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NEW STYLE VENEER TIE ANSWERS THE NEED FOR STIFFNESS AND ADJUSTABILITY

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ABSTRACT

Generally, cavity wall ties and veneer anchors need to provide a high degree of stiffness perpendicular to the plane of a wall while being flexible enough to accommodate in-plane differential movements between the wythes.

Neither the Canadian Standards Association (CSA) Standard for Connectors (1) nor the Masonry Standards Joint Committee (MSJC) Building Code Requirements for Masonry Structures (2) have specific performance requirements for cavity wall ties or veneer anchors. However, the Report on the Behavior of Brick Veneer/ Steel Stud Tie Systems (3) and the Brick institute of America (B.I.A.) Technical Notes on Brick Masonry No. 44, Wall ties for Brick Veneers, (4) recommends a minimum stiffness for the connectors.

Traditional two-legged 5 mm (0.188 in) wire pintle type anchors cannot meet the stiffness requirements recommended by the above authorities when the connector misalignment exceeds 19 mm (0.75 in). This paper reviews the results of a test program to evaluate the performance of a new sheet metal type pintle anchor used in conjunction with typical DUR-O-EYE adjustable type joint reinforcing and D/A 213 veneer anchor assemblies against the above requirements (5,6).

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INTRODUCTION

Two significant changes in the way that cavity walls and veneers are designed and built have caused designers to reevaluate the performance of traditional connectors. One change is the use of board-type insulation in cavity spaces between brick and its backup material. This increases in-plane differential movement between wythes. The other change is the use of flexible sheet metal steel studs for the backup of veneers. This increases the chances to develop bed joint cracking.

IN-PLANE MOVEMENT AND ADJUSTABILITY

The differential temperature movement between wythes of a cavity wall or between the veneer and its backup is increased dramatically when even small amounts of insulation is used in the cavity. The difference in temperature between the inner and outer wythes of a wall is almost three times greater in a wall with two inches of insulation as compared to the same wall without insulation (7). In this situation, the differential horizontal movement between wythes of an insulated wall will be about 3.8 mm (0.15 in) as compared to 2.0 mm (0.08 in) for an uninsulated wall when the spacing of vertical expansion joints is about 7 m (23 ft). Most two-piece anchors can accommodate this amount of horizontal movement.

When steel stud / brick veneers are built, the normal assumption is that a mason can easily locate a veneer anchor vertically within 13 mm (0.5 in) of its design position (aligned with the bed joint to eliminate eccentricity). When additional differential vertical movement is added for the temperature changes, the total design eccentricity should be about 16 mm (0.63 in). It is not the same with a brick and block cavity wall using metric and imperial sized units. In this case, due to reasons unrelated to thermal movements, there can be significantly larger misalignment of the brick and block bed joints resulting in cavity wall tie eccentricities of up to 32 mm (1.25 in).

Most two-piece adjustable connectors can meet these job conditions. Many of them, particularly those using wire triangular ties provide much more adjustability than is needed. This "overkill" can reduce the connector's out-of-plane stiffness in some situations.

OUT-OF-PLANE STIFFNESS

Connector stiffness has been identified as a significant factor in determining the distribution of loads from the outer wythe to its backup through the connectors. Stiffness also affects the loads applied to individual connectors. The B.I.A. has recommended that two-piece adjustable connectors have a minimum stiffness of 350 N/mm (2000 lb/in). Because it combines stiffness with free play in its requirements, the CSA Standard for Connectors effectively requires a much stiffer anchor. When an allowable free play of 1.2

mm (0.05 in) is used as allowed by the above standard, the anchor must have a stiffness of 563 N/mm (3200 lb/in).

LOADS AND DESIGN CRITERIA SUMMARY

Traditionally, veneer anchor and cavity wall tie loads are assumed to be based on their tributary area. When a 1 kPa (20 psf) wind load is imposed on a wall where the connectors are spaced every 0.19 m² (2.0 s.f.), the load per anchor is assumed to be 178 N (40 lbs). With a safety factor of 3, the ultimate strength needs to be 534 N (120 lbs). Additional connectors are used around the edges of wall panels to accommodate load concentrations in these areas. Table 1 shows a summary of the recommended in-plane and out-of-plane adjustability. It also shows the recommended stiffness and strength for a 1 kPa wind pressure when the CSA and MSJC Standards as well as the BIA recommendations are used.

TABLE 1 DESIGN CRITERIA

	CSA	MSJC/BIA
IN-PLANE MOVEMENT	3.8 mm (0.15 in)	3.8 mm (0.15 in)
VERTICAL ADJUSTMENT	32 mm (1.25 in)	32 mm (1.25 in)
ULTIMATE STRENGTH	534 N (120 lb)	534 N (120 lb)
STIFFNESS	563 N/mm (3200 lb/in)	350 N/mm (2000 lb/in)

TEST PROGRAMS

Based on the above conditions, DUR-O-WAL conducted two test programs (5,6) to compare the performance characteristics of a new design for pintle type anchors with traditional wire pintles. In the new design, Figure 1, a sheet metal pintle was used with its DUR-O-EYE and D/A 213 assemblies in lieu of traditional wire pintle because the strength and stiffness of a sheet metal pintle is not limited by the thickness of a joint or the

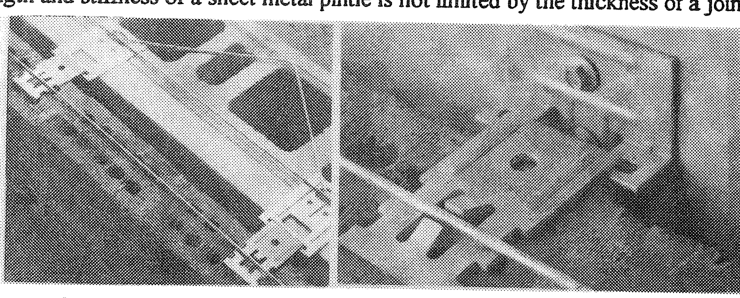

FIGURE 1 D/A 213 ANCHORS AND SEISMIC DUR-O-EYE

number of vertical pintle legs. In addition, sheet metal pintles could accommodate shear lugs. These shear lugs are required by some codes in seismic areas to engage joint reinforcement in veneers.

Tests were conducted on the D/A 213 assemblies since the back plate of this connector is more flexible than the rigid plate used in the DUR-O-EYE assembly. The first series of tests compared 1.9 mm (14 ga, 0.075 in) back plates with 2.8 mm (12 ga, 0.11 in) pintle plates. The second series tested 2.8 mm (12 ga,0.11 in) back plates and 3.0 mm (11ga, 0.12 in) pintles. In each series, the anchors were tested with three different eccentricities.

Test Series 1 and 2 were conducted at Iowa State University and used a test setup consistent with that used in a masonry industry sponsored test series (9) to determine the strength and stiffness of a number of common anchors. Figure 2 shows a schematic view of the Iowa State University test setup. A summary of the test matrix is shown in Table 2 for the anchor types and eccentricities used in the tests, along with the associated plate and pintle thicknesses.

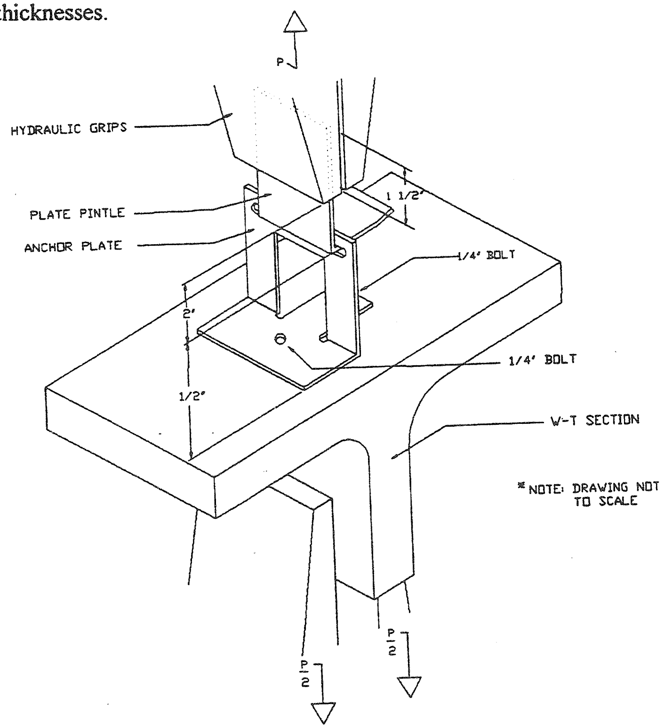

FIGURE 2 TEST SETUP (6)

TABLE 2 TEST MATRIX

ANCHOR SER/TYPE	PLATE THICKNESS	PINTLE THICKNESS	PINTLE ECCENTRIC.	NO. OF TESTS
1 D/A213-0	1.9 mm (0.075 in)	2.7 mm (0.105 in)	0	5
1 D/A213-2	1.9 mm (0.075 in)	2.7 mm (0.105 in)	0	5
1 D/A213-2	1.9 mm (0.075 in)	2.7 mm (0.105 in)	19 mm (0.75 in)	5
1 D/A213-2	1.9 mm (0.075 in)	2.7 mm (0.105 in)	32 mm (1.25 in)	5
2 D/A213-0	2.7 mm (0.105 in)	3.0 mm (0.120 in)	0	5
2 D/A213-0	2.7 mm (0.105 in)	3.0 mm (0.120 in)	19 mm (0.75 in)	5
2 D/A213-0	2.7 mm (0.105in)	3.0 mm (0.120 in)	32 mm (1.25 in)	5

The anchors were tested in tension utilizing a SATEC -- calibrated as a self-contained hydraulic machine. In another test series not reported here, the anchors were tested in compression with normal cavity widths. There was no significant difference noted (8). In Test Series 1 only the ultimate load was measured. This load is defined as the peak load before a negative slope was obtained in the Load/Deflection curve. This is similar to the REM load described below. Stiffness was graphically determined from the initial "elastic" portion of the curve.

In Test Series 2, with the heavy plate and pintle assemblies, the connectors were also tested in tension. These tests developed "peak loads and deflections" (PL), "reasonably expected maximum loads and deflections" (REM), and "reasonably expected elastic loads and deflections" (REEL).

The peak loads and deflections (PL) correspond to peak loads attained prior to a significant decrease in load or an abrupt failure point. If loading continued to increase substantially after a negative slope region, peak loads were taken at the higher load.

The reasonably expected maximum loads and deflections (REM) are the loads and deflections achieved at the end of the inelastic, ductile (somewhat plastic) region of the load - deflection behavior, beyond which much larger deflections occur. The REM is the load that represents the "interpreted maximum" load that should be safely or conservatively considered to be the practical ultimate load. In most cases, loads (peak loads) beyond REM were due to highly inelastic behavior, rotation, extra membrane force contribution, or exaggerated deflections that one would not want to count as part of the correct specimen's capacity. The REM loads are those recommended to which the appropriate safety factors should be applied to arrive at the design value for the connector. If the characteristic REM load is determined (average REM minus two standard deviations), this load could be used as a working load. Characteristic values are shown in Table 3.

TABLE 3 LOADS AND DEFLECTIONS TEST SERIES 1

ANCHOR TYPE	AVERAGE ULTIMATE LOAD	STD. DEVIATION S	CHARACTISTIC VALUE AVG - 2S	STIFFNESS
213 - 0 0 Ecc	3.28 kN (738.0 lb)	95 N (21.3 lb)	3.09 kN (695.4 lb)	NA
213 - 2 0 Ecc	4.23 kN (950.5 lb)	169 N (38.0 lb)	3.89 kN (874.6 lb)	1167 N/mm (6666 lb/in)
213 - 2 19 mm (0.75 in)Ecc	1.07 kN (240.9 lb)	49 N (11.1 lb)	0.97 kN (218.6 lb)	473 N/mm (2700 lb/in)
213 - 2 32 mm (1.25 in)Ecc	0.80 kN (178.9 lb)	48 N (10.7 lb)	0.7 kN (157.6 lb)	103 N/mm (588 lb/in)

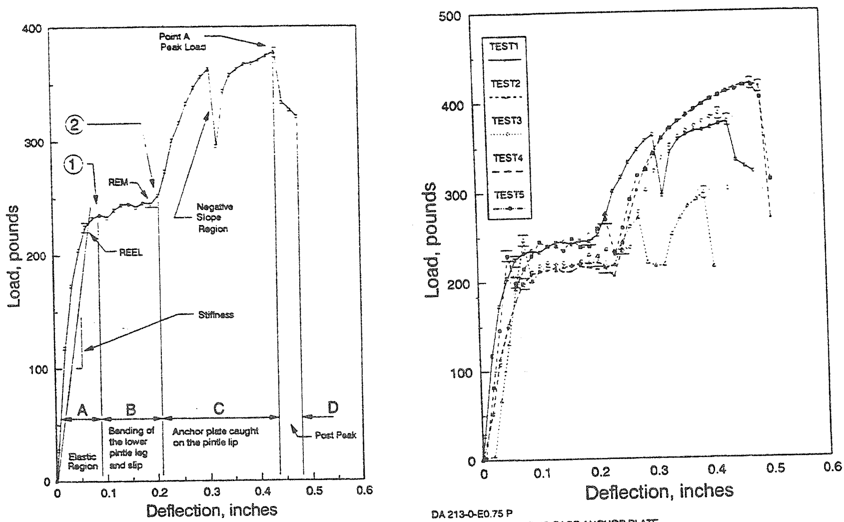

FIGURE 3 REPRESENTATIVE AND ACTUAL LOAD-DEFLECTION CURVES(9)

Figure 3 shows a representative curve for the assembly with 19 mm (0.75 in) eccentricity and a plot of the actual data for the same loading case. Note that the REEL, REM, and PL loads are depicted on the left illustration of Figure 3, as well as the associated Zones A,B,C, and D (elastic, inelastic,contributing inelastic and post peak zones) These zones are also depicted in a previous paper (9).

The reasonably expected elastic load (REEL) and associated deflections are the values taken at the end of the elastic region (the initial straight line portion of the graph) and the start of the inelastic region. The average stiffness shown in Table 3 is that stiffness associated with the elastic portion of the load-deflection curve. These are shown as a conservative prediction of possible design load values. A linear regression was used to obtain the slope of the straight line portion of the curves between zero and the REEL loads.

TEST RESULTS

The test results, shown in Tables 3 and 4 indicated that manufacturers can design anchors to meet a series of requirements once standards writing groups and regulatory bodies decide what properties are needed. In this case, the heavy duty plate and pintle assembly had a stiffness of 369 N/mm (2110 lb/in) at an eccentricity of 32 mm (1.25 in) and, by interpolation 546 N/mm (3116 lb/in) at 25 mm (1.0 in). Even at 32 mm (1.25 in) eccentricity, the assembly had an REM strength in excess of 533 N (120 lb).

Tables 3 and 4 show the connector loads versus eccentricity. As expected, as the eccentricity increases, the Peak Load capacity goes down. Table 4 values of REEL and REM reflect an inelastic behavior at 19 mm (0.75 in) eccentricity. In this case, it is reasonable to set the REEL and REM capacities at least as high as the PL. An overall behavior of load vs eccentricity is shown in Figure 4.

RECOMMENDATIONS AND CONCLUSIONS

The overall characteristic behavior of the connectors was defined based upon zones as determined by three significant loads, namely, REEL, REM, and PL. REEL was the extent of the basic elastic behavior and REM was the practical end of the inelastic action which was believed to be the appropriate maximum load that could be considered as the practical ultimate load.

The CSA Standard for Connectors recommends the use of large and varied safety factors for connectors. From the data presented, anchors can be designed by a manufacturer with consistent properties to achieve the desired stiffness and strength results. Based on these results, a possible design recommendation can be to require that a manufacturer document the properties of an anchor and its "characteristic strength", based on the average value less two standard deviations. This resulting characteristic value should be allowed as the "working strength" of the anchor in that particular failure mode. As an example, if the characteristic REM strength of 890 N (200 lb) as determined for the D/A 213-0 with 19 mm (0.75 in) is used for design, this assembly could be spaced at 0.36 m^2 (4.0 ft^2).

TABLE 4 LOADS AND DEFLECTIONS TEST SERIES 2

ANCH OR TYPE		"REEL"	"REM"	"PL"	AVERAGE STIFF.
213 - 0 0 Ecc.	AVG. LOAD	1.81 kN (407.6 lb)	2.63 kN (591.7 lb)	5.59 kN (1256.1 lb)	1361 N/mm (7780 lb/in)
12ga/11 ga	CHAR. LOAD	1.72 kN (387.8 lb)	2.30 kN (517.4 lb)	5.15 kN 1158.0 lb)	
	DEFLEC.	1.83 mm (0.072 in)	12.14 mm (0.478 in)	31.34 mm (1.234 in)	
213 - 0 19 mm (0.75 in)	AVG. LOAD	0.98 kN (219.3 lb)	1.05 kN (236.4 lb)	1.69 kN (379.3 lb)	721 N/mm (4120 lb/in)
Ecc. 12ga/11 ga	CHAR. LOAD	0.81kN (181.1 lb)	0.88 kN (198.1 lb)	1.29 kN (291.0 lb)	
	DEFLEC.	1.96 mm (0.077 in)	5.54 mm (0.218 in)	NA	
213 - 0 32 mm (1.25 in)	AVG. LOAD	1.41 kN (318.8 lb)	1.59 kN (357.5 lb)	1.61 kN (361.1 lb)	369 N/mm (2110 lb/in)
Ecc. 12ga/11 ga	CHAR. LOAD	1.09 kN (246.1 lb)	1.43 kN (322.2 lb)	1.50 kN (337.9 lb)	
	DEFLEC.	5.05 mm (0.199 in)	9.55 mm (0.376 in)	10.13 mm (0.399 in)	

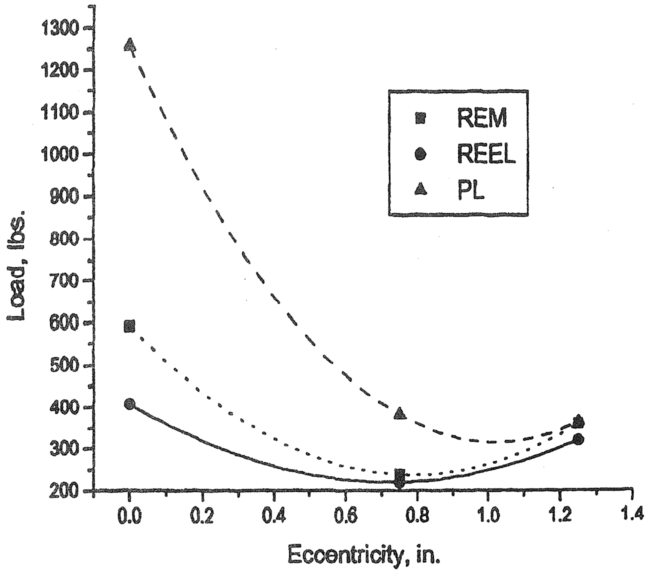

FIGURE 4 LOAD VERSUS ECENTRICITY

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THE PULLOUT OF TIES FROM BRICK VENEER

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ABSTRACT

This paper documents a study to assess the likely performance of masonry ties in tensile pullout from clay masonry. Four different proprietary ties were tested, namely a Z-tie, a straight dovetail, a corrugated dovetail and the Helifix HRT60.

The test procedure for most of the tests involved simple monotonic pullout of the tie from a brick couplet. An initial in-plane vertical compressive stress of either zero or 33 kPa (690 psf) was applied to the brick couplet. The relevant American, British and Canadian standards for pullout testing of ties from brick couplets require different amounts of vertical clamping during testing. The theoretical and experimental aspects of the clamping force and its consequences are discussed.

All the tie systems tested performed satisfactorily. However, the failure mechanism and the nature and extent of damage, as well as the bursting forces imposed on the brick couplet varied considerably.

INTRODUCTION

In order to properly design an exterior wall system with a brick veneer facade, it is necessary to have some knowledge of the structural properties and likely performance of the lateral ties that connect the brickwork to the structural backing. Usually the only vertical load taken by the brick veneer is its own weight. Lateral wind loads are resisted by the overall wall system, with the lateral ties ensuring some degree of composite structural action between the brick veneer and backup. The effects of abnormal loadings such as seismic, impact or explosion also require consideration.

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