REINFORCED MASONRY SHEAR WALL – STATIC EQUILIBRIUM - ASD

Reinforced Masonry Shear Wall Based on Static Equilibrium and Strain Compatibility;

Reference and notes: Reinforced Masonry Handbook use static equilibrium and strain compatibility. Enter gravity forces per length of wall and lateral forces as total forces. The reinforcement is assumed to have no reinforcing ties and therefore will not be considered effective (i.e. it will only resist tension forces). The spacing of the reinforcement is as follows – One bar placed at ends, the length of the wall is divided by the spacing of bars to give N bars. This number is rounded down and a new assumed spacing is used with one less bar than would be used in the field, therefore the calculation should error on the conservative side. The subscript "U" is used for 'factored loads' this should not be confused with strength design. This uses ASD load factors and design stresses/strengths.

Wall Geometry and Forces;

Height of wall; $H=10.0ft$;

Length of Wall; $L=8.0ft$;

Spacing of Vertical Reinforcment; $S=32in$;

Vertical reinforcment size (bar number); $b_{tnum}=4$; $A_b=pi/4*(b_{tnum}*1in/8)^2=\mathbf{0.196}in^2$;

Horizontal shear reinforcement (see spacing below); $A_v=0.11in^2$;

Spacing of horizontal shear reinforcment; $S_v=32in$;

Unfactored Loads per foot of wall;		Total Load on wall
Self Weight of Masonry at location of max moment;	$P_{SW}=0.610$ kip/ft;	$P_{SWT}=L*P_{SW}=\mathbf{4.880}$kip;
Axial load due to dead load;	$P_{DL}=0.5$ kip/ft;	$P_{DLT}=L*P_{DL}=\mathbf{4.000}$kip;
Axial load due to live loads;	$P_{LL}=0.0$ kip/ft;	$P_{LLT}=L*P_{LL}=\mathbf{0.000}$kip;
Axial load due to snow loads;	$P_{SL}=0.25$ kip/ft;	$P_{SLT}=L*P_{SL}=\mathbf{2.000}$kip;
Lateral Load at top of wall;	$V_{LAT}=18$ kip;	
Moment from above due to Lateral loads;	$M_{LAT}=80$ kip_ft;	

Load Combination Factors;		Per Foot;	Total Load on wall
Self weight;	$LF_{SW}=1.0$;	$P_{USW}=LF_{SW}*P_{SW}=\mathbf{0.610}$kip/ft;	$P_{USWT}=L*P_{USW}=\mathbf{4.880}$kip;
Dead Load;	$LF_{DL}=0.6$;	$P_{UDL}=LF_{DL}*P_{DL}=\mathbf{0.300}$kip/ft;	$P_{UDLT}=L*P_{UDL}=\mathbf{2.400}$kip;
Live Load;	$LF_{LL}=1.0$;	$P_{ULL}=LF_{LL}*P_{LL}=\mathbf{0.000}$kip/ft;	$P_{ULLT}=L*P_{ULL}=\mathbf{0.000}$kip;
Snow Load;	$LF_{SL}=1.0$;	$P_{USL}=LF_{SL}*P_{SL}=\mathbf{0.250}$kip/ft;	$P_{USLT}=L*P_{USL}=\mathbf{2.000}$kip;
Moment from above;	$LF_M=1.0$;		$M_{UT}=LF_M*M_{LAT}=\mathbf{80.000}$kip_ft;
Shear;	$LF_V=1.0$;		$V_{UT}=LF_V*V_{LAT}=\mathbf{18.000}$kip;

Total vertical load; $P_{UT}=P_{USWT}+P_{UDLT}+P_{ULLT}+P_{USLT}=\mathbf{9.280}$kip;

Wall Properties and Geometry;

Wall properties are based on spacing of bars and are per foot of wall length quanties. The section properties are calculated for the weak axis (out of plane).

(Assumes face shells, cell at bar and webs each side of bar are grouted)

Width of section use spacing; b=S; (for wall properties);

Thickness of wall (height of x-section); $h=7.625in$;

Flange thickness; $t_f=1.25in$;

Web thickness; $t_w=1.0in$;

Width of unit and width of grout; $b_{unit}=15.625in$; $b_{grout}=0.375in$;

Nominal width of unit; $b_{nunit}=b_{unit}+b_{grout}=\mathbf{16.000}in$;

Width of web cell; $b_{cell}=(b_{unit}-3*t_w)/2=\mathbf{6.313}in$;

Width of web; $b_w=b_{cell}+2*t_w=\mathbf{8.313}in$;

Height of web; $h_w=h-2*t_f=\mathbf{5.125}in$;

Area per foot of wall length; $A_{ft}=(2*b*t_f+h_w*b_w)/S=\mathbf{45.976}in^2/ft$; $A_{tot}=A_{ft}*L$;

Moment of inertia per foot; $I_{ft}=(b*h^3/12-(b-b_w)/2*h_w^3/12*2)/S=\mathbf{343.678}in^4/ft$;; $I_{tot}=I_{ft}*L$;

Radius of gryation; $r=(I_{tot}/A_{tot})^{0.5}$=**2.734**in;

Bar Layout;

The bar layout is based on finding the number of bars based on the length of wall and bar spacing. It then rounds down and places the end bars a distance d' from the ends of the wall. This approximation is conservative as it will contian one less bar than placed in the field.

Distance from end of wall to end bars; d'=4in;

Effective length of wall; L'=L-2*d'=**7.333**ft; b=**32.000**in; L=**8.000**ft; S=**32.000**in;

Number of bars in field (round up and add bar at end); n=L/S=**3.000**; N=int(L/S)=**3.000**; N=ceiling(L/S+1,1)=**4.000**;

Number of bars to use in calc (round down); N'=floor(L/S+1,1)=**4.000**;

Design spacing of rebar; S'=L'/(N'-1)=**29.333**in;

Wall Properties Material (See Tbl 2.2B or IBC 2105.2.2.1.2);

Allowable 1/3rd increase; i=1.33;

Maximum masonry compressive stress; f'_m'=1500psi; $f'_m=f'_m$'*i=**1995.000**psi;

Rebar yeild stress; F_y=60ksi;

Modulus of Elasticity – steel; E_s=29000ksi; $\varepsilon_y=F_y/E_s$=**0.002**in/in

Modulus of Elasticity – masonry; $E_m=900*f'_m$=**1795.500**ksi; (use 700 for clay)

Modular ratio; $n=E_s/E_m$=**16.151**;

Allowable rebar stress; F_s'=max(F_y /2.5,24ksi); $F_s=F_s$'*i=**31920.000**psi;

Allowable compressive stress; $F_m=f'_m/3$=**665.000**psi;

Slenderness Ratio; γ=H/r=**43.890**

Reduction factor for slenderness; $R1=1-(\gamma/140)^2$=**0.902**; $R2=(70/\gamma)^2$=**2.544**;

R=if(γ<=99,R1,R2)=**0.902**;

Allowable axial stress; Fa=0.25*f'_m*R=**449.731**psi;

Eccentricity required from loads; $e_d=M_{UT}/P_{UT}$ =**103.448**in;

Analysis of Stresses;

Assume location of NA; kd=26in;

Assume allowable compressive stress of masonry controls; $f_m=F_m$=**665.000**psi;

Area of masonry compression; $A'=A_{ft}$*kd=**99.614**in^2;

Masonry compression force; $C_m=0.5*f_m*(A')$=**33.122**kip;

Moment arm to compression force; X_m=L/2-kd/3=**3.278**ft;

Moment strength of compressive stress; $M_{cm}=C_m*X_m$=**108.565**kip_ft;

Find Stress in Rebar;

Number of bars to copy (start at bar 2); N'=**4.000**;.

Steel bar number from right end; ib=1; ib_{prev}=ib;

Distance from end of wall to bar; d=d'

Stress in bar; $f'_s=n*(d-kd)*f'_m/kd$=**-27.265**ksi; $f_s=\max(0ksi, \min(F_s,f'_s))$=**0.000**ksi;

Tension in bar; $T=f_s*A_b$=**0.000**kip; T_{prev}=0kip; $T_{tot}=T_{prev}+T$=**0.000**kip;

Moment arm; Xs=L/2-d=**3.667**ft;

Moment strength at bar 1; M=T*Xs=**0.000**kip_ft; Mprev=0kip_ft; Mtot=Mprev+M

Tedds	Project				Job Ref.	
	Section				Sheet no./rev. 3	
	Calc. by RSF	Date 11/9/2012	Chk'd by	Date	App'd by	Date

Steel bar number from right end;	$ib=ib_{prev}+1=\mathbf{2.000}$;	$ib_{prev}=ib$; $ib_{prev}=\mathbf{2.000}$;	
Distance from end of wall to bar;	$d=(ib-1)*S'+d'=\mathbf{2.778}ft$;		
Stress in bar;	$f'_s=n*(d-kd)*f'_m/kd=\mathbf{9.088}ksi$;	$f_s=max(0ksi, min(F_s,f'_s))=\mathbf{9.088}ksi$;	
Tension in bar;	$T=f_s*A_b=\mathbf{1.784}kip$;	$T_{prev}=T_{tot}$;	$T_{tot}=T_{prev}+T=\mathbf{1.784}kip$;
Moment arm;	$Xs=d-L/2=\mathbf{-1.222}ft$;		
Moment strength at rebar;	$M=T*Xs=\mathbf{-2.181}kip\_ft$;	$M_{prev}=M$;	$M_{tot}=M_{prev}+M=\mathbf{-4.362}kip\_ft$;

Steel bar number from right end;	$ib=ib_{prev}+1=\mathbf{3.000}$;	$ib_{prev}=ib$; $ib_{prev}=\mathbf{3.000}$;	
Distance from end of wall to bar;	$d=(ib-1)*S'+d'=\mathbf{5.222}ft$;		
Stress in bar;	$f'_s=n*(d-kd)*f'_m/kd=\mathbf{45.442}ksi$;	$f_s=max(0ksi, min(F_s,f'_s))=\mathbf{31.920}ksi$;	
Tension in bar;	$T=f_s*A_b=\mathbf{6.267}kip$;	$T_{prev}=T_{tot}$;	$T_{tot}=T_{prev}+T=\mathbf{8.052}kip$;
Moment arm;	$Xs=d-L/2=\mathbf{1.222}ft$;		
Moment strength at rebar;	$M=T*Xs=\mathbf{7.660}kip\_ft$;	$M_{prev}=M$;	$M_{tot}=M_{prev}+M=\mathbf{15.321}kip\_ft$;

Steel bar number from right end;	$ib=ib_{prev}+1=\mathbf{4.000}$;	$ib_{prev}=ib$; $ib_{prev}=\mathbf{4.000}$;	
Distance from end of wall to bar;	$d=(ib-1)*S'+d'=\mathbf{7.667}ft$;		
Stress in bar;	$f'_s=n*(d-kd)*f'_m/kd=\mathbf{81.795}ksi$;	$f_s=max(0ksi, min(F_s,f'_s))=\mathbf{31.920}ksi$;	
Tension in bar;	$T=f_s*A_b=\mathbf{6.267}kip$;	$T_{prev}=T_{tot}$;	$T_{tot}=T_{prev}+T=\mathbf{14.319}kip$;
Moment arm;	$Xs=d-L/2=\mathbf{3.667}ft$;		
Moment strength at rebar;	$M=T*Xs=\mathbf{22.981}kip\_ft$;	$M_{prev}=M$;	$M_{tot}=M_{prev}+M=\mathbf{45.962}kip\_ft$;

Nominal axial strength of section;	$P_n=C_m-T_{tot}=\mathbf{18.802}kip$;
Nominal moment strength of section;	$M_n=M_{cm}+M_{tot}=\mathbf{154.527}kip\_ft$;
Eccentricity of section design;	$e_s=M_n/P_n=\mathbf{8.219}ft$;
Eccentricity required by loading;	$e_r=M_{UT}/P_{UT}=\mathbf{8.621}ft$;

Shift NA;

NAS=if(and(kd>L',e_s>e_r),"No Tension", if(e_s>e_r,"Increase kd","Decrease kd"))=**"Decrease kd"**;

Moment and Axial Results;

Moment capacity;	$M_c=M_n=\mathbf{154.527}kip\_ft$;f
Critical Axial Load;	$P_c=P_n*R=\mathbf{16.954}kip$;
Axial stress due to axial load only;	$f_a=P_{UT}/A_{tot}=\mathbf{25.231}psi$;

Shear;

Use shear reinforcement?;	VD=if(A_v>0in^2,"Yes","No")=**"Yes"**;
Shear stress;	$f_v=V_{UT}/A_{tot}=\mathbf{48.939}psi$;
Moment shear ratio;	$M\_V=M_{UT}/(V_{UT}*L')=\mathbf{0.606}$;
Allowable shear stress for reinforced M-V<1;	$Fvr1=min(0.5*(4-M\_V)*(f'_m*1in^2/lb)^{0.5}*1psi*i,(120psi-45*M\_V*1psi)*i)=\mathbf{87.412}psi$;
Allowable shear stress for reinforced "M_V>=1";	$Fvr2=min(1.5*(f'_m*1in^2/lb)^{0.5}*1psi*i,75psi*i)=\mathbf{77.266}psi$;
Allowable shear stress for unreinforced "M_V<1";	$Fvr1=min(1/3*(4-M\_V)*(f'_m*1in^2/lb)^{0.5}*1psi*i,(120psi-45*M\_V*1psi)*i)=\mathbf{58.275}psi$;
Allowable shear stress for unreinforced "M_V>=1";	$Fvr2=min((f'_m*1in^2/lb)^{0.5}*1psi*i,35*1psi*i)=\mathbf{46.550}psi$;

Allowable Shear Stress;	Fv=if(VD==”yes”,if(M_V<1,Fvr1,Fvr2),if(M_V<1,Fvr1,Fvr2))=**58.275**psi;
Required spacing of shear reinforcment;	S_v’=A_v*F_s*L’/V_{UT}=**17.166**in;
Vertical steel required for shear;	A_{vv}=if(A_v>0 in^2,A_v*H/S_v’/3,”Not Req’d”)=**0.256**in^2;

Results;

Moment Capacity;	ChM=if(M_c>M_{UT},”OK”,”NG”)=**"OK"**;
Axial Capacity;	ChP=if(P_c>P_{UT},”OK”,”NG”)=**"OK"**;
Axial stress;	Chfa=if(Fa>f_a,”OK”,”NG”)=**"OK"**;
Shear stress;	Chfv=if(Fv>f_v,”OK”,”NG”)=**"OK"**;
Spacing of horizontal shear reinforcment;	ChSv=if(S_v’>S_v,”OK”,”NG”)=**"NG"**;
Minium vertical steel required for shear;	A_{vv}=**0.256**in^2;