PUSHOVER ANALYSIS OF RC COLUMNS SUBJECTED TO MULTIPLE DEGREES OF TRANSVERSE REINFORCEMENT CORROSION

Mohamed Saad Abbadi, Nouzha Lamdouar
Civil Engineering Laboratory, Mohammadia School of Engineers, Mohammed V University in Rabat, Morocco

ABSTRACT

The seismic performance of old existing bridges remain a major issue of the road network managers, especially in moderate seismic regions were 70’s bridges were designed prior to modern seismic codes and without earthquake actions considerations, or subjected to great damage and confinement loss due to reinforcement corrosion.

A fiber-type section model was employed in order to perform a pushover analysis of columns under multiple degree of confinement losses. The Mander model allow the estimation of the compressive strength and ultimate strain values as a function of the confinement (transverse reinforcing) steel. The model is able to quantify the effects of losing confinement on seismic performance, especially axial-flexural cross section ductility.

A case study conducted on an old exiting multicolumn bridge with apparent confinement loss has showed that transverse reinforcement provides plastic rotational capacity to maintain flexural strength as high curvatures. Different levels of transverse reinforcement losses, from 25% to 100%, in addition to uncorroded case, were considered on a pushover analysis in order to draw performance curves and evaluate ductility reserve.

Results have shown that confinement loss induced by corrosion of transverse reinforcement reduce considerably the ductility of the columns, and special attention must then be given in order to verify the residual plastic hinge capacity.

Keywords: Pushover, Opensees, Capacity Curve, Mander Model, Concrete Confinement

1. INTRODUCTION

In the seismic bridge design, a main objective is to ensure that a structure is capable of deforming in a ductile manner when subjected to a larger earthquake loading. Previous studies have shown that the confinement of concrete by suitable arrangements of transverse reinforcement results in a significant increase in both the strength and the ductility of compressed concrete. For the columns, it provides plastic rotational capacity to maintain flexural strength as high curvatures.

The loss of confinement in the columns of old existing bridges remains one of the major concerns, this loss is due to the lack of cover concrete and the reduction of the area of transverse reinforcement as a consequence of corrosion. In order to assess the existing structure, a pushover analysis is conducted on an old bridge that was built in the early 1970s, presenting visual degradation of cover concrete and apparent transverse reinforcement.

OpenSees, an open source finite element program, has been used for modelling and analysis, while Midas Civil "trial version" was employed for the additional controls of some results, especially dynamic properties of the bridge.

A pushover analysis is conducted in order to assess the residual ductility of the structure, the expected material strength and stress-strain (σ-ε) relation is used for unconfined and confined concrete, as well as reinforcing steel, to more accurately capture the bridge’s capacity and behavior. The reinforcement details of the piers and other major bridge components are required and are based according to the construction design code.

The Mander et al. (1988) model is to be used to represent the uniaxial stress-strain behavior for unconfined and confined concrete. Several configurations of transverse reinforcement loss where modeled in order to evaluate the structural behavior of the bridge.

2. MANDER CONFINED CONCRETE STRESS-STRAIN CURVE

The Mander concrete stress-strain curve calculates the compressive strength and ultimate strain values as a function of the confinement (transverse reinforcing) steel. The model is able to quantify the effects of improving or losing confinement, the following parameters define the Mander confined concrete stress-strain curve:

\[ \varepsilon = \text{Concrete strain} ; \]
\[ f = \text{Concrete stress} ; \]
\[ E = \text{Modulus of elasticity (tangent modulus)} ; \]
\[ E_{sec} = \text{Secant modulus of elasticity} ; \]
\[ f'_{c} = \text{Compressive strength of unconfined concrete} ; \]
\[ f'_{cc} = \text{Compressive strength of confined concrete; this item is dependent on the confinement steel provided in the section and is explained later} ; \]
\[ \varepsilon'_{c} = \text{Concrete strain at } f'_{c} ; \]
\[ \varepsilon_u = \text{Ultimate concrete strain capacity for unconfined concrete and concrete spalling strain for confined concrete} ; \]
\[ \varepsilon'_{cc} = \text{Concrete strain at } f'_{cc} ; \]
\[ \varepsilon_{cut} = \text{Ultimate concrete strain capacity for confined concrete, this item is dependent on the confined steel provided in the section and is explained later} . \]
Figure 1 Stress-strain model proposed for monotonic loading of confined and unconfined concrete (Mander et al., 1988a)

2.1. The Mander Confined Concrete Compressive Strength $f'_{cc}$

The Mander confined concrete stress-strain curve is defined by the following equations:

$$f = \frac{f'_{cc}x}{r - 1 + x^r}$$

Where: $\varepsilon'_{cc} = \left\{5\left(\frac{f'_{cc}}{f'_{c}} - 1\right) + 1\right\} \varepsilon'_c$ and $x = \frac{\varepsilon}{\varepsilon'_{cc}}$

$E_{sec} = \frac{f'_{cc}}{\varepsilon'_{cc}}$ and $r = \frac{E}{E - E_{sec}}$

For circular cores, the Mander Confined Concrete Compressive Strength $f'_{cc}$ is described by the equation:

$$f'_{cc} = f'_{c} \left(2.254 \sqrt{1 + \frac{7.94f'_{l}}{f'_{c}} - 2f'_{l} - 1.254}\right)$$

$f'_{l}$ is the effective lateral pressure on confined concrete provided by the confinement steel, expressed as follow:

$$f'_{l} = k_{e}f_{l}$$

Where: $f_{l}$ is the lateral pressure on confined concrete provided by the confinement steel, described as $\frac{\rho_{s}f_{yh}}{2}$, where $\rho_{s}$ is the volumetric ratio of transverse confinement steel to the concrete core and $f_{yh}$ yield stress of confinement steel.

$k_{e}$ is a coefficient measuring the effectiveness of the confinement steel, equal to $\frac{A_{e}}{A_{cc}}$ where $A_{e}$ is the concrete area that is effectively confined and $A_{cc}$ is the concrete core area excluding longitudinal bars.

2.2. The Mander Ultimate Concrete Compression Strain

To predict the ultimate concrete compressive strain, Mander et al. (1984) proposed a rational method for predicting the longitudinal concrete compressive strain at first hoop fracture based on an energy balance approach. In this approach, the additional ductility available when
concrete members are confined is considered to be due to the energy stored in the transverse reinforcement.

The longitudinal concrete compressive strain corresponding to hoop fracture can be calculated by equating the ultimate strain energy capacity of the confining reinforcement per unit volume of concrete core \((U_{sh})\) to the difference in area between the confined \((U_{cc})\) and the unconfined \((U_{co})\) concrete stress-strain curves, plus additional energy required to maintain yield in the longitudinal steel in compression \((U_{sc})\),

\[
U_{sh} = U_{cc} + U_{sc} - U_{c0}
\]

Developing the equation gives:

\[
\rho_s A_{cc} * \int_0^{\varepsilon_{sf}} f_s \, d\varepsilon_s = A_{cc} * \int_0^{\varepsilon_{cu}} f_c \, d\varepsilon_c + \rho_{cc} A_{cc} * \int_0^{\varepsilon_{cu}} f_{sl} \, d\varepsilon_c - A_{cc} * \int_0^{\varepsilon_{sp}} f_c \, d\varepsilon_c
\]

Where:

\(\rho_s\) = ratio of volume of transverse reinforcement to volume of concrete core;
\(A_{cc}\) = area of concrete core;
\(f_s\) and \(\varepsilon_s\) = stress and strain in transverse reinforcement;
\(\varepsilon_{sf}\) = fracture strain of transverse reinforcement;
\(f_c\) and \(\varepsilon_c\) = longitudinal compressive stress and strain in concrete;
\(\varepsilon_{cu}\) = ultimate longitudinal concrete compressive strain;
\(\rho_{cc}\) = ratio of volume of longitudinal reinforcement to volume of concrete core,
\(f_{sl}\) = stress in longitudinal reinforcement;
\(\varepsilon_{sp}\) = spalling strain of unconfined concrete.

Following works, tests and results of previous authors, the equation simplifies to:

\[
110 * \rho_s = \int_0^{\varepsilon_{cu}} f_c \, d\varepsilon_c + \int_0^{\varepsilon_{cu}} f_{sl} \, d\varepsilon_c - 0.017 \sqrt{f'_{c0}}
\]

Where \(f'_{c0}\) is the quasi-static compressive strength of concrete in MPa.

3. GEOMETRY AND MATERIAL PROPERTIES

In order to construct a reliable numerical model of the existing structure, an appropriate collection of data must be obtained, including geometrical properties, structural details, material properties, and any additional information that count on structural response.

The values used in this study are summarized in the table bellow:

Table 1 Material properties of concrete and reinforcement for different levels of corrosion

<table>
<thead>
<tr>
<th>Concrete</th>
<th>0</th>
<th>25</th>
<th>50</th>
<th>75</th>
<th>100</th>
</tr>
</thead>
<tbody>
<tr>
<td>(E) (GPa)</td>
<td>33</td>
<td>33</td>
<td>33</td>
<td>33</td>
<td>33</td>
</tr>
<tr>
<td>(f'_{cc}) (MPa)</td>
<td>39.18</td>
<td>38.6</td>
<td>34.04</td>
<td>31.05</td>
<td>30</td>
</tr>
<tr>
<td>(\varepsilon_{cc}) (10(^{-3}))</td>
<td>5.06</td>
<td>4.86</td>
<td>3.35</td>
<td>2.35</td>
<td>2</td>
</tr>
<tr>
<td>(\varepsilon_{cu}) (10(^{-3}))</td>
<td>6.25</td>
<td>5.60</td>
<td>3.25</td>
<td>1.55</td>
<td>0.9</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>reinforcement</th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield stress (MPa)</td>
<td>285.75</td>
<td>279.25</td>
<td>255.75</td>
<td>238.75</td>
<td>235</td>
</tr>
<tr>
<td>(\varepsilon_{cu}) (10(^{-3}))</td>
<td>6.25</td>
<td>5.60</td>
<td>3.25</td>
<td>1.55</td>
<td>0.9</td>
</tr>
</tbody>
</table>
4. METHODOLOGY, PRINCIPLES AND HYPOTHESIS OF NONLINEAR STATIC ANALYSIS OF CORROSION RC BRIDGE PIERS

Pushover analysis is a static, nonlinear procedure in which the magnitude of the structural loading is incrementally increased in accordance with a predefined reference load pattern. With the increase in the magnitude of the loading, the benefit of static nonlinear “pushover” method consist on the superposition of a curve representing the resisting capacity of the structure with a curve describing the solicitation provided by a seismic action. The intersection of the two curves represent the performance point, allowing the estimation of maximal displacement of the structure, and thus the level of incursion in the plastic field.

The capacity curve provide different limit states:
• Point A: limited damage corresponding to the first plastification of the first hinge (first curve inflection).
• Point B: significant damage corresponding to a global displacement equal to ¾ of the ultimate displacement when structure collapses.
• Point C: collapse limit state represented by the end of the curve.

In this respect, a pushover analysis was conducted in transversal direction in order to draw performance curve of the columns. Control node for steering progressive displacement is taken at the column top and triangular loading pattern was adopted.

5. DESCRIPTION OF THE NUMERICAL MODEL

The superstructure elements will be modeled as linear-elastic beam-column elements. The modulus of elasticity in the superstructure was determined using the unconfined concrete compressive strength of 30 MPa. No nonlinearities are considered for the superstructure elements for an overall analysis of the bridge, since other elements such as the columns and abutments are designed to undergo inelastic excursions, while the superstructure is protected by a capacity design and is expected to remain in the elastic range of response.

The substructure of the four-span bridge was composed of three, five-circular column piers with the diameters of 0.5 m which had been reinforced with a 1.02 percent longitudinal steel ratio and a transverse reinforcement ratio of 0.62 percent. A beam-column element connects each of the nodes at the geometric centroid of the column top cross section.

Since, the columns were expected to experience nonlinear deformations at their extremes; nonlinear elements "force-based beam-column" in OpenSees software were used to model the five-column piers. Fiber section for the longitudinal column reinforcement, unconfined concrete, and confined concrete were characterized through multi-linear stress-strain curves. The constitutive relationships of both the confined and unconfined concrete were defined by the "Concrete04" option in OpenSees which is a uniaxial Popovics concrete material object. The properties of the confined concrete were then specified in reference to Mander’s model. Table.1 have illustrated this concrete model and its relevant data in the present study, for different transverse reinforcement losses. the bi-linear steel material (Steel01 in OpenSees) with the initial modulus of elasticity (E) of 200 GPa, the yielding stress (Fy) of 235 MPa, and the strain hardening ratio of 5% were used to model plastic field the reinforcements.

The dynamic effects of column axial loads acting through large lateral displacements, otherwise known as P-Δ or second-order effects, is included in this analysis.

During a pushover analysis, the horizontal displacements achieved by the structure can significantly increase P-Δ effects.

The consideration of P-Δ effects helps identify the structural instability hazard of the bridge by capturing the degradation of strength and amplification of the seismic demand on the column bents, caused by the relative displacement between the column top and bottom.

The effect of corrosion on the confined concrete is considered by reducing the volumetric ratio and yield strength of the confinement reinforcement as a function of steel mass loss due to corrosion. The influence of corrosion on reduced ductility is also considered by limiting the maximum strain in confined concrete as a function of reduced ductility of hoop reinforcement.

The Concrete04 (material model) available in the OpenSees is used to model both confined and unconfined concrete. The Mander’s equations (Mander et al., 1988) are used to define the confinement parameters.
6. SEISMIC DEMAND

Seismic movements are characterized by their long return periods, and a great uncertainty in their probability of occurrence.

Given the scarcity and the violence of these phenomena, design practices reserves special treatment to seismic actions, since partial safety factors are almost non-existent and strength calculation are conducted using behavior factors and nonlinear analysis, well beyond usual linear field. Global retrofitting of all bridges for a uniform seismic action is considered economically inadequate.

Definition of differential nominal accelerations for bridge retrofitting needs to take into account site effect, seismic zoning, bridge importance in the road network, probabilistic analysis that enables linking return period and corresponded seismic acceleration.

Seismic reference action can be defined choosing a probability of being exceeded during a theoretical lifetime $T_L$ of the structure. Return period $T_R$ of the seismic event is then calculated by the following equation:

$$T_R = \frac{1}{(1 - (1 - p)^{T_L})}$$

The dependency of acceleration to return period is generally associated to a power-law described as follow:

$$a_{gc} = \left(\frac{T_R}{T_{NCR}}\right)^k a_{gr}$$

where:

- $a_{gc}$ is acceleration value corresponding to a reference return period $T_{Ref}$.
- $a_{gr}$ is acceleration value corresponding to a reference return period $T_{NCR}$.

The exponent « $k$ » depends on the seismic activity of the concerned zone, in this case study the value is 0.444, the nominal accelerations for different return periods are determined accordingly.

The following table summarize the nominal accelerations for the case study site:
Table 2 Nominal accelerations for different return periods and probabilities of exceeding, for a lifetime of 50 and 100 years.

<table>
<thead>
<tr>
<th>$T_L$ (years)</th>
<th>$p$</th>
<th>1%</th>
<th>2%</th>
<th>5%</th>
<th>10%</th>
<th>20%</th>
<th>40%</th>
</tr>
</thead>
<tbody>
<tr>
<td>50 years</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$T_R$ (years)</td>
<td></td>
<td>4975</td>
<td>2475</td>
<td>975</td>
<td>475</td>
<td>225</td>
<td>100</td>
</tr>
<tr>
<td>$A_s$(cm/s²)</td>
<td></td>
<td>128.88</td>
<td>94.53</td>
<td>62.51</td>
<td>45.42</td>
<td>32.57</td>
<td>22.9</td>
</tr>
<tr>
<td>100 years</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$T_R$ (years)</td>
<td></td>
<td>9950</td>
<td>4950</td>
<td>1950</td>
<td>950</td>
<td>449</td>
<td>100</td>
</tr>
<tr>
<td>$A_s$(cm/s²)</td>
<td></td>
<td>175.32</td>
<td>128.59</td>
<td>85.03</td>
<td>61.78</td>
<td>44.28</td>
<td>22.57</td>
</tr>
</tbody>
</table>

7. RESULTS AND DISCUSSIONS

7.1. Eigenvalue Analysis Results

A linear eigenvalue analysis was performed, prior to pushover nonlinear analysis, in order to determine dynamic characteristics of the structure. The periods at which vibrations naturally occur and the mode shapes assumed by the bridge are determined analytically, based on the mass, stiffness, and damping properties of the system. The principal modes of deformation of the structure include the transverse and longitudinal translation of the bridge.

Given that the lack of graphic view in Opensees may conduce to some defects in modeling assumptions, such material properties, stiffness and mass distribution, Additional analysis was performed in Midas Civil trial version, proving the correct modeling in Opensees. Results are summarized below.

![Figure 5: (left) Longitudinal and (right) Transverse fundamental mode shapes](image)

Table 3 Modal periods (seconds) of the bridge

<table>
<thead>
<tr>
<th>Mode</th>
<th>Opensees</th>
<th>Midas</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.788</td>
<td>0.838</td>
</tr>
<tr>
<td>2</td>
<td>0.785</td>
<td>0.835</td>
</tr>
</tbody>
</table>

7.2. Pushover Results

Pushover analysis was conducted in transversal direction, the capacity curves obtained and those representing the seismic demand are superposed in one graph. The benefit of such representation is to visualize the level of displacement and damage for all columns, and for different levels of seismic demand. It demonstrated nearly the same initial elastic behavior regarding different levels of corrosion. Differences began to arise in the plastic field, with an elastic-plastic yield strength varying from 160 to 100, and an ultimate strain varying from 0.24 m to 0.05 m. Slight loss of the ductility is noticed when the transverse reinforcement is...
barely corroded, but the impact of corrosion is palpable for advanced levels of degradation. Performance points, represented by the intersection of the capacity curve and the seismic demand curves, show that corroded columns cannot admit large displacements and therefore collapses under moderate and large seismic movements. For uncorroded or columns with limited corrosion, ductile displacements are provided before total collapse under large seismic movements.

![Pushover curves of the columns under different confinement losses](image)

**Figure 5** Pushover curves of the columns under different confinement losses

8. CONCLUSIONS AND DISCUSSIONS

By performing a pushover analysis of ordinary bridge with multiple levels of corroded transverse reinforcement, leading to a confinement loss of the column concrete. The following conclusions can be drawn out:

- Loss of confinement have a significant effect on the global response of the structure, plastic rotation capacity and plastic hinging mechanisms of the corroded RC elements.
- A considerable decrease in the elastic yield stress as well as in the plastic zone and ultimate strain limit is noted.
- For large seismic movements, based on new seismic zoning from probabilistic seismic hazard analysis, corroded and uncorroded columns collapses. Uncorroded columns behave better under moderate seismic actions than corroded ones, by permitting ductile rotations and displacements. Small seismic actions are supported by both corroded and uncorroded columns, and for a large part remains in the elastic field.
- Consideration needs to be given to the buckling of bars even if the structure is originally designed to have sufficient level of confinement and antibuckling reinforcement.
- The influence of spatial variability of corrosion on nonlinear response of the corroded elements needs to be investigated.
REFERENCES


