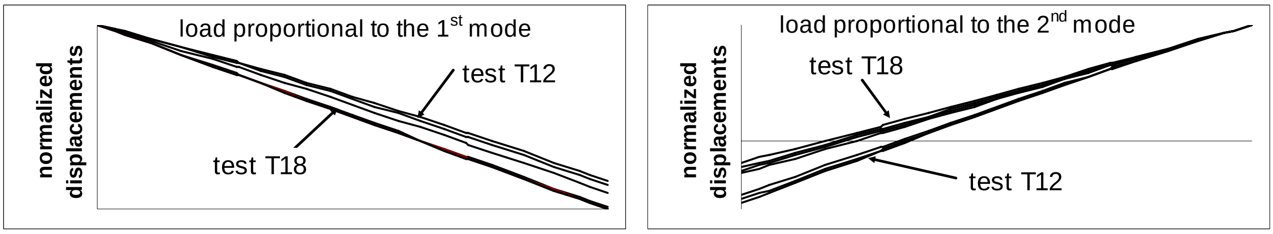
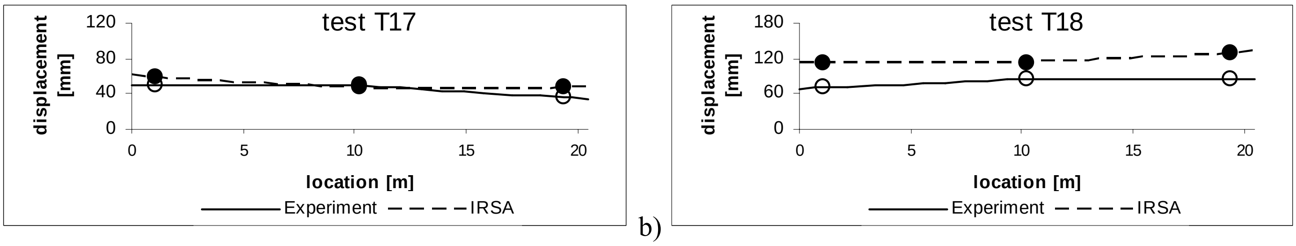


Figure 8. Normalized displacements of the bridge calculated with the MPA



a) b)

Figure 9. Displacements calculated by IRSA after the modifications of the a) damping b) ratio of the stiffness of the side bents in the test T18

6 CONCLUSIONS

A large scale model of two-span two-column bent RC bridge, representing one frame of typical US continuous bridge was experimentally tested on three parallel shake tables at the UNR in the frame of the NEES project. Although the geometry of the bridge was relatively regular, the experimentally observed response of the bridge was quite complex and considerably influenced by the seismic intensity and the ratio of the stiffness of the side bents. At the lower seismic intensities, the left (initially more flexible) side of the bridge was more exposed, resulting in the clockwise rotations and larger displacements at the left side of the bridge. At the higher seismic intensity levels, the right (initially stiffer) bent was more exposed. Due to the damage of this bent the direction of the bridge rotations were gradually changed resulting in the larger displacements at the right (initially stiffer) side. Several reasons for these qualitative changes of the response were detected: the significant damage of the side bents, which makes the response quite sensitive to small variations of the bent properties, the reduced resistance of columns in both (longitudinal and transverse) directions as well as the spectral characteristics of the applied seismic load.

Three typical pushover methods were used to analyze this bridge: a) The N2 method as a standard single-mode pushover method, b) the MPA method as a typical non-adaptive multimode pushover method, and c) the IRSA method as a typical adaptive multimode pushover method. For all considered methods the displacement shapes correlated with the experiment quite well when the lower intensity levels were considered. The N2 method was less effective in the case of higher intensity levels since it was not able to take into account qualitative changes of the deck rotations.

The MPA estimated the displacement shapes quantitatively fairly good for the higher seismic intensities, too. However the detailed analysis of the response detected considerable qualitative differences with the experiment. The non-adaptive MPA method, could not take into account considerable changes of the first mode shape. The second mode coincidentally compensated this discrepancy and displacements of the right side of the bridge match the experimental values well.

The IRSA method successfully identified the changes of the mode shapes as well as the variable importance of the higher modes. The analytical displacement shapes of the deck correlated with the experiment very well. The absolute values of the analytical displacements were larger than the measured displacements. The reasons for this discrepancy could be: a) abrupt changes in the displacement spectra, b) the considered amount of damping. At the last test (T18) considered in the study, the displacement shape correlated with the experimental values worse than in the preceding tests. This discrepancy was mainly caused by the varying ratio of the bent stiffnesses after their yielding. During this test, due to the severe damage of columns, small variations of the side bent stiffness ratio caused qualitative changes of the displacement shape of the superstructure.

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