STRUCTURAL UPGRADING

OF

STEEL STUD-BRICK VENEER CLADDING SYSTEMS

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ABSTRACT

Many residential and office buildings in Canada constructed in the 1970's and early 1980's incorporated steel stud-brick veneer cladding systems. The steel stud backup and brick veneer were usually connected by means of corrugated strip ties. The brick veneer was assumed to transfer the lateral load to the steel stud backup. A number of such cladding systems are experiencing difficulties mainly due to tie corrosion, lack of ties, frame shortening and deflection of structural members supporting the cladding, such as beams and slabs. Guidelines and recommendations for retrofitting such distressed cladding systems are introduced.

This paper also proposes a retrofitting approach that uses a stiff shear connector to be attached to the web or flange of the steel stud and embedded in the brick veneer. This system makes use of the veneer as a structural component working with the stud to resist lateral loads at the same time affording a chance to examine the studs. An experimental program carried out to test the moment capacity of shear connectors attached to the flanges of metal studs is presented and an elastic stress analysis evaluation of the system is also carried out.

Keywords: Steel stud, cladding, retrofitting cladding, brick veneer, cavity wall, cladding rehabilitation, masonry cladding, shear connectors

INTRODUCTION

In the 1960's and 1970's a large number of residential and commercial structures were constructed in Canada and the United States of America with cladding systems consisting of steel stud and brick veneer. The steel studs were designed for a deflection requirement of L/360 or less, and assumptions in designing the system included composite action between the steel studs and the drywall fastened on both sides of the steel studs by means of drywall screws. The thickness of the steel studs commonly used was between 0.45 mm and 0.91 mm (26 to 20 gauge). The assembly was usually nested and fastened into top and bottom tracks of similar thickness, which were then secured to the main structural elements (slabs, beams) typically by power activated small diameter fasteners. The veneer was designed to transfer all lateral loads to the backup studs by means of corrugated metal strip ties spaced at 400 mm horizontally and 600 mm vertically.

A number of these types of structures are now experiencing distress largely due to frame shortening, tie corrosion, absence of ties, and deflection of supporting elements which cause the backup assembly (steel studs) to deform. The absence of horizontal control joints to accommodate frame shortening and deflection of the structural main elements has caused loads to be transferred to the veneer. The latter as a result in many cases is placed under substantial vertical load. In the absence of masonry ties or where the ties are inadequate and deteriorated, this load at times has stabilized the veneer acting as a post-stressing force. Construction tolerances have also contributed to the problem, mainly by causing excessive or undersized cavities to occur for which no allowance was made in the design.

During the last few years considerable knowledge has been gained and changes have been introduced to the design philosophies of these assemblies[1, 2, 3]. Such changes are expected to reduce or eliminate the failures encountered in the past. The overall allowable lateral deflection of the masonry veneer under service load is now generally limited to L/720. It is also recognized that composite action between steel studs and gypsum sheathing is not assured nor rationally quantified. The top connections of both the veneer and backup steel studs to the building structural elements are currently designed to accommodate vertical deflection and deformation of these main supporting elements. The interaction of the ties fastening the veneer to the backup system is also taken into consideration in the design of these assemblies. Although such changes will improve the design of steel stud-brick veneer (SS-BV) wall systems, the repairs and rehabilitation of distressed existing structures have received limited attention.

Retrofitting existing structures or repairing veneer assemblies that are experiencing distress requires the employment of techniques and procedures that can provide verifiable results and at the same time can be shown to satisfy sound engineering principles[4, 5]. This paper proposes a retrofitting approach that uses a stiff shear connector to be attached to the web or flange of the steel stud through small openings in the brick veneer. This system makes use of the veneer as a structural component working with the stud to resist lateral loads at the same time affording a chance to examine the stud from the outside of the building. An experimental program was carried out to test the moment capacity of shear connectors attached to the flanges of metal studs. The results of this test program are presented as well as an elastic analysis investigation of the stresses and deflections in the retrofitted system. Guidelines and recommendations for retrofitting such distressed cladding systems are also presented in this paper.

SIGNS OF DISTRESS

Cracking, bulging and spalling are the most common signs of structural distress of masonry veneer. Complete failure of parts of the veneer has also occurred in a number of cases (Calgary 1991 and Winnipeg 1990). At times loads transferred through the ties to the veneer or deformation of the backup steel stud system caused by frame shortening or deflection of the slabs or beams supporting the steel studs, have resulted in visible outward ridges (bulging) in the veneer. These deformations and movements are noticeable in the vicinity of the steel shelf angle supporting the veneer at floor levels. This is the result of eccentric loading on the veneer caused by deflection of the shelf angle combined with the lack of a control joint below the shelf angle. Small amounts of mortar placed between the shelf angle and the veneer will also provide a load path for eccentric loading on the assembly further complicating the state of stress within the veneer.

Crushing of the interior gypsum sheathing and popping of the drywall screws are also a signs of structural distress within the assembly.

Water penetration or an excessive amount of air infiltration or ex-filtration may cause structural damage which may be difficult to detect, as it affects the structural components supporting the cladding. Corrosion of ties, fasteners, studs and tracks can be linked to excessive air leakage and the incorrect location of the dew point within the wall system. This especially frequent for walls incorporating insulation only within the stud space. For these walls the dew point is within the stud space, and thus the possibility of corrosion is much higher than in walls with the dew point on the exterior side of the backup steel stud wall and within the cavity insulation.

The need for repairs must be established after careful evaluation of the condition of the entire system. This can only be accomplished by a systematic study of the condition of all components and the factors which contributed to the distress. Remedial actions must account for and address the effects of the remedial work on the expected performance of all parts of the assembly. If for example, the distress of a brick veneer is attributed to frame shortening, then providing a control joint at the floor level. The axial load carried by the brick veneer will be transferred to the steel studs. The latter may now be distressed. If the ties are corroded or inadequate, removing the vertical load acting on the veneer can result in a situation where total collapse may occur. In addition, because of tolerances, the condition of the cavity and its size may vary along the height of the building and the ties may be inadequate once the veneer is released of the unintentional vertical load transferred due to frame shortening.

RETROFITTING OPTIONS

At present no specific guidelines exist in retrofitting SS-BV wall assemblies. A variety of retrofit anchors are available for assemblies where the backup consists of solid walls such as concrete, concrete blocks, or structural tile.

For such walls, strengthening devices are installed by drilling through the wall and either mechanically (expansion) or chemically (epoxy) affixing the ties to the backup wall, relying on either adhesion or expansion forces to secure the veneer. This method has been reported to provide adequate results for such backup wall systems. However, this type of strengthening of SS-BV assemblies is limited especially when retrofitted from the exterior of the structure for a number of reasons. It is difficult to locate the steel studs when drilling. However, if the drill does locate a stud, then removal of a large part of the flange of the stud will reduce its ability to resist other loads. Without uncovering the assembly, the condition of the different components can not be verified.

Strengthening the assembly from the interior will result in similar and additional difficulties and costs, with the penetration of the air barrier and damage to interior finishes being the most important. Factors such as occupancy disruption and making good on retrofitting operations also make the process of repairing from the interior very unattractive.

If severe corrosion of the steel stud assembly is not evident, then the backup assembly should be structurally evaluated to determine if it is adequate to carry the lateral loads acting on it. This evaluation should require that the load be transferred to the backup system via proper ties. If the backup system is found structurally inadequate, then strengthening of the wall system is required.

For adequate backup wall strength cases, the assembly can be retrofitted with appropriate ties spaced at 600 mm vertically and 800 mm horizontally overall with the spacing being closer at the top and bottom supports in the manner recommended by CSA CAN3-A370 "Connectors for Masonry." During retrofitting the fastenings of the tracks at the top and bottom must be examined and repairs be made if required. Control joints along the length of the wall at the top of veneer panels can be examined, and introduced if required, but only after the backup system is repaired, so as to relieve the stresses arising in the veneer due to frame shortening and deflection of the supporting elements. Placement of a section of membrane between the gypsum sheathing and the retrofit tie (to protect the gypsum sheathing), may be considered.

If the backup system is found to be structurally inadequate to receive the lateral loads from the veneer, i.e. excessive lateral deflection of the steel stud backup wall is expected, one retrofitting option is the introduction of connectors which can transfer shear across the cavity, allowing the assembly to act as a vertically spanning Vierendeell Truss. This allows the backup system to make rational and controlled use of the veneer as a structural wythe. The proposed system is shown in Fig. 1. These ties can be attached to the backup system in one of two ways.

The approach is quite simple. Small openings are made in the brick veneer at the stud locations. This allows direct examination of the condition of the studs from the outside without disrupting indoor activities. The connectors are then fixed to the studs with metal screws. The gypsum sheeting and vapor barrier should then be repaired The other end of each connector is lastly embedded in the mortar layers of the bricks used to close the small openings.

Ties can be fastened to the web of the studs as shown in Fig. 2(a). Alternately, ties can be placed on the flange of the studs as shown in Fig. 2(b). Details of this tie are shown in Fig. 3. This is possibly a better system since the damage to the insulation gypsum sheeting and vapor barrier is minimal. In this case, however, it is recommended that the gypsum sheathing in the cavity be protected by placing a piece of self-adhesive vapor barrier material on the sheathing at the contact location of the connectors. The moment capacity of this particular tie (Fig. 2b) was examined in a test program at the University of Alberta. Four sizes of metal studs were tested in that investigation. The results will be presented in detail later on in this paper.

The structural performance of the system retrofitted by any one of the above methods can be evaluated using a plane frame analysis computer program. The properties of the components used in this analysis must be carefully chosen to reflect the true system as retrofitted. The interaction of the ties and screws with the components must be based on experimental evaluation of the capacities, such as the evaluation conducted on the ties attached to the flanges of the metal studs.

The following sections present the experimental test program and the methodology and results of a numerical analysis investigation of steel stud-brick veneer assemblies retrofitted with these ties.

EXPERIMENTAL PROGRAM

Sixteen tests were performed on shear connector plates fastened to the flanges of steel studs to investigate their ability to transfer shear. This surface mounted shear connector is shown in Fig. 3. Four thicknesses of steel studs (14, 16, 20, 26 gauge) were tested, with four tests performed on each stud size. The connectors and steel studs were separated by 12.7 mm gypsum sheathing and a vapor barrier (thermal barrier 400) as shown in Fig. 4.

The load frame, shown in Fig. 5, enabled 600 mm long sections of steel stud to be securely fastened to a rigid surface via four bolts through the center of the web of the steel stud. Two bolts, spaced 102 mm apart anchored the steel stud at each end, leaving an unsupported web length of 254 mm between end connections. Blocks of wood were inserted between the flanges at either end of the steel stud giving the flanges an unsupported length of 450 mm.

The shear connectors were fastened at the center of the steel studs using two self-tapping sheet metal screws (one at each end of the shear connector plate), which penetrated the connector, vapor barrier, sheathing and flange of the steel stud. The tensile load was applied to the end of the connector by the "turn of the nut" method. Load and displacement at the end of the shear connector were monitored.

TEST RESULTS

Different failure models were prevalent for steel studs of different thickness. The failure of the connectors attached to 26 gauge steel studs came from the pull out of the sheet metal screws at moments between 0.024 and 0.038 kN.m. Cracks in the gypsum sheathing originated at the toe of the shear connector, just prior to failure for some specimens and at failure for others. The maximum moments for all the shear connectors are shown in Table 1. All 26 gauge steel stud connections were able to sustain deflections of at least 11 mm.

The specimens constructed with 20 gauge steel studs again failed because of the pull-out of the sheet metal screws. Failure occurred between 0.073 and 0.080 kN.m. The gypsum sheathing cracked prior to pull out of the sheet metal screws at approximately 0.055 kN.m. The cracking of the sheathing had no adverse affects on the performance of the connections, as is indicated by Fig. 8. Figure 8 shows representative moment versus displacement curves for specimens with all tested thicknesses of steel studs. There is no discernible deviation from the moment versus connection curve when the gypsum sheathing cracked. All 20 gauge stud connections were able to sustain deflections of at least 20 mm.

The specimens made with 16 gauge steel studs all experienced failure of the connector itself. The sheathing cracked between 0.068 and 0.078 kN.m and the connectors buckled at moments between 0.092 and 0.117 kN.m. All 16 gauge stud connections were able to sustain deflections of at least 14 mm.

Three of the four specimens constructed with 14 gauge steel studs experienced failure by buckling of the connector and one failed due to screw pull-out, but not before the sheathing was crushed. The crushing of the gypsum sheathing was acknowledged when one end of the shear connector had visibly sunk into the sheathing. Cracking of the sheathing occurred at moments between 0.059 and 0.071 kN.m, crushing of the sheathing occurred between 0.078 and 0.098 kN.m and buckling of the connector occurred between 0.092 and 0.106 kN.m. Specimen D1 did not experience connector buckling, but instead failed due to screw pull out. Failure of specimen D1 occurred at 0.117 kN.m. All 14 gauge stud connections were able to sustain deflections of at least 19 mm.

FRAME MODEL

Figure 7 shows schematically the Vierendeell Truss model used to analyze the system. The model is represented by solid lines, while the actual components of the cavity wall are represented by dotted lines. Beam elements are used to model all of the wall components. The elements in the brick veneer and in the steel stud are assumed to span between connector locations. Each shear connector is modeled with four elements as shown in Fig. 7. The first element is the portion of shear connector plate that is connected to the steel stud. A large stiffness is assigned to this element (two orders of magnitude larger than the actual tie stiffness). The second element consists of the portion of the shear connector plate in the cavity. The stiffness of this portion of the plate, as obtained experimentally (Wang 1996), is EA = 600 kN, EI = 18.5 kN.m^2. The third element is the portion of wire in the cavity which is assumed to be 13 mm long. For this element the actual properties of the wire are used. The fourth element is the portion of the wire (V-tie) embedded into the mortar bed of the brick veneer, and is assigned a higher stiffness than the portion of wire in the cavity. All the connections between the elements are assumed fixed, except the connection of V-tie to the shear connector plate which is modeled as a pin joint. The modulus of elasticity for the brick veneer and the steel studs are 3,000 MPa and 200,000 MPa respectively.

The boundary conditions for the frame model are also shown in Fig. 7. Both the veneer and steel studs are modeled as hinged at the bottom end. The top reaction of the steel studs is simulated by a roller which allows vertical movement of the studs. The brick veneer is modeled as free at the top. To model the partial fixed condition at the bottom ends of the steel studs and the brick veneer, rotational springs can be inserted at the bottom hinged ends. This, however, was not considered in this investigation.

Uniformly distributed lateral load along the height of the brick veneer is applied to simulate the lateral wind pressure. Self-weight of the brick veneer and steel stud backup wall are also included in the analysis.

EVALUATION OF RETROFITTED ASSEMBLIES

Consider the assembly shown in Fig. 8, which is a typical exterior wall of residential structures of the 1970's. Assume that the wall is 2400 mm high and consists of 90 mm veneer, 25 mm airspace, building paper, 13 mm exterior grade gypsum sheathing, 102 mm wide by 0.91 mm (20 gauge) thick steel studs, 6 mil polyethylene and painted sheathing. Assume that the veneer is attached to the studs by means of 26 gauge (0.45 mm) mill galvanized corrugated strip ties, that the corrugated strip ties are found to be severely corroded in the mortar joint as a the cavity, as built, is larger than the 25 mm specified in the construction drawings and specifications. This assembly is deemed structurally inadequate and retrofitting is required.

Recognizing that ties at the top and bottom of the walls must resist large loads prior to cracking of the veneer, the vertical spacing of the retrofit ties is at 200 mm from the shelf angle for the first tie, the second tie is placed at 400 mm above the first tie and the remaining ties are spaced at 600 mm vertically until the pattern is reversed at the top of the wall. The horizontal spacing of the ties is 800 mm and thus every second stud is used in the retrofitting process. This arrangement is shown in Fig. 9(a).

Using the properties of the components listed in the previous section with the vertical arrangement of the ties shown in Fig. 1, the wall is analyzed under the action of a 1.0 kPa wind pressure. The maximum tensile stress in the brick veneer is 0.179 MPa and the maximum lateral deflection of the assembly is 0.292 mm. The maximum bending moment in a single stud is 0.216 kN.m. The allowable moment according to supplier tables is 0.679 kN.m. There is also an axial force in the stud, because of the Vierendeel action. This axial force has a maximum value of 2.08 kN over a short distance. If one uses the same allowable stress and the corresponding stud properties, one finds that this stud is now adequate with a P/P_a + M/M_a=0.416. The maximum moment in a stud connector joint is 0.086 this is slightly higher than that detected in the tests of 20 gage steel stud-connector joints reported above. Perhaps then a closer spacing of the connectors (600 mm) could be used under these conditions.

Table 2 contains the results of an elastic analysis of 1000 mm long sections of walls under 1.0 kPa wind pressure using the same model described above for a selection of wall assemblies retrofitted using the proposed method. The studs are all 102 mm deep. The stud gages were varied to have values of 26, 25, 22, 20 and 18. Four wall heights are used; 2400, 2600, 2800 and 3000 mm. The analysis is repeated for connectors spaced horizontally at 400 mm, 600 mm and 800 mm. Studs are normally spaced at horizontal spacing of 400 mm. A 600 mm connector spacing is achieved by connecting two out of every three studs as shown in Fig. 9(b). The model used here ignores the contribution of studs that are not tied to the brick veneer with a shear connector. It could be argued that this is a conservative approach, since the other studs could conceivably share the load through the gypsum board. The table reports the maximum tensile stress in the brick veneer, the maximum deflection and the maximum (P/Pa + M/Ma) ratio for the studs. Table 2 shows that the brick veneer is safe in almost all cases. The deflections are also well within a limit of L/700. The studs, however, exceed the safe limits under the reported conditions for some of the cases where the connectors are spaced at 600 mm and 800 mm for gages 26 and 25. The reader should remember that unconnected studs were ignored in the analysis. More work is needed to ascertain the load sharing afforded by the gypsum boards both exterior and interior.

The maximum tensile force in the screws required to develop the computed moments is 411 N (maximum at the top and bottom connector location), which provides for a factor of safety of 2.8, as obtained by comparing this computed value with experimentally derived values listed in Table 3.

CONCLUSIONS

Structures with inadequate backup systems and/or ties can be retrofitted without completely removing the cladding system. The performance can be enhanced by connecting the veneer to the light steel framing with connectors capable of transferring shear, thereby attaining partial shear connection between the brick veneer and the steel studs. Blind restoration of such wall assembles or the incorporation of relatively large diameter fasteners through the flanges of the studs, cannot be relied upon and may further reduce the ability of the assembly to resist lateral loads by introducing hinges at the location of these penetrations.

Preliminary test and analysis results indicate that it is possible to retrofit steel stud-brick veneer assemblies by utilizing the structural properties of the veneer, ties and steel studs, provided that the induced loads are within the acceptable limits (safety factor). More work is needed to determine the degree of load sharing afforded by unconnected studs. Also more work is needed to determine the capacities of stud-connector joints on a more definitive manner specially for small thickness studs (gages 20, 22, etc.)

ACKNOWLEDGMENTS

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Table 1. Test Results of connector -stud specimens.

Specimen	Metal Stud Gauge	Max. Moment (kN.m)	Displacement at Max. Load (mm)	Mode of Failure
A1	26	0.029	11.8	Screw pull out
A2	26	0.033	14.2	Screw pull out
A3	26	0.028	n/a	Screw pull out
A4	26	0.035	11.6	Screw pull out
B1	20	0.079	21.8	Screw pull out and drywall cracking
B2	20	0.073	22.6	Screw pull out and drywall cracking
B3	20	0.073	20.8	Screw pull out and drywall cracking
B4	20	0.080	20.5	Screw pull out and drywall cracking
C1	16	0.092	22.7	Connector buckling and drywall cracking
C2	16	0.100	15.9	Connector buckling and drywall cracking
C3	16	0.117	n/a	Connector buckling and drywall cracking
C4	16	0.094	14.4	Connector buckling and drywall cracking
D1	14	0.117	25.3	Screw pull out and drywall crushing
D2	14	0.106	24.4	Connector buckling and drywall crushing
D3	14	0.092	19.3	Connector buckling and drywall crushing
D4	14	0.106	n/a	Connector buckling and drywall crushing

Table 2 Performance of brick veneer-metal stud system retrofitted with shear connectors

Connector spacing		400 mm			600 mm			800 mm		
Height (mm)	Stud gage	ft (brick) MPa	Deflection mm	Stud P/Pa+M/Ma	ft (brick) MPa	Deflection mm	Stud P/Pa+M/Ma	ft (brick) MPa	Deflection mm	Stud P/Pa+M/Ma
2400	18	0.098	0.244	0.172	0.134	0.327	0.245	0.163	0.399	0.315
	20	0.111	0.270	0.225	0.149	0.360	0.321	0.179	0.437	0.413
	22	0.140	0.333	0.372	0.181	0.439	0.532	0.212	0.529	0.685
	24	0.167	0.398	0.548	0.209	0.520	0.784	0.240	0.624	1.010
	26	0.173	0.414	0.593	0.216	0.541	0.848	0.246	0.648	1.093
2600	18	0.100	0.286	0.203	0.139	0.386	0.291	0.171	0.473	0.374
	20	0.115	0.319	0.265	0.156	0.429	0.379	0.190	0.524	0.487
	22	0.149	0.401	0.433	0.195	0.533	0.620	0.231	0.645	0.796
	24	0.181	0.488	0.634	0.231	0.641	0.906	0.268	0.770	1.162
	26	0.189	0.509	0.685	0.239	0.668	0.977	0.276	0.801	1.254
2800	18	0.107	0.375	0.223	0.152	0.506	0.318	0.191	0.619	0.409
	20	0.124	0.418	0.291	0.174	0.561	0.416	0.214	0.684	0.534
	22	0.165	0.523	0.477	0.220	0.692	0.681	0.263	0.835	0.875
	24	0.205	0.632	0.699	0.264	0.827	0.996	0.308	0.988	1.278
	26	0.214	0.660	0.755	0.274	0.860	1.076	0.318	1.025	1.379
3000	18	0.130	0.473	0.245	0.175	0.636	0.350	0.213	0.778	0.450
	20	0.145	0.528	0.321	0.194	0.706	0.458	0.240	0.859	0.588
	22	0.185	0.660	0.526	0.248	0.870	0.752	0.298	1.045	0.964
	24	0.231	0.797	0.771	0.299	1.036	1.099	0.350	1.232	1.408
	26	0.242	0.831	0.833	0.311	1.077	1.186	0.361	1.278	1.519

Table 3. **Self Tapping Sheet Metal Screw Ultimate Tensile Pullout Loads**

Self Taping Sheet Metal Screw Size	Thickness of Steel Stud 14 ga. (1.73 mm)	16 ga. (1.44 mm)	20 ga. (0.94 mm)	26 ga. (0.45 mm)
#8	4185 N	3150 N	1290 N	n/a
#10	4425 N	3135 N	1245 N	740 N
#12	4290 N	2835 N	1800 N	n/a
#14	5010 N	3180 N	1575 N	1150 N

Note: **Loads have been modified (reduced) by applying correction factors as per CSA CAN3-A370.**

List of Figures

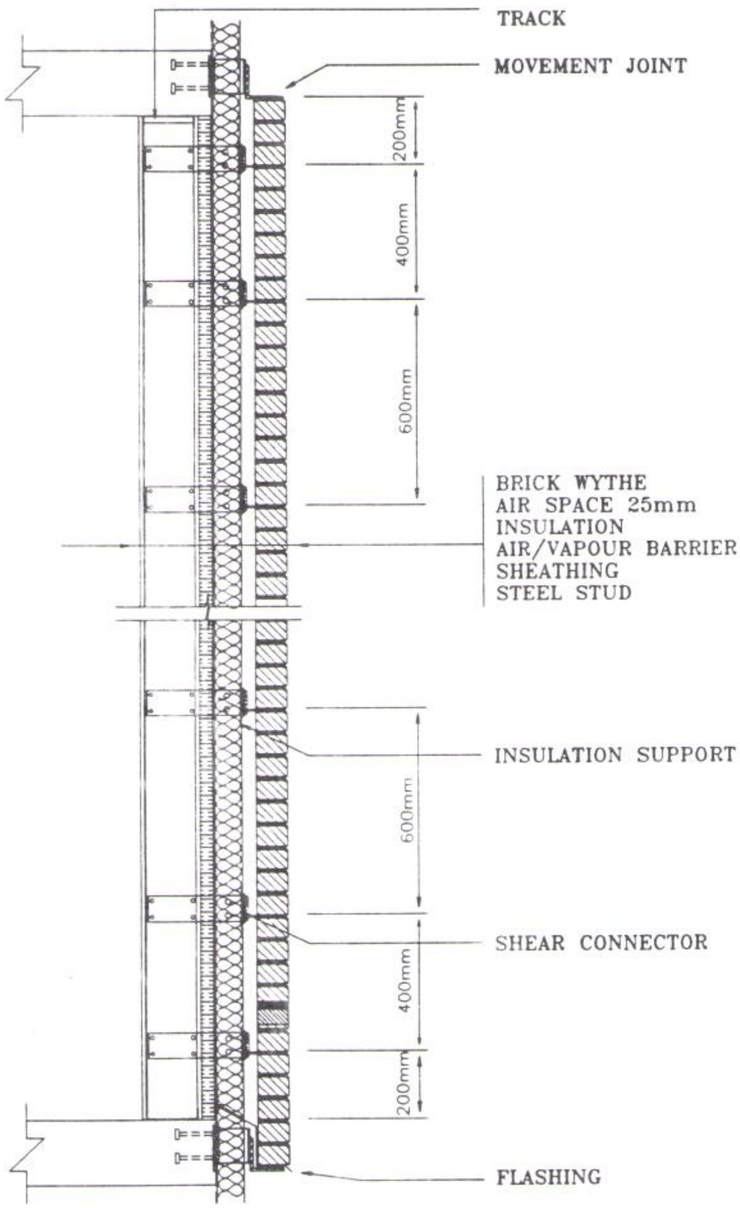
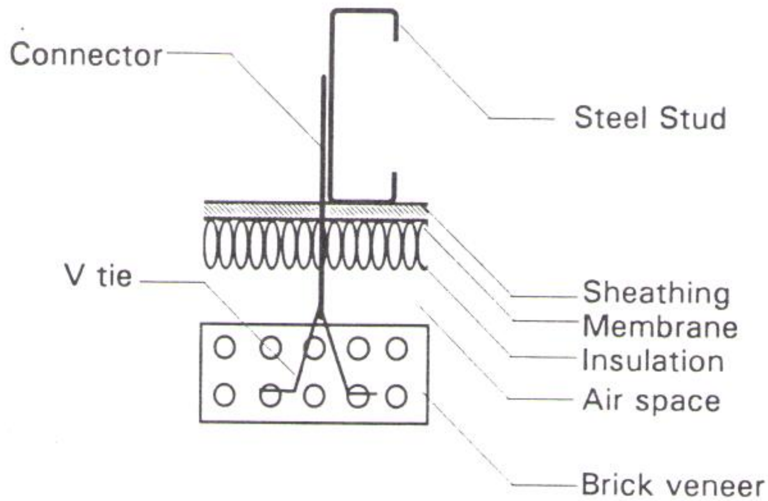
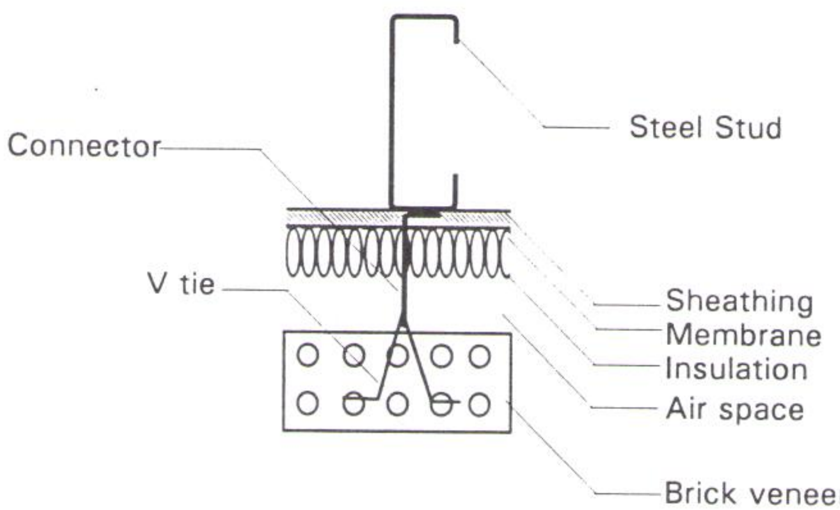


Fig. 1. Typical Location of Retrofitted Ties in Brick Veneer Steel Stud Wall System



(a) Connector attached to the web of the steel stud



(b) Connector attached to the flange of the steel stud

Fig. 2 Different approaches to attaching rigid connectors to steel studs

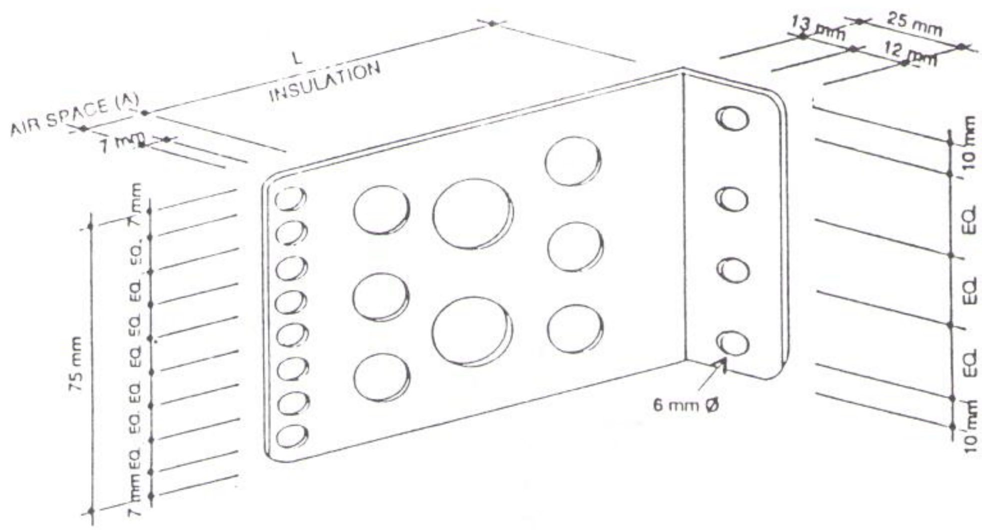


Fig. 3 Dimensions of the shear connector plate used to attach to the flange of the Steel Stud

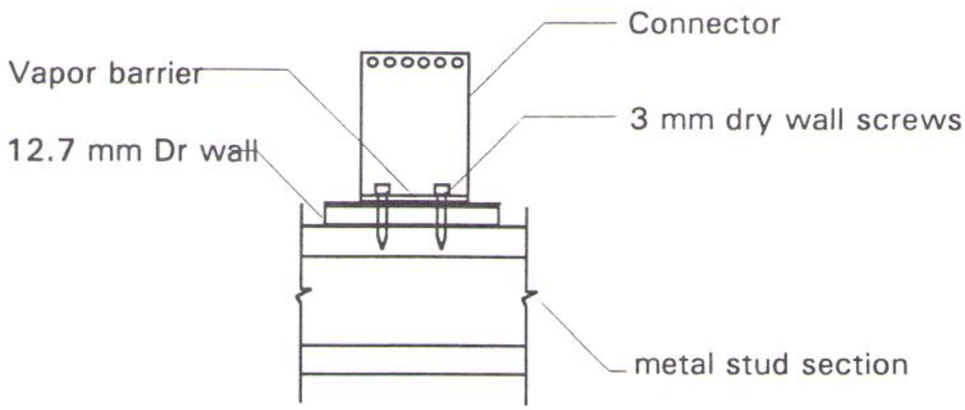


Fig. 4. Shear Connector Set Up

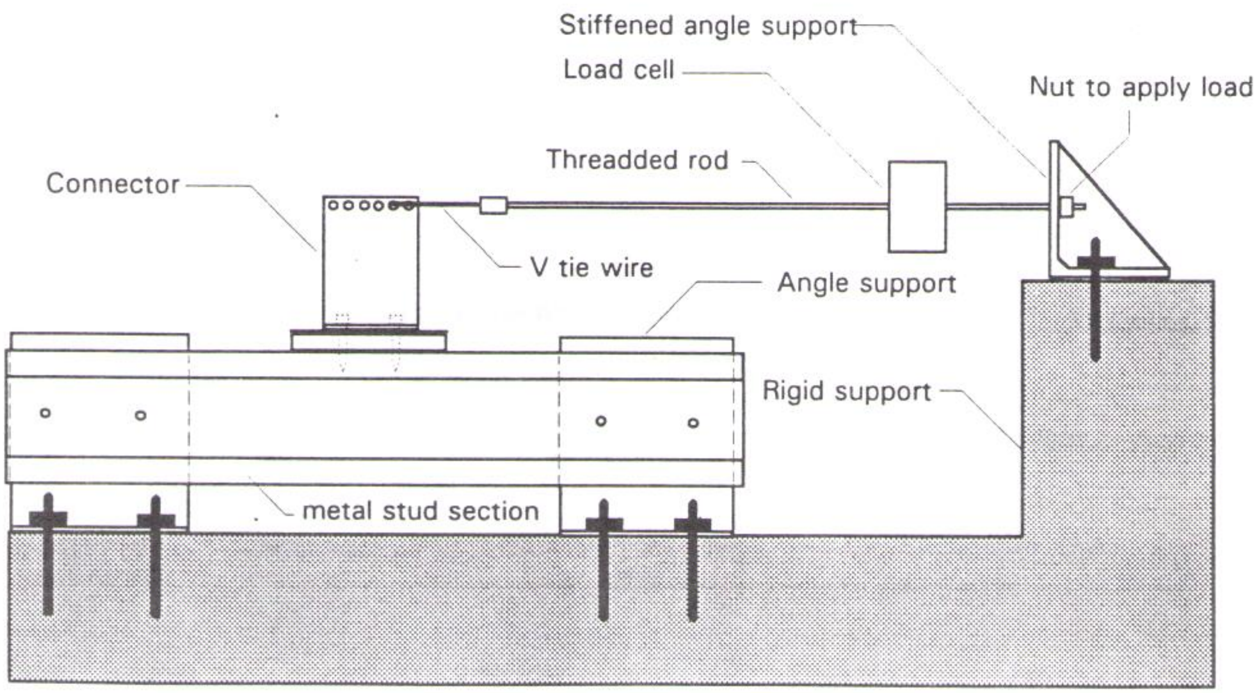


Fig. 5. Schematic of Load Frame

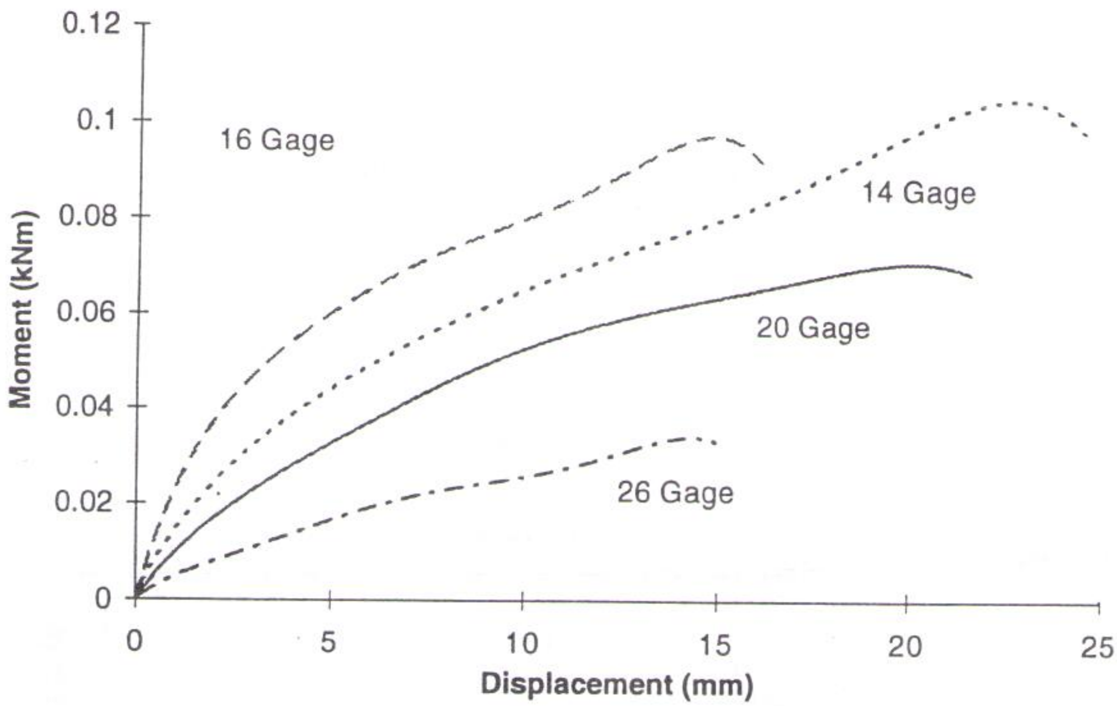


Fig. 6. Moment versus displacement curves for a representative tests

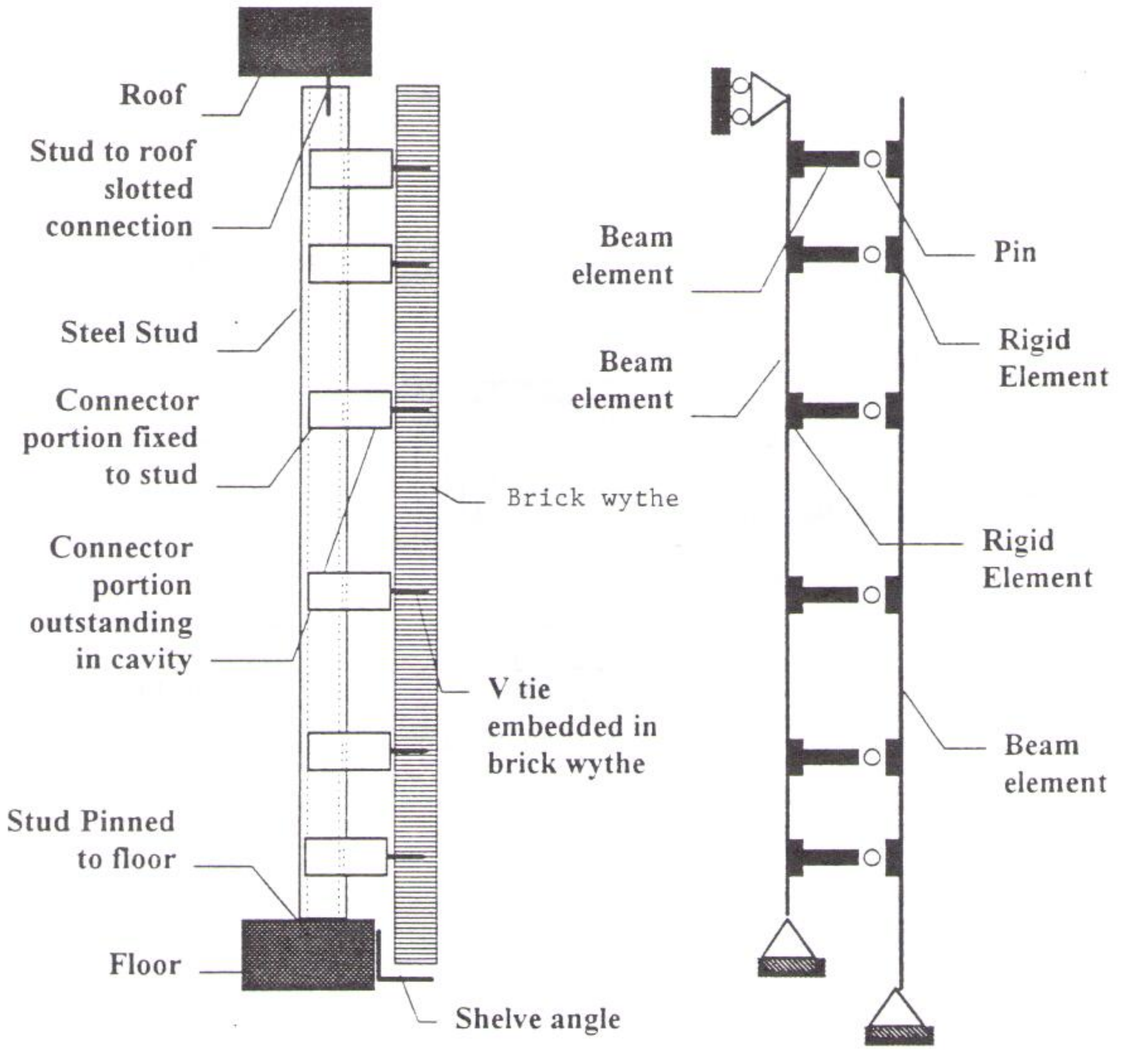


Fig. 7. Frame Model

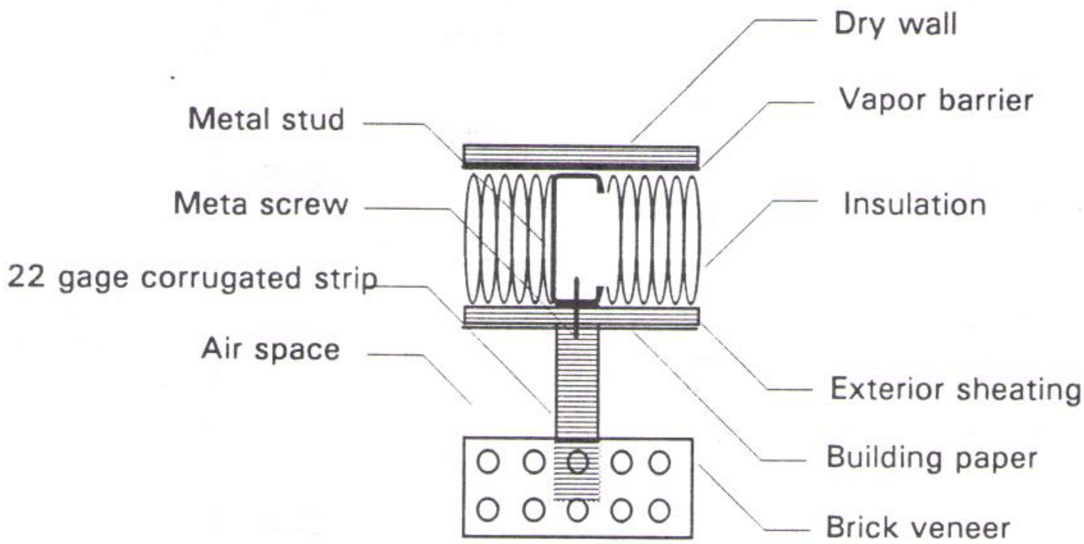


Fig. 8. Typical cross section of brick veneer-steel stud wall system of the 1970's and early 1980's.