

## **CUMBIA-Walls. Set of codes for the analysis of reinforced concrete walls**

Theory and User Guide

[luis.montejo@upr.edu](mailto:luis.montejo@upr.edu)

[arsalan\\_niroomandi@yahoo.co.nz](mailto:arsalan_niroomandi@yahoo.co.nz)

## DISCLAIMER

There are **no warranties**, explicit or implicit, that the codes contained in this collection are free of error, or are consistent with any particular standard of accuracy, or that they will meet your requirements for any particular application. They should not be relied on for any purpose where incorrect results could result in loss of property or personal injury. If you do use the codes for any such purpose it is at your own risk. The authors disclaim all liability of any kind, direct or consequential, resulting from your use of these codes.

CUMBIA-Walls is available from <https://github.com/LuisMontejo/CUMBIA-Walls>

If you use CUMBIA-Walls to produce scientific articles, please cite as:

Montejo, L.A. and Niroomandi, A. 2021. Cumbia-Walls - set of codes for the analysis of reinforced concrete walls. <http://doi.org/10.5281/zenodo.5136688>

# **1. INTRODUCTION**

CUMBIA (Montejo and Kowalsky 2007) is a set of Matlab/Octave codes to perform monotonic moment-curvature analysis and force-displacement response of reinforced concrete (RC) members of rectangular or circular sections. This version of CUMBIA is following the CUMBIARECT code and is specifically adjusted for RC walls. The parts of the code that are identical to the CUMBIARECT code are not repeated here. Therefore, the users should refer to the original CUMBIA Manual that is also available within this package. An axial load – moment interaction analysis can also be performed. The section analysis is performed by tabulating moment and curvature of the member section for increasing levels of concrete strain. The member response is obtained from the section moment-curvature results along with an equivalent plastic hinge length, as presented by Paulay and Priestley (1992). Shear deformations are computed following the procedure described in Priestley, Calvi and Kowalsky (2007). The shear strength envelope for the member is calculated using the shear model proposed by Krollicki, Maffei and Calvi (2011) which is based on the model proposed by Kowalsky and Priestley (2000) and was revised for RC walls. The onset of buckling is determined according to three different models, proposed by Moyer and Kowalsky (2003), Berry and Eberhard (2005) and Alvarado, Rodriguez and Restrepo (2015). The plastic rotation capacity corresponding to lateral instability of the walls is determined according to the equation proposed by Oliver, Boys and Marriott (2012) which is based on the work of Paulay and Priestley (1993).

The constitutive models for the concrete and steel can be easily specified by the user. Nonetheless, the code has some default models. The default models for the unconfined and confined models are those proposed by Mander, Priestley and Park (1988). There are two models for the steel, one is the same used by the King program (1986) and the other is the proposed by Raynor et al. (2002). The code allows the analysis of members subjected to axial load (tension or compression) and single or double bending.

## **2. MATERIAL MODELS**

The constitutive models for the concrete (confined and unconfined) and reinforcing steel can be specified by the user or the default models can be used.

### **2.1. Default Models**

#### **2.1.1. Model for the unconfined and confined concrete**

Mander, Priestley and Park (1988) model was used for both the unconfined and confined behaviour of concrete. The only adjustment in this current version of CUMBIA is that the ultimate strain of concrete is defined based on Equation (1) proposed by fib (2003) which is a revised version of the equation originally proposed by Paulay and Priestley (1992).

$$\varepsilon_{cu} = \max \left( \left( 0.004 + \frac{0.6\rho_s f_{yh} \varepsilon_{sm}}{f'_{cc}} \right), \varepsilon_{spall} \right) \quad (1)$$

In this equation,  $\varepsilon_{sm}$  should not be larger than 0.06 as recommended by Kowalsky (2000).

#### **2.1.2. Reinforcing Steel models**

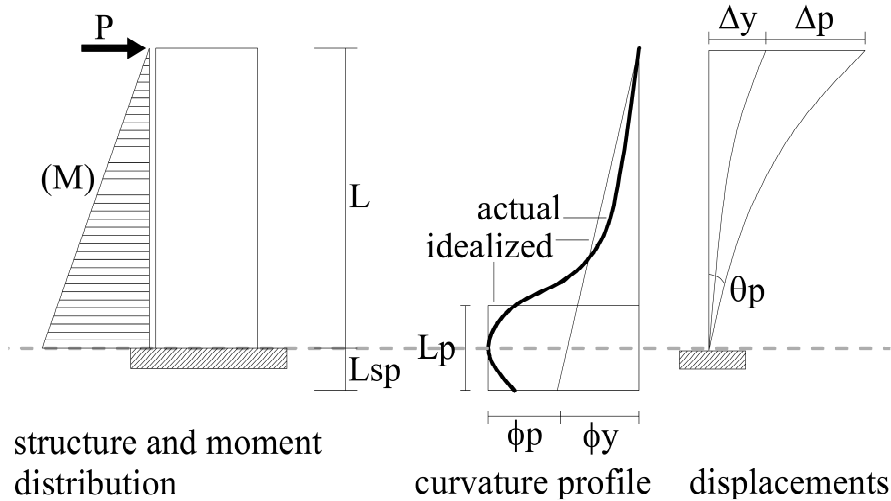
Two different methods proposed by King, Priestley and Park (1986) and Raynor, Lehman and Stanton (2002) are available as default material for steel reinforcement. Both models are identical to the ones used for CUMBIARECT code.

### **3. SECTION ANALYSIS**

The section analysis procedure follows the same approach as the CUMBIARECT code.

## 4. MEMBER RESPONSE

The member response is obtained using the plastic hinge method proposed by Paulay and Priestley (1992). The plastic hinge method replaces the real curvature distribution with an equivalent curvature distribution in order to facilitate the application of the moment area method to find the displacements in the member. The length of the equivalent plastic hinge ( $L_p$ ) is defined as the length over which the maximum curvature can be assumed to be constant. equations (2) and (3) are used to calculate the equivalent plastic hinge and the strain penetration lengths, respectively. An effective length of the member is defined as equations (4) and (5). The flexural displacement of the member before yielding is calculated according to equations (6) and (7). The flexural displacement beyond yield is given by equations (8) and (9).



**Figure 1. Plastic hinge method**

$$L_p = kL_c + 0.1L_w + L_{sp} > 2L_{sp} \quad (2)$$

$$L_{sp} = 0.022f_s d_{bl} \quad f_s \leq f_y \quad (3)$$

Where:

$L_c$  : length from the critical section to the point of contraflexure. In the case of the single bending walls, this length is equal to the effective height of the wall and in the case of the double bending walls it is equal to half the full height.

$$L_{eff} = L + L_{sp} \quad \text{single bending} \quad (4)$$

$$L_{eff} = L + 2L_{sp} \quad \text{double bending} \quad (5)$$

$$\Delta_f = \frac{\phi L_{eff}^2}{3} \quad \text{single bending} \quad (6)$$

$$\Delta_f = \frac{\phi L_{eff}^2}{6} \quad \text{double bending} \quad (7)$$

$$\Delta_f = \Delta'_y \frac{M}{M'_y} + \left[ \phi - \phi'_y \frac{M}{M'_y} \right] L_p (L + L_{sp} - 0.5L_p) \quad \text{single bending} \quad (8)$$

$$\Delta_f = \Delta'_y \frac{M}{M'_y} + \left[ \phi - \phi'_y \frac{M}{M'_y} \right] L_p (L + 2(L_{sp} - 0.5L_p)) \quad \text{double bending} \quad (9)$$

A simplified approach for calculating the shear stiffness in the elastic range of response based on the cracked section stiffness suggested by Priestley, Calvi and Kowalsky (2007) is used to calculate the shear deformations. The premise of this approach is that the elastic shear stiffness is reduced approximately in proportion to the flexural stiffness. After shear cracking and before the section reaches its nominal moment, shear deflections are computed by considering the shear flexibility of an equivalent strut-and-tie model. The shear displacement beyond yield is assumed to increase proportional to the flexural displacement. These are discussed in the following. The total displacement in the member is given then by the equation (10).

$$\Delta = \Delta_f + \Delta_s \quad (10)$$

### Phase 1: Elastic, Prior to Shear Cracking

The shear deformation of the walls before cracking are calculated using equation (11) where,  $V$  is the shear force acting in the member. Shear cracking is assumed to occur when the applied shear is larger than the shear strength of the concrete  $V_c$ , which is obtained from equation (24) with  $\gamma=0.29$  (i.e. the initial concrete strength).

$$\Delta_{s,l} = \frac{V}{k_{seff}}, \quad V < V_c \quad (11)$$

$$k_{seff} = k_{sg} \frac{I_{eff}}{I_g} \quad (12)$$

$$k_{sg} = \frac{G_c A_s}{L} \quad (13)$$

$$I_{eff} = \frac{M'_y}{E_c \phi'_y} \quad (14)$$

$$A_s = \frac{5}{6} A_g \quad (15)$$

$$E_c = 4700 \sqrt{f'_c} \quad (MPa) \quad (16)$$

$$G_c = 0.43 E_c \quad (17)$$

### Phase 2: Elastic, After Shear Cracking

After shear cracking occurs and before the nominal moment is reached, the shear stiffness is calculated using equation (19), where B and d are the effective width and depth of the section.

$$\Delta_{scr} = \Delta_{s,l} + \frac{V - V_c}{k_{scr}}, \quad V_c < V < V_y \quad (18)$$

$$k_{scr} = \frac{0.25 \rho_y}{0.25 + 10 \rho_y} E_s B d \quad (19)$$

$$V_y = \frac{M_y}{L} \quad (20)$$

### Phase 3: Ductile phase

The shear displacement beyond yield is assumed to increase proportional to the flexural deformation after yield as shown in equation (21).

$$\Delta_s = \Delta_f \frac{\Delta_{s,y}}{\Delta_{f,y}}, \quad V > V_y \quad (21)$$

$$\Delta_{s,y} = \Delta_{s,l} + \frac{V_y - V_c}{k_{scr}} \quad (22)$$



## 5. SHEAR CAPACITY

The shear strength envelope for the member is calculated using the shear model proposed by Krollicki, Maffei and Calvi (2011) which is based on the model proposed by Kowalsky and Priestley (2000) and was revised for RC walls. The 0.85 strength reduction factor is due to the uncertainty with the shear capacity of RC walls and not the common design strength reduction factor.

$$V = 0.85(V_c + V_s + V_p) \quad (23)$$

$$V_c = \alpha\beta\gamma\sqrt{f'_c}0.8A_g \quad (24)$$

$$1 \leq \alpha = 3 - \frac{L}{H} \leq 1.5 \quad (25)$$

$$\beta = 0.5 + 20\rho_l \leq 1 \quad (26)$$

$$0.05 \leq \gamma = 0.41 - 0.06\mu_\Delta \leq 0.29 \quad (27)$$

It should be noted that  $\mu_\Delta = \Delta/\Delta_{y,f}$ . Where  $\Delta_{y,f} = \Delta'_{y,f} M_N/M_y$ .

$$V_s = \frac{A_v f_{yh} h_{cr}}{s} \quad (28)$$

$$h_{cr} = \frac{l'}{\tan \theta_{cr}} \leq L \quad (29)$$

$$l' = H - c - c_0 \quad (30)$$

Where,  $c$  is the depth of the neutral axis at  $M_N$  and  $c_0$  is the cover to the main bars.

$$\theta_{cr} = 45 - 7.5 \frac{L}{H} \geq 30 \quad (31)$$

$$V_p = P \frac{H - c}{2L} \quad \text{Single bending} \quad (32)$$

$$V_p = P \frac{H - c}{2L} \quad \text{double bending} \quad (33)$$

$$V_p = 0 \quad \text{for } P < 0 \quad (34)$$

## 6. BAR BUCKLING MODELS

The assessment of reinforcing buckling limit state is implemented in the code following three different methodologies, one is the proposed by Moyer and Kowalsky (2003), the second is the proposed by Berry and Eberhard (2005) and the third is the proposed by Alvarado, Rodriguez and Restrepo (2015). The first two methods are discussed before and only the third method is briefly explained here. According to Alvarado, Rodriguez and Restrepo (2015), the curvature capacity of the wall at the onset of bar buckling is determined according to equation (35). Then, the displacement at the onset of bar buckling can be determined using equations (8) and (9).

$$\phi_{buckling} = \frac{\varepsilon_p^*}{\gamma H} \quad (35)$$

$$0.02 \leq \varepsilon_p^* = \frac{11 - s_h/d_b}{150} \leq 0.06 \quad (36)$$

$$\gamma H = H - 2c_0 \quad (37)$$

## 7. GLOBAL BUCKLING OF THE WALL

The plastic rotation capacity corresponding to lateral instability of the walls is determined according to the equation proposed by Oliver, Boys and Marriott (2012) which is based on the work of Paulay and Priestley (1993).

$$\theta_p = \frac{2\varepsilon_y}{H} L_{p,w} \left( \frac{40b_c^2 E_s \beta_w \xi_c}{7l_o^2 f_y} - 1 \right) \quad (38)$$

Where:

$L_{p,w} = 0.15L \leq 0.5H$  is the wall effective plastic hinge length;

$b_c$  is the effective thickness of wall boundary element within plastic hinge region;

$\beta_w$  is the wall effective width factor; 0.8 for doubly and 0.5 for singly reinforced walls;

$\xi_c = 0.5 + 1.18m - \sqrt{1.38m^2 + 1.18m}$  is the normalised critical out-of-plane displacement;

$m = (\rho_l f_y) / f'_c$  is the mechanical reinforcement ratio of the wall end region;

$\rho_l$  is the longitudinal reinforcement ratio in the end region of the wall;

$L_o = 0.2H + 0.044L \leq 0.8h_n$  Is the critical buckling length of wall;

$h_n$  is the clear height to the next floor above the critical section.

## 8. EXAMPLES

### 8.1. Slender Wall with Flexural Failure

The first example is a flexurally governed slender wall with boundary elements. The example is based on the specimen WSH3 from the experimental study by Dazio, Beyer and Bachman (2009).

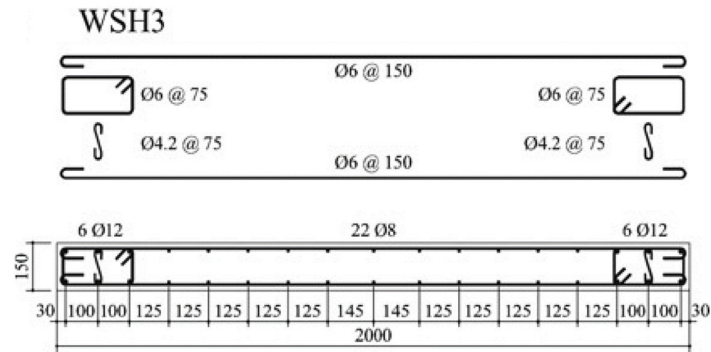


Figure 2. Details of the specimen WSH3 (Dazio et al. 2009)

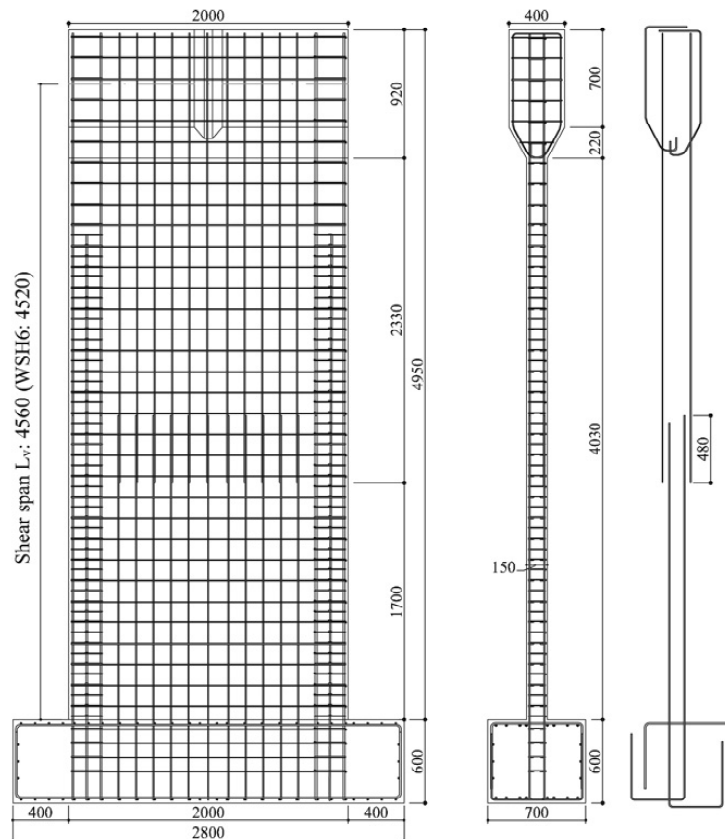


Figure 3. Elevation view of the specimen WSH3 (Dazio et al. 2009)

Other key parameters of the specimen WSH3 are presented below:

$$clb = 24$$

Axial load = 686 kN (compression)

$$f_{pc} = 39.2 \text{ MPa}$$

$$E_c = 35200 \text{ MPa}$$

$$esm = 0.06$$

$$f_y = 601 \text{ MPa}$$

$$f_{yh} = 489 \text{ MPa}$$

$$f_{ys} = 489 \text{ MPa}$$

$$f_{su} = 725.5 \text{ MPa}$$

$$esh = 0.008$$

$$esu = 0.0769$$

```

1
2 %=====
3
4 %
5         CUMBIA WALLS
6
7 %
8         SECTION AND MEMBER RESPONSE OF RC WALLS
9
10 %
11         luis.montejo@upr.edu, arsalan_niroomandi@yahoo.co.nz
12
13 %=====
14
15 clc; clear; close all; format long
16
17 % input data:
18 name = 'WSH3';           %identifies actual work, the output file will be name.xls
19 interaction = 'y';       % if you want to also perform an axial load - moment interaction
20                           % analysis type 'y', otherwise type 'n'
21
22 % section properties:
23
24 H = 2000;                 % wall length (mm)- perp to x
25 B = 150;                 % wall thickness (mm)- perp to y
26 clb = 24;                % cover to longitudinal bars (mm)
27
28 % member properties
29
30 L = 4560;                % Wall height (mm)
31                           % effective height for single bending wall (usually 0.7 times the full height)
32                           %full height for double bending walls
33 hn = 4560;               % clear height to the next floor above the critical section
34
35 bending = 'single';      % single or double
36 reinforcement = 'doubly'; % singly or doubly
37
38 % longitudinal reinforcement details, MLR is a matrix composed by
39 % [distance from the top to bar center (mm) - # of bars - bar diameter (mm)] each row
40 % corresponds to a layer of reinforcement:
41
42 MLR=[30    2   12
43      130   2   12
44      230   2   12
45      355   2    8
46      480   2    8
47      605   2    8
48      730   2    8
49      855   2    8
50      1000  2    8
51      1145  2    8
52      1270  2    8
53      1395  2    8
54      1520  2    8
55      1645  2    8
56      1770  2   12
57      1870  2   12
58      1970  2   12];
59
60 % Boundary element details (reinforcement must be symmetric, as the code is)
61
62 firstRowBoundary = 3;
63 HBZ = MLR(firstRowBoundary, 1); % boundary Elements length (mm) (from extreme compression
64                                % to the centre of the last confined longitudinal bar)
65 nBZ = sum(MLR(1:firstRowBoundary, 2)); % # bars in the boundary elements
66 db = mean(MLR(1:firstRowBoundary, 3)); % longitudinal bar diameter (mm)
67

```

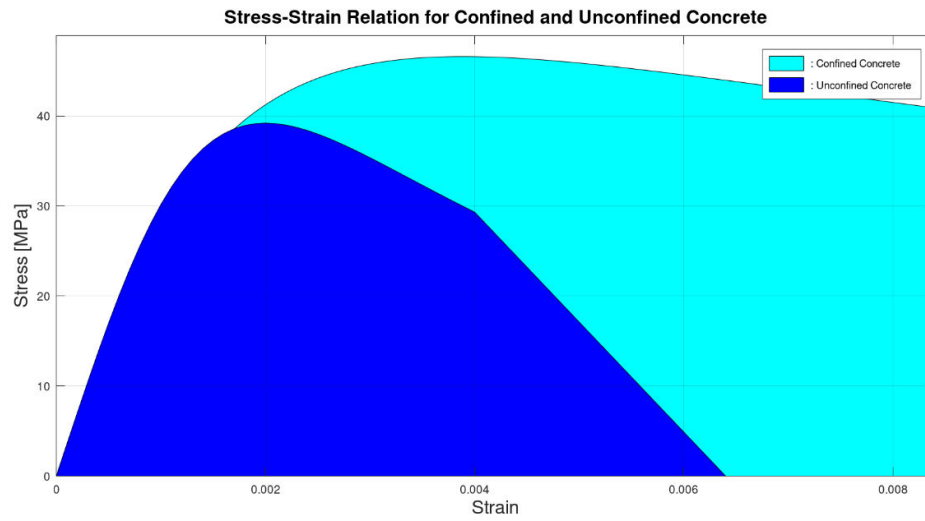
**Figure 4. Input data for Example 1**

```

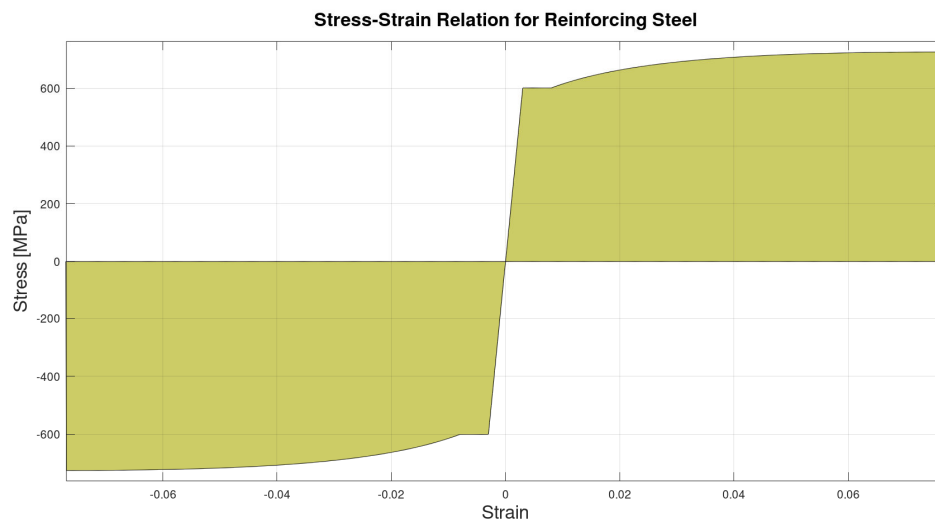
68 ncx = 2; % # legs transv. steel x_dir
69 ncy = 3; % # legs transv. steel y_dir
70 Dh = 5.47; % transverse reinforcement diameter (mm)
71 sh = 75; % spacing of transverse reinforcement (mm)
72
73 % Shear reinforcement details
74
75 Ds = 6; % diameter of shear reinf. (mm)
76 s = 150; % spacing of shear steel (mm)*
77 ns = 2; % # legs shear reinforcement
78
79 % applied loads:
80
81 P = 686; % axial load kN (-) tension (+)compression
82
83 % material models (input the 'name' of the file with the stress-strain relationship
84 % to use the default models: Mander model for confined or unconfined concrete type 'mc' or 'mu'.
85 % For lightweight confined concrete type 'mclw'
86 % King model for the steel 'ks', Raynor model for steel 'ra':
87
88 confined = 'mc'; %mc=mu if no transverse reinforcement
89 unconfined = 'mu';
90 rebar = 'ks';
91
92
93 wi = [88 88 88 88 78 78]; % vector with clear distances between
94 % periferical longitudinal bars properly
95 % restrained
96
97 % material properties
98 fpc = 39.2; % concrete compressive strength (MPa)
99 Ec = 35200; % concrete modulus of elasticity (MPa) or
100 % input 0 for automatic calculation using
101 % 4700(fpc)^0.5
102 eco = 0.002; % unconfined strain (usually 0.002 for normal weight or 0.004 for lightweight)*
103 esm = 0.06; % max transv. steel strain (<0.06)*
104 espall = 0.0064; % max uncon. conc. strain (usually 0.0064)
105
106 fy = 601; % long steel yielding stress (MPa)
107 fyh = 489; % transverse steel yielding stress (MPa)
108 fys = 489; % shear steel yielding stress (MPa)
109 Es = 200000; % steel modulus of elasticity
110 fsu = 725.5; % long steel max stress (MPa)*
111 esh = 0.008; % long steel strain for strain hardening (usually 0.008)*
112 esu = 0.0769; % long. steel maximum strain (usually ~0.10-0.15)*
113
114 Ey = 350; % slope of the yield plateau (MPa)
115 C1 = 3.5; % defines strain hardening curve in the Raynor model [2-6]
116
117 % *this information is used only if the default material models are selected
118
119 % strain limits for yield surface (interaction diagram);
120
121 csid = 0.004; % concrete
122 ssid = 0.015; % steel
123
124 % Deformation Limit States:
125
126 ecser = 0.004; esser = 0.015; % concrete (ecser) and steel (esser) serviceability strain
127 ecdam = 'twth'; esdam = 0.060; % concrete (ecser) and steel (esser) damage control strain
128 % (to use the 2/3 of the ultimate
129 % concrete strain just tipe 'twth')
130
131 % temperature information (in case of freezing conditions)
132 temp = 30; % temperature of the specimen in celsius
133 kLsp = 0.022; % constant to calculate Lsp = kLsp*fy*Db1
134 % (usually 0.022 at ambient temp. or 0.011 at -40C)
135
136 % Shear strength inforamtion
137 phiS = 0.85; % Strength reduction factor for shear

```

**Figure 5. Input data for Example 1 (continued)**

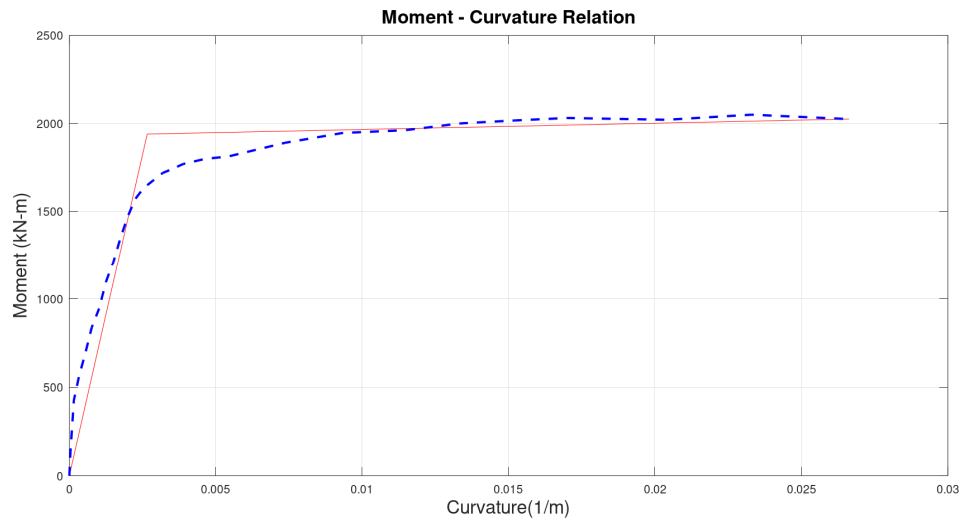


**Figure 6. Concrete Model (Example 1)**

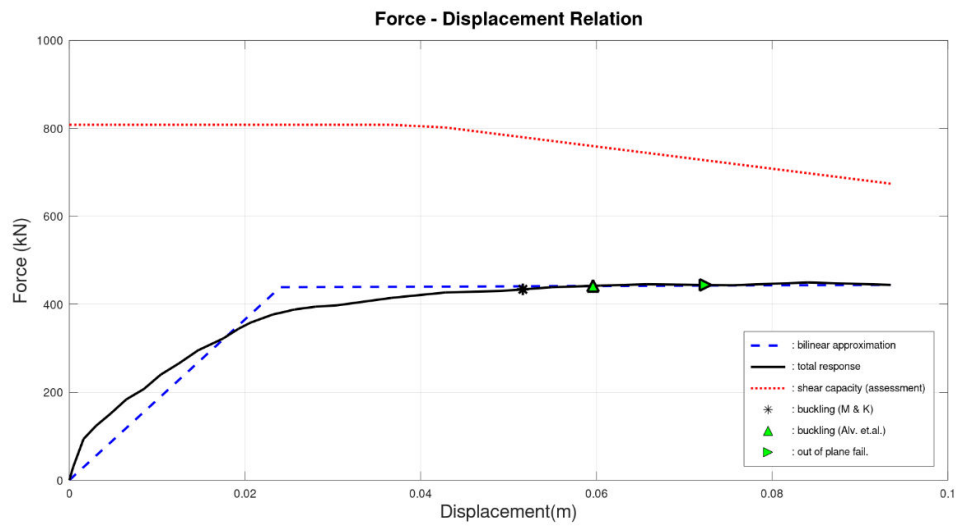


**Figure 7. Steel Reinforcement Model (Example 1)**

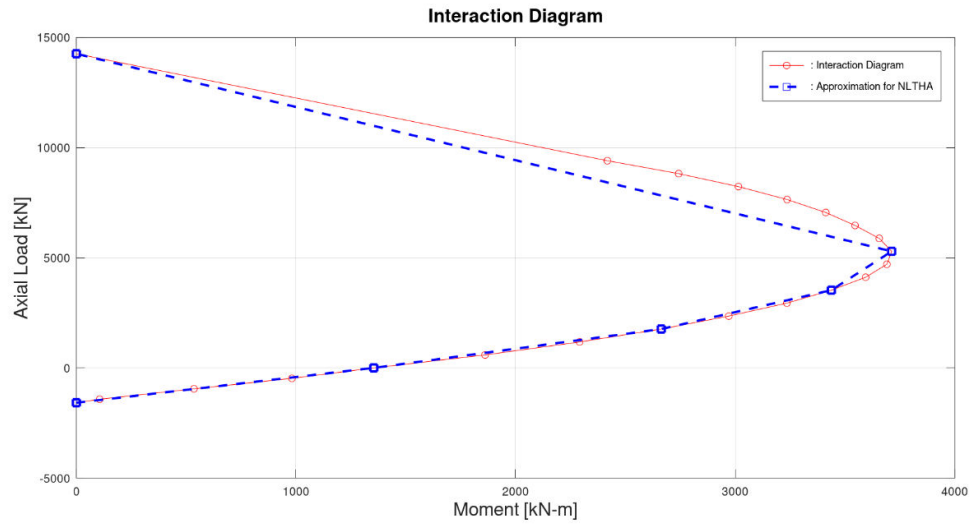




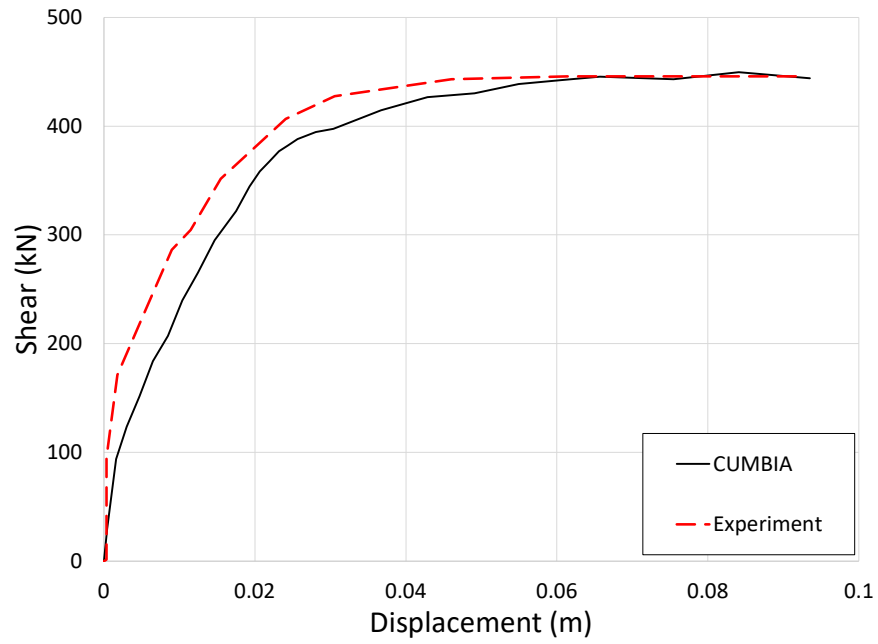
**Figure 8. Moment – Curvature Relation (Example 1)**



**Figure 9. Member Response (Example 1)**



**Figure 10. Interaction Diagram (Example 1)**



**Figure 11. Force-Displacement Curve of the specimen WSH3 (CUMBIA vs Experiment)**

## 8.2. Squat Wall with Shear Failure

For the second example a squat wall with a shear failure was chosen. The example is based on the specimen Wall 1 from the experimental study by Paulay, Priestley and Synge (1982).

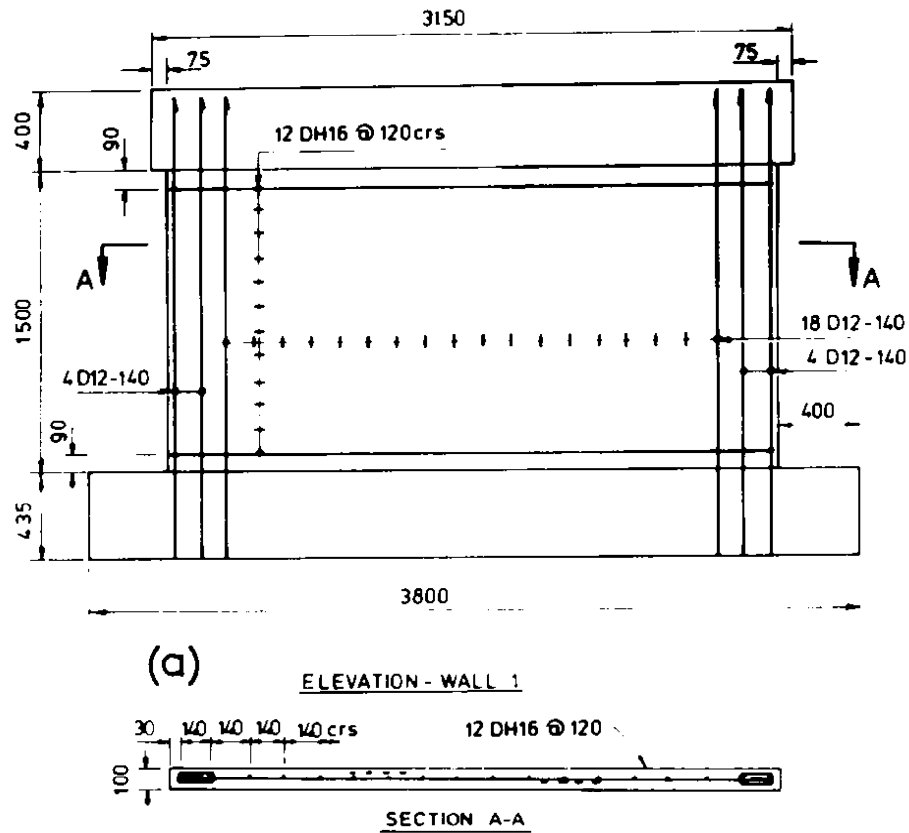


Figure 12. Arrangement of reinforcement of specimen Wall 1 (Paulay et al. 1982)

Other key parameters of the specimen Wall 1 are presented below:

$$B = 100 \text{ mm}$$

$$clb = 24$$

$$ncx = 1$$

$$ncy = 1$$

$$Dh = 6 \text{ mm}$$

$$Sh = 50 \text{ mm}$$

$$\text{Axial load} = 0 \text{ kN}$$

$$f_{pc} = 27.2 \text{ MPa}$$

$$esm = 0.04$$

$$f_y = 300 \text{ MPa}$$

$$f_{yh} = 280 \text{ MPa}$$

$$f_{ys} = 380 \text{ MPa}$$

$$f_{su} = 400 \text{ MPa}$$

$$esh = 0.008$$

$$esu = 0.1$$

```

1
2 %=====
3
4 %
5
6 %
7
8 %
9
10
11 %=====
12
13 clc; clear; close all; format long
14
15 % input data:
16
17 name = 'Wall 1'; %identifies actual work, the output file will be name.xls
18 interaction = 'y'; % if you want to also perform an axial load - moment interaction
19 % analysis type 'y', otherwise type 'n'
20
21
22 % section properties:
23
24 H = 3000; % wall length (mm)- perp to x
25 B = 100; % wall thickness (mm)- perp to y
26 clb = 24; % cover to longitudinal bars (mm)
27
28 % member properties
29
30 L = 1500; % Wall height (mm)
31 % effective height for single bending wall (usually 0.7 times the full height)
32 %full height for double bending walls
33 hn = 1500; % clear height to the next floor above the critical section
34
35 bending = 'single'; % single or double
36 reinforcement = 'singly'; % singly or doubly
37
38 % longitudinal reinforcement details, MLR is a matrix composed by
39 % [distance from the top to bar center (mm) - # of bars - bar diameter (mm)] each row
40 % corresponds to a layer of reinforcement:
41
42 MLR=[30 2 12
43 170 2 12
44 310 1 12
45 450 1 12
46 590 1 12
47 730 1 12
48 870 1 12
49 1010 1 12
50 1150 1 12
51 1290 1 12
52 1430 1 12
53 1570 1 12
54 1710 1 12
55 1850 1 12
56 1990 1 12
57 2130 1 12
58 2270 1 12
59 2410 1 12
60 2550 1 12
61 2690 1 12
62 2830 2 12
63 2970 2 12];
64
65 % Boundary element details (reinforcement must be symmetric, as the code is)
66
67 firstRowBoundary = 2;
68 HBZ = MLR(firstRowBoundary, 1); % boundary Elements length (mm) (from extreme compression
69 % to the centre of the last confined longitudinal bar)
70 nBZ = sum(MLR(1:firstRowBoundary, 2)); % # bars in the boundary elements
71 db = mean(MLR(1:firstRowBoundary, 3)); % longitudinal bar diameter (mm)
72

```

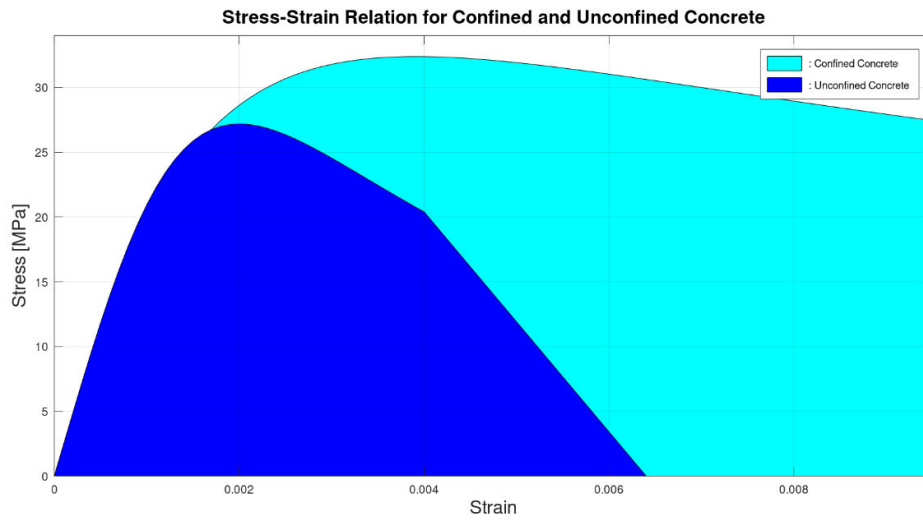
**Figure 13. Input data for Example 2**

```

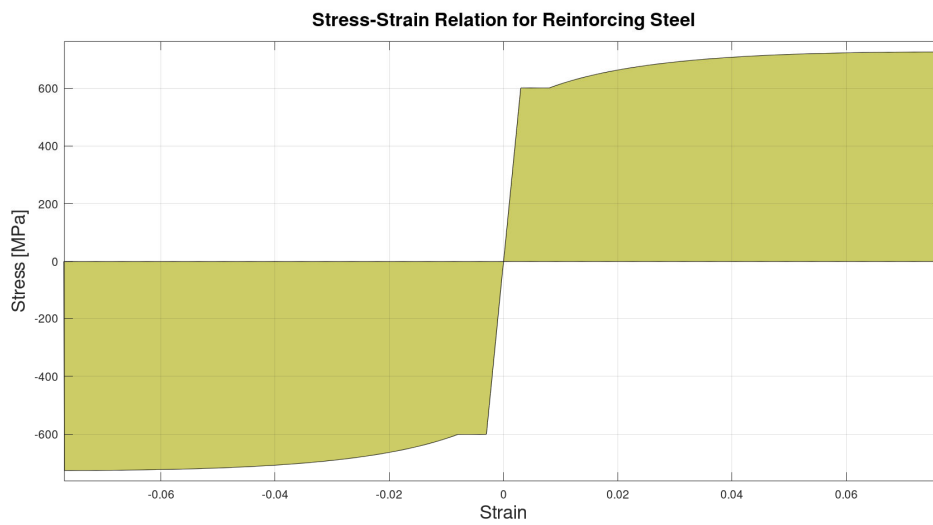
73 ncx = 2; % # legs transv. steel x_dir
74 ncy = 2; % # legs transv. steel y_dir
75 Dh = 6; % transverse reinforcement diameter (mm)
76 sh = 50; % spacing of transverse reinforcement (mm)
77
78 % Shear reinforcement details
79
80 Ds = 16; % diameter of shear reinf. (mm)
81 s = 120; % spacing of shear steel (mm)*
82 ns = 1; % # legs shear reinforcement
83
84 % applied loads:
85
86 P = 0; % axial load kN (-) tension (+)compression
87
88 % material models (input the 'name' of the file with the stress-strain relationship
89 % to use the default models: Mander model for confined or unconfined concrete type 'mc' or 'mu'.
90 % For lightweight confined concrete type 'mclw'
91 % King model for the steel 'ks', Raynor model for steel 'ra':
92
93 confined = 'mc'; %mc=mu if no transverse reinforcement
94 unconfined = 'mu';
95 rebar = 'ks';
96
97
98 wi = [128 128]; % vector with clear distances between
99 % periferical longitudinal bars properly
100 % restrained
101 % material properties
102
103 fpc = 27.2; % concrete compressive strength (MPa)
104 Ec = 0; % concrete modulus of elasticity (MPa) or
105 % input 0 for automatic calculation using
106 % 4700(fpc)^0.5
107 eco = 0.002; % unconfined strain (usually 0.002 for normal weight or 0.004 for lightweight)*
108 esm = 0.04; % max transv. steel strain (<0.06)*
109 espall = 0.0064; % max uncon. conc. strain (usually 0.0064)
110
111 fy = 300; % long steel yielding stress (MPa)
112 fyh = 280; % transverse steel yielding stress (MPa)
113 fys = 380; % shear steel yielding stress (MPa)
114 Es = 200000; % steel modulus of elasticity
115 fsu = 400; % long steel max stress (MPa)*
116 esh = 0.008; % long steel strain for strain hardening (usually 0.008)*
117 esu = 0.1; % long. steel maximum strain (usually ~0.10-0.15)*
118
119 Ey = 350; % slope of the yield plateau (MPa)
120 Cl = 3.5; % defines strain hardening curve in the Raynor model [2-6]
121
122 % *this information is used only if the default matrial models are selected
123
124 % strain limits for yield surface (interaction diagram);
125
126 csid = 0.004; % concrete
127 ssid = 0.015; % steel
128
129 % Deformation Limit States:
130
131 ecser = 0.004; esser = 0.015; % concrete (ecser) and steel (esser) serviceability strain
132 ecdam = 'twth'; esdam = 0.060; % concrete (ecser) and steel (esser) damage control strain
133 % (to use the 2/3 of the ultimate
134 % concrete strain just tipe 'twth')
135
136 % temperature information (in case of freezing conditions)
137 temp = 30; % temperature of the specimen in celsius
138 kLsp = 0.022; % constant to calculate Lsp = kLsp*fy*Db1
139 % (usually 0.022 at ambient temp. or 0.011 at -40C)
140
141 % Shear strength inforamtion
142 phis = 0.85; % Strength reduction factor for shear

```

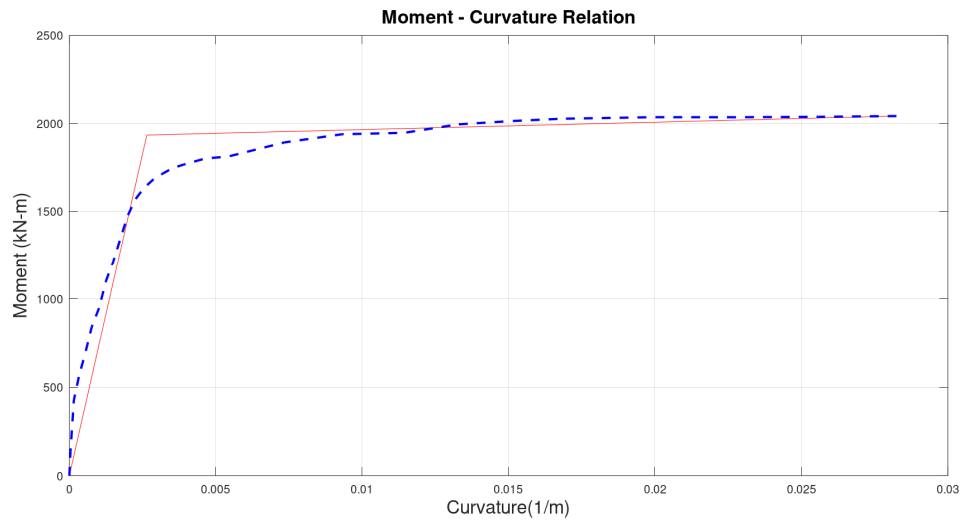
**Figure 14. Input data for Example 2 (continued)**



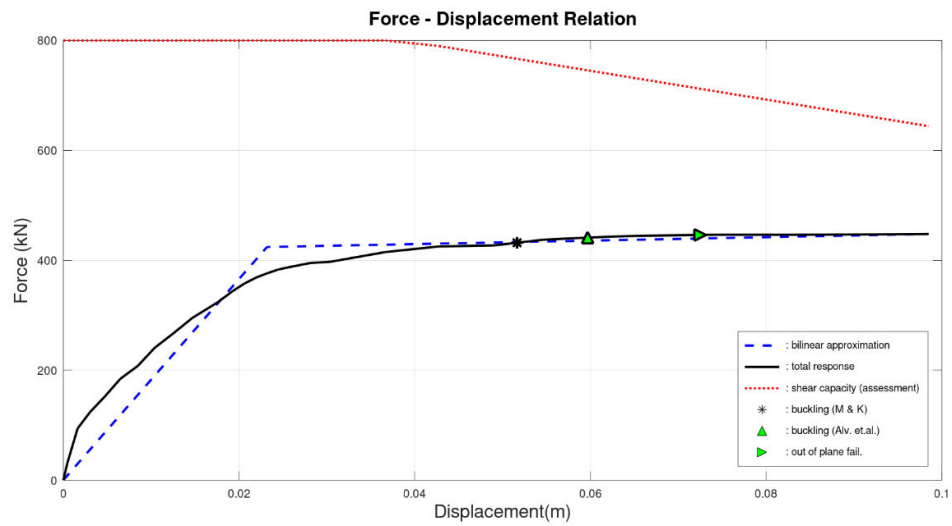
**Figure 15. Concrete Model (Example 2)**



**Figure 16. Steel Reinforcement Model (Example 2)**

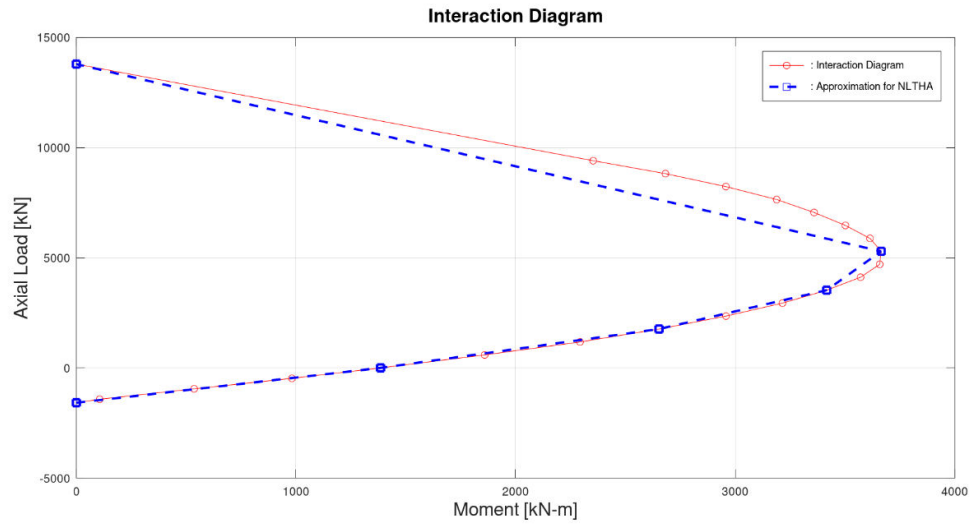


**Figure 17. Moment – Curvature Relation (Example 2)**

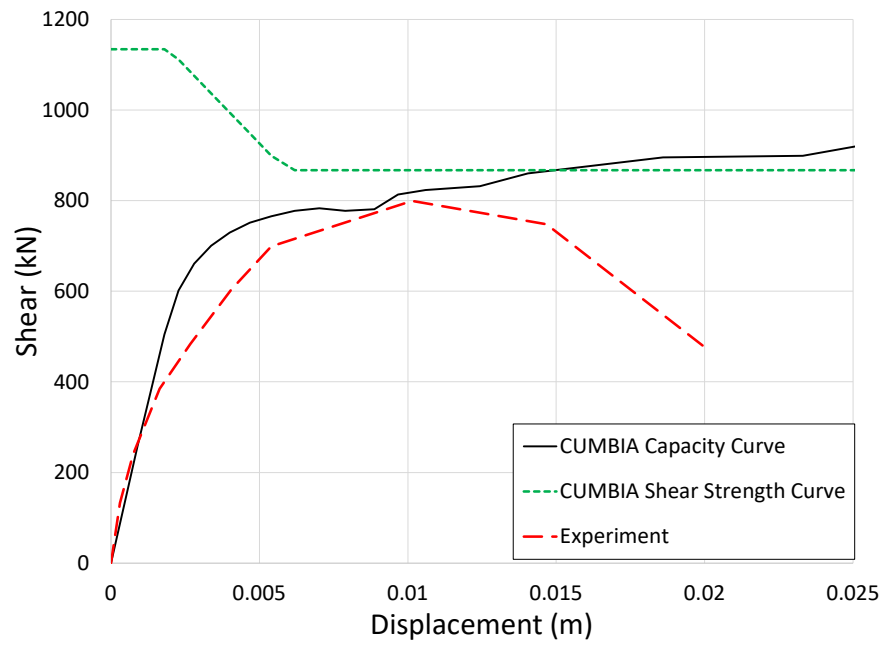


**Figure 18. Member Response (Example 2)**





**Figure 19. Interaction Diagram (Example 2)**



**Figure 20. Force-Displacement Curve of the specimen Wall 1 (CUMBIA vs Experiment)**

## 9. REFERENCES

- Alvarado, I., Rodriguez, E. and Restrepo, I. (2015). Resistencia a flexocompresión y capacidad de deformación lateral de muros rectangulares de concreto reforzado en zonas sísmicas. *Congreso Nacional de Ingeniería Sísmica*.
- Berry, M. P. and Eberhard, M. O. (2005). "Practical performance model for bar buckling." *Journal of Structural Engineering* **131**(7): 1060-1070.
- Dazio, A., Beyer, K. and Bachmann, H. (2009). "Quasi-static cyclic tests and plastic hinge analysis of RC structural walls." *Engineering Structures* **31**(7): 1556-1571.
- FIB (2003). "Displacement-based seismic design of reinforced concrete buildings." *Federación Internacional para Concreto Estructural* **25**.
- King, D. J., Priestley, M. J. N. and Park, R. (1986). computer programs for concrete column design. *University of Canterbury*.
- Kowalsky, M. J. (2000). "Deformation limit states for circular reinforced concrete bridge columns." *Journal of Structural Engineering* **126**(8): 869-878.
- Kowalsky, M. J. and Priestley, M. N. (2000). "Improved analytical model for shear strength of circular reinforced concrete columns in seismic regions." *Structural Journal* **97**(3): 388-396.
- Krolicki, J., Maffei, J. and Calvi, G. M. (2011). "Shear strength of reinforced concrete walls subjected to cyclic loading." *Journal of Earthquake Engineering* **15**(S1): 30-71.
- Mander, J. B., Priestley, M. J. N. and Park, R. (1988). "Theoretical stress-strain model for confined concrete." *Journal of structural engineering* **114**(8): 1804-1826.
- Montejo, L. A., & Kowalsky, M. J. (2007). CUMBIA—Set of codes for the analysis of reinforced concrete members. *CFL technical rep. no. IS-07, 1*.
- Moyer, M. J. and Kowalsky, M. J. (2003). "Influence of tension strain on buckling of reinforcement in concrete columns." *ACI Structural journal* **100**(1): 75-85.
- Oliver, S., Boys, A. and Marriott, D. (2012). Nonlinear Analysis Acceptance Criteria for the Seismic Performance of Existing Reinforced Concrete Buildings. *Proc. NZSEE Annual Conf., 13-15 April 2012*.
- Paulay, T. and Priestley, M. J. N. (1992). *seismic design of reinforced concrete and masonry buildings*. New York, USA, John Wiley & Sons, Inc.
- Paulay, T. and Priestley, M. J. N. (1993). "Stability of ductile structural walls." *ACI Structural Journal* **90**(4).

Paulay, T., Priestley, M. J. N. and Syngé, J. (1982). "Ductility in earthquake resisting squat shearwalls." *ACI Journal* **79**(4): 257-269.

Priestley, M. J. N., Calvi, M. C. and Kowalsky, M. J. (2007). *Displacement-Based Seismic Design of Structures*. IUSS Press, Pavia, Italy.

Raynor, D. J., Lehman, D. E. and Stanton, J. F. (2002). "Bond-slip response of reinforcing bars grouted in ducts." *Structural journal* **99**(5): 568-576.