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MASONRY VENEER WITH STEEL STUD STRUCTURAL BACKING

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ABSTRACT

This paper discusses the design of a masonry veneer with a steel stud structural backing. The design requirements of CSA S304.1-94 "Masonry Design for Buildings (Limit States Design)" are discussed. A design example is given for a typical 2400 high infill stud wall with a window opening.

INTRODUCTION

Recent investigations of steel stud/brick veneer walls in highrise buildings (Drysdale and Suter, 1991) have demonstrated the need for minimum structural requirements for the design of structural backings and ties when the structural backings are not substantially stiffer than the masonry veneer. The new CSA Standard S304.1-94 "Masonry Design for Buildings (Limit States Design)" contains specific requirements for flexible structural backing systems. This paper is based on these requirements unless otherwise noted.

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FLEXIBLE STRUCTURAL BACKING

A flexible structural backing is defined as having a stiffness, EI, less than 2.5 times the uncracked stiffness of the veneer. A steel stud structural backing would be classed as a flexible structural backing.

VENEER

The masonry veneer is required to be at least 75mm thick and to be tied to a structural backing for lateral support. Unless otherwise engineered, the veneer must also bear on noncombustible supports spaced not more than 3.6m vertically when more than 11m above the top of the foundation. Provision should also be made for differential movement and dimensional changes by using horizontal and vertical movement joints.

TIES

Ties are required to be spaced not more than 600mm apart vertically and 800mm apart horizontally. The spacing cannot be staggered. This ensures that the lateral loading is applied reasonably uniformly to the structural backing. Additional requirements around openings and at top and bottom of veneer panels can be found in CSA Standard A370-94 "Connectors for Masonry" - clauses 6.1.2 and 6.1.3 and again in A371-94 "Masonry Construction for Buildings" - clauses 5.6.1.2 and 5.6.1.3. At openings, the ties shall be spaced not more than 600mm apart all around and be located not more than 300mm from the edge of the opening. The distance from the bottom of the panels to the first row of ties can be 600mm unless the bearing support does not provide adequate lateral support, in which case the distance is reduced to 400mm. An example of inadequate lateral support would be the use of a flashing with a low frictional resistance. The distance from the last row of ties to the top of the panel should not exceed 300mm.

For flexible structural backings, each tie is required to be designed for 40% of the tributary lateral load on the vertical line of ties, but not less than double the tributary lateral load on the tie. The 40% rule allows for the uneven distribution of the tie forces on a stud due to the relative flexibility of the stud. The largest tie forces are at the top and bottom of the veneer panel until the veneer panel fails at midheight due to the lateral loading. Once this occurs, the largest tie force can be at midheight.

Alternatively, the tie forces can be determined by a detailed stiffness analysis before and after cracking of the veneer. This might be cost effective for high walls.

Each tie must also conform to strength and stiffness requirements of CSA Standard A370-94. The total free play shall not exceed 1.2mm. The minimum stiffness requirements for ties are a deflection due to the free mechanical play plus a tension or compression of 0.45 kN not to exceed 2.0mm. In addition, stiffer ties can be used to allow a more economical flexible backup for normal height walls. Stiffer ties are defined as having a deflection due to one half the total free play plus a tension or compression of 0.45 kN not exceeding 1.0mm.

STEEL STUD STRUCTURAL BACKING

The structural backing must be designed to resist the entire lateral load applied to the wall system. In addition, for flexible structural backing systems the total deflection of the veneer must not exceed h/600 due to specified wind loads. This means that the steel studs must be stiff enough so that the combined tie deflection and stud deflection does not exceed h/600.

Alternatively, the stud deflection alone can be limited to h/720 provided that the stiffer ties are used. This would be more cost effective for normal height walls.

There is no limit on the stiffness of the top and bottom tracks. However the design example recommends 1mm maximum deflection.

Allowance for differential movement is made by providing a deflection gap at the top track.

MASONRY VENEER / STEEL STUD DESIGN EXAMPLE

INTRODUCTION

This design example is based upon a sheathed design approach which assumes that the sheathing is structurally adequate to resist the torsional component of loads not applied through the shear centre and to resist the effects of lateral instability. Members are designed using simple beam theory.

This example is typical of a Masonry Veneer / Steel Stud (MV/SS) wall that may be found in a residential mid-rise apartment structure.

Figure 1 shows the components of the wind bearing infill stud wall assembly.

GIVEN

- Masonry Veneer / Steel Stud Wall
- Building Height 15 m - Concrete Construction
- H / W < 1
- q (^{1}/30) = .48 kPa
- q (^{1}/10) = .39 kPa
- Stud Spacing = 400 mm
- Storey Height = 2600 mm
- Slab thickness = 200 mm
- Interior and exterior sheathing provides adequate torsional restraint for loads not applied through the shear centre and for lateral stability.
- Deflections to conform to Clause 13 of CSA S304.1
- 1600 x 1600 window opening with sill height = 400 mm

DESIGN WIND LOAD (SEE NOTE ON PAGE 12)

CpCg. CpCg is determined as follows:

- Since H<20 m and H/W<1 use low rise pressure coefficients (NBC Supp/90 Fig. B-8)
- Conservatively assume 1 m^2 tributary area
- At corners (Z Distance) CpCg = -2.1 or +1.8
- At typical wall CpCg = -1.8 or +1.8

Ce. Ce is determined as follows:
Reference height for determining external pressure equals the eave height (NBC Supp/90), H = 15 m.

$$\begin{aligned} Ce &= (H/10)^{1/5} \quad \text{(OBC/90 4.1.8.1 (5) (b))} \\ &= (15/10)^{1/5} \\ &= 1.08 \end{aligned} \qquad [1]$$

Reference height for determining internal pressure equals H/2 (NBC Supp/90, Fig. B-12)

$$\begin{aligned} Ce &= (7.5/10)^{1/5} \\ &= 0.94 > .90 \text{ min.} \end{aligned} \qquad [2]$$

CpiCg. CpiCg is determined as follows:
Assume Category 1 building (NBC Supp/90, Commentary B)
CpiCg = 0.0 to -0.3

Design Wind Loads
Conservatively assume maximum corner load applies everywhere.

Design Wind Load for Strength

$$P = CpCg\ Ce\ q\ (^1/30) + Cpi\ Cg\ Ce\ q\ (^1/30)$$
$$= 2.1\ (1.08)\ (0.48) + 0.3\ (.94)\ (0.48)$$
$$= 1.22\ \text{kPa} \qquad [3]$$

Design Wind Load for Deflection

$$P = CpCg\ Ce\ q\ (^1/10) + Cpi\ Cg\ Ce\ q\ (^1/10)$$
$$= 2.1\ (1.08)\ (0.39) + 0.3\ (.94)\ (0.39)$$
$$= 0.99\ \text{kPa} \qquad [4]$$

$$P\ (\text{Defln})\ /P\ (\text{Strength}) = 0.81 \qquad [5]$$

STUD SELECTION FOR FULL HEIGHT WALL

- Height = 2600 mm - 200 mm = 2.4 m
- Spacing = 400 mm
- Specified Strength Load = 1.22 kPa
- Specified Deflection Load = 0.99 kPa

From a Manufacturers Wind Bearing Stud Allowable Height Table.
Trial Section 92 mm x 1.22 mm.

For Strength $W_f = 1.22\ (1.5)$
$= 1.83$ kPa [6]

From Table for $W_f = 2.15$ kPa
H max = 3.47 m > 2.4 m

For Deflection W = 0.99 kPa

Interpolating from table for allowable deflection = L /720
H max = 2.53 m > 2.40 m

Check Web Crippling Strength
From manufacturer's product literaure for a 92 x 1.22 stud.

$$P_r = 2.46\ \text{kN}$$
$$P_f = (0.4\ (2.4)\ (1.22)\ (1.5))/2 = 0.88\ \text{kN} < 2.46\ \text{kN} \qquad [7]$$

Conclusion
A 92 mm x 1.22 mm stud is suitable for the typical wall and requires no additional web stiffening.

BOTTOM TRACK SELECTION

Tests have indicated (Drysdale and Suter, 1991) for a typical stud to track connection (welded) or screwed, if the thickness of the track is equal to or greater than the thickness of the stud, then web crippling strength can be developed.

Conclusion
Use 92 x 1.22 track to match a 92 x 1.22 stud.

WINDOW FRAMING DETAILS

Refer to Fig. 1 for window details.

Distribution of wind loads on glass to supporting members

$$\text{Height/Width} = 1600/800 = 2 > 1.5 \quad [8]$$

- Use 2 way assumption for wind load i.e. glass spans between jamb and mullion.
- Mullion Loading and reactions (Refer to Fig. 2a)
  $$W = 1.22(0.8) = 0.98 \text{ kN/m} \quad [9]$$
- Sill and head loading and reactions (Refer to Fig. 2b)
  $$W = 1.22\ (0.2) = 0.24 \text{ kN/m} \quad [10]$$
- Jamb stud Loading (Refer to Fig. 2c)
  $$W_1 = 1.22\ (0.4 + 0.2) = 0.73 \text{ kN/m} \quad [11]$$
  $$W_2 = 1.22\ (0.4) = 0.49 \text{ kN/m} \quad [12]$$

Sill / Head Selection
Factored Bending Moment

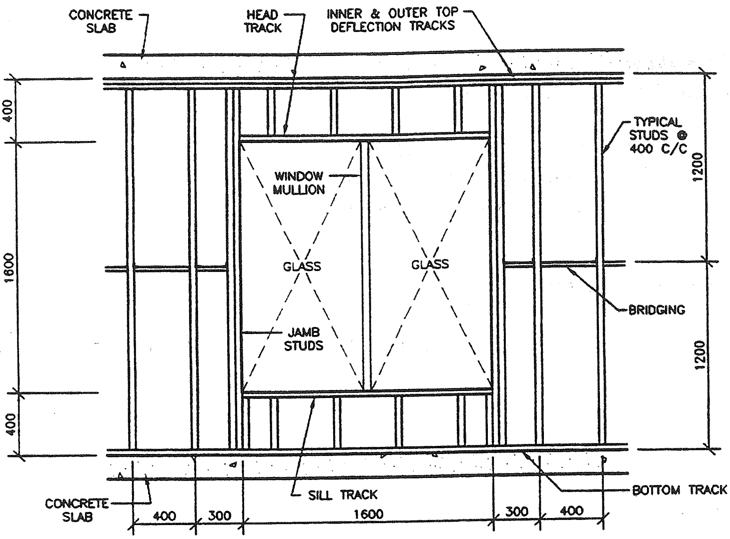
$$M_f = (1.5\ (.78)\ (1.6)\ /4) + (1.5\ (.24)\ (1.6)^2\ /8)$$
$$= .58 \text{ kN m} \quad [13]$$

Deflection

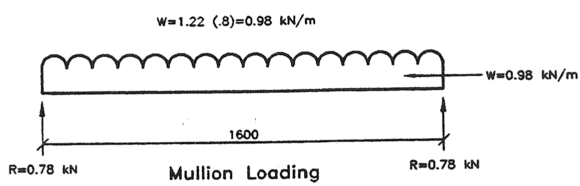
$$\Delta = (.78\ (1600)^3\ (10^3)\ /\ 48\ EI + 5\ (.24)\ (1600)^4\ /\ 384\ EI\ )\ q\ (1/10)\ /\ q(1/30)$$
$$= 0.353 \times 10^6/I \text{ mm/mm}^4 \quad [14]$$

For a Deflection Limit L /720

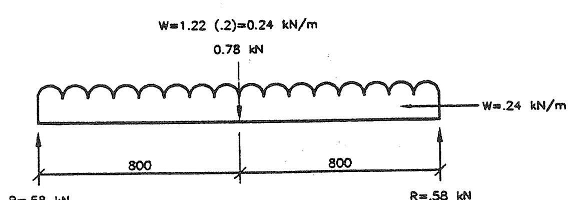
$$\text{Allowable } \Delta = 1600\ /\ 720 = 2.22 \text{ mm}$$
$$\text{Therefore Required } I = 0.353 \times 10^6\ /\ 2.22$$
$$= 159078.4 \text{ mm}^4 \quad [15]$$



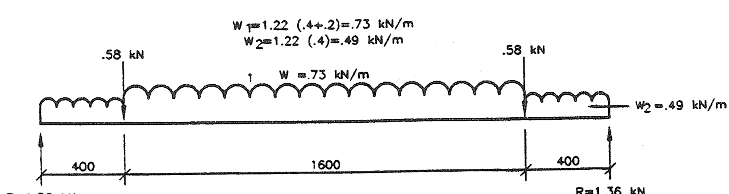
Stud Wall Elevation
Fig. 1



Mullion Loading
Fig. 2a



Sill and Head Loading
Fig. 2b



Jamb Stud Loading
Fig. 2c

From a Manufacturers Literature
Trail Section 92 mm x 1.22 mm Track

$$
\begin{aligned}
Mrx &= 0.85 \text{ kN m} > 0.58 \text{ kN m} \\
Vr &= 13.48 \text{ kN} > 1.5\ (0.58) \text{ kN} \\
Ix &= 220186 \text{ mm}^4 > 196036 \text{ mm}^4
\end{aligned}
$$

Jamb Selection
Factored Bending Moment

$$M_f = (1.5\ (0.49)2.4^2)\ /\ 8 + 1.5\ (0.24)\ (1.6)\ 2.4\ /\ 4 + 1.5\ (.58)\ .4 = 1.22 \text{ kN m} \qquad [16]$$

Shear Calculation

$$V_f = 1.5\ (1.36) = 2.04 \text{ kN} \qquad [17]$$

Deflection Calculation

$$\Delta = .81\ [5(.49\ (2400)^4\ /\ 384\ EI) + (0.24\ (1.6)\ (2400)^3\ 10^3\ /\ 48\ EI) + (.58\ (400)\ (3\ (2400)^2 - 4\ (400)^2)\ 10^3\ /\ 24\ EI)] = 1.51 \times 10^6\ /\ I \text{ mm/mm}^4 \qquad [18]$$

For a L / 720 Deflection Limit

$$
\begin{aligned}
&\text{Allowable } \Delta = 2400\ /\ 720 = 3.33 \text{ mm} \\
&\text{Therefore Required } I = 1.51 \times 10^6\ /\ 3.33 \\
&\qquad = 453453 \text{ mm}^4 \qquad [19]
\end{aligned}
$$

A built up member will be required. Refer to Fig. 3.

$$
\begin{aligned}
Mrx &= 2\ (1.38) + .854 \\
&= 3.61 \text{ kN m} > M_f \qquad [20]
\end{aligned}
$$

$$
\begin{aligned}
Ix &= 2\ (310509) + 220186 \\
&= 841204 \text{ mm}^4 > V_f \qquad [21]
\end{aligned}
$$

$$
\begin{aligned}
Vr &= 2\ [13.48] + 13.48 \\
&= 40.44 \text{ kN} > V_f \qquad [22]
\end{aligned}
$$

$$
\begin{aligned}
Pr &= 2\ (2.46) + 0 \\
&= 4.92 \text{ kN} > V_f \qquad [23]
\end{aligned}
$$

OUTER TOP TRACK SELECTION

The inner/outer track detail is required to accommodate slab deflection construction tolerance and anchor geometry.

For this example allow for a 12 mm ± slab deflection and a 8 mm ± tolerance. At time of installation the deflection space should be 20 mm but due to the tolerance could be a minimum of 12 mm to a maximum of 28 mm. The design deflection gap should be 20 + 8 + 12 = 40 mm which could occur if the base deflects and the top slab does not. (Refer to Fig. 4). In addtion we will include 4 mm to accomodate the head of the connection anchor.

Size Outer Top Track

One leg of the outer top track is assumed to be loaded uniformly by the inner top track which spreads the concentrated reactions from the studs (Refer to Fig. 5).

$$Pf = (2.4)\,(1.5)\,(1.22)\,/2$$
$$= 2.20 \text{ kN/m}$$
$$= 2.20 \text{ N/mm}$$

$$M_f = 44\,(2.20)$$
$$= 96.8 \text{ N mm/mm}$$

M_r is based upon the elastic section modulus and assuming a steel yield = 345 MPa

For t = 1.52 mm

$$M_r = .9\,(bt^2/6)\,Fy = .9\,(1)\,(1.52)^2\,345\,/\,6$$
$$= 119.56 \text{ N mm / mm} \qquad [24]$$

Horizontal movement can be calculated as follows (Trestain, 1991)

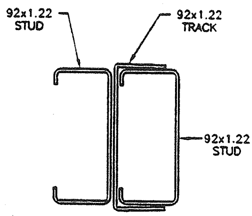
$$\Delta = (P/EI)\,[(L_2{}^2L_1/8) + (L_2{}^3/3)]\,(.81)$$

Where:

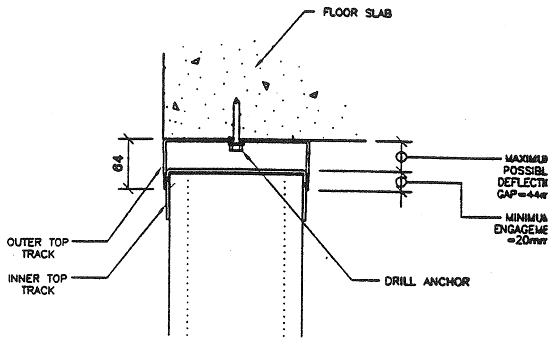
$P = 1.47$ N/mm
$L_1 = 92$ mm Track Width
$L_2 = 44$ mm Max Gap
$E = 203\,000$ MPa
$I = bt^3/12$
$= .293$ mm^4 /mm [25]

$\Delta = (1.47/EI)\,(44^2(92)\,/\,8 + 44^3\,/\,3)\,(.81) = 1.01 \text{ mm} \approx 1\text{mm}$

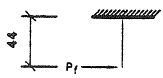
Note: 1 mm is an arbitrary parameter chosen to control cracking and displacements of air barriers, sealants and sheathing.



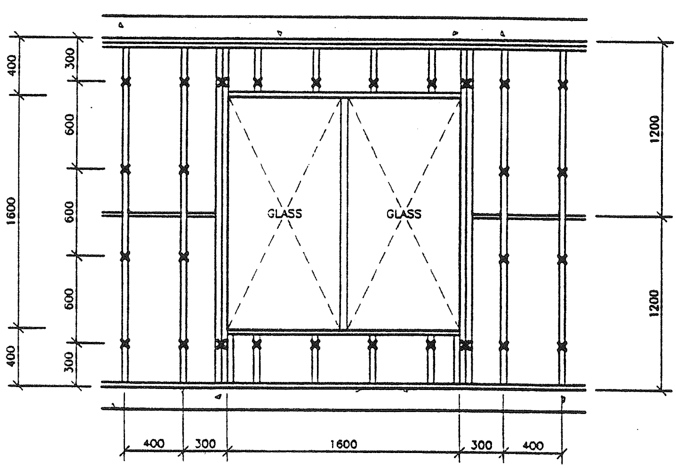
Built Up Member
Fig. 3



Top Track Configuration
Fig. 4



Single Leg Loading
of Outer Top Track
Fig. 5



Brick Tie Spacing
Fig. 6

BRICK TIE SELECTION

Tie Design Load

Subclause 13.2.3 of S304.1 dictates tie loads as follows: "all ties shall be designed for 40% of the tributary lateral load on one vertical line of ties, but not less than double the tributary lateral load on the tie."

Using this Clause and the maximum vertical tie spacing of 600 mm and the maximum horizontal tributary width which occurs at the jamb the tie design load is the greater of:

$$P_{tie} = .5\,(.4)\,(2.4)\,(1.22) = .59 \text{ kN} \qquad [26]$$

or

$$P_{tie} = 2\,(.5)\,(.6)\,(1.22) = .73 \text{ kN} \qquad [27]$$

Tie Selection

Clause 13.3.3 Note (2)(b) of S304.1 requires that "tie deflection due to one half of the total mechanical play plus a tension or compression load of 0.45 kN does not exceed 1.0 mm."

It would be necessary to select a tie that meets this stiffness requirement.

The factored tie load is 1.5 x .73 = 1.10 kN [28]

The factored resistance of a tie is given in CSA A370-94 clause 8.4.2.1.2 as the ultimate test strength multiplied by the appropriate resistance factor, ϕ. It would be necessary to select a tie that meets this strength requirement.

Tie Spacing

CSA Standard S304.1 calls for maximum tie spacing at 600 mm on centre vertically and 800 mm on centre horizontally and that the tie spacing shall not be staggered.

Given our 400 mm stud spacing we are therefore compelled to space the ties at 600 on centre vertically and 400 on centre horizontally.

CSA Standard A370-94 "Connectors for Masonry" in Clause 6 dictates spacing of ties adjacent to openings at a maximum of 600 mm apart and not more than 300 mm from the edge of openings. In addition the distance from the top of the wall to the first row of ties is not to exceed 300 mm and the distance from the bearing support to the first row of ties above shall not exceed 400 mm where the bearing support does not provide adequate lateral resistance.

Given a slab thickness of 200 mm the first tie could be spaced 300 mm above the slab if the masonry is supported on a ledger angle detailed to provide adequate lateral resistance at the underside of slab. The resulting tie spacing is as indicated on Fig. 6.

NOTE REGARDING DESIGN WIND LOAD

The National Building Code of Canada 1990 requires only q (1/10) for cladding. However, this example follows the guidelines issued by CSSBI (Trestain 1991) which uses q (1/30) for the strength design of steel stud infill assemblies.

REFERENCES

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