DISPLACEMENT-BASED SEISMIC DESIGN OF CONCRETE CONTINUE BRIDGES

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ABSTRACT: Several efforts have been made in the last decades to address the importance of changing the focus of current seismic design codes from merely preventing collapse in major earthquakes and controlling the damage in minor earthquakes to a more general design philosophy, which takes into account multiple performance objectives based on quantifiable performance criteria; this design philosophy is referred to as Performance Based Design (PBD).

A displacement-based seismic design procedure is proposed and elaborated for concrete bridges with continuous deck integral with the piers. It includes a simple estimation of inelastic deformation demands (chord or plastic hinge rotations in piers, curvatures for the deck) via elastic 5%-damped modal response spectrum analysis [4]. The applicability of the equal displacement rule at the level of member deformations is checked through nonlinear static analyses of one representative regular four Spans Bridge, steel box girders and piers of various cross-sections and about equal and very different heights.

A four-span bridge of 100 meters in total length was analyzed using both the Nonlinear Static Procedure and Displacement-based design method.

The main aim and product of the research line has been the formation of a model code for displacement-based seismic design.

KEYWORDS: Concrete bridges, Displacement-based design, Nonlinear Static Procedure, Eurocode 8, Seismic design of bridge, higher modes

1 INTRODUCTION

Seismic design is currently going through a transitional period. Most of the seismic codes to date utilize force-based seismic design, or what can also be called strength-based design procedures. However, it is now widely recognized that force and damage are poorly correlated and that strength is of minor importance when designing for earthquake loading than for other actions.

These, together with other problems and inconsistencies with force-based design, [Priestley, 2003], have led to the development of more reliable seismic design methodologies under the framework of what has been termed Performance-Based Seismic Design (PBSD). PBSD represents basically the
philosophy of designing a structure to perform within a predefined level of damage under a predefined level of earthquake intensity.

2 PROPOSED DISPLACEMENT-BASED SEISMIC DESIGN PROCEDURE
The displacement-based design of multi-degree-of-freedom Bridge structures is based on the concepts the displacement-based design of single-degree-of-freedom. However, some specific issues must be considered carefully during the process. The design displacement shape is a function of the relative stiffness between columns, abutments and the deck. Resistance to transverse seismic excitation is mainly provided by bending of the bridge piers, which are designed to respond inelastically and, if the abutments provide some restraint to transverse displacements, superstructure bending will also develop.

In normal seismic design practice the bridge deck is required to remain elastic under the design level earthquake. As a consequence the seismic inertia forces developed in the deck are taken by two different load paths, one portion is transmitted to the piers foundations by column inelastic bending and the remainder transmitted to the abutments by superstructure elastic bending.

The portion of load carried by each of the two different load paths is unknown at the start of the design process and depends strongly on the relative effective-column and deck stiffness's as well as on the degree of lateral restrain provided by the abutments. Since column stiffness's are also unknown at the start of the design process, an iterative procedure is required.

The design procedure presented here considers the discretization of the deck mass as lumped masses at the top of the piers and at the abutments. A portion of the column masses and the cap beam masses can also be lumped at the top, following the recommendations given in [Priestley, et al., 1996].

The Direct Displacement-Based Design procedure for multi-degree-of-freedom bridge structures can be summarized according to bottom flowchart:
3 REGULAR AND IRREGULAR BRIDGE CONFIGURATIONS
As previous studies were done in bridges with regular configurations [Alvarez Botero, 2004], in this paper a Regular Bridge will be defined as a bridge in which the structure center of mass, CM, coincides with the structure center of strength, CV. In this case the translational modes of vibration rule the seismic
response and the rotational ones are not excited and consequently do not participate in the seismic response of the structure.

An Irregular Bridge will be defined as a bridge in which the structure center of mass, CM, does not coincide with the structure center of strength, CV. In this case, the seismic response is a combination of the translational and rotational modes of vibration.

Figure 2. Regular and irregular bridges [3]

3.1 Design displaced shape
Bridge structure composed by several columns connected to a superstructure of defined flexibility will deform in a manner that is influenced by variations in strength, stiffness and mass distribution. The transverse displaced shape will depend strongly on the relative column stiffness, and more considerably, on the degree of lateral restrain provided at the abutments. Figure 3 depicts two different bridge configurations and the possible transverse displaced shapes indicated for the different abutment conditions.
Generally a parabolic displaced shape between abutments and piers can be initially assumed for design purposes.

Deck first-mode deformed shape can be obtained either by solving the Eigen-problem for the deck or by using an approximate first mode shape function as the one shown in Eq.(1) based on a parabolic loading shape.

4 MODELING ISSUES
A simplified plan model of the structure, as depicted in Figure 4, was constructed for each of the bridges. The bridge deck was modeled by means of elastic frame elements. Piers were characterized by inelastic springs, while the abutments by linear elastic springs and dashpots that represented the additional elastic energy dissipation associated to them.

4.1 Bridge design
Described in this section, is one design example to demonstrate the direct displacement-based design approach for continuous bridges. The example utilizes the results given in table 2 to identify the displacement pattern and design the bridge for a rigid translational deflected shape and a result of nonlinear static analyses for the bridge were shown in Figures 3, 4.
Displacement-based seismic design of concrete continue bridges

Step 1: Define initial input parameters
Bridge information and assumption material
Concrete and reinforcing steel properties used for design purposes are presented in table:

<table>
<thead>
<tr>
<th>Material properties for design</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Concrete</strong></td>
</tr>
<tr>
<td>f’c=28 MPa compression strength</td>
</tr>
<tr>
<td>Ec=26457.51 MPa Elastic modulus</td>
</tr>
<tr>
<td>Wc=24 kN/m^3 unit weight</td>
</tr>
<tr>
<td><strong>Reinforcing steel</strong></td>
</tr>
<tr>
<td>fy=420 MPa yield stress</td>
</tr>
<tr>
<td>Es=200000 MPa Elastic modulus</td>
</tr>
<tr>
<td>dbl=0.036 m Longitudinal bar diameter</td>
</tr>
<tr>
<td><strong>Abutment</strong></td>
</tr>
<tr>
<td>KA=KB 3.75E+04 kN/m m_{A1}=3570 kN</td>
</tr>
<tr>
<td>ζ=(%) 0.08 m_{A2}=3570 kN</td>
</tr>
<tr>
<td>Δ(AButment)_{1,2}=0.1m</td>
</tr>
<tr>
<td><strong>Bridge Deck</strong></td>
</tr>
<tr>
<td>I_{yy} = 1.47m^4</td>
</tr>
<tr>
<td>Elastic modulus = 26457.51MPa</td>
</tr>
<tr>
<td>W_{deck} = 175KN/m</td>
</tr>
<tr>
<td>L = 100m ζ=5%</td>
</tr>
<tr>
<td><strong>Column</strong></td>
</tr>
<tr>
<td>H_{1,3} = 11M</td>
</tr>
<tr>
<td>H_{2} = 22m</td>
</tr>
<tr>
<td>W_{1,3} (Column+ Cap beam) = 1343KN</td>
</tr>
<tr>
<td>W_{2} (Column+ Cap beam) = 870KN</td>
</tr>
<tr>
<td>X_{1,4} = 20M</td>
</tr>
<tr>
<td>X_{2,3} = 30m</td>
</tr>
<tr>
<td>D1 = 2M</td>
</tr>
<tr>
<td>D2 = 2.2m</td>
</tr>
<tr>
<td>m_{1,3} = 10470KN</td>
</tr>
<tr>
<td>m_{2} = 15534KN</td>
</tr>
</tbody>
</table>

ζ=0.0021
Figure 5. Elevation and position of regular bridge

Figure 6. Cross section of the bridge

It is worth mentioning that for this work abutment have been assumed to behave elastically, and the procedure presented based on this assumption.

**Step 2: Assume displaced shape and obtain the target displacement pattern based on an approximate first mode shape function**

\[
\delta_i = \left(\frac{16}{5}L^4\right)\left(4x^4-2Lx^3+L^3x\right)
\]

(approximate first mode shape function)

Evaluation of the limiting displacements for each of the piers is also required in order to determine the target displacement pattern. Normally the shortest column governs the selection of displacement pattern. For more details about the definition of each of the previous parameters and how to estimate it, see Priestley et al., 1996[1].
Table 2. Approximate first mode shape function

<table>
<thead>
<tr>
<th>δ(A_1)</th>
<th>0.074084 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>δ(x_1)</td>
<td>0.44 m</td>
</tr>
<tr>
<td>δ(x_2)</td>
<td>0.740841 m</td>
</tr>
<tr>
<td>δ(x_3)</td>
<td>0.44 m</td>
</tr>
<tr>
<td>δ(A_2)</td>
<td>0.074084 m</td>
</tr>
<tr>
<td>δ(x)</td>
<td>≈ 0.04*h</td>
</tr>
</tbody>
</table>

Figure 7. Approximate first mode shape

**Step 3: Assume proportion of load carried by superstructure bending** \( SS = 0.01 \)

**Step 4: Estimate displacement ductility demands and damping for individual piers**

\( \Delta y \), for a cantilever pier is given by Eq. (2), where \( k_y \) is the yield curvature and \( H_e \) is the effective pier height, that considers yield penetration, [Priestley et al., 1996].

\[
\Delta y_1 = k_y \cdot \frac{H_e^2}{3} = 0.20227525m \quad (2)
\]

\[
H_{e1} = H + L_{sp} = 11.33264m \quad (3)
\]

\[
L_{sp1} = 0.022f_y \cdot d_{bl} = 0.33264m \quad (4)
\]

\[
k_{y1} = 2.25 \cdot \Delta y/D = 0.004725 \quad (5)
\]

\[
\Delta_1 = 0.44m
\]

\[
\mu_{\Delta_1} = \Delta/\Delta y_1 = 2.17525378 \quad (6)
\]

\[
\zeta_1 = 0.05 + 0.444*(\mu - 1)/(\pi \mu) = 0.126358 \quad (7)
\]

\[
\Delta y_2 = k_y \cdot \frac{H_e^2}{3} = 0.65460519m \quad (2')
\]

\[
H_{e2} = H + L_{sp} = 22.33264m \quad (3')
\]

\[
L_{sp2} = 0.022f_y \cdot d_{bl} = 0.33264m \quad (4')
\]
### Step 5: Characterize equivalent SDOF

\[ \zeta_{sys} = \frac{\Delta d}{\Delta_{(T,5)}} \times 0.114 \]  
\[ \Delta_{sys} = \Delta d = \Delta (T,5) = \Delta (T,5) = \Delta (T,5) \]  
\[ M_{eff} = \Sigma (m_i \Delta_i) / \Delta d = \Sigma (m_i \Delta_i) / \Delta (T,5) = 13869.97 \text{KN} \]  
\[ \Delta_{T,5} = S_a * g / W^2 = S_d = 0.195686 \text{m} \]  
\[ \Delta_{T,3} = \Delta (T,3) = (10 / (5 + \zeta))^0.5 = 0.15263 \text{m} \]  
\[ T_e = T / \Delta (T,3) = 3.891 \text{sec} \]

\[ \zeta_3 = 0.05 + 0.444 \times (\mu - 1) / (\pi \mu) = 0.126358 \]  
\[ \Delta_{3} = 0.44 \text{m} \]  
\[ \mu_{3} = \Delta / \Delta y \]  
\[ \mu_{3} = 0.022 \text{fy} \times \text{dbl} = 0.33264 \text{m} \]  
\[ H_{3} = H + \text{Lsp} = 1.1317364 \text{m} \]  
\[ H_{e3} = H + \text{Lsp} = 11.33264 \text{m} \]  
\[ L_{sp3} = 0.022 \text{fy} \times \text{dbl} = 0.33264 \text{m} \]  
\[ k_{y3} = 2.25 \times \Delta y / D = 0.004725 \]  
\[ k_{y3} = 2.25 \times \Delta y / D = 0.004725 \]  
\[ k_{y3} = 2.25 \times \Delta y / D = 0.004725 \]  
\[ k_{y3} = 2.25 \times \Delta y / D = 0.004725 \]  
\[ k_{y3} = 2.25 \times \Delta y / D = 0.004725 \]

### Step 6: Obtain \( T_{eff} \) from displacement spectra and \( K_{eff} \) for the equivalent SDOF system

\[ K_{eff} = 4 \pi^2 \times M_{eff} / T_{eff}^2 = 36176 \text{KN/m} \]

### Step 7: Obtain lateral design force for the equivalent SDOF system

\[ V_B = K_{eff} \times \Delta d = 21481.6383 \text{KN} \]

### Step 8: Distribute base shear as seismic forces

<table>
<thead>
<tr>
<th>Force</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>( F_1 )</td>
<td>4656.84679KN</td>
</tr>
<tr>
<td>( F_2 )</td>
<td>11633.2381KN</td>
</tr>
<tr>
<td>( F_3 )</td>
<td>4656.84679KN</td>
</tr>
<tr>
<td>( F_{A1} )</td>
<td>267.353291KN</td>
</tr>
<tr>
<td>( F_{A2} )</td>
<td>267.353291KN</td>
</tr>
</tbody>
</table>
Figure 8. Distribute base shear as seismic forces

**Step 9: Obtain shear forces in the members**

\[ V_i = SDF_i \cdot V_B, \quad SDF = \frac{1}{hi} \sum \left( \frac{1}{hi} \right) F_C \] (16)

\[
\begin{align*}
V_1 &= 8652.8053 \text{KN} \\
V_2 &= 4390.8436 \text{KN} \\
V_3 &= 8652.8053 \text{KN}
\end{align*}
\]

Seismic force carried by the abutment of the following Eq. (17):

\[
SDF_{A1} = \frac{\Delta_{A1}}{\Delta_{A1} + \Delta_{A2}} SS
\]

\[
SDF_{A2} = \frac{\Delta_{A2}}{\Delta_{A1} + \Delta_{A2}} SS
\] (17)

\[
\begin{align*}
SDF_{A1} &= -0.005 \quad V_{A1} = -107.41 \text{KN} \\
SDF_{A1} &= -0.005 \quad V_{A2} = -107.41 \text{ KN}
\end{align*}
\]

**Step 10: Compute member effective stiffness’s**

\[
\begin{align*}
K_{A1} &= 37500.00 \text{KN/m} \\
K_{S1} &= 19665.47 \text{KN/m} \quad K_{S1} = V_{i}/\Delta_i \\
K_{S2} &= 5926.84 \text{KN/m} \quad K_{S2} = V_{i}/\Delta_i \\
K_{S3} &= 196665.47 \text{KN/m} \quad K_{S3} = V_{i}/\Delta_i \\
K_{A2} &= 37500.00 \text{KN/m}
\end{align*}
\] (18)

**Step 11: Solve equivalent elastic system under seismic forces \( F_i \), obtain revised**
displacement pattern: $\Delta n$ and revised proportion of load carried by superstructure

### Table 3. Summary of design for transverse section of bridge

<table>
<thead>
<tr>
<th>$\zeta_{sys}$ (%)</th>
<th>$\Delta d$ (m)</th>
<th>$M_{eff}$ (kN)</th>
<th>$T_{eff}$ (s)</th>
<th>$K_{eff}$ (kN/m)</th>
<th>$V_B$ (kN)</th>
<th>SS (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.114</td>
<td>0.153</td>
<td>13869.97</td>
<td>3.890</td>
<td>36175.94</td>
<td>21481.638</td>
<td>-1.0</td>
</tr>
</tbody>
</table>

### Table 4. Summary of design for longitudinal section of bridge

<table>
<thead>
<tr>
<th>$\zeta_{sys}$ (%)</th>
<th>$\Delta d$ (m)</th>
<th>$M_{eff}$ (kN)</th>
<th>$T_{eff}$ (s)</th>
<th>$K_{eff}$ (kN/m)</th>
<th>$V_B$ (kN)</th>
<th>$V_1$</th>
<th>$V_2$</th>
<th>$V_3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>15.04</td>
<td>0.594</td>
<td>13869.9</td>
<td>3.514</td>
<td>44336.2</td>
<td>26327.3</td>
<td>12389.3</td>
<td>1548.6</td>
<td>12389.3</td>
</tr>
</tbody>
</table>
4.2 Finite element model
The structural analysis program, SAP2000 Version 14.0.0 Advanced [6], was used to perform analyses. Geometric nonlinearity through considering P-Delta effect was applied to this bridge in addition to material nonlinearity.

4.3 Seismic loading
To perform analysis of structure, the next step after modeling is applying loads. Design response spectrum should be available in order to perform NSP. This bridge is to be built in the Tehran city in a seismic zone with an acceleration coefficient of (PGA = 0.35g). This estimation is based on the AASHTO (2007) specification for an earthquake of 10% probability of occurrence in 50 years, which is equivalent to a recurrence period of 475 years [7].

5 RESULTS AND PARAMETRIC STUDY
Analyses were performed for live Safety level of seismic load intensity. Comparison is performed for the maximum displacement, total base shear resulting from the NSP and the corresponding results from the DDBD. The lateral load behavior of the bridge in longitudinal and Transverse Direction is depicted in Fig. 11.

5.1 Longitudinal direction
Period of the first mode in this direction is 1 seconds and the modal participation mass ratio is shown in Table 5. Pushover curve for this direction is shown in Fig. 11. The formula shown below (FEMA-356) is used to estimate the target displacement:

$$\delta_t = C_0C_1C_2C_3S_a \frac{T^2}{4\pi^2}$$

where C_0, C_1, C_2, and C_3 are modification factors to consider different parameters affect the control node displacement, and the rest of the formulae is the equal displacement rule. For more details about the definition of each of the...
previous parameters and how to estimate it, see FEMA-356. The estimated target displacement is 10.42cm for the Design Level.

5.2 Transverse direction
Period of the first mode in this direction is 1 seconds and the modal participation mass ratio for this mode is shown in Table 5. Pushover curve for this direction is shown in Fig 11. The same formula shown in Eq. 19 is implemented to estimate the target displacement. The estimated target displacement is 9.33cm for the Design Level.

5.3 Evaluation of performance level
Using acceptance criteria provided by FEMA-356 to evaluate performance level of this bridge, rotation of plastic hinges should not exceed the following values for the corresponding performance levels: 0.005 for immediate occupancy, 0.01 for life safety, and 0.017 for collapse prevention. In the longitudinal direction, the bridge satisfies life safety performance level for Design Level. In the transverse direction; this bridge satisfies the life safety performance level for Design Level.

<table>
<thead>
<tr>
<th>OutputCase</th>
<th>StepType</th>
<th>StepNum</th>
<th>Period</th>
<th>SumUX</th>
<th>SumUY</th>
<th>SumRZ</th>
</tr>
</thead>
<tbody>
<tr>
<td>Text</td>
<td>Text</td>
<td>Unitless</td>
<td>Sec</td>
<td>Unitless</td>
<td>Unitless</td>
<td>Unitless</td>
</tr>
<tr>
<td>MODAL</td>
<td>Mode 1</td>
<td>1</td>
<td>1.000749</td>
<td>4.70238E-20</td>
<td>0.927621</td>
<td>0.707766</td>
</tr>
<tr>
<td>MODAL</td>
<td>Mode 2</td>
<td>2</td>
<td>0.599413</td>
<td>1.52612E-19</td>
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<td>0.931045</td>
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<tr>
<td>MODAL</td>
<td>Mode 3</td>
<td>3</td>
<td>0.531914</td>
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<td>0.931045</td>
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<tr>
<td>MODAL</td>
<td>Mode 4</td>
<td>4</td>
<td>0.431656</td>
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<td>0.927621</td>
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</tr>
<tr>
<td>MODAL</td>
<td>Mode 5</td>
<td>5</td>
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<td>0.927621</td>
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<tr>
<td>MODAL</td>
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<td>6</td>
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</tr>
<tr>
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<td>Mode 7</td>
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<td>0.347756</td>
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<td>0.965456</td>
</tr>
<tr>
<td>MODAL</td>
<td>Mode 8</td>
<td>8</td>
<td>0.290404</td>
<td>0.371218493</td>
<td>0.968418</td>
<td>0.965456</td>
</tr>
<tr>
<td>MODAL</td>
<td>Mode 9</td>
<td>9</td>
<td>0.242127</td>
<td>0.371218493</td>
<td>0.968418</td>
<td>0.969104</td>
</tr>
<tr>
<td>MODAL</td>
<td>Mode 10</td>
<td>10</td>
<td>0.215351</td>
<td>0.371218493</td>
<td>0.97320</td>
<td>0.969104</td>
</tr>
<tr>
<td>MODAL</td>
<td>Mode 11</td>
<td>11</td>
<td>0.197230</td>
<td>0.371218493</td>
<td>0.97320</td>
<td>0.97060</td>
</tr>
<tr>
<td>MODAL</td>
<td>Mode 12</td>
<td>12</td>
<td>0.190209</td>
<td>0.371218493</td>
<td>0.978894</td>
<td>0.974953</td>
</tr>
<tr>
<td>MODAL</td>
<td>Mode 13</td>
<td>13</td>
<td>0.165004</td>
<td>0.415947943</td>
<td>0.978894</td>
<td>0.974953</td>
</tr>
<tr>
<td>MODAL</td>
<td>Mode 14</td>
<td>14</td>
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<td>0.978894</td>
<td>0.976625</td>
</tr>
<tr>
<td>MODAL</td>
<td>Mode 15</td>
<td>15</td>
<td>0.159176</td>
<td>0.415947943</td>
<td>0.979065</td>
<td>0.976756</td>
</tr>
<tr>
<td>MODAL</td>
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<td>16</td>
<td>0.156756</td>
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<td>Mode 17</td>
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<td>0.149517</td>
<td>0.904035723</td>
<td>0.979065</td>
<td>0.976781</td>
</tr>
</tbody>
</table>
Displacement-based seismic design of concrete continue bridges

Figure 12. Pushover curve for longitudinal direction

Figure 13. Pushover curve for transverse direction

Figure 14. Displaced shape of the bridge by using different methods of analysis
CONCLUSIONS

In this work, a displacement-based design procedure for the transversal seismic design of continuous multi-span reinforced concrete bridge, first proposed by Priestley, has been presented. The method is an iterative procedure which, based on initial assumptions of the bridge displacement pattern and the proportion of the total lateral force carried by the superstructure through the abutments, aims to design a bridge that will reach the design limit state of deflection. As result, comparison of results obtained from displacement-based design with the Nonlinear Static Procedure.

The method utilizes the Substitute Structure approach, [Gulkan and Sozen, 1974], to model the inelastic structure as an equivalent single-degree-of-freedom system. The equivalent SDOF is characterized by the secant stiffness, $K_{\text{eff}}$, at maximum displacement and an equivalent viscous damping, $\xi_{\text{sys}}$, appropriate for the level of hysteretic energy absorption associated with the inelastic response.

The iterative design procedure was found to be efficient and easy to implement. Very little iteration is required, even if initial assumptions are poor. However, some suggestions are made to provide the method with good initial estimates of the inelastic displaced shape and the proportion of load taken by the abutments. The procedure is then very easy to implement in any programming software.

Finally, it is believed than more investigation is required on the topic, especially for the cases in which the fundamental elastic and inelastic mode shapes differ. Even though all the bridges in this work considered flexible
lateral support at the abutments, the procedure can be successfully applied for fixed abutments condition. Application of the method to free abutments condition was not considered and can be also investigated. Moreover consideration of nonlinear inelastic behavior of the abutments could also be implemented in the design procedure.

Additional analysis will be carried out in the future for more bridge configurations, abutment conditions and variable span lengths. The goal will be to identify the inelastic displacement patterns and to estimate the amount of shear force carried by the abutments in the case of a bridge with restrained abutments.

REFERENCES

Received: July 6, 2015     Accepted: Sept. 24, 2015
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