Seismic Design of Reinforced Concrete Bridge Columns at Subfreezing Temperatures

by Luis A. Montejo, Elmer Marx, and Mervyn J. Kowalsky

INTRODUCTION

Results obtained from low temperature experimental tests have shown that reinforced concrete (RC) columns exposed to the combined effect of subfreezing temperatures and cyclic reversals undergo a gradual increase in strength and stiffness coupled with a reduction in displacement capacity. In this paper, the experimental results from past research are used to calibrate a fiber-based lumped-plasticity model capable of simulating the response of RC columns to cyclic load reversals while subjected to subfreezing temperatures. The model is used to analyze typical RC bridge bent configurations. To determine the impact of subfreezing temperatures on the seismic response of RC bridge bents, the bent models were subjected to inelastic lateral pushovers and a series of incremental inelastic time-history analyses using a set of spectrum-compatible records. Based on the results obtained from the experimental tests, the nonlinear simulations and a moment-curvature parametric analysis, a simple methodology was developed to account for the low temperature flexural overstrength and reduction in ductility capacity.

LOW TEMPERATURE EFFECT AT MATERIAL LEVEL

Previous research has shown that, when exposed to subfreezing temperatures, plain concrete and reinforcing steel exhibit a remarkable increase in strength without any loss in the deformation capacity. Concrete compressive strength $f'_c$ at low temperatures depends on the moisture content ($w$, percent of concrete dry weight) and can be properly estimated using the equation proposed by Browne and Banforth (Eq. (1)). Concrete moisture content is related to the concrete pore volume, which is governed by the age of the material, curing conditions, water-cement ratio ($w/c$), and aggregate gradation. Moisture content of in-place air-dried concrete is usually in the range 3 to 5%, and it will increase depending on the grade of exposure. The moisture content of water-saturated concrete can be as high as 8%. Figure 1 shows the increase in $f'_c$ with low temperatures according to Eq. (1) for a room temperature concrete strength $f'_c$ of 35 MPa (5 ksi). At $-40^\circ$C ($-40^\circ$F), the estimated increase in the compressive strength of water-saturated concrete is approximately twice that of in-place air-dried concrete.

$$f'_c(T) = f'_c(20^\circ C) - Tw/12 \quad 0^\circ C > T > -120^\circ C$$
$$f'_c(T) = f'_c(68^\circ F) - 5(T - 32)w/108 \quad 32^\circ F > T > -184^\circ F \quad (1)$$

In the case of reinforcing steel, yield stress and tensile strength increase with low temperatures at approximately the same rate. For design purposes, Eq. (2) can be used to estimate the reinforcing steel tensile stress at low temperatures (Fig. 2). No significant effect of low temperatures on the deformation capacity of reinforcing steel or plain concrete has been reported or was observed in the low temperature material tests performed as part of this research. Other properties such as steel-concrete bond strength, concrete fracture energy, and concrete tensile strength also increase with low temperatures in a substantial manner.

$$f_s(T) = (1 - 0.004T)f_s(20^\circ C) \quad 0^\circ C > T > -25^\circ C$$
$$f_s(T) = 1.1f_s(20^\circ C) \quad -25^\circ C > T > -40^\circ C \quad (2)$$
$$f_s(T) = (1 - T/450)f_s(68^\circ F) \quad 32^\circ F > T > -13^\circ F$$
$$f_s(T) = 1.1f_s(68^\circ F) \quad -13^\circ F > T > -40^\circ F$$

LOW TEMPERATURE EFFECT ON RC COLUMNS

The results from a series of large-scale tests aimed to identify the effect of subfreezing temperatures on the seismic

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behavior of RC columns is presented in Montejo et al. The columns tested in this study can be divided into three groups according to their structural behavior: 1) flexural-dominated ordinary RC (ORC) columns; 2) flexural-dominated RC-filled steel tube (RCFST) columns; and 3) shear-dominated ORC columns. The results obtained show that the flexural-dominated columns tested at low temperatures (-40°C [-40°F]) exhibit an increase in the flexural strength and a reduction in the displacement capacity. The increase in the flexural strength of the column can be explained by the enhancement in the mechanical properties of plain concrete and steel reinforcement when exposed to subfreezing temperatures. The decrement on the displacement capacity of the flexural-dominated columns was attributed to a substantial reduction in the spread of plasticity of the specimens tested at low temperatures, causing an increase in the curvature demand at the base of the column. The reduction on the spread of plasticity at low temperatures was evident from the condition of the specimens after the test. The reduced spread of plasticity was further confirmed with the curvature profiles obtained during the tests and the subsequent calculation of equivalent plastic hinge lengths. It was proposed to reduce the equivalent plastic hinge length $L_p$ used for room temperature calculations by a factor of 0.57 for the low-temperature condition as indicated in Eq. (3).

$$L_p = kL + L_{sp} \geq 2L_{sp} \quad T > 0°C(32°F)$$

$$L_p = 0.57(kL + L_{sp} \geq 2L_{sp})$$

$$0°C(32°F) \geq T \geq -40°C(-40°F)$$

where

$$L_{sp} = 0.022f_{y}d_{bl} (\text{MPa}) \quad 1 MPa = 0.145 \text{ ksi}$$

$$k = 0.2\left(\frac{f_{su}}{f_{y}} - 1\right) \leq 0.08$$

In the previous equations, $L$ is the distance from the base to the inflection point, $f_y$ is the expected longitudinal bar yield stress, $f_{su}$ is the expected longitudinal bar tensile strength, and $d_{bl}$ is the diameter of the longitudinal bar. It was shown that the theoretical monotonic envelopes can be obtained by applying the equivalent plastic hinge method with the proposed reduced plastic hinge length for low temperatures (Eq. (3)) and the appropriate temperature-dependent material properties (Eq. (1) and (2)). Figure 3 shows, for example, the experimental results (in the form of average first cycle envelopes) and predicted monotonic envelopes of a pair of flexural-dominated columns tested at warm and cold temperatures. Identified in the theoretical predictions are the stages when the extreme tension reinforcement in the base of the column reaches the yield strain ($\epsilon_y = 0.0023$) and strains of 0.015 and 0.07, which define the first yield, serviceability, and damage control limits, respectively (Table 1). In the case of shear-dominated RC columns, it has been shown that specimens tested at low temperatures exhibit an increase in the flexural strength of the column can be explained by the enhancement in the mechanical properties of plain concrete and a reduction in the displacement capacity. The increase in the flexural strength of the column is attributed to a substantial reduction in the spread of plasticity of the specimens tested at low temperatures, causing an increase in the curvature demand at the base of the column. The reduction on the spread of plasticity at low temperatures was evident from the condition of the specimens after the test. The reduced spread of plasticity was further confirmed with the curvature profiles obtained during the tests and the subsequent calculation of equivalent plastic hinge lengths. It was proposed to reduce the equivalent plastic hinge length $L_p$ used for room temperature calculations by a factor of 0.57 for the low-temperature condition as indicated in Eq. (3).

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increase in their shear strength. Flexural strength also increases at low temperatures, resulting in an increased shear demand; however, the shear capacity increases at an even higher proportion, thus delaying the onset of shear failure at low temperatures. Figure 4 shows the average first cycle force-displacement envelopes from the testing under cyclic reversals of a pair of identical shear dominated RC columns. One specimen was tested at room temperature conditions (22°C [71.6°F]) and the other at subfreezing temperatures (−36°C [−32.8°F]). Figure 4 also shows the theoretical shear strength envelopes calculated using the revised UCSD model. The increase in the shear strength is effectively captured by the UCSD model when the appropriate temperature-dependent material properties are used.

**FINITE ELEMENT MODEL**

Available experimental results from the testing of flexural dominated columns at low temperatures were used to calibrate a fiber-based lumped plasticity model. In a fiber-based model, the flexural member is represented by unidirectional fibers and constitutive material relationships are specified to each type of fiber. In RC members, for example, fibers representing the reinforcing steel, cover concrete (unconfined), and core concrete (confined) can be employed (Fig. 5). Once the model was calibrated, it was used to analyze the nonlinear seismic behavior of RC bridge bents exposed to subfreezing conditions. The simulations were performed in the OpenSees (Open System for Earthquake Engineering Simulation) software framework system using the BeamWithHinges element. This element confines the nonlinear constitutive behavior to plastic hinge regions of a specified length while maintaining numerical accuracy and objectivity. The model uses the force-based fiber beam-column element formulation for the hinge region and the section response between hinges is assumed linear elastic (Fig. 5). Confined and unconfined concrete fibers are modeled using the Concrete021 material. The input data required: maximum compressive strength, strain at maximum strength, crushing strength, and strain at crushing, were calculated as proposed by Mander et al. Longitudinal steel bars are modeled using the ReinforcingSteel material, this model accounts for degradation of strength and stiffness due to cycling according to a Coffin and Manson fatigue model through the factors \( \alpha, C_f, \) and \( C_d \). The damage strain range constant \( \alpha \) is used to relate damage from one strain range to an equivalent damage at another strain range and is constant for a material type. The ductility constant \( C_f \) is used to adjust the number of cycles to failure. A higher value of \( C_f \) translates to a larger number of cycles to failure. The strength reduction constant \( C_d \) controls the amount of degradation per cycle. A larger value for \( C_d \) will result in a lower reduction of strength for each cycle. Suggested values by Mohle and Kunnath for bars with a slenderness (ratio between the bar unsupported length and the bar diameter) of 6 are \( \alpha = 0.506, C_f = 0.26, \) and \( C_d = 0.38 \). Berry calibrated the model constants using the experimental results from 20 specimens of the bridge-column database in which bar fractures were reported; and the values recommended were \( \alpha = 0.506, C_f = 0.26, \) and \( C_d = 0.45 \). In general, these values are expected to change with the steel type, bar diameter, and the confinement provided to the section.

Material properties used for the distributed plasticity part of the BeamWithHinges element are those obtained from experimental tests at the corresponding temperatures and the plastic hinge lengths are calculated using Eq. (3). The elastic part of the element is modeled with the effective section inertia \( (E_I)^{eff} \) which is calculated from the moment-curvature response of the section as the slope of the line from the origin to the point of first yield of the longitudinal steel.

Response simulation results of the pair of columns presented in Fig. 3 are shown in Fig. 6 and 7. For each

<table>
<thead>
<tr>
<th>Limit state</th>
<th>Concrete compressive strain limit</th>
<th>Steel tensile strain limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Serviceability</td>
<td>−0.004</td>
<td>0.015</td>
</tr>
<tr>
<td>Damage control</td>
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<td>0.07</td>
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</tbody>
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Fig. 4—Low temperature effect in cyclic behavior of shear-dominated RC columns. (Note: 1°C = 9/5 × °F.)

Fig. 5—Fiber-based lumped-plasticity model.

Fig. 6—Experimental versus simulated: (a) hysteretic response; (b) average first-cycle envelope; (c) area-based hysteretic damping; and (d) base-curvature ductility for the column tested at room temperature conditions (22°C [71.6°F]).
column, the force-displacement hysteretic response, the average force displacement first cycle envelope, the area-based hysteretic damping, and the base-curvature ductility for column tested at low temperature conditions (−36°C [−32.8°F]).

Fig. 7—Experimental versus simulated: (a) hysteretic response; (b) average first-cycle envelope; (c) area-based hysteretic damping; and (d) base-curvature ductility for column tested at low temperature conditions (−36°C [−32.8°F]).

Optimal values obtained for the factors $\alpha$, $C_f$, and $C_d$ were 0.48, 0.22, and 0.30, respectively, for the columns reinforced with ASTM A706 No. 5 bars and 0.51, 0.34, and 0.45 for the columns reinforced with ASTM A615 No. 9 bars. The values of $\alpha$, $C_f$, and $C_d$ were kept constant for each pair of identical specimens, that is, the effect of low temperatures was captured by just incorporating the increase in the stiffness of the constitutive materials and the reduction in the plastic hinge length. In general, the degradation factors values vary with the steel type, bar diameter, and column transverse detailing. To limit the number of variables, the steel degradation factors were kept constant for all the following simulations. The values used are those proposed by Berry (α = 0.506, $C_f$ = 0.260, and $C_d$ = 0.45), which were obtained by calibration against a larger column database.

LOW TEMPERATURE EFFECT ON RC BRIDGE BENTS

The effect of subfreezing temperatures on the seismic response of RC bridge bents was analyzed by means of inelastic static pushover analyses and inelastic time history analyses. Pushover analyses were used to track levels of strain and formation of plastic hinges to determine displacement limit states, which were initially given in terms of material strain. The limit strain definitions used in this research were those recommended by Kowalsky for well-detailed RC columns (Table 1). Once the deformation limit states were obtained from the pushover analysis, an incremental dynamic analysis (IDA) was performed to determine the seismic level of intensity required to reach each limit state. The elastic damping in all the dynamic simulations is represented by 0.5% tangent stiffness proportional damping, that is, the damping coefficient is proportional to the instantaneous value of the stiffness and it is updated whenever the stiffness changes. The seismic input for the nonlinear time history analyses consisted of seven spectrum-compatible records generated through adjustments of recorded accelerograms using wavelet theory. The target spectrum is obtained from the 2003 NEHRP (National Earthquake Hazards Reduction Program) seismic design provisions (FEMA 450) for a site class “C” near Caribou Creek, Alaska. Figure 8 shows the target spectrum along with the response spectra of the modified records.

The bents analyzed are representative of typical Alaska Department of Transportation design practice. Different bent configurations are analyzed including single and multiple circular column bents. The column’s aspect ratios were varied between 3 and 7, and the columns’ longitudinal and
transverse reinforcement was maintained constant at approximately 2% and 1%, respectively. Figures 9 and 10 show the results obtained for a single column bent. From the pushover analysis (Fig. 9), it is seen that the average increase in flexural strength at low temperatures is approximately 12%. The lateral displacement demand required to reach the serviceability and damage control limit states at freezing conditions are 85% and 72%, respectively, of the displacements required to reach the same limit states at room temperature conditions. The damage control limit changed from being controlled by the concrete strain for the bent at room temperature to being controlled by the steel strain for the bent at low temperatures. Results obtained from the incremental dynamic analysis (Fig. 10) show that maximum lateral responses are almost identical up to peak ground acceleration (PGA) of approximately 0.5g where the serviceability limit is reached in both scenarios; after this point, lateral displacements are slightly larger at the room temperature condition.

Similar results are obtained for a four-column bent (Fig. 11 and 12). The displacement required to reach the serviceability and damage control limit states at subfreezing temperatures were 94% and 76%, respectively, of the lateral displacements required to reach the same limit states at room temperatures. The increase in the flexural strength at low temperatures was approximately 12%. Serviceability and damage control limits were controlled by the concrete compressive strain in the top hinges. More details on the inelastic simulations and the results obtained are presented elsewhere.18

PARAMETRIC STUDY

It has been shown that the response of RC columns subjected to the combined effect of low temperatures and lateral loads can be properly estimated using moment-curvature analysis along with the equivalent plastic hinge method if the appropriate temperature-dependent material properties and plastic hinge length are used. A parametric study was performed with the aim of discovering trends that can assist with the seismic design of RC bridge columns in cold regions. The main objective was to find a simple way to calculate the reduction in ductility capacity and increase in strength observed at low temperatures.

To quantify the increase of flexural strength at low temperatures, a series of moment curvature analyses were performed using the computer code CUMBIA.19 Three different section diameters were analyzed (457, 914, and 2440 mm [18, 36, and 96 in.]). For each section diameter, the axial load ratio was varied between 0 and 30%, and the longitudinal reinforcement ratio was varied between 1 and 4%. A total of 95 section configurations were analyzed, for each configuration, two moment-curvature analyses were performed, one with warm temperature (+20°C [68°F]) material properties and the other with low temperature (–40°C [–40°F]) material properties. The low temperature overstrength was defined as the ratio between the low temperature and room temperature nominal moments (defined at a cover concrete strain of 0.004). Based on the temperature-dependent material properties (Eq. (1) and (2); Fig. 1 and 2), concrete compressive strength and steel tensile strength were estimated to be 40% and 10%, respectively, larger than that employed for the room temperature condition. Figure 13 shows the overstrengths for all of the 95 section configurations along with the overstrengths obtained from the flexural units tested in Montejo et al.1,2 A low temperature flexural overstrength factor of 1.15 thus seems appropriate for the seismic design of RC columns exposed to subfreezing temperatures.

Determining the reduction in displacement-ductility capacity due to low temperatures requires the calculation of the force-displacement response of the member, which is extrapolated from the section (moment-curvature) analysis using the equivalent plastic hinge method. Material properties are the same as used for the overstrength factor analysis. Temperature-dependent equivalent plastic hinge lengths are calculated using Eq. (3). Two different section diameters (914 and 2440 mm [36 and 96 in.] were analyzed. Other variables analyzed included the axial load ratio (0% ≤ ALR ≤ 20%), longitudinal reinforcement ratio (1% ≤ ρl ≤ 4%), transverse reinforcement ratio (0.4% ≤ ρt ≤ 1.3%), and aspect ratio (3 ≤ L/D ≤ 11). For each member configuration, the force-displacement response was calculated for room and low temperature conditions. The reduction in the ductility capacity at low temperatures is defined by the ratio between the displacement ductilities at low and room temperatures at a given level of strain. Figures 14 and 15...
show the reduction of displacement ductility at low temperatures as a function of concrete and steel strain, respectively. It is seen from these graphs that the reduction in ductility increases with the lateral demand. Equations (4) and (5) were derived from the data presented in Fig. 14 and 15 and can be used to estimate the low temperature ductility capacity reduction for a given concrete strain, steel strain, or displacement ductility.

Alternatively, if the limit states are defined in terms of ductility instead of strains, Eq. (6) (Fig. 16) can be used.

$$\frac{\mu_{-40^\circ C(-40^\circ F)}}{\mu_{+20^\circ C(+68^\circ F)}} = 0.43\varepsilon_c^{-0.12} \quad R^2 = 0.93$$  (4)

$$\frac{\mu_{-40^\circ C(-40^\circ F)}}{\mu_{+20^\circ C(+68^\circ F)}} = 0.46\varepsilon_s^{-0.12} \quad R^2 = 0.95$$  (5)

$$\frac{\mu_{-40^\circ C(-40^\circ F)}}{\mu_{+20^\circ C(+68^\circ F)}} = 0.88(\mu_{+20^\circ C(+68^\circ F)})^{-0.17} \quad R^2 = 0.91$$  (6)

CONCLUSIONS

It is recommended to use a low-temperature overstrength factor (LTOF) of 1.15. This factor must be applied when determining the moment that the column will transmit to the cap beam or footing (if present) and to calculate the design shear force in the column. The LTOF should be applied in addition to any material overstrength factor conventionally used.

It was found that the reduction in ductility due to low temperatures increases with lateral demand. Equations (4) to (6) were developed to estimate the low-temperature ductility capacity reduction for a given concrete strain, steel strain, or displacement ductility.

If a displacement-based design approach is used, the effect of low temperatures can be directly taken into account by using the appropriate temperature-dependent material properties (Eq. (1) and (2)) and equivalent plastic hinge lengths (Eq. (3)).

All of the design recommendations are given based on a low temperature of –40°C (–40°F), as all of the available experimental tests were performed at approximately this temperature. The authors consider that the proposed equations can be conservatively used for the design of RC columns exposed to temperatures between 0 and –40°C (–40°F).

It is important to note that Eq. (3) was developed based on a limited number of tests and column configurations. Additional low temperature tests using different column configurations are desired to verify or calibrate the equations proposed in this manuscript.

REFERENCES


