STUDY REPORT

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Critical properties of mortar for good seismic performance of brick veneer

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Preface

The quality of brick veneer construction has improved markedly in recent years, with requirements for the ties to be screw-fixed to the timber framing and with the advent of lighter bricks with vertical penetrations.

This is the sixth BRANZ investigation of a series looking into the seismic performance of brick veneer. Previously:

- In Phase 1 slow cyclic tests were performed on two full-scale veneer specimens where the veneer clad a rectangular room which had both window and door openings.
- In Phases 2 and 3 a shake table test was performed on a clay brick veneer-clad room and concrete brick veneer-clad room respectively. These used an inertial mass to simulate roof loads.
- In Phase 4 a two-storey brick veneer building was cyclically racked to investigate its seismic performance.
- In Phase 5 the seismic performance of brick veneer walls was tested in the out-of-plane direction using a shake table.

The above studies all used the same pre-mixed mortar. In this latest study the effect of mortar type and quality is examined and it consisted of three parts:

- In Stage 1 the properties of mortar collected on-site were measured.
- Stage 2 consisted of elemental laboratory tests measuring mortar bond and compression strength properties with variables including sand type and admixtures used.
- In Stage 3 shake table tests were performed using a range of mortar properties.

The interaction of the factors which affect brick veneer performance that are relevant to this project are summarised in a block chart on the next page.

Acknowledgments

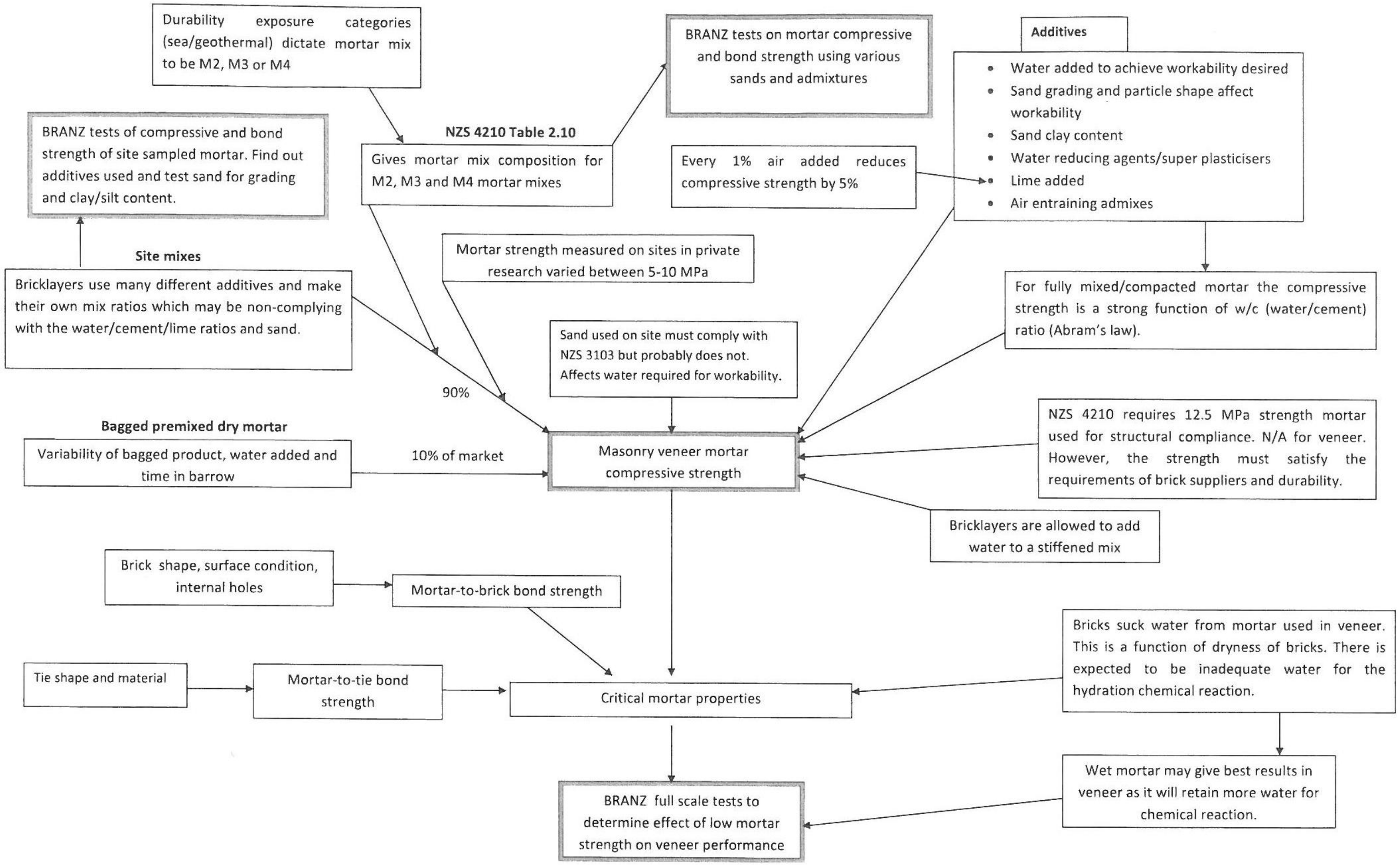
This work was funded by the Building Research Levy. Elephant Plasterboard New Zealand donated the wall linings used in the testing described herein. Monier Brickmakers Ltd donated the bricks and Eagle Wire Products Ltd donated the brick ties used in the testing. Various companies donated sand and admixtures used in the testing.

BRANZ acknowledges the work of John Oliver of Lifetime Promotions Promotions Ltd for the testing described in Section 3.3, and Duigald Myers of Materials Advisory and Testing Services Ltd for the testing described in Section 3.4.

The goodwill of the tradesmen who allowed mortar samples to be taken from their barrows in site investigations is acknowledged.

Note

This report is intended for structural engineers, standards development and brick masonry committees and associations, and fellow researchers studying the seismic performance of brick veneer.



Frontispiece. Flow chart of factors affecting mortar properties and veneer seismic performance considered in this project

Critical properties of mortar for good seismic performance of brick veneer

BRANZ Study Report SR 258

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Reference

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Abstract

Recent cyclic displacement tests and shake table tests on full-scale buildings by the author have shown that modern single and two-storey brick veneer New Zealand houses, constructed using good mortar, will perform very well under both in-plane and out-of-plane testing. The veneer can be relied on to carry a significant portion of the seismic design load.

In this project mortar compression strength and the mortar-to-brick bond strength were measured using both mortar taken from building sites and laboratory mixes (but made under site conditions to a desired workability by an experienced masonry tradesman). The strength of the bond between brick tie and mortar was also measured in the laboratory. The laboratory mixes were designed to cover the range of mortar being used on New Zealand sites.

The seismic performance of brick veneer walls constructed using a selected set of the laboratory mixes was determined by shake table testing. By examining which walls gave good performance, and which gave unsatisfactory performance, minimum mortar properties to ensure good seismic performance were determined. Good seismic performance was defined as the veneer being able to resist the design level earthquake without any masonry shedding.

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1. INTRODUCTION AND OUTLINE OF WORK

BRANZ has performed several in-plane and out-of-plane tests on full-scale brick veneer buildings in the last few years. This work and a summary of tests by others on the out-of-plane performance of brick veneer is described by Thurston and Beattie (2011). This report focuses on the influence of mortar on the seismic resistance of brick veneer.

Concerns have been expressed by the industry about the compressive strength of mortar that is being used for masonry construction in New Zealand buildings. NZS 4210 (SNZ 2001) *Masonry construction – materials and workmanship* requires that the seven day compressive strength of mortar "be not less than 12.5 MPa for structural compliance with NZS 3604". (However, as brick veneer is "non-structural" this requirement does not apply to mortar for brick veneer.) A member of the NZS 4210 Committee has advised that the value of 12.5 MPa has no technical basis as such and there is no clear understanding of what effect mortar strength has on the overall structural performance of masonry.

Pre-mixed mortar, that is designed to deliver this strength, is commercially available but is only used in 5-10% of brick veneer construction (by far the most common approach is to use site-mixed mortar). A recent private pilot-scale survey of 10 sites in Auckland found an average strength of site-mixed mortar of 7.08 MPa with a minimum of 5.2 MPa and a maximum of 10.0 MPa.

The research hypothesis for this project is that "a threshold measurable value for mortar compressive or bond strength can be established below which the seismic performance of brick veneer construction is compromised". The project consisted of a series of experiments on a range of mortar strengths and components to ascertain the minimum level that achieves satisfactory seismic performance. This is the basis for the recommendations given in this report.

The project consisted of three stages which are briefly described below. More details are given in the body of this report. The interaction of the factors which affect brick veneer seismic performance that are relevant to this project are summarised in a block chart in the frontispiece before the table of contents of this report.

Stage I. Investigation of mortar properties used on building sites. Building sites where brick veneer was being constructed were visited in various regions of New Zealand and mortar samples were taken from bricklayers' wheelbarrows. The mortar was used to make brick couplets and compression cylinders which were eventually tested. Tests were also performed on sand sampled at the site and a questionnaire filled in about admixtures used etc.

Stage II. Laboratory mortar investigation. A registered masonry tradesman mixed mortar in a rotary mixer (similar to that commonly used on-site) using a range of mortar mix designs. Variables considered were admixture type, mix time, cement quantity, sand type, brick wetness and presence of brick cores. The bond strengths were measured from brick couplets, direct tension cruciforms and brick tie pullout specimens. This information was used to select mortar used in Stage III of the project.

Stage III. Laboratory shake table tests. Four 2.4 m high brick veneer walls, each made with a different mortar mix design, were fixed to timber-framed wall framing and tested under simulated seismic loading. By comparing the bond strength of the mortar in the tested walls with the ability of the test walls to resist design level shaking, minimum bond strengths to give "satisfactory performance" were determined.

"Satisfactory performance" is defined as the ability to resist a design level earthquake for locations where the seismic "Z" factor in Table 3.3 of NZS 1170.5 (SNZ 2004) = 0.40 (e.g. Wellington) without any masonry shedding. Many may suggest that some masonry shedding is acceptable as the consequent life risk on a single-storey building is low and the veneer can be repaired for a moderate cost. However, for the purposes of this project, for economic and social reasons it is considered unacceptable for a large number of houses to shed brick veneer should a design earthquake strike a major metropolitan area.

2. LITERATURE SURVEY

This project is intended to determine what mortar properties are necessary to ensure brick veneer has good seismic performance. The literature survey examines the various properties of mortar to identify key parameters found by others.

BDRI (1977) lists the four functions of mortar as:

1. To provide a joint of sufficient thickness to accommodate bricks of slightly different sizes – 10 mm average mortar thickness is regarded as being adequate for this purpose;
2. To provide bond of sufficient strength to resist the lateral (wind and earthquake) loads to which the wall will be subjected;
3. To provide an even bedding for the bricks with sufficient strength to carry the compressive loads within the brickwork; and
4. To give a weather-tight and durable wall.

This project is focused on item (2). Regarding item (3), the compressive strength of a brick veneer is usually not critical. In relation to item (4), in New Zealand some water penetration is usually deemed to be acceptable as the water which does penetrate the veneer is adequately handled by the cavity.

2.1 Mortar components and properties

Mortars contain cement, aggregate, water and admixtures and each contributes to its performance. ASTM C270-08C (2008) summarises the attributes of the mortar components as follows:

- Portland cement contributes to mortar strength and durability.
- Lime, in its hydroxide state, provides workability, water-retentivity and elasticity. Both Portland cement and lime contribute to bond strength.
- Sand acts as a filler and enables the unset mortar to retain its shape and thickness under the weight of subsequent courses of masonry.
- Water is the mixing agent which gives fluidity and causes cement hydration to take place.

2.1.1 Mortar-to-brick tensile bond strength

Section C2.2.3.2 of NZS 4210 (SNZ 2001) states that the "bond between mortar and brick is the most important single factor affecting (transverse) strength" and for "unreinforced two storey veneers, mortar bond strength becomes a significant factor in the overall performance of veneers". ASTM C270-08C 2008 gives similar conclusions and advises that the following actions will increase tensile bond strength:

- increase the cement content of the mortar;
- keep air content of the mortar to a minimum;

- use mortars having high water retentivity;
- mix mortar to the maximum water content compatible with workability;
- use masonry units having moderate initial rates of absorption when laid;
- use bricks with a rough surface;
- minimise the time between spreading mortar and placing masonry unity;
- apply pressure between bricks in forming the mortar joint and do not subsequently disturb laid units; and
- tool the joints.

Plummer and Blume (1980) advise that masonry bond test results show huge scatter. The variables include flow of mortar, elapsed time between spreading mortar and placing brick in contact with it, suction of bricks, pressure or tapping applied to joint during forming and texture of the brick surface. They advise that there is no consistent relationship between mortar compressive strength and tensile bond strength. However, they found that the correlation between mortar flow and tensile bond strength is strong, with bond strength increasing as flow increases for all mortars.

As the bond strength is affected by the suction of the masonry unit, ASTM C270-08C advises that optimum bond is obtained when the mortar has properties which are compatible with the masonry units to be used.

The bond strength is affected by curing and climatic conditions and can be seriously impaired if no curing is provided and bricks are laid dry in hot and dry weather (Oliver 2009).

The Cement and Concrete Association of Australia (CCA 2001) wrote that bond strength was important for the development of sufficient tensile strength in the masonry to resist wind and earthquake forces and minor movement. Inadequate bond strength will inevitably lead to cracking in masonry construction. Because this cracking is a brittle mode of failure, there is often little scope for redistribution of stresses, and hence there is potential for widespread damage if bond strength is inadequate. The weakness might become apparent only when the masonry is subjected to an extreme load event, such as a high wind or an earthquake, when it might lead to collapse (CCA 2001).

Cracking might also be caused by minor movements in the footings or by thermal gradients (CCA 2001). The effect of this type of damage is primarily aesthetic, although it can also lead to long-term degradation, ingress of water into the building and a general lack of serviceability.

ASTM C270-08C states that the mortar tensile strength usually far exceeds the tensile bond strength between the mortar and the masonry unit. Therefore, under tensile or shear stresses bond failures occur at the interface of brick and mortar rather than within the mortar. Further, Plummer and Blume (1980) found that for most mortars, and with most commercially-produced bricks, a failure will generally be of adhesion or bond between mortar and brick rather than in the brick itself.

NZS 4210 (SNZ 2001) requires the seven day masonry-to-mortar bond strengths be not less than 200 kPa for structural compliance with NZS 3604 (SNZ 1999). However, as veneer is not a structural element, it actually need not comply with this value. Section C 2.2.3.2 of NZS 4210 states that typical bond strengths for mortars meeting NZS 4210 requirements are 500-1000 kPa for clay brick masonry and 400-900 kPa for concrete brick masonry.

Both NZS 4210 and AS 3700 (SA 2001) are based on the bond strength measured at an age of seven days, as it is not usually convenient to continue testing beyond seven days where site control monitoring of strength is required. As the strength of veneer mortar is usually limited by lack of water for the hydration process due to water loss from evaporation or being "sucked" out by the brick, the seven day strengths may be close to the strength finally achieved over time. The relationship between bond strength and time is examined in the paragraph below.

CCA (2001) found that for specimens air cured in the laboratory the bond strength tested at 28 days increased by 9% from that tested at seven days for clay bricks. However, if the specimens were cured under plastic the increase was 21% as there was less water loss by evaporation. The average ratio of bond strengths for outside and inside air cured specimens in Australia was 0.79 at seven days and 0.94 at 28 days. That these ratios are less than 1.0 indicates that more moisture was lost from evaporation from outside exposure and less hydration occurred. The results also highlight the beneficial effect of keeping new veneers moist and curing under plastic to facilitate hydration of the cement. However, excessive wetting of new masonry construction can lead to problems with shrinkage and efflorescence and is not recommended by the CCA.

2.1.2 Mortar-to-tie shear bond strength

Plummer and Blume (1980) reported that high compressive strength mortars have higher bond strength between the mortar and steel. The compressive strength of mortar increases with increased proportions of cement and with decreased water-cement ratio.

Thurston and Beattie (2009) found that when brick ties became loose in the mortar, over a large area of veneer, the veneer becomes weak under face loading. Hence, mortar-to-tie bond strength is critical to good seismic performance.

Thurston and Beattie (2009) also found that tie pullout strength from the mortar is reduced if the mortar joint is cracked (from face or in-plane loading).

2.1.3 Dry-bedding or full embedment of ties in mortar

Section 2.9.5.1 of NZS 4210 states that: "Wall ties shall be installed so that they are contained within the mortar bed over the full contact length, with a layer of mortar both above and below the tie". However, this full embedment procedure is generally disliked by tradespeople due to the extra time involved during brick-laying and because they prefer the dry-bedding process. Both processes are described below:

(a) *Full embedment.* The ideal practice is to put daubs of mortar onto the surface of a tie before inverting the tie and placing it onto the bottom brick so the mortar is now encapsulated between the tie and bottom brick. The tie is then screwed to the adjacent stud. Soon afterwards, a layer of mortar is placed onto the top surface of the bottom layer of bricks (and therefore the tie top surface) and the next layer of bricks placed.

For efficiency most tradespeople like to fix veneer ties for a whole row of bricks before placing the bricks and mortar above. To "achieve" full embedment, some tradespeople place daubs of mortar on the bottom brick adjacent to each stud for a whole row of bricks and then: (1) place the ties in the (by now) partially-stiffened mortar; and (2) screw the ties to the studs with a power tool. Bricks will later be mortared into position over the ties for the full row of bricks, but the delay at each tie location can be significant. This can lead to inconsistent tie bond strengths.

Placing a daub of mortar at a single tie location (whether on the tie or the brick surface) and then screwing the tie to the adjacent stud before moving on to the next tie location is cumbersome, as it requires picking up and putting down both the power tool and trowel for each tie.

(b) *Dry embedment.* For a whole row of bricks ties are placed on the bottom brick with no mortar between the underside of the tie and brick and screwed to the adjacent stud. Some time later a bed of mortar is placed onto the bottom brick (and therefore onto the top surface of the tie) in the normal way.

Based on brick tie tests to AS/NZS 2699.1 (SNZ 2000), BRANZ has provided an opinion for several brick tie types for given tie classifications that they may be used with dry embedment. Nowadays, most brick veneer in New Zealand is laid using dry-bedding. For this reason the tests in this report have generally used ties having dry-bedding, although for comparative purposes full tie embedment was also tested.

2.1.4 Mortar compressive (crushing) strength

Testing mortar compression strength requires mortar samples to be taken at the time of mixing and stored for 28 days (at 100% humidity and 21°C) before testing in accordance with NZS 3112 (1986).

NZS 4210 (SNZ 2001) calls for a compressive 28 day mortar strength of 12.5 MPa for structural compliance with NZS 3604:1999 *Timber framed buildings,* although this is not intended to apply to veneers. The standard also states that the strength of mortars for veneers shall follow the requirements of masonry suppliers. Some of these specify a strength of 12.5 MPa, and others rely on the mortar mix compositions listed in Table 2.1 of NZS 4210. The minimum compressive strength for veneers needs to be clarified in future revisions of the standard.

The compressive strength of mortar is sometimes used as a principal criterion for selecting mortar type, since compressive strength is relatively easy to measure and gives consistent and reproducible results. It is useful as a check on gross changes in the properties and proportions of the mortar constituents. The BS 5628-3 (2005) and ASTM C270-08a (2008) standards imply the measured strength of 7.3 MPa is satisfactory for mortar batched to the M3 (as defined in NZS 4210) durability exposure. AS 3700 (2001) does not stipulate the required/expected value of mortar compressive strength.

Plummer and Blume (1980) state that while the compressive strength of mortar per se is not of primary importance in most masonry structures, it has a relation to other properties of mortar. Also, for similar ingredients it is a reasonably accurate check on the proportioning and mixing. The determination of strength can be made relatively simply and the standard test methods give consistent and reproducible results. For these reasons, compressive strength is usually considered an important basis for comparing mortars.

ASTM C270-08a (2008) states that the importance of compressive strength of mortar is over-emphasised and that bond strength, good workability and water retentivity are more important characteristics.

Compressive strength of mortar increases with an increase in cement content and decreases with an increase in lime, sand, water or air content. As durability also increases with cement content (Guirguis et al 2003), the compressive strength of mortar can be an indicator of durability.

2.1.5 Water content

ASTM C270-08a (2008) states that water content is possibly the most misunderstood aspect of masonry mortar, probably due to the confusion between mortar and concrete requirements. Water requirement for mortar is quite different from that for concrete where a low water:cement ratio is desirable. Mortars should contain the maximum amount of water consistent with optimum workability. Mortar should also be re-tempered to replace water lost by evaporation.

BDRI (1977) state that wet mortar encourages complete intimate contact with the brick to assist in the development of bond strength. In their tests in Australia the high water-to-cement ratio implied by this technique was not sustained for long as within about 20 minutes of placing mortar between two bricks about 40-70% of the total water in the mortar was sucked out by the brick. This movement of water into the brick was stated to carry a concentration of cementitious material to the interface of the brick and mortar to enhance the strength of the adhesive bond between them.

Plummer and Blume (1980) recommend that mortar for masonry construction should be mixed with the maximum amount of water consistent with workability.

2.1.6 Workability

BDRI (1977) define workability as the combination of flow and cohesiveness. They state that it can only be judged by how the mortar feels on the trowel. Workability is a function of the cement, lime, air and water content and the sand grading. Well-graded sands are not always available. In these instances extra plasticising may be necessary, such as the addition of lime or admixtures. BDRI state that good workability was essential for maximum bond with masonry units.

ASTM C270-08a (2008) rate workability as the most important property of unset (i.e. plastic) mortar and states that the mortar must spread easily with a trowel into the separations and crevices of the masonry unit to allow it to adhere to the masonry surfaces. It also must readily extrude from the mortar joints when the mason applies pressure to bring the unit into alignment. However, when plastic it must still be stiff enough to support the weight of the masonry units above.

2.1.7 Flow/water retentivity

BDRI (1977) advise that “initial flow” is a laboratory-measured property of mortar that indicates the percentage increase in diameter of the base of a truncated cone of mortar when it is placed on a flow table and mechanically raised and dropped 25 times in 15 seconds. “Flow after suction” is another laboratory property which is determined by the same test, but performed on a mortar sample which has had some water removed by a vacuum.

“Water retention” is the ratio of flow after suction to initial flow, expressed as a percentage. It is a measure of the ability of a mortar under suction to retain its mixing water. This mortar property gives the mason time to place and adjust a masonry unit without the mortar stiffening. Water retention is increased through higher lime or air content, addition of sand fines within allowable gradation limits, or the use of water-retaining admixtures. Mortar in masonry stiffens through loss of water and hardens through normal setting of cement. This transformation may be accelerated by heat or retarded by cold. BDRI (1977) recommend that for good bond, high suction bricks should be combined with mortars with high water retention values and that, with low suction bricks, lower water retention is desirable.

ASTM C270-08a (2008) states that bond strength increases as flow increases to the point where detectable bleeding begins. Bleeding was defined as migration of free water through the mortar to its surface. Plummer and Blume (1980) also found that the tensile bond strength between mortar and brick increases with increased flow of the mortar.

ASTM C270-08a states that mortars used on-site normally have initial flows in the range of 130-150% to produce a workability satisfactory to the mason. Glencross-Grant and Walker (2003) suggested 105-115% and Plummer and Blume (1980) 110%-140%. However, those measured at BRANZ in Section 4 of this report were in the range of 66-134% with an average of 98%.

2.1.8 Sand

The bulk of mortar is sand. An increase in sand content increases the setting time and reduces potential cracking due to shrinkage of the mortar joint. The well-graded sand specified in standards such as NZS 3103 (1991) (called "standard sand" and often used for laboratory mortar tests) may produce quite different test results from sand that is used on-site.

Sand is usually described by the masses of different sized grains that have been separated as they pass through a group of standard sieves. A range of proportions of each of the grain sizes is permitted by a sand specification, and when this range is plotted graphically it produces a *grading envelope.* The result of a test on a particular sand is plotted as its *grading curve.* Both of these concepts are shown in Figure 1. The size of sand particles and their relative proportions plays an important part in determining mortar properties in both the wet and set states.

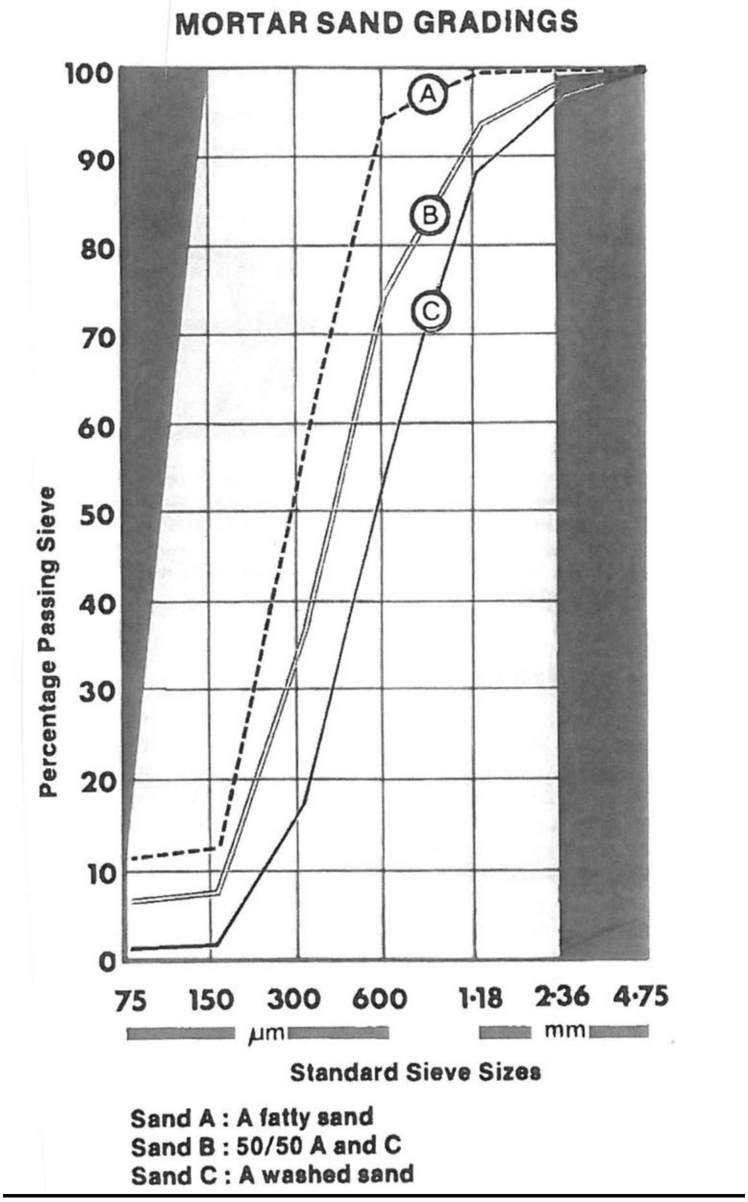


Figure 1. Sand grading curves from BDRI (1980)

NZS 4210 requires that mortar sand must comply with NZS 3103 *Specification for sands for mortars and plasters* (SNZ 1991). To comply with this standard the sands:

- must not retain more than 1% of a 2.36 mm sieve (limit of amount of large particle sizes)
- have a "sand equivalence" test result of 60 or greater (limits amount of dust and clay)
- the flow time and voids content of the sands must be within defined limits.

To "prove" the sands do not contain deleterious material (e.g. organic impurities), NZS 3103 requires one of the following to be satisfied:

(a) Proven to the satisfaction of the specifying authority that the sand has satisfactory service record over the previous 12 months;

(b) Satisfies (1) and (2) below:
   (1) Has been tested as a mortar at least twice in the last 12 months and has achieved in each instance at least 80% of the compressive strength of mortar made using standard sands;
   (2) Has successful history of flow time and voids content test results.

The writer questions whether this methodology will prove sands do not contain deleterious material. The procedure is very complex for tradespeople to achieve or Building Consent Authorities (BCAs) to oversee. Most mortar sand suppliers in New Zealand do not claim to supply sand which conform to NZS 3103. One of the reasons sometimes given is the difficulty in obtaining the “standard sand” which is necessary for the comparative tests. Thus, mortar used in New Zealand is generally from non-complying sand.

ASTM C270-089 (2008) states that well-graded sand reduces bleeding and improves workability. Sands deficient in fines produce coarse mortars of low workability, while sands with excessive fines produce weak mortars. CCA(2001) recommends that only clean sharp sand be used.

BDRI (1977) also recommend using well-graded sand and to avoid sand that is too coarse or too fine. Very fine material is likely to be clay. Although small proportions of clay in mortar act as plasticisers and slow the too-rapid absorption of water by high-suction bricks, too much clay can make wet mortar sticky and the set mortar has poor bond and compressive strength and has low durability. Excess clay in the mortar can also result in traces of mortar (smears) on the face of the brickwork that are hard to remove.

Section C2.2.2.1 of NZS 4210 (SNZ 2001) states that workability of mortars is significantly affected by sand grading and particle shape. For most mortars in New Zealand, it is likely that the use of an admixture or hydrated lime will be necessary to produce the desired workability.

Section 2.1.7.2 of NZS 4210 requires the chloride content of sands for mortar exposed to the weather to be less than or equal to 0.04% by mass.

To establish the influence of sand type on mortar characteristics, Glencross-Grant and Walker (2001) collected 50 sand samples in NSW with 18 selected for subsequent mortar testing, each representing the broad grading categories. A 1:1:6 cement:lime:sand by volume mortar mix was used throughout. No admixtures (such as air-entraining agents, set-retarders or water thickeners) were used in preparation of the mortars. Water sufficient for an initial flow table consistency between 105-115% was found by trial-and-error for each mix. Their conclusions were:

- Building mortar sands should be sharp, hard, durable and clean and free from contaminants (clay, iron, pyrites, salts, organic matter) likely to impair hardening, strength and durability. The mortar strength reduced and shrinkage increased as grading of the sand became finer, particularly when moisture-sensitive fines such as clay were present. The fines increased mortar shrinkage and impaired mechanical bonding. However, they state that bricklayers commonly prefer relatively high fines content, as it improves its cohesive and workability properties commonly referred to as “fattiness”.

- Grading alone was not sufficient to classify mortar sand quality. Particle shape and size, surface area and characteristic of fines were also important.

- There was a very strong correlation between bond strength and fines content, with a significant reduction in bond strength, and with increasing fines as shown in Figure 2. This is attributable to clay and other moisture sensitive fines preventing development of proper bonding and leading to high levels of shrinkage.

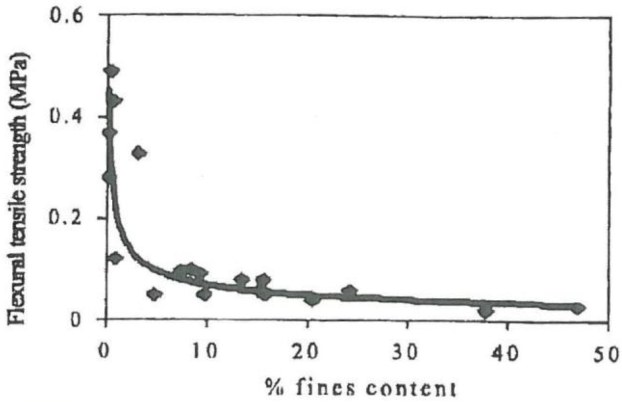


Figure 2. Graph of motar-to-brick bond strength versus fines content

- There was little experimental correlation between mortar compressive strength and bond strength. Indeed, the highest recorded bond strengths were associated with some of the weakest crushing strengths and least fatty mortars. They concluded that specifying mortar suitability on compressive strength clearly cannot be relied on to yield adequate flexural masonry bond strength.

- In general, the coarser coastal and estuary dredged sands proved better than finer inland loams. The rock crusher dust proved poorest of all sands in bond testing. They recommended that the use of admixtures such as fire-clay should be avoided, and that where water retentivity was inadequate then hydrated lime or other admixture (not unduly harmful to bond strength and durability) should be used rather than simply adding 'some dirt' as preferred by some bricklayers.

2.1.9 Admixtures

Admixtures (also called additives) are usually commercially prepared products and their compositions are not generally disclosed. They are intended to induce air entrainment, water retentivity, workability, retard the set, make the set mortar more impermeable, add colour and so on. ASTM C270-08a (2008) states that limited data ia available regarding the effect of proprietary admixtures on mortar bond and compressive strength, but that field experience has indicated that detrimental results frequently occurred. In Section 4 of this report, these and other mortar properties are measured for commonly available admixtures in New Zealand

ASTM C270-08a (2008) states that admixtures are necessary for factory-made pre-mixed mortars. There are also some special situations where the use of admixtures may be advantageous when added at the job site mixer. In general, however, ASTM C270-08a does not recommend the use of admixture. It also states that careful selection of the mortar mix, use of quality materials and good practice will usually result in a sound masonry veneer and that improprieties of the mix cannot be corrected by admixtures. However, now that lime is relatively rarely used apart from in the southern part of the South Island, admixtures are almost universally used in New Zealand.

ASTM C270-08a states that air entrainers should not be used, except under the most special circumstances, because this decreases bond and compressive strength. They do note though that most highly air-entrained mortar systems can support higher sand contents without the mortar losing workability.

BDRI (1980) states that air-entraining agents are the most common plasticising admixtures used in Australia. The purpose is sometimes to reduce the stickiness of mortar made with fatty (clay-rich) sands. Although they frequently improve the workability of wet mortar, BDRI consider that they can seriously diminish the strength and durability of set mortar and recommend that they should only be used as a last resort and be accurately measured. In Section 4 of this report, the effect on bond strength from overdosing admixtures and from over-mixing is measured.

BDRI (1977) state that although air-entraining agents suit the bricklayer, there is a danger that the resulting mortar will not suit the brickwork. Their tests showed that:

- from a level of around 5% by volume of air in a straight cement:lime:sand reference mix, the air content of chemically plasticised mortars increased by between 20-44%
- in comparison with cement:lime:sand reference mixes, the compressive strength of the plasticised mortars dropped by between 50-90%.

BDRI (1977) consider that there were two parameters which control adhesion to brickwork: (1) the gluing strength of the mortar; and (2) the area of contact. No contact can occur where there is an air bubble at the interface, and it follows that air entrained in mortar reduces the mortar’s ability to bond with brick.

In a survey, CCA (2001) found that the use of air-entrainers was widespread in Australia and detergent was the next most common workability enhancement used, despite the fact that AS 3700 prohibits it. They noted that it is common to overdose air entrainers because of the large gain in workability achieved. However, this overdosing produces severe reductions in bond strength as air bubbles consume the cementitious paste in the formation of shells, leaving less of the paste to form the interlocking layer at the surface of the brick, leading to reduced bond strength.

2.1.10 Colouring admixtures

Finely-ground synthetic metallic oxides are especially suitable for the purpose of pigmenting mortar. They are inert and chemically stable and are therefore unlikely to cause trouble in the mortars in which they are included. However, because oxide pigments are finely ground, and thus add more fine material to one often already having it in excess, BDRI (1977) state that it is unwise to add them to a mortar mix to a greater extent than about 15% of the mass of the cement. At this level they cease to have an appreciable effect in intensifying mortar colour.

2.1.11 Lime

Although lime is strictly an admixture, it is considered separately in this report. Nowadays, apart from in the south of the South Island, lime is rarely used in mortar in New Zealand and admixtures are used to give the required workability.

Lime was the traditional single cementitious material in masonry mortar and provides a cementing function, although significantly less than that of Portland cement which is used these days. Lime also has important functions in making wet mortar plastic and in imparting to set mortar the property of self-healing (often called autogenous healing) which will close up the hair cracks that can develop in masonry during setting and loading.

Section 2.2.2.1 of NZS 4210 states that hydrated lime may be omitted from the required mixes given in Table 2.1 if it can be demonstrated that the performance requirements of Sections 2.2.3.1 and 2.2.3.2 will be achieved. However, the strengths specified in the Section 2.2.3.1 and 2.2.3.2 are only for "structural compliance" and thus not applicable to veneer (as it is not a structural element). Thus, the compressive and bond strength values specified need not be complied with. Hence, NZS 4210 permits lime to be omitted from mortar mixes without any specific requirements.

ASTM 270-08a (2008) states that hydrated lime contributes to workability, water retentivity and elasticity. Lime mortars carbonate gradually under the influence of carbon dioxide in the air. Complete hardening occurs very slowly. Thus, residual lime can cause healing which is the re-cementing of small hairline cracks. Here, the lime goes into solution when water is present and migrates through the masonry where it can be deposited in cracks and crevices as water evaporates. Successive deposits may eventually fill the cracks. Such autogenous healing will tend to reduce water permeance. However, this can also cause some leaching.

Lime is particularly useful in providing workability in mortar made from poorly graded sands.

Portland cement produces approximately 25% of its weight in calcium hydroxide at complete hydration. This calcium hydroxide performs the same as lime during carbonation and redepositing.

2.1.12 Properties of mortar in a veneer

Once veneer is laid and dried, there is currently no reliable testing available to verify its compressive strength for compliance with specifications. There are several mortar hardness (scratch and impact) tests to assess the in-place durability and quality of mortar (Guirguis et al 2003), but these have not found widespread acceptance in the industry.

CCA (2007) reports on a durability test which may be used to assess the potential long-term performance of masonry mortar. This is based on a controlled scratching of the mortar surface during which the penetration into the mortar is measured (called the scratch index). The test simulates and accelerates the physical forces that can cause mortar degradation in service. A set of performance criteria is included in the Australian standard for masonry structures – AS 3700 (2001).

Oliver (2009) states that, in practice, determining the quality of the mortar in a laid-up veneer comes down to common sense, observation and experience. If the mortar is powdery, its quality and strength should be questioned. Someone must make the call that the mortar will be able to perform its functions.

2.1.13 Durability

The durability of mortar is an important topic that has not been covered in this study which focuses on the structural performance of brick veneer. The main aspects are: corrosion of metal embedded in the veneer; the water-tightness of the veneer (some water is expected to penetrate the veneer, but this is expected to be handled by the cavity) and the mortar fretting away under wind-driven abrasion; salt crystallisation; and chemical attack or freeze-thaw action so that the veneer loses its face load strength to resist wind pressure and earthquake forces. (Axial strength is expected to be of lesser significance.) Any damage also has an aesthetic consequence. A lack of bond at the interface of the mortar and the masonry unit may lead to moisture penetration through these areas.

NZS 4210 (2001) classifies various zones of New Zealand depending on the severity of exposure to wind-driven sea salt or geothermal gases. AS/NZS 2699.1 (2000) then specifies the durability of metal components used in the veneer construction to meet the 50-year durability requirement with B2 of the New Zealand Building Code (NZBC) current at the time. The current B2 (DBH 2011) has similar requirements. This has resulted in all brick ties being either galvanised steel or stainless steel. Comments from the field indicate that little corrosion is being seen on these ties when circumstances allow them to be examined.

Section 2.9.5.1 of NZS 4210:2001 requires ties to be fully embedded (i.e. encapsulated) in the mortar, which is apparently partly for durability reasons. However, BRANZ has provided opinions that specific ties may be dry-bedded as they were shown to meet the strength/deformation requirements of AS/NZS 2699.1 (SNZ 2000). The opinions now cover virtually all the ties sold in New Zealand. Tradespeople strongly dislike using full embedment, and if used sometimes they place whole rows of ties in small deposits of mortar at each tie location before returning to the first brick in the row and then commencing laying the course of mortar and placing the top bricks. Shrinkage/drying of the small deposits of mortar has taken place by the time the tradesperson reaches the ends of the rows, which will affect bond and maybe durability too.

Ties are required to be protected by an end cover of 15 mm in the mortar (Section 2.9.5.2 of NZS 4210). Thus, the portion of the ties which are exposed in the cavity, where salt build-up may occur, may be more vulnerable to corrosion. So it is probable that corrosion is not a particular issue with dry-bedded ties.

The durability performance requirement in AS 3700 (2001) is that a masonry member or structure shall withstand the expected wear and deterioration throughout its intended life (taking into account the exposure environment and the importance of the structure) without the need for undue maintenance. The standard recommends the use of a range of classes of mortar for different exposure conditions. It also gives deemed-to-satisfy mortar compositions for each class of mortar.

NZS 4210 (SNZ 2001) requires mortar to consist of Portland cement, sand and hydrated lime given in composition ratios specified in Table 2.1 of the standard. These ratios are a function of the site exposure category and the mixes are intended to ensure adequate mortar durability. The required resistance of the masonry units to salt attack is also given. Section 2.2.2.1 of NZS 4210 states that admixtures may be used (subject to the same provisions).

Brick veneer will not be damaged due to freezing unless the brickwork is nearly saturated (ASTM C270-08a). Masonry walls, heated on one side, will stand many years before requiring maintenance. However, ASTM C270-08a considers that parapets, masonry paving, retaining walls and other masonry exposed to freezing while saturated represents extreme exposure and requires a more durable mortar.

Properly entrained air in masonry mortar generally increases its resistance to freeze-thaw damage where extreme exposure exists (such as repeated cycles of freezing and thawing while saturated with water). Air contents do not have to be large for this to be achieved. Durability is adversely affected by over-sanded or over-tempered mortars as well as the use of highly absorbent masonry units.

CCA (2007) also consider that most masonry structures exhibit excellent long-term performance with comparatively low maintenance cost. They note that the veneer durability is influenced by the durability of both the masonry units and mortar.

2.2 Masonry units

ASTM C270-08a (2008) notes that masonry units are absorbent by nature, with the result that water is extracted from the mortar as soon as the masonry unit and mortar come into contact. The amount of water removal affects the the bond strength.

The suction exerted by the masonry unit affects the development of bond. Masonry units vary widely in the initial rate of absorption (suction). It is therefore necessary that the mortar chosen has properties that will provide compatibility with the properties of the masonry unit being used, as well as environmental conditions that exist during construction and the construction practices peculiar to the job.

ASTM C270-08a considers that the extraction of too much or too little of the available water in the mortar tends to reduce the bond between the masonry unit and the mortar. A loss of too much water from the mortar can be caused by low water retentivity mortar, high suction masonry units, or dry windy conditions. Where lowering the suction by pre-wetting the units is not possible, the time lapse between spreading the mortar and laying of a masonry unit should be kept to a minimum. On the other hand, a very low suction masonry unit also usually results in poor bond.

2.3 Workmanship and construction procedures

2.3.1 Poor construction techniques

Once the mortar between adjacent units has begun to stiffen, any movement of the masonry units breaks the bond between the mortar and the masonry unit. The mortar will then not be sufficiently plastic to re-establish adherence to the masonry unit. CCA (2001) states that movement of the units after initial contact with the mortar is a common problem leading to inadequate bond strength, and that any disturbance of units more than a few seconds after placement should be avoided.

CCA (2001) states that one of the most significant sources of problems with bond strength is poor construction techniques. Placing a long line of mortar in a course before placing the bricks on it allows the loss of too much moisture to the lower course and consequently weakens the bond to the upper course.

ASTM 270-08a (2008) came to similar conclusions and states that workmanship has a substantial effect on bond strength. They also state that the time lapse between spreading mortar and placing masonry units should be kept to a minimum because the flow will be reduced through suction of the unit on which it is first placed. This is especially so in hot, dry and windy conditions, or with use of highly absorbent masonry units.

Historically, masonry head joints were often only filled with fillets at the external edges, but this practice is rare nowadays. This can severely weaken a veneer (CCA 2001). Allen (1986) warns of the dangers of this practice and states that these small dabs of mortar are more susceptible to suction of the masonry units and evaporation, with the result that shrinkage cracks invariably open up. These cracks allow water ingress, sound and noise penetration, directly contribute to loss of masonry strength, and are weak under wind and seismic loading. ASTM 270-08a (2008) also stresses the importance of providing full head joints.

Section 2.7.4.2 of NZS 4210 limits the depth of furrowing (grooves) of bed joints to a depth of 25% of joint thickness. ASTM 270-08a (2008) recommends elimination of deep furrows in horizontal bed joints. Allan (1986) notes that deep furrowing of the mortar in a cored veneer is unnecessary, because the perforations allow levelling of the brick through excess mortar being squeezed into the cavities. Even with solid concrete units, a light surface roughening is all that is required.

2.3.2 Tooling mortar joints

Tooling mortar joints is the process of pressing/sliding a metal tool across a fresh mortar joint.

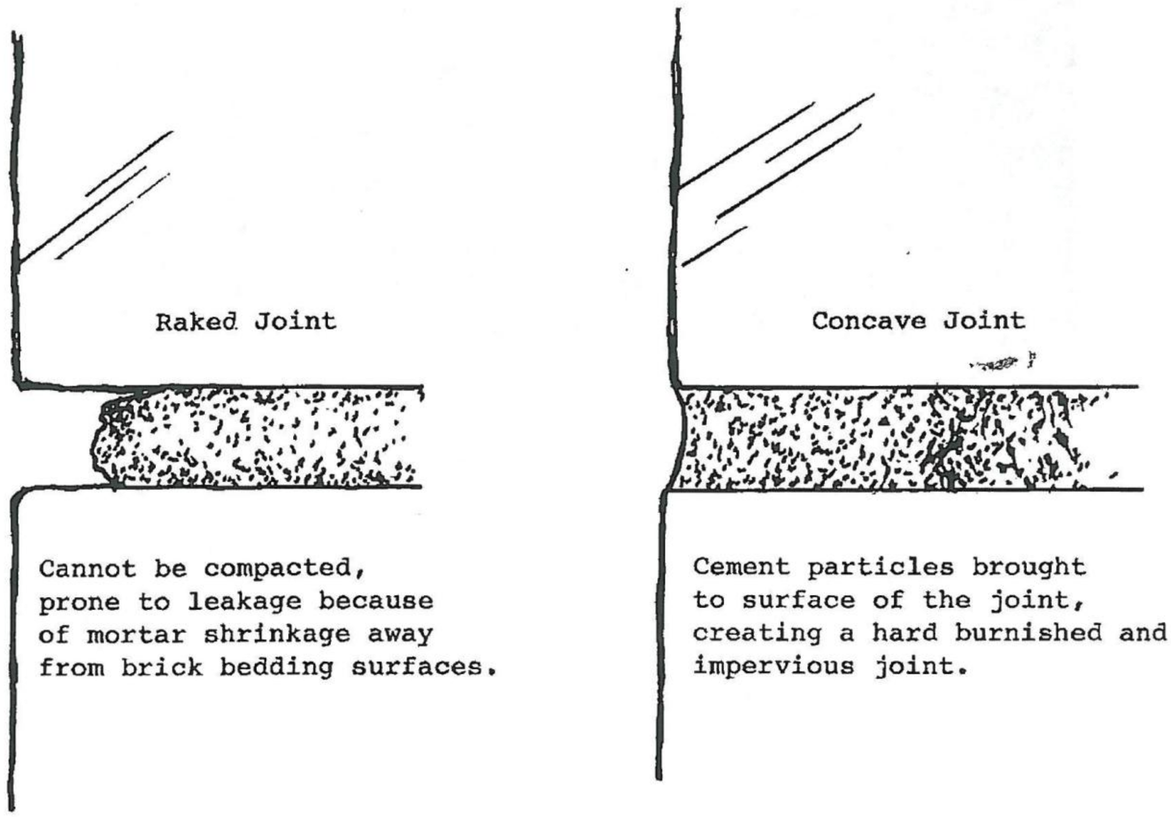
Tooling of the mortar joint should be done when its surface is thumb-print hard utilising a jointer having a diameter slightly larger than the mortar joint width. ASTM 270-08a (2008) states that joint configurations other than concave can result in increased water permeance of the masonry assemblage. Finishing is not only for appearance, but to seal the interface between mortar and masonry unit, while densifying the surface of the mortar joint.

Section 2.7.7.1 of NZS 4210 (SNZ 2001) requires mortar joints to be:

(a) Concave tooled to a depth not exceeding 6 mm and burnished after the initial stiffening has occurred, or
(b) Raked out, pointed and tooled to a depth not exceeding 6 mm after the initial stiffening has occurred.

By its omission from the above list, the “flush” and “weathered” joints, e.g. as detailed by Monier (2006), are not allowed. Many veneer mortar joints in New Zealand are raked and brushed without tooling. This is expected to result in a weaker non-complying finish. To investigate the relative strength of this and other joints, couplet tests were done as detailed in Section 4.

Allen (1986) discusses the prevalence of raked untooled joints in older construction and has the following comments. Raked joints will provide small ledges on which water can sit. In the procedure of raking joints, mortar is pulled away from the brick bedding surfaces. This promotes mortar shrinkage, thereby making water entry more easy. Deeply raked joints will in some cases touch on brick perforations, entrapping water for much longer periods before being able to dry out. For these reasons, raked joints have been found to be about five times more prone to water penetration than properly executed concave joints.



2.3.3 Tapping

After the top unit is pressed into position many tradespeople tap the top brick several times on the top surface with the trowel. Plummer and Blume (1980) found mortar bond strengths obtained where the tapping technique was used exceeded those from the specimens where hand pressure alone was used in all cases by 50-100%. This suggests that the contact between mortar and brick is much improved by impact or by the vibration resulting from impact.

2.3.4 Wetting bricks before laying

Section 2.19 of NZS 4210 allows masonry units to be lightly dampened before laying, particularly in air temperatures above 25ºC. However, Section 2.7.2.2 of this standard requires masonry units to be protected from the weather prior to laying to ensure they are not laid in a saturated state. CCA (2001) reports that dampening or wetting the bricks prior to laying reduces the suction of the masonry which leads to poor bond. However, BRANZ tests in Section 4 found the opposite results.

A survey in Australia by CCA (2001) found that most tradespeople dampened both clay and concrete bricks before laying. CCA comments that wetting bricks affects the delicate balance between transport of moisture and fines to the interface and subsequent hydration of the cement to form a strong bond. They consider that it is better to match the mortar to the suction of the units, by means such as the addition of lime to the mix, rather than to wet the units before laying. However, Plummer and Blume (1980) advise that specimens assembled with bricks which have been dampened prior to laying develop substantially higher bond strengths than those assembled with dry brick. They recommend that bricks with high suction should be wet before laying.

2.3.5 Re-tempering

Re-tempering is the process of adding water and re-mixing mortar that has stiffened in the barrow. It therefore results in a decrease in mortar compressive strength. The amount of the reduction increases with the amount of water addition and time between mixing and re-tempering. However, most standards allow some controlled re-tempering as stiff mortar has low bond strength and it is considered expedient to sacrifice some compressive strength of the mortar in favour of improved bond.

BDRI (1980) recommend that mortar is mixed thoroughly so that it is as wet as possible and, up to one-and-a-half hours after mixing, to replace water lost through evaporation by the addition of more water (but discard unused mortar after two hours).

ASTM C270-08a (2008) recommends that the addition of water to mortar to replace evaporation within specified time limits should not be prohibited. Although the compressive strength of the mortar is reduced slightly by re-tempering, bond strength is usually increased. Because re-tempering is harmful after mortar has begun to set, ASTM C270-08a recommends that all site-prepared mortar should be placed in its final position as soon as possible, but always within two-and-a-half hours after the original mixing (or the mortar is discarded).

Section 2.2.2.2 (d) of NZS 4210 (SNZ 2001) states that stiffened mortar may be re-tempered providing the water is added to a basin formed by the mortar and the mortar is carefully worked into it. However, mortar not used within one-and-a-half hours after the addition of cement to the mix must be discarded, except this may be increased to two hours if the temperature is less than 5ºC.

2.3.6 Mixing time

Section 2.2.2.2(a) of NZS 4210 (SNZ 2001) requires the mortar to be mixed in a mechanical mixer for a minimum time of five minutes. It does not nominate a maximum time, although time in the mixer will increase the mortar air content which will result in weaker bond strength (ASTM C270-08a). This is particularly important if an air-entraining admixture has been used.

2.3.7 Weather conditions

Sections 2.18 and 2.19 of NZS 4210 discuss the precautions necessary for laying masonry in cold (less than 5ºC) and hot (greater than 25ºC) temperatures. The cold conditions require the masonry to be covered for the first 24 hours and prohibit the use on materials containing ice. The hot conditions allow the masonry units to be lightly dampened, require the mortar to be kept moist, and require the constructed veneer to be cured for the first 24 hours.

ASTM C270-08a (2008) states that during warm, dry, windy, summer weather, mortar must have a high water retentivity to minimise the effect of water lost by evaporation. In winter, a lower water retentivity has merit because it facilitates water loss from the mortar to the units prior to a freeze. Units should not be laid in temperatures less than 0°C.

ASTM C270-08a recommends very light wetting of the in-place masonry (such as fog spray) under hot, dry and windy conditions. However, they consider that curing of the mortar by the addition of considerable water to the masonry assemblage could prove to be detrimental as the excess moisture might saturate the masonry, creating movements which decrease the adhesion between mortar and masonry unit.

Oliver (2009) discusses why the brick veneer in a particular New Zealand house needed to be removed and replaced. The veneer had been laid in hot weather with a warm drying wind. He surmises that the moisture had rapidly evaporated from the fresh mortar resulting in inadequate hydration and strength. The mortar powdered easily.

2.3.8 Efflorescence

Efflorescence is a crystalline deposit (usually white) of water soluble salts on the surface of masonry. ASTM C270-08a (2008) states that the principal objection to efflorescence is the visual appearance of the salts on the surface and the nuisance of their removal. Under certain circumstances, particularly when exterior coatings are present, salts can be deposited below the surface of the masonry units. When this crypto florescence occurs, the force of crystallisation can cause disintegration of the masonry.

A combination of circumstances is necessary for the formation of efflorescence. First, there must be a source of soluble salts. Second, there must be moisture present to pick up the soluble salts and carry them to the surface. Third, evaporation or hydrostatic pressure must cause the solution to migrate. ASTM C270-08a states that if any one of these conditions is eliminated efflorescence will not occur. They note that full bed and head joints, along with compacting finish on a concave mortar joint, will reduce water penetration and reduce efflorescence.

Salts may be found in the masonry units, mortar components, admixtures or other secondary sources. These can leach out and be concentrated on the surface. Very little is required to cause efflorescence problems. Removal of efflorescence from the face of the masonry can frequently be achieved by dry brushing. Since many salts are highly soluble in water, they will disappear of their own accord under normal weathering processes. Some salts, however, may require harsh physical (or even chemical) treatment if they are to be removed.

2.3.9 Non-complying construction

Allen (1986) summarises the consequences of non-complying construction. Hopefully few of these apply nowadays, but it makes interesting reading and some aspects are summarised below:

- Walls constructed by inexperienced labourers showed up to a 50% reduction in the ultimate shear strength, but this should not be regarded as the limit of strength reduction that could occur where there are lapses of workmanship and inspection.
- In 1986 the greater percentage of veneers were fixed using “grey” ties, i.e. ties which have not been advertised, did not have information sheets or technical back-up, and had not been shown to comply with the relevant New Zealand Standards. Allen showed photographs of 10 non-complying ties which were often just variations of bent No 8 fence wire. (Hopefully these would not be accepted by today’s authorities and it will not surprising if older-style veneers perform badly in future earthquakes.)
- Allen was concerned about re-use of masonry units without evidence of crushing strength, mortar bond strengths, and certification as to soundness and durability. He argued that demolition bricks are salvaged from an era of building when bricks were

not produced to a standard. When vintage buildings are torn down, there was no possible way of differentiating between good bricks, commons, and underfired or doughboys. Further, bricks which are retrieved from vintage buildings are not “de/aired”. De/airing is a relatively modern clay-working process in which the wet clay particles are passed through a vacuum chamber with a negative pressure of about 90 kPa. This knitted the clay particles together to form a tight cohesive body which fired well and is more regular in configuration. Also, used bricks have their bedding surface pore structures clogged with lime particles, dirt or other deleterious matter. These will adversely affect the bond of mortar to brick. Allen wrote that the Ceramic Industries Association, mindful of its industry responsibilities and a corresponding obligation to the building trade, has issued the following policy statement:

“Demolition brick cannot be shown to comply with the New Zealand Standards in respect of crushing strengths, soundness, durability or masonry-to-mortar bond strengths. Their use is therefore prohibited”.

2.3.10 Separation of veneer at corners and windows for earthquake resistance

NZS 4210 (SNZ 2001) states that in the Coalings (USA) earthquake, veneer with poor fixings collapsed but veneer with correctly installed ties survived undamaged despite the absence of a special separation details. Veneer damage at Edgecumbe in 1987 revealed primarily a problem in inadequate tie provision and fixing.

However, NZS 4210 also states that “the adoption of full corner and window separation details is seen as a logical engineering step towards limiting damage in veneers. Veneer construction in these cases is likely to make use of the special flexible ties whose performance is set out in AS/NZS 2699.1”. In contrast to this, Thurston and Beattie (2009) found that such separation is not necessary and brick veneer buildings are likely to perform better without the separations. Veneer corner elements simply rock over and sustain a little damage.

3. STAGE I. SAMPLING AND TESTING MORTAR AND SAND FROM BUILDING SITES

3.1 Separate private study

A private study in 2008 of 10 unidentified building sites in the greater Auckland region found average (from three samples) mortar compressive strengths in the range 5.2 to 10.0 MPa, with an average overall value of 7.1 MPa. Nine of the 10 sites used volume batching generally in a sand:cement ratio of 4:1, although one site used a ratio of 5:1 and one 3.5:1. The 10th site shovel batched the mortar in an approximate ratio of 3:1. The cylinders from this site gave the highest compressive strength.

3.2 BRANZ mortar sample from Kapiti

As a pilot project to that described in Sections 3.3 and 3.4, the compressive strength of mortar from one Kapiti site was measured. Mortar cylinders were made in steel moulds and gave an average compressive strength to NZS 3112 Part 1 (SNZ 1986) of 7.3 MPa. The mix air content was measured on-site as 22% (using the Air Meter Test of NZS 3112 Part 1). The sand grading curve is given in Figure 3. The sand was stated by the tradesman to be the same as the Winstones Aggregate sand called Sand Y, and did in fact give a similar grading curve (cf Figure 17).

The sand:cement ratio was 4:1 and was measured using 20 L pails. Two capfuls of the admixture No. 2 from Table 22 was used per mix which BRANZ measured as 40 mL. This was calculated to be 22% more than the manufacturer's recommended dose of 100 mL per bag of cement.

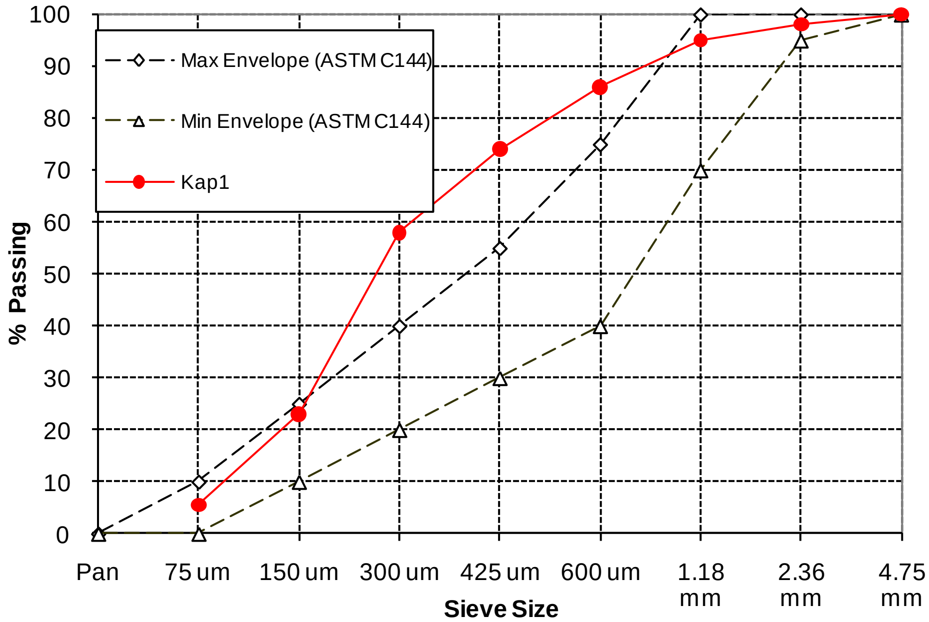


Figure 3. Grading curve of the BRANZ Kapiti sand sample

3.3 Mortar sampled at building sites in the greater Auckland region

Mortar was sampled from 15 building sites: eight in the Auckland region, five from the Waikato region and two from the Bay of Plenty (BOP) region. The method used was to randomly arrive on-site, sometimes unannounced, and in most cases with the masonry tradesman being unaware of the purpose of the visit. The investigator would wait until the masonry tradesman had a barrow partially full of mortar and then requested they:

- provide details of the mortar composition
- let the investigator make compression test specimens from the mortar
- make couplet specimens to enable the mortar bond strength to be measured
- let the investigator take sand samples.

The tradesman was advised that nothing in this study would identify them or the site with the mortar taken (thus only general information is given in this report).

3.3.1 Summary from questionnaires filled out on-site

At all sites where mortar was site batched it always contained one and only one admixture. No mix used lime. Mortar details are summarised in Table 1 with a cross-reference to admixture details in Table 22.

Table 1 contains the stated source of the sand used. In 11 of the 15 cases the sand and cement were volume batched in a 4:1 ratio. In two cases the sand and cement were only batched by the number of shovel loads when placing the material into the mixer, but here the target ratio was also 4:1. In one case volume batching with a ratio of 3:1 was used. The remaining cases used a pre-mixed commercially available mortar (Type 8 in Table 22) and the tradesman merely added water and mixed.

The amount of cement in each mixer load was unknown but is expected to be close to half of a 40 kg bag. Based on this assumption and the volume of admixture the tradesman said they used, Table 1 compares the dosage used with that recommended. That is, it gives (as a percentage) the ratio of the quantity of admixture used to the average quantity of admixture recommended by the manufacturer. It can be seen that in nine of the 14 cases this was greater than 100%; in two cases it was less than 100% and the remaining three cases were 100%. It is unlikely that more than half a bag of cement was used per mortar batch and possible that less than half a bag was used. Thus, the overdosing may be greater than Table 1 suggests.

Mixing times did not exceed 10 minutes, except that in one instance it was 20 minutes. The time in the barrow when sampling occurred is also given as this may affect the measured mortar strengths.

Table 2 contains information on the brickwork and Table 3 is a legend for Table 1 and Table 2.

3.3.2 Mortar compression tests

At each site mortar taken from the barrow was used to make three mortar compression test cylinders in accordance with NZS 3112 Part 1 (SNZ 1986), except that the test specimens were made in 200 x 100 mm cardboard cylinders. These were covered and left overnight on the site. The next day these specimens were fully submerged in a bucket of water, delivered to an accredited testing laboratory, and stored at 100% humidity until tested for compressive strength at age 28 days.

3.3.3 Mortar couplet bond tests

At each site the masonry tradesman made five brick couplets using two dry bricks (as described in Section 4.2) supplied by BRANZ. They did this by placing the mortar on the bottom brick, pressing the second brick on top, and lightly tapping the two together if necessary. The mortar was struck flush on all edges. The brick couplets were numbered 1–5 and dated. The samples were left on-site in a secure area for 24 hours to dry. The specimens were then wedged into and stored in a specially prepared plastic box for transportation and stored under cover.

The investigator eventually performed bond couplet tests as per Section 4.3, except that instead of applying a force by hand and measuring it via a load cell, he gently added sand to a bucket which was later weighed (see this test set-up in Figure 4). Instead of the tests being done at seven days as per NZS 4210 (SNZ 2001) the tests were performed at times ranging from 19-49 days as shown in Table 4.

Figure 4. System for measuring bond strength from brick couplets in the greater Auckland region

3.3.4 Mortar crushing and couplet bond strengths

The average mortar crushing strength was 9.6 MPa for the Auckland region and 8.0 MPa for the BOP/Waikato region (overall 8.9 MPa). The Coefficient of Variations (C.o.V.s) from each group of three specimens were small with an average value of 0.05 which gives confidence in the results. The two lowest averages were 4.8 and 5.2 MPa.

The average couplet bond strength was 311 kPa for the Auckland region and 205 kPa for the BOP/Waikato region (overall 281 kPa). The C.o.V.s were large with an average value of 0.39. The average bond strengths were slightly lower than the values measured in the BRANZ laboratory tests (see Table 7 in Section 4). This was even though they were measured at an average age of 25 days as against the age of seven days in the laboratory tests at the BRANZ site. The C.o.V.s were also slightly higher than measured at BRANZ.

Figure 5 shows the greater Auckland region site test results have a poor correlation between mortar compressive and bond strength. The best fit curve does not pass through the origin.

Table 1. Site mixes sampled in the greater Auckland region

Label	Mix	Location	Sand: Cement Ratio	Sand Equiv.	Fineness Modulus	Admixture Type	Admixture quantity ratio	Mix time (Minutes)	Time in barrow (Minutes)
J1	Volume	Auckland	4	82	1.1	3	133%	5	5
J2	Volume	Auckland	4	86	0.96	3	133%	10	15
J3	Volume	Auckland	N/A	N/A		8	N/A	5	5
J4	Volume	Auckland	4	79	1.52	6	100%	20	10
J5	Volume	Auckland	4	See J4		3	100%	7	20
J6	Shovel	Auckland	3.5*	80	1.11	3	67%	7	5
J7	Volume	Auckland	4	See J6		3	133%	5	5
J8	Volume	Auckland	3	See J6		3	67%	5	10
J9	Volume	Waikato	4	73	1.7	2	120%	10	60
J10	Volume	Huntly	4	71	1.09	2	120%	8	45
J11	Volume	Waikato	4	See J9		3	133%	5	50
J12	Volume	Waikato	4	83	2.42	4	200%	4	30
J13	Volume	Matamata	4	80	1.29	7	100%	8	60
J14	Volume	Bay of Plenty	4	74	1.79	4	133%	5	60
J15	Shovel	Bay of Plenty	4**	86	1.4	4	200%	10	15

Table 2. More details of site mixes sampled in the greater Auckland region

Label	Mortar finish on house	Tie embedment	Joint tooled smooth?	Note 1	Note 2
J1	Flush	Dry bedded		Veneer to be plastered	50% bricks kept dry rest left open to weather
J2	Rake 8 mm	Dry bedded		Face veneer	
J3					
J4	Rake 6 mm	Full bedded	Yes		50% bricks kept dry rest left open to weather
J5				Veneer is to be bagged	
J6	Rake 6 mm	50:50	No		
J7	Rake 7 mm	Dry bedded	No		
J8		Dry bedded		Bricks are fairly wet	Bricks still in plastic but broken
J9	Rake 6 mm	Dry bedded	No		
J10	Rake 6 mm	Dry bedded	Yes	White cement with colouring	50% bricks kept dry rest left open to weather
J11	Flush	Dry bedded		Veneer to be plastered	
J12	Flush	Dry bedded		Veneer to be plastered	50% bricks kept dry rest left open to weather
J13	Cove, ironed	Dry bedded		Sand/admixture stated to be VG	
J14	Flush	Dry bedded	No	Veneer to be plastered	
J15	Rake 6 mm	Dry bedded	No		

Table 3. Legend to tables giving site mixes sampled in the greater Auckland region

Heading	Name	Description
Mix	Volume	Sand and cement for the mortar were batched using containers of fixed volume
	Shovel	Sand and cement for the mortar was batched by the number of shovel fills
	Bagged	Mortar was from a commercially bagged supply of premixed dry ingredients
Admixture quantity ratio	Percentage of mean of range of manufacturer's recommendations assuming that half a bag of cement is used in each mixer load	
Sand:cement ratio	* 9 shovels/half bag of cement = approx 3.5:1.	
	** 10 shovels/half bag of cement = approx 4:1.	
Mixing time	Time used to mix the mortar in the rotating drum before tipping into the barrow	
Time in barrow	Time mortar had sat in the barrow before the mortar samples were taken	

Table 4. Mortar strengths measured from site samples collected in the greater Auckland region

Label	Location	Age (days) at testing couplet bond strength	Couplet bond strength (kPa)		Compressive strength (MPa)		Density kN/m^3
			Ave	C.O.V.	Ave	C.O.V.	
J1	Auckland	33	275	0.16	7.83	0.037	1694
J2	Auckland	33	135	0.27	6.83	0.042	1540
J3	Auckland	27	338	0.20	11.66	0.157	1693
J4	Auckland	27	353	0.18	13.66	0.042	1763
J5	Auckland	27	353	0.20	9.16	0.063	1663
J6	Auckland	49	317	0.23	10.00	0.087	1713
J7	Auckland	49	433	0.14	9.67	0.030	1663
J8	Auckland	49	283	0.43	8.00	0.000	1663
Average from Auckland		**36.8**	**311**	**0.23**	**9.60**	**0.06**	**1674**
J9	Waikato	33	317	0.23	5.20	0.056	1513
J10	Huntly	33	317	0.23	10.20	0.028	1650
J11	Waikato	33	367	0.68	8.00	0.063	1710
J12	Waikato	24	27	0.83	7.00	0.000	1593
J13	Matamata	24	85	1.21	10.50	0.048	1543
J14	Bay of Plenty	19	15	0.53	4.83	0.060	1590
J15	Bay of Plenty	19	304	0.37	10.50	0.048	1656
Average from BOP/Waikato		**26.4**	**205**	**0.59**	**8.03**	**0.04**	**1608**
Average of all regions		**25.5**	**261**	**0.39**	**8.87**	**0.05**	**1643**

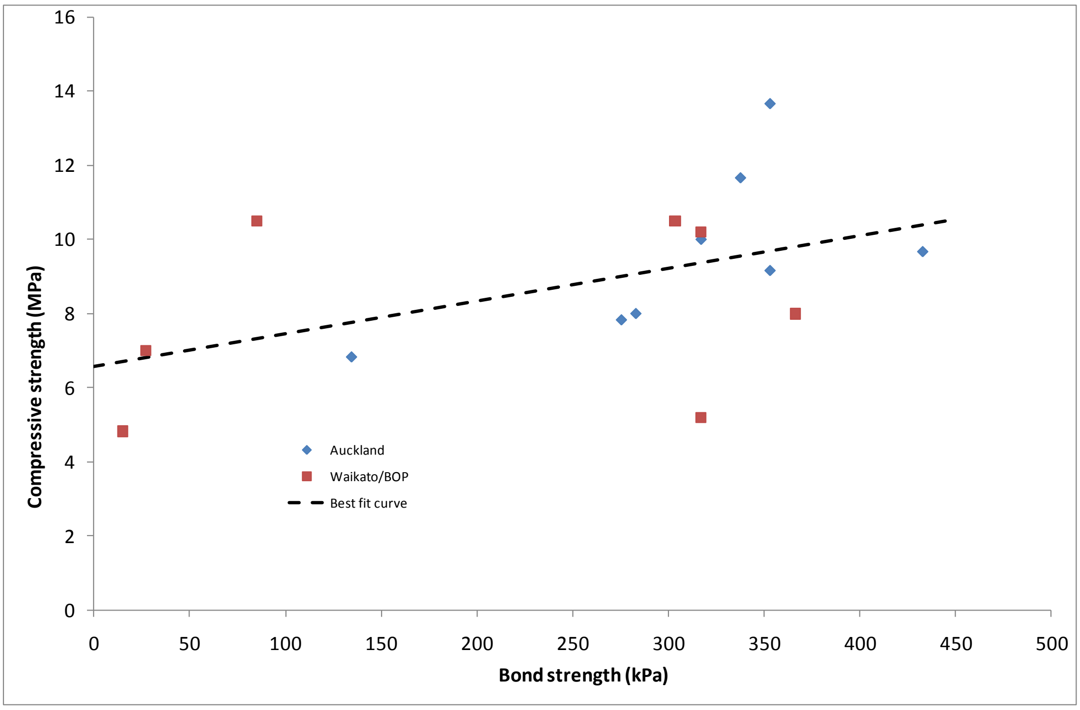


Figure 5. Relationship between mortar bond and compressive strengths from mortar sampled in the greater Auckland region

3.3.5 Sand samples

Figure 6 and Figure 7 show the measured grading of sand samples collected in the greater Auckland region: J3 was a pre-mixed mortar; the sand source for J5, J6, J7 and J8 was the same; and similarly the sand source for J9 and J11 was the same.

Many of the sand gradings showed wide divergence from the standard curves which may explain why the measured bond strengths were low. Some had a steep portion indicating a big proportion of the sand the similar sand particle size (J1, J2, J6, J10), with the worst being J2. J10 has the largest discontinuity between the 300 µm and 150 µm sieves. Most were low on fines which indicates that a large amount of admixture or cement would be needed to give good workability.

3.4 Mortar sampled at building sites in the Wellington region

Mortar was sampled at 12 sites in the Wellington region in a similar manner to that done in the greater Auckland region. However, it was found that the same sand had been used at each site (called Sand Y) and the couplet strengths were measured at age seven days. Mortar cylinder compressive strengths were measured at age 28 days.

The results in Table 5 showed very low bond strengths and only moderate cylinder compressive strengths. Scatter was high in both instances. In six of the 70 bond couplet specimens the specimens broke during transport. These results are ignored. With couplets taken from one particular building site (called “MATS8”), four specimens broke as the bucket was being added to the bar. These specimens (and also for an additional four specimens taken from other sites where this happened) were assigned strengths consistent with them breaking from the weight of the bucket alone.

Figure 8 shows the specimens taken at various sites in the Wellington region have a poor correlation between mortar compressive and bond strength.

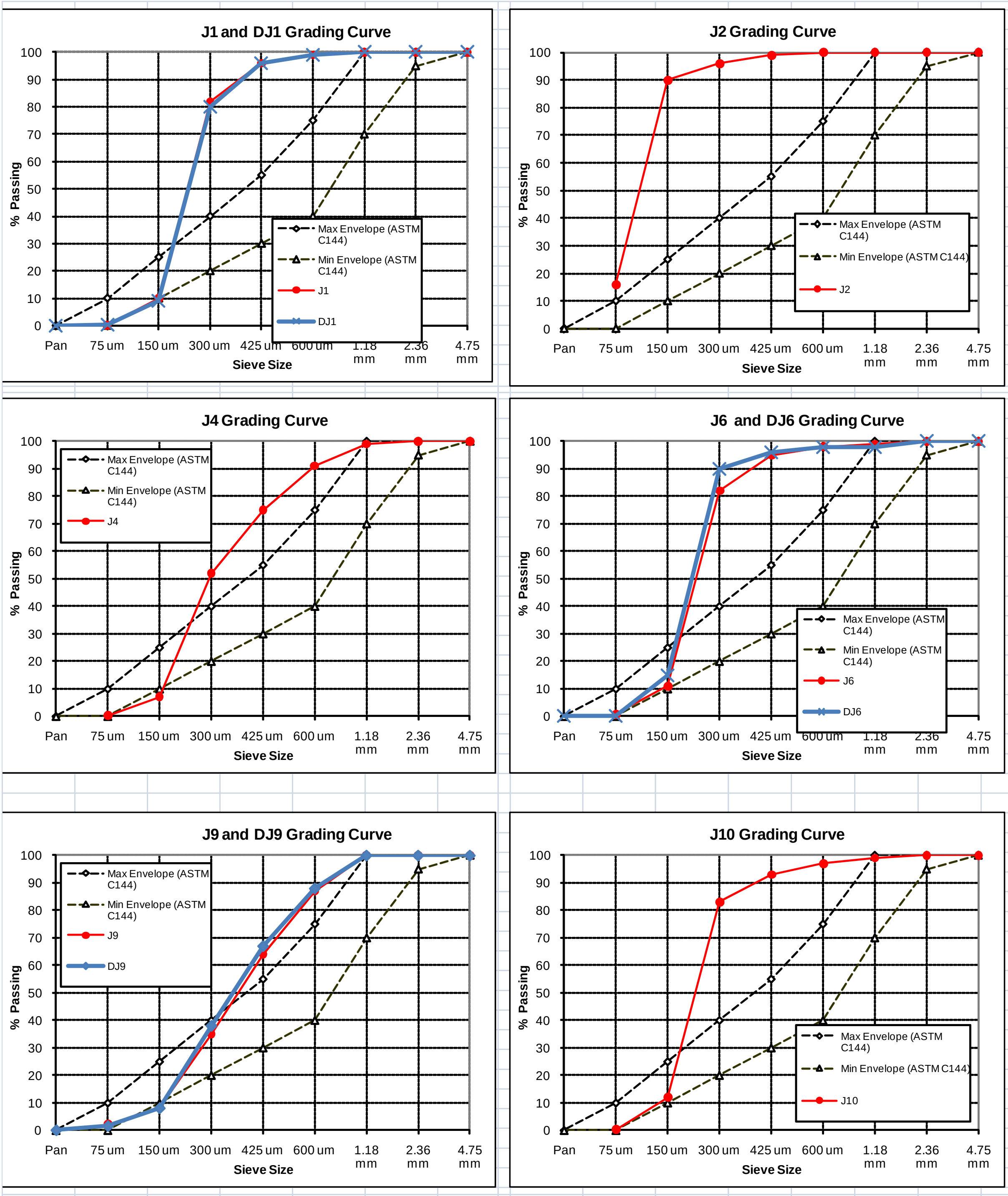


Figure 6. Sand gradings for J1 to J10

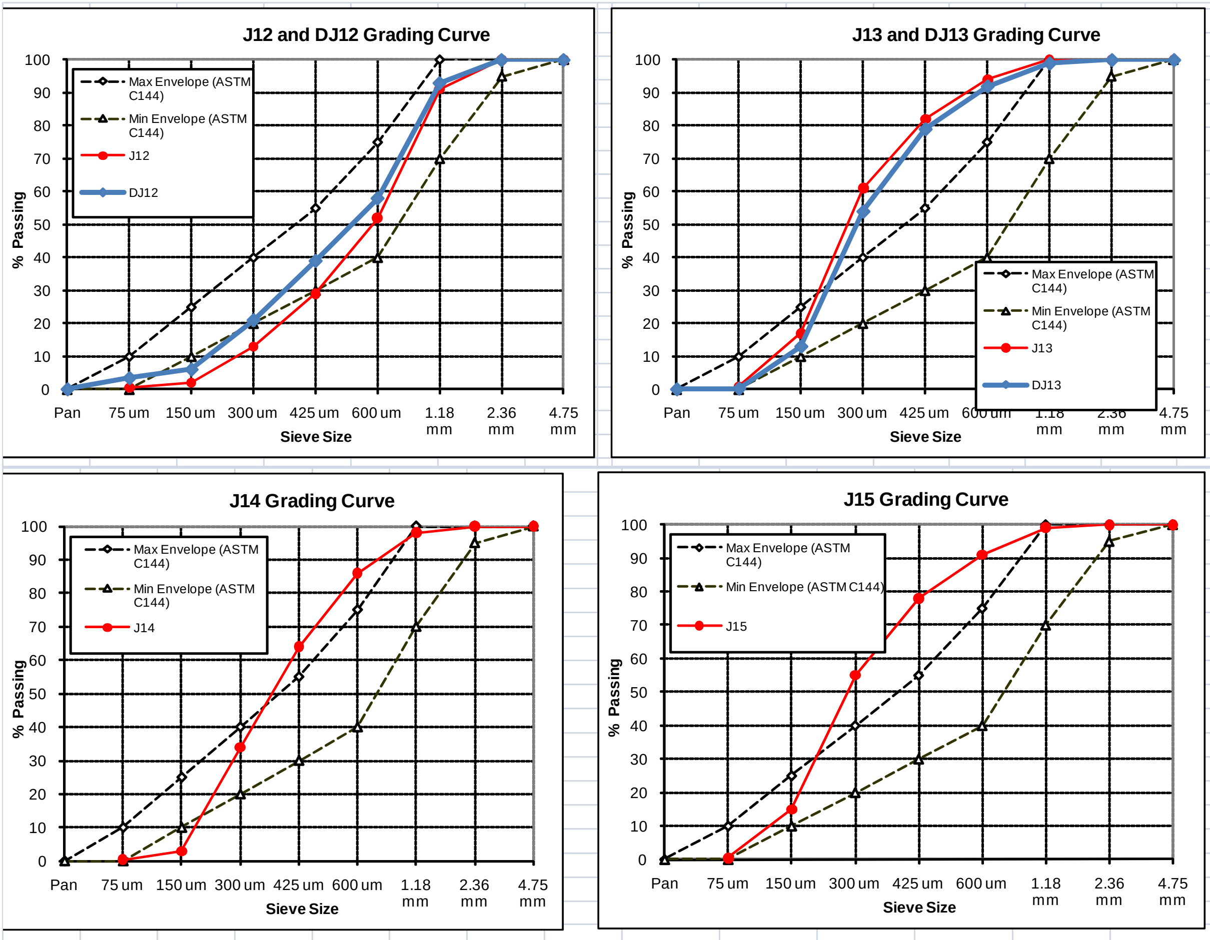


Figure 7. Sand gradings for samples taken from sites J12 to J15

Table 5. Mortar strengths measured in the Wellington region

	Couplet bond strength (kPa)		Compressive strength (MPa)	
Label	**Ave**	**C.O.V.**	**Ave**	**C.O.V.**
MATS1	38	0.05	11.33	0.02
MATS2	344	0.20	8.33	0.02
MATS3	36	0.32	12.65	0.00
MATS4	451	0.37	14.42	0.04
MATS5	153	1.19	9.68	0.02
MATS6	43	0.73	3.91	0.04
MATS7	240	0.79	15.28	0.02
MATS8	24		4.69	0.02
MATS9	101	0.26	8.80	0.03
MATS10	101	0.57	13.35	0.07
MATS11	40	0.47	13.26	0.01
MATS12	82	0.49	6.89	0.03
Average	**138**	**0.49**	**10**	**0.03**
C.O.V.	**100%**		**37%**	

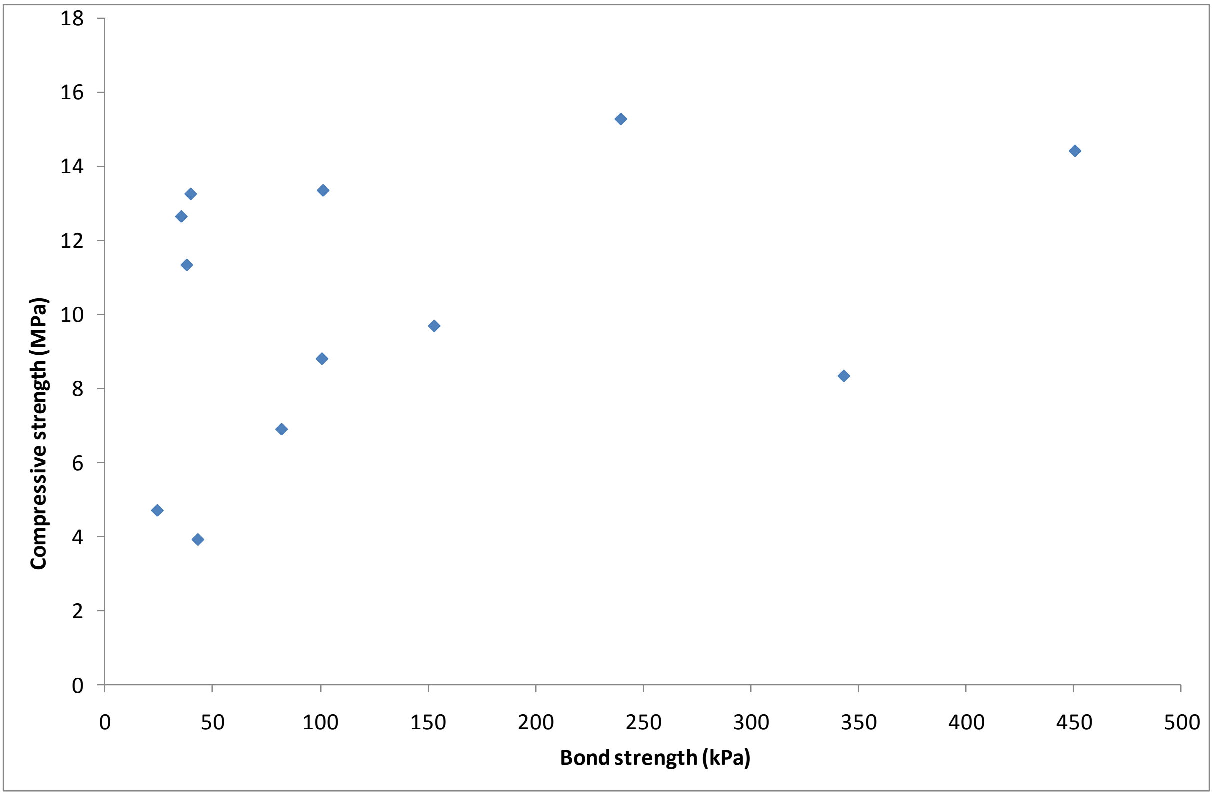


Figure 8. Relationship between mortar bond and compressive strengths from mortar taken from sites in the Wellington region

4. STAGE II. MORTAR ELEMENTAL TESTING AT BRANZ

4.1 Work performed

An experienced masonry tradesman mixed mortar to a consistency he deemed suitable for site use using a range of mortar mix designs. Variables considered were admixture type (as listed in Table 22), mix time, cement quantity and sand type. The effect of wetting the bricks and presence or absence of brick vertical core holes was also examined. Mortar bond and compressive strengths measured from the various test mixes were used to select the four mortar mixes used for test walls in Stage III of this project (see Section 5).

Mortars used for elemental testing were made in four periods in 2009 as follows:

- Day One (29 June). This used only a single sand type (Sand Y) using the same sand:cement ratio (4.5:1) (with one exception) and the only variable was the admixture type.
- Day Two (13 July). This used only a single sand type (Sand Y) except for some pre-mixed mortars tested, and the only variables in the mortar constituents were the admixture type and the sand:cement ratio.
- Day Three (23 November). The mortars used were mainly chosen to examine the effect of sand type on the mortar mixes and the same sand:cement ratio (4.5:1) (with one exception) was used.

- Day Four (16 December to 23 December 2010). These were the mortars used for constructing the test walls which were later tested on the shake table. The elemental tests on this mortar were mainly to confirm consistency with the results of the elemental testing on the other days. Also, a few of the mortar mixes from earlier tests were repeated to verify or otherwise some of the previous results the writer had found surprising.

The mortars for Days One to Three were made in the Concrete Laboratory whereas the mortars for Day Four were made in the Structures Laboratory. The dates on which particular mortar mixes were made are noted in the tables in this report.

Test specimens made with the various mortars were:

1. Brick couplet test specimens to measure the bond strength of brick-to-brick mortared joints under flexural loading.
2. Brick tension test specimens to measure the bond strength of brick-to-brick mortared joints under direct tension loading.
3. Brick tie pullout test specimens to measure the force required to pull a brick tie from two bricks mortared together.
4. Mortar cylinders to measure mortar compressive (crushing) strength.

Compression test specimens were stripped at three days and transferred to a 100% humidity fog room and tested at 28 days. The other specimens were stored in the same room in which they were made and tested at age seven days as stipulated in NZS 4210 for bond strength. The exception was that some specimens were tested at the time of the wall shake table tests as identified in this report.

4.2 Products used

The Monier Bricks brand clay bricks used had dimensions 230 mm long x 76 high x 70 mm wide. When assembled using the standard 10 mm of mortar between bricks, the veneer weighs approximately 130 kg/m^2.

The bricks used had five vertical holes, of cross-section 32 x 23 mm, for the full brick depth which partially filled with mortar during laying. In some bond couplet test specimens these holes were prefilled to help determine the influence the mortar “dowels” (i.e. mortar which penetrated the holes during the normal mortar laying operation) had on the apparent mortar bond strength.

In tie test specimens, hot-dipped galvanised steel, 85 mm long, 70 series, Eagle brand brick ties were either dry-bedded onto the bricks or fully encapsulated within the mortar to measure the tie resistance to pullout. The ties were stated to be rated “heavy earthquake to NZS 3604:1999 and AS/NZS 2699.1”.

A variety of sands were used. For clarity, these are described with the test results for mortar made from the sands.

Admixtures used are listed in Table 22. This covers most of the common admixtures used in New Zealand. The decision was made not to publish the admixture name.

The mortar was mixed in an ordinary rotating barrel mixer shown in Figure 9. This is a common type of mixer used on New Zealand house building sites. The cement used was Type GP Golden Bay Premium Portland Cement.

Each concrete mixer load of mortar was assigned a Concrete Mix (CM) Number. A Test Group Label was assigned to mortar taken out of the mixer at the same time for each CM Number. Thus, mortar taken from CM Number having five minutes mixing had a different Test Group Label from that retained in the mixer and given a total of 25 minutes mixing, but of course they had the same CM Number.

There are several examples where the same mix ingredients were used on different days and each of these received different CM Numbers, and the test specimens were given different Test Group Labels.

The first two letters of the mortar Test Group Label defines the cement content based on Table 2.1 of NZS 4210 as described in Table 6. The exception is Test Group Label "MX", which is similar to M3 but uses a 4:1 ratio rather than 4.5:1 ratio. A Test Group Label was assigned to all specimens made after the same mixing time from the same CM Number irrespective of whether they were couplet, tension or tie specimens.

As an example, the first concrete mix load on Day One (CM = 1) had a batch taken off after five minutes mixing called M3B1. Twenty minutes later a second batch was taken off called M3B2. On Day Two the same ingredients were tested and were called M3B21 for five minutes mixing and M3B22 for 25 minutes mixing.

Table 6. Relationship between test group label and mortar cement content

First letters of Test Group Label	Sand:Cement ratio
M2	6:1
M3	4.5:1
M4	3:1
MX	4:1

Figure 9. Concrete mixer used for the BRANZ tests

4.3 Mortar couplet bond tests

Generally five couplet test specimens were made for each Test Group. However, to provide more confidence in the average result, 10 specimens were made for tests investigating the effect of tooling and for tests investigating the influence of different sands.

Couplet test specimens were made by the masonry tradesman by placing mortar on the bottom brick, placing the top brick and applying downward pressure, and then (for stiffer mortars only) lightly tapping the top brick with the back of his trowel twice. The excess mortar from the joint was stuck flush on all four sides and the specimen then moved less than 2 m to where it stayed for the next seven days until it was tested.

The tests at BRANZ were performed to AS 3700 (2001) Appendix D6 with some variations as described herein. The bottom brick was clamped in a vice between plywood packers (as shown in Figure 10) and a pipe clamp gripped the top brick. A thrust bar was used to manually apply a slowly increasing downward force on the end of the pipe clamp as shown in Figure 11. The applied force was measured by load cell. Peak load was usually achieved in approximately 10 seconds. The couplet bond strengths were calculated as described in AS 3700, and no account was taken of the vertical core holes in the bricks through which mortar “dowels” formed in the couplet test specimens.

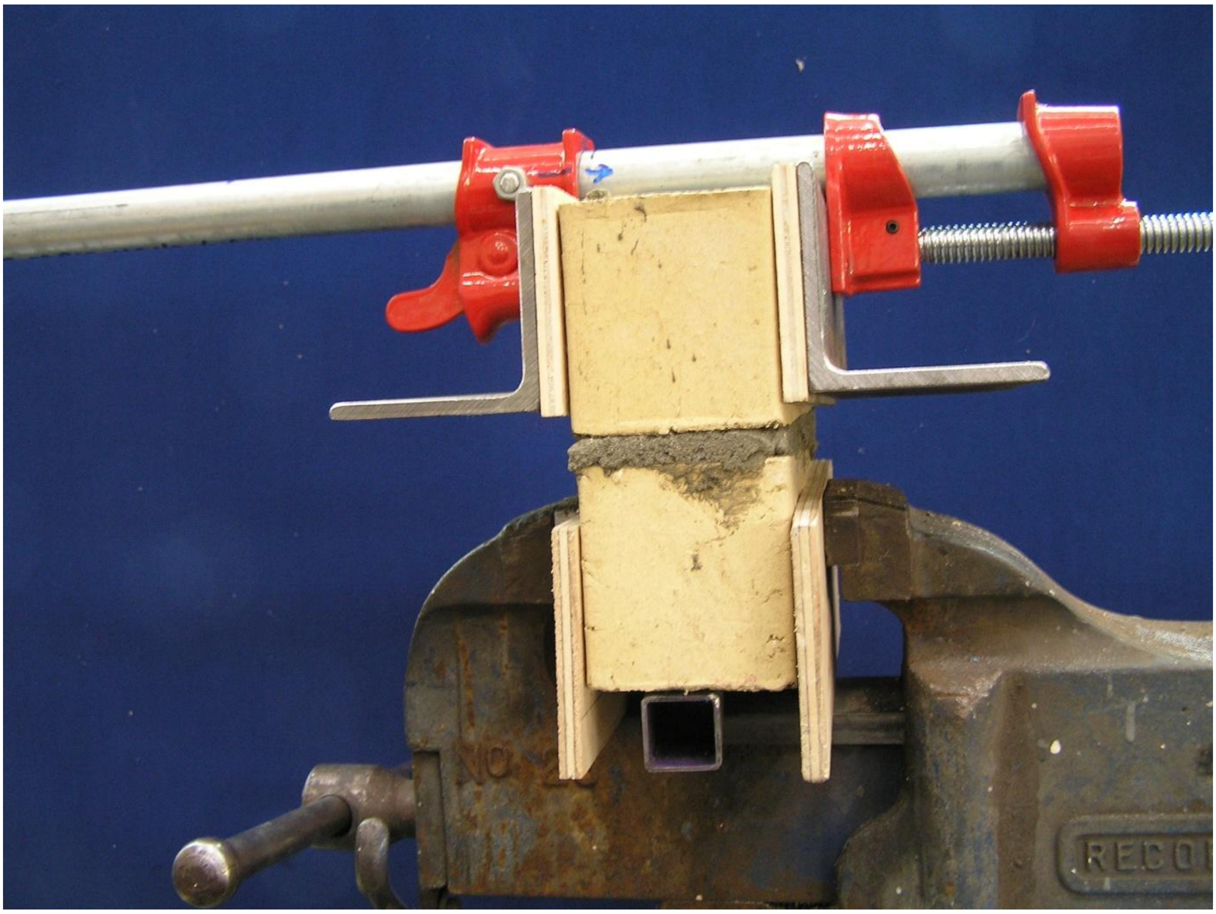


Figure 10. Couplet test – couplet specimen, timber packers, vice and pipe clamp

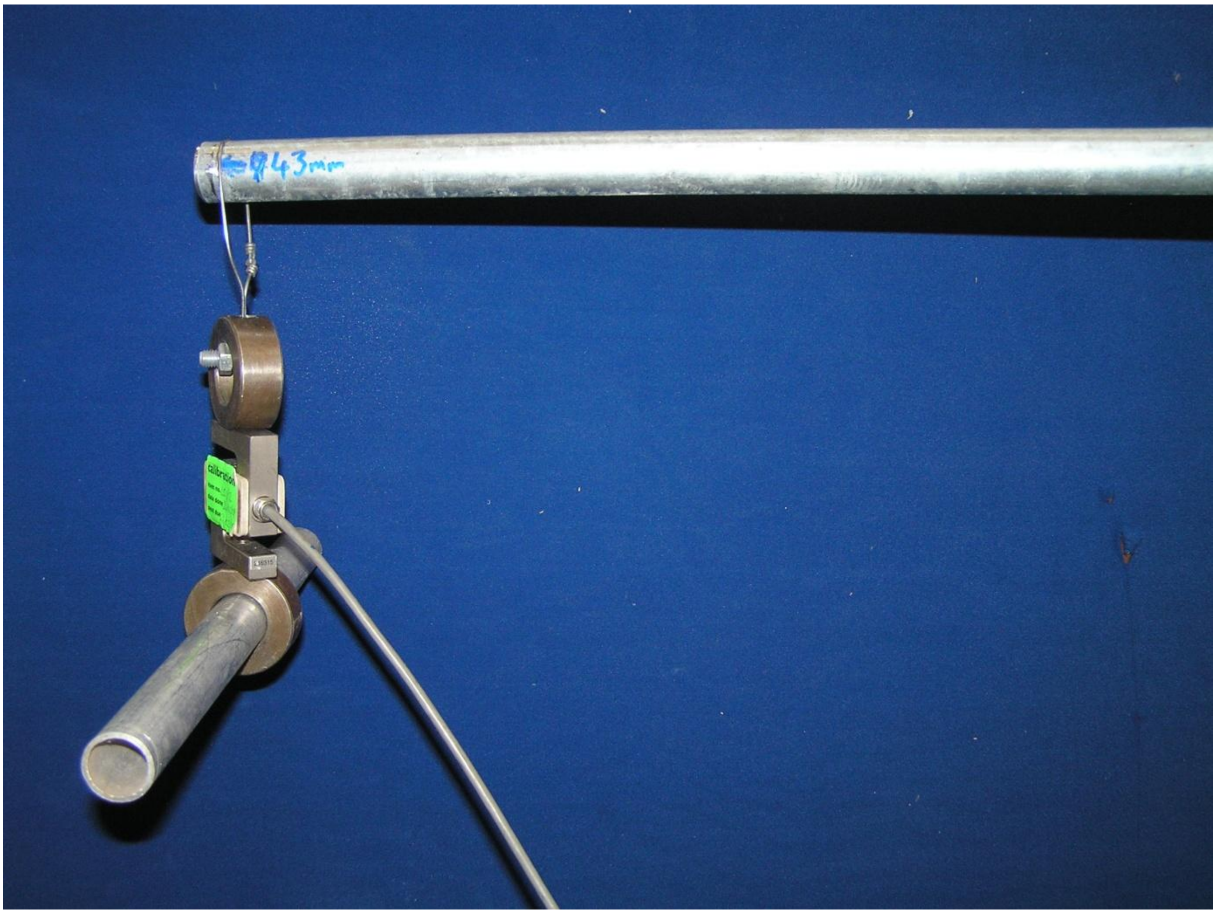


Figure 11. Couplet test – thrust bar and load cell on the end of the pipe

4.4 Mortar bond direct tension tests using brick cruciforms

Five cruciform test specimens were made for each Test Group for specimens made on Day One. However, as it was found there was similar variability for this test arrangement as for the couplet tests (and therefore little reason for doing this additional test), this test type was subsequently discontinued.

Cruciform test specimens were made by the masonry tradesman by placing mortar on the bottom brick, placing the top brick at right angles as shown in Figure 12 and then applying downward pressure, and then (for stiffer mortars only) lightly tapping the top brick with the back of his trowel twice. The excess mortar from the joint was stuck flush on all four sides and the specimen then moved less than 2 m to where it stayed for the next seven days until it was tested.

The ad hoc direct tension tests at BRANZ were performed as shown in Figure 13, which effectively prised the bricks apart. The top apparatus applied a compressive load and was connected to the test machine via a universal joint and pressed on the bottom bricks through ball joints so that any misalignment did not result in non-uniform loading. The applied force was measured by load cell. Peak load was usually achieved in approximately 10 seconds. The direct tension failure bond strengths were calculated from the applied load divided by the contact area.

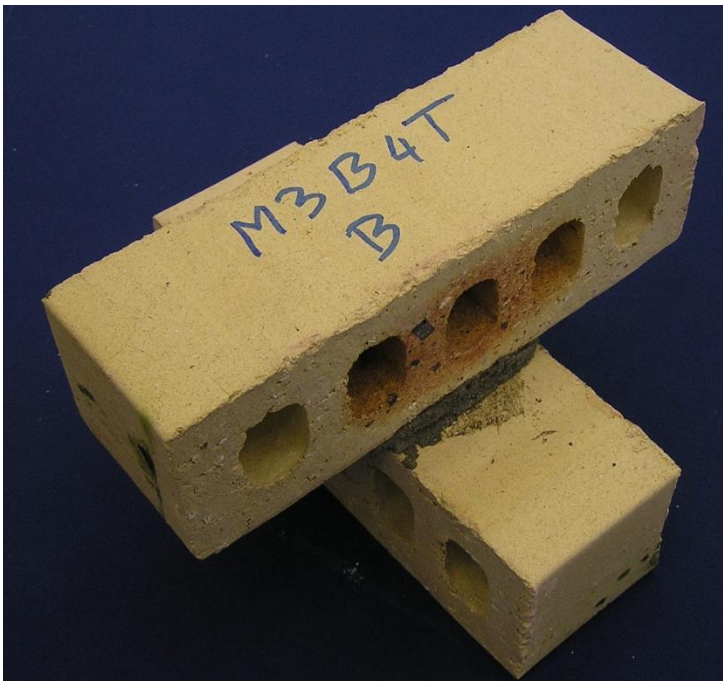


Figure 12. Cruciform test specimen

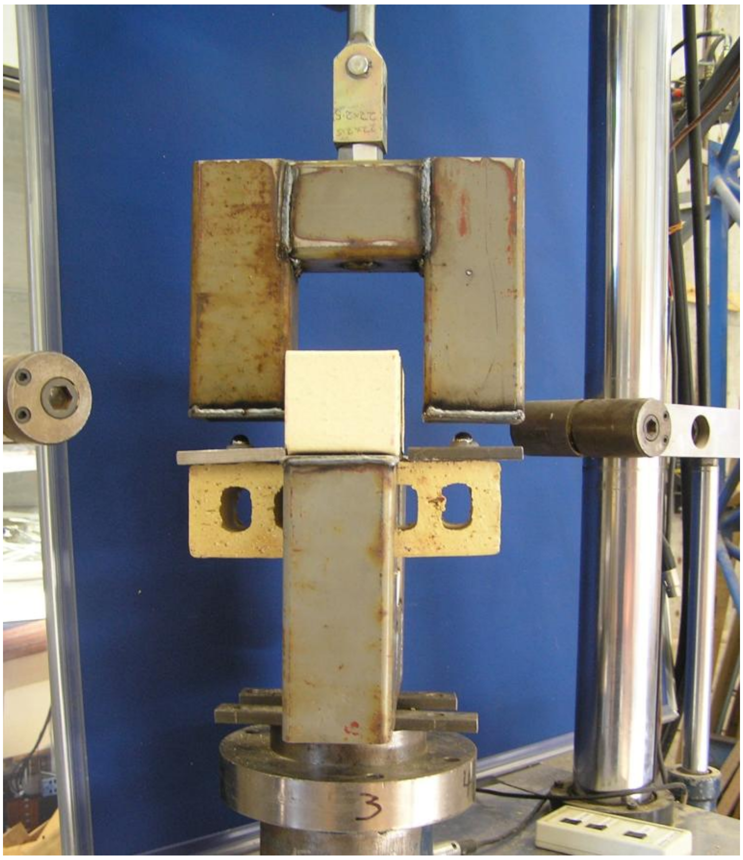


Figure 13. Cruciform direct tension test

4.5 Brick tie pullout test

Generally three tie pullout test specimens were made for each Test Group. However, five specimens in each group were made for tests investigating the influence of different sands in Day Three.

For the purpose of this tests the 'L-shaped' ties were flattened in a vice as shown in Figure 14. The tie pullout test specimens for dry-bedding were made by the masonry tradesman by placing the bottom brick onto the angle shown in Figure 14 and placing a flattened tie onto the brick in the same position used in normal bricklaying, except the tie passed through a slot in the angle. The bricklayer then placed a layer of mortar onto the bottom brick and then placed the top brick on this. Keeping the two bricks pressed against the vertical face of the angle, he applied downward pressure and (for stiffer mortars only) lightly tapped the top brick with the back of his trowel twice. The specimen was then moved less than 2 m to where it stayed for the next seven days until it was tested. Note that the steel angle was kept with the specimen to contain and protect the specimen until the test was complete. The mortar from the joint made uniform contact with the steel angle, as can be seen by observing the smooth surface in Figure 15.

These ad-hoc tie pullout tests were performed as shown in Figure 16. Tension was gradually applied by tightening the shaft with the ring spanner. The applied force was measured by load cell. Peak load was usually achieved in approximately 20 seconds. In approximately 70% of the cases the tie pulled out of the mortar without disrupting the brick-brick bond and in the remaining cases the bricks separated. Actual tie embedment depth in the mortar was measured after each test.

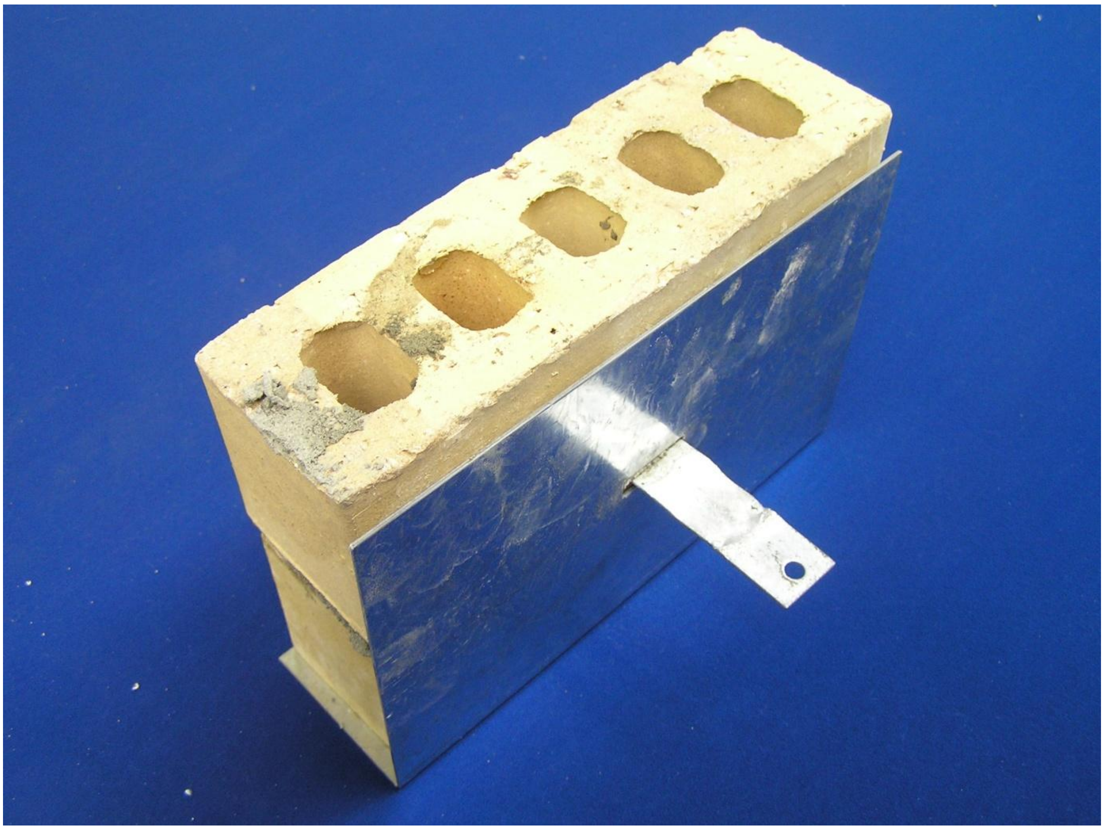


Figure 14. Tie test specimen

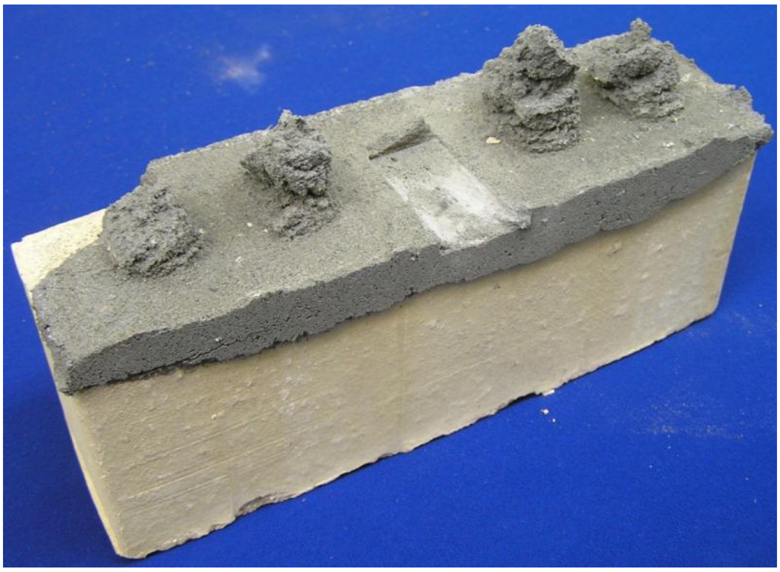


Figure 15. Typical failure surface in the mortar after the tie test

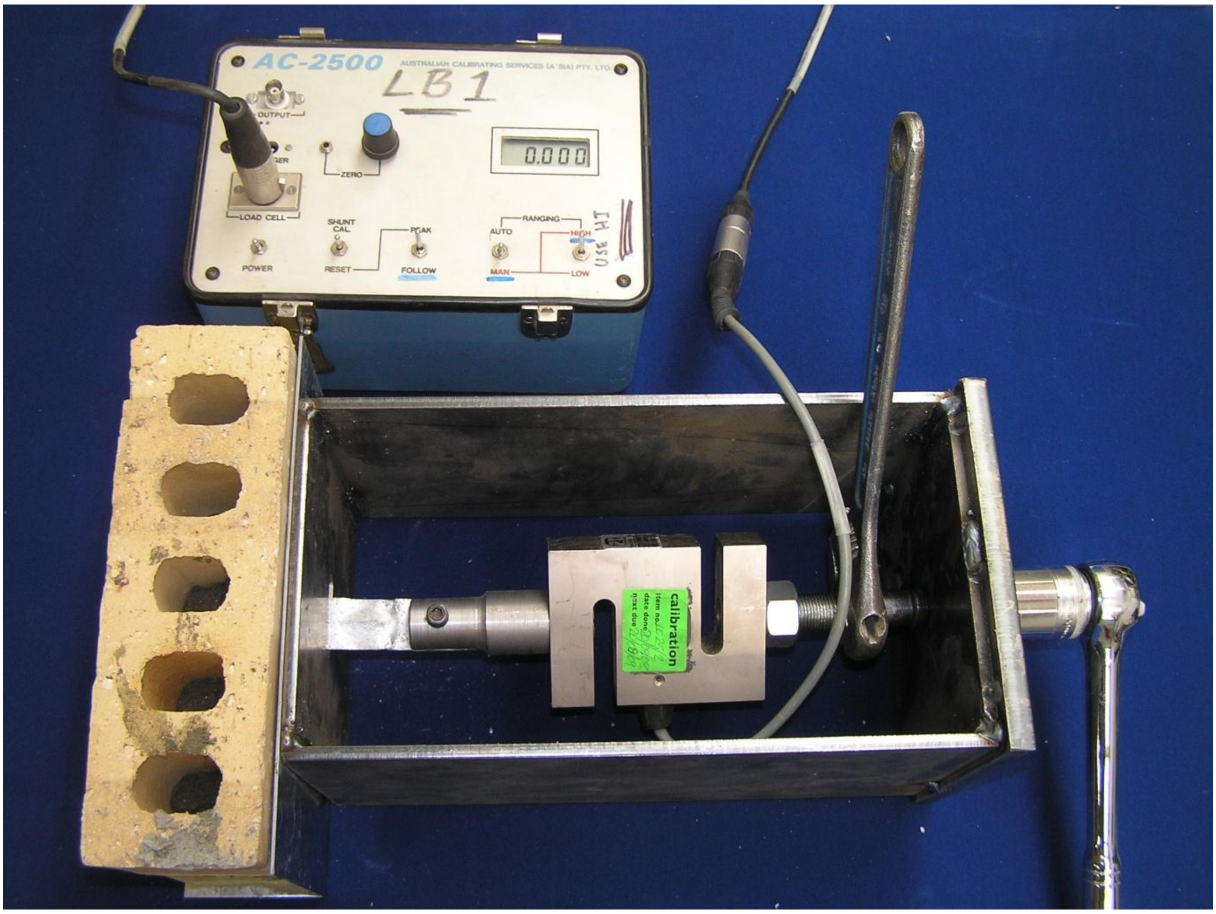


Figure 16. Tie pullout test

4.6 Test results on elemental specimens using Sand Y

4.6.1 Properties of Sand Y used

NZS 3103 (SNZ 1991) specifies the following limits for the mortar sand:

1. The percentage oversize material in the sand shall not exceed 1% by mass.
2. The flow limits shall be in a stipulated range (which for a voids content of 46.5% is between 22 and 28 seconds).
3. The voids content shall not exceed 48%.
4. The minimum value of sand equivalence is 60.

Apart from pre-mixed mortar, the mortar made on Day One and Day Two used sand from a plant in Wellington and is referred to as Sand Y. The sand, like most being used in New Zealand, did not comply with NZS 3103. In particular:

1. The measured percentage oversize was 4%.
2. The voids content was 46.5% (which does comply) but the flow time was only 20.6 seconds.
3. The sand equivalence was 48.
4. The grading curves were outside the ASTM C144 (2004) limits shown in Figure 17.
5. The sand had a fineness modulus of 1.56.

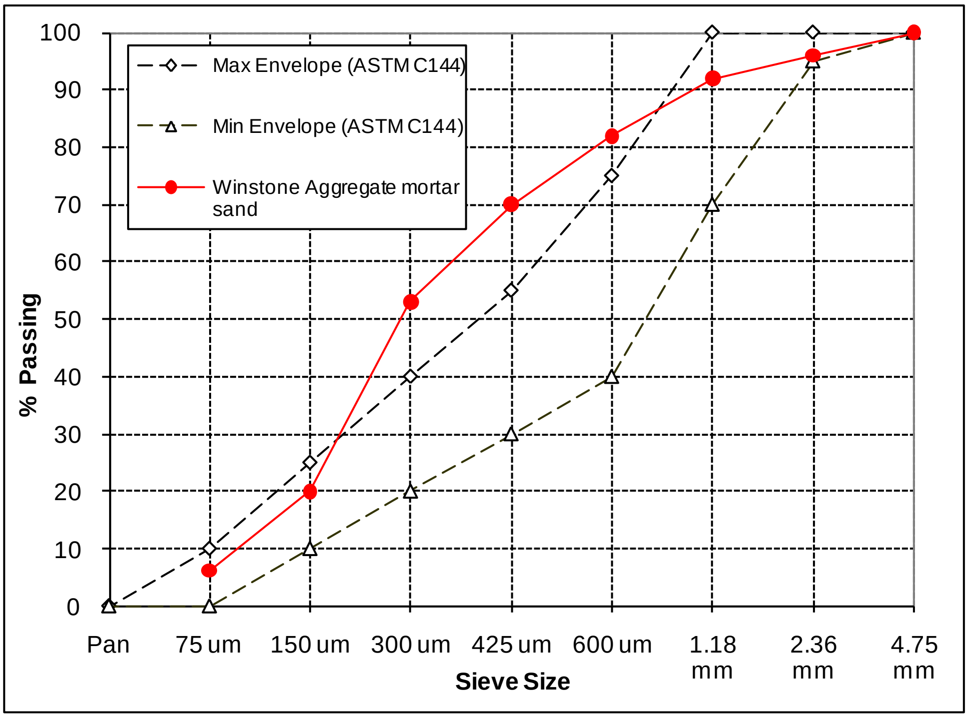


Figure 17. Grading curve for Sand Y used in mortar made on 29 June and 13 July 2009

4.6.2 Test results from samples made on Day One and Day Two

Table 7 summarises the test results from the 10 different concrete mixes (CM Numbers) of mortar which had five minutes mixing. The quantity of admixtures used in all the 10 mixes complied with the manufacturer's instructions. Half the mortar was retained in the concrete mixer and given an extra 20 minutes mixing for the first eight of these mixes. Table 8 summarises the results for these eight mixes.

Seven mixes were also given three times the manufacturer's recommended admixture dosage and then mixed for five minutes. Table 9 summarises the results for these seven mixes. The tradesman was cautious adding water to these mixes and the flow measured was relatively low. The writer's observation was that these mixes appeared to be at least equally fluid as other mixes, but there seemed to be a stickiness which resisted spread when placed on the flow table.

The mortar flow was measured for all mortar mixes, but sometimes there was insufficient time available to measure the air content. In these instances the air content was calculated from the 28 day mortar density, as discussed below and as illustrated in Figure 22, with the values so determined being shown in bold in Table 7 to Table 9.

To check the repeatability of the test results for mixes made on Day One and Day Two, two mixes were replicated with both the original and replicate tests shown shaded in the tables. The repeat tests showed significant variability, particularly for couplet bond strength, which is expected to be largely due to the different "flow" of mortar made the second day.

Tensile bond tests were performed because they were expected to give the average strength of the mortar bond and were less dependent on the flexural tensile strength at the outside edge of the mortar joint. A lower C.o.V. was expected. However, the results in the tables did not bear this out and so the tensile samples were only made on Day One.

The measured tie bond and compressive strengths showed less than half the C.o.V. of the couplet bond strengths.

Table 10 gives the ratio of measured mortar properties for mortar mixed for five minutes to that mixed for 25 minutes. On average the air content had increased from 12% to 26% and the flow from 98% to 108%. Probably because of this increased flow, the 20 minutes extra mixing gave increased the couplet bond strength. However, the tie bond strength and the compressive strength both reduced.

Table 11 gives the ratio of measured mortar properties for mortar mixed for five minutes with the recommended dose of admixtures to that with three times the recommended dose. In each case the mortar was mixed for five minutes. Possibly because the masonry tradesman was cautious with this excessive dosing, and consequently reluctant to add water to the mix, the average flow reduced by 9% and the average air content only increased by 8%. However, the mixtures with three times the admixtures did look very fluid (runny) and the relatively low flows measured came out as a surprise. The average reduction in couplet and tie bond strength was large and hence it is concluded that excessive overdosing results in inferior mortar properties.

Table 12 presents results for standard mixes from Table 2.1 of NZS 4210 when the recommended quantity of lime is used. Thus, both lime and cement content are inter-related variables, with smaller quantities of lime being used when the cement content is increased. Increasing the cement content (and reducing the lime content) increased the compressive strength significantly and the tie bond strength to some degree.

However, it had little effect on the couplet strength. The highest strength was for sample M3B1 where an intermediate lime and cement content was used.

Table 13 summarises results where cement content is the only variable and the admixture is No. 2 from Table 22. Increasing the cement content again increased the compressive strength significantly but had no trend influence on the tie bond strength. The couplet bond strength was lowest with the maximum cement content.

In Table 12 the admixture is lime whereas in Table 13 Admixture 2 was used (see Table 22). Comparing these two tables it is concluded that the compressive strength was lower when lime rather than the admixture was used but the tie bond strength was similar.

Because the results for Table 12 and Table 13 did not show an increase in mortar bond strength with an increase in cement content, as many engineers would expect to happen, the tests were repeated on Day Four but using 10 mortar bond samples per Test Group Label and three brick tie samples. The corresponding results in Table 14 and Table 15, respectively, largely confirm the conclusion that bond strength has little correlation with cement content. Figure 18 plots the couplet bond strength versus flow from Table 12 to Table 15. Bond strength increased with flow. As the flows were greater on Day Four this explains why greater bond strengths were obtained on Day Four compared with Day One and Day Two.

Wetting the bricks by dipping them into water for five seconds approximately five minutes before the couplet was made increased the average couplet mortar bond strength by 48% as shown in Table 16. The weather at the time was cold (approximately 8ºC) and overcast and a greater increase from wetting may occur in dry hot weather.

Approximately two weeks before the test specimens were made, the brick vertical core holes in some specimens were filled with mortar by placing the bricks on plastic and filling from the top. Such bricks are referred to as “filled bricks”. Couplets made with these “filled bricks” (using the side which had been against the plastic during filling at the interface of the bricks in the couplet) gave an average strength of 392 kPa compared with the 253 kPa for corresponding couplets made with cored bricks as shown in Table 17. It was concluded that the dowel action from mortar penetrating the core holes did not enhance the couplet bond strength determined from the test.

The ratio of strength for full tie embedment to those ties with dry embedment is given in Table 18. On average, the full tie embedment samples were 48% stronger than the samples using dry embedment. All corresponding specimens were made with the same mortar mix.

Cardboard cylinders were used to make most of the concrete compression cylinders in the laboratory and all of those used by the Auckland collection (see Section 3.3). To ensure that this did not unduly affect test results, samples were also taken using 100 x 50 mm steel cylinders. Table 19 shows this premise to be true for all samples, except Test Group Label M4C1 where the compression strength measured using steel cylinders was significantly stronger. The reason for the difference with this particular mix is not known.

The mortar properties measured with three different pre-mixed dry bagged mortars is shown in Table 20. The mortar pre-mix number given in Table 20 can be used to find the mortar type from Table 22. Although the average compressive strength of the pre-mixed mortar (25.4 MPa) was greater than the corresponding average from the other mixes in Table 7 (12.5 MPa), the average brick couplet mortar bond strengths (265 kPa as against 278 kPa) and the average tie bond strength were similar (41 N/mm as against 40 N/mm).

The effect of tooling joints on mortar bond strength is shown in Table 21. The weatherstruck and ironed concave finishes gave greatest bond strengths. Raking reduced the bond strength. In this test series tooling the raked joint did not increase the measured bond strength.

Figure 19 plots the relationship between flow and couplet bond strength for all mortars that had not been provided pre-mixed. This shows that increased flow results in increased couplet bond strength. This was independent of whether there was five or 25 minutes mixing or whether three times the recommended admixture was used, although the latter did give slightly lower couplet bond strengths for corresponding flows.

Figure 20 shows that there was also a moderate relationship between couplet and direct tension bond strength, with the best fit line indicating that the couplet bond strength was 43% greater.

Figure 21 shows that there is little correlation between couplet bond strength and mortar compression strength.

Figure 22 shows that there was also a good relationship between measured air content and mortar density. The best fit curves were used to estimate air content of mortar where it was not measured directly.

Figure 23 shows that there may be a relationship between couplet bond strength and air content with the bond strength reducing as air content increases. This strength reduction relationship is more distinct for mortar compressive strength as shown in Figure 24.

The average tie pullout strengths increased with increased couplet bond strength (Figure 25), although this relationship was not strong and appeared to depend on time in the mixer. A longer time in the mixer increased the brick bond strength but slightly reduced the tie bond strength as discussed above.

There was a trend for the tie bond strength to increase with mortar compressive strength (Figure 26), although the relationship was weak.

There was also a trend for the tie bond strength to increase with flow (Figure 27), although the relationship was weak and showed much scatter.

Table 7. Samples made with Sand Y after five minutes mixing using specified levels of admixtures

(shading represents replicates made on different dates)

Mortar CM number	Day made	Test Group Label	Admixture No	Admixture Dosage per 40 kg bag of cement	Flow %	Air content	Couplet bond strength (kPa) Ave.	Couplet bond strength (kPa) C.O.V.	Tension bond strength (kPa) Ave.	Tension bond strength (kPa) C.O.V.	Tie bond strength (N/mm) Ave.	Tie bond strength (N/mm) C.O.V.	Compressive strength (MPa) Ave.	Compressive strength (MPa) C.O.V.	Density kN/m^3	Ratios couplet: tension	Ratios couplet: Tie bond
1	One	M3B1	1	0.5:1 by volume	98	6%	305	0.11	254	0.23	49	0.12	18.0	0.028	2070	1.20	6.24
2	One	M3B3	2	100 ml	101	18%	330	0.20	267	0.27	48	0.08	11.5	0.130	1893	1.24	6.93
3	Two	M3B21	2	100 ml	87	20%	257	0.18			35	0.20	10.0	0.050	1823		7.27
4	One	M3B5	3	25 gms	103	**12%**	303	0.29	208	0.43	40	0.10	11.3	0.243	1963	1.46	7.58
5	One	M3B8	4	60 ml	96	15%	384	0.28	352	0.16	52	0.03	15.0	0.220	1895	1.09	7.37
6	Two	M3B18	4	60 ml	94	**16%**	204	0.23			31	0.09	15.5	0.116	1890		6.68
7	One	M3B11	5	30 gms	109	10%	396	0.25	307	0.17	36	0.07	12.8	0.045	1980	1.29	10.87
8	Two	M3B15	6	30 gms	93	**14%**	212	0.18			39	0.03	11.0	0.045	1930		5.45
9	Two	MX	7	658 gms	94	**30%**	123	0.17			26	0.22	8.7	0.120	1683		4.67
10	Two	M3Tool	2	100 ml	94	20%							11.0	0.120	1830		
				Averages	**97**	**16%**	**279**	**0.21**	**278**	**0.25**	**40**	**0.10**	**12.5**	**0.11**	**1896**	**1.25**	**7.01**

Table 8. Samples made with Sand Y after 25 minutes mixing using specified levels of admixtures

Mortar CM number	Day made	Test Group Label	Admixture No	Admixture Dosage per 40 kg bag of cement	Flow %	Air content	Couplet bond strength (kPa) Ave.	Couplet bond strength (kPa) C.O.V.	Tension bond strength (kPa) Ave.	Tension bond strength (kPa) C.O.V.	Tie bond strength (N/mm) Ave.	Tie bond strength (N/mm) C.O.V.	Compressive strength (MPa) Ave.	Compressive strength (MPa) C.O.V.	Density kN/m^3	Ratios couplet: tension	Ratios couplet: Tie bond
1	One	M3B2	1	0.5:1 by volume	100	**5%**	436	0.28	226	0.20	56	0.20	16.0	0.031	2063	1.93	7.76
2	One	M3B4	2	100 ml	109	33%	282	0.06	178	0.13	25	0.08	7.8	0.037	1627	1.59	11.36
3	Two	M3B22	2	100 ml	98	**32%**	295	0.16			28	0.13	7.0	0.000	1647		10.36
4	One	M3B6	3	25 gms	120	**30%**	439	0.16	186	0.14	31	0.16	7.0	0.124	1670	2.36	14.24
5	One	M3B9	4	60 ml	104	24%	363	0.12	321	0.24	39	0.05	9.5	0.053	1773	1.13	9.23
6	Two	M3B19	4	60 ml	105	**25%**	358	0.30			38	0.12	13.0	0.077	1760		9.49
7	One	M3B12	5	30 gms	134	**20%**	618	0.22	306	0.21	42	0.07	9.2	0.031	1833	2.02	14.77
8	Two	M3B16	6	30 gms	104	**37%**	328	0.38			21	0.09	5.3	0.054	1573		15.94
				Averages	**109**	**26%**	**390**	**0.21**	**243**	**0.18**	**35**	**0.11**	**9.4**	**0.05**	**1743**	**1.81**	**11.64**

Table 9. Samples made with Sand Y after five minutes mixing using 3 x specified levels of admixtures

Mortar CM number	Day made	Test Group Label	Admixture		Flow %	Air content	Couplet bond strength (kPa)		Tension bond strength (kPa)		Tie bond strength (N/mm)		Compressive strength (MPa)		Density kN/m^3	Ratios	
			No	Dosage per 40 kg bag of cement			Ave.	C.O.V.	Ave.	C.O.V.	Ave.	C.O.V.	Ave.	C.O.V.		couplet: tension	couplet: Tie bond
11	One	M3B14	2	300 mL	66	29%	107	0.69	110	0.40	20	0.14	9.7	0.030	1723	0.97	5.22
12	Two	M3B23	2	300 mL	86	**27%**	124	0.27			18.28	0.17	9.8	0.029	1723		6.81
13	One	M3B7	3	75 gms	108	**18%**	214	0.15	129	0.34	31	0.13	8.8	0.131	1863	1.66	6.85
14	One	M3B10	4	180 mL	81	**22%**	166	0.33	104	0.29	32	0.07	9.5	0.091	1803	1.59	5.21
15	Two	M3B20	4	180 mL	97	**25%**	167	0.16			16.15	0.03	11.5	0.115	1760		10.36
16	One	M3B13	5	90 gms	101	**17%**	314	0.33	108	0.45	32	0.13	9.3	0.164	1873	2.91	9.87
17	Two	M3B17	6	90 gms	82	**24%**	95	0.23			24	0.05	10.0	0.050	1767		4.05
				Averages	**89**	**23%**	**170**	**0.31**	**113**	**0.37**	**25**	**0.10**	**9.8**	**0.09**	**1787**	**1.78**	**6.91**

Table 10. Ratio of strengths of samples made with Sand Y after five minutes mixing to those made after 25 minutes mixing

Mortar CM number	Day made	Test Group1 Label	Test Group2 Label	Admixture		Flow (%)		Air content		Couplet bond strength (kPa)	Tension bond strength (kPa)	Tie bond strength (N/mm)	Compressive strength (MPa)
				No	Dosage per 40 kg bag of cement	At 5 min	At 25 min	At 5 min	At 25 min	Ave.	Ave.	Ave.	Ave.
1	One	M3B1	M3B2	1	0.5:1 by volume	98	100	6%	5%	0.70	1.13	0.87	1.13
2	One	M3B3	M3B4	2	100 ml	101	109	18%	33%	1.17	1.50	1.92	1.47
3	Two	M3B21	M3B22	2	100 ml	87	98	20%	32%	0.87		1.24	1.43
4	One	M3B5	M3B6	3	25 gms	103	120	12%	30%	0.69	1.12	1.29	1.61
5	One	M3B8	M3B9	4	60 ml	96	104	15%	24%	1.06	1.10	1.32	1.58
6	Two	M3B18	M3B19	4	60 ml	94	105	16%	25%	0.57		0.81	1.19
7	One	M3B11	M3B12	5	30 gms	109	134	10%	20%	0.64	1.00	0.87	1.39
8	Two	M3B15	M3B16	6	30 gms	93	104	14%	37%	0.65		1.89	2.08
					Average	**98**	**109**	**14%**	**26%**	**0.79**	**1.17**	**1.28**	**1.49**

Table 11. Ratio of strengths of samples made with Sand Y after five minutes mixing to those made with 3 x admixture

Day made	Test Group1 Label	Test Group2 Label	Admixture No	Admixture Standard dose per 40 kg bag of cement	Flow (%) 1 x dose	Flow (%) 3 x dose	Air content 1 x dose	Air content 3 x dose	Couplet bond strength (kPa) Ave.	Tension bond strength (kPa) Ave.	Tie bond strength (N/mm) Ave.	Compressive strength (MPa) Ave.
One	M3B3	M3B14	2	100 ml	101	66	18%	29%	3.51		3.08	1.63
Two	M3B21	M3B23	2	100 ml	87	86	20%	27%	1.32	1.38	0.80	0.89
One	M3B5	M3B7	3	25 gms	103	108	12%	18%	1.78	0.00	0.90	0.74
One	M3B8	M3B10	4	60 ml	96	81	15%	22%	2.63		1.91	0.61
Two	M3B18	M3B20	4	60 ml	94	97	16%	25%	1.15	2.97	1.23	1.02
One	M3B11	M3B13	5	30 gms	109	101	10%	17%	3.76		1.60	1.30
Two	M3B15	M3B17	6	30 gms	93	82	14%	24%	3.64	2.72	1.69	0.94
				Average	**98**	**89**	**15%**	**23%**	**2.54**	**1.77**	**1.60**	**1.02**

Table 12. Samples made with Sand Y after five minutes mixing with cement as a variable and admixture is lime

Mortar CM number	Day made	Test Group Label	Sand to cement ratio	Admixture No	Admixture Dosage by volume Lime:cement ratio	Flow %	Air content	Couplet bond strength (kPa) Ave.	Couplet bond C.O.V.	Tension bond strength (kPa) Ave.	Tension bond C.O.V.	Tie bond strength (N/mm) Ave.	Tie bond C.O.V.	Compressive strength (MPa) Ave.	Compressive C.O.V.	Density kN/m^3	Ratios couplet: tension	Ratios couplet: Tie bond
18	Two	M4C2	3	1	0.25	101		226	0.52			52	0.09	31.7	0.01	2093		4.39
1	One	M3B1	4.5	1	0.5	98	6%	305	0.11	254	0.23	49	0.12	18.0	0.03	2070	1.20	6.24
19	Two	M2C4	6	1	1	107		269	0.26			38	0.18	8.3	0.07	2033		7.00
					Average	**102**		**267**	**0.30**	**254**	**0.23**	**46**	**0.13**	**19.3**	**0.04**	**2065**		

Table 13. Samples made with Sand Y after five minutes mixing with cement as a variable and Admixture is 2

Mortar CM number	Day made	Test Group Label	Sand to cement ratio	Admixture No	Admixture Dosage per 40 kg bag of cement	Flow %	Air content	Couplet bond strength (kPa) Ave.	Couplet bond C.O.V.	Tension bond strength (kPa) Ave.	Tension bond C.O.V.	Tie bond strength (N/mm) Ave.	Tie bond C.O.V.	Compressive strength (MPa) Ave.	Compressive C.O.V.	Density kN/m^3	Ratios couplet: tension	Ratios couplet: Tie bond
20	Two	M4C1	3	2	100 ml	88		153	0.20			42	0.29	18.3	0.08	1703		3.61
2	One	M3B3	4.5	2	100 ml	101	18%	330	0.20	267	0.27	48	0.08	11.5	0.13	1893	1.24	6.93
21	Two	M2C3	6	2	100 ml	90		344	0.16			37	0.26	6.0	0.00	1880		9.21
					Average	**93**		**276**	**0.19**	**267**	**0.27**	**42**	**0.21**	**11.9**	**0.07**	**1825**		

Table 14. Repeat tests: samples made with Sand Y after five minutes mixing with cement as a variable and admixture is lime

Original Test Group Label	Repeat Test Group Label	Sand to cement ratio	Admixture		Flow %	Couplet bond strength (kPa)		Tie bond strength (N/mm)		ratios couplet: Tie bond
			No	Dosage by volume Lime:cement ratio		Ave.	C.O.V.	Ave.	C.O.V.	
M4C2	M4C6	3	1	0.25	99	448	0.43	55	0.09	8.17
M3B1	M3B25	4.5	1	0.5	117	460	0.25	46	0.13	9.97
M2C4	M2C8	6	1	1	117	336	0.26	37	0.11	8.95
				Average	**111**	**415**	**0.31**	**46**	**0.11**	

Table 15. Repeat tests: samples made with Sand Y after five minutes mixing with cement as a variable and admixture is 2

Original Test Group Label	Repeat Test Group Label	Sand to cement ratio	Admixture		Flow %	Couplet bond strength (kPa)		Tie bond strength (N/mm)		ratios couplet: Tie bond
			No	Dosage per 40 kg bag of cement		Ave.	C.O.V.	Ave.	C.O.V.	
M4C1	M4C5	3	2	100 ml	103	324	0.29	34	0.33	9.57
M3B3	M3B24	4.5	2	100 ml	110	412	0.15	42	0.09	9.79
M2C3	M2C7	6	2	100 ml	120	608	0.26	41	0.13	14.95
				Average	**111**	**448**	**0.23**	**39**	**0.18**	

Table 16. Effect of wetting bricks using mortar with Sand Y

Day made	Label	Sand to cement ratio	Admixture No	Admixture Dosage per 40 kg bag of cement	Flow (mm)	Air content	Dry Couplet bond strength (kPa) Ave.	Dry Couplet bond strength (kPa) C.O.V.	Wet Couplet bond strength (kPa) Ave.	Wet Couplet bond strength (kPa) C.O.V.	Ratio of strengths Wet:Dry
Two	M3B21	4.5	2	100 ml	87	20%	257	0.18	326	0.14	1.27
Two	M3B18	4.5	4	60 ml	94	16%	204	0.23	384	0.16	1.88
Two	M3B15	4.5	6	30 gms	93	14%	212	0.18	269	0.28	1.27
One	M3B1	4.5	1	0.5:1 by volume	98	6%	305	0.11	479	0.21	1.57
Two	M4C1	3	2	100 ml			153	0.20	364	0.25	2.38
Two	M2C3	6	2	100 ml			344	0.16	357	0.24	1.04
Two	Premix	N/A	8	N/A			369	0.30	494	0.09	1.34
Two	Premix	N/A	9	N/A			138	0.72	395	0.30	2.86
Two	Premix	N/A	10	N/A			283	0.15	286	0.23	1.01
						Averages	**252**	**0.25**	**373**	**0.21**	
								Ratio of average wet:dry			**1.48**

Table 17. Effect of filling core holes using mortar with Sand Y

Mortar batch number	Day made	Test group label	Admixture No	Admixture Dosage per 40 kg bag of cement	Flow (mm)	Air content	Couplet bond strength (kPa) Cored bricks Ave.	Cored bricks C.O.V.	Filled bricks Ave.	Filled bricks C.O.V.	Ratio of strengths Filled:cored
1	One	M3B1	1	0.5:1 by volume	98	6%	305	0.11	453	0.25	1.48
8	Two	M3B15	6	30 gms	93		212	0.18	167	0.16	0.79
				Average	**96**	**6%**	**259**	**0.14**	**310**	**0.20**	**1.14**

Table 18. Effect of full tie embedment using mortar with Sand Y

Mortar batch number	Day made	Test group label	Admixture		Flow %	Air content	Tie bond strength (N/mm)				Ratio of strengths Filled: dry bedded
			No	Dosage per 40 kg bag of cement			Dry bedded		Full embedment		
							Ave.	C.O.V.	Ave.	C.O.V.	
1	One	M3B1	1	0.5:1 by volume	98	6%	49	0.12	65	0.08	1.33
8	Two	M3B15	6	30 gms	93	**14%**	39	0.03	36	0.38	0.93
20	Two	M4C1	2	100 mL	88		42	0.29	75	0.06	1.77
21	Two	M2C3	2	100 mL	90		37	0.26	72	0.11	1.92
6	Two	M3B18	4	100 mL	94	**16%**	31	0.09	44	0.26	1.45
				Average	**93**	**12%**	**40**	**0.16**	**58**	**0.18**	**1.48**

Table 19. Effect of using cardboard as against steel cylinders using mortar with Sand Y

Mortar batch number	Day made	Test group label	Admixture		Cylinder strengths for different moulds (MPa)				Ratio of strengths card./steel
			No	Dosage per 40 kg bag of cement	Cardboard		Steel		
					Ave.	C.O.V.	Ave.	C.O.V.	
4	One	M3B6	2	25 gms	7.00	0.12	7.30	0.04	1.04
22	Two	Premix	8		31.30	0.05	31.50	0.02	1.01
8	Two	M3B15	2	25 gms	11.00	0.05	11.00	0.08	1.00
20	Two	M4C1	2	25 gms	18.30	0.08	23.70	0.01	1.30
				Average	**16.90**	**0.07**	**18.38**	**0.04**	**1.09**

Table 20. Properties determined using pre-mixed mortars

Mortar CM number	Day made	Premix No	Flow %	Couplet bond strength (kPa)		Tie bond strength (N/mm)		Compressive strength (MPa)		Density kN/m^3	Ratios couplet: Tie bond
				Ave.	C.O.V.	Ave.	C.O.V.	Ave.	C.O.V.		
22	Two	8	96	369	0.30	49	0.42	0.0	8.333	2020	7.53
23	Two	9	85	138	0.72	37	0.10	0.0	6.000	1880	3.71
24	Two	10	57	283	0.15	37	0.23	0.0	9.833	1870	7.70
		Averages	**79**	**263**	**0.39**	**41**	**0.25**	**0.0**	**8.06**	**1923**	**6.31**

Table 21. Effect of tooling joints

	# specimens	Flow	air content	Notional bond strength (kPa)	C.o.V.
Ironed concave	10	94%	20%	276	0.26
Weatherstruck	10	94%	20%	304	0.30
Raked and tooled	10	94%	20%	214	0.24
Raked and brushed.	10	94%	20%	212	0.44
M3B3 struck flush	5	101%	18%	330	0.20
M3B21 struck flush	5	87%	20%	259	0.18

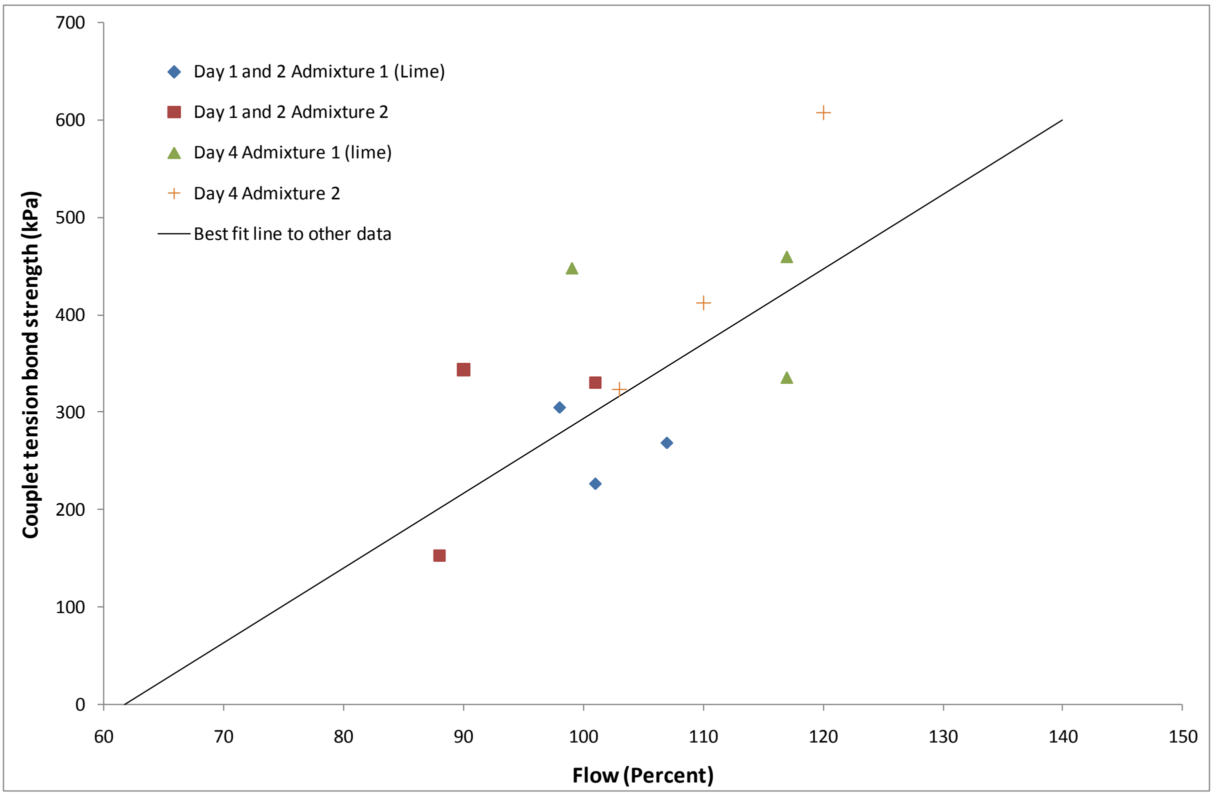


Figure 18. Relationship between seven day couplet bond strength and mortar flow when cement content varied

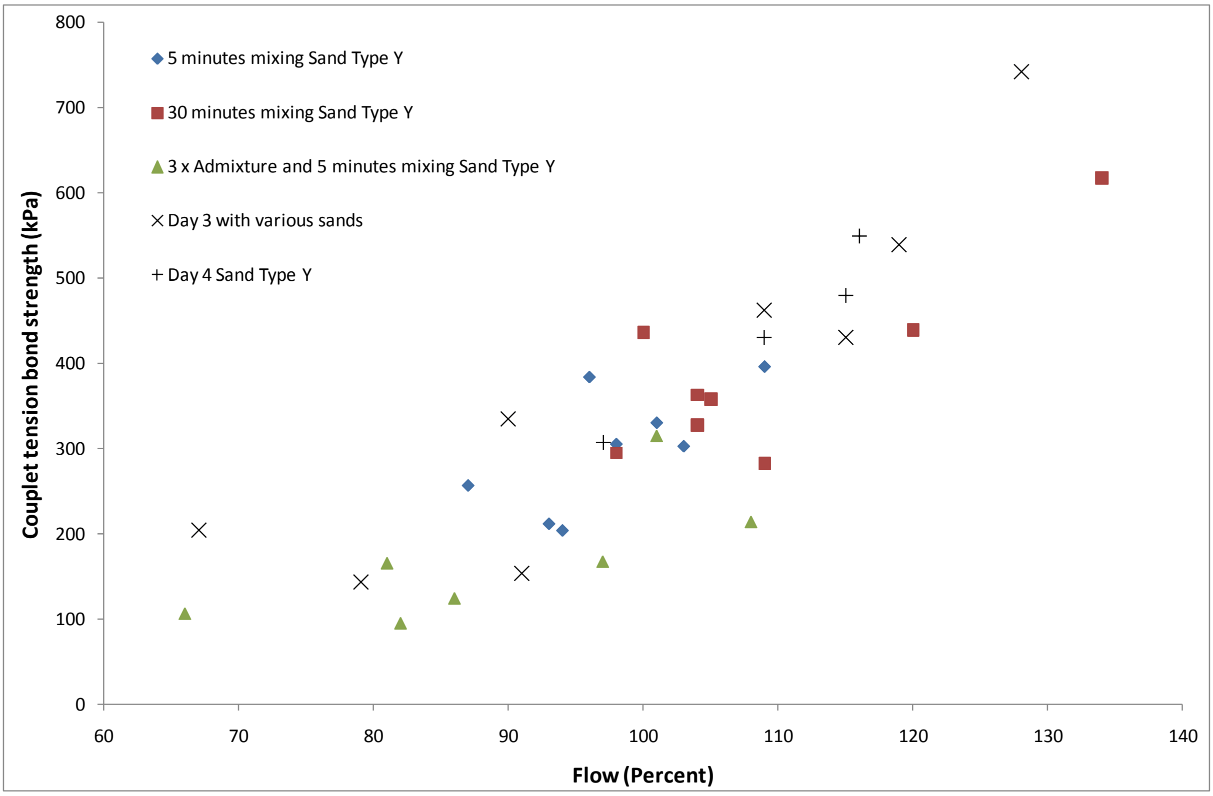


Figure 19. Relationship between seven day couplet bond strength and mortar flow for constant cement content

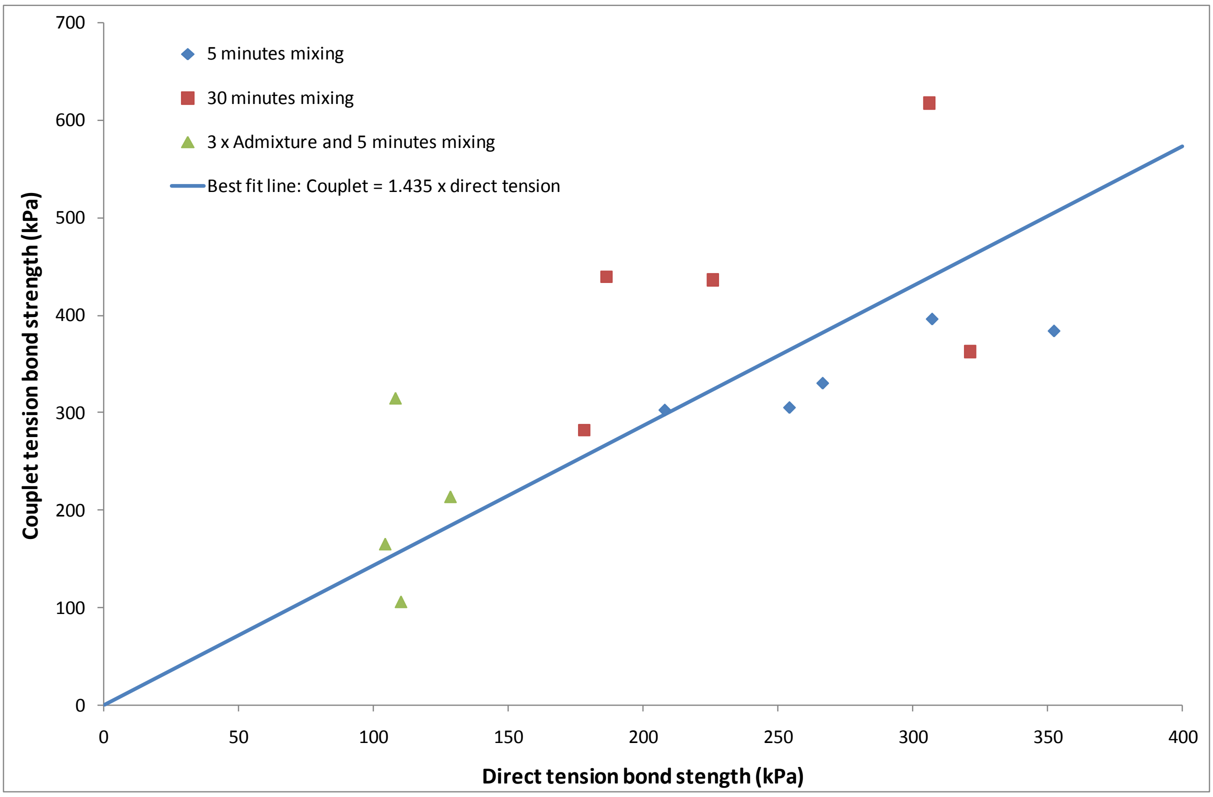


Figure 20. Relationship between couplet and direct tension bond strength

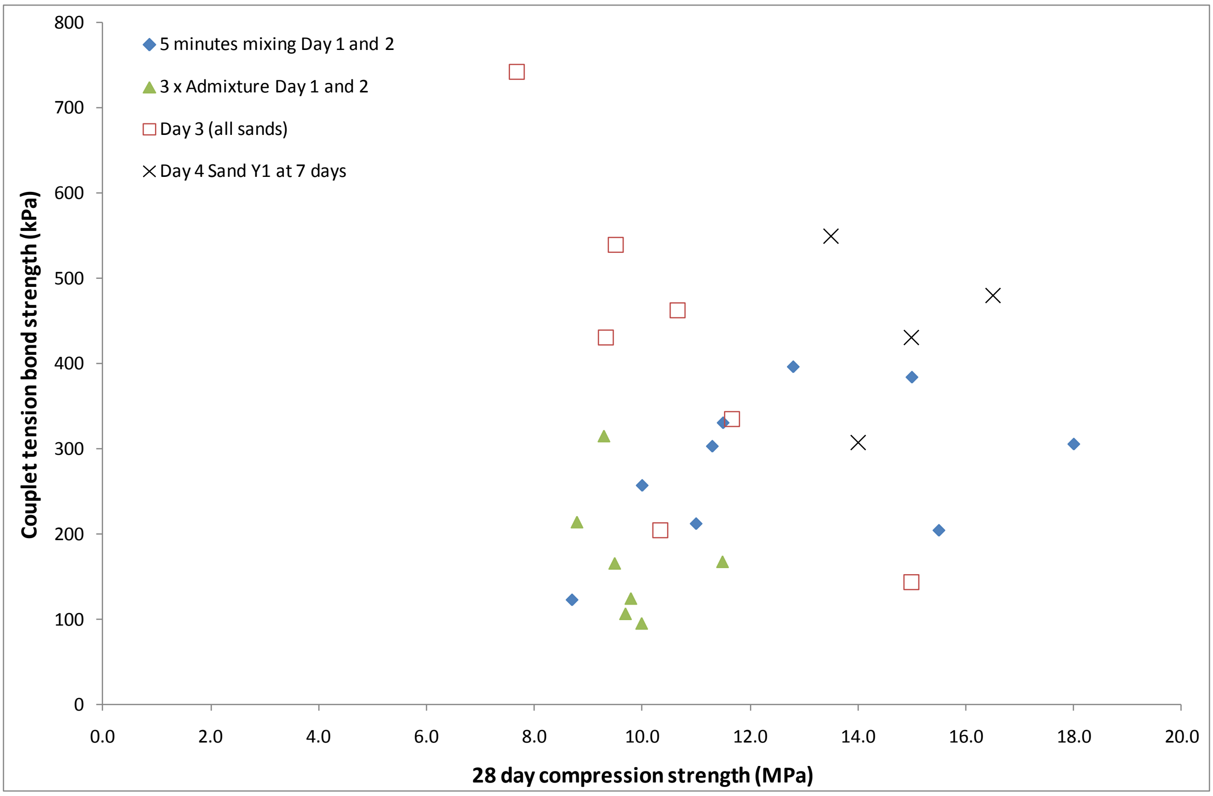


Figure 21. Relationship between seven day couplet bond strength and cylinder compressive strength

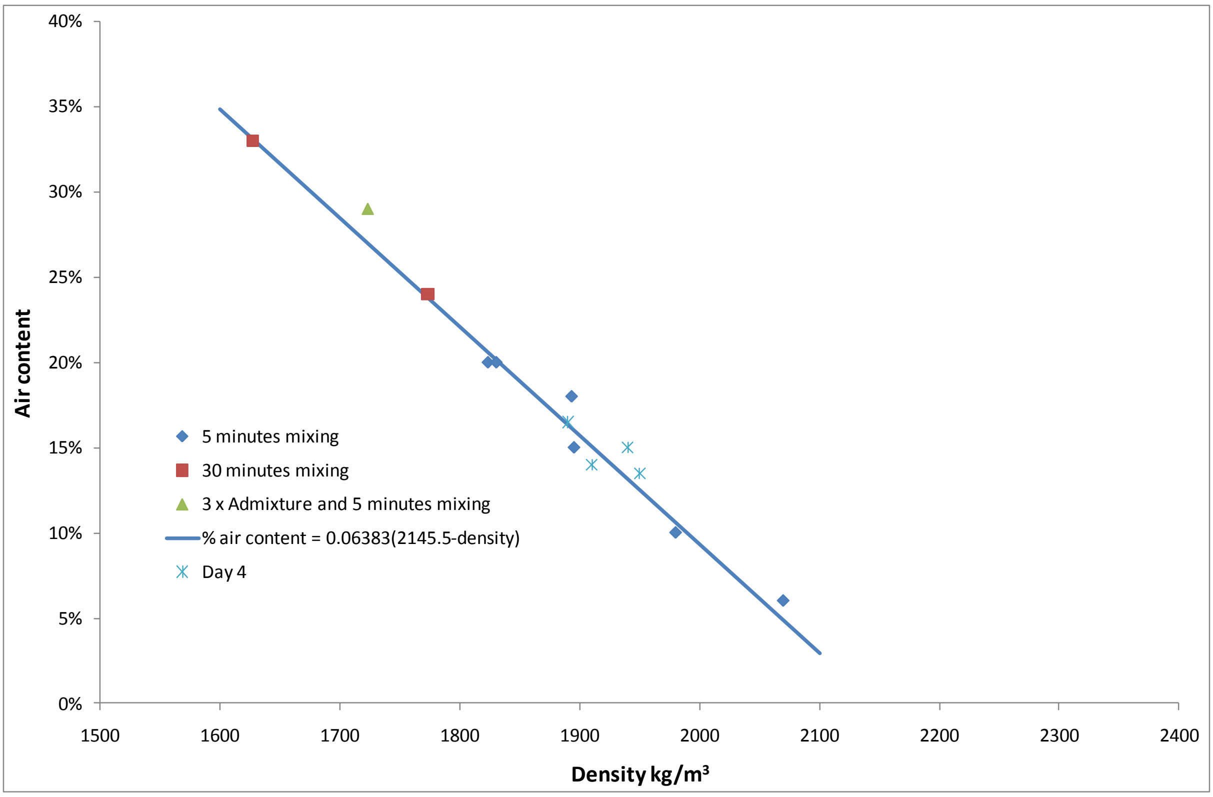


Figure 22. Relationship between air content and mortar density for Sand Y

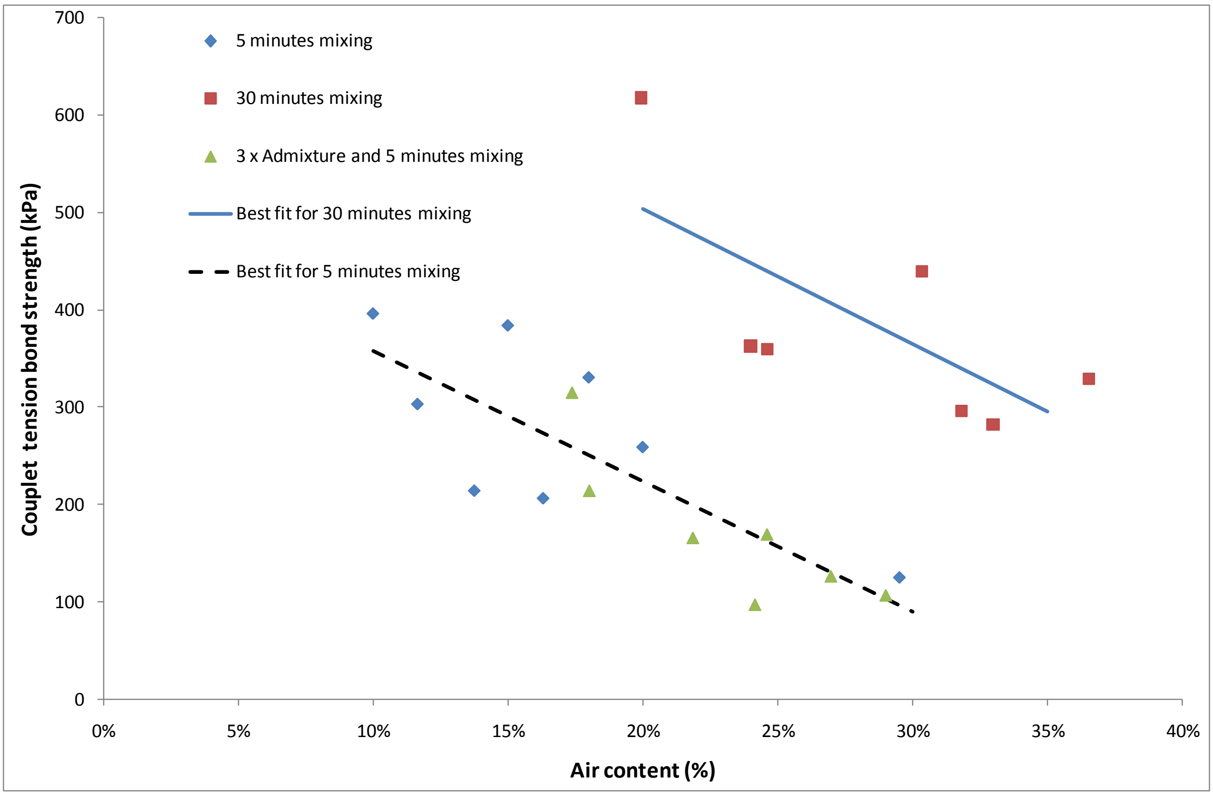


Figure 23. Relationship between couplet bond strength and mortar air content for Sand Y

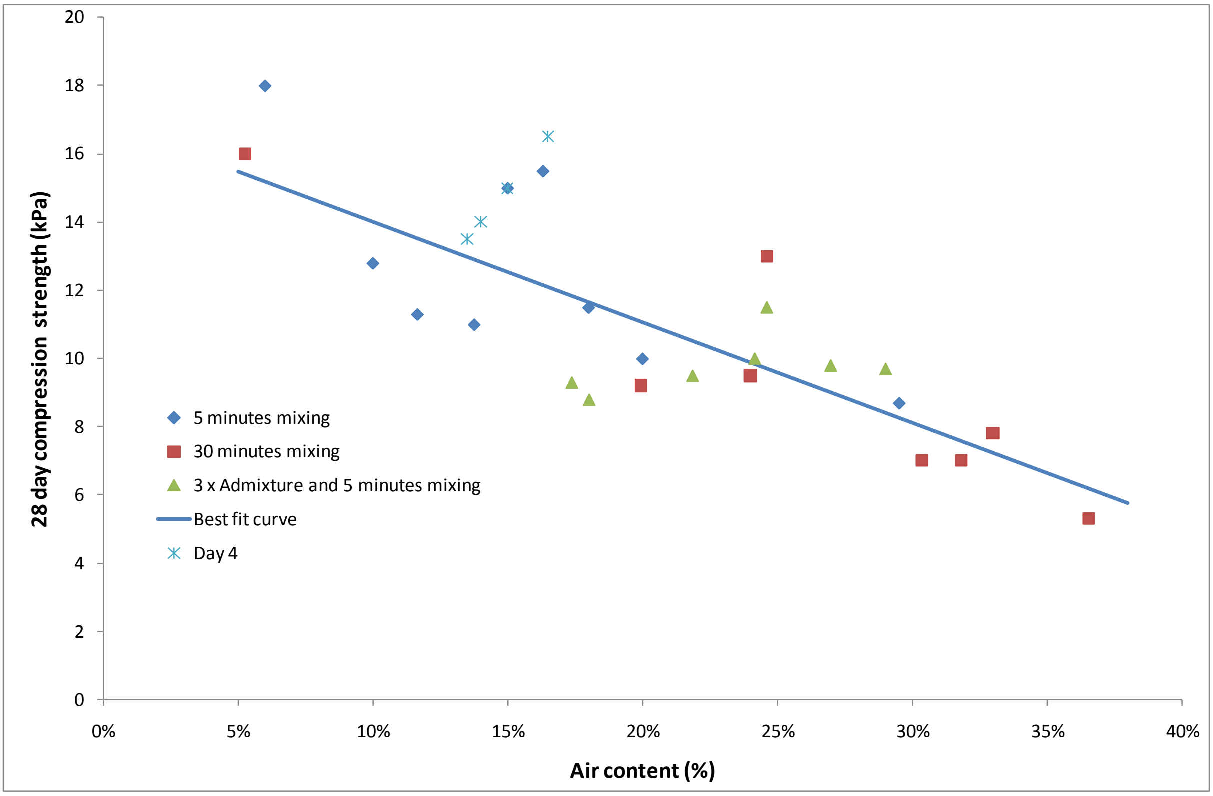


Figure 24. Relationship between mortar compressive strength and mortar air content for Sand Y

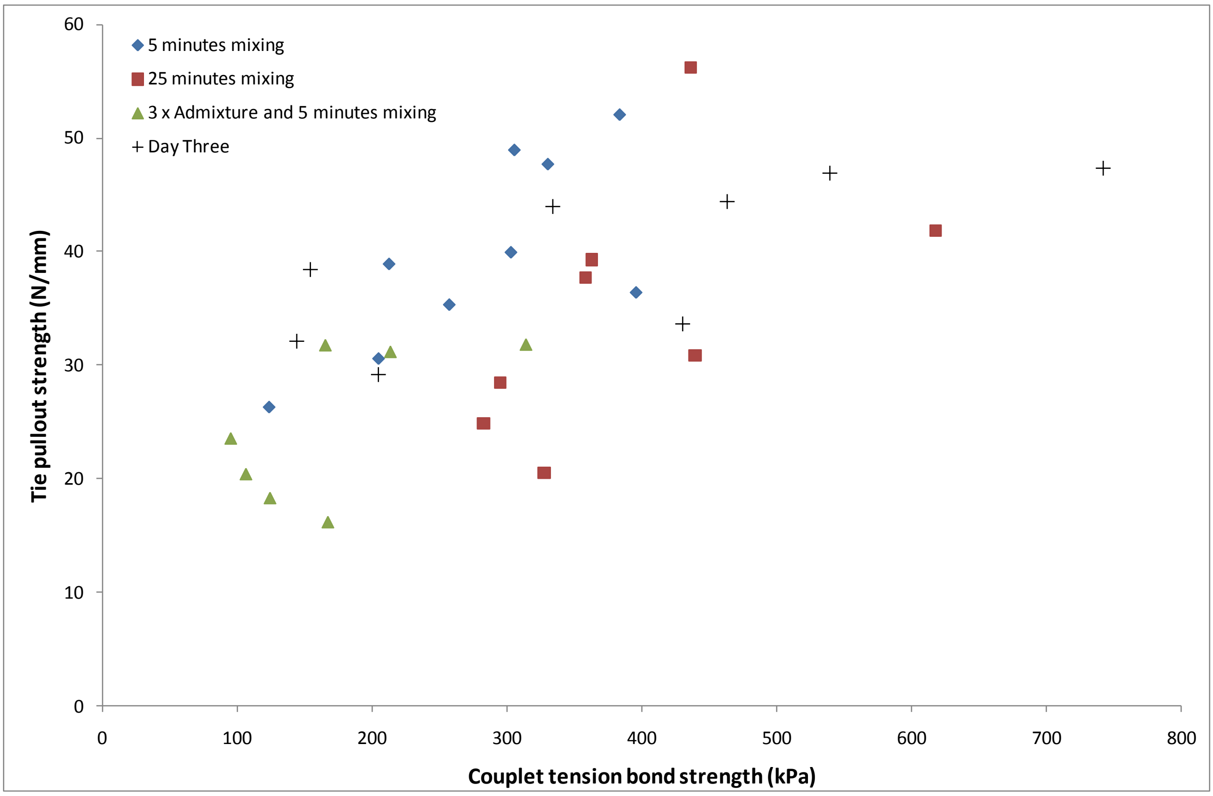


Figure 25. Relationship between mortar tie bond strength and mortar couplet bond strength from Day One and Two test results

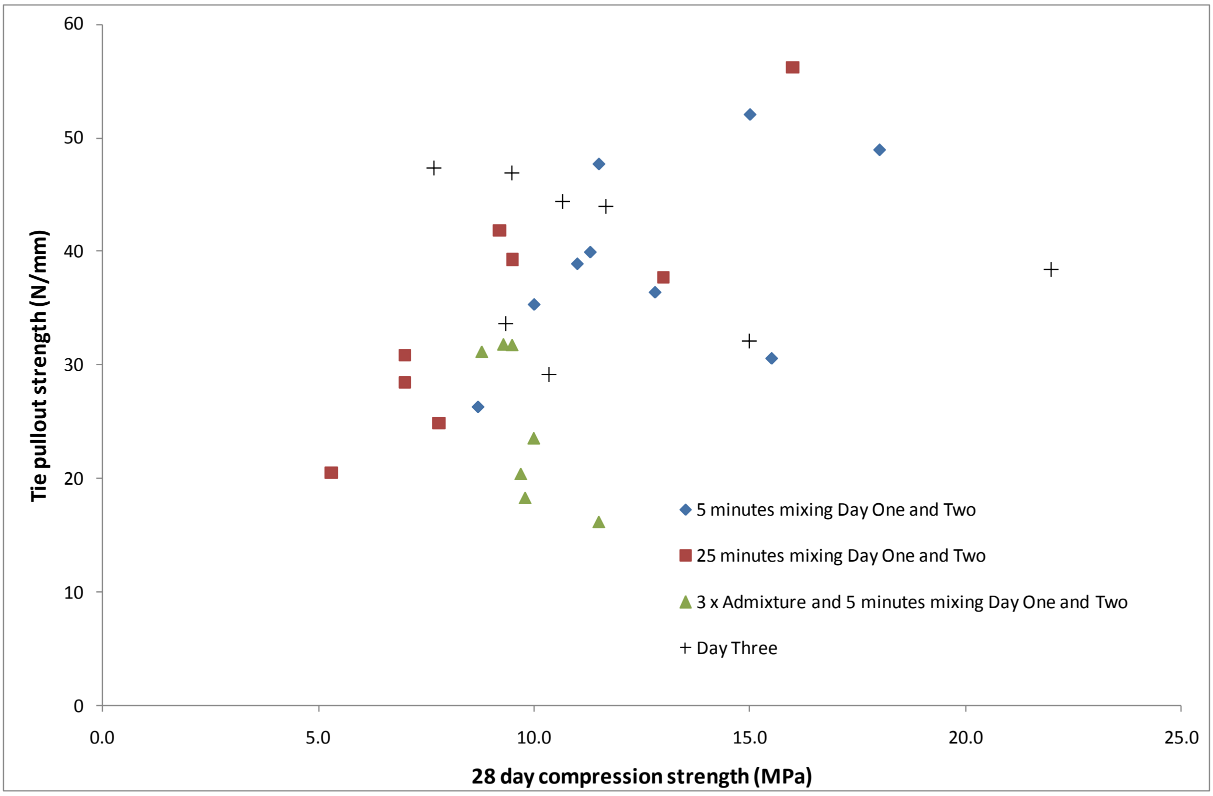


Figure 26. Relationship between mortar tie bond strength and mortar compressive strength

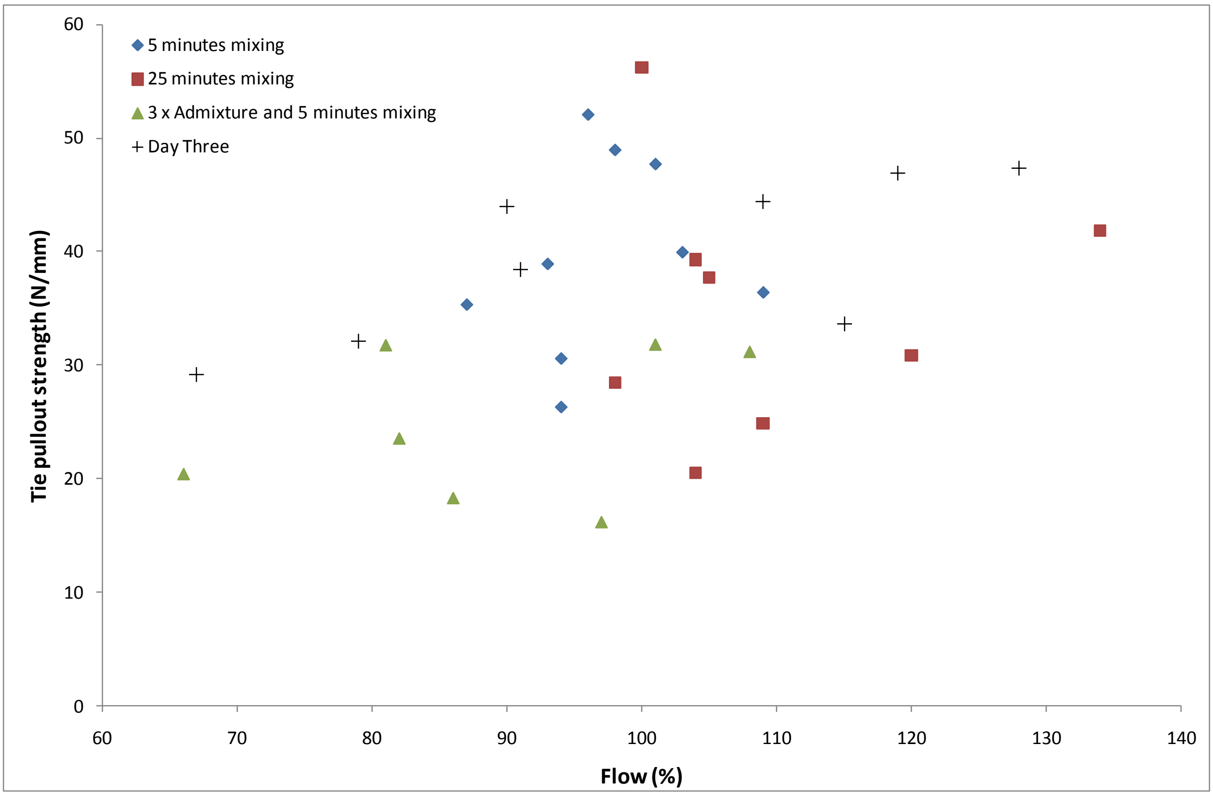


Figure 27. Relationship between mortar tie bond strength and mortar flow

Table 22. Admixtures or pre-mix type used in the BRANZ testing program

Number	Admixture or Pre-mix label
1	Hydrated lime
2	A2 admixture
3	A3 admixture
4	A4 admixture
5	A5 admixture
6	A6 admixture
7	A7 admixture
8	B1 pre-mixed mortar
9	B2 pre-mixed mortar
10	B3 pre-mixed mortar

4.7 Test results on elemental specimens using various sands

4.7.1 Properties of sands used

Small samples of sand were collected from various suppliers around the country and were sent to BRANZ. They were subsequently sent to an accredited laboratory for sand sieve analysis. Based on this information the seven sands listed in the first column of Table 23 were selected for detailed mortar testing as they were considered to represent the extremes of good and bad from the spectrum of sands used in the country. A larger quantity of sand was then obtained, reputably from the original source, and the letter D placed in front of the label to represent “Duplicate” as shown in the second column of Table 23. The reasons for the selection of the sands are given in the third column of Table 23.

The remaining sand in Table 23 was Sand Y. This was the control sand and is described in Section 4.6.1. It was purchased in bulk from the beginning of the project.

The sand grading for the first two sands in Table 23 is given in Figure 28 and for the next five sands in Figure 6 and Figure 7. The grading for Sand Y is shown in Figure 17.

The bricklayer’s comments on the sands when made into mortar on Day Three using Admixture 2 (see Table 22) are given in Table 24. The comments correlated moderately well with the measured flow as shown in Figure 29. Sands that he considered inferior required more water added to achieve a satisfactory workability and this resulted in a higher flow. However, as flow correlated well with bond strength (Figure 30), the “inferior” sands gave the paradoxical result of greater bond strength as shown in Figure 29. Hence, it would appear that providing adequate mortar flow is more critical than the selection of sand type.

The bricklayer’s comments on the same sands, but made on Day Four while he was building the veneer walls, are also given in Table 24. They did vary from his initial assessment for DX1 (Wall W11) but remained constant for Sand Y (Wall W9) and similar for DJ1 (Wall 10). As the admixture was different for Wall W12, no comparison can be made for comments on the mortar in this wall and Day Three comments.

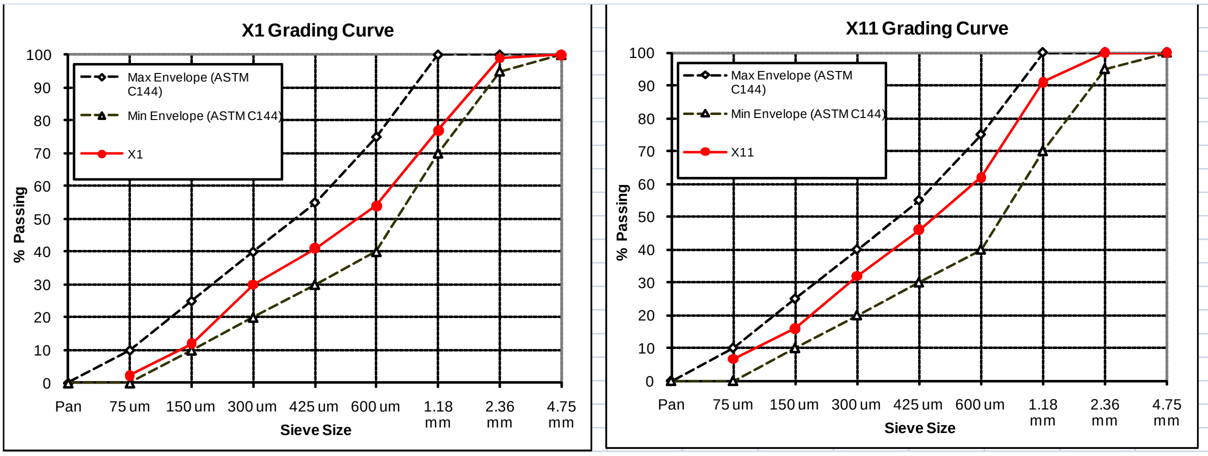


Figure 28. Grading curves for sands X1 and X11

Table 23. Sands used in Tests in Section 4.7

Sand label from original small packet	Sand label from larger volume of the same sand	Reason for selection of sand
X1	DX1	Good fit to grading envelope, fairly clean
X11	DX11	A moderately good grading curve, but dirty
J1	DJ1	A single sized very fine sand
J6	DJ6	Another example of a single sized very fine sand
J9	DJ9	Good fit to grading envelope, clean
J12	DJ12	A large discontinuity between 300 mm and 150 mm sieves
J13	DJ13	Similar to J6
-	Y	Control specimen for testing at different dates

Table 24. Bricklayer's comments on mortar made from the following sands

Sand Label	Comments	Relative rating (1= good)	Measured flow
DX1	Sand/mortar "felt very good"	1	79
DJ1	He considered this sand to be too fine and "hungry for water", fatty, and did not bind well. He thought that the mortar would improve if more cement or admixture was added.	7	115
DJ6	He considered this to be a fine beach sand. He described the mortar as being "stogie", "dead", had "no life" and had "no give" with the bricks. It was similar to, but worse than, Sand DJ1.	8	128
DJ9	This sand was coarser than DJ1. It made mortar which was not too sticky, but was not as good as DX1. The cement mixed right away and the	3	90

	water did not separate. There was time to work it. It had life in it.		
DX11	This limestone sand was similar to the DJ9 sand. However, the mortar absorbed water OK and did not separate but was slightly sticky. However, he rated it slightly worse than DJ9.	4	91
DJ12	He described this as a coarse silica sand. It felt like “silver” sand but it was a bit coarser and had a bit of grittiness. The mortar was slightly hungry for water and the mix needed more cement. It was good, but not quite as good as mortar from DX1 sand.	2	67
DJ13	He considered this beach sand to be similar to DJ1, but not as bad. It made mortar which was fatty and hungry for water but it bound OK.	6	109
Wall W9 (Sand Y)	He considered this to be coarse river sand which contained some foreign particles. It was a clay loam Sand Y that did not mix up easily. However, it made mortar which felt good on the trowel, had good water retention and did not separate. The mortar was slightly fatty and stodgy and needed more admixture.	5	119
Wall W11 (Sand DX1)	This was a gritty, coarse and sharp sand and needed more fines but the mortar stuck to the trowel OK. Being a coarse sand, water got sucked out of the mortar quickly and so after the bricks are placed the bricks could not be move easily in the fresh mortar.		
Wall W10 (Sand DJ1)	This was a fine silica beach sand and the he had more time to work with the mortar made from it but it separated easily. It needed a lot of water added but then changed from being too dry to “custard” with little change in the water added. It was hard to move the brick on the fresh mortar bed – stodgy.		
Wall W12 (Sand Y)	This was Admixture 7 rather than that used in all the other mortars in this table which used Admixture 2. The mortar was described as being very “fluffy”.		

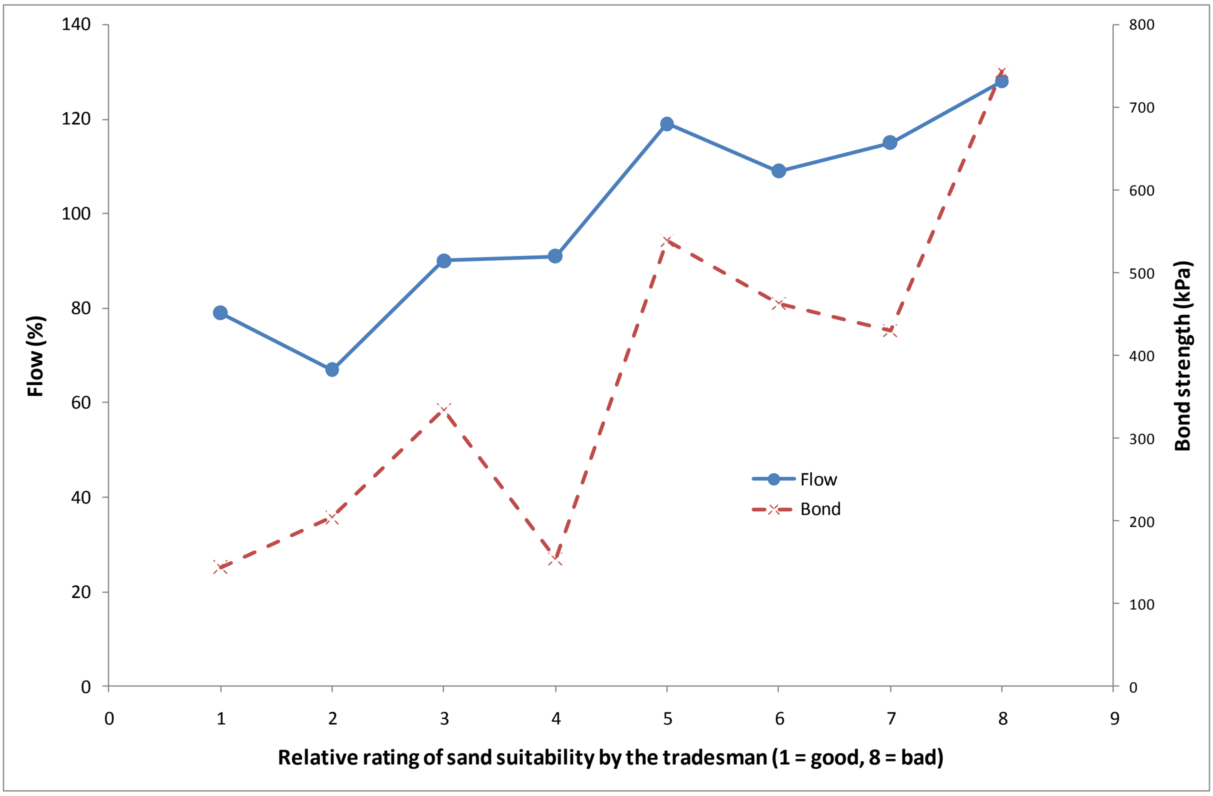


Figure 29. Flow and bond strength versus sand rating for the range of tested sands

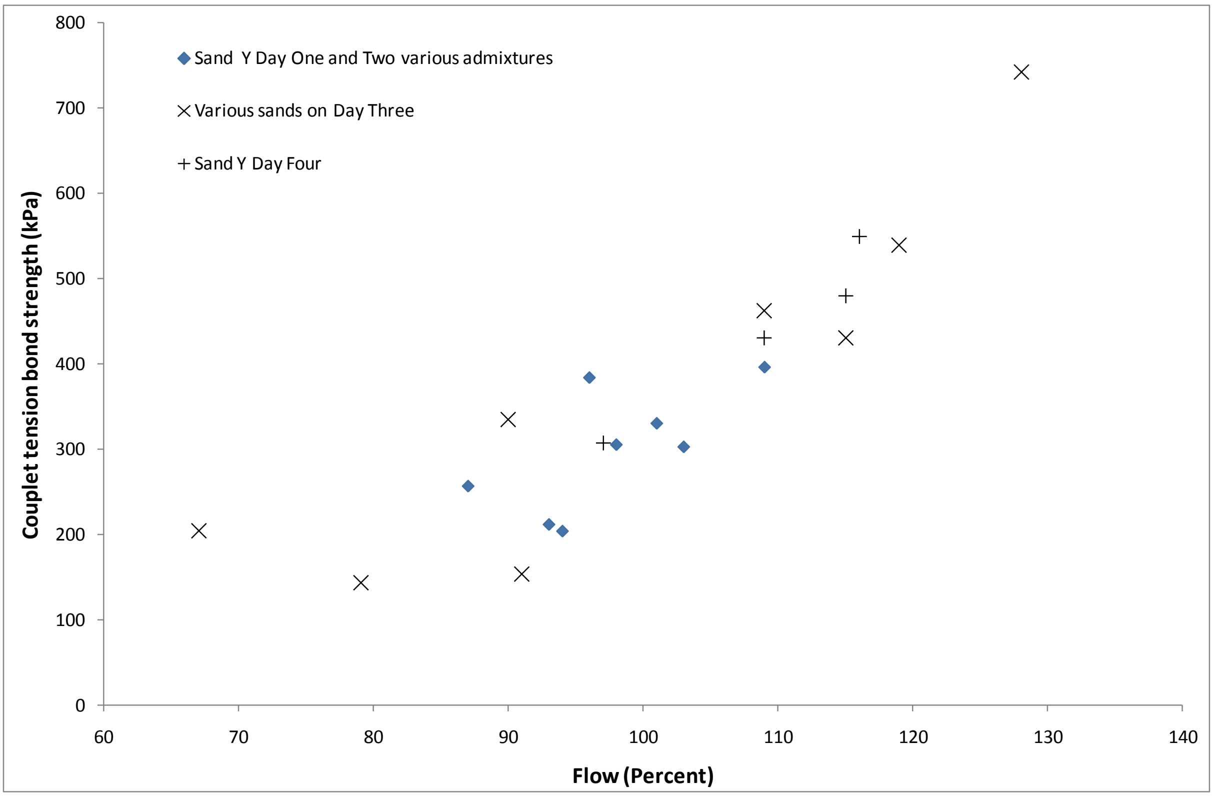


Figure 30. Bond strength versus flow for the range of tested sands on Day Three plus comparisons with Sand Y (all tests at seven days with five minutes mixing)

4.7.2 Test results

In an attempt to obtain more accurate bond strengths, 12 bond strength couplets were tested for each of the eight sands of Table 23. Also, five brick tie pullout tests brick couplets and three 100 x 50 mm steel cylinders were taken for mortar compressive strength testing for each sand type. All were mixed in the ratio of 4.5 sand to 1.0 cement and the standard ratio of Admixture 2 (see Table 22) of 100 ml/40 kg bag of cement.

Test results are shown in Table 25 and the couplet bond strength is plotted against flow in Figure 30. Also shown in this plot are results from Sand Y tested at mortar age seven days from other elemental tests. Although the results differ significantly on each test day, this is attributed to the difference of flow on each of the three days and the results are compatible as can be seen in the plot.

The tie bond strength was generally better for the mortars the tradesman considered best (see Figure 31), with the exception being the mortar from sand DJ6 which was rated worst but had one of the highest tie bond strengths.

The mortar compressive strength tended to reduce with increased mortar bond strength as shown in Figure 32, although there was considerable scatter of results.

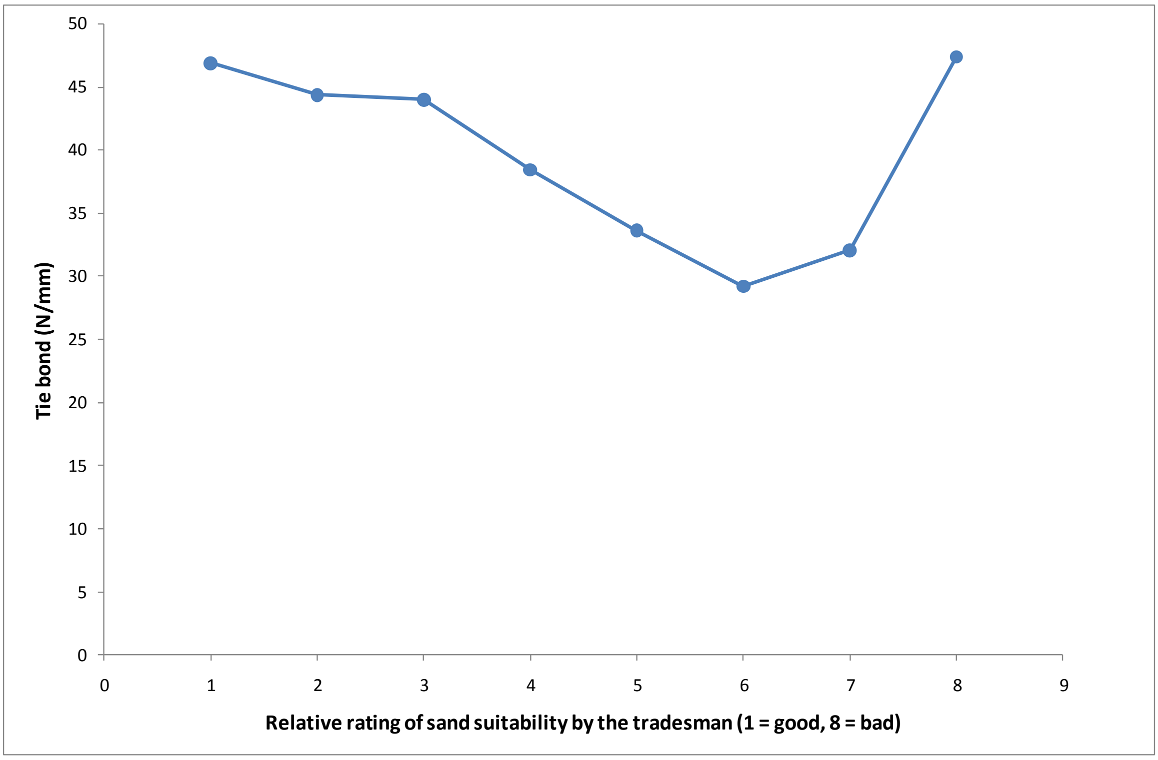


Figure 31. Relationship between tie bond strength and sand rating

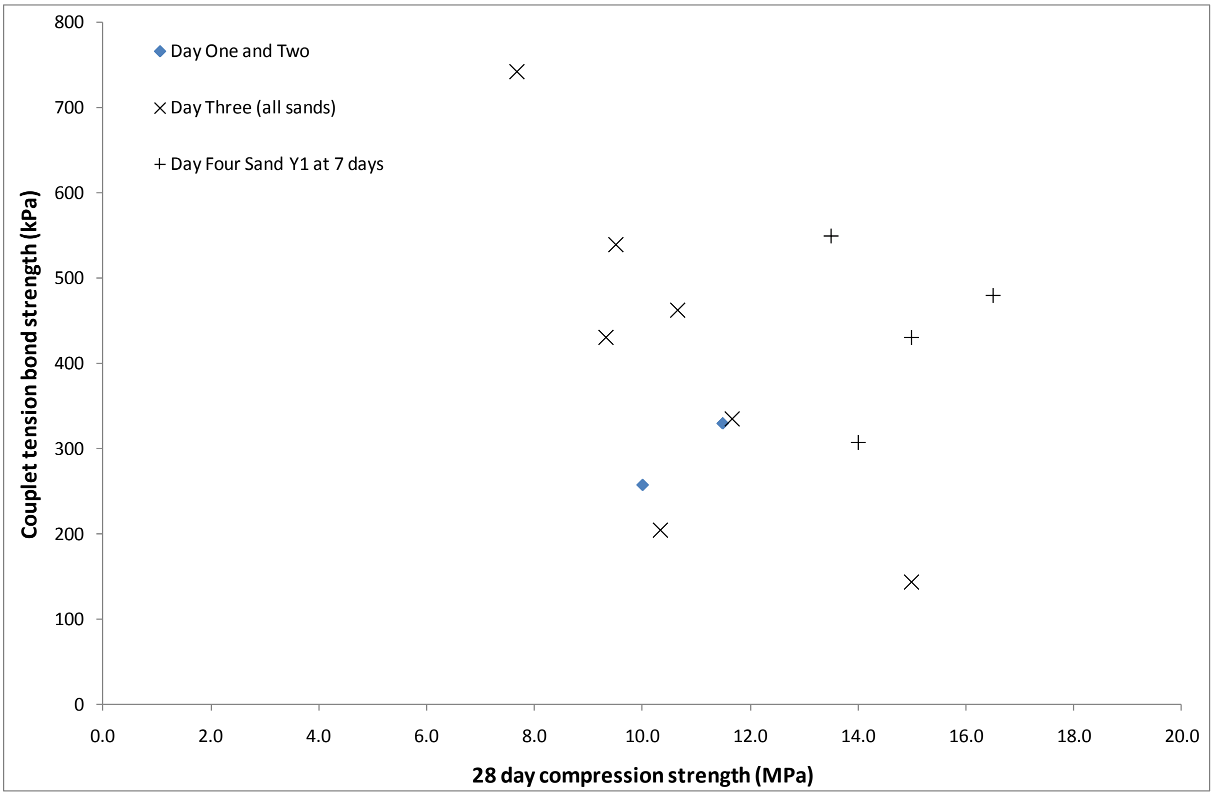


Figure 32. Relationship between mortar bond strength and mortar compressive strength for Sand Y Admixture 2

Table 25. Test results from elemental tests on mortar on Day Three using various sands

Sand Type	Flow %	Couplet bond strength (kPa)		Tie bond strength (N/mm)		Compressive strength (MPa)		Density kN/m^3	Ratio couplet: Tie bond	Later used for Wall Number
		Ave.	C.O.V.	Ave.	C.O.V.	Ave.	C.O.V.			
Y	119	539	0.25	47	0.09	9.5	0.091	1880	11.50	**W9 and W12**
DJ13	109	463	0.17	44	0.11	10.7	0.027	1680	10.43	
DJ9	90	334	0.27	44	0.16	11.7	0.124	1707	7.60	
DX11	91	154	0.40	38	0.10	22.0	0.060	1953	4.00	
DJ1	115	430	0.24	34	0.18	9.3	0.062	1800	12.80	**W10**
DJ12	67	204	0.52	29	0.12	10.3	0.028	1640	7.00	
DX1	79	144	0.36	32	0.17	15.0	0.000	1797	4.48	**W11**
DJ6	128	743	0.18	47	0.18	7.7	0.136	1860	15.67	
Average	**99**	**360**	**0.28**	**40**	**0.14**	**11.8**	**0.07**	**1803**	**8.77**	

5. STAGE III. SHAKE TABLE TESTS ON BRICK VENEER WALLS

Four brick veneer walls were constructed with each using a different mortar mix. They were first racked in the in-plane direction and then shaken to destruction on the BRANZ shake table at a mortar age of 49 to 58 days.

- Section 5.1 describes the elemental tests performed on the mortar used
- Section 5.2 describes the wall construction
- Section 5.3 describes the in-plane tests
- Section 5.4 derives the seismic demand load on the test walls to enable a comparison to be made of the measured strength of the test walls and the required demand load
- Section 5.5 describes the shake table tests
- Section 5.6 analyses the test results.

5.1 Elemental test results

Based on the elemental tests of Section 4.7, four mortar mixes were selected for brick veneer walls to be tested on the BRANZ shake table. These were based on the maximum and minimum bond strengths from the mixes as summarised in Table 26. The walls were called W9, W10, W11 and W12 and were constructed over a period of three days.

During construction of the test walls, five brick bond couplets and three brick tie test specimen were made and one mortar cylinder was filled from each wheelbarrow of mortar used to make the walls. A minimum of three wheelbarrows of mortar were used per wall. The couplets and brick tie specimen were tested prior to the shake test (age 42 to 46 days) and the cylinders were tested at 28 days. A flow test was performed and an air content sample was taken from the mortar from each wheelbarrow.

Also, five brick bond couplets per wheelbarrow were taken for Wall W9 and these were tested at age seven days.

A comparison of the measured strengths from mortar used for the test walls and those from prior elemental testing is given in Table 26. It can be seen that the mortar bond strengths at the time of testing had increased significantly from the seven day strength. It can be seen that the bond strength of Wall W10 has now surpassed that of Wall W9 and the Wall W11 strength has increased significantly, which may be due to the increase of flow on Wall W11 mortar. The compressive strengths of the mortar used in the test walls were far higher than from the elemental tests and it is difficult to assess why this was so as the C.o.V.'s in each group were low.

5.1.1 Relationship between flow and bond strength for all elemental tests

The relationship between flow and bond strength for couplets tested at seven days was strong. This is plotted in Figure 33 for Sand Y with Admixture 2 after five minutes mixing only, similar to that used in the shake table test on Wall W12. Note that the values for Day Four are the average of five couplets from each of four wheelbarrows.

When all sands admixtures and mixing times are included (Figure 34) the relationship is still strong but shows more scatter. Couplets tested at a mortar age of 42-45 days (which is a little less than the age at wall testing) showed a significant increase in strength from the seven day mortar bond strength as shown in Table 26 and Figure 35. The brick tie strengths (Table 26) also show some strength enhancement when tested at an older age.

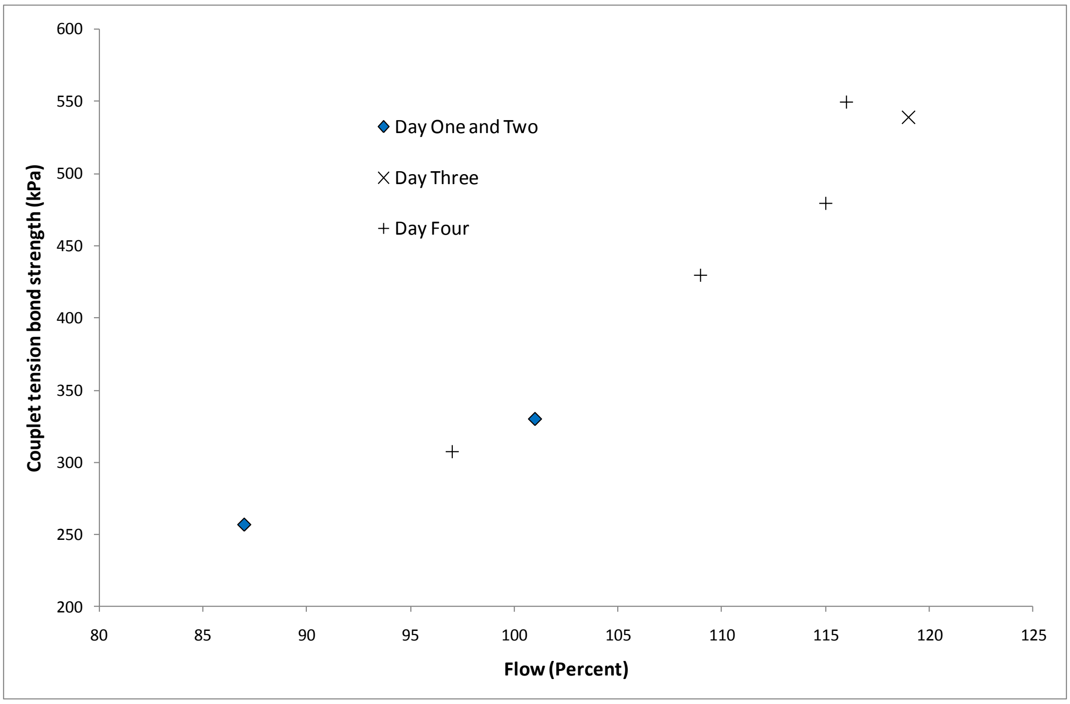


Figure 33. Relationship between couplet bond strength and flow for mortar using Sand Y and Admixture 2 from elemental tests at mortar age seven days

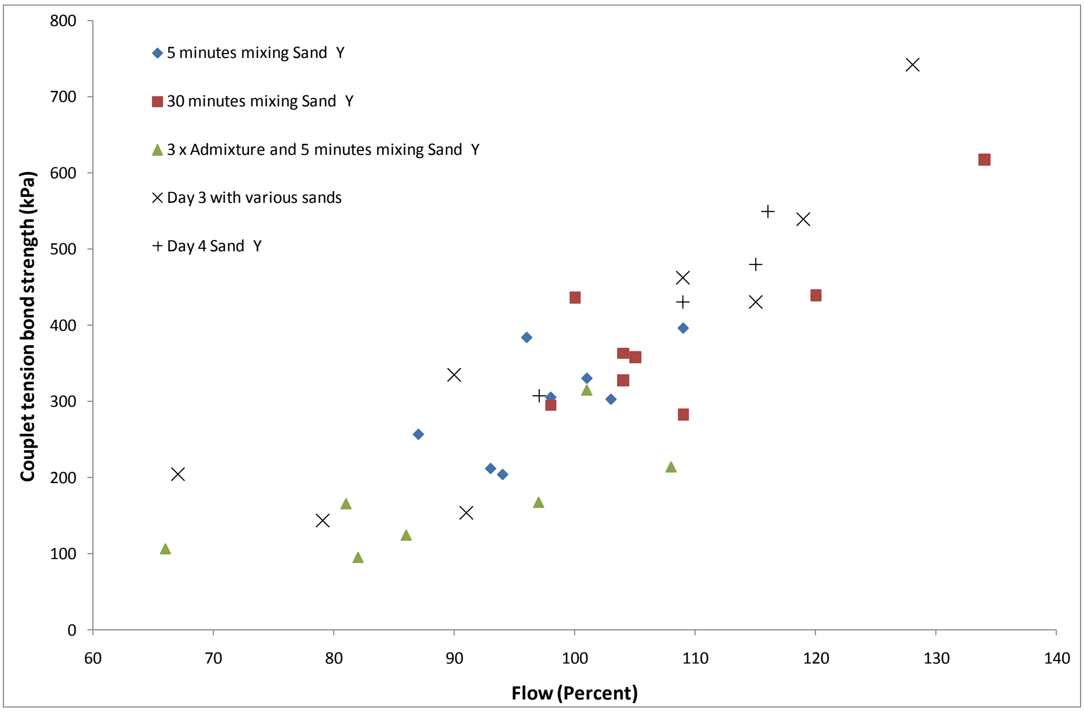


Figure 34. Relationship between couplet bond strength and flow for all tests at mortar age seven days

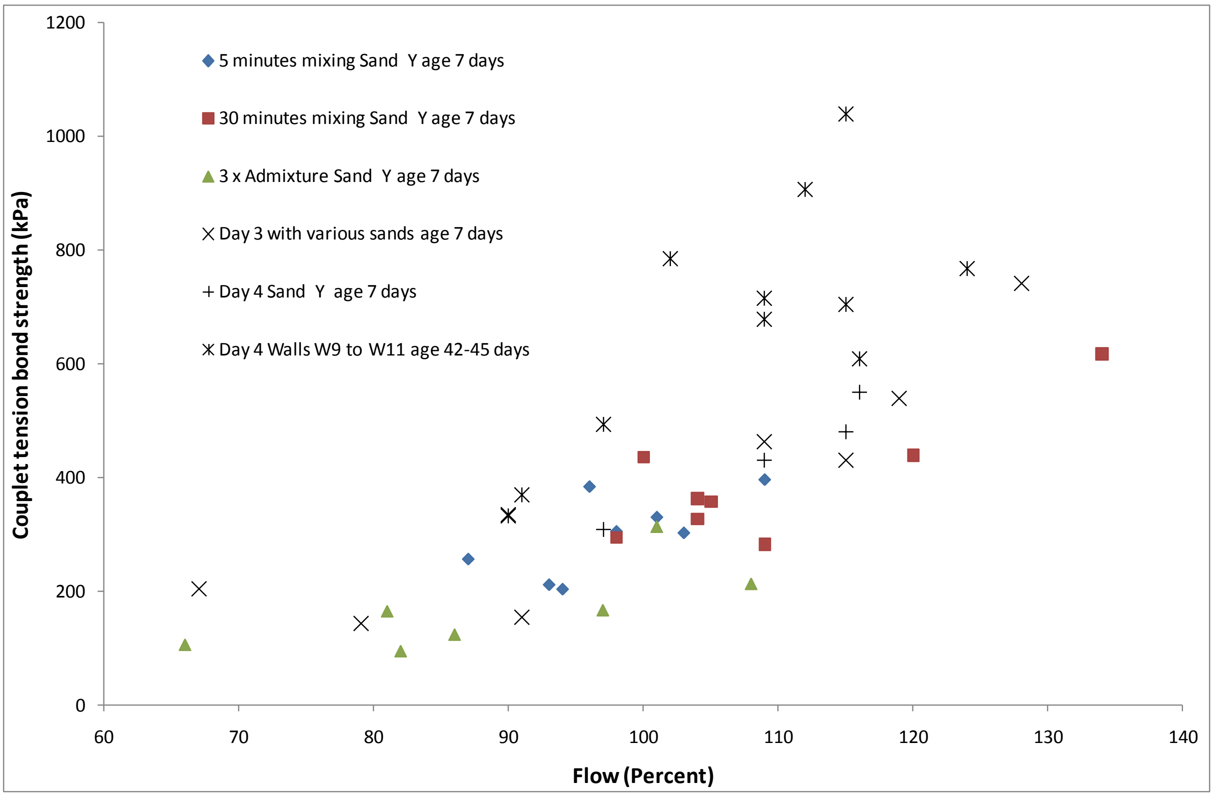


Figure 35. Relationship between couplet bond strength and flow for all tests irrespective of mortar age

Table 26. Test results on mortar used in shake table tests

Wall Label	Sand Type *	Admixture Used **	Elemental test results								
			From Day3 except W12 is from Day1				From samples taken during wall construction				
			Flow	Brick bond strength at 7 days (kPa)	Tie bond strength N/mm	28 day compressive strength (MPa)	Flow	Brick bond strength at 7 days (kPa)	Brick bond strength at 42 to 45 days (kPa)	Tie bond strength at 43 to 46 days N/mm	28 day compressive strength (MPa)
W9	Y	2	119	539	46.9	9.5	110	455	686	59	15.3
W10	DJ1	2	115	430	33.6	9.3	102	-	835	56	12.7
W11	DX1	2	79	144	32.1	15.0	109	-	490	47	17.7
W12	Y	7	94	123	26.0	8.7	104	-	240	40	16.2

Legend

* Sand type is described in Section 4.7.1.
** Admixtures are described in Table 22.

Notes on Table 26

- Dosage of Admixture 2 was 100 ml/40 kg of cement which is that recommended by the manufacturer.
- Dosage of Admixture 7 was 658 gm/40 kg of cement which is that recommended by the manufacturer.
- The sand:cement ratio was 4.5 for all walls except Wall W12 where the ratio was 4:1 as recommended by the manufacturer.

5.2 Specimen construction

The veneer was 2.4 m wide and consisted of 26 courses of brick resulting in a veneer height of 2.23 m. Photographs of the specimens can be seen in Figure 36 and Figure 37. The ties were at the 2nd and 24th mortar joint and every 4th joint between. The walls did not include window or door openings. The wall framing was 1.8 m wide and 2.42 m high and consisted of 90 x 45 MSG eight radiata pine studs at 600 mm centres. The ties were dry-bedded in the mortar.

The bricks were laid by a tradesman using the mortar described in Table 26. The mortar strength test results are also summarised in this table. Approximately 10 mm thick mortar joints were used between the bricks on both horizontal and vertical joints. They were concave tooled and burnished after the initial stiffening had occurred.

Hot-dipped Eagle brand galvanised brick ties, which were 85 mm long, were spaced at 340 mm vertically and 600 mm horizontally. Ties were secured to the face of the timber studs using 35 mm long galvanised, self-drilling Tek screws.

5.3 in-plane testing prior to the out-of-plane tests

The earthquake's direction will usually not align with the main axis of a building and so veneer walls need to be designed for seismic load in both in-plane and out-of-plane directions. This was simulated by first slow cyclically racking the test walls in the in-plane direction and then performing shake tests in the out-of-plane direction. To facilitate this the walls were built on strong steel foundations which could be easily lifted and moved and then bolted to either the strong floor as shown in Figure 36 or the shake table as shown in Figure 37. The wall framing was fixed to preclude stud uplift. The brick veneer was constructed on the concrete filled channel also shown in Figure 36. The base of the veneer was blocked in the in-plane tests to prevent the veneer sliding on the concrete foundation.

The framed walls were lined with plasterboard for the in-plane tests. However, this was removed for the out-of-plane shaking tests to enable the movement between veneer and timber studs to be observed. In similar tests previously done at BRANZ (Thurston and Beattie 2011) the top plate sometimes separated from the studs. As the intention of the current study was to test the relative strength of the four brick veneers, and not the timber framing, the top plate in the current tests was strapped to the studs to ensure the top-plate to stud joint did not fail during testing.

An actuator applied four cycles of ±16 mm top plate displacement to each framed wall (with the brick veneer precluded from rocking using tie down rods as shown in Figure 36), and then four cycles to ±24 mm with the rocking restraint removed. The four cycles to ±16 mm was to simulate the deformation regime a tie must undergo as specified in AS/NZS 2699.1, and the four cycles to ±24 mm with rocking allowed was to duplicate the maximum displacements expected in a design level earthquake.

Most of the wall framing movement was taken up by tie distortion. However, the veneer walls cracked at the base of the wall.


Figure 36. Test set-up for in-plane racking tests


Figure 37. Test set-up for in-plane racking tests

5.4 Seismic design loads

This study measures the seismic resistance of brick veneer under face load. To determine whether this is adequate it is necessary to calculate the seismic demand loads stipulated by the New Zealand loadings standard NZS 1170.5 (2004). This demand load is calculated below for single and two-storey residential constructions.

5.4.1 Single-storey veneer construction

Section 8 (requirements for parts and components) of NZS 1170.5 (SNZ 2004) states that (for category P.1, P.2 and P.3 parts) that the scope of Section 8 was limited to parts that weigh more than 10 kg and are able to fall more than 3 m onto a publically accessible area. Further, that "when a part is supported directly on the ground it shall be designed as a separate structure with design actions derived in accordance with Section 5" of NZS 1170.5. Consequently, single-storey brick veneer need not be designed as a "part" and Eqn. 5.2(1) of NZS 1170.5 was used to derive the horizontal design action coefficient, $C_d(T_1)$ for out-of-plane loading on a single-storey brick veneer house located on soft soil in Wellington. It was assumed that the building period was less than 0.4 seconds and the ductility factor, μ, was 1.0. Thus;

$$C_d(T_1) \quad = \quad \frac{C\,(T_1)\,S_p}{K_\mu} = \frac{1.2\ x\,1.0}{1.0} = 1.2$$

Note that based on the face load tie strength requirement of Table 2 of AS/NZS 2699.1 (SNZ 2000) the tie spacing given in Table 2.3 of NZS 4210 (SNZ 2001) implicitly assumes $C_d(T_1)$ = 1.74 for Wellington (Thurston and Beattie 2011).

5.4.2 Upper storey of two-storey veneer

Section 8 specifies that the design action on a part, F_{ph}, is given by:

$F_{ph} = C_p(T_p)\ C_{ph}\ R_p\ W_p = C\ (0)\ C_{Hi}\ C_i(T_p)\ C_{ph}\ R_p\ W_p = 1.12 \times 1.5 \times 2.0 \times 1.0 \times 1.0\ W_p = 3.36\ W_p$

To calculate C_{Hi}, assumptions needed to be made of the height of veneer attachment to the building and height of the uppermost seismic mass (which could be open to debate).

In conclusion, the design action coefficient derived from NZS 1170.5 is 1.74 for single-storey buildings and 3.36 for two-storey buildings.

5.5 Out-of-plane testing

5.5.1 Shake table tests

Shake table tests were performed on four brick veneer walls fixed to a shake table in a similar manner to previous BRANZ tests (Thurston and Beattie 2009).

For each wall, the table was subjected to a series of pseudo-earthquakes (P-EQs) controlled by sinusoidal voltage traces sent to the shake table controller. The excitation voltages sent are proportional to the target table displacement.

In each P-EQ, the voltage trace (i.e. table shaking excitation) can be considered to be a train of 13 carriages, with each carriage being two cycles of sine waves at different frequencies. These were 8, 7, 6.5, 6, 5.5, 5, 4.5, 4, 3.5, 3, 2.5, 2, 1.5 and 1.0 Hz as shown in Figure 45. There was a single transition cycle at the beginning of the train and also at each frequency change (see Figure 46). This transition started and finished at the correct table speed and did not exceed the target maximum acceleration of the P-EQ. The "train" terminated with a relatively sharp stop, of maximum acceleration 0.5 g, which caused the test wall to exhibit a decay vibration which was used to determine the natural frequency and damping of the test wall.

Note that the P-EQ focused on the frequency range 6.5 to 3.5 Hz as the natural frequency of the walls is expected to be in this zone (see Section 5.5.3).

The first P-EQ applied (called Set A) imposed the lowest level peak displacements, with good fidelity being achieved for both the target displacements and accelerations. The magnitude of the voltage trace was designed to produce the same table accelerations at each frequency. Thus, at each frequency (i.e. carriage of the train) the sine wave induced different table peak displacements but the same peak accelerations.

At each successive P-EQ from Set B to Set G greater magnitudes of excitation voltage trace were applied to produce greater peak accelerations. However, at the lower frequencies the trace was modified to take into account the physical limits of the equipment, and the accelerations at these lower frequencies was less than the target accelerations.

Figure 45 shows the target acceleration record for two P-EQs called Set A and Set G. (Set B to Set F are not shown for clarity.) The first portion of Figure 45 is expanded in Figure 46 to show the higher frequency accelerations more clearly. With the low accelerations of Set A the table is capable of following the whole record. In Set G, for frequencies below 3.5 Hz, the table velocity and displacement limitations prevented the full peaks from being achieved as shown in Figure 45.

5.5.2 Test set-up

The bottom of each specimen was bolted to the shake table as shown in Figure 38. The actuator applying the motion was fixed to mid-height of a stiff space frame (also shown in Figure 38). The space frame was rigidly connected to both the shake table and to the top plate of the wall framing. Thus, the accelerations imposed on the table were also (approximately) imposed to the top of the framing. This arrangement simulated the situation where the side walls and ceiling/roof diaphragm of a house are extremely rigid and is one extreme for worst case out-of-plane loading of veneer walls.

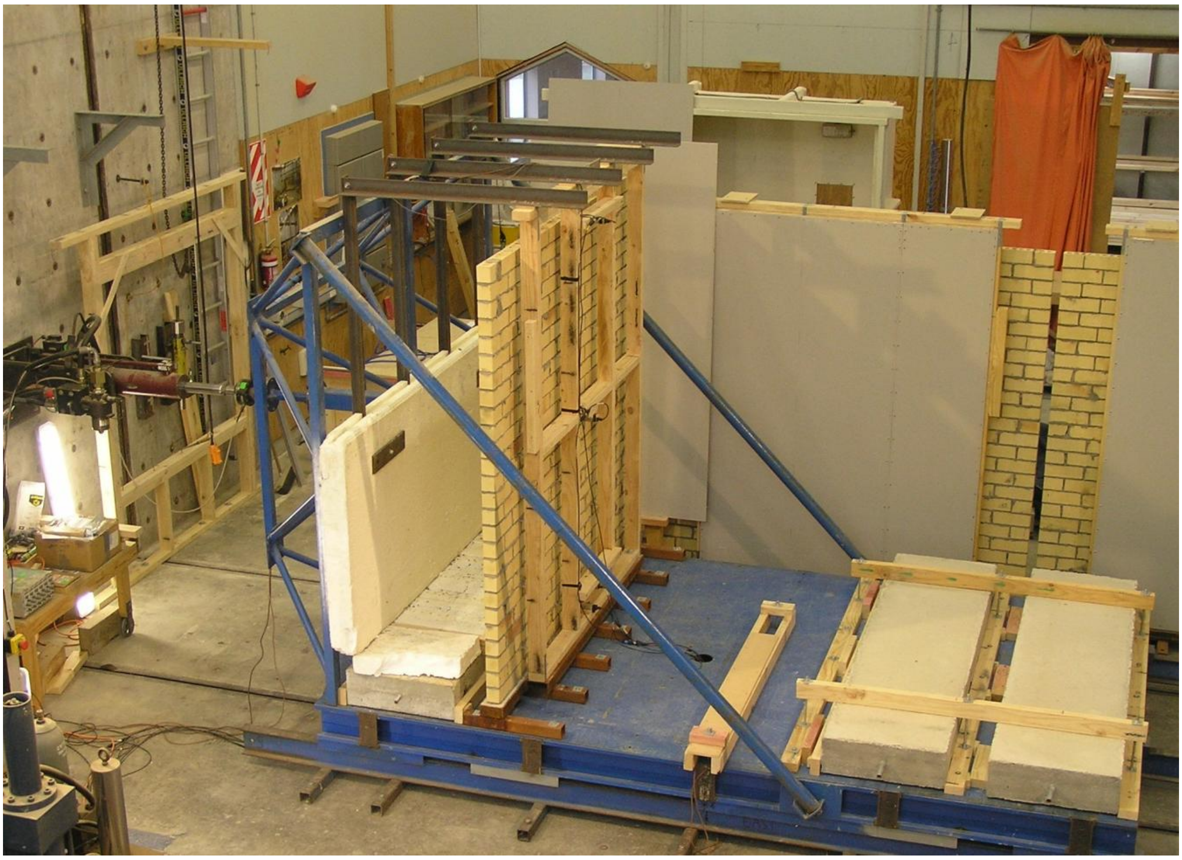


Figure 38. Space frame and general set-up for the out-of-plane shake table tests

5.5.3 Natural frequency measurements

Priestley et al (1979) developed a formula for calculating the fundamental frequency of veneer panels. The method for calculating the natural frequency of cracked veneer panels simply assumed that the veneer contributes no stiffness to the wall. Priestley et al found the measured natural frequency of their test walls commenced at close to the theoretical uncracked natural frequency, but this migrated to close to the theoretical cracked natural frequency as the tests progressed. Inserting the construction dimensions as used in the BRANZ tests into their formula results in an uncracked natural frequency of 9.1 Hz and a cracked natural frequency of 4.5 Hz for a wall without openings.

5.5.4 Free vibration tests

At each stage of testing the natural frequency of each test wall was determined by abruptly bringing the shake table motion to a stop as discussed in Section 5.5.1. This created a pulse motion on the wall, which quickly settled into a decaying free vibration allowing the wall natural frequency and damping to be calculated.

An example of the differential displacement between veneer and framing at mid-height of the veneer recorded during such free vibration motion is shown in Figure 39. The natural period is simply determined by extracting the time elapsed between a given number of peaks as illustrated by the '+' symbols in Figure 39. The damping was obtained from the rate the motion decayed using standard formulae. Almost identical natural frequencies were obtained from the measured veneer mid-height acceleration and also the differential displacement between veneer and framing at the veneer mid-height. The plots were not purely sinusoidal, and showed the influence of several contributing frequencies.

Shaking on all walls commenced with EQ Set A except for Wall W12 which commenced with EQ Set B. The measured natural frequency of all test walls at the end of this first low level shake test (P-EQ Set A or in the case of Wall W12, EQ Set B) varied between 6.7 and 7.5 Hz, indicating that the walls were partially cracked. The damping varied between 2.2% and 3.7%. It is expected that the walls had been partially 'pre-cracked' during the in-plane testing.

At the end of the P-EQ set, immediately prior to that where failure occurred, the measured natural frequency in Walls W9, W10 and W11 varied between 4.8 Hz and 4.9 Hz and the damping varied between 7.1% and 10.4%. This indicated that the lower level shake tests had cracked the walls or loosened the ties and had thus decreased the natural frequency and increased the damping.

Comparing the wall measured natural frequency with the theoretical values of 9.1 Hz for an uncracked wall and 4.5 Hz for a cracked wall (see Section 5.5.3) indicates that the veneer was partially cracked after the first P-EQ set and almost fully cracked in the P-EQ set prior to that which resulted in veneer shedding.

5.6 Out-of-plane results

5.6.1 Veneer collapse mechanisms

The wall failure mechanisms are shown in Figure 42. In three tests the veneer collapse occurred in the body of the veneer, whereas in the fourth test (Wall W12) the top portion of bricks toppled in a cantilever action. In a previously tested series of eight walls (Thurston and Beattie 2011), six of the walls toppled in a cantilever action and only two failed in the body of the walls.

Generally veneer shedding was preceded by a group of ties pulling out of the mortar. However occasionally the ties partially pulled out under tension, jammed and then buckled under subsequent compression load, and then finally ruptured under a buckling fatigue mode.

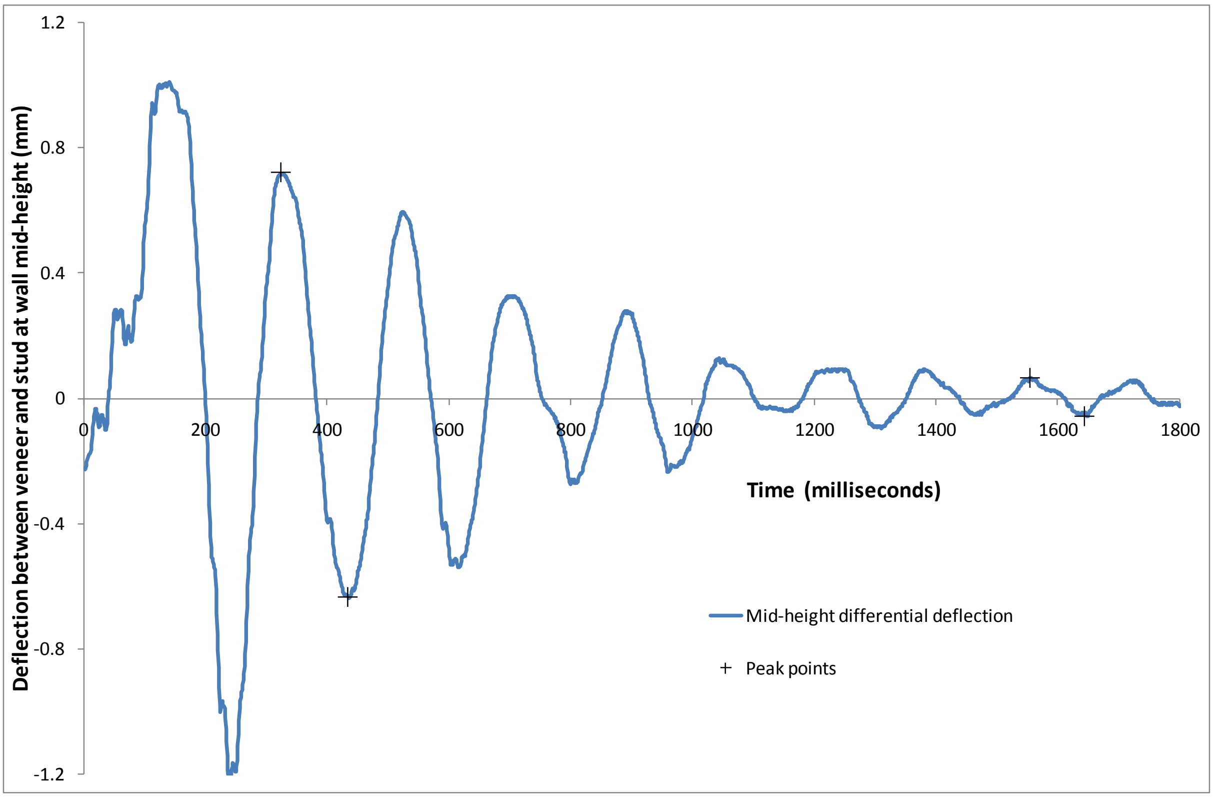


Figure 39. Free vibration wall response

5.6.2 Response spectra

Although the sinusoidal accelerations applied in the tests are vastly different to the signals from real earthquakes, it is possible to correlate results obtained from these tests with expected performance in an earthquake. This is because the panels in any test behave in an essentially elastic manner (i.e. natural frequency and damping remain constant). Thus, if these values are known, then the veneer accelerations (and hence seismic coefficient) can be computed from the measured table accelerations or else measured directly from accelerometers on the veneer.

The applied table displacements were a smooth sinusoidal record. However, due to friction effects the table acceleration was a rougher signal, which was not necessarily a disadvantage. The acceleration record was used to compute acceleration spectra as shown in Figure 43. A generalised shape of these spectra at successive stages of test is shown in Figure 40. It consists of the following components:

- Line A, which defines the limitation on the final spectra due to the maximum velocity which is able to be imposed on the BRANZ shake table. Thus, at the lower frequencies the maximum spectra was limited by the table itself rather than the failure of the test wall.
- Apart from the limitations of Line A, the table displacement was selected to give a constant horizontal line for each stage of test shown in Figure 40. However due to table feedback there was actually some increase in the spectra near the wall's natural frequency.

- Between 6-8 Hz the spectra tended to increase. This was again attributed to feedback from the table and end support frame. As this frequency range was not critical to the performance of the test walls, this feature is considered to be of little consequence.

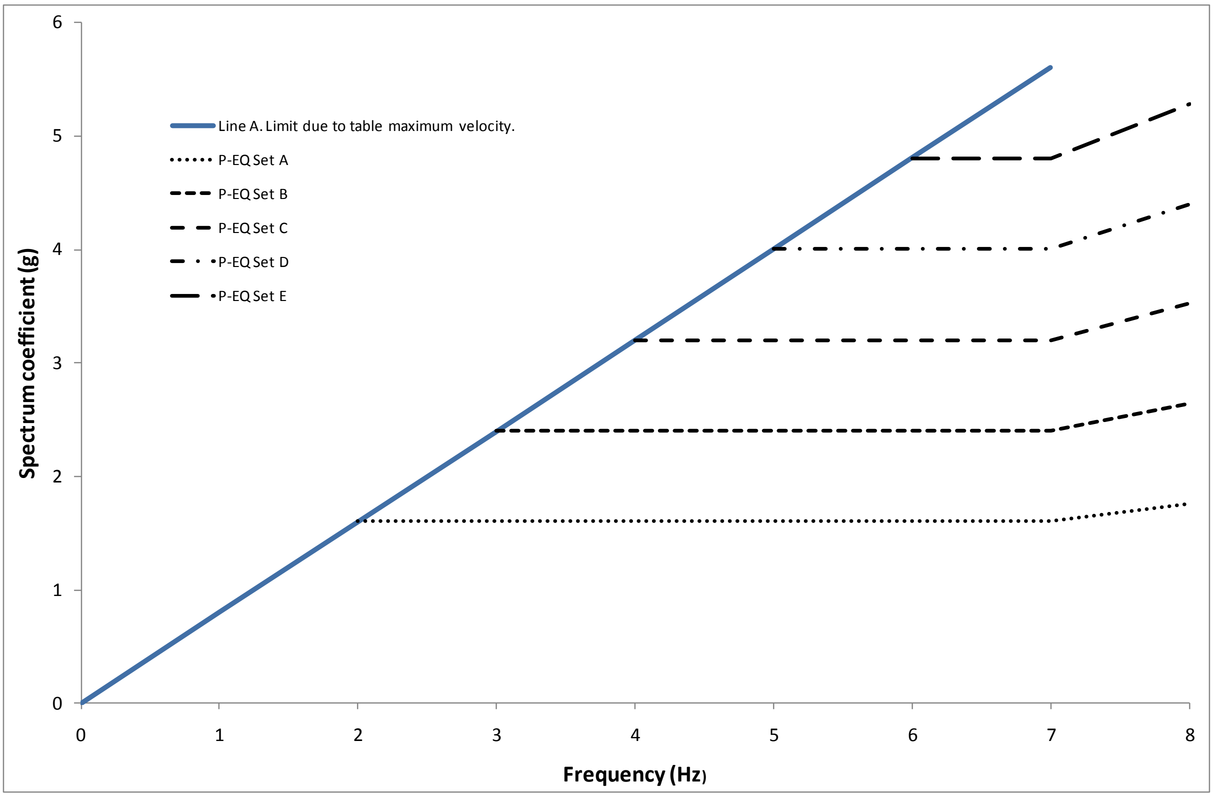


Figure 40. Typical spectra calculated from measured table accelerations

The spectra presented herein were derived for 5% damping as this is what was assumed when the NZS 1170.5 design spectra were derived. Measured wall damping given in Section 5.5.4 was lower in the early stages of testing (which would have increased the derived spectra) but higher just prior to wall failure.

For each wall the acceleration records were used to compute acceleration spectra for each P-EQ from Set A up to the instance of failure. Figure 43 presents the results for Wall W9. This failed during the shaking at 3.5 Hz in P-EQ Set F and thus the trace for Set F stops just prior to this frequency. For each wall an envelope of spectra was derived from all shaking prior to failure.

An arrow has been drawn on Figure 43 joining the tops of a “hump” which would appear to be a resonance effect at the natural frequency of the wall. There is a second “hump” at higher frequencies as discussed three paragraphs above.

The envelope of spectra imposed on each of the four walls is plotted in Figure 44. Also plotted on this figure is the design shear value for both single and two-storey buildings calculated as described in Section 5.4 using NZS 1170.5. The four walls each have very different maximum spectra with Wall W9 being the best and Wall W12 the worst.

With the exception of Wall W12, all walls resisted spectra greater than the design spectra for single-storey buildings. The seven day bond strength for Wall W11 was 144 kPa and for Wall W12 was 123 kPa. It was concluded that seven day mortar bond strength over 200 kPa was suitable for single-storey buildings.

Wall W9 and W10 resisted spectra greater than the design spectra for two-storey buildings. The seven day bond strength for Wall W9 was 455 kPa and for Wall W10 it was 430 kPa. It was concluded that seven day mortar bond strength over 500 kPa was suitable for two-storey buildings.

6. DISCUSSION AND CONCLUSIONS

6.1 Comments on AS/NZS 2699.1

Currently brick veneer ties are classified as light, medium or heavy duty depending on their performance in the tests to Appendix A of AS/NZS 2699.1 (SNZ 2000). Table 2.3 of NZS 4210 (SNZ 2001) stipulates the tie classification required for a particular building as a function of seismic zone, veneer weight and tie spacing. This was determined by ensuring the tie strength (as measured by the AS/NZS 2699.1 tie test) exceeded the seismic demand force on the tie. This is defined as the tributary weight of the veneer per tie (i.e. weight of an area of veneer = tie vertical spacing x tie horizontal spacing) factored by a horizontal seismic coefficient which varies with seismic zone. Hence, if the coefficient is selected on sound principles the tie connection should not fail under face loading in a design earthquake.

Limitations of the test method in AS/NZS 2699.1 are discussed below:

1. Masonry units. The strength determined from the test depends on the mortar bond strength. The method does not specify whether clay, concrete, calcium silicate or AAC units should be used. Tie manufacturers have generally only tested to AS/NZS 2699.1 using clay units which have core holes. The tie classification resulting from these tests is then applied to construction using all types of masonry units. Note that most concrete units do not have core holes. As some can have a dusty surface, they may therefore have lower bond strengths.
2. Mortar. The test method does specify the ratios of the mortar components although the sand grading, water content, flow and bond strength are not specified. Bond strength is very dependent on the flow. It is recommended that the test method specifies a mortar bond strength not exceeding 220 kPa and that the test be performed with clay-brick units with core holes no wider than the bricks may be in practice. If NZS 4210 is modified to require single-storey construction to achieve a bond strength of at least 200 kPa, then good tie performance can be assured in a design earthquake.
3. Dry embedment. As most veneer is currently constructed using dry-bedded ties, despite NZS 4210 requiring full embedment, the tests to AS/NZS 2699.1 should be performed using dry embedment. Practical difficulties arise on building veneer with full tie embedment as discussed in Section 2.1.3 of this report. It is recommended that NZS 4210 allow dry tie embedment as long as AS/NZS 2699.1 also calls for dry embedment.
4. In BRANZ shake table tests on brick veneer walls it was observed that ties tended to pull out of cracked joints as the cracks opened due to wall out-of-plane flexural movement (see Figure 41). The test set-up does not allow joints to crack in AS/NZS

2699.1 tests. Thus, ties can pull out of joints more readily in practice than simulated in the AS/NZS 2699.1 tests. Small corrugations and protrusions in the tie are not very effective in a cracked joint. A better simulation of a cracked joint in a revised AS/NZS 2699.1 test is likely to result in use of ties with deeper corrugations or an upturned end. However, although desirable, devising such a test will be difficult.

5. Section A7.3 of AS/NZS 2699.1 requires a compressive pressure between 10 and 100 kPa be placed across the mortar joint of the test specimen. This is a huge range, which can significantly affect test results. It is equivalent to a range of approximately 0.5 to 5 m of veneer weight above the joint. For a tie of area of say 50 x 20 = 1000 mm^2 in the joint, a pressure on the tie of 100 kPa, assuming a friction coefficient of 1.0, will resist a sliding force of 2 x 1000 x 100^{-6} = 0.2 kN, which is 40% of the required strength for EL ties. However, in most laboratory tests the level of compression is not measured and the equipment used may strongly resist any expansion of the joint, which means that corrugated or dimpled tie shapes need to ream out the mortar or the brick surface before they will pull out. The actual pressure on the tie may therefore be greater than 100 kPa at tie pullout. In actual construction some joint expansion is possible meaning that the high pressure cannot be maintained. It is recommended that AS/NZS 2699.1 stipulates the compression pressure across the mortar joint be 20 kPa, require this to be monitored during the test (or use a dead load), and for the set-up not to impose additional restraints on the joint expanding.

6.2 Discussion of parameters affecting brick veneer seismic performance

Based on the writer's previous testing (Thurston and Beattie 2009), it is considered that the performance of brick veneer under face load seismic forces is more important than its performance under in-plane seismic load. The critical parameters for good performance of New Zealand brick veneer walls under face loading are discussed below.

6.2.1 Spacing of the ties

NZS 4210 (SNZ 2001) stipulates that ties between timber framing and brick veneer are fixed to each stud at a tie spacing not exceeding 400 mm. The maximum contributing area is therefore 0.6 x 0.4 = 0.24 m^2. No change is recommended to this requirement.

6.2.2 Brick-to-mortar flexural-tensile bond strength

This research has shown that the critical parameter for obtaining good mortar-to-brick bond strength is the mortar flow (i.e. how far a cone of fresh mortar will spread when vibrated). Best bond is obtained when the mortar is at the maximum flow before detectable bleeding begins but still being workable by the tradesperson. A literature search shows others have come to a similar conclusion (see Section 2.1.5). Wetting bricks before they are laid, pressing and tapping the bricks to firmly embed them in the mortar, not dislodging the bricks once placed, minimising the time between spreading mortar, and placing the bricks and tooling the mortar joints all increase this bond strength. In hot dry weather, adequate curing of the freshly constructed veneer should be mandatory. Clay and silt in the mortar can increase workability, but these and any organic material reduce bond strength.

BRANZ tests showed that mixing mortar too long in the barrow, and also using excessive admixture quantities compared with manufacturer's recommendations, increased the air content of the mortar and reduced the mortar-to-brick bond strength.

This research found that the in-plane loading sometimes induced cracks in the mortar. Such cracks can result in early tie pullout and thereby early veneer collapse. By comparing mortar bond strength used in the test walls with wall resistance to the shake table loading the following recommendation was made:

> "To ensure a brick veneer complying with NZS 4210 can resist the New Zealand demand earthquake loads without any masonry shedding, brick veneer should use a mortar with minimum seven day bond strength of:
>
> (a) Single-storey brick veneer: 200 kPa
> (b) Two-storey brick veneer: 500 kPa".

6.2.3 Strength of the connection between brick ties and the brick veneer

Only brick ties which have had the strength/stiffness of their connection between timber framing and brick veneer tested to AS/NZS 2699.1 may be used in construction to NZS 4210. As discussed in Section 6, the results from the AS/NZS 2699.1 test, veneer weight and seismic zone determine whether a specific tie may be used on a specific project.

NZS 4210 stipulates that ties must be screwed to studs. With most ties on the market a robust Tek screw is used, and the writer has not encountered a failure with this connection either in the field or in the laboratory.

Tie rupture is also rare, and the writer has only observed it to occur after the tie has partly pulled out of the mortar and then jammed on the reverse compression load, buckled and subsequently suffered low cycle fatigue failure.

The weakest link between veneer and frame is the tie pulling out of the mortar. In the face load veneer shake tests, it was noted that brick ties tended to pull out of the mortar at a mortar crack. When this crack opens up as the wall flexes, a tie located at this cracked joint does not need to ream out a slot of mortar or brick to withdraw from the joint as shown in Figure 41. Nevertheless, brick tie-to-mortar bond strength is expected to be an important parameter because most of the brick veneer face loaded test walls failed because the brick ties slid out of the mortar joints.

The laboratory tests described in this research showed that full tie embedment within the mortar improved the tie pullout resistance, although in practice such construction is difficult to build. Dry embedment is the common practice.

The elemental test results showed only moderate correlation between mortar-to-tie and mortar-to-brick bond strengths. There was a poor correlation between mortar-to-tie and mortar compressive strengths (whereas in reinforced concrete there is a good correlation between reinforcing bond strength and concrete compressive strength).

Instead of trying to increase the tie-to-mortar bond strength, it may be more effective to upturn the tie end lips (say 6 mm high) or have deep corrugations in the tie and thus mobilise a mechanical connection.

6.2.4 Mortar properties

6.2.4.1 Mortar workability

It is important that mortar has adequate workability, defined as the ease of spreading the mortar and its cohesiveness. Cement, lime and most commercial admixtures increase workability, particularly when the sand is coarse. The research has shown that the sand used, while important for good workability, was not critical to the bond strength.

Paradoxically, the sand considered worst by the tradesman gave high mortar bond strength, because to achieve a suitable workability he was required to add more water and hence a high mortar flow was achieved.

6.2.4.2 Mortar compressive strength

The elemental test results showed that increases in cement content increases mortar compressive strength. Mortar compressive strength had poor correlation with the mortar-to-brick bond strength. Mixing the mortar too long in the barrow, and also using excessive admixtures compared with the manufacturer's recommendations reduced the mortar compressive strength.

Specifying mortar suitability on compressive strength alone cannot be relied on to yield adequate masonry bond strength. However, it would be imprudent to suggest that the cement content of mortar can be reduced as the cement plays a major part in the mortar durability.

NZS 4210 (SNZ 2001) calls for a compressive 28 day mortar strength of 12.5 MPa for structural compliance with NZS 3604, although this is not intended to apply to veneers. The standard also states that the strength of mortars for veneers shall follow the requirements of masonry suppliers. Some of these specify strength of 12.5 MPa, and others rely on the mortar mix compositions listed in Table 2.1 of NZS 4210. The minimum compressive strength needs to be clarified in future revisions of the standard. It is recommended that the mortar compressive strength of brick veneer be at least 6 MPa to maintain an acceptable level of durability.

7. GENERAL RECOMMENDATIONS FOR REVISION OF STANDARDS

7.1 NZS 4210

7.1.1 Tie embedment

It is recommended that dry tie embedment be allowed, provided AS/NZS 2699.1 also requires the tests to be conducted using dry-bedded ties.

7.1.2 Curing of newly placed veneer

It is recommended that in hot dry weather, adequate curing of the freshly constructed veneer should be mandatory. The method to achieve this needs to be specified.

7.1.3 Mortar bond strength

It is recommended that brick veneer should use a mortar with minimum seven day bond strength of:

(a) Single-storey brick veneer: 200 kPa
(b) Two-storey brick veneer: 500 kPa.

7.1.4 Mortar compressive strength

The minimum compressive strength of mortar for brick veneer needs to be clarified as discussed above. ***It is recommended*** that the mortar compressive strength of brick veneer be stipulated to be at least 6 MPa.

7.1.5 Quality assurance of mortar bond strength

It has been shown that bond strength is mainly influenced by mortar flow. On this basis the following recommendation is made:

It is recommended that bond strength is measured for each new brick or mortar combination used on-site to ensure the target bond strength results are achieved. Also, each tradesperson should be required to have the bond strength measured every 12 months. The couplets should be made on-site, left on-site under shelter (but not under plastic) to cure for seven days and then tested by an accredited laboratory. At the same time as these test samples are made, a slump test should be preformed and the spread recorded. The tradesperson should daily perform the same slump test on-site to ensure the spread matches or exceeds that obtained when the mortar bond strength was measured.

Although the equipment to do the mortar bond test on-site is simple, it is not considered appropriate for the tradesperson to do the bond tests as test results are sensitive to handling and the mortar-brick bond test results show a large scatter. The bond strength depends to some extent on the curing conditions (temperature and humidity). It will be difficult to remove this variable in practice without undue expense. ***It is recommended***, therefore, that daily slump tests (but not bond tests) are carried out.

7.2 AS/NZS 2699.1 test method

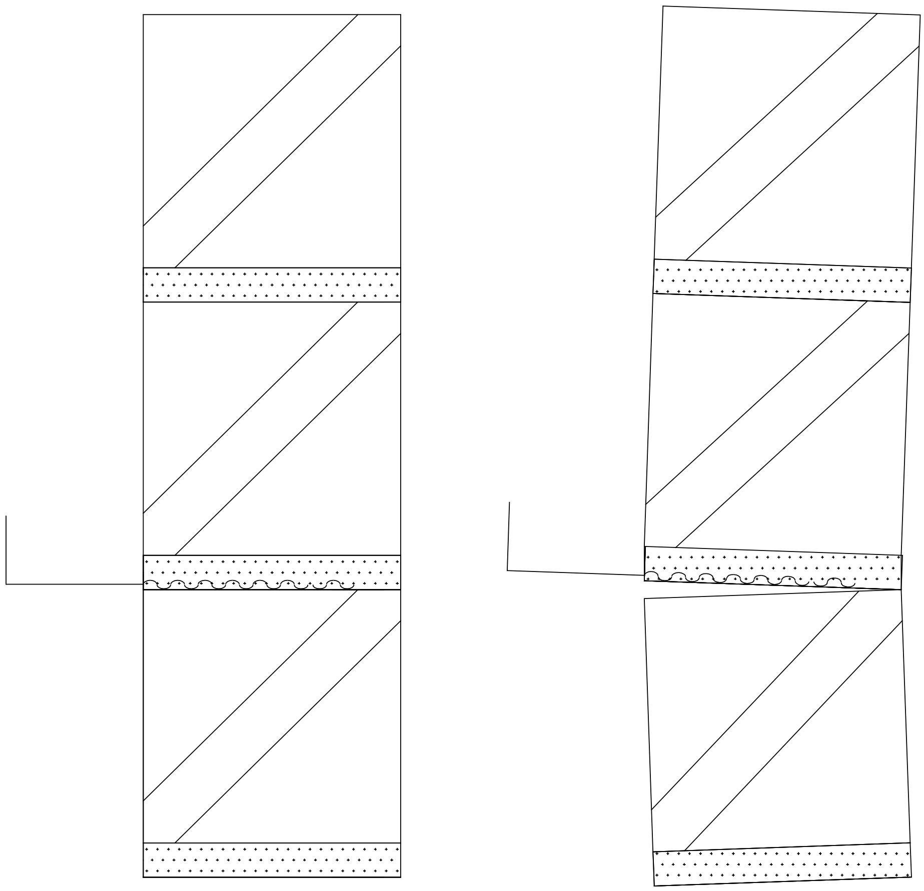
It is recommended that the test method specify a mortar bond strength not exceeding 200 kPa and the test be performed with solid clay-brick units with core holes. The bricks should have the minimum width they can have in practice – eg 70 mm. NZS 4210 should also be modified to require single-storey construction to achieve a bond strength of at least 200 kPa.

It is recommended that the test method in Appendix A reduce the compressive pressure across the mortar joint to 20 kPa, require this to maintained during the test, and for the set-up not to impose additional restraints on the mortar joint expanding.

7.3 Future research

It is recommended that a study be performed to make tests to Appendix A of AS/NZS 2699.1 a more realistic simulation of seismic loading on a brick tie and to take into account the potential for a mortar crack opening up during a seismic loading of actual construction.Thus, the cracks will be able to open up in the test as shown in Figure 41. This may lead to future ties having end lips (say 6 mm high) or having deep corrugations.

The test stiffness and strength pass/fail criteria should be examined and the method should be made applicable to both timber and steel wall framing.



(a) Tie pullout no wall flexing (b) Tie pullout with wall flexing

Figure 41. Influence of wall flexing on tie pullout from the mortar bed for dry-bedded construction

Figure 42. Photographs of veneer shedding mechanisms for the four test walls

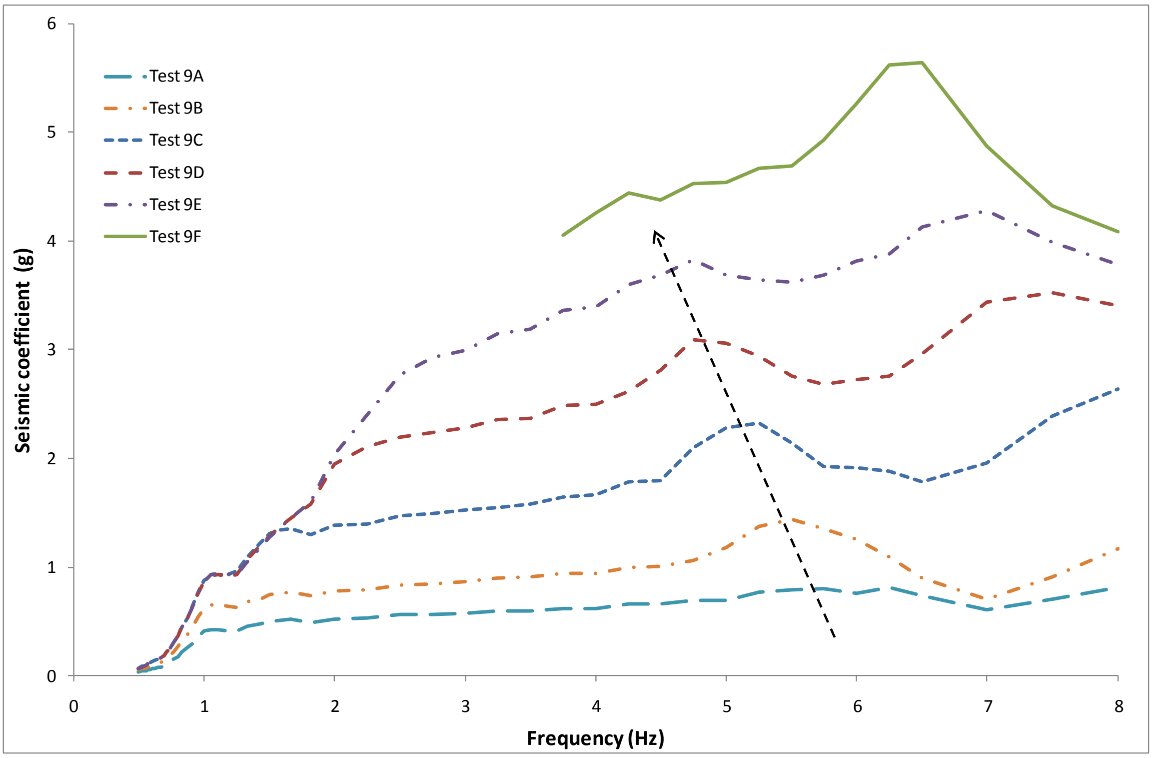


Figure 43. Spectra calculated from measured table accelerations for Wall W9

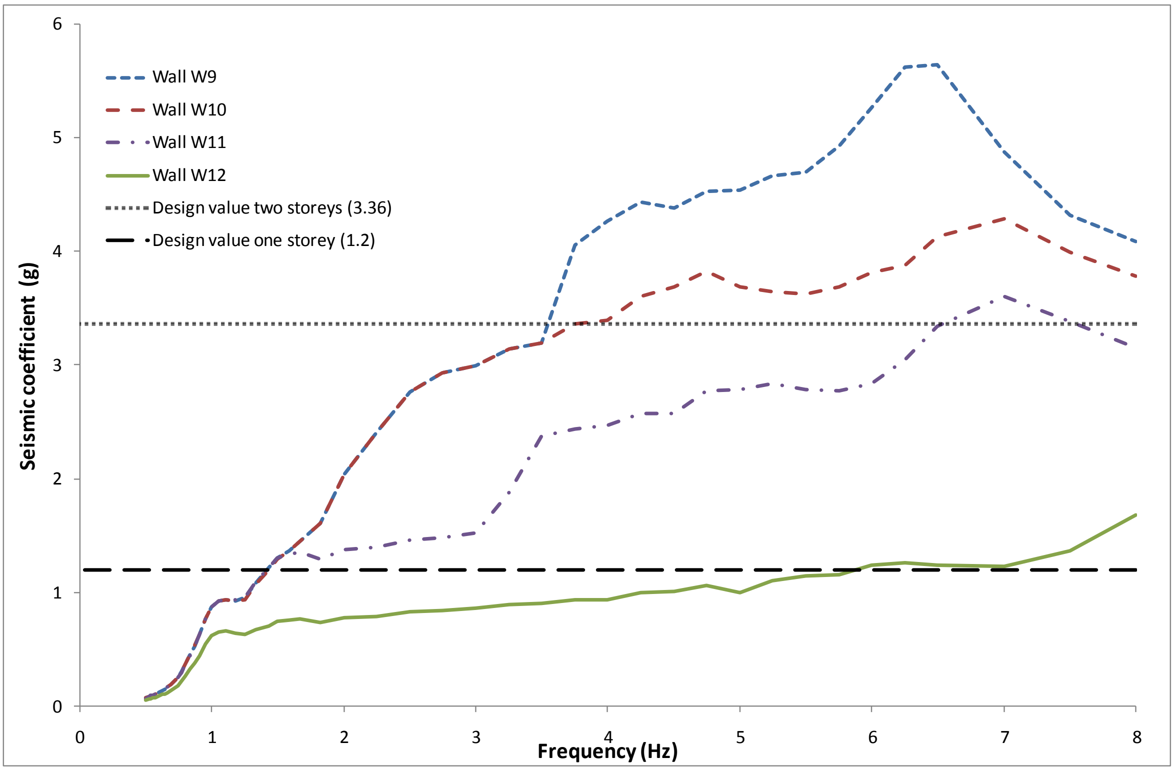


Figure 44. Maximum spectra calculated from measured table accelerations for all walls

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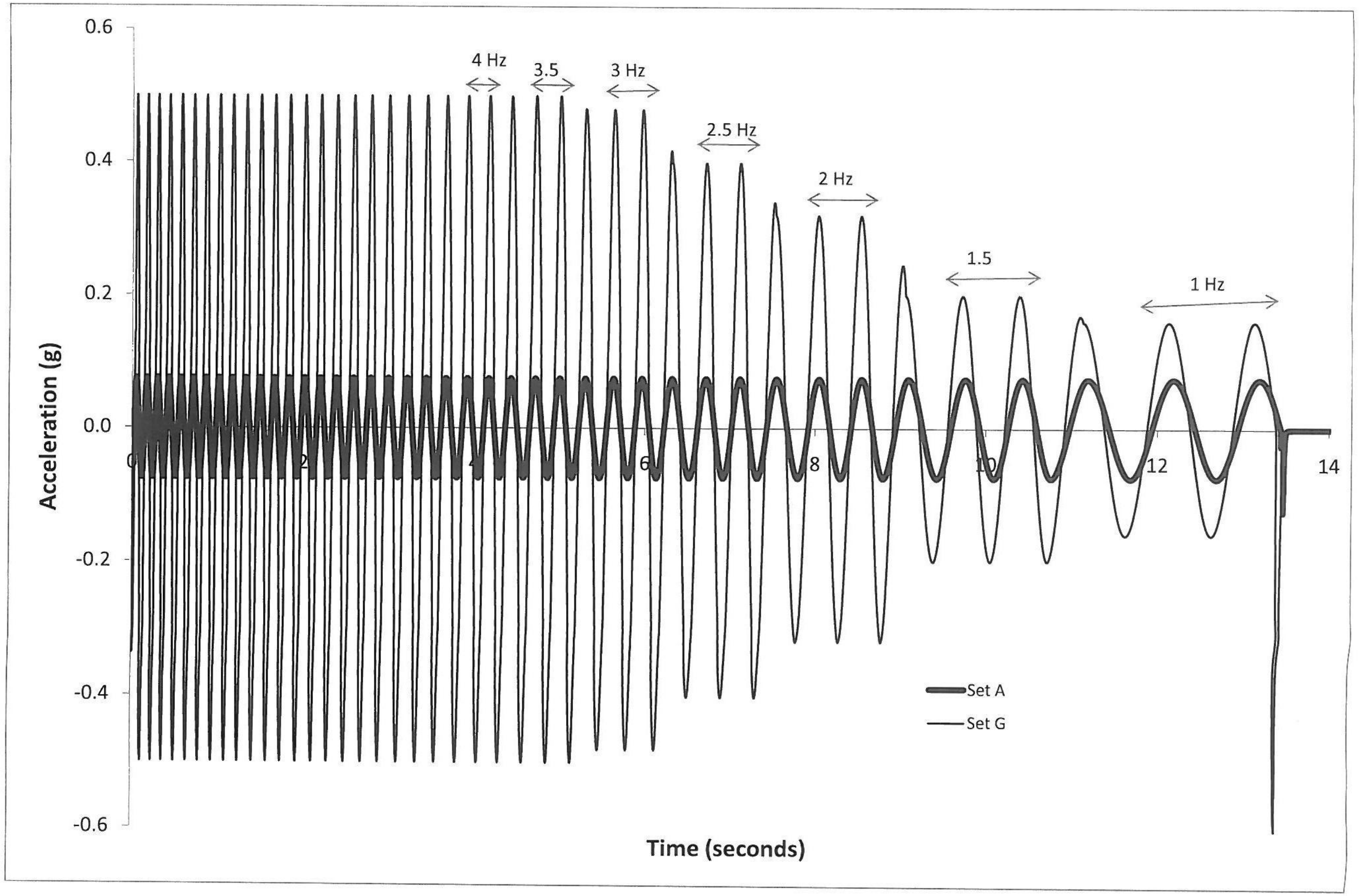

Figure 45. Full acceleration excitation record

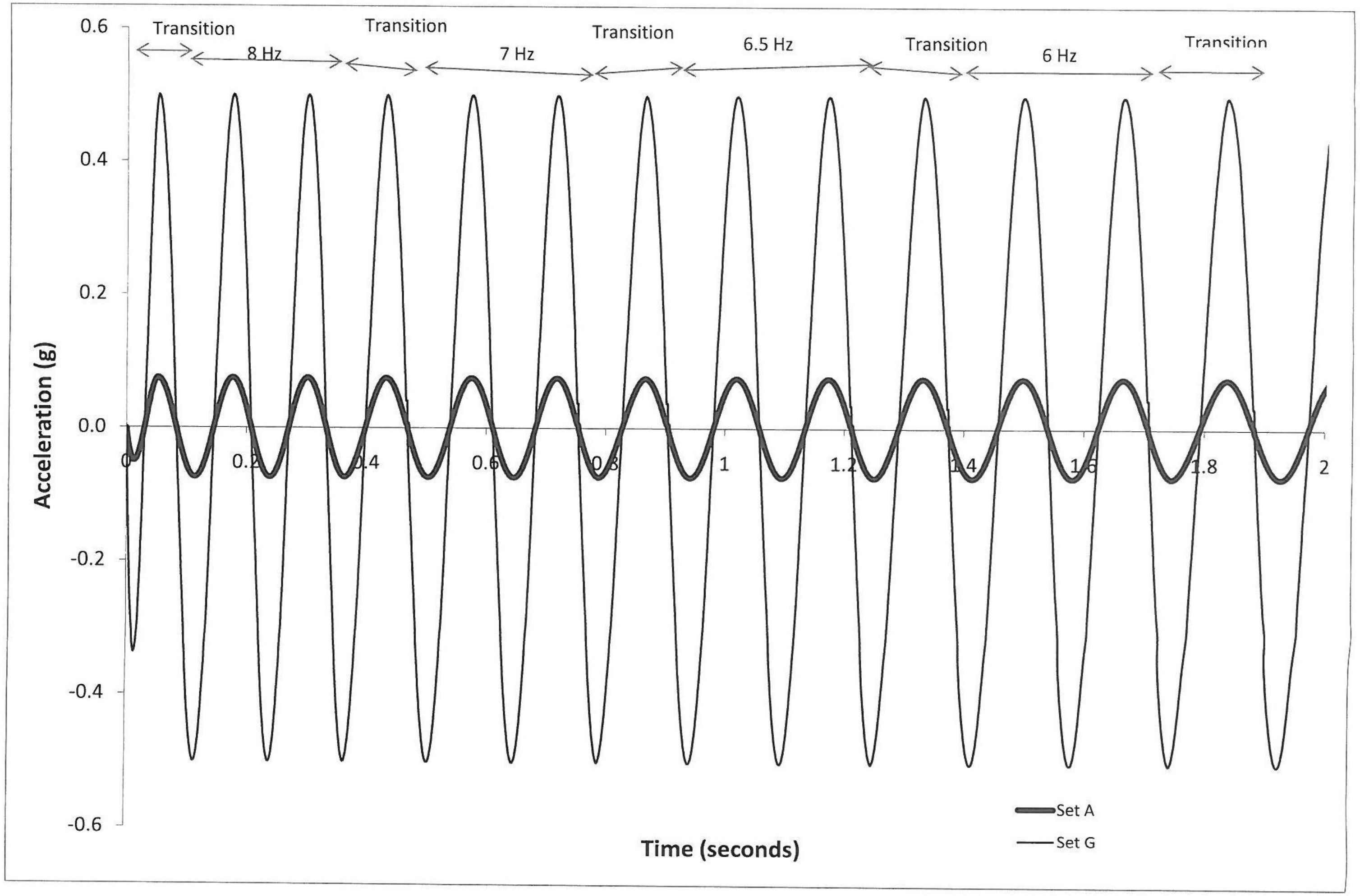

Figure 46. Initial portion of acceleration excitation record