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Priority: Standard
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6 November 2017

Mr. Edward Carley
Engineering Supervisor – License Renewal
NextEra Energy Seabrook, LLC.
P.O. Box 300, Lafayette Road
Seabrook, NH 03874

Project 170444 – License Amendment Request & NRC Review Support, NextEra Energy Seabrook Station, Seabrook, NH

Reference - Evaluation of Containment Enclosure Ventilation Area (CEVA), Calculation 160268-CA-05, Rev. 0 (FP101122), 22 March 2017

Document Number: 170444-L-001

Dear Mr. Carley:

The structural evaluation of the Containment Enclosure Ventilation Area (CEVA), referenced above, showed that its north wall between EL +3 and +19 ft was bowed outward with a maximum out-of-plumbness of 1.25 in., measured during SGH field observation in February 2017. The referenced calculation showed that the wall can move by an additional 25% (reaching to 1.5 in. at the point of maximum bowing) before conducting a more robust analysis or repairing the wall becomes necessary.

The bowing limit of 1.5 in. was calculated conservatively in the referenced calculation by neglecting the compression reinforcement in calculating the moment-curvature behavior of the wall. The limit of bowing is recalculated as shown in Attachment A considering the compression reinforcement in the moment-curvature calculation, and including the effects of upward vertical movement of CEVA. The upward movement of CEVA can impose vertical tension in the north wall of CEVA. Accordingly, moment-curvature relationships are calculated at different levels of tension.

At each level of axial force, a limiting curvature is determined to avoid unacceptable behavior. As explained in Attachment A, the curvature value of 0.003 1/in. corresponds either to the buckling of compression reinforcement (for cases with low level of axial tension) or rupture of tension reinforcement (for cases with high level of axial tension). Conservatively, 90% of this curvature value (0.0027 1/in.) is selected for computing the maximum bowing that the wall can resist.

The computed moment-curvature relationships are assigned to a finite element model of the wall, and nonlinear analyses are performed to determine the lateral displacement at which the curvature of the wall at the point of maximum bowing reaches the limiting curvature. Figure A2 in Attachment A shows that 2 in. of lateral wall deformation induces curvature of 0.0027 1/in. in the wall with a low level of axial force, while the same displacement induces curvature of 0.0023 1/in. in the wall with a high level of axial tension. Therefore, the wall can resist a lateral bowing of 2 in.

before buckling or rupture of the vertical reinforcement will occur (with 10% margin), and it is recommended that the wall be retrofitted prior to it experiencing 2 in. of bowing.

The site visit report (170444-SVR-01-R0, FP101192-00) for the recent measurements indicates that the wall moved outward since the previous measurements; bulging has increased to 1-7/16 in. Additionally, at a localized area, hammer sounding indicated portions of the concrete cover potentially susceptible to separation, especially at areas with larger horizontal cracks. Therefore, sensitive equipment or processes should be protected against possible falling pieces of concrete cover from this wall before it is retrofitted.

Sincerely yours,

Said Bolourchi
Senior Principal

Michael Mudlock, P.E.
Senior Project Manager
NH License No. 14808

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ATTACHMENT A

MOMENT CURVATURE RELATION FOR THE CEVA NORTH WALL

A1. REVISION HISTORY

Revision 0: Initial document.

A2. OBJECTIVE OF CALCULATION

The objective of this calculation is to compute moment-curvature diagrams for the north wall of Containment Enclosure Ventilation Area (CEVA) structure subjected to different tensile force.

A3. RESULTS AND CONCLUSIONS

Figure A2 presents moment-curvature diagram for the wall section subjected to different tensile force. As can be seen from the figure, by increasing the magnitude of tensile force, the failure mode changes from compressive rebar buckling to tensile rebar rupture. For all cases, the point of severe damage initiation approximately corresponds to curvature value of 0.003 (1/in.). Accordingly, this point is selected as a limiting curvature and the wall is allowed to deform up to this curvature.

Figure A3 provides a comparison between two cases, one with accounting for the effect of compressive rebars and the other one without considering the compressive rebars. As can be seen from the figure, including the resistance offered by compressive reinforcement does not affect the ultimate capacity noticeably, however, it increases the ductility of a section.

A4. DESIGN DATA / CRITERIA

See Section 4 of the calculation main body (160268-CA-05 Rev.0).

A5. ASSUMPTIONS

A5.1 Justified assumptions

There are no justified assumptions.

A5.2 Unverified assumptions

There are no unverified assumptions.

A6. METHODOLOGY

To calculate the moment-curvature diagrams, sectional analysis based on fiber section method for integrating over the cross section is used. In this method, the cross section is discretized into fibers (or layers subjected to unidirectional bending), and an appropriate material model is assigned to each fiber. Figure A1 demonstrates a typical fiber section discretization. In this study, each section is discretized into 20 fibers/layers. The concrete material is represented by Kent and Park [A5] model in compression and Steven's exponential softening model in tension [A3]. Reinforcing steel bars are modeled using elastic perfectly plastic material with strain cutoff of 0.007 in tension and accounting for inelastic buckling model of Kashani et al. [A4] in compression.

Moment-curvature diagrams are then calculated for axial tension of 0, 10, 30 and 60 kips. Each point is derived by selecting an axial force, and then changing curvature value and calculating the moment. Figure A2 presents the computed moment-curvature diagrams.

An additional study is conducted to show the effect of including or excluding compressive reinforcement. The study shows accounting for the effect of compressive rebars increases the ductility of a member as presented in Figure A3; therefore, the member can accommodate more displacement.

A7. REFERENCES

[A1] Simpson Gumpertz & Heger Inc., *Evaluation of Containment Enclosure Ventilation Area,* 160268-CD-05 Rev. 1, Waltham, MA, Mar. 2017.

[A2] United Engineers & Constructors Inc., Seabrook Station Structural Design Drawings.

[A3] Yuan Lu, and Marios Panagiotou. "Three-dimensional cyclic beam-truss model for nonplanar reinforced concrete walls." Journal of Structural Engineering, 2013, 140(3): 04013071.

[A4] Mohammad M. Kashani, Laura N. Lowes, Adam J. Crewe, Nicholas A. Alexander, Phenomenological hysteretic model for corroded reinforcing bars including inelastic buckling and low-cycle fatigue degradation, Computer and Structures, 156 (1), 58-71, 2015.

[A5] Dudley. C. Kent, and Robert Park, Flexural members with confined concrete, ASCE Journal of Structural Division, 97 (ST7), 1969-1990, 1971.

A8. COMPUTATION

Geometry

Rebar diameter $d_b := 1.128in$

Reinforcement ratio $\rho := \frac{2 \cdot 1in^2}{12in \cdot 24in} = 6.944 \times 10^{-3}$

Concrete Material Model

Compressive strength of concrete $f_c := -3ksi$

Kent & Park Model

Strain at Peak compressive strength $\varepsilon_{co} := -0.002$

Strain at 50% compressive strength $\varepsilon_{50u} := \frac{3 - 0.002 \cdot \frac{f_c}{psi}}{\frac{f_c}{psi} + 1000} = -4.5 \times 10^{-3}$

Model parameter $Z := \frac{0.5}{\varepsilon_{50u} - \varepsilon_{co}} = -200$

Residual compressive strength $f_{c.res} := f_c \cdot 0.025 = -75 \cdot psi$

Steven's Model

Young's modulus of concrete $E_c := 3120ksi$

Tensile strength of concrete $f_t := 5 \cdot \sqrt{\left|f_c\right| \cdot psi} = 273.861\,psi$

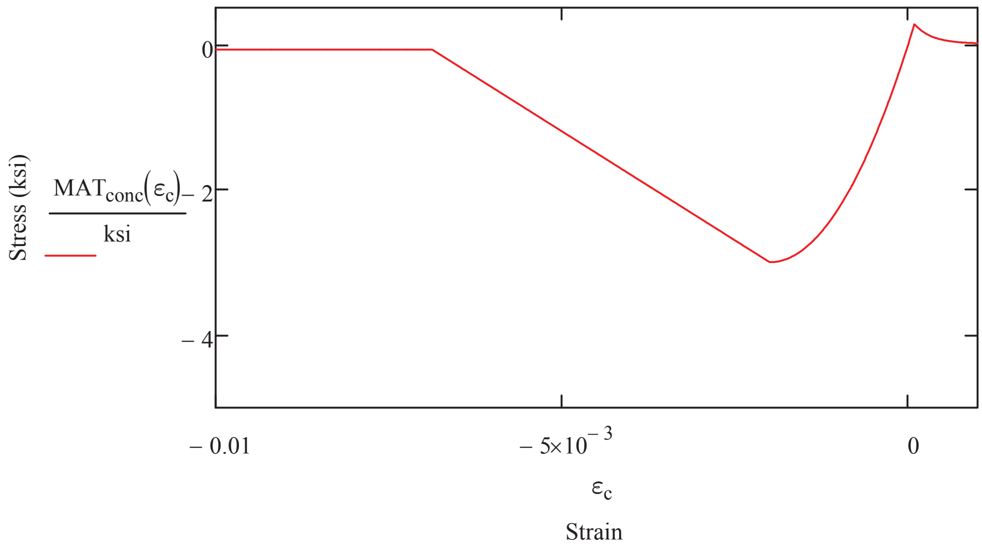
Strain at cracking $\varepsilon_{cr} := \frac{f_t}{E_c} = 8.778 \times 10^{-5}$

Model parameter that controls residual $M := 75mm \frac{\rho}{d_b} = 0.018$

Model parameter that controls softening slope $\lambda t := \frac{540}{\sqrt{M}} = 4.005 \times 10^3$

Constitutive model for concrete

$$MAT_{conc}(\varepsilon) := \begin{cases} \min\left[f_{c.res}, f_c \cdot \left[1 - Z \cdot \left(\varepsilon - \varepsilon_{co}\right)\right]\right] & \text{if } \varepsilon < \varepsilon_{co} \\ f_c \cdot \left[\frac{2 \cdot \varepsilon}{\varepsilon_{co}} - \left(\frac{\varepsilon}{\varepsilon_{co}}\right)^2\right] & \text{if } \varepsilon_{co} \le \varepsilon < 0 \\ E_c \cdot \varepsilon & \text{if } 0 \le \varepsilon < \varepsilon_{cr} \\ f_t \cdot \left[(1 - M) \cdot e^{-\lambda t \cdot \left(\varepsilon - \varepsilon_{cr}\right)} + M\right] & \text{if } \varepsilon_{cr} \le \varepsilon \end{cases}$$



Steel Material Model

Yield strength of steel $f_y := 60ksi$

Young's modulus of steel $E_s := 29000ksi$

Yield strain $\varepsilon_y := \frac{f_y}{E_s} = 2.069 \times 10^{-3}$

Rupture strain $\varepsilon_{sr} := 0.07$

Account for buckling $Buckling := 1$ Put 0 to ignore, put 1 to consider

Kashani Model (buckling)

Unbraced length/Diameter $LD := 15$

Slenderness ratio $\lambda p := \sqrt{\frac{f_y}{100MPa}} \cdot LD = 30.509$

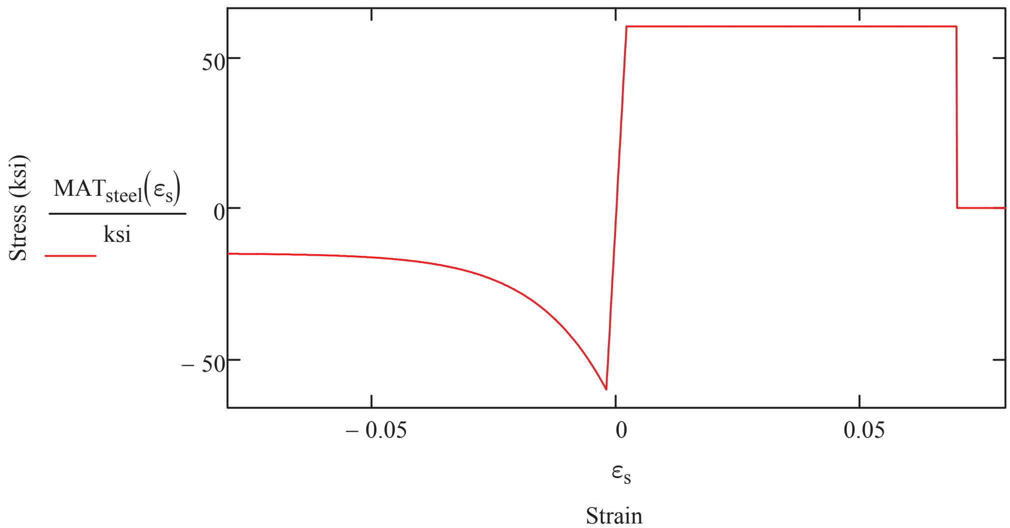
Model parameters $\rho 1 := 4.572 \cdot \lambda p - 74.43 = 65.057$

$\rho 2 := 318.4 \cdot e^{-0.071 \cdot \lambda p} = 36.495$

$\sigma_{star} := 3.75 \cdot \frac{f_y}{LD} = 15 \cdot ksi$

Constitutive model for steel

$$MAT_{steel}(\varepsilon) := \begin{cases} \begin{cases} -f_y & \text{if Buckling} = 0 \\ -\left[\sigma_{star} + (f_y - \sigma_{star})\cdot e^{\left[-\left(\rho 1 + \rho 2\cdot\sqrt{|\varepsilon| - \varepsilon_y}\right)\cdot\left(|\varepsilon| - \varepsilon_y\right)\right]}\right] & \text{otherwise} \end{cases} & \text{if } \varepsilon < -\varepsilon_y \\ f_y & \text{if } \varepsilon_y \le \varepsilon < \varepsilon_{sr} \\ 0 & \text{if } \varepsilon_{sr} \le \varepsilon \\ (E_s\cdot\varepsilon) & \text{otherwise} \end{cases}$$



Concrete Fibers

Ref. 2, 2ft thick wall

Width of fibers $b := 12in$

Total thickness or height $h := 24in$

Number of fibers $Conc_{Num} := 20$

Height of fibers $Conc_H := \frac{h}{Conc_{Num}} = 1.2\cdot in$

Concrete fiber coordinates

$$Conc_y := \left| \begin{array}{l} \text{for } i \in 1..Conc_{Num} \\ \quad ans_i \leftarrow -\frac{h}{2} + \frac{Conc_H}{2} + (i-1)\cdot Conc_H \\ ans \end{array} \right.$$

Concrete fiber strain

$$Conc_\varepsilon(\varepsilon_o, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Conc_{Num} \\ \quad ans_i \leftarrow \varepsilon_o - \varphi\cdot Conc_{y_i} \\ ans \end{array} \right.$$

Concrete fiber stress

$$Conc_{\sigma}(\varepsilon_o, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Conc_{Num} \\ \quad ans_i \leftarrow MAT_{conc}\left(Conc_{\varepsilon}(\varepsilon_o, \varphi)_i\right) \\ ans \end{array} \right.$$

Concrete fiber force

$$Conc_{F}(\varepsilon_o, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Conc_{Num} \\ \quad ans_i \leftarrow Conc_{\sigma}(\varepsilon_o, \varphi)_i \cdot (b \cdot Conc_H) \\ ans \end{array} \right.$$

Reinforcement/Steel fibers

Depth to reinforcement $d := 20.5in$ Ref. 2, 2ft thick wall

Area of tensile reinforcement (#9@12 in.) $A_s := 1in^2$

Number of reinforcement in row, e.g. equal to 2 for tensile and compressive $Steel_{Num} := 2$

Depth to reinforcement fiber

$$Steel_{y_1} := -\left(d - \frac{h}{2}\right) = -8.5 \cdot in$$

$$Steel_{y_2} := d - \frac{h}{2} = 8.5 \cdot in$$

Area of reinforcement fiber

$$Steel_{As_1} := A_s = 1 \cdot in^2$$

$$Steel_{As_2} := A_s = 1 \cdot in^2$$

Steel fiber strain

$$Steel_{\varepsilon}(\varepsilon_o, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Steel_{Num} \\ \quad ans_i \leftarrow \varepsilon_o - \varphi \cdot Steel_{y_i} \\ ans \end{array} \right.$$

Steel fiber stress

$$Steel_{\sigma}(\varepsilon_o, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Steel_{Num} \\ \quad ans_i \leftarrow MAT_{steel}\left(Steel_{\varepsilon}(\varepsilon_o, \varphi)_i\right) \\ ans \end{array} \right.$$

Steel fiber force

$$Steel_{F}(\varepsilon_o, \varphi) := \left| \begin{array}{l} \text{for } i \in 1..Steel_{Num} \\ \quad ans_i \leftarrow Steel_{\sigma}(\varepsilon_o, \varphi)_i \cdot Steel_{As_i} \\ ans \end{array} \right.$$

Axial Equilibrium

$$\text{Force}(\varepsilon_o, \varphi) := \begin{array}{|l} ans1 \leftarrow 0 \\ \text{for } i \in 1..Conc_{Num} \\ \quad ans1 \leftarrow ans1 + Conc_F(\varepsilon_o, \varphi)_i \\ ans2 \leftarrow 0 \\ \text{for } i \in 1..Steel_{Num} \\ \quad ans2 \leftarrow ans2 + Steel_F(\varepsilon_o, \varphi)_i \\ ans \leftarrow ans1 + ans2 \end{array}$$

Moment Equilibrium

$$\text{Moment}(\varepsilon_o, \varphi) := \begin{array}{|l} ans1 \leftarrow 0 \\ \text{for } i \in 1..Conc_{Num} \\ \quad ans1 \leftarrow ans1 + -1 \cdot Conc_F(\varepsilon_o, \varphi)_i \cdot Conc_{y_i} \\ ans2 \leftarrow 0 \\ \text{for } i \in 1..Steel_{Num} \\ \quad ans2 \leftarrow ans2 + -1 \cdot Steel_F(\varepsilon_o, \varphi)_i \cdot Steel_{y_i} \\ ans \leftarrow ans1 + ans2 \end{array}$$

Solution

Known parameters

Axial force $P := 60kip$

Iteration

Curvature $\phi := 0.003 \cdot \frac{1}{in}$

Solve for strain at centroid

Axial strain at centroid (initial guess) $x_o := 0.03$ Requires iteration

Axial force equilibrium $f(x) := Force(x, \phi) - P$

$\varepsilon_{cent} := root(f(x_o), x_o) = 0.027$

Sectional forces

$Force(\varepsilon_{cent}, \phi) = 60 \cdot kip$

$Moment(\varepsilon_{cent}, \phi) = 48.079 \cdot kip \cdot ft$

Stress and strain in concrete and steel

Steel fiber stress and strain

$$Rebar_{\varepsilon} := Steel_{\varepsilon}(\varepsilon_{cent}, \phi) = \begin{pmatrix} 0.052 \\ 1.266 \times 10^{-3} \end{pmatrix}$$

$$Rebar_{\sigma} := Steel_{\sigma}(\varepsilon_{cent}, \phi) = \begin{pmatrix} 60 \\ 36.723 \end{pmatrix} \cdot ksi$$

$$Steel_{F}(\varepsilon_{cent}, \phi) = \begin{pmatrix} 60 \\ 36.723 \end{pmatrix} \cdot kip$$

$$Concrete_{y} := Conc_{y}$$

Concrete fiber stress and strain

$$Concrete_{\varepsilon} := Conc_{\varepsilon}(\varepsilon_{cent}, \phi)$$

$$Concrete_{\sigma} := Conc_{\sigma}(\varepsilon_{cent}, \phi)$$

Maximum compressive strain in concrete

$$\varepsilon_{max.comp} := \frac{Rebar_{\varepsilon_2} - Rebar_{\varepsilon_1}}{Steel_{y_2} - Steel_{y_1}} \cdot \left(\frac{h}{2} - Steel_{y_1}\right) + Rebar_{\varepsilon_1} = -9.234 \times 10^{-3}$$

A9. TABLES

There are no tables.

A10. FIGURES

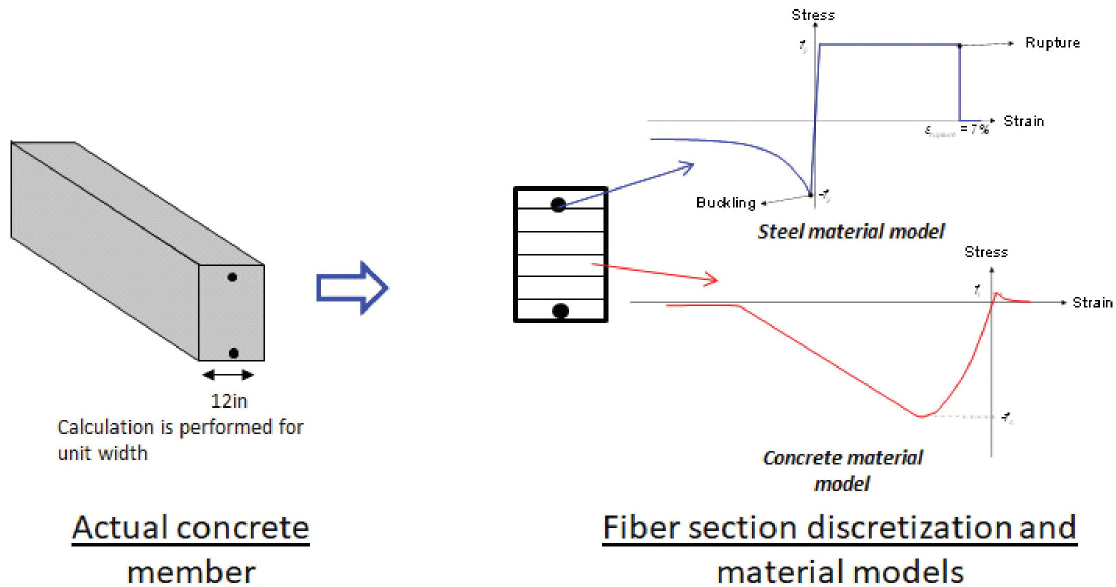


Figure A1: Schematic representation of fiber section method

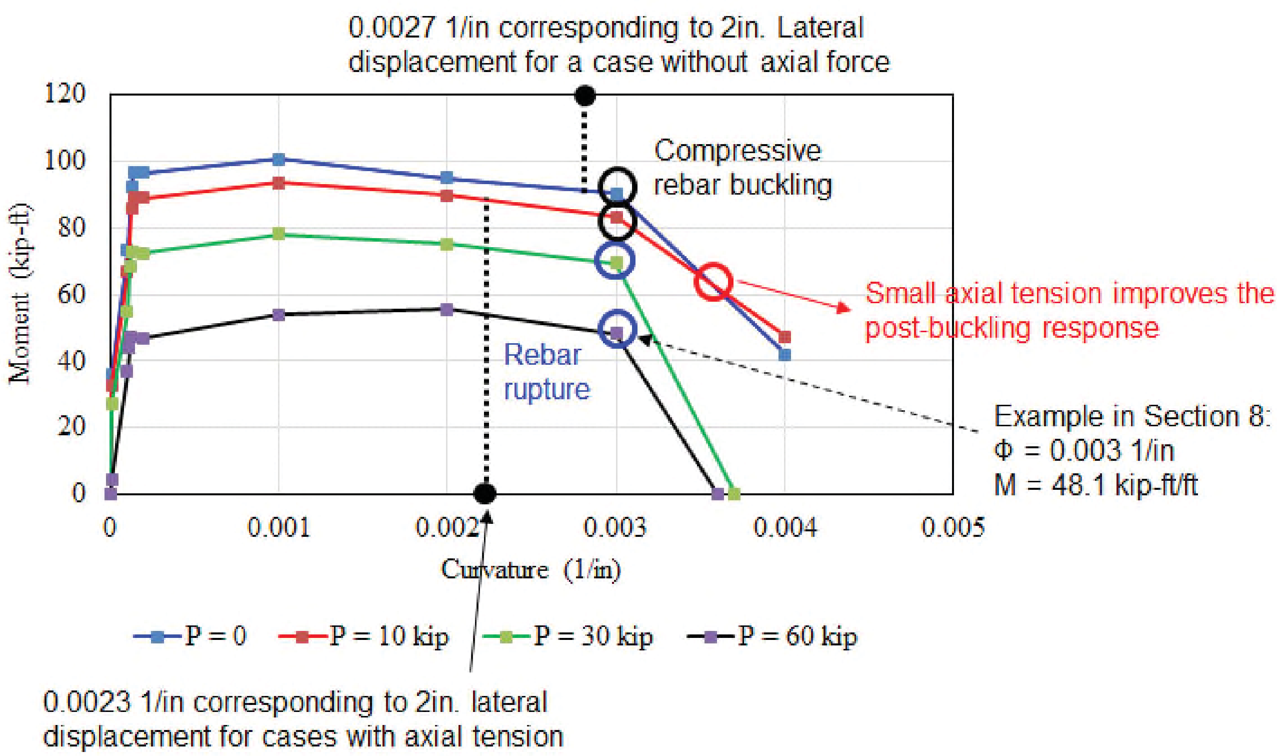


Figure A2: Moment – curvature diagrams for different axial force

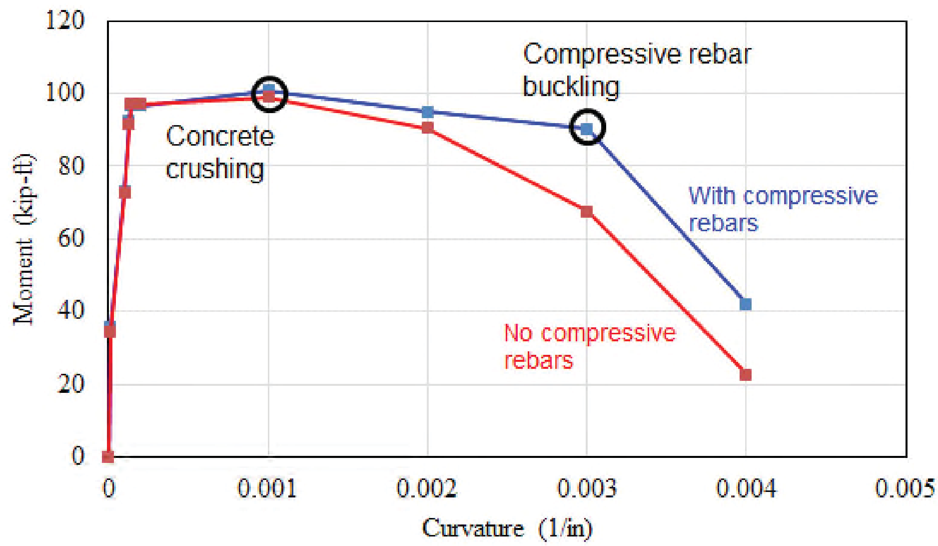


Figure A3: Moment – curvature diagram considering the effect of including or excluding compressive rebars