VERIFYING THE SEISMIC PERFORMANCE OF CONCRETE BRIDGE COLUMNS ACCORDING TO CSA S6-14

S. Ashtari¹, C. Ventura², W.D. L. Finn³

ABSTRACT

The latest edition of the Canadian Highway Bridge Design Code (CSA S6-14) has introduced a performance-based design approach for designing reinforced concrete bridges, and requires meeting certain strain limits in steel and concrete for various specified performance levels. Appropriate computer models are necessary to verify the specified strain limits for concrete bridges. The global and local responses of reinforced concrete members - including displacements, curvatures, and strains - modelled with distributed plasticity elements, show mesh-dependency due to localization of strains in a single element or at a critical fiber section. One of the techniques used to deal with the mesh-dependency issue is material model regularization, which ensures achieving mesh-independent global response of concrete members. There are also recommendations on obtaining mesh-independent local curvatures for these models. However, there are no clear recommendations so far on how the mesh dependency in the local strain response of the regularized models can be rectified. This is especially important for checking the strain limits of CSA S6-14 for reinforced concrete bridges. This paper aims to address this need by providing a simple method to verify the strain response of the regularized models of concrete members. First, the material regularization is applied to the distributed plasticity models of a tested bridge column. The global responses of the regularized models are compared against the test results. Next, a method is developed and applied to verify the strain response of the regularized models against the strain limits of the code.

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ABSTRACT

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Introduction

The performance-based seismic design of RC bridges according to the Canadian Highway Bridge Design Code (CSA S6-14) [1] requires meeting several local strain limits and global displacement criteria at different performance levels. The evaluation of these limits is only

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appropriate, if objective local and global responses are obtained for demand and capacity. An objective response in this context is a response that is independent of the choice of mesh in the finite element model. An advantageous choice of modelling to obtain strain values is to employ distributed plasticity beam-column elements with fiber sections. The use of these models for design and assessment purposes has become very common in practice in recent years. The two common formulations of the distributed plasticity beam-column elements are force-based elements (FBE) and displacement-based elements (DBE). However, it has been shown [2, 3] that the distributed plasticity models of RC components, which are expected to respond in the post-peak strain-softening region of concrete material, demonstrate mesh-dependency due to the localization of plastic strains in a critical element or at a critical fiber section. The local and global responses of these models are not objective and cannot be used in performance evaluations, unless the mesh-dependency issue is addressed properly.

There are very few structural design and analysis guidelines, which recognize this problem and provide a solution. PEER/ATC-72-1 [4] recommends using an element size equal to the empirical plastic hinge length. A common practice is to set the length associated with the critical fiber section equal to the empirical plastic hinge length [5]. However, a more refined solution to the mesh-dependency issue is material model regularization, proposed initially by Coleman and Spacone [2] for FBE. The technique has its roots in fracture mechanics and the use of the crack-band model proposed by Bazant and colleagues [6-8]. Material regularization ensures objective global response that is independent of the mesh-size, but the local response of the regularized model needs further post-processing. Coleman and Spacone suggested a simple post-processing procedure for achieving mesh-independent local curvatures for FBE elements. It is possible to obtain strains for the regularized models from the post-processed curvatures. However, a much simpler and more direct way of checking strain limits for the regularized models is possible based on mapping the specified strain limits from the reference material model to the regularized material model. This paper demonstrates how this can be achieved and presents the necessary equations. First, it is shown how the localization and mesh-dependency issue affect the predictions of the global and local responses of a tested bridge column model. Next, the material models are regularized for models with varying mesh sizes and the objective responses of the models are compared against the test results. Finally, the strain limits are mapped to check the performance of the bridge column using the regularized models.

Material Model Regularization

The material regularization is a technique applied to distributed plasticity elements to address the mesh dependency issue. The technique is based on preserving a constant fracture energy of concrete in compression, referred to as the crushing energy. Using this concept, Coleman and Spacone suggested a simple regularization technique for a fiber section FBE. Crushing energy is defined as the area under the post-peak stress-displacement curve of concrete. This energy can be related to the material stress-strain curve through a characteristic length, $h$. For FBE, this is the length associated with the critical integration point and is denoted by $L_{IP}$ in Eq. 1, as follows:

$$G_{fc} = h \int \sigma \, d\varepsilon_i = L_{IP} \int \sigma \, d\varepsilon_i$$

(1)

in which, $G_{fc}$ is the crushing energy of concrete, and $\sigma$ and $\varepsilon$ are concrete stress and strain, respectively. If $G_{fc}$ is preserved constant, then the production of the characteristic length and the
area under the stress-strain curve of concrete in the post-peak region should remain constant. Consequently, changing the characteristic length by varying either the mesh size or the number of integration points requires adjusting the area under the stress-strain curve correspondingly. This can be achieved by adjusting the strain value, at which concrete loses 80% strength, denoted by \( \varepsilon_{20u} \) for unconfined concrete and \( \varepsilon_{20c} \) for confined concrete. Coleman and Spacone presented an expression for calculating \( \varepsilon_{20u} \) and \( \varepsilon_{20c} \) assuming the modified Kent and Park model [9] for concrete, as follows:

\[
\varepsilon_{20u} = \frac{G_{fc}}{0.6f'_cL_{IP}} \cdot \frac{0.8f'_c}{E_c} + \varepsilon_o
\]

(2)

\[
\varepsilon_{20c} = \frac{G_{fcc}}{0.6f'_{cc}L_{IP}} \cdot \frac{0.8f'_{cc}}{E_{cc}} + \varepsilon_{oc}
\]

(3)

In the above relations, \( G_{fc} \) and \( G_{fcc} \) are the crushing energy of unconfined and confined concrete, \( f'_c \) and \( f'_{cc} \) are the specified strength of unconfined and confined concrete, \( E_c \) and \( E_{cc} \) are unconfined and confined concrete elastic moduli, and \( \varepsilon_o \) and \( \varepsilon_{oc} \) are the strains corresponding to the peak strength of unconfined and confined concrete, respectively.

One of the most recent extensive studies on material model regularization was conducted by Pugh [3]. He studied the localization issues in distributed plasticity models for numerical simulation of reinforced concrete shearwalls. His main contribution was to recommend a more scientific formulation for the crushing energy of both unconfined and confined concrete. Previously, Coleman and Spacone recommended using values of 20-30 N/mm for the crushing energy of unconfined concrete based on the recommendation of Jansen and Shah [10] and 180 N/mm for the crushing energy of confined concrete (about six times the value for the unconfined concrete crushing energy). Pugh observed that the crushing energy can be formulated in terms of the specified strengths of unconfined and confined concrete. He recommended using \( G_{fc}=2f'_c \) for FBE and \( G_{fcc}=1.7G_{fc} \).

Pugh also suggested material regularization to be applied to reinforcing steel as well as concrete. Steel shows a strain-hardening behaviour in the post-yield region of the stress-strain curve. However, in a reinforced concrete element, for which concrete exhibits strain-softening at

![Figure 1](image-url)
the critical section, steel strains also localize at the critical section to conform to compatibility conditions. The post-yield energy of steel material is referred to as hardening energy and can be related to the stress-strain curve through a length measure. For steel, this length is the length, along which the inelastic deformation localizes and is taken equal to the gage length, $L_{gage}$, used in laboratory test. The scaling parameter to conserve the hardening energy is the rupture strain of reinforcing steel, calculated from the following expression:

$$\varepsilon_{su} = \varepsilon_y + \left(\varepsilon_{su,exp} - \varepsilon_y\right) \frac{L_{gage}}{L_{IP}}$$

(4)

where $\varepsilon_{su,exp}$ is the expected rupture strain and $\varepsilon_y$ is the yield strain of reinforcing steel material.

**Proposed Method for Verifying Local Strain Response**

The regularization of material models can be viewed as a mapping between the post-peak region of a reference material model and the post-peak region of the regularized model. This is illustrated in Fig.1 for confined concrete and reinforcing steel material models. The shaded areas belong to the reference material models before regularization and the transformed post-peak regions belong to the regularized models. The reference material models are assumed to have a characteristic length equal to the empirical plastic hinge of the column, $L_p$, which can be calculated from the following expression by Paulay and Priestley [11]:

$$L_p = 0.08L + 0.022f_{ye}d_b > 0.044f_{ye}d_b \text{ (mm, MPa)}$$

(5)

Where $L$ is the member length from the point of maximum moment to the point of contraflexure, $f_{ye}$ is the expected yield strength of the longitudinal rebars, and $d_b$ is the nominal diameter of the longitudinal rebars.

The question posed here is how to map the strain limits specified for the reference concrete model, $\varepsilon_{c1}$, and the reinforcing material model, $\varepsilon_{s1}$, to $\varepsilon_{c2}$ and $\varepsilon_{s2}$ in the corresponding regularized models. This is possible considering that the ratio of the post-peak energy up to the specified strain limit to the total post-peak energy is similar for the reference and the regularized material models. A detailed description of the mapping process and derivation of the corresponding expressions is given in the thesis of the first author [12]. The mapped strain limits for unconfined concrete, confined concrete, and reinforcing steel can be obtained from Eq. 6, 7, and 8, respectively:

$$\varepsilon_{c2} = \left(\varepsilon_{20u} - \varepsilon_o\right) \frac{\varepsilon_{c1} - \varepsilon_o}{\varepsilon_{u} - \varepsilon_o} + \varepsilon_o$$

(6)

$$\varepsilon_{c2} = \left(\varepsilon_{20c} - \varepsilon_{oc}\right) \frac{\varepsilon_{c1} - \varepsilon_{oc}}{\varepsilon_{cu} - \varepsilon_{oc}} + \varepsilon_{oc}$$

(7)

$$\varepsilon_{s2} = \varepsilon_y + \left(\varepsilon_{s1} - \varepsilon_y\right) \frac{L_{gage}}{L_{IP}}$$

(8)

In the above expressions, $\varepsilon_u$ and $\varepsilon_{cu}$ are the ultimate strains of the reference unconfined and
confined concrete material models. By mapping the strain limits to the regularized material models, it is possible to check the strains directly from the outputs of the regularized models, without any further post-processing of the strain values, or indirectly obtaining them from the post-processed curvatures. This would be very advantageous for checking the strain limits of CSA S6-14.

The steps of the proposed method for verifying the local strain response can therefore be summarized as: (1) regularizing the concrete and reinforcing steel material models using Eq. 2 to 4, (2) performing structural analysis of the regularized model, (3) mapping the specified strain limits from to the regularized material models using Eq. 6 to 8, (5) comparing the strain outputs of the regularized model against the mapped strain limits.

**Selected Bridge Column Test**

A reinforced concrete bridge column test was selected for study from the PEER structural performance database [13]. The column has a circular cross-section and is laterally reinforced with spirals, with varying spacing along the length of the column. The selected test is part of the experimental program developed by Lehman and colleagues at PEER [14] to study the cyclic performance of concrete bridge columns detailed for ductile flexural response in high seismicity zones. A schematic picture of the overall geometry and reinforcement details of the tested column and the applied lateral displacement time history are shown in Fig.2. Table 1 lists the reinforcement details and the axial load ratio of the column. The measured compressive strength of concrete was 34 MPa. The measured yield and ultimate strengths of the reinforcing steel were 448 and 634 MPa, respectively, and the yield strength of the spirals was 607 MPa. First the axial load was applied to the high strength rods on either sides of the specimen and was transferred to the column using a spreader beam. Next, the axial load was maintained constant, while the lateral load was applied. The loading protocol for the post-yield cycles included three cycles at each amplitude and a following cycle with one-third of the amplitude. The amplitude of the imposed

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![Figure 2](Figure2.png)

Figure 2. (a) The overall geometry and reinforcement details and (b) the lateral displacement time history of Column 328 [14].
Table 1. Reinforcement details and axial load ratio of the tested column

<table>
<thead>
<tr>
<th>Reinforcement</th>
<th>Column</th>
<th>Longitudinal $\rho_l$ (%)</th>
<th>Spiral Spacing (mm)</th>
<th>$\rho_s$ (%)</th>
<th>Axial Load Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>328</td>
<td>28 No. 6</td>
<td>2.8</td>
<td>25/50</td>
<td>0.87</td>
<td>0.1</td>
</tr>
</tbody>
</table>

lateral displacement was 1, 3, 5, 10 mm prior to yield, and 15, 20, 30, 51, 71, 102, and 132 mm for the post yield-cycles.

**Bridge Column Models in OpenSees**

The test column was modelled using inelastic beam-column elements with fiber sections. Both DBE and FBE were employed in the models, but due to space limitations on the paper only the results for the FBE models are presented. An elastic shear section was integrated into the fiber section in the FBE models to account for the shear stiffness loss of the column. The base of the column was restrained in all degrees-of-freedom to mimic the fixed-base condition in the test. A concentrated vertical load equal to the axial load in the test and a unit lateral load were applied at the top of the column. The lateral load was imposed at the center of the loading zone at the top of the column, from where the height of the column is measured. A displacement-controlled integrator was employed in the analysis, for which the control node was the node where the lateral load is applied. Two sections were defined for the plastic hinge zone and the rest of the column, due to different confinement conditions. Concrete02 is used for both of the unconfined and confined concrete material models, which is developed based on the Kent-Scott-Park constitutive relationship [9]. OpenSees does not automatically apply the confinement effects. Therefore, the confined concrete material properties were manually obtained using the Mander et al. confined concrete model [15].

For the reinforcing steel, the Steel02 material model was employed. This is a Giuffre-Menegotto-Pinto material model [16] with the addition of isotropic strain hardening proposed by Filippou et al. [17]. The strain hardening parameter, defined as the ratio of the post-yield stiffness to the initial elastic stiffness, was calculated as 0.01. The ultimate strain capacity of the reinforcing steel was set to 0.09, based on the recommendation of Caltrans SDC [18] for the reduced ultimate tensile strain of bar #10 or smaller. To impose this limit on the response of the steel fibers, a MinMax uniaxial material model was additionally assigned to the column sections with the limiting strain of 0.09 in tension and infinity in compression. This material model will ensure that the steel fibers strength reaches zero when the strain in those fibers is equal to 0.09, which will mimic the effect of rupture in the reinforcing steel.

**Global Response Verification**

To verify the global response of the models, first the cyclic force-displacement response of the FBE models were compared with the test results, without performing material regularization. The goal was not to calibrate the models so that they would reproduce the test results with the utmost accuracy; rather, it was to illustrate how well the current modelling techniques can predict the performance of RC bridge columns, when testing is not an option. This comparison is shown in Fig.3.a and b for the FBE models with 0.610 m mesh size and 2 integration points per element ($L_{IP}=0.305$ m) and 0.914 m mesh size and 3 integration points per element ($L_{IP}=0.152$ m). The two models, although having similar material models, yields very different results. The
response of the first model ($L_{IP}$ 0.305) gives a better match to the test results, because $L_{IP}$ for this model is closer to the empirical plastic hinge of the column, which is obtained as 0.376 m using Eq.5. Fig.3.a’ and b’ show the results of the same models after regularization. It is evident that the response of the two models becomes almost identical after regularization and in both cases, the models underestimate the displacement, at which the strength suddenly drops (failure displacement).

Figure 3. Comparison of the cyclic force-displacement response of the Column 328 FBE models with test results: (a) 0.610 m 2IPs before Reg. (a’) 0.610 m 2IPs after Reg. (b) 0.914 m 3IPs before Reg. (b’) 0.914 m 3IPs after Reg.

Figure 4. Monotonic response of the FBE models of Column 328 (a) before and (b) after material regularization.
Once the cyclic response of the Column 328 models were compared against the test results, the same models were employed to predict the monotonic response of the column, although there are no monotonic test results to allow a comparison. This comparison is shown in Fig.4 for four different FBE models with varying mesh size. Similarly, the monotonic responses of the models become almost identical after regularization, while they vary considerably before that (the FBE 1.828 m 4 IP has similar $L_{IP}$ as the FBE 0.914 m 3IP, and therefore they have similar predictions in Part a). The failure displacements of the four models before and after regularization are listed in Table 2. Although the regularized models underestimate the failure displacement of the column, they are the preferred choice for verifying the performance of the column, since otherwise, the response of the models and therefore the outcome of the performance evaluation becomes mesh dependent.

### Table 2. Failure displacements of the FBE models before and after regularization.

<table>
<thead>
<tr>
<th>Type</th>
<th>$L_{ele}$ (m)</th>
<th>IPs</th>
<th>$L_{IP}$ (m)</th>
<th>Failure Displacement (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FBE</td>
<td>0.610</td>
<td>2</td>
<td>0.305</td>
<td>0.155 0.096</td>
</tr>
<tr>
<td>FBE</td>
<td>0.914</td>
<td>3</td>
<td>0.152</td>
<td>0.081 0.100</td>
</tr>
<tr>
<td>FBE</td>
<td>1.829</td>
<td>4</td>
<td>0.152</td>
<td>0.081 0.102</td>
</tr>
<tr>
<td>FBE</td>
<td>1.829</td>
<td>6</td>
<td>0.061</td>
<td>0.039 0.104</td>
</tr>
</tbody>
</table>

Local Response Verification

To test the derived equations for verifying local strain limits, they were applied to predict the displacements of the regularized models of Column 328. The considered strain limits included the reinforcing steel strain of 0.002 (yielding) and 0.025, unconfined concrete strain of -0.004 (cover spalling), and confined concrete strain of -0.018 (core crushing). These strain limits were defined with respect to the reference material models, with $\varepsilon_u$ of -0.004, $\varepsilon_{cu}$ of -0.018, and $\varepsilon_{su,exp}$ of 0.09. The first step was to find the displacement of the top of the column from the monotonic response of the regularized models that would correspond to the first occurrence of these strain limits. These displacements are listed in Table 3.a. By inspecting the tabulated values, it is evident that except for the yielding criterion, which is not in the post-peak region, the regularized FBE models predict mesh-dependent displacements for the considered strain limits. For instance, the regularized FBE model with 0.610 m mesh size and 2 integrations points, predicts the crushing of confined concrete at the displacement of 0.071m, while the model with 1.828 m mesh size and 6 integration points predicts the same displacement at 0.027 m, less than half of the first value. It should be noted that all these models produce mesh-independent global force-displacement responses, since they are regularized. So the inconsistency in predicting these displacements relates to the difference in the local strain response of the regularized models.

The second step was to map the considered strain limits using Eq.6 to 8. These are listed in Table 3.b. For instance the confined concrete strain limit of -0.018 for the reference material model corresponds to strain limit of -0.022 for the regularized FBE 0.610 2 IP, and -0.094 for the regularized FBE 1.828 6IP. This significant difference in the mapped strain limits explains why the values of the displacements in Part (a) varied considerably.

Once the strain limits were mapped, now the displacements corresponding to the first occurrence of the mapped strain limits were found for the regularized models. These are listed in Table 3.c. Comparing the results in Part (c) with Part (a) shows that the discrepancy in predicting...
the displacements corresponding to the post-peak strain limits is considerably reduced for the FBE models. Considering the failure displacement of the models in Table 2, the predicted displacements for the mapped strain limits seem more reasonable. For instance, for the crushing of confined concrete, the mapped strain limit predicts displacement values between 0.078 to 0.081 m, and the reference strain limit predicts values between 0.027 to 0.071 m, while the predicted failure displacements were between 0.10 to 0.13 m.

Overall, it seems that the suggested method for verifying the local strain response is working very effectively with FBE models. It is very easy to implement for practical purposes and addresses an important issue with using distributed plasticity models for verifying the performance-based design of concrete bridges according to CSA S6-14.

Table 3. Column 328 regularized FBE models: (a) the displacements corresponding to the specified strain limits for the reference material models, (b) mapped strain limits, (c) the displacements corresponding to the mapped strain limits.

<table>
<thead>
<tr>
<th>Displacement (m)</th>
<th>$\varepsilon_s = \varepsilon_y = 0.002$</th>
<th>$\varepsilon_s = 0.025$</th>
<th>$\varepsilon_c = -0.004$</th>
<th>$\varepsilon_{cc} = -0.018$</th>
</tr>
</thead>
<tbody>
<tr>
<td>FBE 0.610 2IP</td>
<td>0.007</td>
<td>0.041</td>
<td>0.018</td>
<td>0.071</td>
</tr>
<tr>
<td>FBE 0.914 3IP</td>
<td>0.007</td>
<td>0.025</td>
<td>0.014</td>
<td>0.045</td>
</tr>
<tr>
<td>FBE 1.828 4IP</td>
<td>0.007</td>
<td>0.025</td>
<td>0.014</td>
<td>0.044</td>
</tr>
<tr>
<td>FBE 1.828 6IP</td>
<td>0.007</td>
<td>0.017</td>
<td>0.013</td>
<td>0.027</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Mapped Strain Limits</th>
<th>$\varepsilon_s = \varepsilon_y = 0.002$</th>
<th>$\varepsilon_s = 0.025$</th>
<th>$\varepsilon_c = -0.004$</th>
<th>$\varepsilon_{cc} = -0.018$</th>
</tr>
</thead>
<tbody>
<tr>
<td>FBE 0.610 2IP</td>
<td>0.002</td>
<td>0.017</td>
<td>-0.005</td>
<td>-0.022</td>
</tr>
<tr>
<td>FBE 0.914 3IP</td>
<td>0.002</td>
<td>0.033</td>
<td>-0.007</td>
<td>-0.040</td>
</tr>
<tr>
<td>FBE 1.828 4IP</td>
<td>0.002</td>
<td>0.033</td>
<td>-0.007</td>
<td>-0.040</td>
</tr>
<tr>
<td>FBE 1.828 6IP</td>
<td>0.002</td>
<td>0.078</td>
<td>-0.016</td>
<td>-0.094</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Corresponding Displacement (m)</th>
<th>$\varepsilon_s = \varepsilon_y = 0.002$</th>
<th>$\varepsilon_s = 0.025$</th>
<th>$\varepsilon_c = -0.004$</th>
<th>$\varepsilon_{cc} = -0.018$</th>
</tr>
</thead>
<tbody>
<tr>
<td>FBE 0.610 2IP</td>
<td>0.007</td>
<td>0.030</td>
<td>0.021</td>
<td>0.081</td>
</tr>
<tr>
<td>FBE 0.914 3IP</td>
<td>0.007</td>
<td>0.030</td>
<td>0.020</td>
<td>0.080</td>
</tr>
<tr>
<td>FBE 1.828 4IP</td>
<td>0.007</td>
<td>0.030</td>
<td>0.019</td>
<td>0.078</td>
</tr>
<tr>
<td>FBE 1.828 6IP</td>
<td>0.007</td>
<td>0.033</td>
<td>0.023</td>
<td>0.079</td>
</tr>
</tbody>
</table>

**Conclusions**

The use of distributed plasticity beam-column elements with fiber sections for verifying the performance of RC bridge components requires that the localization and mesh-dependency issue in these models are addressed properly. Material model regularization can be utilized to address the mesh-dependency issue. Further research however, is necessary to improve the equations used to predict the crushing energy of confined concrete.

A simple method for mapping the strain limits from reference material models of RC components to the regularized models was presented. The method proved to be an effective and
practical for checking the strain limits for performance-based design of RC bridges according to CSA S6-14.

Acknowledgments

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