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Seismic Performance of Masonry Veneers and Ties

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Preface

This is the first of a series of reports prepared during research into masonry veneers and their seismic performance. Future reports will cover in-plane and out-of-plane behaviour and the problem of deflection incompatibility at corners.

Acknowledgments

The work was jointly funded by the *Building* Research Levy and the Foundation for Research, Science and Technology from the Public Good Science Fund.

Readership

This report is intended for researchers, structural engineers, manufacturers and other workers in the field of masonry veneers.

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Masonry, Veneer, Timber Framing, Veneer Ties, Seismic Performance, Dynamic Testing, Testing, Performance Criteria.

ABSTRACT

This report describes the first stage of an investigation into the performance of masonry veneer on framed backing walls under seismic loading. Both elemental tests on veneer ties and dynamic tests on full-scale walls were carried out.

The report concludes that the current practice of installing ties by hammering nails into the framing is unsatisfactory, and results in poor performance under seismic loading. The report recommends changes in tie installation procedures to overcome this.

A further conclusion is that the criteria for tie performance as currently covered by international standards, and the test methods to establish code compliance, are unsatisfactory. Suggested directions for improvement are listed.

Contents

PHOTOGRAPHS, FIGURES AND TABLES

1.0 INTRODUCTION

The widely used expression "bricks and mortar" is an indication of the strong place that brick masonry holds in our culture. Use of brick masonry in New Zealand stems from years of experience in Great Britain and Europe. There, spectacular examples of load-bearing masonry structures, still standing after hundreds of years service, are visible proof of fitness for purpose and durability.

de Vekey (1990a) suggests that cavity masonry was first introduced in the nineteenth century as a means of improving resistance to weathering in coastal southern England. Early cavity walls consisted of a full or half brick rain shield, tied to the structural wall by brick headers, or cast iron or wrought iron ties. Cavity walls have also developed as two skins of masonry, acting compositely to share loads through the stiffness of the cavity ties (or header masonry units) - the familiar "double brick wall" commonly used early this century. Cavity ties, therefore, have more demanding stiffness requirements than do veneer ties and are not addressed in this study.

Load-bearing masonry construction was imported to New Zealand and used extensively throughout the country, until found wanting in the Napier earthquake of 1931 and other similar events. More recently, the outer "rain shield" skin of masonry has been used as an attractive, low maintenance cladding (or veneer) to buildings constructed of timber or (very recently) steel framing. Since the 1930's, brick masonry veneer cladding tied to a timber frame backing has gained increasing popularity in New Zealand. Now, a significant portion of housing stock contains brick veneer as part or the whole of the external cladding. In new housing this proportion may be as high as 30% (Monier 1992). Use of veneer also extends into the low-rise commercial area.

Until relatively recently, veneer ties used in New Zealand were a development of the UK "butterfly" wire tie. These were nail or staple fixed to the side of the stud some time after the veneer was laid and the mortar cured. In the 1970's, with recognition of the importance of building insulation, building paper fixed to the outside of the timber frame became used universally in brick veneer work (NZ Technical Correspondence Institute 1977). Side-fixed ties which required penetration of the building paper became unacceptable and veneer ties evolved into the face-fixed metal strips commonly used today. The resulting configuration, therefore, is not based on structural requirements. Within the last five years the recognition of the desirability of ductile behaviour under seismic action has spread into the masonry veneer world, and the La Palle ductile tie has been developed (Allen 1988).

The original research programme, of which this report covers the first part, was conceived as an attempt to understand the seismic behaviour of the composite structure consisting of veneer and backing frame, under both in-plane and out-of-plane action. The importance of the veneer ties was always understood. However, as deficiencies in their anchorage to the brick veneer became apparent during the early stages of the work, the emphasis of the programme shifted to finding a satisfactory solution to this problem, before tackling the wider issue of composite behaviour. This report, therefore, concentrates on backgrounding the subject, and the tie anchorage problem. Subsequent reports will cover composite in-plane and out-of-plane behaviour, and the corner incompatibility problem.

2.0 SEISMIC PERFORMANCE OF MASONRY

In spite of having many desirable qualities as a building material, masonry does have inherent limitations with regard to its seismic performance during earthquakes. It is a relatively heavy material and therefore subject to high inertial loading, it is brittle and non ductile and, because it is made up of individual units requiring interconnection, it can be adversely affected by poor workmanship.

Masonry's poor performance in past earthquakes both within New Zealand and overseas has been widely documented by both the press and the technical literature. This has resulted in a lack of public and professional confidence in the material. This perception is based largely on the performance of load-bearing masonry rather than masonry veneer walls. In fact, reported evidence relating specifically to veneer wall behaviour is rather sparse.

Features of veneer construction thought to contribute to failure during earthquakes are as follows:

Napier 1931: Inadequate tying of the outer unreinforced skin of cavity brick to the structural frame. Examples given were shop fronts, gables, parapets and architectural facades. (Brodie and Harris 1933)

Seddon 1966: Inadequate ties to veneers, which cracked at extremities of the buildings and wall junctions. (Adams et al. 1970)

Inangahua 1968: Some bricks dislodged from walls. Incompatible stiffness of veneers and timber frames resulted in brickwork shattering at corners and near windows and doors. (Cooney 1979)
Edgecumbe 1987: Staple-fixed ties pulled loose, veneer panels between windows being particularly vulnerable. In some buildings, however, the veneer apparently

supported the structure through stiff wall ties, until ultimate failure occurred. (BRANZ 1987)

Newcastle 1989: Missing or corroded ties. Melchers and Page (1990) noted that veneered houses performed better than cavity brick houses.

Brick chimneys, in particular, have been criticised strongly. However, few new houses have brick chimneys, and those that are built are likely to be reinforced and specifically designed.

Kenna (1965) and Cooney (1979) are two of the few authors to assemble and summarise previous accounts. Both are quoted widely for their analyses of house performance in earthquakes. Cooney, in particular, succinctly expressed the fundamental problem of incompatible stiffness between the veneer and timber frame. He suggested a number of avenues for improving the behaviour of veneers in earthquakes, including isolating corner veneer planes, using ties which allow free in-plane movement between frame and veneer; he even contemplated reinforced veneer.

Unfortunately, a number of commentators have confused poor workmanship (and even in some cases unfinished or unbraced structures) with fundamental behaviour. While it may be a feature that, in the past, masonry has been subject to gross breaches of workmanship, that is no reason to write off an otherwise satisfactory building system. As Holgate (1932) said: "*Since the Napier (1931) earthquake, brick construction has received severe criticism in some quarters, but the writer submits such criticism was very ill-advised, being based solely on a superficial inspection of the area of destruction. It is quite safe to say no material, however good, would be of any use against earthquakes if the design and workmanship were faulty.*" This statement seems as true today as it was then.

3.0 BACKGROUND TO MASONRY VENEER CONSTRUCTION

Historically, developments in masonry construction have depended on the skills and craftsmanship of the master masons, largely through trial and error. Over the last 30 years, methods based on formal scientific study have provided the basis for rational design procedures for structural masonry. To a lesser extent this is also true of brick veneer construction where, generally speaking, rules for building methods have been developed from the experience gained from successful constructions from the past. Recently, however, some scientific studies have been carried out; some of these have led to the development of more sophisticated codes of practice (e.g. NZS 4230, NZS 4210 [SNZ 1990a, 1989] and AS 3700 [SA 1988]).

3.1 Literature Survey

The literature shows that, generally speaking, New Zealand researchers, reacting to experiences of past earthquake damage, have concentrated on the interaction of brick veneers with timber frames under seismic lateral loading. Almost all overseas workers have studied wind loading effects.

North American experience of masonry cladding has been concentrated almost solely in the field of steel stud framing. In the United Kingdom, the main focus has been on corrosion of cavity ties. Thus, very little of the overseas information is directly relevant to New Zealand. It seems to be a field where, in New Zealand, we are largely "on our own".

The findings of the literature survey are grouped, with Australasian work first, followed by UK findings, and then those from North America.

The first known New Zealand analysis of masonry veneer on timber frame backing was carried out by the Pottery and Ceramics Research Association (PACRA) (Kenna 1963). Although he commented that "*there is little or no test or analysis data available on this form of construction,*" the NZ Standard Model Building Bylaw NZSS 95 (NZSI 1955) contained provisions stipulating that veneer should be tied to the frame, and that the frame should be adequate to resist all applied loadings without requiring a contribution to strength from unreinforced masonry. However, seismic design loadings (at 8%g) were rather less than required by today's codes, resulting in wind loads governing most designs. Following on a suggestion by Thomas (1953), Kenna carried out an elastic stiffness analysis, both of unreinforced veneer spanning between the ties, and of the veneer/stud assembly. He concluded that tensile stresses in the veneer under code loads were about the same as the ultimate tensile flexural strength of the brick masonry. He also concluded that lack of diaphragm action at roof level would have serious consequences under lateral loads from wind or earthquake. Based on this work, he presented recommended veneer tie spacings.

Kenna (1965) tested veneer/timber frame panels in flexure under quarter point loading. He reported a 28% improvement in bending performance (to first cracking in the veneer) by tying the unreinforced veneer to the frame backing. He suggested that ultimate performance would be limited by the strength of the studs "*with possibly an intermediate yield stage occurring due to the buckling collapse of the wire ties*".

Priestley et al. (1979) tested seven unreinforced and two reinforced brick veneer panels under dynamic face loading. The panels were single storey, approximately 2.4 m high, and of various lengths. These tests were a continuation of the series begun by Kenna (1965). The panels, tied to timber-frame backing, were subjected to inertial loading by sinusoidal acceleration within the expected range of earthquake frequencies. However, the timber framing of the veneer panels was fixed through the top and bottom plates to the shaking frame. So, although dynamic face loading was applied at about the centre of gravity, the specimen was nevertheless subjected to the same acceleration over its full height. Thus, the concern of Kenna relating to diaphragm action at roof level was not pursued. Priestley et al. concluded that unreinforced brick masonry veneers, built according to standard practice, can survive response accelerations well in excess of those expected under the design level earthquake of the (then) current loading code (NZ 4203, SNZ 1976). Initial cracking at the mid-height of the veneer occurred at a minimum acceleration of 0.6g, with ultimate failure through shedding of the masonry occurring at a minimum peak acceleration of 1.7g. Damping of the specimens decreased at the onset of cracking and stabilised at a minimum value of between 4-5% of critical. Priestley et al. found that neither preformed crack inducement in the veneer nor variation of stud spacing had significant influence on the ultimate performance of the veneers, and that failure of the veneer panels was normally preceded by failure of the studs in flexure.

Cooney and Fowkes (1981) observed that, while most attention had concentrated on face loading, the current flat strip configuration of veneer ties meant that significant in-plane shear loads between veneer and wall were able to be transferred. This, in turn, affects the corresponding amount of veneer damage, particularly at corners.

Between 1983 and 1985 a series of in-plane static, racking tests was carried out for the New Zealand ceramics industry and reported on by Fenwick (1983), Compton (1985), and Fenwick (1986). These tests were summarised and analysed by Allen and Lapish (1986).

The first test (Fenwick 1983) used a specimen consisting of a brick veneer with a timber framed backing wall. The timber wall had plywood cladding on its outer face, and plasterboard lining. The test was carried out under displacement control by loading the timber frame top-plate at very slow displacement rates up to +/- 40 mm. A crack formed in the mortar joint at the base of the wall at a displacement of approximately 24 mm. It was reported that the satisfactory performance of the brick veneer wall depended on how well ties remained anchored in the mortar. Extending these tests, Compton (1985) described racking tests of three timber framed walls with and without brick veneer. Cracking developed between the veneer and the base beam at displacements of between 5 and 6 mm. It was concluded that addition of veneer and a "heavy duty" end stud-to-floor attachment reduced the displacement at which strength degradation started to occur. This was suggested to be that because of the stiffening effect, the fixings were overstressed. (However, in practice, such a "heavy duty" base fixing detail is unlikely to be feasible in typical buildings.) It was also concluded that, while the wall ties appeared to remain anchored satisfactorily, a reduction in horizontal stiffness would improve overall frame ductility and strength reserves.

Concluding this series of tests, Fenwick (1986) reported on wall tests which modelled the effect of two sizes of window openings. For each window size, one wall was tested without brick veneer while another was tested with veneer. It was observed that the presence of openings (with or without brick veneer) caused the walls to behave in a non-linear manner, right from the start, by comparison with Compton's tests. Cracking of the brick veneer developed at the window sill level at frame displacements of 3 mm and 16 mm for the bigger and smaller openings, respectively. It was observed that after the brick veneer cracked, the wall moved as a rigid body, rotating about the closed end of the crack. The already noted trend that the presence of brick veneers increased wall performance was also observed. This increase in load capacity was thought to be due to the increased load required to rock the veneer about one corner. It was concluded that the maximum displacements between the timber wall and veneer, and hence the load on the veneer ties, were limited in the tests by the formation of cracks in the veneer.

In a paper presented to the New Zealand Building Inspectors' Conference Allen (1986) highlighted the widespread use of non-complying veneer ties by bricklayers who bought solely on the basis of price. Since the revision of the code criteria in the late 1980's, even those ties being promoted by Allen at that time do not now comply with building codes.

Computer modelling of face-loaded one, two and three storeyed veneer-clad timber-framed walls was carried out by Lapish and Allen (1986). Computers now allowed designers to expand Kenna's work and enabled more parameters and geometries to be investigated. The composite veneer/tie/timber frame was modelled as an elastic frame structure incorporating a range of tie stiffnesses. Face loads were obtained from the Parts and Portions section of the NZ Loadings Code (NZS 4203, SNZ 1984). Lapish and Allen confirmed that, as might be expected under static analysis, ties adjacent to stiff supports (such as floor and ceiling diaphragms) attracted a disproportionately greater share of high loads. Nevertheless, these findings were at odds with the then current international design procedure of calculating the axial tie load on the basis of the tributary area supported by the tie, multiplied by a seismic mass or wind constant. In fact, Lapish and Allen suggested that with the brick/mortar bond strengths that are routinely achieved in practice, tie loads were well in excess of values specified in the Masonry Design Code (NZS 4230P, SNZ 1985). They concluded that yielding (or ductile) ties are required to enable redistribution of loads throughout the wall, thereby reducing peak tie loads to acceptable levels. This recommendation has found its way into the current version of the code (NZS 4230, SNZ:1990a). No comment was made on the appropriateness, or otherwise, of the code-specified face loading, which will influence tie loads significantly.

Allen (1988) described the background to the invention of the La Palle ductile, flexible tie connector. In the same paper he also presented numerous examples of failures of masonry claddings due to differential movement between cladding and supporting structure. The tie is claimed by its promoters to be a rationally designed connector which exhibits predictable levels of stiffness and ductility while allowing for code-imposed vertical and horizontal movements. Horizontal and vertical in-plane movement capability enables the separations between the veneer and the backing seismic-resisting structure to be achieved. The ductile behaviour in the axial direction permits the face loads on the connectors to be distributed to other connectors as described in Lapish and Allen (1986).

Following on from Allen, Lapish (1988) considered "*the interrelationship of the masonry, wall ties and timber stud framing in veneer constructions subjected to earthquake forces*" and proposed "*a design rationale by which brick veneers can achieve better performance levels than those obtained from the empirical techniques used to date*." The design rationale separates veneer walls into in-plane and out-of-plane actions and provides for vertical separations between the two. The timber-framed backing structure, with attached window and door frames and other penetrations, being more flexible in the in-plane direction, is detailed to be able to slide behind the veneer without interference. (Several of these features were first suggested by Cooney [1979]).

Lapish's work, in particular, has had considerable influence on the development of those areas of New Zealand masonry codes of practice which concentrate on veneer (NZS 4230:1990 "Code of practice for design of masonry structures"; NZS 4210:1989 "Code of practice for masonry construction: Materials and workmanship"; and also, indirectly, NZS 3604: "1990 Code of practice for light timber framed buildings not requiring specific design" [SNZ 1989, 1990, 1990a]).

Fenwick (1989) undertook cyclic load tests on a type of flat strip, nail face-fixed ties in common use at the time. The results showed a significant fall-off of axial load capacity resulting from in-plane distortion caused by racking loads of flexible support structures.

Megget (1989) reported on tests carried out on the La Palle tie described by Allen (1988). The results indicated that the tie was capable of ductile behaviour with "*excellent repeatability*", and the pull-out loads from mortar bedding were well in excess of those required to mobilise the ductile behaviour of the assembly.

Jacks and Beattie (1990) investigated how two types of wall ties commonly used in New Zealand performed when subjected to dynamic face loading. Static pilot pull-out tests were also conducted on both types of ties. For full-sized tests, two veneer/wall specimens were fixed at their top and bottom plates to a loading frame installed onto a shaking table. Using the E1 Centro earthquake records, wall specimens were subjected to three levels of intensity. They were then subjected to a sinusoidal displacement signal of 5 Hz. The displacement was increased until failure occurred. A decrease in natural frequency and an increase in damping ratio of the specimens after the earthquake signal was reported. They also reported that ties slipped in the mortar bedding, the first time this had been reported in the literature. The results of the static tests indicated that the tie pullout strength in the static state was more than 1.4 times greater than the code design strength for multi-storey construction. Under reversed dynamic loading, however, it was reported that both tie types showed initial signs of failure at load levels below the design value of 778 N. The failure mechanism was one of nail pullout from the timber framing for both tie types.

The background behind the development of the veneer provisions of the masonry construction code NZS 4210 was given by Lapish (1991). In particular, he explained the distinction made in the code between stiff and flexible ties (where stiff and flexible refers to in-plane characteristics). This is the first time that this distinction has been made, and was largely based on the findings of Fenwick (1989). In support of this, Lapish plotted the fall-off in tie axial load capacity due to prior in-plane displacement as reported by Fenwick. Evidence was also presented of brittleness of some ties subjected to in-plane displacement prior to axial loading.

In Australia, Reardon and Mahendran (1988) reported on simulated wind-loading tests on a full size, single storey brick veneer timber-framed house constructed at the testing laboratory. Large steel frames, hydraulic rams and load-spreading devices were used to apply the simulated uplift and racking pressures equivalent to those generated by cyclones or severe wind storms. General findings regarding those components associated with the brick veneers were:

(a) brickwork strength depended on the ability of the wall ties to transfer face loads from the brick skin to the timber-framed walls;

(b) wall ties used in the house were effective in transferring positive and negative code-design wind pressures from the brick veneer face to the timber framework;

(c) there was no evidence of the transfer of in-plane racking forces (in either direction) between the timber framework and the brick veneer skin; the in-plane stiffness of the brickwork was far too great to receive or transfer forces through the wall ties;

(d) the racking strength of the brick veneer skin depended upon the tensile strength of the mortar, and was affected by the presence of window openings - which effectively isolated the lengths between windows.

In the United Kingdom, Thomas (1970) discussed the functions and properties of cavity wall ties and their suitability with respect to differential movement, fire resistance, corrosive conditions and frost. He described research on the shear and pullout values of wall ties used for securing brick panels to structural concrete members, and included a table of recommended safe loads. He discussed the suitability of polypropylene ties, and the strength of wall ties designed to anchor in a dovetail slot cast in the concrete backing structure. Thomas's work was the first known documented testing or analysis carried out on ties which, until then, had generally been developed on the basis of experience in use.

de Vekey (1984) comprehensively analysed the performance criteria for wall ties. The report was intended to bridge the gap between the prescriptive requirements of BS 1243 (BSI 1978), and performance-based standards, and followed the limit states design philosophy. Before this, very little consideration had been given to defining the exact engineering requirements of wall ties. He concluded that although durability criteria needed to be prescriptive, a performance-based standard would allow development of more economical and structurally efficient ties, and allow the industry to adapt its products more easily to changing wall forms and structural demands. He proposed four grades of duty for use at standard spacings, and suggested face load deflection limits based on cracking serviceability limit state.

Many of the criteria and recommendations from this work have found their way into DD 140:1986 (BSI 1986), which has not yet been superseded. The principle is that, by means of testing, ties are allocated a classification which then determines where they may be used. The UK criteria, of course, do not include provision for seismic loading.

de Vekey (1990a) summarised the history and development of cavity and veneer ties, especially relating to durability. The Building Research Establishment's (BRE) experience has shown that not until 1981 did the material specification in BS 1243 provide for enough durability to ensure that ties were likely to last more than 50 years in typical masonry structures.

As early as 1976, Grimm in the United States commented on the lack of test data and analysis of ties, and pointed out that "*rational analysis of the physical and chemical properties of ties in masonry walls had not kept pace with the structural analysis of masonry.*" He summarised the dimensions and strength properties of ties and categorised them into rigid (shear resisting), and flexible anchors for bolting, welding, screwing or nailing to the structural backing, or for dovetail fixing to concrete backing. He concluded that test methods needed to be developed for metal ties and anchors.

In agreement with Grimm, the introduction to ASTM E754-80 (ASTM 1980) states that "*the attachment of masonry walls to building exteriors has been based largely on experience and professional judgement. Requirements specify size and spacing* *of fasteners, rather than forces to be resisted*". The test method describes an interim means of determining pullout resistance of ties under no lateral displacement.

McGinley et al.(1986) reported that although the brick veneer/steel stud curtain wall cladding system was popular in North America in the 1970's and early 1980's, the design methods were deficient because they neglected the interaction between the veneer ties and the metal studs. The methods assumed that all lateral loads were carried by studs, with a deflection limit to preclude cracking of the veneer. McGinley et al. investigated the load/deflection behaviour of full-size stud walls subjected to face loading (ties in compression only) as well as testing of the individual components. Results indicated significant movement of both the top of the veneer and the stud, thus causing wall ties at the top of the wall to attract higher load. When additional ties were installed at the top, veneer deflection was increased and the applied face load at which the ties buckled was increased. An important finding in the context of increasing interest in steel framing in New Zealand was the influence of the interaction between tie and "C" section steel stud.

Following on from a comprehensive test program, Chin et al. (1988) examined the issue of the relative stiffness of brick veneer and metal stud. They showed, using computer analysis, that veneer flexural stress is greatly reduced when veneer stiffness, stud stiffness, and stud fixity are taken into account, compared with normal methods of design. Stud fixity (within a range of practical limits) appears to have the greatest effect. They concluded that these factors explained the satisfactory behaviour of the brick veneer/metal stud systems constructed before the Brick Institute of America (BIA) recommendations were introduced.

The only known overseas study of brick veneer performance under seismic loading was that sponsored by an American masonry manufacturers' association (KPFF 1989). Unfortunately, this was also based on a metal stud frame system so not all of its conclusions are relevant to New Zealand's timber frame environment. The building configurations studied were generally multi-storey structures with the veneer supported on shelf angles at each floor level. However, this is the only known study to question code seismic loadings and to use more realistic dynamic analyses based on earthquake input spectra. They used the ATC 0.4g S2 spectrum (ATC 1978). This loading spectrum produced force levels three times as great as the 0.3g specified by the UBC code, (UBC 1985). This finding must question the code basis used to date. They also found that stiffer building types (i.e. braced frames) produced greater amplification before veneer cracking, but that this amplification reduced after cracking. As found by other researchers, they reported an elastic distribution of load between veneer and frame before cracking which varied only with tie stiffness. After cracking they found that the studs carried most of the load. They proposed a design procedure based on the UBC procedure which takes into account the following parameters:

(a) Brick veneer allowed to crack under service wind and seismic load.

(b) Studs carry full load (uniformly distributed) assuming veneer is cracked.

(c) Stud size and spacing designed for maximum deflection of L/360 under a serviceability load level. (There is, however, some debate on this value.)

(d) Establish tie force demand based on an upper bound assessment with a rigid tie and the assumption that the veneer carries 100% of the face load before to cracking (or 2g, whichever is greater).

(e) Masonry modulus of rupture of 180 psi.

(f) For ties less stiff than 2000 lb/in, the required capacity is 1.25 times calculated value. For ties stiffer than 2000 lb/in, required capacity is 2 times calculated value.

Arumala (1991a) fatigue tested of some metal ties used commonly in the UK. He found that large displacements were required to affect the behaviour of the ties. The only significant failures occurred at the location of the nailing point; this tended to be manufacturer dependent.

Arumala (1991b) also carried out an experimental and analytical study of face loading on brick veneer walls with metal stud backing. In particular, he presented a mathematical model to study the inelastic behaviour of the wall after the development of a crack in the veneer. A method of predicting the cracking load of the veneer was also presented. He found, along with other researchers, that boundary conditions, tie stiffness and flexural rigidity of the backing wall system affected performance. He also found that significant lateral load transfers to the backup wall system after the veneer cracks.

3.2 Summary of Work to Date

Amongst those workers who have attempted to quantify the distribution of forces applied to ties in the composite action under face loads, there appears to be a discrepancy in the level of force applied to each tie. Those carrying out simple elastic analyses have found that tie loads are much higher at stiff points of support such as at floors or roofs, and much lower within the span of the more flexible studs. The most extreme example of this was reported by Lapish and Allen (1986), where tie loads were shown to increase by up to three to eight times the load based on tributary area, depending on what axial stiffness was assumed in the calculation. This finding has been confirmed (more or less) by static testing simulating wind loading, and seems to be independent of whether steel or timber framing is being used as backing (McGinley et al. 1986, Chin et al. 1988, Arumala 1991b). However, this phenomenon has not been reported by any of the workers carrying out dynamic face-load testing. For example, Jacks and Beattie (1990) did not observe that ties near rigid fixing points (such as at the base and at the top of the wall) exhibited a higher level of load than those away from fixing points.

A further discrepancy observed when considering seismic loading is the influence of the actual dynamic response of the wall system on the overall level of tie loads. Most were content to apply "code loads" in the same way as for wind loads. Only KPFF (1989) considered earthquake input spectra, and found the resulting tie load-levels to be greatly different to the levels assumed in the code loadings.

Very few attempts have been made to model the stiffness (or dynamic response) of the whole support structure in either face loading tests or analysis. The flexible diaphragm action noted by Kenna (1963) appears to have been completely ignored to date.

None of the face-load tests reported included in-plane displacements before or after face loading. In view of the findings of Fenwick (1989) and Lapish (1991), this lack may have compromised the reported behaviour of the ties under axial load. This aspect of seismic behaviour is clearly important and needs further investigation.

All of the in-plane testing to date has been carried out at slow loading rates. Dynamic effects of in-plane action have not been investigated, although it may well be that dynamic effects are not as critical to in-plane wall performance as they are for face load action.

A number of workers have expressed concern that either "rigid" or "flexible" ties may have an adverse effect on in-plane performance. The findings of the various workers above regarding the use of "flexible" and "stiff" wall ties are not consistent. Others (Reardon and Mahendran 1988) concluded that there is no transfer of load in either direction between the brick and timber-framed walls.

The actual shear (or in-plane) behaviour of the wide, flat, strip tie now almost exclusively used in New Zealand has received very little experimental or analytical attention, and its contribution to composite in-plane behaviour should be quantified and assessed.

The other area requiring attention, as noted by several observers, is the corner effect. This is where two planes of brickwork intersect with a resulting deflection incompatibility under lateral movement of the building structure. The solution proposed by Lapish of creating a vertical separation joint at the corners, although sound technically, does not find wide acceptance among consumers. This is partly due to its cost, but mostly because it is not aesthetically pleasing, in spite of attempts by some designers to hide it behind rainwater downpipes.

All these issues need resolution if realistic levels of loading on ties are to be included in codes covering design or testing.

3.3 Code Provisions

The basic framework for building delivery in New Zealand is the New Zealand Building Code, which is contained in the first schedule of the Building Regulations (NZ Government 1992). This is a performance based code rather than being prescriptive, and is intended to encourage innovation in the building process. The structural performance requirements of the code are:

B1.3.1 "*Buildings, building elements, and sitework shall have a low probability of rupturing, becoming unstable, losing equilibrium or collapsing during construction or alteration and throughout their lives.*" This is a re-statement of the "Ultimate limit state" as defined by NZS 4203:1992 (SNZ 1992).

B1.3.2 "*Buildings, building elements, and sitework shall have a low probability of causing loss of amenity through undue deformation, vibratory response, degradation or other physical characteristics throughout their lives or during construction or alteration when the building is in use.*" This is a restatement of the "Serviceability limit state" as defined by NZS 4203:1992.

Of pertinence to brick veneer construction, durability (clause B2) and weather tightness (clause E2) provisions are also included. The range of New Zealand standards broadly covering design are cited in the Approved Documents of the Code as "Verification Methods", whereas standards specifying materials and workmanship are cited as "Acceptable Solutions". "Alternative Solutions" are offered as a means of introducing new materials, components, and construction methods which are not covered in the Approved Documents, but nevertheless comply with the Building Code.

A logical interpretation of the structural requirements is that damage to brick veneers should be minimised in a moderate earthquake or severe windstorm, and in a major earthquake or extreme wind, injury to (or loss of) life is to be minimised. Therefore, it must be accepted that some damage to brick veneer (such as cracking) will occur, but damage needs to be controlled so that people are safeguarded from injury caused by large pieces becoming detached and falling.

The current approach to the design of buildings incorporating masonry veneer in New Zealand will generally follow one of two paths:

(a) Buildings not requiring specific structural design are covered by NZS 3604 (SNZ 1990) for timber framed buildings, and NZS 4229 (SNZ 1986) for concrete masonry buildings. These codes both include prescriptive requirements, and limitations as to application, for the design of unreinforced masonry veneer cladding; they refer to NZS 4210 (SNZ 1989) for construction and workmanship requirements. These documents are cited as "Acceptable Solutions" by the NZ Building Code.

(b) Buildings which are subject to specific design by structural engineers are governed by the loading code NZS 4203 (SNZ 1992) and the masonry materials design code NZS 4230 (SNZ 1990a). Again, NZS 4210 (SNZ 1989) is cited for construction and workmanship requirements.

NZS 3604:1990 "Code of practice for light timber framed buildings not requiring specific design" (SNZ 1990) covers the requirements for construction of timber framed buildings and includes veneer claddings in an appendix. It limits the height of veneer to 4 metres (single storey only), and refers to NZS 4210 (SNZ 1989) for veneer and tie, material and workmanship requirements.

A change was introduced in the 1990 revision of NZS 3604 which significantly affects the requirements for ties. Previously, ties and their fixings were required to resist a horizontal load of twice the weight of masonry veneer supported without elongation or shortening by more than 1.5 mm. This simple test was similar to the provisions of most overseas codes of practice and did not recognise the demands placed on ties by seismic action. The new requirement is conformity with the **stiff tie** provisions of NZS 4210, as described below.

NZS 4229:1986 "Concrete masonry buildings not requiring specific design" (SNZ 1986) is the equivalent code for concrete masonry buildings.

NZS 4203:1992 "Code of practice for general structural design and design loading for buildings" (SNZ 1992) covers general methods of structural design and minimum loads. Of particular importance to veneers, the code sets limits on building deflections under both wind and seismic actions. The "requirements for parts" section of the code places limits on the required ductility of connections, and contains a method of calculating the out-of-plane loads on veneers and wall ties.

NZS 4230:1990 "Code of practice for design of masonry structures" (SNZ 1990a) is the code of practice for specific design of masonry structures, including veneers. The code identifies general design principles for veneers, including separation from the structure, the accommodation of in-service differential movement, and in-plane and face-load design to resist seismic loads. Although essentially a "design" code, the veneer section of NZS 4230 does set prescriptive requirements on veneers by imposing height limits (depending on the nature of the backing structure and the potential risk to life), thickness limits, and tie spacing. It requires the use of **flexible ties** for veneers with timber or steel backing structures and allows the use of **stiff ties** in conjunction with concrete backing structures. As with the codes mentioned above, this also refers to NZS 4210 (SNZ 1989) for material and workmanship requirements.

NZS 4210:1989 "Code of practice for masonry construction: Materials and workmanship" (SNZ 1989) specifies the materials and requirements for construction and workmanship of masonry, and thus complements NZS 3604, NZS 4229 and NZS 4230. NZS 4210 stipulates spacing and layout of veneer ties and also their class of duty and category (stiff or flexible), depending on area of use. (For example, **stiff** ties are specified for use in buildings which are within the scope of NZS 3604 or NZS 4229. Stiff **or** flexible ties may be used for specifically designed structures, depending on the stiffness of the backing structure.) In addition, Part 3 specifies tie performance requirements and prescribes methods of testing which enable compliance to be demonstrated. In recognising the randomness of seismic action and the resulting demands on veneer tie in-plane and face load performance, the test method is greatly more complicated than equivalent methods used in other countries. It is also more complicated than the previous New Zealand test method (NZS 3604, SNZ 1984a). Until this project began, there were few in the New Zealand masonry industry who were aware of this.

As a result of testing, ties are to be categorised as "stiff" or "flexible" depending on their **in-plane** behaviour, and into medium duty (MD), heavy duty (HD), or extra heavy duty (EH), depending on their strength, stiffness and ductility under axial loading. NZS 4210 comments that *"single storey brick veneer timber-framed buildings supported by a reinforced concrete or masonry foundation wall at floor level, to which veneers are attached with stiff wall ties, have proved to be effective in limiting the lateral deflections of the structure, thereby reducing seismic damage to the veneer at corners and openings".*

The distinction between stiff and flexible ties was enumerated by Lapish (1991): if ties are "stiff" under in-plane action, then building seismic response is less, and consequently tie load demands are less. "Flexible" ties do not affect building response and are also less likely to be damaged under in-plane action, resulting in more reliable face load performance. Thus, stiff ties only are permitted in the more limited buildings covered by NZS 3604 (SNZ 1990). However, apart from limiting their area of use, no provisions of the above standards recognise this difference in performance between stiff and flexible ties.

The inclusion of some prescriptive unreinforced brick veneer provisions in the specific design masonry standard NZS 4230 is inappropriate. Also, the requirements for brick veneers are inconsistent between NZS 3604 and NZS 4229. There would appear to be a case for a simplified set of criteria for the modest buildings covered by these standards, with perhaps a more encompassing set of criteria (and associated test methods) for buildings requiring specific design.

The hierarchy of Australian standards is similar to New Zealand's, with AS 1170 (SAA 1989) setting loads, AS 3700 (SAA 1988) design and workmanship, and AS 2699 (SAA 1984) covering wall ties.

AS 2699: 1984 "Wall ties for masonry construction" (SA 1984) is similar to Part 3 of NZS 4210, but does differ in two important respects: there is no requirement to assess in-plane tie performance, and the test method prescribes that ties are to be tested fully bedded in mortar, not laid dry on the bricks as in NZS 4210. This latter requirement reflects differences in construction practices in the two countries. Strength of veneer ties is assessed at an axial displacement of 1.5 mm, so the criterion is effectively a stiffness criterion. The ultimate strength of the tie is not tested. Even so, the criterion is termed "characteristic strength".

BS 1243:1978 "Specification for metal ties for cavity wall construction" (BSI 1978) first introduced in 1945 and revised several times up to 1981, prescribes three types of standard cavity tie, namely: vertical twist, butterfly, and double triangle.

DD 140:1986 "Draft for development. Wall ties" (BSI 1986) was prepared at the request of the manufacturing industry for guidance on ties not conforming with BS 1243. As well as covering cavity ties, it also describes test methods for determining strength of wall ties intended for connecting one leaf of a masonry wall to a timber frame. Provision is made for specimens constructed either in a wall or as couplets. Although the emphasis is on axial tension and compression testing, a method of in-plane shear testing is also described. The philosophy is essentially the same as that of de Vekey (1984); that is, ties are classified in accordance with their situation and location and testing is required to prove that the tie being used meets the criteria appropriate to that class.

It is notable that New Zealand, Australian and United Kingdom requirements all specify a relative vertical and horizontal movement between timber frame and veneer to simulate timber shrinkage, brick growth, thermal effects, or elastic shortening before ties are axially tested.

ASTM E754 -80 "Standard test method for pullout resistance of ties and anchors embedded in masonry mortar joints" (ASTM 1980) "*is intended to determine conservative ultimate pullout values of fasteners*". The test method merely establishes maximum axial pullout resistance of a tie in the mortar bedding of couplets. No initial in-plane displacement is required.

CAN-A370-M84 "Connectors for masonry" (CSA 1984) provides for various types of ties (on a prescriptive basis), and gives criteria for design and testing of non-standard ties. The test method subjects couplet specimens to a pure axial load up to failure, in a manner similar to ASTM E754.

4.0 BACKGROUND TO THE TEST PROGRAMME

The testing programme consisted of five series of tests on individual ties (elemental tie tests) carried out at BRANZ, and two full-size wall tests carried out at Works Central Laboratories (WCL). All the work was carried out during the period between August 1991 and July 1992.

4.1 Objectives of the Original Test Programme

The original objective of the testing programme was to obtain results from both the full size and elemental tests, to use for calibrating an analytical computer model. Once calibrated, this model was then to be used to extend the test findings to cover a range of wall configurations that may be expected to occur in practice.

A further objective was to carry out a series of tie tests to establish the stiffness and strength parameters of each of the ties readily commercially available in New Zealand. These parameters were also to be used as input for the analytical modelling phase of the work, and also for calibrating the model with the dynamic load tests.

The WCL programme was to consist of two tests. Both involved subjecting a full-size veneer/timber stud wall specimen to dynamic face loading applied through a rigid steel loading frame. In the first test, the specimen was to be fixed rigidly to the loading frame at floor and roof level, as has been done by other researchers. For the second test the specimen was to have been flexibly connected to the loading frame. This connection was intended to simulate the interaction of the flexible components of a typical timber building frame structure in which post-elastic degradation had just commenced, which has not been studied previously. (This avenue was first suggested by Kenna [1963], but not subsequently followed up).

4.2 Problems

During the initial phase of the elemental tie test programme, a number of problems were encountered. The most crucial of these subsequently affected the whole research programme and required adjustments to several of its objectives.

This crucial problem was the lack of anchorage of the veneer ties, caused by looseness of the ties in the bedding mortar. The nature of the problem is described in more detail in section 5.1. The consequence was that the first series of elemental tie tests at BRANZ was not completed, because both stiffness and strength values were well below expectations based on the NZS 4210 (SNZ 1989) criteria.

The first WCL test was also affected by the same problem. This is described more fully by Lawrance and Stevenson (1992) and summarised in section 5.5.

That the same problem was encountered in both the tie specimens and in the first full-size specimen, led to the supposition that the situation may well be widespread throughout New Zealand in buildings using face nailed ties.

The second problem was that the wording of the code (NZS 4210) was abstruse and very hard to understand, especially relating to tie performance criteria and test procedures. In fact, it was necessary to correspond with SANZ in order to devise a test method that met the intent, if not the actual wording, of the code.

Once the second problem was overcome, and the tie tests conducted, yet another problem arose. This was that none of the ties tested met the criteria specified by the code. The area of non-compliance which all of the ties fell most short of was in-plane stiffness, but many also failed in axial strength and in other more minor ways.

4.3 Revised Programme

These problems meant that the main thrust of the first stage of the project shifted from an analytical investigation into the composite behaviour of the whole wall, to one that focused more sharply on the veneer ties. Because the fundamental purpose of the ties is to anchor the veneer to the backing structure, and because both tests had demonstrated that this was not occurring, it was felt that solving the tie problem was the first priority, and that other phases of the project should be postponed. A similar conclusion on the importance of the ties (although arrived at for different reasons) has been reached by other researchers (e.g. Grimm [1976], de Vekey [1984], Lapish [1988]).

The second of the originally intended WCL tests (using flexible connections to the bracing structure) was deferred. In its place the first test was duplicated, but with measures taken to ensure that the ties would be adequately anchored. As a result, the second test achieved the objectives intended for the original first test. Subsequent attempts to attract FRST or industry funding have to date proved ineffective, and this phase of the work remains to be completed.

Because the results of the second (originally planned) WCL test were not available, the intended correlation with the analysis model was not possible. The analytical phase of the work has therefore been deferred. In its place, a revised series of elemental tie tests was undertaken to give a better understanding of tie behaviour. The second and third series of tie tests were designed to investigate the difference in behaviour between 90 series (90 mm wide) and 70 series (70 mm wide) bricks. This was partly in response to requests from the masonry industry, which was concerned to find out whether there was any change in performance between the traditional 90 mm brick and the narrower bricks permitted by the new standard (NZS 4210:1989). A further series of tie tests (series 4) was designed to investigate the construction and workmanship factors causing tie looseness in mortar bedding.

4.4 Tie Test Method

Because the test method used for assessing tie performance became fundamental to the project, it will be described here rather than in the description associated with the individual tests.

4.4.1 Test procedure

NZS 4210, Appendix 3C (SNZ 1989) describes a testing procedure for determining the mechanical properties of veneer ties.

The procedure differs markedly from the requirements of earlier New Zealand standards, and also of the Australian (AS 2699, SA 1984), British (DD 140, BS 1986), Canadian (CAN3-A370-M84, CSA 1984), and American (ASTM E754-80, ASTM 1980) standards. The major difference is the requirement for displacement in the plane of the wall (shear loading of the tie) in addition to the more usual face loading (axial loading of the tie). This test regime was introduced to the New Zealand standard to simulate the response of flexible (relative to the stiffer veneer) framed structures to earthquakes. Veneer ties are required to remain functional after loads and displacements induced by random ground motions from any direction. This is a severe test, and it is therefore to be expected that ties shown to be suitable for use under overseas standards, still may not meet NZS 4210 criteria.

In essence, the method requires that brick couplets, together with a tie and short length of stud, be subjected to cyclic loading in the horizontal plane of the wall (in-plane or shear loading), followed by a single displacement in the vertical plane of the wall (to simulate timber shrinkage and/or brick growth). This is then followed by cyclic loading in the axial direction of the tie (to simulate seismic face loading).

However, a study of the detailed procedure of NZS 4210 revealed that the test method and assessment of results to determine criteria were far from clear. Correspondence with Standards New Zealand clarified some ambiguities, but the test methods actually used for this project are still open to some doubt in terms of whether they absolutely fulfil the test requirements of NZS 4210 Appendix 3. However, they are believed to follow the intent, if not the actual wording, of the standard. For this reason, the test procedures used in the various series of elemental tie tests are described in detail in Appendix 1.

4.4.2 Test set up

The test rig was developed specifically to carry out tests on veneer ties in accordance with the method specified by Appendix 3.C of NZS 4210. The test set-up is shown diagrammatically in Figure 1 and in Photograph 1.

Brick couplets, complete with the tie and a short length of stud, were clamped in the base. Polystyrene packing was used under the holding down clamps to avoid damaging the bricks or breaking the mortar bond. Clamping screws "A" and "B" have sufficient travel to provide the means to impose the in-plane vertical displacement required by the test. The side clamping system bore directly against the sides of the bricks without packing thus preventing any movement of bricks with respect to the base. The two actuators with their connectors and the stabiliser bar, act together to restrain the stud from twisting or tilting with respect to the brick. During the in-plane section of the testing, the stud suffered minor twisting due to the eccentric configuration of the tie connection. In a real wall, with only the internal lining to restrain it, it may be expected that the stud would also be likely to undergo some twisting, so it is believed that this was not a serious shortcoming in the test set-up.

In-plane cyclic loads were applied to the stud by actuator "2" acting through a load cell, universal joint and connecter. Resulting displacements were measured by potentiometer "2". Axial (or out-of-plane) loads were applied to the stud by actuator "1" acting through a load cell, universal joint and connector. Resulting displacements were measured by potentiometer "1".

4.4.3 Test equipment

Actuators 1 and 2 were 30 kN and 100 kN respectively, servo-controlled, electro-hydraulic rams. The load applied by actuator 1 was measured by a 4.5 kN load cell; the actuator 2 load was measured by a 90 kN load cell. Both linear potentiometers have been calibrated to an accuracy of +/- 0.20%.

Loads and displacements were recorded using an IBM-compatible PC, running software to record data in real-time mode. Data reduction, plots and graphs were processed using proprietary spreadsheet software.

5.0 TEST PROGRAMME

5.1 Elemental Tie Tests, Series 1

5.1.1 Construction of test specimens

Specimens incorporating tie types 1, 2, and 3 were constructed using 90 series clay bricks.

Because the method of construction of the specimens is of crucial importance in the performance of veneer construction, the method used will be described in detail, following a description of trade practice as normally followed by bricklayers:

Normal trade practice

(a) The backing frame is erected, bracing installed (if used), and building paper attached.

(b) The required number of courses of bricks are laid on the foundation.

(c) Ties, at the required spacing, are laid directly on top of the upper brick course.

(d) *Ties are nailed to the faces of the studs of the backing frame.*

(e) Mortar is placed on top of the ties and subsequent courses of bricks laid.

Construction of test specimens

(a) The backing frame consisting of 2.4 m high, 100 x 50 timber studs was erected. To reduce the number of bricks used, studs were spaced at 300 mm centres (instead of the more usual 600 mm) and connected to common top and bottom plates, in accordance with the provisions of NZS 3604.

(b) One course of bricks was laid with a 50 mm cavity spacing and open perpend joints for ease of dismantling (as required by NZS 4210).

(c) Ties, one at each stud position, were laid directly on top of the bricks.

(d) Ties were nailed to the faces of the studs of the backing frame.

(e) Mortar was laid on top of the bricks and a second course laid in stack bond.

(f) The third course of bricks was positioned on a polystyrene bond breaker to facilitate dismantling.

(g) Subsequent courses were laid similarly, repeating steps (b) to (e).

(h) After 28 days, the specimens were dismantled and studs carefully sawn into suitable lengths for testing.

Steps (b) to (g) were undertaken by a registered bricklayer experienced in this type of work. The general set-up is shown in Photograph 2.

This construction method was different in several respects from that specified by NZS 4210 (SNZ 1989), but was adopted for the series 1 tests because it was expected to more closely reflect the condition of ties in practice. Although the mass of bricks associated with each tie was rather less than in practice, it was still enough to demonstrate that stud vibration had a significant effect on tie anchorage. Thus tie anchorage in a "real" building is likely to be less effective than shown by these tests. The identification of the anchorage problem at this early stage proved to be of great significance.

Upon disassembly for testing, it was found that most ties were slightly loose in their mortar bedding. Subsequent discussions with the bricklayer suggested that the hammering action while driving the tie fixing nails was sufficient to deflect the studs enough to disturb the partially hardened bedding of the row of ties below. It was also suggested that the method of construction incorporating the polystyrene bond-breaker, and the closer than normal vertical spacing of the ties, may have unduly adversely affected the bond.

5.1.2 Observations and results

Figure 2 shows a generic veneer tie in both the (a) "as constructed" and (b) "displaced" configurations immediately before the test. Because of the need for face fixing to the framing over building paper, and the requirement to provide hammering space, all three types of ties tested had a dimension "d" of about 15 mm. This offset between the axis of the tie and the level of the fixing point allowed the tie to twist about its fixing under in-plane displacement (Figure 3). This twisting action was accommodated by vertical displacement of the fixing nails in the timber of the stud. As a result, the ties were generally able to withstand the cycles of ±10 mm in-plane displacement without noticeable distress, although the nails (or teeth) of the type 3 ties were partly withdrawn from the timber.

However, the lack of anchorage caused by the looseness of the ties in their mortar bedding produced erratic results, particularly under axial displacements. Loads and stiffnesses achieved under the applied displacements were well below the levels set by the criteria of NZS 4210 Part 3 and the tests were discontinued.

5.2 Elemental Tie Tests, Series 2

In an effort to obtain useful results which could be used to calibrate other sections of the project, the construction method of the second series of specimens was changed from that of the first series, thus avoiding loose ties in the mortar. This was done by constructing the specimens as isolated couplets, thereby allowing prenailing to the studs (Refer to Photograph 3). In all other respects the tests were the same as in the first series.

5.2.1 Construction of series 2 test specimens

Specimens were constructed using 90 series clay bricks. The same bricklayer used for series 1 constructed this series of specimens as well.

(a) The ties were nailed to studs which had been pre-cut into suitable lengths for fitting into the testing rig.

(b) Each tie was placed directly on top of the lower brick.

(c) Mortar was applied over brick and tie, and the second brick laid in the normal trade method.

(d) Specimens were stored under cover for 28 days.

This construction procedure was the same in all important respects as that of Appendix 3.C of NZS 4210.

In all, 23 type 1 ties, 19 type 2 ties, and 22 type 3 ties were tested under the series 2 tests.

5.2.2 Observations and results

No ties moved in the mortar bedding in any part of the test regime. It seems apparent that as long as the tie is adequately bedded during construction, then failure will not occur in the tie anchorage to the brick. If failure does occur, then it will be elsewhere in the veneer fixing system.

The same twisting action of the ties under the cycles of in-plane displacement observed in series 1 tests was also apparent in this series. The distortion of the ties resulting from this part of the test is evident in Photographs 4, 5 and 6, showing typical examples of each of tie types 1, 2 and 3, respectively.

In tie type 1, the nails did not noticeably withdraw under this action, and subsequent behaviour was not badly affected. The tie appeared to behave in a predictable manner with all of the axial cyclic displacements accommodated by the bending of the steel body of the tie. Photograph 7 shows a typical example of a tie on completion of the test.

The vertical movement of the nails of tie type 2 induced by twisting caused them to partly loosen in the timber, with some resulting reduction of strength and stiffness in the later parts of the test. Also, during the axial displacement cycles, ductile behaviour didn't happen under load cycles starting with compression because the ties tended to buckle rather suddenly on the first compression cycle. This had the effect of withdrawing the nails sufficiently to severely reduce the axial load-resisting capability in subsequent cycles (Photograph 8). The reason for the difference in this compression behaviour compared with tie type 1, is the increase in metal thickness. The resulting stiffer section has the ability to lever the nails out of the stud under the buckling action, compared with the rather more "gentle" bending action of tie type 1.

Tie type 3 was more seriously affected by the twisting action of the in-plane displacement. Although the body of the tie coped well with the distortion, the nails (or teeth) were quite noticeably withdrawn from the timber. Had the ties been fixed through a sheet of plywood, as is often the case when wall bracing is present, then the grip of the teeth remaining in the stud would have been minimal. A further effect on this tie of the in-plane displacement cycles is that the tightly bent section of metal where the teeth join the flat section is severely strained (see Photograph 6). This has caused work hardening of the steel, leading to embrittlement and tearing of several of the ties.

The result of this is that, during subsequent axial cyclic loading, some ties failed by nail withdrawal (Photograph 9), some failed by nails breaking off (Photograph 10), and some survived the test (Photograph 11). The unpredictability of failures rules out any possibility of ductile behaviour, which is desirable for good seismic performance.

Typical examples of load/displacement graphs for the three types of ties are shown in Figures 4 to 8. The hysteresis loops under in-plane cycling of tie type 3 (as shown in Figure 4) are well shaped with minimal drop off in load resistance after the first cycle. However, the observed damage to the tie is not apparent from this result, and in any case, the code stiffness criterion under in-plane displacement is not met. Of particular note is the axial compression loading achieved by tie type 2 before buckling occurred. After the initial high load, the fall-off in resistance on subsequent cycles can be clearly seen (Figure 7).

Under cycling beginning with tension, most of the ties exhibited load/displacement graphs for which the deflection at first yield (and therefore ductility), as defined by NZS 4210 Appendix 3.C, was extremely difficult to determine. In fact, the shape of the graphs was more indicative of structural components based on nail/timber interaction rather than the elasto-plastic behaviour assumed by NZS 4210. The contrast between this type of hysteresis loop (e.g. Figure 5) and the NZS 4210 loops (reproduced in Figure 9) is clear.

However, under cycling beginning with compression, the buckling behaviour on the first cycle shows on the graphs as though it were the ductile behaviour as defined by NZS 4210. This is shown in Figure 6 (the first cycle appears in the bottom left quadrant).

Results of the series 2 tests are set out in Table 1. For comparison, the NZS 4210 criteria are also included.

5.3 Elemental Tie Tests, Series 3

With the introduction of the 1990 editions of both NZS 3604 and NZS 4230 (SNZ 1990), the minimum allowable masonry veneer thickness was reduced from 90 mm to 70 mm. The "Kiwi Brick" as it is known in Australia, is becoming increasingly popular in both countries, particularly for use in low-cost housing. This has resulted in considerable industry interest in whether the narrower brick, with its smaller embedment distance of 35 mm, would perform the same as the more "standard" 90 mm brick with its 45 mm embedment.

To determine this, a third series of tie tests was carried out, using 70 series clay bricks.

Three specimens of each tie type were subjected to the same cyclic test as series 2 above. One further specimen of each type was then modified in the way described below, in an effort to simulate a configuration which would be capable of passing the water transfer test of NZS 4210 Appendix 3.B. This test method requires that veneer ties for use with timber frame backing shall be displaced before testing, such that the frame end of the assembly is lowered by 20 mm with respect to the veneer end, to simulate both timber shrinkage and brick growth. Figure 10a shows a tie of current configuration displaced in accordance with this requirement. The test then requires that water running down the inner face of the bricks will not wet the blotting paper. It is self evident that the tie set up like this cannot pass the test. Figure 10b shows a tie which has been altered to ensure compliance with the water transfer test. The three specimens were modified in accordance with Figure 10b before being subjected to the same test sequence as the other specimens.

To simplify the test, while at the same time subjecting specimens to a load regime which would fully test the embedment depth comparison, the monotonic in-plane stiffness test was omitted. In every other respect, the tests were the same as the series 2 tests.

At the same time, a sample of Australian-made ties were tested (type 4) to both NZS 4210 and AS 2699 (SA 1984) test methods. The load resisted with the ties pre-displaced as required under AS 2699 was so low that further specimens were tested without the pre-test displacement.

A total of four type 1 ties, four type 2 ties, four type 3 ties, and six type 4 ties was tested in the series 3 tests.

5.3.1 Observations and results

Observed tie behaviour was substantially the same as for the series 2 tests. At no stage of the tests did any of the ties become loose in their mortar bedding. The same twisting behaviour (under in-plane displacement) was observed as occurred in series 1 and 2 tests. Also, the change in configuration for compliance with the water transfer test appeared to have no noticeable influence on the tie behaviour.

Because very few ties exhibited the elasto-plastic type "ductile" behaviour required to assess their performance under the code criteria, the results comparison of Table 2 shows only the peak load resisted. (The minimum value under either tension or compression). Unfortunately, due to an equipment malfunction, no axial results were produced for tie type 2.

Typical load/deflection graphs are presented in Figures 11 and 12. Figure 11 may be directly compared with Figure 4, which shows the same tie under the same loading, but using 90 mm brick. Figures 11 and 12 indicate comparison of ties with and without the 20 mm displacement for the water transfer test.

Although the size of the sample means that results are only indicative, it seems that the 20 mm displacement had little significant effect on tie behaviour. Also, the difference in results achieved between ties using the different embedment distances for the 70 and 90 series bricks is insignificant and within the range of results achieved for any of the ties separately.

5.4 Elemental Tie Tests, Series 4

In view of the poor performance of ties inadequately anchored due to looseness in the bedding mortar at the time of loading, a further series of tests was carried out to investigate the influence of the construction method on this anchorage. A veneer wall was built using different tie types, fixed and bedded in different ways for comparison. The wall was built by the same bricklayer as for the previous series of tests.

5.4.1 Specimen construction

The arrangement is shown diagrammatically in Figure 13 and in Photograph 12. Construction was designed to simulate as closely as possible actual on-site practice, while still allowing dismantling into specimens for testing. Polythene bond breaker strips were installed in the mortar joints above and below each tie course to facilitate separation. The ties indicated in the figure as N (non-bedded) were laid directly on the brick below as for normal New Zealand trade practice, whereas others (indicated as B [bedded]) were laid on a bed of mortar before being fixed to the stud. These latter ties were also covered with mortar as the next course of bricks was laid. Thus, they were completely embedded within the mortar. Ties marked as

N were fixed using the two plain galvanised nails supplied with the ties. Ties marked as A were fixed with two annular grooved nails. Ties marked as S were fixed with one self-drilling screw.

During bricklaying, the relative movement between studs and bricks under the hammering action of tie installation was monitored with a potentiometer (see Photograph 13). Studs moved up to 3mm under this action, measured at the tie below the one being installed. The lower tie was clearly observed moving in the partially hardened mortar.

After 28 days, the wall was dismantled to provide couplet specimens for testing. Unfortunately, in spite of the precautions taken during construction, the wall proved extremely difficult to dismantle. Both the sawing of studs into testable lengths and breaking of the brick perpend joints created disturbance to the mortar joints containing the ties. Several of the specimens had to be rejected before testing because of disturbance to the tie anchorage; several test results may be suspect for the same reason.

5.4.2 Test procedure

Specimens were set up in a 10 kN Instron Universal Testing Machine, with an axial compression load of 100 kPa applied perpendicular to the bedding joints to simulate support from the surrounding brickwork. The stud section was gripped in the upper jaws of the test machine and the tie subjected to an axial force under deflection control until peak load was reached. Load and deflection data were recorded and the mode of failure observed.

5.4.3 Observations and results

Results are summarised in Table 3. Specimens damaged and not tested are marked as such. A failure mode marked as "mortar" indicates that the tie pulled out from the mortar joint; "nail" indicates that the nails pulled out from the timber.

There appears to be no noticeable trend in the results up the height of the stud. The difference in bedding method is not significant for tie type 1, which has a well formed anchorage portion. Tie type 2, whose anchorage is almost flat, is sensitive to the method of bedding. Tie type 3 has by far the best anchorage, but has very poor nail-holding capacity.

Performance of tie types 1 and 2 is greatly enhanced using annular grooved nails or screws. Screws, in particular, are a very promising method of fixing because they cause little disturbance to the mortar bedding during installation.

Unfortunately, because of the disturbance caused while dismantling the wall, results will have to be regarded as indicative only but are, nevertheless, valuable in helping to isolate the effects that construction practices have on tie behaviour.

5.5 Elemental Tie Tests, Series 5

This series of tests was primarily intended as a means of developing tie fixings and anchorage, and also as a first step in deriving an alternative tie test method to that of NZS 4210 Appendix 3.C.

The results of the series 4 tests had suggested that screw fixing of the ties to the studs was a promising avenue for achieving improved performance. The first stage of the series 5 tests was therefore designed to help select a suitable screw which could be used for the remainder of the test series. Parameters studied were driveability and withdrawal resistance.

To obtain further comparative data on the performance of bedded and unbedded ties, two sets of brick couplet specimens were made, one with ties laid directly on the lower brick, and one with ties fully embedded in the mortar.

To determine the effects of in-plane displacement on axial load capacity, the remaining series 5 tie specimens were held directly in the test rig without brick couplets and subjected to varying amounts of pre-test in-plane displacements. The appropriate amount of in-plane displacement for a standard test procedure will depend on a study of the composite in-plane behaviour of veneer walls; as such, it is outside the scope of this phase of the project.

5.5.1 Fixing selection

A range of screws commonly available to the building industry and judged to be of suitable size, were used to fix type 1 ties to short lengths of stud. For comparison, three specimens were similarly fixed with the nails supplied by the manufacturer for use with the type 1 ties. All specimens were then loaded in tension until failure.

5.5.1.1 Specimen construction

To assess the driveability of each fixing, the specimens were constructed while clamped to a timber stud fixed at top and bottom, as would be the case in a normal frame installation. The general arrangement is shown in Figure 14. While the tie fixing was being installed, the potentiometer recorded stud deflection. The recording speed was set at 200 Hz to capture the transient information. In a normal site situation this deflection, transferred through the stud, is what affects the anchorage of the lower ties (by loosening them in their partially hardened mortar bedding). Table 4 shows the stud deflections measured during the installation of each of the fasteners. The plots of Figure 15 show the contrast between the stud deflections recorded during the installations of a typical screw and nail fixing.

5.5.1.2 Fixing pullout tests

The specimens were tested under axial tension loading in a Dartec Universal Testing Machine. The stud section was bolted to the base of the test machine and the tie and fixing subjected to an axial force under deflection control until peak load was reached. The maximum pullout load recorded is tabulated in Table 4, along with (where appropriate) the load specified by the relevant standard.

The No 12 x 40mm Hex head self drilling screws (Type 17 of AS 3566 [SAA 1988]) were chosen for the tests of the type 1 ties because they had adequate load capacity, and because the hex head is robust enough for site installation using the cordless screwdrivers commonly available to the building industry. The screws are also available in a variety of corrosion resistant finishes as required for roof fixing installations.

Countersunk self-tapping screws (35 x 8g) were used to fix the type 4 ties, as these most closely fitted the hole provided in the tie. In addition, some of the type 3 ties were partly straightened, drilled and fixed to the studs using the same screws.

5.5.2 Tie testing

5.5.2.1 Specimen construction

Some of the specimens were built as couplets, as for the series 2 tests. Roughly half of these had the tie laid directly on the lower brick, whereas the other half were fully embedded in the mortar.

The remainder were cast into dental plaster and set in a steel RHS section mould. This mould was then clamped in the test rig as shown in Photograph 14. The change in specimen set-up was done to save the 28-day delay period while the brick couplet mortar cured, allowing a quicker test turnaround. As this test series attempted to study the test method and tie fixings rather than the tie anchorage, the actual nature of the tie attachment was not considered critical.

Table 5 indicates the construction method used for each specimen.

5.5.2.2 Test procedure

The loading regime was based on the procedure of Park (1989).

Each specimen, while prevented from twisting or tilting, was subjected to four cycles of in-plane displacement to + or - either 0, 10 or 20 mm. The specimen was then displaced in the tie's axial direction using the following sequence:

+2 mm, -2 mm (1 cycle)
+5 mm, -5 mm (2 cycles)
+10 mm, -10 mm (4 cycles)

About half were displaced in tension first and the other half in compression first. Axial load and deflection data were recorded and the tests were videotaped.

5.5.2.3 Observations and results

Similar tie behaviour was observed as for test series 2 and 3. One of the partly bedded type 1 ties was observed to be loose in its mortar during the test, even though it was built as a couplet. This resulted in substantially lower loads being resisted for this specimen.

Results are summarised in Table 5. Each value tabulated is the maximum load resisted by the tie at the completion of the displacement cycle indicated (2 mm, 5 mm, or 10 mm). A negative value indicates tension loading.

Examples of two axial load/displacements graphs are shown in Figures 16 and 17.

A means of interpreting these results is included in section 8.

5.6 Works Central Laboratory Tests

These two tests, carried out in collaboration with BRANZ, are fully described by Lawrance and Stevenson (1992). The work consisted of subjecting two full-sized, two-storey veneer walls to increasing amplitudes of sinusoidal accelerations until failure. Although the specimen design was outside the scope of the masonry design code (NZS 4230), in that flexible ties are required for veneer heights greater than 4 m when backed by a timber frame, the test explored how stiffness of the backing structure influenced the composite face load behaviour. In-plane tie flexibility was not considered critical for a study of face load behaviour, and so the more commonly used stiff ties were chosen. Anchorage of the veneer ties in the first wall was inadequate; for the second, precautions were taken during tie installation to ensure adequate anchorage in the veneer. Practical means to achieve the same result were the objective of the series 4 and 5 elemental tie tests already described. Significant findings from the tests worth repeating here are: (Quotes in *italics* are verbatim from Lawrance and Stevenson [1992]).

"It was noticed that the tie was placed on the lower brick and nailed to the studs prior to the mortar being applied ... As a consequence, little mortar, if any, extruded beneath the tie, reducing the bond area." This was referring to the construction of the first wall, and describes standard New Zealand bricklaying practice. For the second wall, special measures were taken to ensure that the ties were well anchored. Investigation of these measures was the primary reason for the series 4 elemental tie tests covered in section 5.4 above.

Only a sinusoidal acceleration input at a frequency of 5 Hz was possible because of equipment limitations. The response of the walls to a random frequency input typical of earthquake records and, in particular, at a much greater displacement amplitude, would have been an interesting extension to the test. However, the two specimens, which were identical in all respects except for the adequacy of the tie anchorage, did enable a comparison to be made between their respective dynamic performances.

Tie forces, veneer accelerations, timber and veneer displacements, all increased as the level of input acceleration was increased (for both walls). Significantly, however, this increase occurred at a lower level of input acceleration in the first wall than in the second. Veneer cracking also occurred at lower acceleration levels for the first wall. The natural frequency for the first wall was consistently lower, due to the tie slippage. As might be expected, the damping was also higher. The total wall reactions measured at first floor and roof levels were, however, similar for both walls.

"For both walls, larger tie forces at floor and roof levels where connections were adequate would indicate that more ties should be placed at these levels." This in fact was only true at low acceleration levels. At higher acceleration levels this isn't so conclusive. In particular, for the wall with loose ties, the peak tie forces varied during the course of the test as various tie connections became loose and shed load to those more able to accept it.

"The failure of the roof connection had initiated the collapse of the veneer." This refers to the final failure of the second wall, at an input acceleration about twice that of the first one.

The most striking observation was the great difference in performance, once steps had been taken to ensure adequate tie anchorage. This was graphically demonstrated to industry representatives present during the first test, when a large portion of veneer became detached and fell in a most alarming manner at the end of the test.

6.0 TIE COMPLIANCE WITH STANDARDS REQUIREMENTS

The criteria of NZS 4210 for stiff ties require that in-plane characteristic stiffness and in-plane characteristic cyclic strength are not less than 500 N/mm and 900 N, respectively. Results from Table 1 show that none of the ties tested using the NZS 4210 procedure met both criteria. They may not, therefore, be classified as stiff ties. The standard also requires that flexible ties must not reduce the cavity width by more than 2 mm when accommodating in-plane movement. NZS 4210 does not specify what this movement should be, but if we assume that it is the vertical and horizontal displacement for flexible ties given in Table 3.C.1, then simple geometry will show that the strip tie will decrease the cavity width by greater than 2 mm for a cavity width of 50 mm, thus not complying with NZS 4210.

Types 1 and 2 ties met the axial stiffness criteria (240 N/mm) for medium duty ties in compression, but not in tension (individual ties of all three types met the stiffness values). NZS 4210 is not clear if this would be a "pass" or "fail" of this section of the criteria. Because none of the ties conformed with the ideal "elasto-plastic" axial behaviour as envisaged by the standard, none could be considered ductile. Additionally, only the type 2 ties met the criteria for characteristic strength of elastic ties (1400 N), in compression only, but not the characteristic cyclic strength for ductile ties (700 N).

Thus, none of the ties tested comply with the NZS 4210 provisions for either stiff or flexible ties. Whether these criteria are realistic in the context of stiff veneer ties is debatable, and this is discussed in the conclusion to this report.

7.0 TIE SURVEY

Once the problem of inadequate tie anchorage caused by looseness in the bedding mortar had been revealed by the elemental tie tests, and confirmed by the construction of the first WCL specimen, it seemed reasonable to suppose that the problem could be widespread throughout New Zealand. This could have serious consequences. Buildings most likely to be at risk are those constructed since the early 1970's, using veneer ties fixed by nailing to the face of the framing. To ascertain the magnitude of the problem, an informal survey of brick clad houses under construction was carried out.

With the help of the brick industry, two houses in the Wellington area and five in Auckland were chosen and surveyed. These houses were selected so they were all at that relatively brief stage of construction of having veneer completed, but no internal linings in place. Five locations in each house were selected at random and the anchorage of the ties checked from inside the house by removing a portion of building paper and observing the behaviour of the tie when the stud was hit sharply with the heel of the hand. Looseness of the tie was easily picked up visually by relative movement at the junction of the tie (attached to the stud) and the mortar (restrained by the mass of the veneer). Every tie examined in this way was found to be loose. We concluded that the problem is widespread in houses constructed in New Zealand since nailed, face-fixed ties became universal in the 1970's. Bricklayers spoken to during this survey acknowledged the likelihood of this, but seemed to be unaware of its significance.

At one of the Wellington houses, the bricklayer was given some type 2 ties (which have a suitable hole already formed) and 12g x 40 self-drilling hex head screws, and asked to construct a portion of the work using the same fixing method as was used for the series 5 tests. Photograph 14 shows tie installation under way. Once the background was explained to him, there was no reluctance on the bricklayer's part to adapt to the new method of installation. Indeed, the opinion was expressed that as long as all bricklayers were treated equally, the trade as a whole would accept the imposition of screw-fixed ties. Although no times were recorded, observers thought that screw fixing took little longer (if at all) than nail fixing.

8.0 DEVELOPMENT OF TIE PERFORMANCE CRITERIA

The New Zealand masonry industry urgently needs two things: a definitive statement of performance criteria for veneer ties; and the development of methods to establish compliance. A good starting point would be the basic requirements identified by de Vekey (1984). AS 2699 (SAA 1984) also contains a succinct list of performance requirements. Although neither list contains specific reference to a seismic environment, the additional requirements are relatively small and concern the nature of the applied load rather than product performance.

Having regard to the above, we suggest the following requirements are needed to achieve acceptable performance, as defined by the New Zealand Building Code:

(a) The tie must prevent the transfer of external moisture from the veneer to the internal frame.

(b) The tie must be sufficiently flexible to allow in-plane differential movement between veneer and backing frame during normal in-service conditions. It must also cope with the inevitable installation tolerances associated with the interface between two trades (e.g. bricklaying and carpentry).

(c) Under serviceability levels of loading, the axial stiffness of the tie, in conjunction with the backing structure, must be sufficient to avoid unacceptable cracking of the veneer.

(d) Under ultimate seismic actions in any direction, the tie must prevent the veneer from becoming dislodged and falling. Veneer damage at this level of load is expected and acceptable. Replacement of damaged portions is quite feasible.

(e) The tie must be sufficiently durable to perform these functions over the expected life of the structure.

External moisture and durability - items (a) and (e) are being studied by others, and although they are vital issues for wall performance, the main thrust of this research effort is structural considerations. Therefore, the following discussion of performance criteria is concentrated on the mechanical requirements of items (b), (c) and (d).

Although the basic concept of the mechanical part of the test regime of NZS 4210 (SNZ 1989) is appropriate for a seismic region, the performance criteria implied are open to question. In addition, the test methods prescribed to establish compliance are confusing, and need to be simplified and clarified.

The tests discussed in this report amply demonstrate that the method of tie installation has a vital influence on tie performance. Therefore the method of specimen construction must closely simulate actual on-site practice to enable test results to adequately represent real site performance. Thus, a simple means of specimen construction that models these effects needs to be established.

To simulate how the random ground movement of earthquakes influences tie behaviour, the inclusion in the test regime of in-plane displacement cycles is clearly desirable. The magnitude of that displacement remains to be determined. This procedure forms part of the test method of NZS 4210 (SNZ 1989) but indications are that the ±10 mm figure may be too low, especially if the procedure is to be used for increased veneer heights.

Recording in-plane behaviour of the tie adds considerable complication to what should be a simple test, and adds a further set of criteria that must be assessed. The advantage of such information is the ability to determine where a tie may be used depending on its in-plane stiffness. This is an aspect that requires further study.

The main technical shortcomings of the currently available test methods, and criteria of NZS 4210 centre around the need for, and definition of, ductility. Park (1989) addresses this problem for structural assemblages which are able to be designed using structural engineering methods. Veneer ties, on the other hand, are generally considered as non-structural components and are seldom subjected to detailed engineering scrutiny. In brief, Park's test procedure consists of two parts: a load-controlled sequence to establish the initial yield displacement, followed by a displacement-controlled sequence to establish available displacement ductility. The load-controlled sequence is dependent on being able to predict beforehand what the ultimate strength of the assemblage will be. This would not be true in the case of a simple routine test of a component such as a veneer tie. However, it is becoming a widely accepted procedure and can be used as a starting point.

There is a further difficulty in testing assemblages whose configuration (and therefore, mode of behaviour) is variable or unknown. In veneer ties, depending on the configuration, some types act in direct tension or compression, some in bending, and most in a combination of all three. For example, Figures 16 and 17 show that the tie appears ductile under compression loading (top right quadrant). In fact, this shape is caused by buckling of the tie, causing its behaviour to change from axial compression to bending. A definition of first yield in accordance with normally accepted principles (Park summarises several methods) would, in this instance, actually be defining the buckling load of the tie. Under tension loading (lower left quadrant), ties that are well anchored are able to utilise the tensile strength of the metal, whereas other tie configurations are acting in bending.

In addition, veneer ties are not really required to behave in a ductile manner during earthquakes in so far that displacement ductility is considered a primary prerequisite of a building's response to seismic events. Rather, as secondary elements, they are required to exhibit **toughness.** Toughness is defined in this context as the ability to survive a number of cycles of (inelastic) displacement without a significant loss of strength.

The cyclic displacement regime proposed by Park, when applied to veneer ties in the manner of the series 5 tests, is capable of demonstrating a specimen's toughness in an intuitive, visual manner on observation of the load/deflection graphs. For example, compare Figure 16 with Figure 17. However, a suitable performance indicator is required to quantify the tie's ability to accept face loading. The requirements for such a performance indicator are:

(a) It must reflect the toughness of the specimen;
(b) It must give some measure of load-resisting capability; and
(c) It must be straightforward to determine.

One possible performance indicator, which is easy to extract from the test data and does not depend on prior assumptions of behaviour mode, is the accumulated load resisted at the maximum displacement of each load cycle (both tension and compression taken as additive). An alternative is to terminate the accumulation when the load has reduced to 80% of the maximum value reached (either tension or compression, whichever occurs first).

For example, using the load/deflection graph of Figure 16:

$$PI = 1.12 + .83 + 1.76 + 1.64 + 1.71 + 1.56 + 1.56 + 2.81 + 1.37 + 2.32 + 1.22 + 2.12 + 1.12 + 1.9 = 23.1$$

$$\text{Alternative PI} = 1.12 + .83 + 1.76 + 1.64 + 1.71 + 1.56 + 1.56 + 2.81 = 13.0$$

and for Figure 17:

$$PI = .59 + 1.1 + .51 + 1.27 + .59 + 1.39 + .54 + 1.17 + .42 + 1.03 + .29 + .95 + .20 + .85 = 10.9$$

$$\text{Alternative PI} = .59 + 1.1 + .51 + 1.27 + .59 + 1.39 + .54 + 1.17 = 7.16$$

This indicator is a good "filter" of tie performance in that it assesses how well the tie can sustain load from beginning to end of any nominal load sequence. That load sequence need not follow in detail the response of a structure to an actual earthquake. As Park suggests: "*Instead, a more simple displacement history can be applied to enable an assessment to be made as to whether the structure is tough enough to be likely to perform satisfactorily during a severe earthquake.*" However, there is no direct relationship between load applied to the tie, and the performance indicator. This needs further investigation.

This procedure was used for the series 5 tests and proved quick and economical to carry out using manual displacement control of the hydraulic actuator. Specimen construction and installation in the test rig took up most of the total test time.

Future work on this project will be based on these test procedures but there is a need quantify the performance indicator with respect to the forces that the ties are required to resist in a real structure under dynamic excitation. There is also a need to confirm, by means of a study of in-plane composite behaviour, whether or not the recording of in-plane data, with its additional assessment demands, is necessary.

9.0 FUTURE WORK

The continuing research under the project aims primarily to investigate the criteria discussed above. However, there are wider issues to be addressed. These are:

(a) Composite behaviour between veneer and backing under both in-plane and out-of-plane actions;

(b) Incompatibility at corners between intersecting planes of veneer undergoing lateral deflections;

(c) Differences in behaviour between concrete and clay bricks as they affect seismic performance;

(d) Level of loads applied to the structure under dynamic seismic actions;

(e) Level of displacement at which cracking occurs in the masonry; and

(f) Benefit or otherwise gained by deliberate weakening of the mortar courses to allow controlled cracking of the veneer.

Work undertaken under the current research programme (93/94) includes the following:

- Static in-plane testing of a representative veneer wall with a window opening, followed by an analytical computer model of the same structure. Once calibrated with the test, the model can then cover other wall configurations.

- A final series of elemental tie tests to isolate the differences in behaviour between concrete and clay bricks, and to further calibrate the performance indicators.

- To cover the initial work proposed, which was deferred because of the problems encountered in the current programme, as well as studying outstanding issues, future work needs to include:

  - Dynamic testing using flexible supports to simulate the behaviour of a real structure, thus completing the original face-load work programme.

  - A pseudo-static face-load test on the same structural configuration, to allow closer study of system performance under realistic levels of random cyclic displacement.

  - Analytical computer modelling of both of these tests as well as the WCL test. As for the in-plane work, an accurately calibrated model will allow ecoverage of a realistically wide range of wall configurations. This will lead to development of performance criteria to match the tie testing criteria envisaged for code revision.

  - Static tests of prospective solutions to the corner deflection incompatibility problem. This should provide alternatives, which the brick industry clearly needs, to the only currently available solution, that of a physical separation joint.

10.0 CONCLUSIONS

The conclusions resulting from this work are:

1. Veneer ties are the key links between masonry veneer and the backing structure, particularly under seismic action. Although required to transfer face loads in the relatively rare event of an earthquake, they are also required to accommodate differential in-plane movement occurring during the life of the structure.

2. The wording of the test method for veneer ties contained in Appendix 3C of NZS 4210:1989 is abstruse and needs clarifying to determine whether or not ties meet the code criteria.

3. The prescribed test methods and assessment criteria of NZS 4210:1989 Appendix 3C are greatly more complicated than equivalent methods used in other countries, and also more complicated than the previous New Zealand test method. Nevertheless, in making this step, NZS 4210 is a major advance over any other known veneer tie standard. The danger is that its complexity is likely to hinder its acceptance in areas which would benefit from its principles. Steps should be taken to simplify the test method and clarify and remove any ambiguities, while still maintaining the general thrust.

4. Veneer ties are almost certain to be inadequately anchored because of disturbance to the bedding mortar during installation. This is because of the ways ties are installed in New Zealand (laying the ties dry on the lower brick and fixing to studs by means of face nailing).

5. Masonry veneers constructed using ties which are inadequately anchored are likely to have a greatly inferior seismic performance compared with those using adequately anchored ties.

6. However, current test practices do not allow inadequate anchorage to be identified. Thus, the test method should be revised to ensure that such inadequacies are clearly identified. The alternative "simple" solution of merely prescribing the method of fixing would discourage innovation and would be unacceptable under the philosophy of the New Zealand Building Code.

7. Ties in common use throughout New Zealand which are adequately anchored, whether deemed stiff or flexible, meet the broad criteria of Conclusion 1.

8. However, no known ties meet the criteria for stiff ties as set out in NZS 4210, even when adequately anchored. The requirements of both the light timber framing standard (NZS 3604) and the non-specific design masonry standard (NZS 4229) is that masonry veneers are tied to the backing structure by stiff ties complying with NZS 4210. This means that masonry veneer clad houses currently being built using these ties may not be in compliance with the New Zealand Building Code, as these standards are the only currently available acceptable solutions for non-specifically designed structures within that document.

9. The modification of structural response and the fall-off in tie performance caused by the effect on stiff ties of in-plane differential movement requires further investigation.

10. Screw fixing of ties resolves the problem of attaining fixity to flexible timber framing, thereby achieving adequate anchorage in the mortar bedding.

Thus, manufacturers should be encouraged to carry out product development aimed at improving tie performance, so that full Code compliance is attained. The most pressing problem is to ensure that ties are fully anchored in the mortar. Results of this project indicate that screw fixing is a promising avenue for development. Although requiring a change from current trade practice, most bricklayers seem to accept that it is inevitable. The common availability and ease of use of cordless screw-drivers should make the change more palatable to the trade. However, the evidence needs to be presented in a convincing manner. The manufacturers and the trade associations will all be required to help in this education process.

APPENDIX 1. INTERPRETATION OF NZS 4210 PART 3

STIFF TIES, TEST PROCEDURE

Note:

In-plane means in the plane of the veneer (horizontal test)
Axial means along the axis of the tie (vertical test)

Required:

6 specimens under in-plane monotonic (**Test [A]**) (3 left, 3 right)
12 specimens under in-plane cyclic (**Test [B]**) followed by axial monotonic (**Test [C]**) (6 tension, 6 compression)
6 specimens under in-plane cyclic (**Test [B]**) followed by axial cyclic **Test [D]** (3 tension 3 compression)

[A] In-plane stiffness and monotonic strength: [cl. 3.C8.2]

1st tie:
No displacement [cl. 3. C8.2(a)]
In-plane shear left.
Increase to maximum force. Record force/displacement.
Repeat for 2 other ties.

Repeat for in-plane shear right (remaining 3 ties).

Assess performance against clause 3.5.2. Discard ties.

[B] In-plane horizontal ductility and cyclic strength: [cl. 3.C11.1(b)&(f)]

1st tie:
4 cycles +/- 10mm (shear left then right)
Record force/displacement.
Repeat for 2 other ties.

4th tie:
4 cycles +/- 10mm (shear right then left)
Record force/displacement.
Repeat for 2 other ties.

Assess performance against clause 3.5.2. **Use these ties for tests [C] & [D].**

[C] Axial stiffness and monotinic strength: [cl. 3.C8.1]
[Note: these are the same ties as used for in-plane horizontal ductility and cyclic strength as [B] above]
1st & 4th ties:
Displace 10x10 [as cl. 3.C7.4]
Axial tension.
Increase to maximum force. Record force/displacement.

2nd & 5th ties:
No displacement [cl. 3.C8.1 (e)]
Axial tension.
Increase to maximum force. Record force/displacement.

3rd & 6th ties:
No displacement [cl. 3.C8.1(e)]
Axial tension.
Increase to maximum force. Record force/displacement.

Repeat for compression (remaining 6 ties).

Assess performance against clause 3.5.1 and table 3.5.1. Discard ties.

[D] Axial ductility and cyclic strength: [cl. 3.C11.1(a) & (d)]
[Note: these are the same ties as used for in-plane horizontal ductility and cyclic strength as [B] above]

1st tie:
Displace 10 (vert) x 10 (horiz)
1 cycle +/- 1mm (start with tension).
4 cycles +/- 9mm.
Record force/displacement.
Repeat for 2 other ties.

4th tie:
Displace 10 x 10
1 cycle +/- 1mm (start with compression).
4 cycles +/- 9mm
Record force/displacement.
Repeat for 2 other ties

Assess performance against clause 3.5.1 and table 3.5.1. Discard ties.

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APPENDIX 3: PRODUCTS USED IN THIS PROJECT

The following products were used in this project:

Tie type 1 Wiremakers: "Superior wall tie"

Tie type 2 Royal NZ Foundation for the Blind: "Duo tie"

Tie type 3 Lumberlok: "Brick tie"

Tie type 4 Abey: "Sherrif Face Fixed Tie"

Monier Brickmakers: 90 and 70 series clay bricks.

Firth Concrete Products: Concrete bricks.

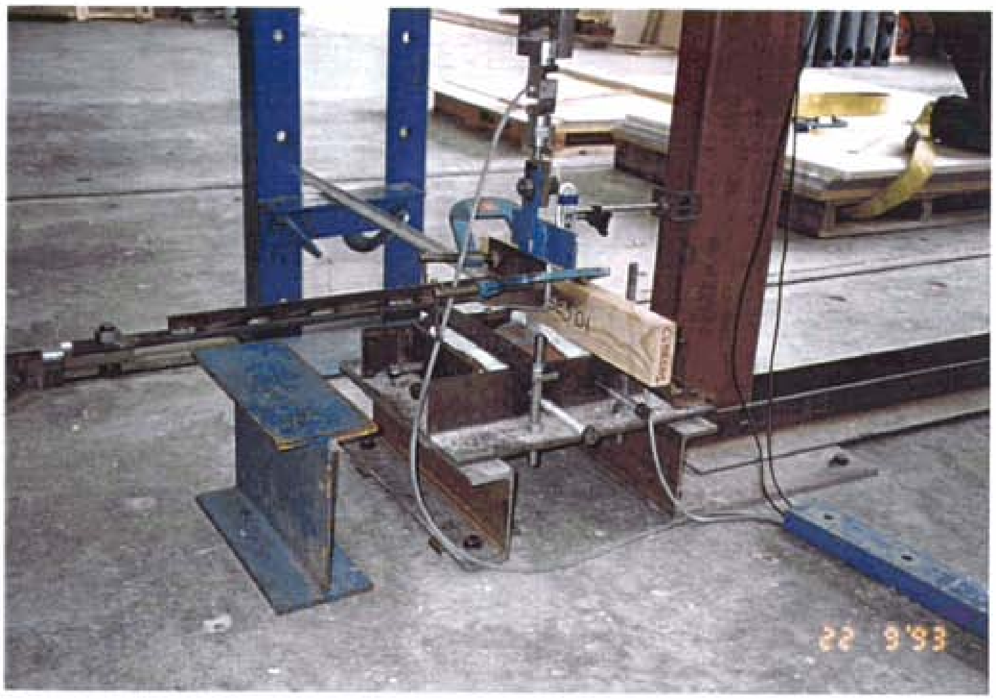
These products were either supplied by their manufacturers or their agents or purchased on the open market.

NOTE

The results obtained in this study relate only to the samples tested, and not to any other item of the same or similar description. BRANZ does not necessarily test all brands or types available within the class of items tested and exclusion of any brand or type is not to be taken as any reflection on it.

Further, the listing of trade or brand names above does not represent endorsement of any named product nor imply that it is better or worse than any other available product of its type. A laboratory test may not be exactly representative of the performance of the item in general use.

This work was carried out for specific research purposes, and may not have assessed all aspects of the products named which would be relevant in any specific use. For this reason, BRANZ disclaims all liability for any loss or other deficit, following use of the named products, which is claimed to be caused by reliance on the results published here.



Photograph 1: Set-up for tie tests

Photograph 2: Construction of series 1 specimens

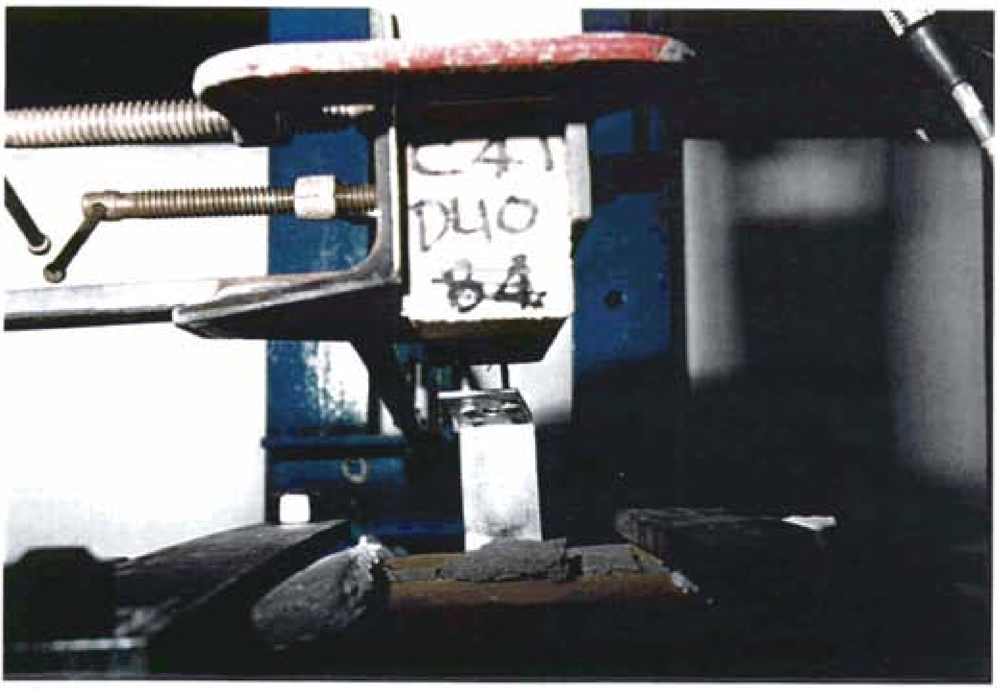
Photograph 3: Series 2 specimens

Photograph 4: Type 1 tie after in-plane cycling

Photograph 5: Type 2 tie after in-plane cycling

Photograph 6: Type 3 tie after in-plane cycling

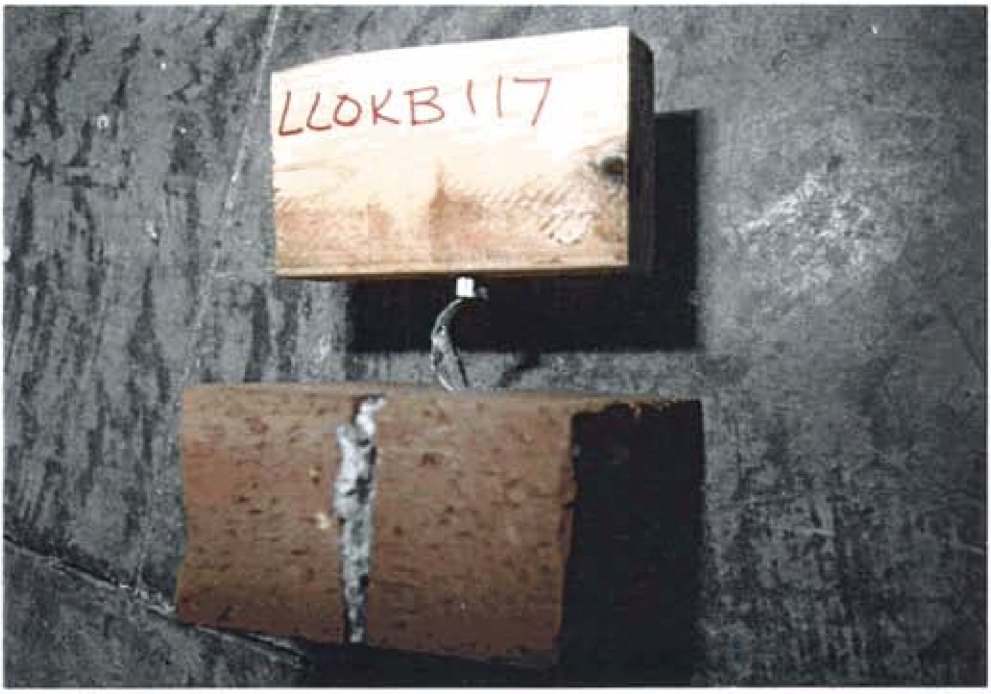
Photograph 7: Type 1 tie after axial cycling



Photograph 8: Type 2 tie after axial cycling, showing nail withdrawal due to 'levering' action during compression

Photograph 9: Type 3 tie after axial cycling showing nail withdrawal

Photograph 10: Type 3 tie after axial cycling showing fracture of teeth

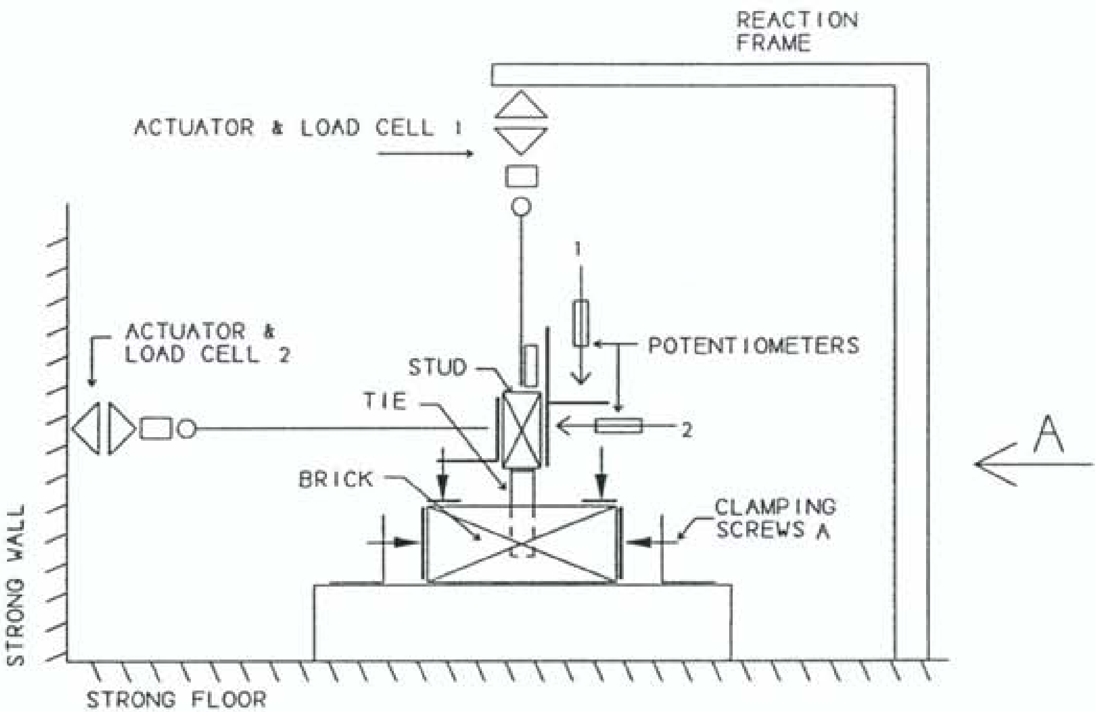


Photograph 11: Type 3 tie after axial cycling

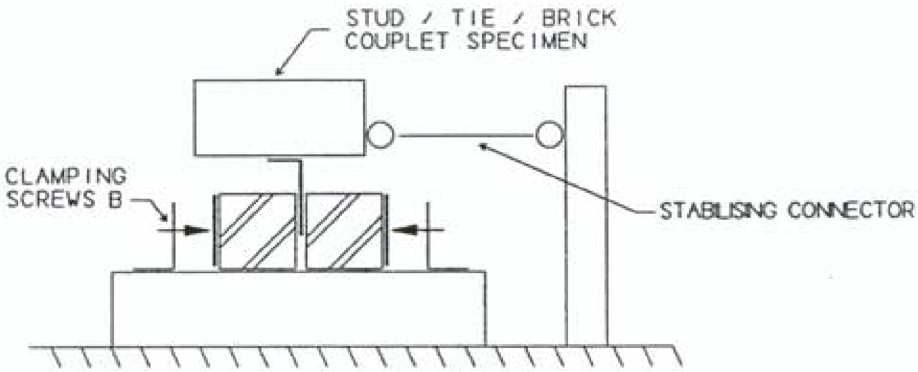
Photograph 12: Construction of series 4 specimens

Photograph 13: Monitoring stud movement under tie installation

Photograph 14: Installing ties using screwdriver



General View



View A
(Only part shown for clarity)

Figure 1 : Tie test set-up

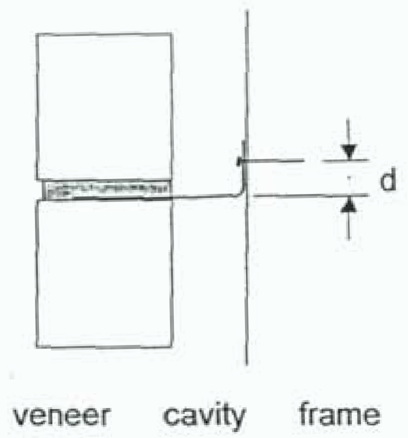


Figure 2a: Typical veneer tie as installed

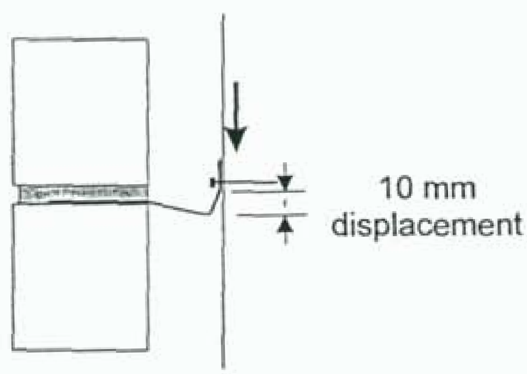


Figure 2b: Tie displaced for test

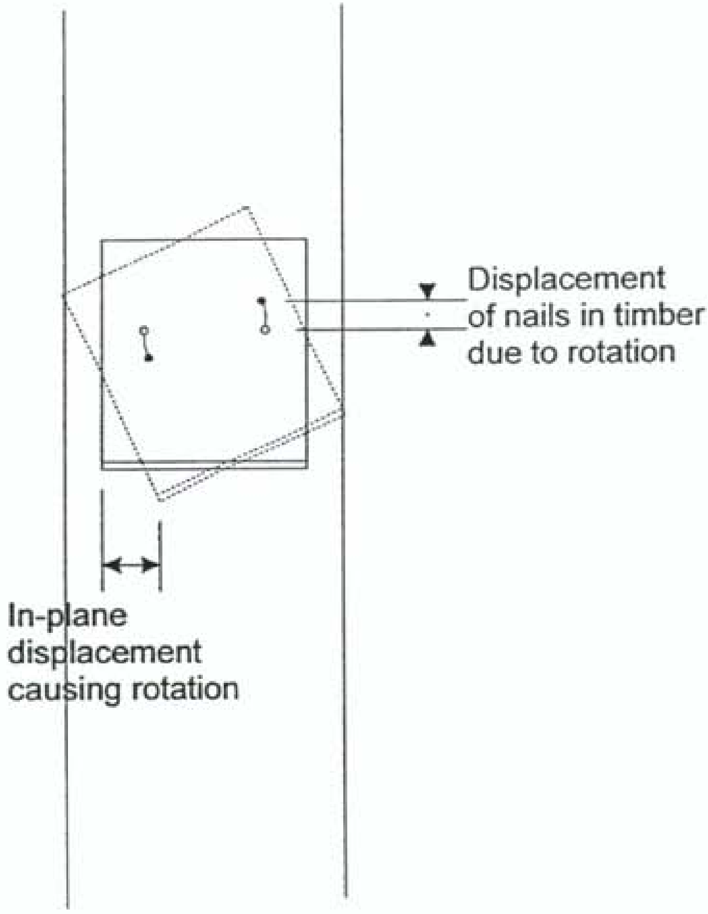


Figure 3: Displacement of tie and nails during in-plane test

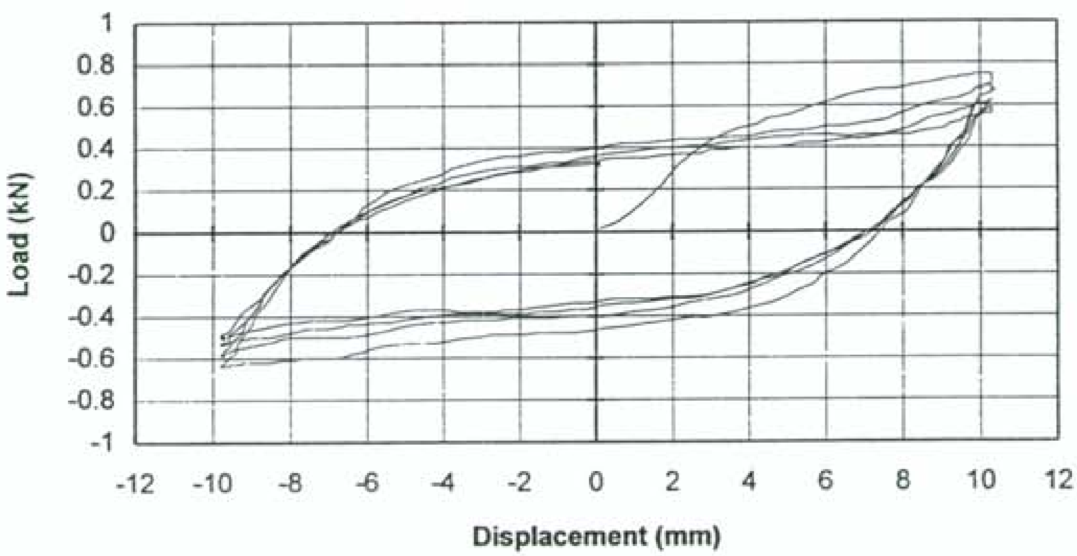


Figure 4: Tie type 3. In-plane cycling

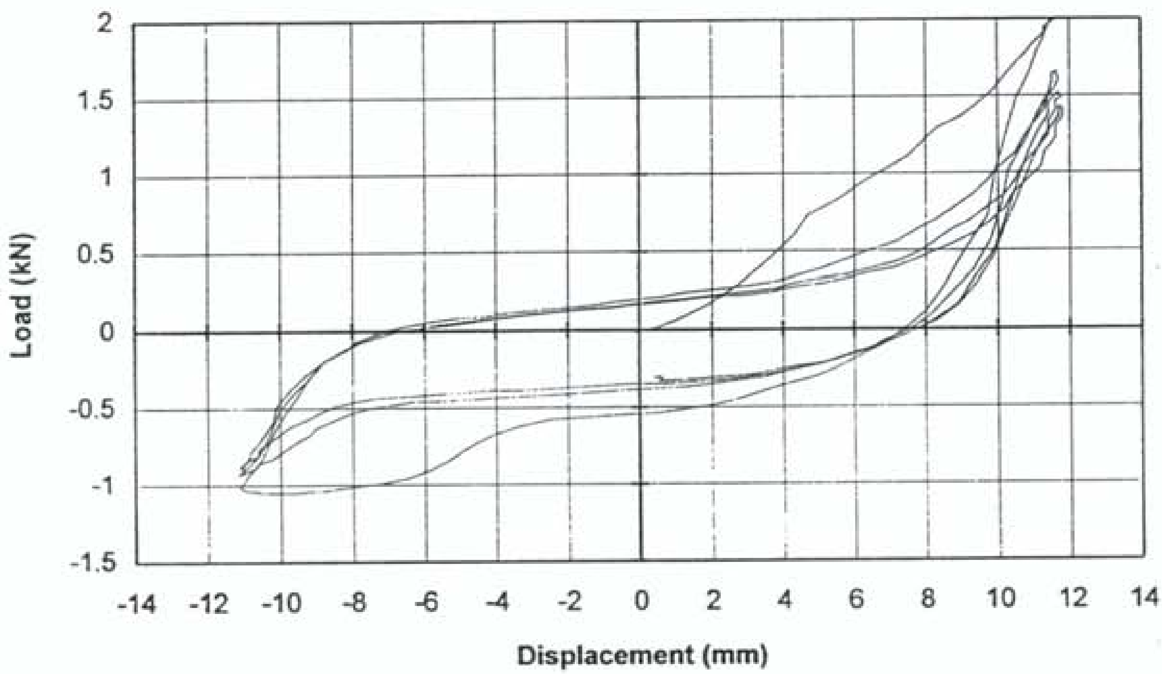


Figure 5: Tie type 1. Axial cycling beginning with tension

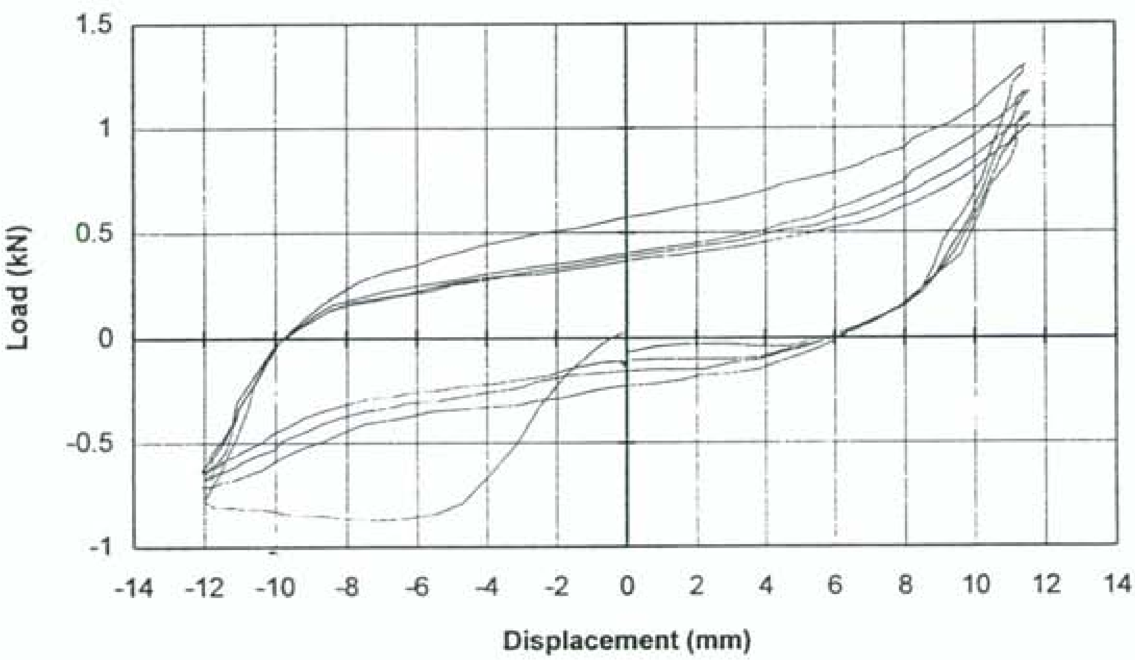


Figure 6: Tie type 1. Axial cycling beginning with compression

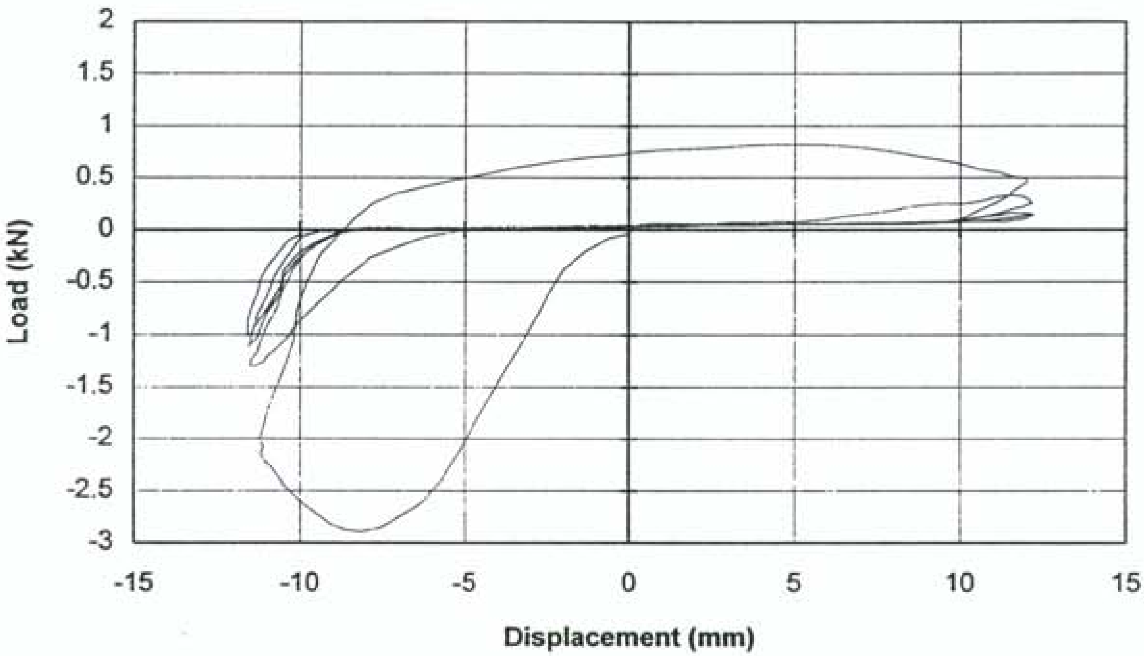


Figure 7: Tie type 2. Axial cycling compression

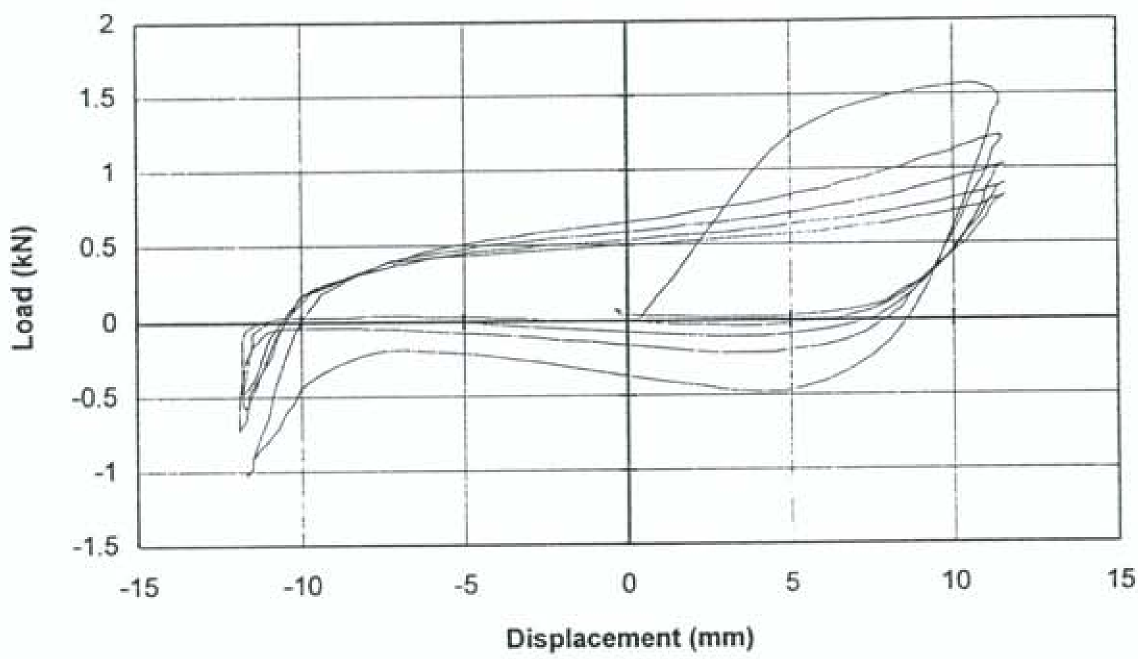


Figure 8: Tie type 3. Axial cycling

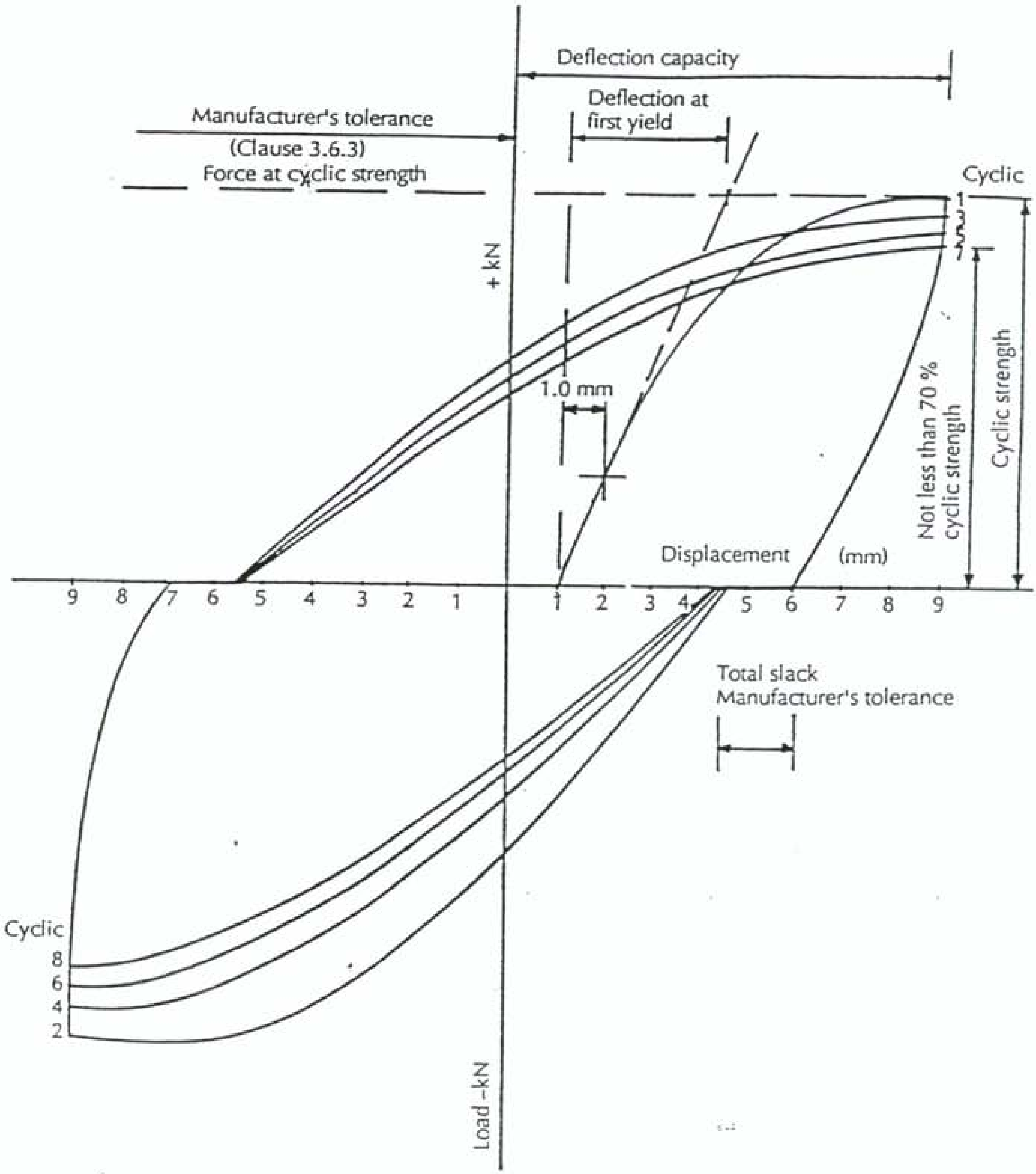


Figure 9: (Reproduced from NZS 4210). Hysteresis loops for tie tests

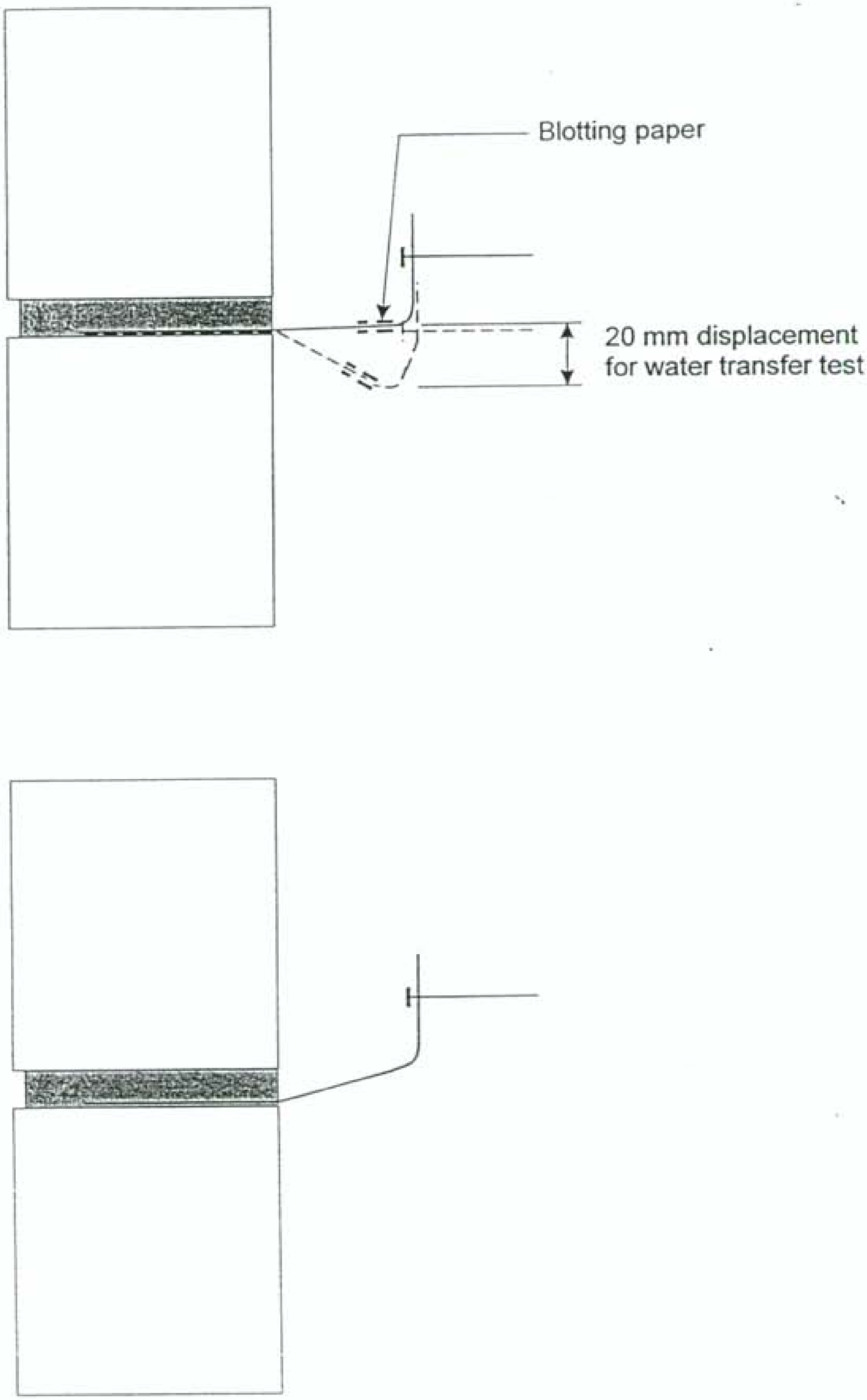


Figure 10: Water transfer test of NZS 4210

(a + b) Appendix 3.B

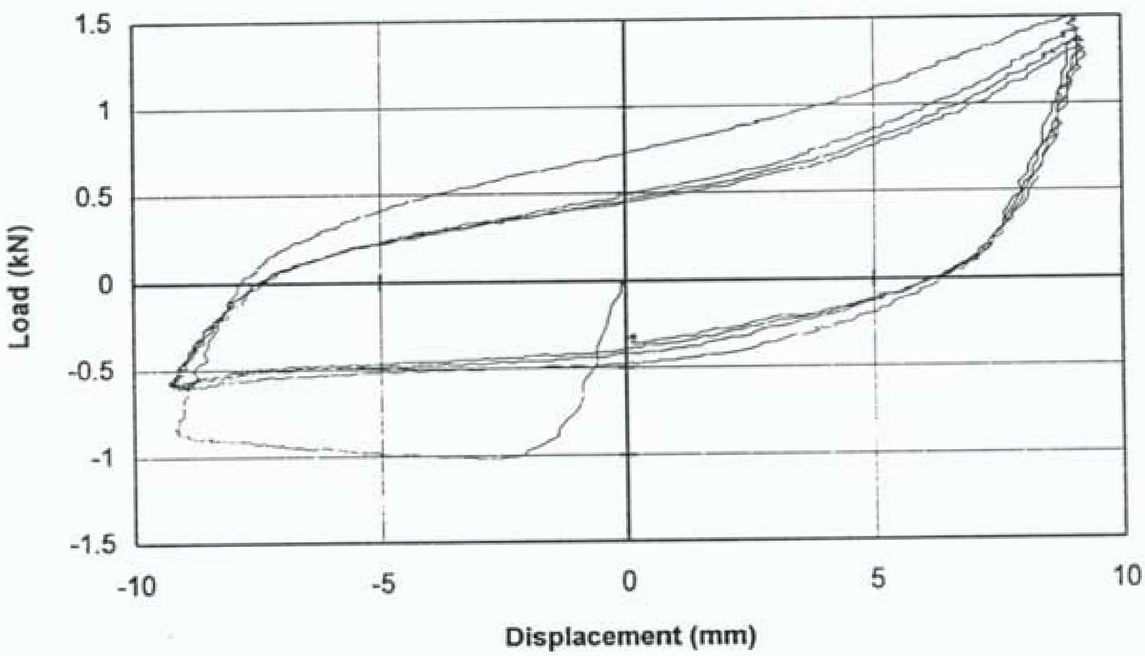


Figure 11: Tie type 1. Axial cycling in 70 series brick

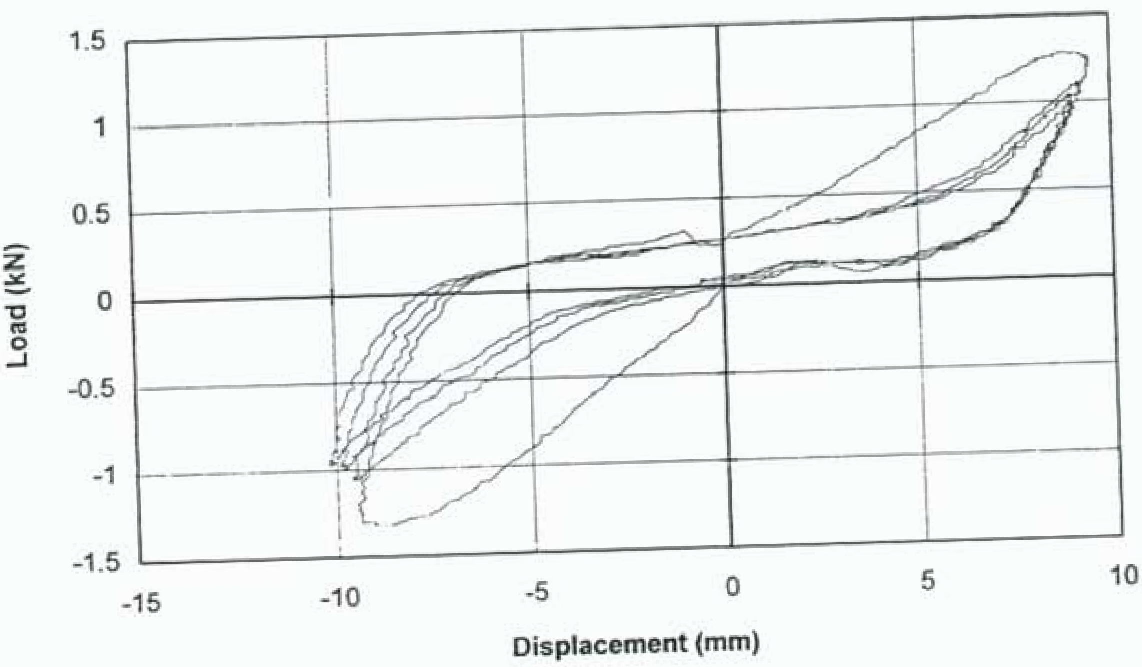
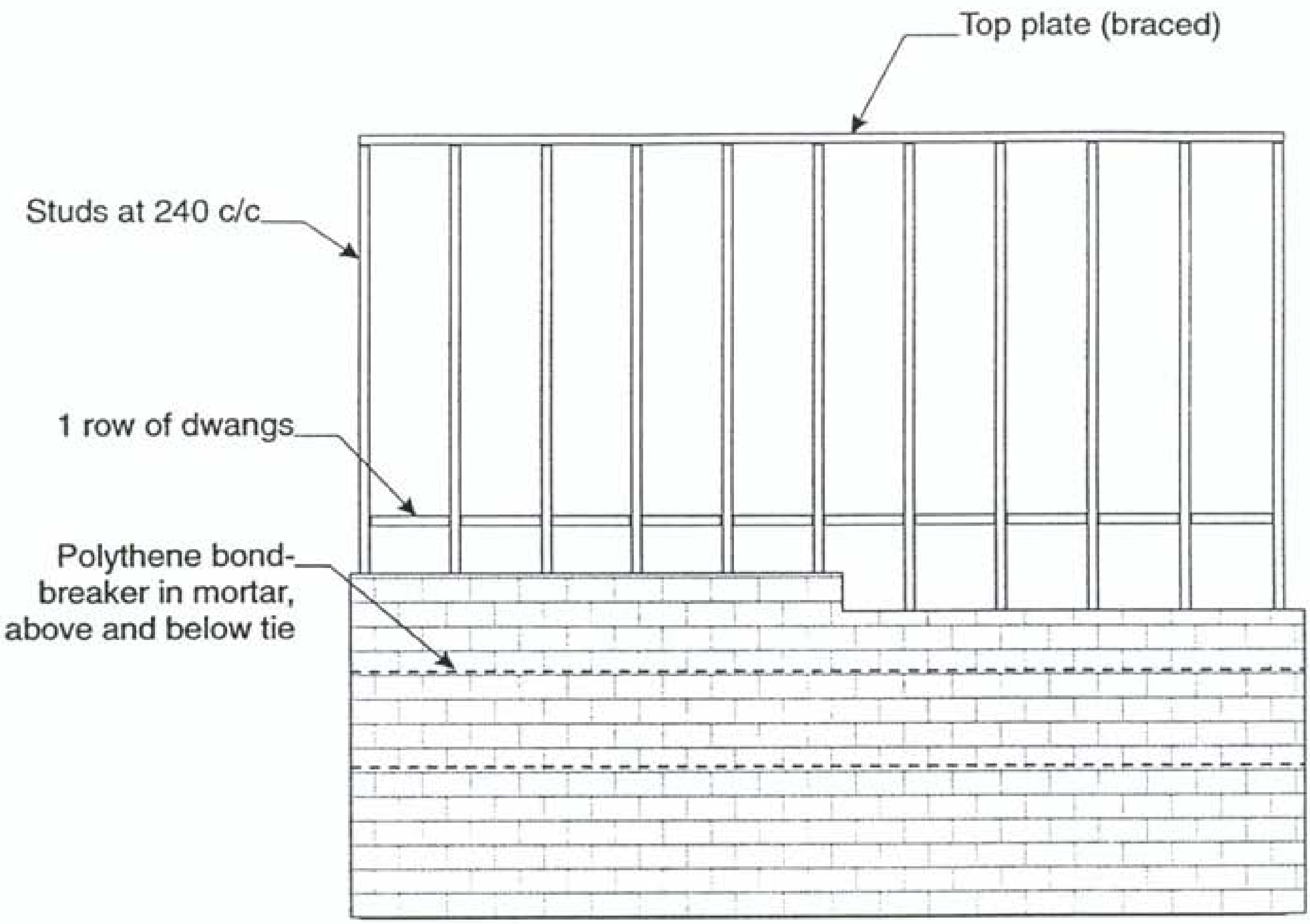


Figure 12: Tie type 1. Axial cycling after 20mm displacement for water transfer test



Tie type	3	3	1	1	1	2	2	2	2
Bedding:	B	N	B	N	B	N	B	B	N
Fixing:	-	-	N	N	A	N	N	A	S

Note: B = Tie fully bedded in mortar
N = Tie laid dry on lower brick

Fixings: N = 2 - 40x2.5 galvanised flat head nails
A = 2 - 45x2.3 annular grooved nails
S = 1 - 12g x40 self drilling screw (Type 17)

Figure 13: Specimen construction. Series 4 tests

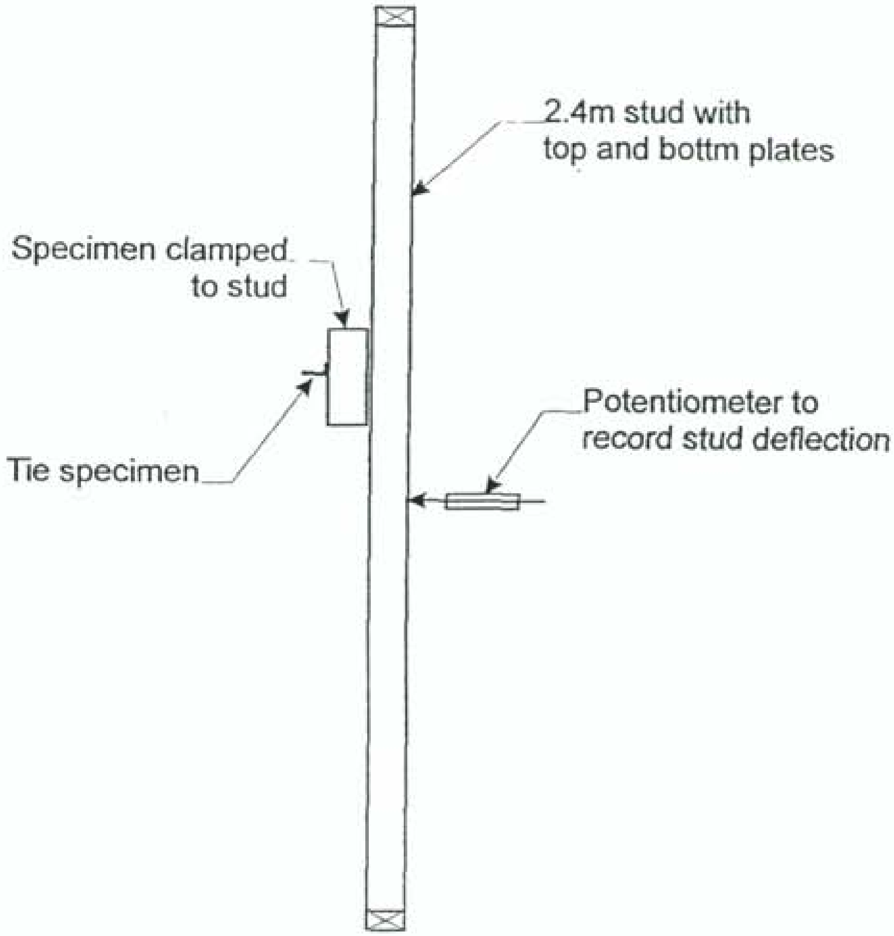


Figure 14: Series 5 tie tests. Set-up for installation of tie fixing

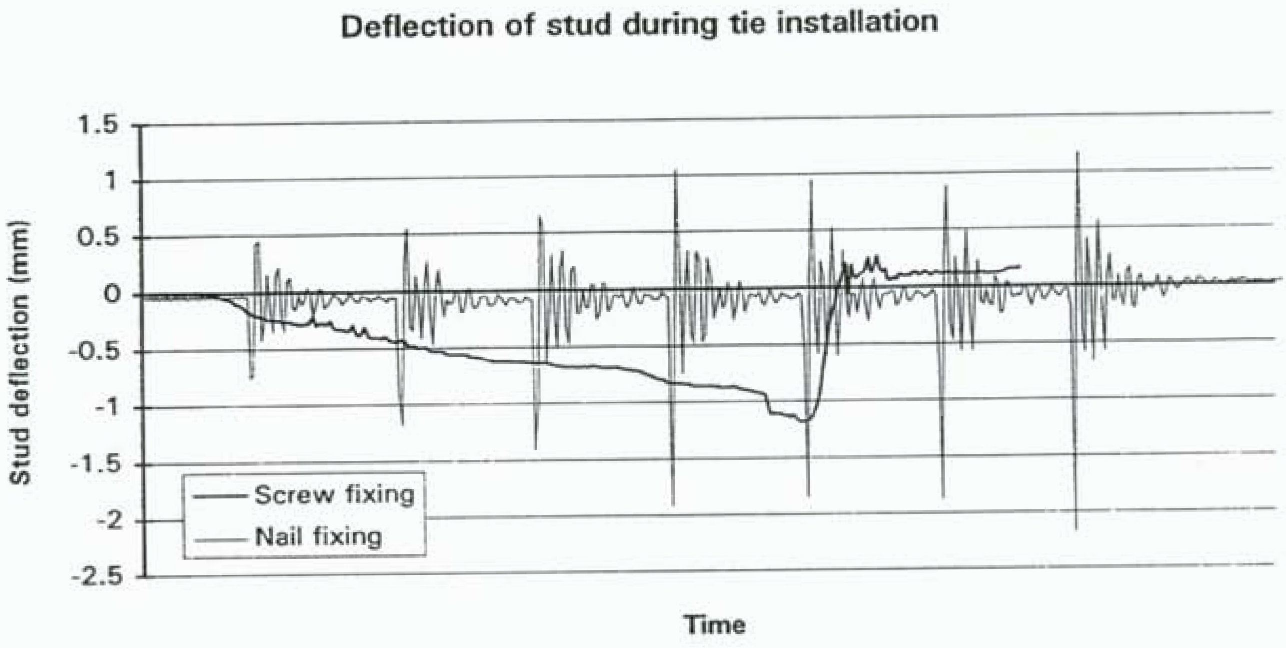


Figure 15: Small-scale tie / tests series 5: Deflection of stud during tie installation

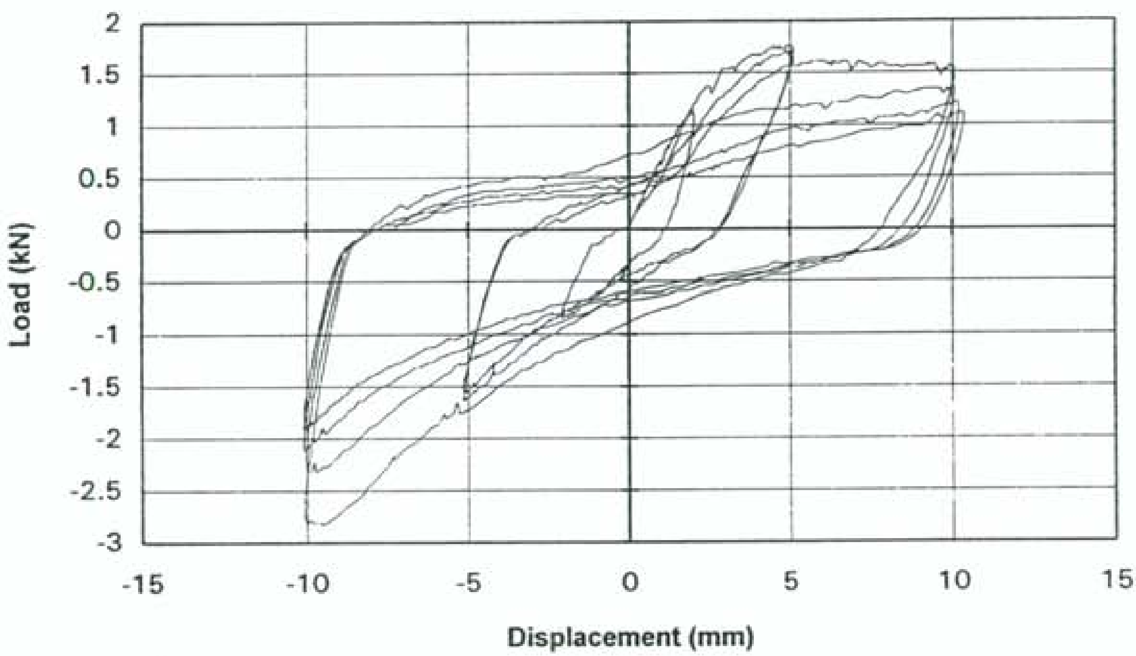


Figure 16: Series 5 tests. Tie type 1 axial load / displacement graph

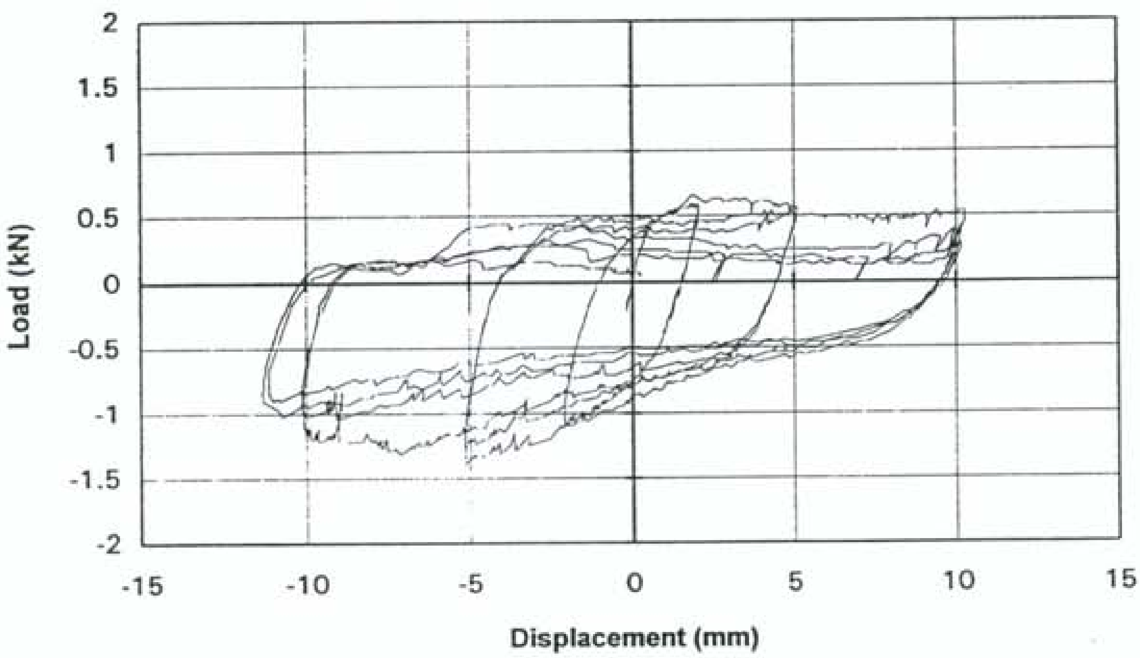


Figure 17: Series 5 tests. Tie type 3 axial load / displacement graph

Specimen	In-plane: Stiffness (N/mm)	In-plane: Cyclic strength (N)	Axial tension: Stiffness (N/mm)	Axial tension: Strength (N)	Axial compression: Stiffness (N/mm)	Axial compression: Strength (N)	Axial cycling: Cyclic strength (N)
Type 1 ties							
1A1	252						
1A2	181						
1A3	200						
1A4	261						
1A5	272						
1A6	189						
1B1		1250 1030	367	3600			
1B2		1300 970	226	3450			
1B3		1170 740	189	2800			
1B4		1050 900	153	2330			
1B5		920 960	181	1670			
1B6		1080 1110	189	1730			
1B7		1200 860			238	960	
1B8		1110 760			250	1410?	
1B9		1150 1000			238	900	
1B10		850 1060			240	1190	
1B11		910 1030			240	1200	
1B13		1180 820					1440 730
1B14		970 670					1400 950
1B15		1180 710					1470 690
1B16		1040 880					2000 1050
1B17		1000 890					1170 1520
1B18		880 840					870 1300
Type 1 x	226	872	218	2600	241	1130	867
s	40	112	77	832	5	205	203
X	160	687	91	1227	233	792	531
NZS 4210 criteria	500	900	240	1400	240	1400	700
Type 2 ties							
2A1	186						
2A2	226						
2A3	219						
2A4	277						
2A5	252						
2A6	340						
2B1		900 850	234	930			
2B2		920 1040	184	830			
2B3		1210 1110	194	1310			
2B4		1170 940	151	880			
2B5		1250 930	348	1540			
2B6		1490 1030	243	2410			
2B7		1110 920			453	2040	
2B8		1310 1000			503	2030	
2B9		1230 1050			367	1980	
2B13		1160 990					1650 1070
2B14		1400 1040					1190 1230
2B15		1270 1220					1820 1360
2B16		1180 960					518 2140
2B17		1530 1180					1360 2500
2B18		1220 910					670 1360
Type 2 x	250	996	253	1320	441	2020	998
s	54	103	60	604	69	32	350
X	161	826	154	323	327	1910	421
NZS 4210 criteria	500	900	240	1400	240	1400	700

Note: x = sample mean value
s = sample standard deviation
X = characteristic value

Table 1: Series 2 elemental tie tests - results

Specimen	In-plane:			Axial tension:		Axial compression:		Axial cycling:	
	Stiffness (N/mm)	Cyclic strength (N)		Stiffness (N/mm)	Strength (N)	Stiffness (N/mm)	Strength (N)	Cyclic strength (N)	
Type 3 ties									
3A1	203								
3A2	223								
3A3	143								
3A4	172								
3A5	176								
3A6	148								
3B1		810	940	186	1460				
3B2		750	720	331	1550				
3B3		670	710	302	1670				
3B4		830	630	266	1380				
3B5		830	760	-	-				
3B6		750	640	240	1210				
3B7		690	750			340	1180		
3B8		870	950			146	1890		
3B9		690	770			240	1260		
3B10		850	750			125	580		
3B13		920	820					1300	407
3B14		810	530					1000	1430
3B15		770	630					1570	1040
3B16		920	660					420	1300
3B17		800	720					-	-
3B18		960	660					370	480
Type 3 x	177	704		265	1450	213	1230	645	
s	31	85		56	174	98	536	343	
X	126	563		172	1163	51	346	80	
NZS 4210 criteria	500	900		240	1400	240	1400	700	

Note: x = sample mean value
s = sample standard deviation
X = characteristic value

Test to NZS 4210

Tie type:		Cyclic strength:		Cyclic strength : (after 20 mm displacement)	
		In-plane (N)	Axial (N)	In-plane (N)	Axial (N)
1	x	918	940	980	1210
	s	63	70		
	X	814	824		
2	x	966	-	1070	500
	s	50	-		
	X	883	-		
3	x	728	470	590	870
	s	97	76		
	X	567	348		
4	x	170	650	120	460
	s				
	X				

Tension only test (to AS 2699)

Tie type:		Force displaced: (N)		Force undisplaced: (N)	
		at 1.5 mm	at 10 mm	at 1.5 mm	at 10 mm
4	x	195	760	545	890

Note: x = sample mean value
s = sample standard deviation
X = characteristic value

Table 2: Series 3 elemental tie tests - results

Tie type:	Bedding	Fixing	Tie row:				Mean result:
			1 (bottom)	2	3	4 (mid height)	
1	Bed	N	738 15.5 mortar	358 8.2 mortar	Damaged before test	1037 21.8 mortar	711
1	Non	N	841 34.5	damaged	damaged	557 7.7 nails	699
1	Bed	A	damaged	805 26.6 mortar	352 21.1 mortar	1666 42.8 mortar	941
2	Bed	N	826 13 mortar	336 6.1 mortar	damaged	1699 30.1 mortar	954
2	Non	N	705 10.1 mortar	621 8.6 mortar	371 7.6 mortar	725 16.3 mortar	606
2	Bed	A	1036 24 mortar	2010 21.7 mortar	459 7.4 mortar	1455 24.3 mortar	1240
2	Non	S	1066 17.5 mortar	1648 13.1 mortar	1648 14.4 mortar	1709 26 brick cracked	1520
3	Bed	-	damaged	688 12.7 nail	635 7.3 nail	498 3.9 nail	607
3	Non	-	621 11 nail	257 3.5 mortar	535 7 nail	439 4.5 nail	463

Notes: Top line is peak load (N)
Middle line is displacement at peak load (mm)
Bottom line is failure mode

Bedding: Bed = Tie fully bedded in mortar
Non = Tie laid dry on lower brick

Fixing: N = Plain nails
A = Annular grooved nails
S = Screw fixed

Table 3: Series 4 elemental tie tests - results summary

Fixing type	Stud deflection (mm)	Pullout load (kN)	Code load (kN)	Comments
45 x 3 galvanised flathead nail	3.36	2.28	0.58	NZS 3603 (For bright nails)
25 x 10g self tapping screw	0.54	2.73	-	
40 x 10g self tapping screw	0.61	4.30	-	
60 x 9g Hex head "S" point screw	1.72	2.88	-	
19 x 8g Pan head Chipboard screw	0.69	2.29	-	
25 x 8g Pan head Chipboard screw	1.63	2.54	-	
35 x 8g Pan head Chipboard screw	1.33	3.47	-	
25 x 10g Pan head Chipboard screw	1.24	2.93	-	
25 x 10g Wafer head self drilling screw	1.31	2.49	-	
45 x 10g Wafer head self drilling screw	1.08	4.00	-	
No 12 x 25 Hex head self drilling screw	1.51	2.94	2.33	AS 3566
No 12 x40 Hex head self drilling screw	1.39	4.55	3.73	AS 3566

Note: 1. Refer to Figure 15 for graphs of stud deflection for these fixings.
2. Stud deflection is the sum of the peak positive and negative amplitudes.

Table 4: Series 5 elemental tie tests. Fixing pullout tests - results summary

Specimen construction	Tie	In-plane cycling to: (+/-mm)	Peak axial load (kN) resisted at:					
			2 mm	2 mm	5 mm	5 mm	10 mm	10 mm
Tie laid directly	Type 1	20	-1.10	1.10	-1.80	1.40	-2.90	1.10
on lower brick					-1.80	1.30	-2.20	0.80
(partly bedded)								
Partly bedded	Type 1	20	-1.20	0.90	-2.20	1.10	-2.40	0.80
					-2.00	1.00	-1.80	0.70
Partly bedded	Type 1	20	0.46	-0.49	1.71	-1.20	1.42	-
					1.64	-0.98	1.12	-
Partly bedded	Type 1	20	0.94	-1.10	1.39	-1.81	1.28	-1.83
					1.33	-1.78	1.03	-1.22
Partly bedded	Type 1	20	1.20	-0.80	1.75	-1.60	1.55	-2.80
					1.70	-1.55	1.10	-1.90
Tie laid in	Type 1	20	-0.95	0.90	-1.55	1.10	-2.40	1.00
mortar bedding					-1.55	1.00	-1.75	0.75
(fully bedded)								
Fully bedded	Type 1	20	-1.10	1.10	-1.25	1.30	-1.55	1.70
					-1.25	1.30	-1.25	1.25
Fully bedded	Type 1	20	-1.15	1.15	-2.00	1.25	-3.50	1.20
					-1.90	1.20	-2.50	0.85
Fully bedded	Type 1	20	1.70	-1.80	2.50	-2.90	3.10	-3.40
					2.50	-2.30	1.90	1.95
Partly bedded	Type 3	10	0.53	-0.59	0.78	-0.42	0.27	-0.21
					0.38	-0.26	0.17	-0.21
Fully bedded	Type 3	20	-0.30	0.08	-0.24	0.12	-0.22	0.53
					-0.20	0.12	-0.22	0.05
Fully bedded	Type 3	10	-0.93	0.68	-0.50	0.64	-0.27	0.25
					-0.32	0.37	-0.22	0.05
Fully bedded	Type 3	10	0.61	-0.58	0.57	-0.47	0.44	-0.25
					0.44	-0.45	0.17	-0.14

(Upper line is load at first cycle, lower line at last cycle)

Table 5: Small scale tie tests series 5 - results summary

Specimen construction	Tie	In-plane cycling to: (+/-mm)	Peak axial load (kN) resisted at:					
			2 mm	2 mm	5 mm	5 mm	10 mm	10 mm
Tie cast in	Type 1	20	0.65	-0.80	0.95	-1.25	0.90	-3.10
dental plaster					0.90	-1.20	0.85	-2.55
Dental	Type 4	20	0.84	-0.70	0.66	-0.83	0.31	-1.00
plaster					0.53	-0.83	0.15	-0.83
Dental	Type 4	20	-0.65	0.51	-0.58	0.55	-0.88	0.37
plaster					-0.58	0.50	-0.71	0.28
Dental	Type 3	10	0.81	-1.07	0.95	-1.61	0.56	-2.83
plaster	(screwed)				0.78	-1.51	0.51	-2.00
Dental	Type 3	10	-1.00	0.97	-1.43	0.89	-2.57	0.56
plaster	(screwed)				-1.31	0.83	-1.71	0.43
Dental	Type 3	20	0.49	-0.46	0.54	-0.88	0.59	-1.42
plaster	(screwed)				0.61	-0.85	0.46	-1.03
Dental	Type 3	20	-0.27	0.32	-0.54	0.46	-0.93	0.71
plaster	(screwed)				-0.59	0.42	-0.73	0.56
Dental	Type 4	0	0.95	-0.81	1.32	-0.93	1.12	-2.10
plaster					1.25	-1.00	1.00	-1.34
Dental	Type 4	0	-0.73	1.03	-0.85	1.42	-1.00	1.03
plaster					-0.81	1.27	-0.98	0.61
Dental	Type 4	10	1.15	-0.68	1.54	-0.81	0.81	-1.00
plaster					1.37	-0.78	0.22	-0.88
Dental	Type 4	10	-0.73	1.00	-0.83	1.37	-0.90	0.93
plaster					-0.78	1.22	-0.88	0.49
Dental	Type 4	20	0.98	-0.76	1.34	-0.83	0.88	-1.03
plaster					1.15	-0.73	0.66	-0.88
Dental	Type 4	20	-0.78	1.05	-0.90	1.32	-0.95	0.68
plaster					-0.88	1.17	-0.93	0.32

(Upper line is load at first cycle, lower line at last cycle)

Specimen construction	Tie	Performance indicator (Where tie has been subject to in-plane displacement cycling of:) +/- 0 mm	+/- 10 mm	+/- 20 mm
Tie placed on mortar bedding (fully bedded)	Type 3		5.02	3.06
			2.75	2.47
			0.05	0.05
Fully bedded	Type 1			18.90
				16.30
				0.75
Fully bedded	Type 1			18.70
				16.10
				1.25
Fully bedded	Type 1			22.00
				15.90
				0.85
Fully bedded	Type 1			33.30
				22.90
				1.95
Tie laid directly on lower brick (partly bedded)	Type 3		4.68	
			2.32	
			0.17	
Partly bedded	Type 1			20.00
				16.20
				0.80
Partly bedded	Type 1			19.80
				18.00
				0.70
Partly bedded (This tie was loose in mortar at time of test)	Type 1			12.70
				10.80
				0.25
Partly bedded	Type 1			19.10
				16.60
				1.03
Partly bedded	Type 1			22.90
				16.60
				1.10

Top line of each set is the accumulated sum of axial loads resisted (Tension and compression).
2nd line is accumulated sum as above, but stopped when load falls below 80% of peak load.
3rd line is residual load resisted at end of test.
(All in kN)

Table 6: Performance indicator applied to results of series 5 tests

Specimen construction	Tie	Performance indicator (Where tie has been subject to in-plane displacement cycling of:)		
		+/- 0 mm	+/- 10 mm	+/- 20 mm
Dental plaster	Type 3	12.60	4.96	
		4.51	2.23	
		0.20	0.14	
Dental plaster	Type 3	7.83	10.70	
		6.39	8.97	
		0.10	0.20	
Dental plaster	Type 3		6.29	
			5.38	
			0.00	
Dental plaster	Type 3 (Screwed)		18.50	10.90
			16.00	9.39
			0.51	0.46
Dental plaster	Type 3 (Screwed)		16.90	8.46
			12.30	7.17
			0.51	0.42
Dental plaster	Type 4	17.60	11.60	7.73
		15.20	8.14	7.45
		0.93	0.22	0.28
Dental plaster	Type 4	13.10	11.90	8.81
		8.14	7.74	8.28
		0.61	0.49	0.15
Dental plaster	Type 4			11.00
				9.28
				0.20
Dental plaster	Type 4			11.30
				7.41
				0.32

Top line of each set is the accumulated sum of axial loads resisted (Tension and compression).
2nd line is accumulated sum as above, but stopped when load falls below 80% of peak load.
3rd line is residual load resisted at end of test.
(All in kN)

Seismic performance of masonry veneers and ties.
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