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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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In Canada, for a number of reasons (corrosion, cost, code revisions, etc.) the nature of masonry ties and their likely response have been and still are contentious. For example, the issues of structural serviceability and durability need to be explicitly and properly addressed. One innovation that has been introduced to North America is the Helifix, stainless steel, helical tie. This is a British development that has been found to be particularly effective for remedial work in stone, brick and concrete masonry.

The project reported on in this paper, and in more detail in a proprietary report (Postma, Burnett, 1992), had a variety of purposes. The first was to experimentally assess the performance of the Helifix HRT60 tie when pulled out from brickwork compared to the performance of three commonly used proprietary systems. The second objective was to document the response of brick ties at relatively large displacements in order to assess their potential for use in, for example, seismic areas. The third objective was to examine the influence and magnitude of likely clamping forces during pullout. By extension the fourth objective was to compare Canadian, American and British practice with regard to pullout testing of ties from brick couplets.

TEST PROGRAM

Four different tie systems were tested and each series of tests was labeled as follows :

- Series 1 - **Helifix HRT60 Tie** - The Helifix HRT60 is a helical tie made from Grade 304 stainless steel. The overall diameter of the HRT60 tie is 6.0 mm and the pitch length is 15 mm. A straight length of HRT60 was used.
- Series 2 - **Z-TIE** - The Z-Tie is a cylindrical wire tie made from hot dipped galvanized steel with a diameter of approximately 5mm.
- Series 3 - **Corrugated Dovetail Tie** - The corrugated dovetail tie is hot dipped galvanized steel with an approximate width of 25 mm and a thickness of approximately 1.5 mm. The end of the tie that was embedded in the mortar joint had corrugations with an amplitude of approximately 2mm and a pitch of approximately 10mm.
- Series 4 - **Straight Dovetail Tie** - The straight dovetail tie is hot dipped galvanized steel with an approximate width of 25 mm and a thickness of 1mm. The straight dovetail has a lip on the end which gives it anchorage in the mortar.

Each series involved a minimum of three or, preferably, five identical tests. Two brick high mortared couplets were used with the tie embedded centrally in the mortar joint. Only pullout or tensile load was applied with a clear length of 25 mm left between the face of the couplet and the loading clamp. This tie clamp was restrained against rotation during pullout.

Preparation of Brick Couplets

The couplets were fabricated in accordance with ASTM E 754. Materials and skilled masons were provided by a local contractor. Every effort was made to ensure that all the mortar joints were of a uniform thickness of 10 mm and that the ties were laid at the center of the brick couplet perpendicular to the face of the couplet. The ties were centered lengthwise and cast in the brick couplet so that the end was at the middle of the brick, i.e., an embedment of 45 mm. This embedment was slightly less than the recommended minimum of 50 mm. The embedment was deliberately reduced, to be conservative and to provide for construction tolerances. The brick couplets were not disturbed after being made and were cured under damp burlap in a laboratory maintained at approximately 23 Celsius and 50 % RH.

The mix proportions for the mortar were 1 : 1 : 5.5, W : C : S by weight. The average of all compression tests on 50 mm mortar cubes was 12.4 MPa at 28 days, which is comparable to the required minimum for Type S Masonry mortar in ASTM C270-89. Testing commenced 28 days after fabrication.

Test Set-up

British, Canadian (CSA) and American (ASTM) standard documents for testing ties in masonry mortar joints all require the use of brick couplets. Brick couplets are single-wythe, stack-bonded, two-unit prisms that contain one tie or anchor centered within the brick couplet. Test procedures require tensile pullout of the tie from the brick couplet, with the pullout load and the corresponding displacement being continuously recorded.

A tie located in the bottom courses of a real veneer wall, which may span one or two floors, will have a relatively large in-plane compression force due to the weight of the masonry above. A tie placed in the top few courses of a wall will have little in-plane compression due to the dead weight of the masonry. The British document BSI DD140, the Canadian standard CAN3-A370-M94 and ASTM standard E 754 all differ in their approach to allowing for this in-plane constraining force on the brick couplet.

BSI DD140 - This British document requires a constant compressive stress to be applied to the clay brick couplet to simulate the dead load of the veneer above the tie, i.e.,

> *"compression device arranged to maintain a compressive stress of 100 +/- 5 kPa as the specimen deforms."*

CSA Standard - The Canadian standard requires a uniformly distributed surcharge of 10 kPa to be applied over the bearing surfaces of the masonry units. The CSA standard gives the following explanation for the use of a 10 kPa surcharge :

> *"This surcharge is intended to simulate a nominal dead load as might be imposed on a connector located near the top of a wall. Increased gravity loading at a lower position in the structure may increase the strength of the anchorage of the connector through clamping action."*

ASTM Standard - The ASTM standard recommends placing the couplet on rollers and avoiding any in-plane force. The authors of the ASTM standard recognize that the standard is conservative, as is indicated in their introduction :

> *"This test method is recommended for determining conservative ultimate pullout values of masonry fasteners under conditions that approach those found in the upper courses of masonry wall construction, which experience little or practically no vertical load restraint."*

If couplet tests are conducted without any in-plane constraint, the tension pullout force is not necessarily a measure of the strength of the connection, but instead is more indicative of the bond between the brick unit and the mortar. When the tie is pulled out of the brick couplet, a bursting force is exerted on the brick couplet and this force can split the interface between the mortar and the brick.

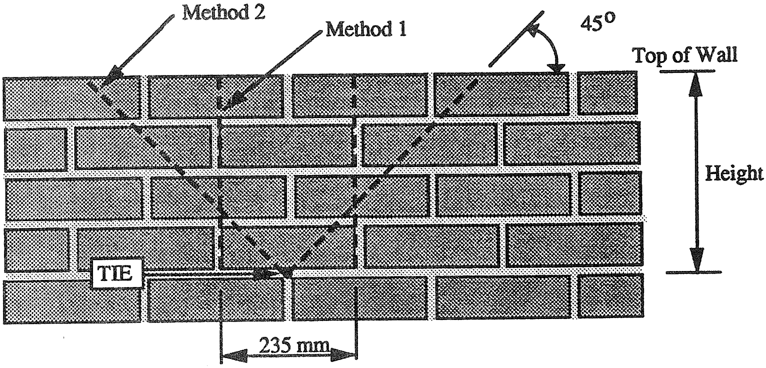
We believe that the ASTM test with zero clamping force is not an appropriate vehicle for assessing the likely performance of ties. Also, the small surcharge required by the CSA standard is appropriate for the top row of ties only and does not adequately represent the performance of the majority of the ties in the wall.

It is both more realistic and instructive to apply an initial (dead weight) force and measure the in-plane consequences of pulling out the tie. In reality there are at least two contributions to in-plane constraint, namely, the dead load surcharge and the interfacial tensile resistance of all the joints involved or engaged. In a real wall the latter is significant. We consider that with a two brick couplet the pre-load constraint models the surcharge and the prevention of in-plane movement accounts for the tensile bond resistance. This method of applying a clamping force and subsequently measuring the restraint that is developed, is, in our view, a better model of real wall response.

Magnitude of Clamping Force

Determining the magnitude of the likely compression clamping force to apply to the brick couplets during testing can be done in a number of ways. One approach is to consider the weight of a column of bricks one length of brick wide. A second approach is to consider the weight of brickwork within a 90 degree triangle to the top level of the bricks (see Fig. 1).

For these two approaches the compressive clamping force on the tie can be calculated for any height of brick above the tie and, knowing the area of the brick couplet, the approximate compressive stress for testing can also be calculated. Fig. 2 shows the compressive force and related stress for both methods for various heights of wall. A compression stress of 33 kPa, which is equivalent to a load of 0.66kN (150 Lbf), was selected as the value to be used. This 33 kPa value is one third of the value, 100 kPa, that is specified in BSI DD140 and more than three times the value, 10 kPa, that is specified in the CSA standard.



Method 1 : Area = 235 x Height

Method 2 : Area = 1/2 x Height x (2 x Height) = Height^2

Fig. 1: Determining the Clamping Force

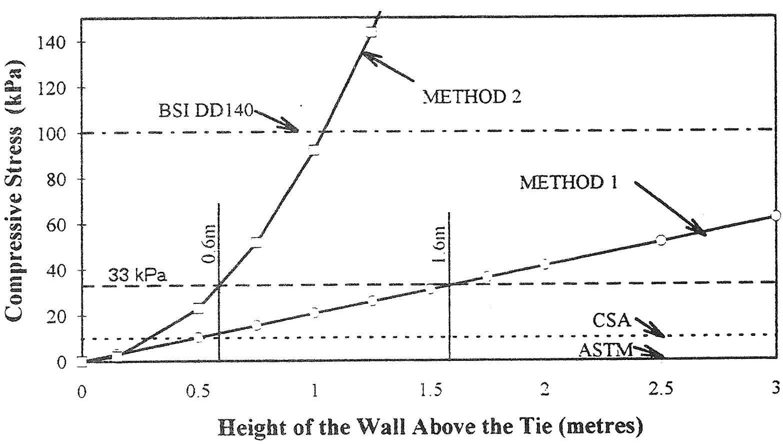


Fig. 2 Possible Clamping Effects

A clamping device or frame was made with a load cell positioned so that the force imposed on the brick couplet (by tightening a screw onto two end plates) could be continuously measured and recorded on a x-y plotter together with the pullout force and displacement. Prior to each test the brick couplet was carefully placed into this frame and the load increased to the desired amount.

Test Procedure
Tests were conducted in a MTS electro-hydraulic test facility at the University of Waterloo. The direct pullout tests were conducted under stroke control at a rate of 6 mm/min. For the pre-conditioning a sinusoidal loading with amplitudes of 0.2 kN (45 Lbf) tension and 0.2 kN (45 Lbf) compression was applied for 1000 cycles at 1.0 Hz, subsequently the specimen was loaded to failure under stroke controlled, monotonic loading. Pullout load and displacement, as well as overall clamping force, were continuously monitored.

TIE PERFORMANCE

Overall Response
This test program involved a total of 28 tests. The various series of tests, tie type and either 0 or 0.66 kN initial clamping force, plus the relevant test values at the proportional limit, P, and maximum capacity, M, are listed in Table 1. Composite versions of the relationship between pullout force and pullout displacement are provided in Figs. 3 and 4.

In general, each tie type pulls out of the brickwork couplet as follows :

- There is an initial, essentially linear elastic, tensile response up to a relatively well defined limit, P, at which point there is a fundamental change in the mechanics of anchorage. Some degree of failure at the tie-mortar interface occurs, and the transition in mechanics is usually accompanied by a temporary drop in resistance, a release in energy is only apparent because load is being applied under stroke (i.e. displacement) control.
- All the ties have considerable pullout capacity subsequent to P. Between P and the maximum pullout capacity, M, the equivalent stiffness of each tie type is very different.
- Subsequent to M, some of the ties are relatively brittle, but the nature of the post-M response is seen to be greatly influenced by constraint conditions.

Structural Serviceability
The recent Canadian standard CSA A370-94 requires that :

(i) under a tensile (or compressive) load of 0.45 kN, the overall displacement (sum of displacement plus any free play) shall not be more than 2.0 mm.
(ii) the total free play should not exceed 1.2 mm.

To address the free play criterion, a pre-conditioning phase involving 1000 cycles of +/- 0.2 kN (45 Lbf) at 1.0 Hz was imposed on selected series of tests (many more than 28 tests were actually performed). In no case was the free play limit approached. In fact there was very little measurable slack and, as is evident in Table 1, the displacement associated with the proportional limit is always much less than 1.2 mm, let alone 2 mm. Since the proportional limit load, P_p, is always greater than 0.45 kN, both stated serviceability criteria are readily satisfied by all four types of tie. From Table 1 it is evident that the HRT60 tie has a lower P_p value than the other three tie types.

Table 1 -- Summary of Mean Load and Displacement Values

TIE TYPE	Initial Clamping Force kN	N	At Proportional Limit (P)			At Maximum (M)		P_m/P_p	Δ_m/Δ_p
			P_p kN	Δ_p mm	P_p/Δ_p kN/mm	P_m kN	Δ_m mm		
HRT60	0	3	0.49	0.13	3.77	1.64	8.48	3.35	65.2
	0.66	5	0.74	0.03	24.70	2.05	14.98	2.77	499
RATIO			**1.51**	**0.28**	**6.5**	**1.25**	**1.77**	**0.83**	**7.66**
Z-TIE	0	3	1.19	0.08	14.90	2.47	1.10	2.08	23.8
	0.66	5	1.71	0.21	8.14	2.59	0.93	1.51	4.43
RATIO			**1.44**	**2.63**	**0.55**	**1.05**	**0.85**	**0.73**	**0.19**
Corrugated Dovetail	0	3	1.40	0.33	4.24	2.61	1.18	1.86	3.58
	0.66	5	2.34	0.30	7.80	2.72	3.42	1.16	11.4
RATIO			**1.67**	**0.91**	**1.84**	**1.04**	**2.90**	**0.62**	**3.18**
Straight Dovetail	0	1	1.52	0.00*	N.A.	3.20	0.30	2.10	N.A.
	0.66	3	3.14	0.50	6.28	3.33	2.05	1.06	4.1
RATIO			**2.07**	**N.A.**	**N.A.**	**1.04**	**6.83**	**0.51**	**N.A.**

* No measurable displacement

The first criterion may be extended to imply that the proportional limit must be greater than 0.45 kN (to ensure linear elasticity) and that the initial stiffness must not be less than 0.225 kN/mm (0.45/2.0). This implicit provision is illustrated in Figs. 3 and 4. In both figures it is evident that all four tie types readily meet all these structural serviceability criteria irrespective of the initial clamping force.

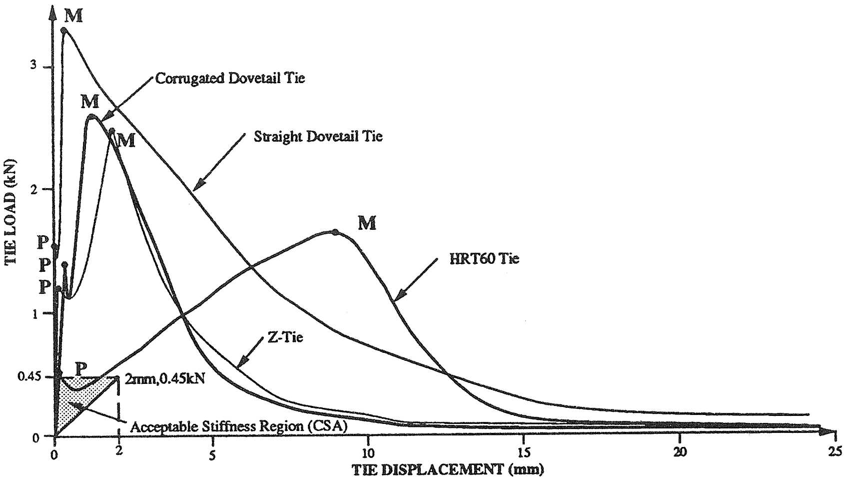


Fig. 3: - Pullout Response of Ties with Zero Initial Clamping Force

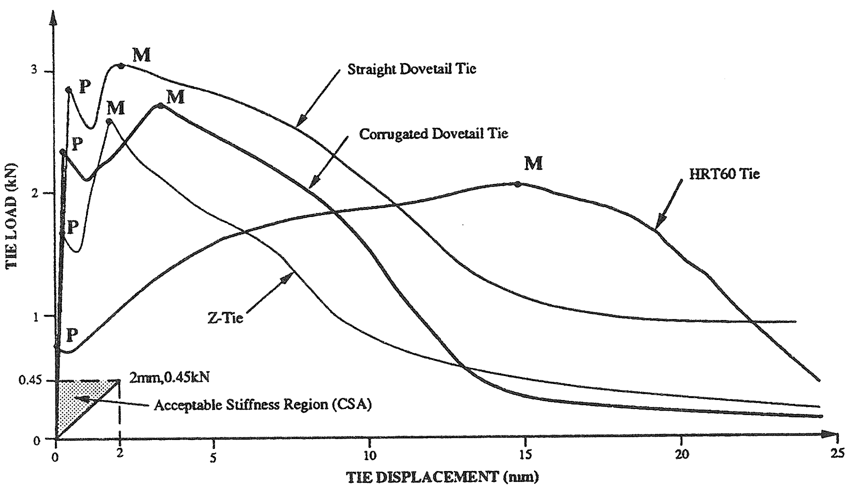


Fig. 4: Pullout Response of Ties with 0.66 kN Initial Clamping Force

Structural Safety

These tests were not done to determine equivalent safe design values; to do so would require a minimum of 10 samples in order to establish statistically valid measures of standard deviation. Instead, structural safety is assessed here on the basis of three characteristics :

(i) Maximum capacity, P_m, and, since P_p is a service load limitation, on the ratio, P_m/P_p

(ii) Ductility ratio, Δ_m/Δ_p , which is a measure of performance when the connection experiences an excursion into the post-P range of response. Ductility is particularly important not only for extreme wind loads but also for seismic, impact or explosive loadings. Bear in mind that ties may be required to redistribute load, to absorb and to release energy, and to avoid fracture of both the tie and the attached material.

(iii) For similar reasons the post-M response should not be brittle.

The Canadian standard requires an "ultimate strength" of not less than 1 kN and clearly all these types of tie readily satisfied this criterion.

On the basis of the above three characteristics the HRT60, despite having the lowest P_m and P_p values, is far superior to the other three tie types. As is evident from Table 1 and Figs. 3 and 4 the Helifix tie is unique in its ability to sustain both load and displacement.

For the purpose of design it would be necessary to develop safe design loads for each tie. These safe design loads would have to take both structural safety and serviceability criteria into account. Obviously these safe design loads would be least for the HRT60 and greatest for the straight dovetail tie. However, these values would not necessarily reflect the nature of response of the tie when overloaded - especially by an abnormal loading such as earthquake or explosion. This aspect of tie performance needs to be given more attention in the future.

Constraint Effects

It is clearly evident from Table 1 and Figs. 3 and 4 that the vertical constraint - both the initial clamping stress and the vertical restraint continuously applied during testing - has a significant influence. Note that in the tests with a zero initial clamping force, the brick couplet remained unrestrained during testing. To assess the effect of this initial and continuing restraint refer to Fig. 5, which illustrates the magnitude of the initial and developed clamping force during testing. In this figure it is evident that, whereas the clamping force remains constant during pullout for the Helifix tie, the other three tie types experience a significant increase in clamping restraint (or a bursting pressure) for post-P response. These findings are significant, especially for ties within a few courses of the unrestrained top of a brick veneer wall.

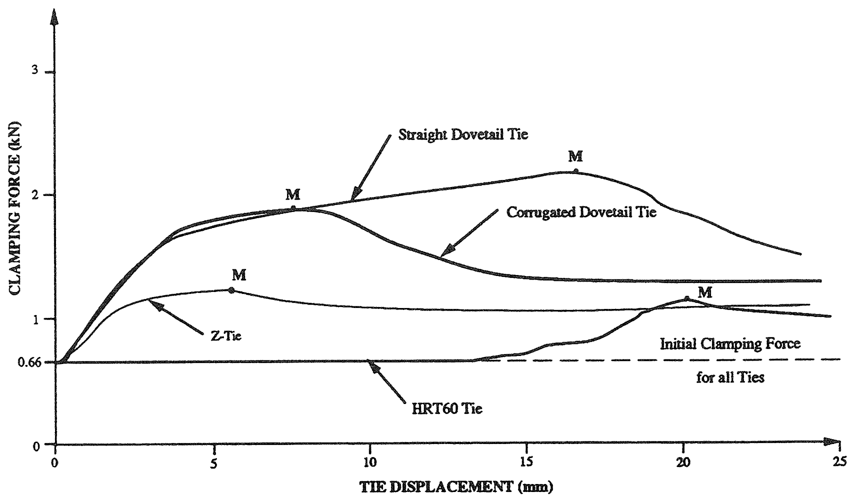


Fig. 5: Clamping Force Variation During Pullout of Tie

Correspondingly Table 1 indicates that the HRT60 tie benefits most from the initial restraint as P_m and P_p are both increased. But P_m/P_p is decreased, which is advantageous as it means that the serviceability limit is raised. The initial stiffness is also increased and both post-P deformability and ductility ratios are increased.

In general restraint is beneficial in that :

- P_p values are increased significantly, from 44% to 107%.
- P_m values are not significantly increased except for the HRT60 tie which experiences a 25% gain.
- P_m/P_p values are all reduced which is generally beneficial.
- Δ_p values are reduced for the Z-Tie and the straight dovetail tie. However, these differences in displacement and stiffness (P_p/Δ_p) are not particularly significant, as the displacement values are small and difficult to measure accurately. For the Z-Tie the increase may be due to the fact that better restraint increases P_p and thus disproportionately increases pullout displacement.
- With one exception, the value for Δ_m is increased, i.e., with larger P_m and more restraint increased deformability is likely to be the case. The one exception is the Z-tie which appears to have received some stiffening for post-P response as a consequence of vertical restraint.
- The ductility ratio, Δ_m/Δ_p , is increased for both the Helifix and corrugated dovetail tie. The opposite occurs for the Z-tie and, probably, the straight dovetail tie as both rely on a hook to anchor the tie - in which case mortar fracture must occur. In the case of the HRT60 and, to some extent, the corrugated dovetail tie, the tie is sliding in the sense that mortar failure (bearing and shear) is largely at or close to the tie-mortar interface. The effect of the clamping force is not consistent for the displacement of these four tie systems.

Comparative Performance

Any comparison that does not consider the cost to purchase and install the tie is incomplete, but some indication of relative performance is provided in Table 2.

The clamping force values quantify the fact that during stable pullout, P to M, the Helifix spiral tie generates much less bursting pressure than the other ties. Not only does the spiral tie have more stable deformability but the amount of direct damage and, probably, the risk of consequential damage is less.

A comparison of equivalent stress (P/A) values at P and M clearly demonstrates that :

(i) At P, the proportional limit, all the tie types have developed comparable levels of stress; the corrugated dovetail being the least materially efficient at this stage of loading.

(ii) At M, the maximum capacity, the Helifix tie is clearly the most, in material terms, efficient while the corrugated dovetail is the least efficient.

(iii) The increase in tensile pullout capacity due to in-plane restraint is much greater for the Helifix ties than the other three tie types.

Table 2 -- Comparative Performance Considerations

TIE TYPE	SIZE	Area of Metal mm^2	Initial Clamping Force kN	Clamping Force at P kN	Clamping Force at M kN	P_p/A kN/mm^2	P_m/A kN/mm^2
HRT60	6 mm	8	0 0.66	- 0.66	- 0.75	6.13 9.25	20.5 42.0
Z-TIE	4.76 mm	17.8	0 0.66	- 0.76	- 0.92	6.69 9.61	13.9 14.6
Corrugated Dovetail	1.52 x 25 mm	38.0	0 0.66	- 0.69	- 1.45	3.69 6.16	6.37 7.16
Straight Dovetail	1.0 x 25 mm	25.0	0 0.66	- 0.72	- 1.23	6.08 12.6	12.8 13.3

CONCLUSIONS

In this paper we have refrained from developing design recommendations mainly because:

(i) more tests are needed to establish statistically valid values

(ii) current standards do not fully recognize the need for load cycling and post-P ductility in order to fully assess structural serviceability and safety

(iii) the two brick couplet test method and clamping conditions needs to be reassessed.

Nonetheless, this limited series of tests permits the following conclusions :

- All tie systems tested fully satisfied the structural serviceability and safety requirements outlined in the current CSA standard.
- The Helifix tie, specifically the HRT60, behaves in a manner that is very different from the behaviour of other types of ties. Whereas the pullout force values at the proportional limit and at maximum capacity are relatively low, the ductility ratio is large. That post-M response is not brittle, that any initial clamping effect is beneficial, and that little if any bursting forces are generated during pullout are distinct advantages. These features are particularly useful in seismic areas and for both new and retrofit situations.
- The other three tie types all have significantly higher P_m and, more importantly, Pp values. The pullout response of these ties is not necessarily brittle.
- The Helifix tie is much more efficient at maximum strength than the other three tie types.

It is clearly evident that the presence and variation of the in-plane restraint has a significant effect on the performance of these ties during pullout testing. The brick couplet test procedure needs to be reassessed. In particular two issues need to be examined, the magnitude of the initial clamping force and the need for continuously monitoring this force.

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COMPOSITE WALLS

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ABSTRACT

Composite walls are defined as multi-wythe walls in which the wythes are tied together by connectors, filled collar joints, bonding units, or other means to ensure shear transfer between the wythes and effective composite action.

These types of walls have been used in North America for many years with variable results. There is evidence of separation at the collar joints between dissimilar masonry wythes bonded together by mortar or grout, and there is evidence of good performance of composite walls connected by means of a header course.

Recently composite walls have received a renewed interest with the introduction of connectors capable of transferring shear across the cavity. This paper provides an overview of composite walls and provides information on the current trends of cavity wall performance.

INTRODUCTION

In addition to the many other positive qualities of the system, the ability of a cavity wall to accommodate insulation and to provide for the structural integrity of the air/vapour barrier is well known. Cavity walls were designed to resist vertical and lateral loads by assuming that all loads were ultimately resisted by only one of the wythes, the interior wythe. Many masonry walls have been built over the years, and continue to be built, with clay brick and concrete block connected by wire ties, with the collar joint filled solid with mortar or grout. These walls are known as composite walls and can be insulated on the interior, or the cores of the inner wythe can be filled with insulating material. The evolution of

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