10th Canadian Masonry Symposium, Banff, Alberta, June 8 – 12, 2005

PROPOSED DESIGN METHODOLOGY FOR INTERNALLY FRP-REINFORCED MASONRY

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ABSTRACT

In spite of the rapid increase in the application of Fibre Reinforced Polymers (FRPs) to masonry structures, theory and rigorous analysis seem to lag behind. Most of the research work focused on the proof of the effectiveness of the application. There is a need for developing rational design approaches to enable designers and architects to utilize FRPs to their full potential. There are presently no codes of practice available for the design of neither FRP-reinforced or FRP-strengthened masonry structures. There are, however, a limited number of documents that provide design guidelines on reinforcing and strengthening concrete structures using FRPs.

In this paper, an approach for the design of FRP-reinforced masonry is presented. The approach is based on the accumulated body of knowledge on the use of FRPs to reinforce masonry and concrete structures. The proposed design approach is discussed in the context of the provisions of the Canadian Standards CSA S304.1-04 Design of Masonry Structures and CSA S806-02 Design and Construction of Building Components with Fibre Reinforced Polymers. The principles of the limit states design method are followed.

KEYWORDS: Fibre Reinforced Polymers, Reinforced Masonry, Design, Analysis

INTRODUCTION

In the past decade, Fibre Reinforced Polymers (FRPs) have been successfully used in reinforced masonry construction and, perhaps more significantly, to strengthen and retrofit existing masonry buildings. There are many reasons why FRPs are viable and, in many aspects, a superior alternative to conventional steel reinforcement. The tensile strength of FRPs is much higher than that of steel depending on the type of fibre. FRPs are immune to corrosion and very light in weight, which makes them very easy to handle and apply. FRPs are also electrically and magnetically neutral and are desirable for reinforcing structures, where signal interference has to be avoided.

FRPs are not intended to replace conventional steel. Rather, FRPs are a new generation of reinforcing materials that could be more efficient and economic than steel in particular applications. Situations such as strengthening basement walls for higher resistance to soil and ground water pressures, strengthening façades to enhance their resistance to lateral loads, and strengthening masonry infill shear walls to increase the overall lateral resistance of the structure are a few examples of where and when the use of FRPs would be desirable and efficient.

A proposed methodology for the design and analysis of masonry members internally reinforced with FRPs is presented in this paper. Due to space limitation, much of the presentation is confined to FRP-reinforced masonry beams. Nonetheless, the design example at the end of the paper demonstrates the extension of the method to FRP-reinforced masonry walls. Interested readers are directed to the more detailed design guide [6] developed for Masonry Canada. The guide covers the design of both internally and externally FRP-reinforced masonry. It addresses FRP material characteristics, FRP durability issues, and other design considerations.

FLEXURAL DESIGN PHILOSOPHY

The design philosophy of steel-reinforced sections is based on avoiding failure by crushing of masonry/concrete. To give warning and avoid catastrophic failure, the ductile yield-of-steel mode of failure is desirable. Therefore, masonry and concrete design codes adopted the under-reinforced section design philosophy to ensure that steel yields before masonry/concrete crushes. With the elastic linear response of FRPs, failure is sudden and brittle whether it is flexure tension or flexure compression. However, compression failure provides some warning due to plastic deformations of masonry/concrete and is therefore preferred. Hence, the philosophy adopted in all FRPs design guidelines is to design for over-reinforced sections. Reinforcement ratios from 1.33 to 1.4 times the balanced reinforcement ratio are recommended [1]. However, such high reinforcement ratios might result in impractical reinforcement areas when the compression zone is large, as in the case of grouted masonry walls.

For FRP reinforcement, serviceability requirement is usually the controlling design factor. Because the modulus of elasticity of FRPs is typically lower than that of steel, larger deflections and crack widths develop for FRP-reinforced sections. Since large curvatures subsequent to cracking lead to large strains in FRPs, it is crucial to limit crack width and deflection. This is achieved by maintaining curvature at service load at an acceptable limit of that at ultimate load. This is referred to as the deformability requirement. Since FRPs do not yield, the term deformability is used to describe the load deflection response of FRP-reinforced members. Deformability is quantified [1 and 5] by the deformability factor given in Equation 1 which is required to be equal to or higher than four to limit curvature. FRP reinforcement in Equation 1 is assigned a strain of 0.002 at service load. Unlike steel, it is advisable to design first for the serviceability limit state, assuming FRP strain of 0.002, and then check the ultimate limit state.

$$\text{Deformability Factor} = \frac{y_{\mathrm{u}} M_u}{y_s M_s} \geq 4 \qquad \text{Equation 1}$$

BALANCED, MINIMUM AND MAXIMUM FRP REINFORCEMENT

The balanced condition for FRP-reinforced sections is when masonry and FRP reinforcement reach their ultimate strains at the same time as shown in Figure 1 . Contrary to steel-reinforced sections, sudden failure will take place under this condition, which is referred to as balanced failure strain condition. From strain compatibility (Equation 2) and equilibrium of force (Equation 3), the balanced reinforcement ratio can be found (Equation 4). Failure modes are governed by whether the reinforcement ratio is lower or higher than balanced. When the reinforcement ratio is lower than balanced, failure takes place by rupture of FRP. Whereas flexural members will fail by masonry crushing for reinforcement ratios higher than balanced.

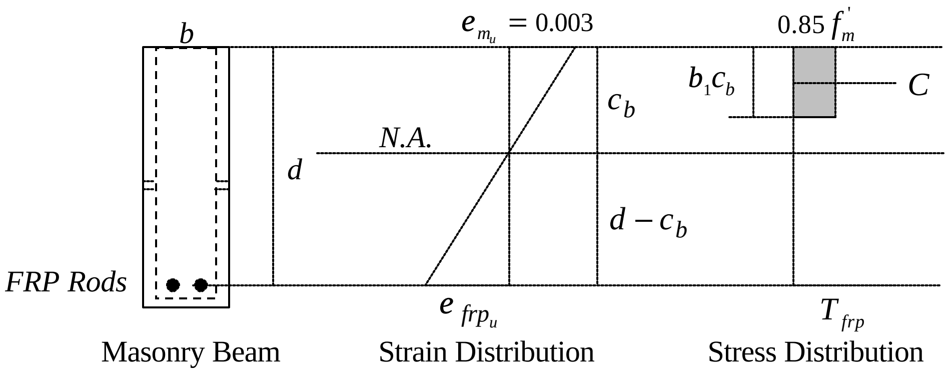


Figure 1 Balanced Failure Strain Condition of FRP-Reinforced Masonry

$$\frac{c_b}{d} = \frac{e_{m_u}}{e_{m_u} + e_{frp_u}} \qquad \text{Equation 2}$$

$$C = T_{frp} \qquad \text{Equation 3}$$

where, $C = f_m c\, 0.85 f_m^{'} b_1 c_b b$ and $T_{frp} = f_{frp} A_{frp_b} f_{frp\,u}$

$$r_{frp_b} = \frac{A_{frp_b}}{bd} = 0.85 b_1 c \,\frac{f_m f_m^{'}}{f_{frp} f_{frp_u}} \left(\frac{e_{m_u}}{e_{m_u} + e_{frp_u}} \right) \qquad \text{Equation 4}$$

To prevent immediate failure after cracking, resistance should be sufficiently higher than the cracking resistance. A minimum FRP reinforcement ratio that results in a resistance at least 1.5 times the cracking moment is recommended [1 and 3].

$$r_{frp_{\min}} \rightarrow M_r > 1.5\, M_{cr} \qquad \text{Equation 5}$$

To satisfy the deformability requirement, the reinforcement ratio should not exceed a maximum value defined by Equation 6. This limiting ratio was originally suggested [1] for concrete. The average concrete stress ($a_1 f_c^{'}$) is replaced here by the average masonry stress ($0.85 f_m^{'}$). For cost effectiveness, reinforcement ratios within 1.33 to 1.4 the balanced ratio should be used.

$$r_{frp_{\max}} = 0.2\left(0.85 f_m^{'} / f_{frp_s}\right) \qquad \text{Equation 6}$$

FLEXURAL RESISTANCE OF FRP-REINFORCED MASONRY

Most masonry design codes apply the limit states design method. The design approach described here follows the same method and is based on the classical rules of equilibrium of force and strain compatibility. The following assumptions have been made:

- Maximum compressive strain of masonry is 0.003.
- Strain of FRP under service load is limited to 0.002.
- Plane sections before deformation remain plane after deformation.
- Strain in FRP is equal to strain in masonry at the level of FRP (perfect bond).
- Contribution of FRP reinforcement in compression, if any, is ignored.

Design of FRP-reinforced sections is an iterative process regardless of whether the section is over-reinforced or under-reinforced. For over-reinforced sections where failure is due to masonry crushing, FRP tensile strain and the depth of neutral axis are unknown. For under-reinforced sections where failure takes place by rupture of FRP, masonry compressive strain and the depth of neutral axis are unknown. Starting the design process with satisfying serviceability requirements by limiting FRP strain to 0.002 under service loads facilitates the design procedure. The flexural design procedure can be summarized as follows:

1- Determine A_{frp} required to resist the service load moment assuming FRP strain of 0.002.
2- Calculate ρ_{frp_b} to determine the mode of failure. (Equation 4)
3- Check resistance at ultimate ($M_r > M_u$).
4- If $\rho_{frp} < \rho_{frp_b}$, check that $\rho_{frp} > \rho_{frp\,\min}$. (Equation 5)
   If $\rho_{frp} > \rho_{frp_b}$, check that $\rho_{frp} < \rho_{frp\,\max}$. (Equation 6)

SHEAR DESIGN PHILOSOPHY

Due to the limited experience on FRPs as shear reinforcement, design guidelines tend to be very conservative. The possible shear failure modes are either rupture of stirrups due to reaching its tensile capacity (shear-tension) or crushing of masonry (shear-compression). As the shear reinforcement increases, the mode of failure changes from shear tension to shear compression. Shear compression is associated with larger deflections, which gives warning to collapse, and is favourable. The following are main differences between steel and FRP as shear reinforcement:

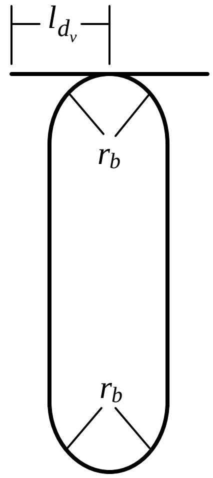
- FRPs have low modulus of elasticity and consequently low axial stiffness ($E_{frp}A_{frp}$). This results in wider shear cracks and reduced compression zone, which minimizes the shear resistance contribution from mechanical interlock and compressed masonry.
- FRPs have lower dowel resistance than steel. Although the contribution of the dowel action of FRP reinforcement has not been quantified yet, it is believed that it is less than that of steel due to the lower transverse strength and stiffness of FRPs.
- Tensile strength of FRP stirrups is significantly lower than that of FRP bars due to stress concentrations at bend regions. Studies [7] show that the strength of stirrups could be as low as 40% than that of straight bars.

The main factors controlling the stirrups' strength are the bend radius (r_b) and embedment length (l_{d_v}) shown in Figure 2 . To reduce stress concentration, the ratio of bend radius to stirrup's diameter should be greater than three and stirrups should be closed with 90° hooks. A development length beyond the hook of at least 12 times the stirrup diameter is required to transfer the tensile force to concrete [1 and 2] A similar development length is suggested for masonry beams.

SHEAR RESISTANCE OF FRP-REINFORCED MASONRY

Similar to steel reinforcement, the factored shear resistance of FRP-reinforced masonry beam can be expressed as the summation of masonry and FRP shear resistances as given in Equation 7.

$$V_r = V_m + V_{frp} \qquad \text{Equation 7}$$



d_e = effective stirrup diameter = $\sqrt{\dfrac{4A_{frp_V}}{p}}$

r_b = bend radius > 3 d_e

l_{d_v} = development length > 12 d_e

$$V_{frp} = 0.4\left(f_{frp} A_{frp_V}\, f_{frpu}\, d/s\right) < 0.36 f_m \sqrt{f'_m}\, b_w d \qquad \text{Equation 8}$$

Figure 2 Typical Configuration of FRP Stirrups. (adapted from [7])

Equation 8, adapted from [3], is similar to that for steel shear reinforcement with the exception that only 40% of the ultimate strength of FRP is considered for shear strength and hence the 0.4 factor. The assumption that shear capacities of masonry and FRP reinforcement are added is valid when shear cracks are controlled. Therefore, the tensile strain of FRP stirrups under service load should not exceed a specified value to limit crack width. A limiting strain value of 0.002 as defined by Equation 9, adapted from (2), is suggested.

$$e_{frp_{v_{ser}}} = \frac{s(V_{ser} - V_m)}{A_{frp_V} E_{frp} d} \leq 0.002 \qquad \text{Equation 9}$$

According to the Canadian masonry code S304.1-04, minimum shear reinforcement is required if V_f exceeds 0.5 V_m. For steel reinforcement, the minimum shear reinforcement required by S304.1-04 is given in Equation 10. For FRP shear reinforcement, Equation 11 is suggested where f_y of steel is replaced by $0.4 f_{frp_u}$ for FRP reinforcement. The recommended maximum spacing between stirrups is 600 mm or d/2, which ensures that each crack is intercepted by at least one stirrup. The shear design procedure can be summarized in the following steps:

1- Calculate V_m and check if shear reinforcement is required.
2- Calculate A_{frp_V}. (Equation 8)
3- Check strain limit under service loads. (Equation 9)
4- Check $A_{frp_{V\min}}$. (Equation 11)

$$A_{s_V \min} = \frac{0.35 b_w s}{f_y} \qquad \text{Equation 10}$$

$$A_{frp_v \min} = \frac{0.35 b_w s}{0.4 f_{frp_u}} \qquad \text{Equation 11}$$

DEVELOPMENT LENGTH

Insufficient development length may result in premature failure by pull out of reinforcement or splitting of the reinforced medium. In masonry construction, large cover prevents the splitting mode of failure. To control the pull out mode of failure for FRP-reinforced masonry, the modification to the minimum development length expression for steel-reinforced masonry [4] given in Equation 12 is suggested. Application of Equation 12 requires a minimum bar spacing of $2d_b$. Unlike steel, FRP rods require longer development lengths due to their higher strength. Substituting in Equation 12 for f_{frp_u} of CFRP rods (2250 MPa) assuming grout strength, f'_{gr} of 25 MPa and $K_3 = 0.8$, the minimum development length is approximately 160 d_b. This is much larger than the 30-40 d_b range for steel reinforcement.

$$l_{d_{frp}} = 0.45 K_1 K_3 K_4 K_5 \frac{f_{frp_u}}{\sqrt{f'_{gr}}} d_b \qquad \text{Equation 12}$$

CRACK CONTROL

Presence of cracks encourages the ingress of moisture. This can lead to the disintegration of mortar due to freeze-thaw activity. Notwithstanding the deformability requirement, Equation 13 is adopted [3] to control crack width in FRP-reinforced concrete and is suggested for FRP-reinforced masonry beams. Unlike steel, FRPs are not susceptible to moisture corrosion and therefore, higher Z values than specified in S304.1-04 for steel-reinforced masonry can be tolerated. When e_{frps} exceeds 0.0015, the quantity Z should be less than 45 kN/mm for exterior exposure and 38 kN/mm for interior exposure [3]. Cracking in masonry walls is controlled through deflection limits.

$$Z = K_b \frac{E_s}{E_{frp}} f_{frp} \sqrt[3]{d_c A} \qquad \text{Equation 13}$$

DESIGN EXAMPLE

The loadbearing wall shown in Figure 3 is constructed using type S mortar and fully grouted 40 MPa concrete blocks and is reinforced with 20M vertical steel bars @ 400 mm spacing.

a) Determine the resistance of the wall under pure bending.
b) Find the area of CFRP required to reach the same capacity in (a).
c) Calculate the axial load capacity of the wall for both types of reinforcement.
d) Construct the axial load-moment interaction diagram for both types of reinforcement.

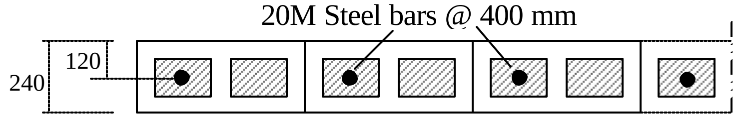


Figure 3 Steel-Reinforced Concrete Masonry Wall

Table 1 Material Design Properties

Masonry	Steel	CFRP
f'_m **= 17 MPa**	f_y **= 400 MPa**	f_{frp_u} **= 2250 MPa**
E_m **= 14.45 MPa**	E_s **= 200 GPa**	E_{frp} **= 150 GPa**
e_{m_u} **= 0.003**	e_s **= 0.002**	e_{frp_u} **= 0.015**

f_m = 0.6 and f_s = 0.85 (CSA S304.1-04) f_{frp} = 0.8 (CSA S806-02)

a) Pure Bending (P=0)

$C = f_m c 0.85 f'_m b b_1 c = 0.6*1.0*0.85*17*1000*0.8c = 6936\ c$ kN/m

A_s of 20M = 300 mm^2

A_s/m = 300*1000/400 = 750 mm^2/m

Assume steel yields, $T_s = 0.85 A_s f_y$ = 0.85* 750* 400 = 255 kN/m

From equilibrium, $C = T_s \ \rightarrow\ c = 36.8$ mm

$e_s = 0.0068 > 0.002$ *Therefore, assumption of yield of steel is correct.*

$$M_r = T_s\left(d - \frac{b_1 c}{2}\right) = 255(120 - 14.7)/1000 = 26.85 \text{ kN-m/m}$$ <u>*Ans.*</u>

b) Equivalent CFRP Reinforcement

Assume compression failure $\rightarrow C = 6936\ c$ (*as before*)

$$M_r = C\left(d - \frac{b_1 c}{2}\right)$$

$\therefore 26.85\ (10)^6 = 6936\ c*(120 - 0.4c)$, $c = 36.8$ mm (*as before*)

$T_{frp} = f_{frp} A_{frp} E_{frp} e_{frp}$

From strain compatibility, $e_{frp} = e_m\left(\dfrac{d-c}{c}\right) = 0.003\left(\dfrac{120-c}{c}\right)$

$$T_{frp} = 0.75* A_{frp} *150{,}000*0.003\left(\frac{120-c}{c}\right) = 338\left(\frac{120-36.8}{36.8}\right)A_{frp} = 255 \text{ kN/m}$$

$\therefore A_{frp} = 334$ mm^2/m

$e_{frp} = 0.0068 << 0.015$ *Therefore, assumption of compression failure is correct.*

$r_{frp_{\max}} = 0.01$ (from Equation 6)

$A_{frp_{\max}} = 1200$ mm^2/m > A_{frp} <u>*OK*</u>

Therefore, use one 16 mm CFRP rod @ 600 mm spacing (A_{frp}=333mm^2/m) <u>*Ans*</u>.

c) Axial Load (M=0)

Contribution of steel under compression is not accounted for unless it is tied. Contribution of FRPs in compression is always neglected.

Steel-reinforced wall

Assuming that steel is tied, $P_r = 0.8[f_m 0.85 f'_m A_e + j_s A_s f_y]$

$P_r = 0.8[0.6*0.85*17*(1000*240) + (0.85*750*400)] = 1869$ kN/m <u>*Ans*</u>.

FRP-reinforced wall

$P_r = 0.8 f_m 0.85 f'_m A_e = 0.8*0.6*0.85*17*(1000*240) = 1665$ kN/m <u>*Ans*</u>.

d) Interaction Diagrams

Steel-reinforced wall, $C = 6936\, c$ (*as before*)

$c_b = 72$ mm

For c values $< c_b \;\rightarrow\; T_s = 255{,}000$ N/m

For c values $> c_b \;\rightarrow\; T_s = 0.85 A_s E_s e_s$

$$T_s = 0.85*750*200{,}000*0.003\left(\frac{120-c}{c}\right) = 382500\left(\frac{120-c}{c}\right)$$

$$P_r = C - T_s \text{ and } M_r = C\left(d - \frac{b_1 c}{2}\right)$$

FRP-reinforced wall, $C = 6936\, c$ (*as before*)

$$T_{frp} = f_{frp} A_{frp} E_{frp} e_{frp} = 0.75*313.3*150{,}000*0.003\left(\frac{120-c}{c}\right) = 105739\left(\frac{120-c}{c}\right)$$

$$P_r = C - T_{frp} \text{ and } M_r = C\left(d - \frac{b_1 c}{2}\right)$$

A spreadsheet was used to construct the interaction diagrams shown in Figure 4 . For c values higher than $t/2$, contribution of CFRP in compression was neglected whereas contribution of steel was accounted for and hence the higher axial load resistance. A summary of the resistance values is given in Table 2 . The higher moment capacity of steel-reinforced over CFRP-reinforced section is due to the significant difference in axial stiffness (EA). Under the same strain, the developed tensile strength in steel is much higher than that developed in CFRP.

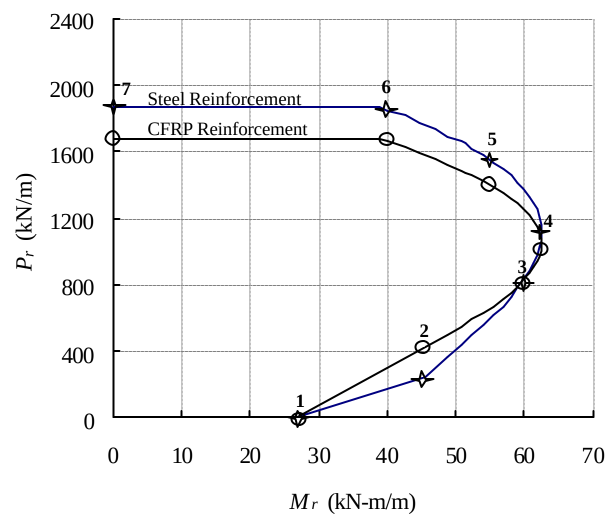


Figure 4 Interaction Diagrams of a Reinforced Masonry Wall.

Table 2 Resistance Values for Figure 4

Point #	c (mm)	P_r (kN/m)		M_r (kN-m/m)	Comments
		Steel	CFRP		
1	36.8	-	-	26.9	Pure Bending
2	72.0	244	429	45.5	Yield of Steel
3	120.0	832	832	59.9	N.A. at Centre
4	150.0	1117	1040	62.4	Peak Moment
5	200.0	1540	1387	55.5	
6	240.0	1856	1665	40.0	N.A. at Edge
7	∞	1869	1665	-	Axial Load

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NOTATIONS

A	effective tension area surrounding flexure tension reinforcement
A_e	effective cross-sectional area of masonry
A_{frp}	area of FRP reinforcement
A_{frp_b} , $A_{frp_{\max}}$	balanced and maximum areas of FRP reinforcement; respectively
A_{frp_V}	area of FRP shear reinforcement
$A_{frp_{V\min}}$	minimum area of FRP shear reinforcement
A_s , A_v	areas of longitudinal and transverse steel reinforcement; respectively
$A_{v\min}$	minimum area of steel shear reinforcement
C	masonry compressive force
E_{frp} , E_m , E_s	modulus of elasticity of FRP, masonry, and steel; respectively
K_1	rod location factor as defined in Clause 9.3.4 of S806-02
K_3 , K_5	rod size and surface profile factors as defined in Clause 9.3.4 of S806-02; respectively
K_4	fibre type factor as defined in Clause 9.3.4 of CSA S806-02
K_b	bond characteristics coefficient as defined in Clause 8.3.1.1 of S806-02
M_{cr} , M_u	bending moments under cracking and ultimate loads; respectively
M_r	moment of resistance of masonry

M_s	bending moment under service loads
P_r	compression resistance of masonry
T_{frp}, T_s	tensile forces in FRP reinforcement and steel reinforcement; respectively
V_f	shear force under factored loads
V_{frp}, V_m	factored shear resistances of FRP reinforcement and masonry; respectively
V_r	total factored shear resistance of a reinforced masonry section
V_{ser}	shear force under service loads
b	width of masonry beam
b_w	minimum width of a masonry section for shear calculations
c	distance of neutral axis from extreme compression fibre
c_b	distance of neutral axis c at balanced strain condition
d	effective depth of masonry section
d_b	diameter of reinforcing bar
d_c	distance from extreme tension fibre to the centreline of closest longitudinal rod
d_e	effective FRP stirrup diameter
f_{frp}	stress in FRP under specified loads as defined in Clause 8.3.1.1 of CSA S806-02
f_{frp_u}	ultimate tensile stress of FRP reinforcement
f_{frp_s}	tensile stress of FRP reinforcement under service loads $= E_{frp} e_{frp_s}$ and $e_{frp_s} = 0.002$
f'_{gr}	in-situ compressive stress of grout
f'_m	compressive stress of masonry at 28 days
f_y	yield stress of steel
$l_{d_{frp}}$, l_{d_V}	development lengths of FRP reinforcement and FRP stirrups; respectively
r_b	FRP stirrup bend radius
s	spacing of shear reinforcement
b_1	ratio of depth of rectangular compression block to depth of the neutral axis
e_{frp_u}	ultimate tensile strain of FRP reinforcement
e_{frp_s}	tensile strain of FRP reinforcement under service loads
$e_{frp_{vser}}$	tensile strain of FRP shear reinforcement under service load
e_{m_u}	ultimate compressive strain of masonry
e_y	yield strain of steel reinforcement
r_{frp}	area of FRP reinforcement to the effective masonry area
r_{frp_b}	area of FRP reinforcement to the effective masonry area at balanced condition
$r_{frp_{\max}}$, $r_{frp_{\min}}$	maximum and minimum FRP reinforcement ratios
f_{frp}, f_m, f_s	material resistance factors for FRP, masonry, and steel; respectively
c	factor accounts for the direction of compressive stress relative to bed joints
y_s, y_u	curvature under service and ultimate loads; respectively