CUMBIA—Set of codes for the analysis of reinforced concrete members – Walls

Theory and User Guide

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DISCLAIMER

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

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1. INTRODUCTION

CUMBIA is a set of Matlab codes to perform monotonic moment-curvature analysis and force-displacement response of reinforced concrete (RC) members of rectangular or circular sections written by Luis Montejo. This version of CUMBIA is following the CUMBIARECT code and is specifically adjusted for RC walls. The parts of the code that is identical to the CUMBIARECT code was 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 Krolicki, 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).

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

In this equation, ε_{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).

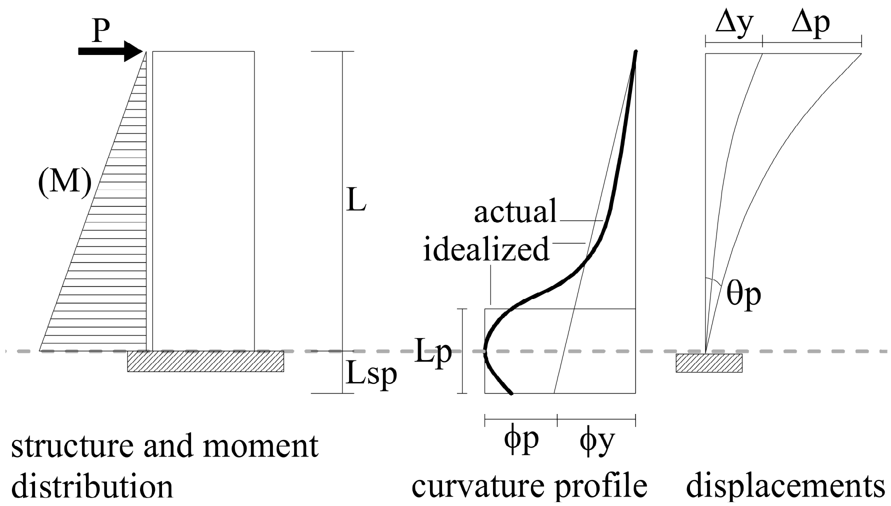

Figure 1. Plastic hinge method

$$L_p = kL_c + 0.1L_w + L_{sp} > 2L_{sp} \tag{2}$$

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

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.

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

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

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

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

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

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

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).

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

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).

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

$$k_{seff} = k_{sg}\frac{I_{eff}}{I_g} \tag{12}$$

$$k_{sg} = \frac{G_c A_s}{L} \tag{13}$$

$$I_{eff} = \frac{M'_y}{E_c \phi'_y} \tag{14}$$

$$A_s = \frac{5}{6}A_g \tag{15}$$

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

$$G_c = 0.43E_c \tag{17}$$

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.

$$\Delta_{scr} = \Delta_{s,l} + \frac{V - V_c}{k_{scr}}, \qquad V_c < V < V_y \tag{18}$$

$$k_{scr} = \frac{0.25\rho_y}{0.25 + 10\rho_y}E_s Bd \tag{19}$$

$$V_y = \frac{M_y}{L} \tag{20}$$

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).

$$\Delta_s = \Delta_f\frac{\Delta_{s,y}}{\Delta_{f,y}}, \qquad V > V_y \tag{21}$$

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

5. SHEAR CAPACITY

The shear strength envelope for the member is calculated using the shear model proposed by Krolicki, 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.

$$V = 0.85(V_c + V_s + V_p) \tag{23}$$

$$V_c = \alpha\beta\gamma\sqrt{f_c'}0.8A_g \tag{24}$$

$$1 \leq \alpha = 3 - \frac{L}{H} \leq 1.5 \tag{25}$$

$$\beta = 0.5 + 20\rho_l \leq 1 \tag{26}$$

$$0.05 \leq \gamma = 0.41 - 0.06\mu_\Delta \leq 0.29 \tag{27}$$

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

$$V_s = \frac{A_v f_{yh} h_{cr}}{s} \tag{28}$$

$$h_{cr} = \frac{l'}{\tan\theta_{cr}} \leq L \tag{29}$$

$$l' = H - c - c_0 \tag{30}$$

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

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

$$V_p = P\frac{H - c}{2L} \qquad \text{Single bending} \tag{32}$$

$$V_p = P\frac{H - c}{2L} \qquad \text{double bending} \tag{33}$$

$$V_p = 0 \qquad for\ P < 0 \tag{34}$$

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).

$$\phi_{buckling} = \frac{\varepsilon_p^*}{\gamma H} \tag{35}$$

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

$$\gamma H = H - 2c_0 \tag{37}$$

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).

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

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;

β_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;

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

ρ_l is the longitudinal reinforcement ratio in the end region of the wall;

$L_0 = 0.2H + 0.044L \leq 0.8h_n$ 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).

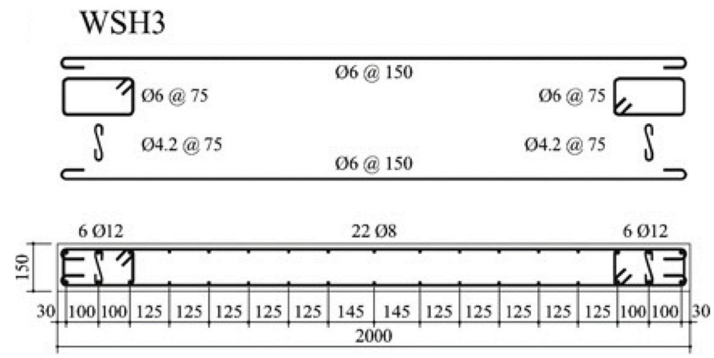

Figure 2. Details of the specimen WSH3 (Dazio et al. 2009)

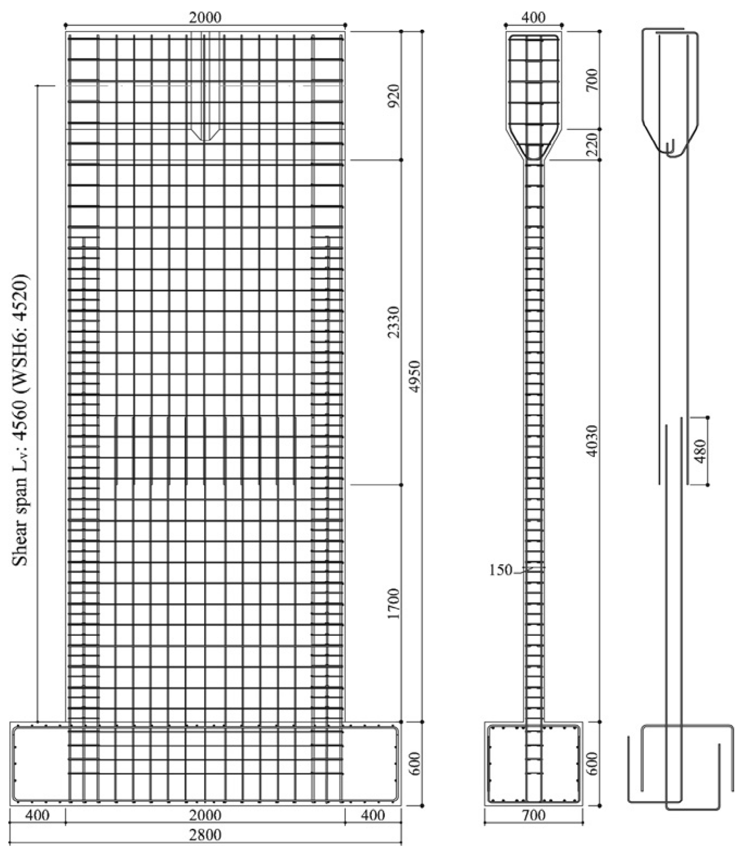

Figure 3. Elevation view of the specimen WSH3 (Dazio et al. 2009)

Other key parameters of the specimen WSH3 are presented below:

clb = 24

Axial load = 686 kN (compression)

fpc = 39.2 MPa

E_c = 35200 MPa

esm = 0.06

fy = 601 MPa

fyh = 489 MPa

fys = 489 MPa

fsu = 725.5 MPa

esh = 0.008

esu = 0.0769

```matlab

%==========================================================================================

%                                    CUMBIA WALLS

%                      SECTION AND MEMBER RESPONSE OF RC WALLS

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%==========================================================================================

clc; clear; close all; format long

% input data:

name = 'WSH3';      %identifies actual work, the output file will be name.xls
interaction = 'y';  % if you want to also perform an axial load - moment interaction
                    % analysis type 'y', otherwise type 'n'


% section properties:

H    = 2000;                              % wall length (mm)- perp to x
B    = 150;                               % wall thickness  (mm)- perp to y
clb  = 24;                                % cover to longitudinal bars (mm)

% member properties

L  = 4560;                       % Wall height (mm)
                                 % effective height for single bending wall (usually 0.7 times the full height
                                 %full height for double bending walls
hn = 4560;                       % clear height to the next floor above the critical section

bending = 'single';              % single or double
reinforcement = 'doubly';        % singly or doubly

% longitudinal reinforcement details, MLR is a matrix composed by
% [distance from the top to bar center (mm) - # of bars - bar diameter (mm)] each row
% corresponds to a layer of reinforcement:

MLR=[30    2  12
     130   2  12
     230   2  12
     355   2  8
     480   2  8
     605   2  8
     730   2  8
     855   2  8
     1000  2  8
     1145  2  8
     1270  2  8
     1395  2  8
     1520  2  8
     1645  2  8
     1770  2  12
     1870  2  12
     1970  2  12];

% Boundary element details (reinforcement must be symmetric, as the code is)

firstRowBoundary = 3;
HBZ  = MLR(firstRowBoundary, 1);          % boundary Elements length (mm) (from extreme compression
                                          % to the centre of the last confined longitudinal bar)
nBZ  = sum(MLR(1:firstRowBoundary, 2));   % # bars in the boundary elements
db   = mean(MLR(1:firstRowBoundary, 3));  % longitudinal bar diameter (mm)

```

Figure 4. Input data for Example 1

```
ncx  = 2;                                % # legs transv. steel x_dir
ncy  = 3;                                % # legs transv. steel y_dir
Dh   = 5.47;                               % transverse reinforcement diameter (mm)
sh   = 75;                              % spacing of transverse reinforcement (mm)

% Shear reinforcement details

Ds      = 6;                                 % diameter of shear reinf. (mm)
s       = 150;                                % spacing of shear steel (mm)*
ns      = 2;                                  % # legs shear reinforcement

% applieed loads:

P      = 686;               % axial load kN (-) tension (+)compression

% material models (input the 'name' of the file with the stress-strain relationship
% to use the default models: Mander model for confined or unconfined  concrete type 'mc' or 'mu'.
% For lightweight confined concrete type 'mclw'
% King model for the steel 'ks', Raynor model for steel 'ra':

confined   = 'mc';              %mc=mu if no transverse reinforcement
unconfined = 'mu';
rebar      = 'ks';


wi   = [88 88 88 88 78 78];                                   % vector with clear distances between
                                                  % periferical longitudinal bars properly
                                                  % restrained
% material properties

fpc     = 39.2;              % concrete compressive strength (MPa)
Ec      = 35200;                 % concrete modulus of elasticity (MPa) or
                             % input 0 for automatic calculation using
                             % 4700(fpc)^0.5
eco     = 0.002;             % unconfined strain (usually 0.002 for normal weight or 0.004 for lightweight)*
esm     = 0.06;              % max transv. steel strain (<0.06)*
espall  = 0.0064;            % max uncon. conc. strain (usually 0.0064)

fy      = 601;               % long steel yielding stress (MPa)
fyh     = 489;               % transverse steel yielding stress (MPa)
fys     = 489;               % shear steel yielding stress (MPa)
Es      = 200000;            % steel modulus of elasticity
fsu     = 725.5;             % long steel max stress (MPa)*
esh     = 0.008;             % long steel strain for strain hardening (usually 0.008)*
esu     = 0.0769;            % long. steel maximum strain (usually ~0.10-0.15)*

Ey     =  350;                % slope of the yield plateau (MPa)
C1     =  3.5;                % defines strain hardening curve in the Raynor model [2-6]

% *this information is used only if the default matrial models are selected

% strain limits for yield surface (interaction diagram);

csid = 0.004;  % concrete
ssid = 0.015;  % steel

% Deformation Limit States:

ecser = 0.004;    esser = 0.015;   % concrete (ecser) and steel (esser) serviceability strain
ecdam = 'twth';   esdam = 0.060;     % concrete (ecser) and steel (esser) damage control strain
                                   % (to use the 2/3 of the ultimate
                                   % concrete strain just tipe 'twth')

% temperature information (in case of freezing conditions)
temp  = 30;           % temperature of the specimen in celsius
kLsp  = 0.022;        % constant to calculate Lsp = kLsp*fy*Dbl
                      % (usually 0.022 at ambient temp. or 0.011 at -40C)

% Shear strength inforamtion
phiS    = 0.85;         % Strength reduction factor for shear
```

Figure 5. Input data for Example 1 (continued)

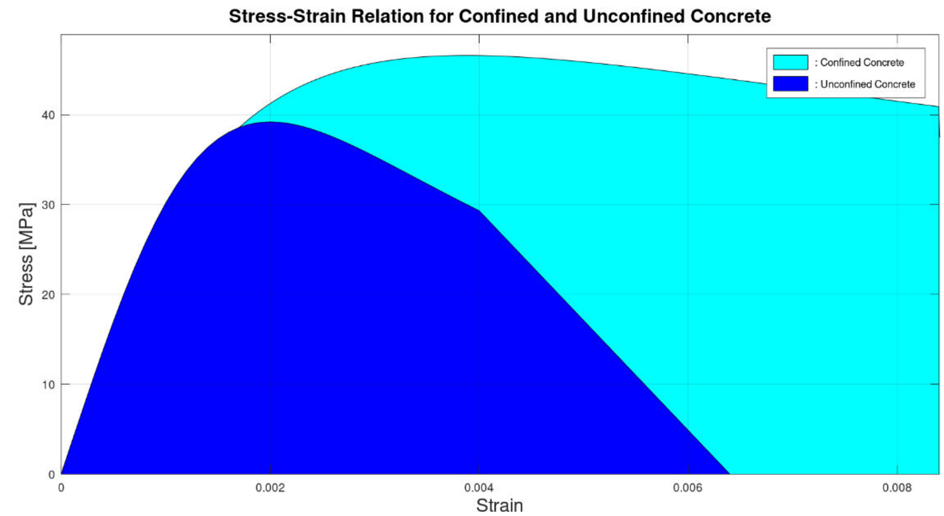

Figure 6. Concrete Model (Example 1)

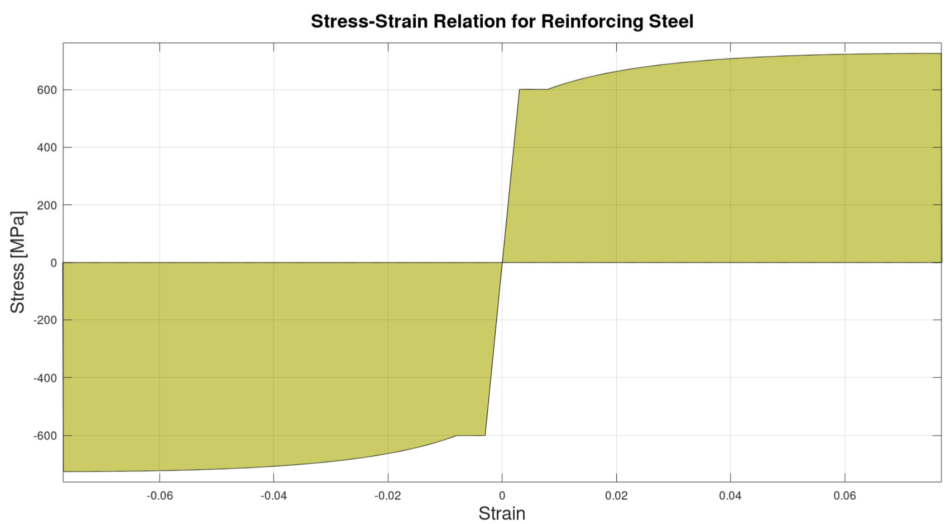

Figure 7. Steel Reinforcement Model (Example 1)

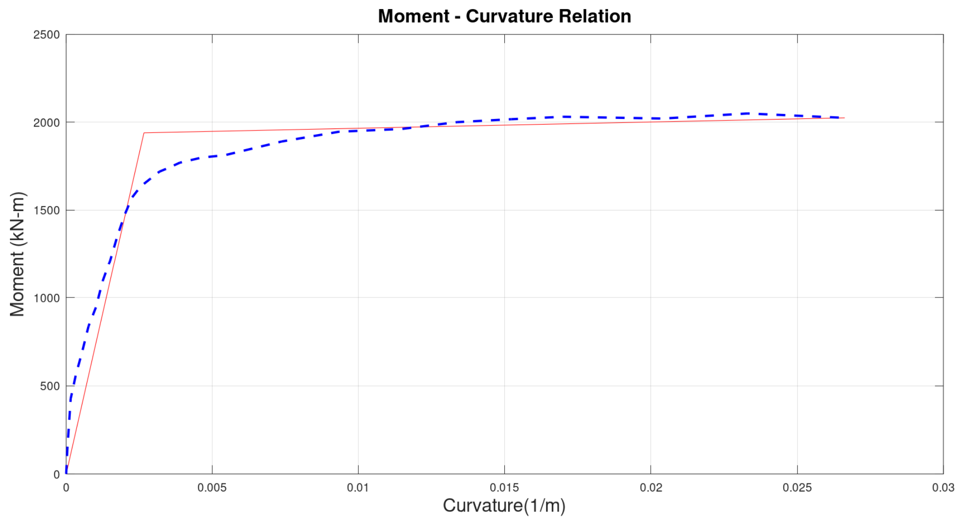

Figure 8. Moment – Curvature Relation (Example 1)

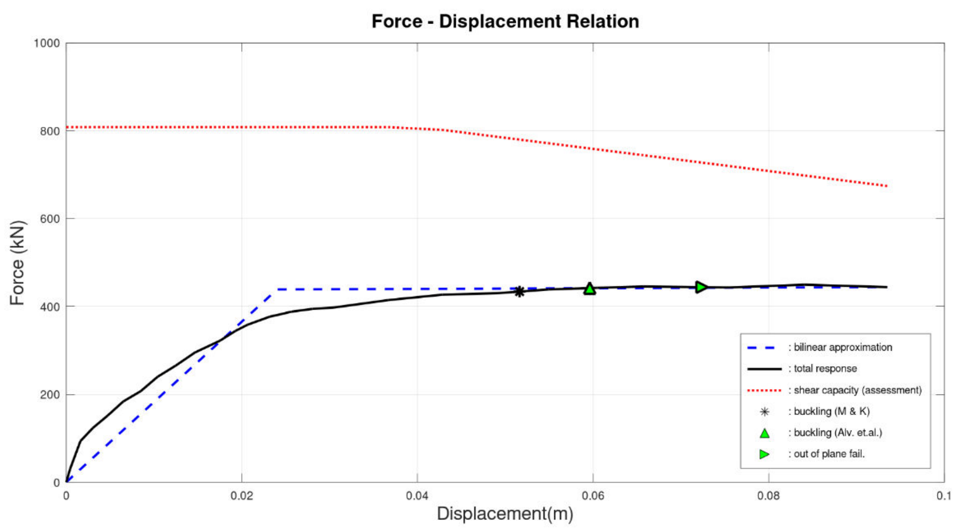

Figure 9. Member Response (Example 1)

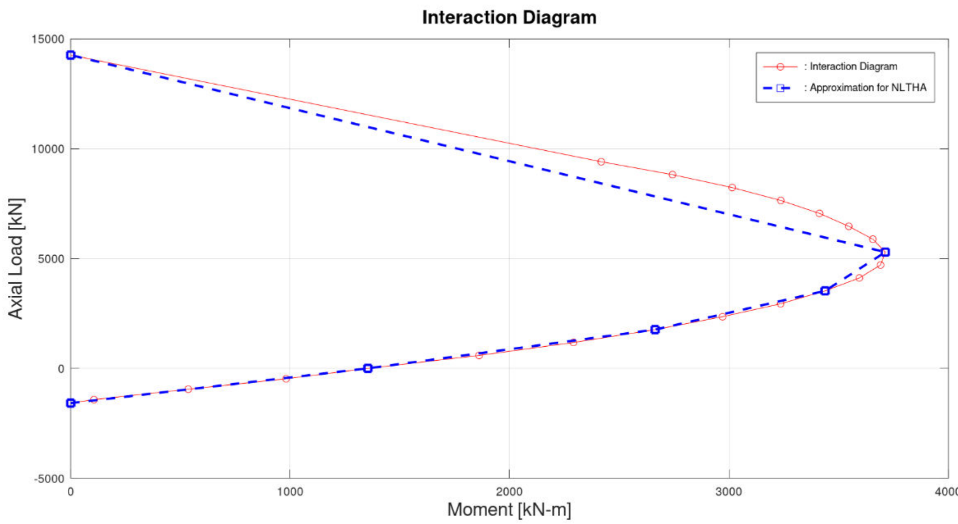

Figure 10. Interaction Diagram (Example 1)

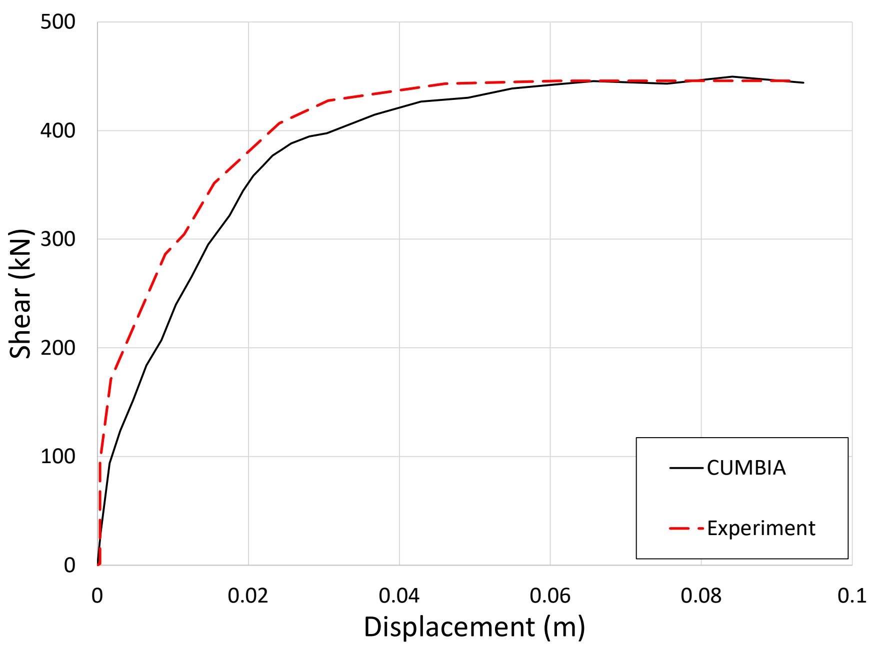

Figure 11. Force-Displacement Curve of the specimen WSH3 (CUMBIA vs Experiment)

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).

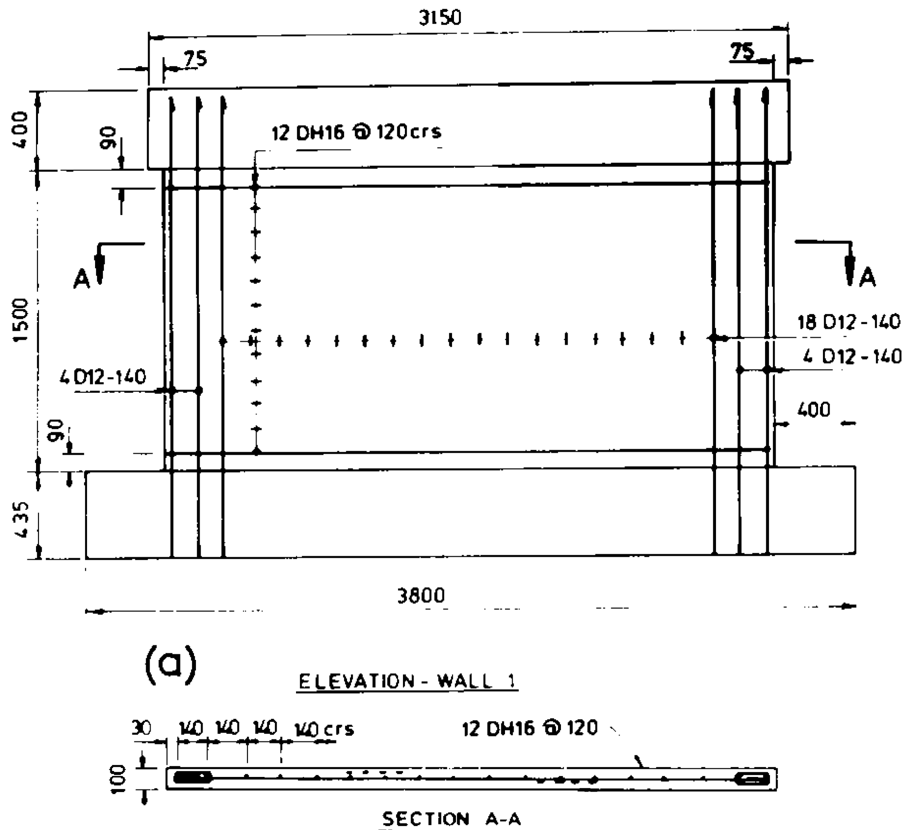

Figure 12. Arrangement of reinforcement of specimen Wall 1 (Paulay et al. 1982)

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

B = 100 mm

clb = 24

ncx = 1

ncy = 1

Dh = 6 mm

Sh = 50 mm

Axial load = 0 kN

fpc = 27.2 MPa

esm = 0.04

fy = 300 MPa

fyh = 280 MPa

fys = 380 MPa

fsu = 400 MPa

esh = 0.008

esu = 0.1

```
%=========================================================================================

%                                    CUMBIA WALLS

%                      SECTION AND MEMBER RESPONSE OF RC WALLS

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%=========================================================================================

clc; clear; close all; format long

% input data:

name = 'Wall 1';      %identifies actual work, the output file will be name.xls
interaction = 'y';  % if you want to also perform an axial load - moment interaction
                    % analysis type 'y', otherwise type 'n'


% section properties:

H    = 3000;                                 % wall length (mm)- perp to x
B    = 100;                                  % wall thickness  (mm)- perp to y
clb  = 24;                                   % cover to longitudinal bars (mm)

% member properties

L  = 1500;                        % Wall height (mm)
                                  % effective height for single bending wall (usually 0.7 times the full height
                                  %full height for double bending walls
hn = 1500;                        % clear height to the next floor above the critical section

bending = 'single';               % single or double
reinforcement = 'singly';         % singly or doubly

% longitudinal reinforcement details, MLR is a matrix composed by
% [distance from the top to bar center (mm) - # of bars - bar diameter (mm)] each row
% corresponds to a layer of reinforcement:

MLR=[30     2  12
     170    2  12
     310    1  12
     450    1  12
     590    1  12
     730    1  12
     870    1  12
     1010   1  12
     1150   1  12
     1290   1  12
     1430   1  12
     1570   1  12
     1710   1  12
     1850   1  12
     1990   1  12
     2130   1  12
     2270   1  12
     2410   1  12
     2550   1  12
     2690   1  12
     2830   2  12
     2970   2  12];

% Boundary element details (reinforcement must be symmetric, as the code is)

firstRowBoundary = 2;
HBZ  = MLR(firstRowBoundary, 1);          % boundary Elements length (mm) (from extreme compression
                                          % to the centre of the last confined longitudinal bar)
nBZ  = sum(MLR(1:firstRowBoundary, 2));   % # bars in the boundary elements
db   = mean(MLR(1:firstRowBoundary, 3));  % longitudinal bar diameter (mm)

```

Figure 13. Input data for Example 2

```
ncx  = 2;                                   % # legs transv. steel x_dir
ncy  = 2;                                   % # legs transv. steel y_dir
Dh   = 6;                                  % transverse reinforcement diameter (mm)
sh   = 50;                                 % spacing of transverse reinforcement (mm)

% Shear reinforcement details

Ds      = 16;                                    % diameter of shear reinf. (mm)
s       = 120;                                   % spacing of shear steel (mm)*
ns      = 1;                                     % # legs shear reinforcement

% applieed loads:

P      = 0;               % axial load kN (-) tension (+)compression

% material models (input the 'name' of the file with the stress-strain relationship
% to use the default models: Mander model for confined or unconfined  concrete type 'mc' or 'mu'.
% For lightweight confined concrete type 'mclw'
% King model for the steel 'ks', Raynor model for steel 'ra':

confined   = 'mc';              %mc=mu if no transverse reinforcement
unconfined = 'mu';
rebar      = 'ks';


wi   = [128 128];                                      % vector with clear distances between
                                                   % periferical longitudinal bars properly
                                                   % restrained
% material properties

fpc     = 27.2;              % concrete compressive strength (MPa)
Ec      = 0;                 % concrete modulus of elasticity (MPa) or
                             % input 0 for automatic calculation using
                             % 4700(fpc)^0.5
eco     = 0.002;             % unconfined strain (usually 0.002 for normal weight or 0.004 for lightweight)*
esm     = 0.04;              % max transv. steel strain (<0.06)*
espall  = 0.0064;            % max uncon. conc. strain (usually 0.0064)

fy      = 300;               % long steel yielding stress (MPa)
fyh     = 280;               % transverse steel yielding stress (MPa)
fys     = 380;               % shear steel yielding stress (MPa)
Es      = 200000;            % steel modulus of elasticity
fsu     = 400;             % long steel max stress (MPa)*
esh     = 0.008;             % long steel strain for strain hardening (usually 0.008)*
esu     = 0.1;            % long. steel maximum strain (usually ~0.10-0.15)*

Ey     =  350;                % slope of the yield plateau (MPa)
C1     =  3.5;                % defines strain hardening curve in the Raynor model [2-6]

% *this information is used only if the default matrial models are selected

% strain limits for yield surface (interaction diagram);

csid = 0.004;  % concrete
ssid = 0.015;  % steel

% Deformation Limit States:

ecser = 0.004;    esser = 0.015;   % concrete (ecser) and steel (esser) serviceability strain
ecdam = 'twth';   esdam = 0.060;     % concrete (ecser) and steel (esser) damage control strain
                                   % (to use the 2/3 of the ultimate
                                   % concrete strain just tipe 'twth')

% temperature information (in case of freezing conditions)
temp  = 30;           % temperature of the specimen in celsius
kLsp  = 0.022;        % constant to calculate Lsp = kLsp*fy*Dbl
                      % (usually 0.022 at ambient temp. or 0.011 at -40C)

% Shear strength inforamtion
phiS    = 0.85;         % Strength reduction factor for shear
```

Figure 14. Input data for Example 2 (continued)

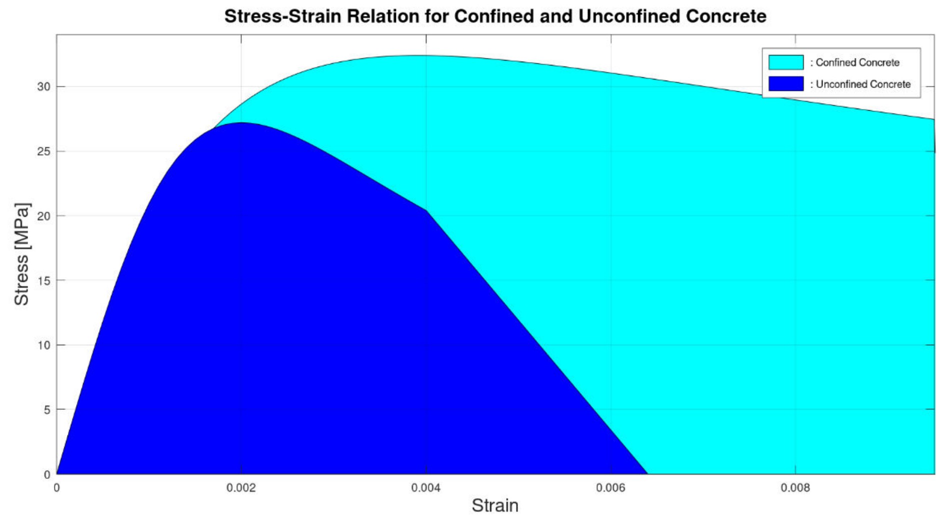

Figure 15. Concrete Model (Example 2)

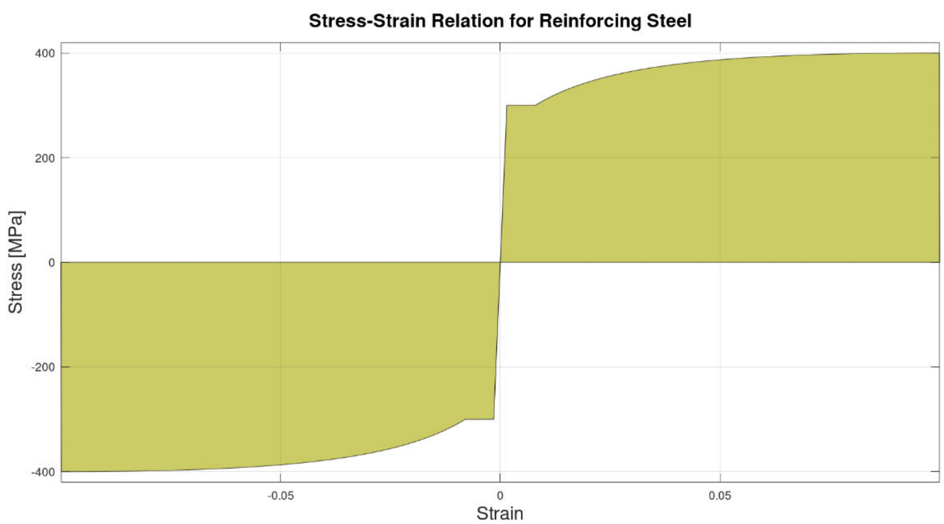

Figure 16. Steel Reinforcement Model (Example 2)

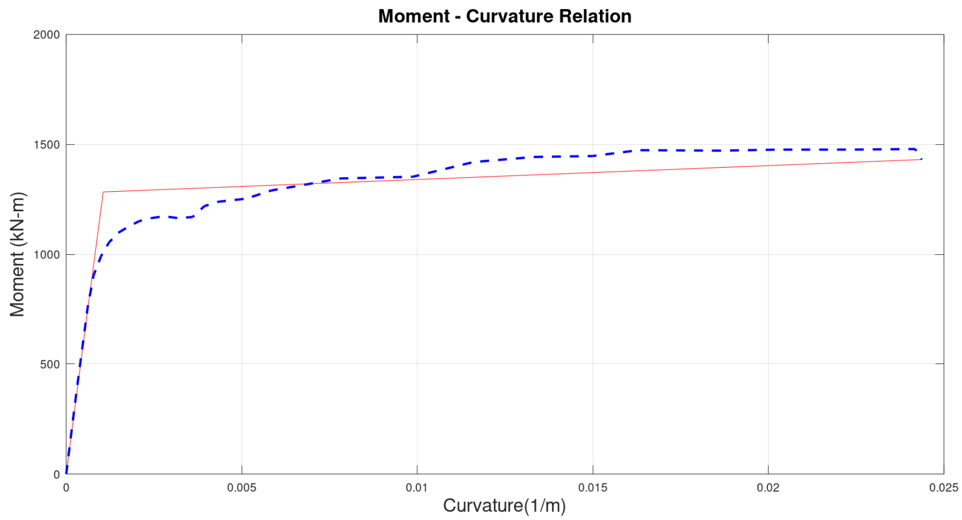

Figure 17. Moment – Curvature Relation (Example 2)

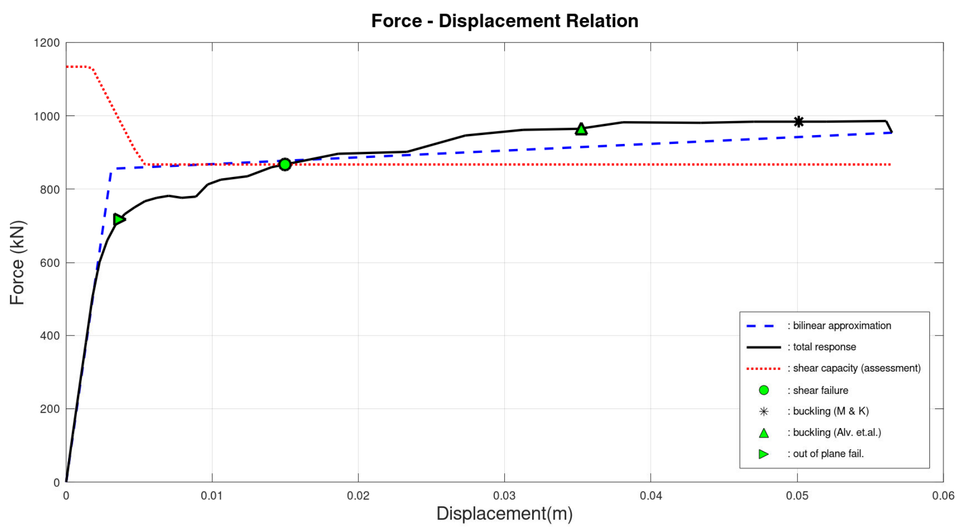

Figure 18. Member Response (Example 2)

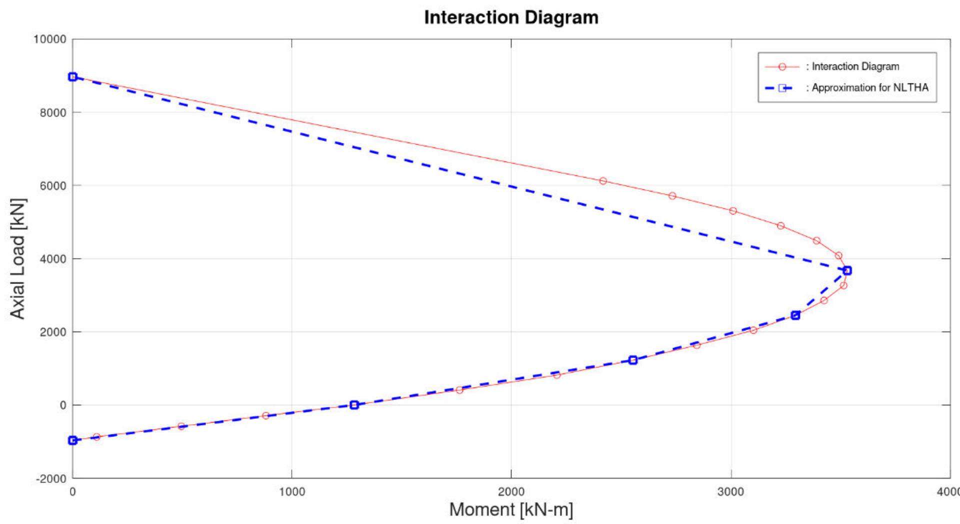

Figure 19. Interaction Diagram (Example 2)

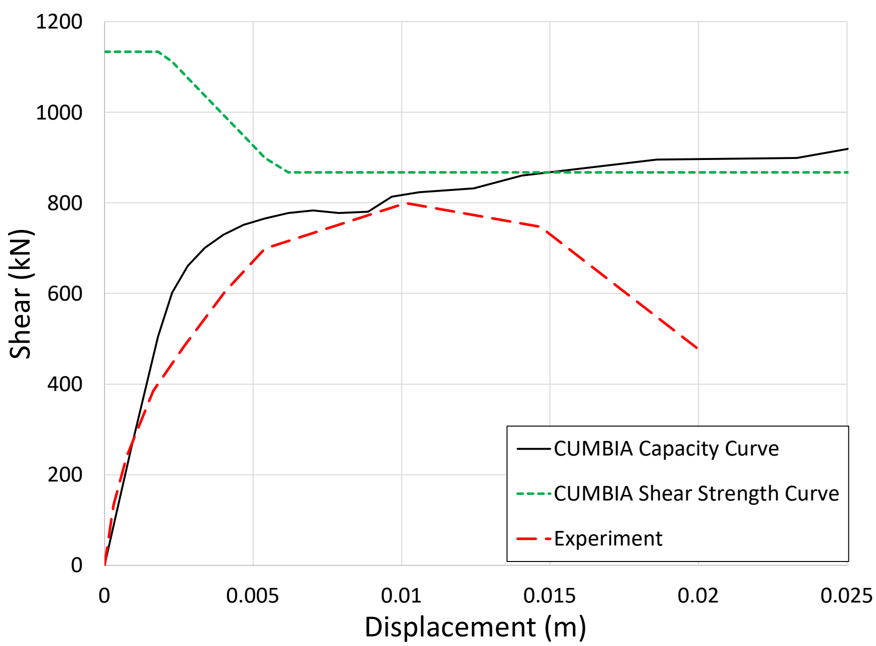

Figure 20. Force-Displacement Curve of the specimen Wall 1 (CUMBIA vs Experiment)

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