

DYNAMIC PERFORMANCE OF BRICK MASONRY VENEER PANELS

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SYNOPSIS

Dynamic tests on seven unreinforced and two reinforced brick masonry veneer panels are reported. The panels, tied to conventional timber-frame backings, were subjected to inertial loading by sinusoidal accelerations within the expected range of seismic frequencies. Variables investigated included stud spacing, veneer-tie type, and initial distribution of preformed cracking. Results indicate that properly constructed brick masonry veneers can withstand seismic attack levels substantially in excess of that implied by NZS 4203(1) for Zone A, with only minor cracking.

1. INTRODUCTION

Brick masonry veneer tied to timber frame backing is in widespread usage for domestic dwellings in many parts of New Zealand. This form of construction integrates two readily available local materials in an economical and maintenance free manner. The role of the masonry veneer is non-structural, as its purpose is to act solely as a weatherproof surface, and the timber framing provides the necessary strength to support gravity loads, and lateral loads from wind and seismic inertial effects. Consequently the veneer need only sustain the forces developed in transmitting the lateral loads from wind-storm and seismic attack to the timber framing. As forces developed under design earthquake levels greatly exceed those resulting from design wind storm⁽¹⁾, discussion will be restricted to the former in this paper.

Despite the apparently simple functional requirement, the reputation of unreinforced masonry veneers under seismic attack is poor. Much blame for the lack of public and professional confidence in unreinforced masonry can be traced to the failures of brick masonry facades and walls in the 1931 Napier earthquake, and to damage caused to a few brick veneer houses in the 1968 Inangahua earthquake⁽²⁾. The Napier failures represent inadequate performance of a construction method no longer used in New Zealand, and widely recognised as unsafe for seismic regions, while the Inangahua houses, built in the 1930's, did not conform to modern veneer dwelling requirements in terms of veneer ties, timber frame stiffness, detailing, and foundation construction practice.

The series of tests reported in this paper was initiated in order to provide more realistic data on which to base assessment of the likely performance of modern brick masonry veneer structures under seismic loads. The programme was primarily directed towards the response of unreinforced brick masonry veneers, as this represents past and present construction practice for domestic dwellings. However, limited testing of reinforced veneers was carried out to investigate the possibilities for masonry veneers in commercial buildings of more than single-storey heights.

2. DESIGN LOADS FOR VENEER PANELS

Lateral forces are induced in veneer panels under seismic loading due to inertial response of the heavy veneer material. Face loading is of prime concern, as the timber frame bracing required by the appropriate code of practice⁽³⁾ is intended to carry the entire in-plane shear loading resulting from inertia of roof and walls. The face loads are transmitted back to the timber framing by suitable wall ties. However, as a result of high in-plane stiffness of the brick masonry veneer in the uncracked state, relative to the timber framing stiffness, it is probable that considerable initial in-plane (racking) load will be carried by the veneer. This may be sufficient to induce shear cracking of the veneer panel, possibly affecting the ability of the veneer to support the inertial face loads. After cracking under in-plane forces, the contribution of the veneer stiffness to in-plane load transfer is unlikely to be significant.

Satisfactory performance of a masonry veneer under seismic attack requires that the veneer should survive the design-level earthquake without collapse. Although not clearly defined in appropriate codes of practice, it is assumed in this paper that the design-level earthquake corresponds to that specified for a building in the same seismic zone by the current loadings-code⁽¹⁾. Failure could occur by fracture or pull-out of veneer wall ties, by failure of the timber framing in flexure, shear or joint collapse, or by shedding masonry units without prior failure of veneer ties or framing. Cracking of the veneer under the design-level earthquake cannot be taken to constitute failure, as this would imply a more severe requirement than demanded of engineered structures. Repair of a competent, but cracked veneer after an earthquake should merely be a matter of repointing.

3. TEST PANELS - DETAILS

The veneer panels tested in this programme were designed to simulate actual anticipated conditions as closely as possible. This required realistic construction and support details, and simulation of effects of possible initial cracking resulting from settlement, prior face loading, or racking load, as described above.

All panels were 2.44m tall, corresponding to a typical domestic storey-height veneer, were attached to conventional timber backings, and were supported on concrete foundation beams. Figs. 1 and 2 show typical construction details adopted, and Table 1 gives details of the individual wall panels. Variables investigated included the

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influence of presence and type of preformed cracking, stud spacing, connection details, and reinforcement.

3.1 Preformed Cracking

Of the unreinforced veneers, two (UV3 and UV3R, see Table 1) were tested without preformed cracks. The remaining five panels were constructed with preformed zero tensile bond strength beds (and perpend, where appropriate) to simulate the existence of cracking. Panel UV4R contained three horizontal zerobond courses, at approximately $\frac{1}{4}$ height, midheight, and $\frac{3}{4}$ height of the panel, while the remaining four walls contained diagonal zerobonds passing through the wall centre, to model shear cracking resulting from settlement or in-plane seismic loading. The zerobonds were achieved by liberally coating the laid bricks with hydraulic oil before placing the next mortar bed or perpend, thus preventing formation of bond by inhibiting the brick absorption of water and cement paste from the mortar. Tests on brick couplets made in this manner confirmed that zero tensile bond strength was in fact achieved.

3.2 Timber Frame - Stud Spacing

The veneers were tied to a conventional plate/stud/dwang timber frame. Each frame contained four vertical studs spaced at either 600mm (panels UV3, UV3R, UV4R, UV5, UV5R) or 400mm (UV8, UV8R), and was constructed from dressed 100mm x 50mm dry No. 1 framing grade Pinus Radiata. The studs were framed with top and bottom plates, and three equally spaced rows of dwangs. Veneer panel width was 1880mm for 600mm stud spacing and 1460mm for 400mm stud spacing.

3.3 Connection Details

In preliminary tests on brick masonry veneers (4) collapse had eventually occurred as a result of stud/plate and tie/stud connection failures. In order to further investigate the veneer strength, panels in the UV series were constructed using special ties (see fig. 2a) to avoid slippage and staple withdrawal associated with the No. 8 gauge wire ties used in the preliminary tests. Angle brackets (fig. 2a) were used at the stud/plate connections to prevent nail shear failures which had occurred in one preliminary veneer test. In contrast to the special connection details of the UV panels, all UVR panels were built following current trade practice, using face nailing veneer tiles, and stud/plate connections of two 100mm x 4.5mm steel nails per connection, skew nailed at the bottom plate, and vertically nailed through the top plate. Details are illustrated in fig. 2.

In all panels, veneer ties were attached to each of the studs. For the UV series panels average vertical spacing of veneer ties in the central region of the wall was six courses, while the conventional ties for the UVR series were spaced at 4 courses (340mm centres).

The UV and UVR series of panels also differed in design of the base simulating the veneer foundation wall. UV panels were constructed on a 150mm x 75mm steel channel filled with concrete, which was simply supported at each end, spanning a distance of 1950mm. Prior to testing it was observed that a crack existed in the base mortar bed, due to deflection of the foundation beam under the weight of the veneer. A stiffer base made from a 150mm x 150mm steel UC section was

used for the UVR walls, eliminating the base crack. Base details are included in fig. 2.

3.4 Reinforced Veneers

Panels RV1 and RV3 (see Table 1) each contained four grade 275 D10 deformed reinforcing bars within the cells of reinforceable veneer bricks. To simulate realistic construction details, a 600mm lap was provided at the wall base with starter bars threaded and bolted to the bottom of a foundation beam similar to that used for the UVR series walls. Cells containing reinforcing steel were filled with a fine grout in three lifts during construction, with construction joints arranged to not coincide with mortar joints.

Both reinforced veneers used the conventional construction techniques for framing and connection described above for the UVR series panels. The panels differed in stud spacing (450mm and 600mm) and the inclusion of three horizontal zerobonds in the mortar courses of panel RV3 in the vicinity of the wall midheight.

4. MATERIAL PROPERTIES

4.1 Bricks

All unreinforced veneers were constructed from AB Brick Ltd Standard Golden Buff 8 in. x 4 in. x 3 in. (200 x 100 x 75mm) brick. The two reinforced veneers were constructed from McSkimmings Industries Ltd half-reinforcing veneer brick (230 x 110 x 70mm).

4.2 Mortar

Mortar used for the unreinforced veneers had a composition of

$$\text{sand} : \text{cement} : \text{lime} : \text{water} = 5 : 1 : 0.67 = 1.37.$$

This mix was selected from a series of trial mixes and was intended to achieve a brick/mortar tensile bond strength close to the (then) specified minimum of 210kPa. In fact, considerable difficulty was experienced in obtaining a bond strength this low. The mortar mix used for the reinforced veneers was similar to the above, having a composition of

$$\text{sand} : \text{cement} : \text{lime} : \text{water} = 5 : 1 : 0.48 : 1.27.$$

The slight modification from the mix for the unreinforced veneers was necessary to compensate for the different absorption properties of the bricks. Brick/mortar bond strength was tested for each panel, using brick couplets and the method specified in reference 10. Results are summarized in Table 1.

4.3 Grout

Grout composition by weight for the two reinforced veneers was

$$\text{Aggregate} : \text{cement} : \text{water} = 4 : 1 : 0.82.$$

This mix was selected after experimentation to obtain a slump value greater than 125mm and a spread less than the maximum value of 530mm allowed by reference 10. Because of the small cell size of the reinforceable bricks used, a maximum aggregate size of 4.75mm was adopted. Strength results, based on 150mm x 75mm dia. cylinders tested at 28 days are included in

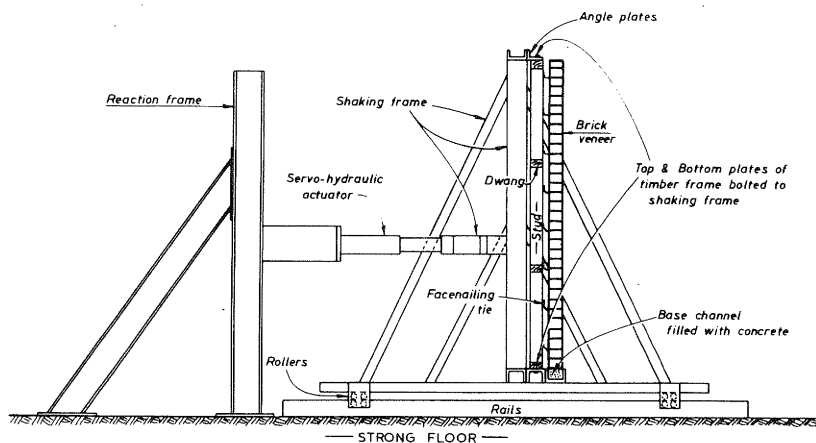
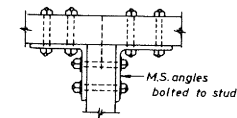
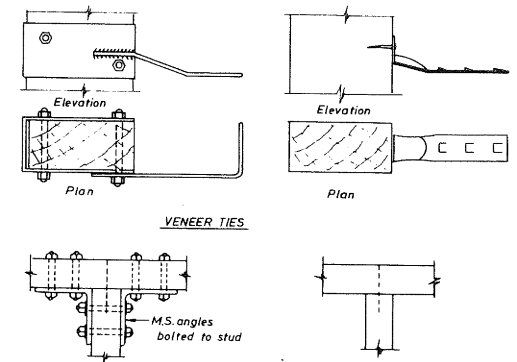
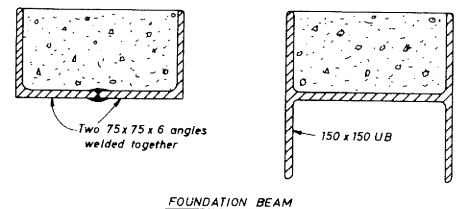


Fig. 1 Veneer Test Set Up



STUD/TOP PLATE CONNECTION



(a) UV SERIES PANELS

(b) UVR SERIES PANELS

Fig. 2 Construction Details

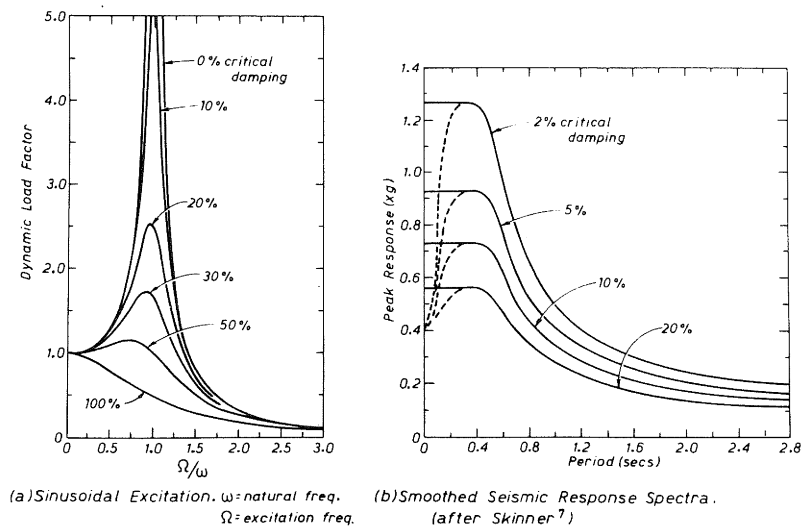


Fig. 3 Response of Single degree of freedom Oscillators

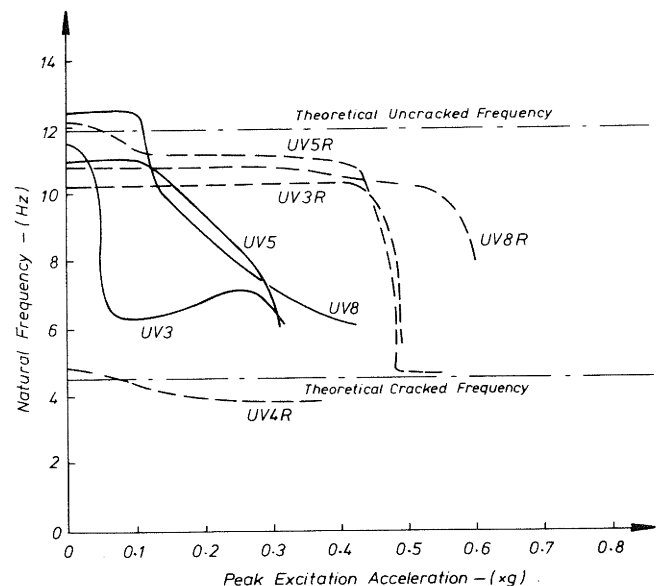


Fig. 4 Natural frequencies, Unreinforced veneers

Table 1. For both panels the grout strength was less than the minimum value of 17.5 MPa specified in reference 10. To investigate the significance of this, panel RV1 was broken after completion of testing to expose the grout cores. The grout was observed to be dense with a strong bond to the brickwork. A construction joint was inspected, and appeared to have ample bond.

More complete details on panel construction are included in refs. (5) and (6).

5. METHOD OF TESTING

The timber framing of the veneer panels was bolted through the top and bottom plates to the specially constructed shaking frame shown in fig. 1. Dynamic loading was applied by a 50 kN capacity double-acting servo-hydraulically controlled actuator, driving the frame at the appropriate level of the centre of gravity of the frame and panel. The shaking frame ran on wheels on an I-beam rail system bolted to a prestressed concrete structural strong floor (see fig.1).

Load was applied to the shaking frame as a series of short bursts of sinusoidal acceleration input of gradually increasing amplitude, at 5.0 Hz frequency. Condition of the panel, response accelerations, and natural frequency and damping, were recorded for each level of acceleration input. The excitation frequency of 5.0 Hz was chosen because this is a frequency typical of the major energy content of recorded earthquakes, and also because calculations indicated that 5.0 Hz was likely to be close to the natural frequency of the panels subsequent to veneer cracking. This enabled large response acceleration to be obtained for modest input accelerations.

Although real earthquakes are not characterised by simple sinusoidal accelerations, it is possible to directly correlate results obtained from these tests with expected performance in a given earthquake. This is because response of the panels is essentially elastic, though the dynamic elastic characteristics (natural frequency and damping) may vary as a result of cracking of the veneer panel. If these characteristics are known, the required elastic response of a veneer panel under a specified earthquake may be assessed from the acceleration response spectra for the earthquake. This required value can be compared with peak response accelerations obtained during sinusoidal testing either measured directly by accelerometers or displacement transducers, or calculated from theoretical response spectra for sinusoidal excitation, such as shown in fig. 3a.

For interpretation of these tests, the smoothed composite earthquake response spectra proposed by Skinner(7), and shown in fig. 3b was adopted, since they form the basis of the seismic loading provisions in NZS 4203 : 1976⁽¹⁾, particularly for short period structures. The response curves are modified as shown to have flat plateaux for periods lower than the peak response, in the same fashion adopted for the design coefficient curves of reference(1). Predicted responses based on measured frequency and damping and the Skinner spectra of fig. 3b, are thus referred to as Zone A design earthquake responses. Earthquake shaking of this intensity has been estimated by Smith(8) to have a typical return period of about 150 years in Zone A, and considerably longer return periods for most locations in Zones B and C.

6. EXPERIMENTAL MEASUREMENTS

Bursts of sinusoidal input acceleration were imposed on the panels in increments of approximately 0.04g intensity. During each test measurements of shaking frame displacements, absolute panel displacement at midheight and panel top, and relative timber frame/wall displacement at midheight were made with strain-gauged deflectometers. At the end of every second acceleration increment the shaking frame was clamped to the strong floor and the veneer firmly hit at midheight. Natural frequency and damping were obtained from measurements of the resulting decay response.

After each acceleration burst the panels were examined and a photographic record taken of any damage. Cracks in the veneer were visually emphasized using a felt pen, and damage to the veneer ties indicated by circles drawn at the appropriate location on the front of the veneer. Dotted circles were marked at the first sign of tie movement, and full circles were marked when it was considered that substantial movement of the tie would take place before it could apply any restraining force. Testing proceeded as bursts of increasing amplitude until the wall response approached the limits of the measuring transducers, or collapse was felt to be imminent. The transducers were then removed to prevent them being damaged. At higher levels of input acceleration the peak response was estimated from the input displacement and frequency, and the natural frequency and damping measurements, as noted earlier.

7. RESULTS

7.1 Natural Frequency and Damping

Theoretical values for the fundamental frequency of the veneer panels can be found by consideration of the stiffness of the veneer and the timber backing. Prior to cracking the stiffnesses will be additive, so the fundamental frequency of the panels will be given by

$$f_0 = \frac{1}{2\pi} \frac{n^2 \pi^2}{l^2} \sqrt{\frac{(EI)_v + (EI)_f}{m}} \quad \text{Hz} \quad (1)$$

where $(EI)_v$ = veneer stiffness

$(EI)_f$ = timber frame stiffness

m = total mass/unit height

l = vertical span = 2.44m. and $n = 1$

From measurements of wall dimensions and brick mass,

$$I_v = 1.34 \times 10^{-4} \text{m}^4, I_f = 1.09 \times 10^{-5} \text{m}^4 \text{ and } m = 304 \text{ kg/m.}$$

On the basis of earlier veneer tests by Kenna⁽⁹⁾, the veneer modulus of elasticity was taken as $E_v = 4.0 \text{ GPa}$. A value of $E_f = 8.0 \text{ GPa}$ was assumed.

$$\text{Consequently: } (EI)_v = 5.36 \times 10^5 \text{ Nm}^2$$

$$(EI)_f = 8.72 \times 10^4 \text{ Nm}^2$$

Thus in the uncracked state the veneer stiffness is 6.15 times stiffer than the timber frame, and 86% of the inertia face load will be carried by flexure of the veneer. Substituting the above values into Eqn. (1) yields

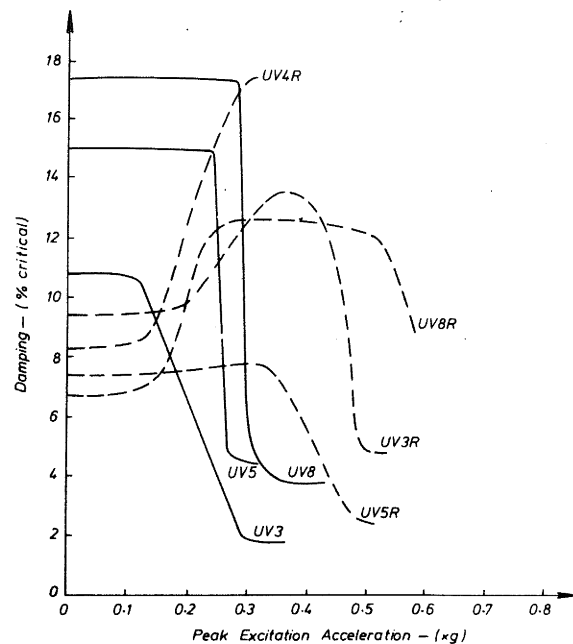


FIG. 5 DAMPING, UNREINFORCED VENEERS

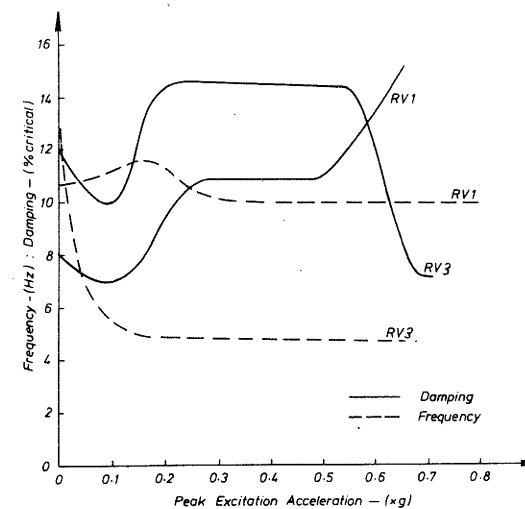


FIG. 6 FREQUENCY AND DAMPING, REINFORCED VENEERS

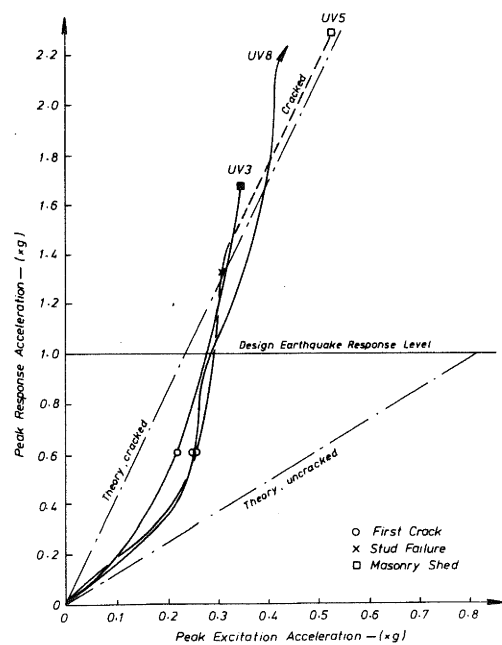


FIG. 7 ACCELERATION RESPONSE OF UV SERIES VENEERS

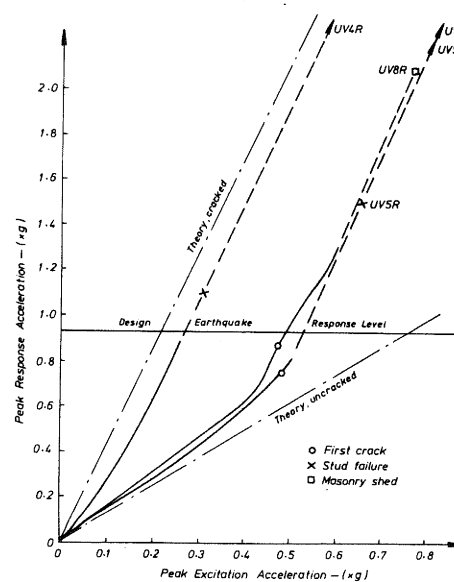


FIG. 8 ACCELERATION RESPONSE OF UVR SERIES VENEERS

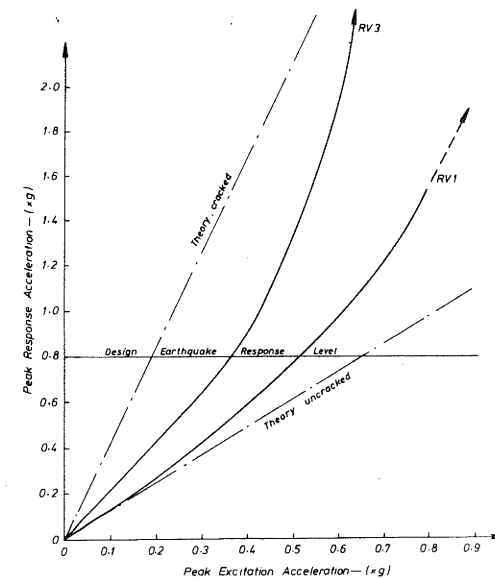


FIG. 9 ACCELERATION RESPONSE OF REINFORCED VENEERS

$$f_o = 11.5 \text{ Hz}$$

After cracking occurs, the total stiffness will reduce substantially, and in the limit, when a large number of horizontal cracks have developed, the veneer stiffness will effectively be zero. The fundamental frequency will then be governed by the timber frame stiffness. Substitution into Eqn. (1) with $(EI)_v = 0$ yields

$$f_{cr} = 4.5 \text{ Hz}.$$

For UV8 and UV8R, the reduced stud spacing has an insignificant effect on the uncracked-veneer frequency, but increases the cracked-veneer frequency to

$$f_{cr} = 5.0 \text{ Hz}.$$

Similar values are obtained for the two reinforced veneers, though the lower limit would not be expected to be obtained in practice, since the reinforcement should maintain some veneer stiffness after cracking.

Figs. 4 - 6 plot the variation of natural frequency and damping with peak excitation acceleration. For comparison, the theoretical values calculated above are included. It can be seen that the natural frequencies of the unreinforced veneer panels (fig. 4) conform well to the expected limits imposed by the uncracked and fully cracked states, with behaviour characterised by a reduction from an initial value close to the theoretical uncracked-veneer natural frequency to a value close to the theoretical cracked-veneer natural frequency. The exception to this pattern is panel UV4 R which had three horizontal zerobonds, and whose natural frequency was close to the fully cracked value for all excitation levels.

A significant difference in behaviour between the UV series and the UVR series veneers is indicated. Natural frequencies of the UV series panels gradually decreased from the uncracked to the fully-cracked value, while the UVR veneers maintained the uncracked frequency until first cracking was observed, whereupon a sudden and rapid reduction to the cracked-veneer natural frequency occurred. It is felt that this difference is a result of the different connection details used for the two series, and is particularly related to the more flexible base incorporated in the UV series panels.

It should be noted that the preformed diagonal cracking of UV5, UV5R, UV8 and UV8R did not influence the initial natural frequency, or the acceleration at which natural frequency reduction occurred.

Damping for the unreinforced veneers, shown in fig. 5, also reveals differences in behaviour between the UV and UVR series veneers. The UV panels had high damping in the uncracked state, with a rapid reduction to an average value of about 4% critical after cracking. The UVR veneers generally exhibited an increase in damping from an initial value of about 8%, with a reduction to an average value of about 5% critical after cracking. It appears that the bulk of the damping is initially provided by movement of the stud/plate joints. In the case of the UVR series veneers, additional damping is provided by movement of the veneer tie connections at higher levels of response acceleration. After cracking, response accelerations are greatly increased due to resonant

response conditions, and the joints and connections considerably loosened, particularly at the low amplitudes of the natural frequency impact tests, with a consequent reduction in damping.

Fig. 6 plots the natural frequency and damping of the reinforced veneers against peak excitation acceleration. The natural frequency of the precracked panel RV3 dropped rapidly to a value close to the theoretical cracked-veneer frequency once sufficient response acceleration was developed to crack the grout cores at the zerobond courses, but the frequency response of RV1 remained effectively constant up to excitation accelerations of about 0.8g. Cracking of this veneer was first observed at an excitation acceleration of 0.89g. Damping of the reinforced veneers remained high after cracking as a result of the reinforcement tending to hold the crack faces in contact, providing frictional energy dissipation.

7.2 Acceleration Response of Veneer Panels

The dynamic performance of the veneer panels is summarized in figs. 7-9 and Table 2. The plots of midheight response accelerations include two theoretical relationships based on the dynamic amplification factors appropriate to the ratios of the uncracked and cracked frequency respectively, to the excitation frequency of 5.0 Hz, using the curves of fig. 3a. It will be noted that as a result of the near coincidence of the excitation frequency and natural frequency in the cracked state, predicted response in this condition is high. Values for damping used in calculating the theoretical response relationships were based on conservative measured damping of 4% for the UV series, 5% for the UVR series and 8% critical for the reinforced veneer panels.

UV Series Veneers (fig. 7)

Minor tie movement was observed prior to development of flexural cracking at midheight at about 0.5 - 0.6g response. Shedding of masonry was preceded by flexural failure of one or more studs close to midheight (fig. 10), and fracture of the special veneer ties (fig 14), at response accelerations in excess of 1.7g. This corresponds to a minimum timber flexural stress at fracture of 15.3 MPa.

Examination of fig. 7 shows that initial response agreed with theoretical predictions based on the uncracked natural frequency. After cracking, response accelerations were close to, but generally exceeded those predicted on the basis of the fully-cracked stiffness. This is expected, as the panel frequency degrades to a value slightly in excess of the theoretical cracked-veneer stiffness, and thus close to the excitation frequency of 5 Hz, resulting in higher dynamic amplification.

Response after removal of transducers monitoring displacement of the veneer has been conservatively based on the theoretical cracked-stiffness response, and the measured excitation acceleration, and is shown dashed in fig. 7. The required response acceleration for satisfactory performance under the design level earthquake (from fig. 3b, using the measured 4% critical damping) is estimated at 1.0g, and is shown in fig. 7 for comparison.

UVR Series Veneers (fig. 8)

Before flexural cracking, which occurred

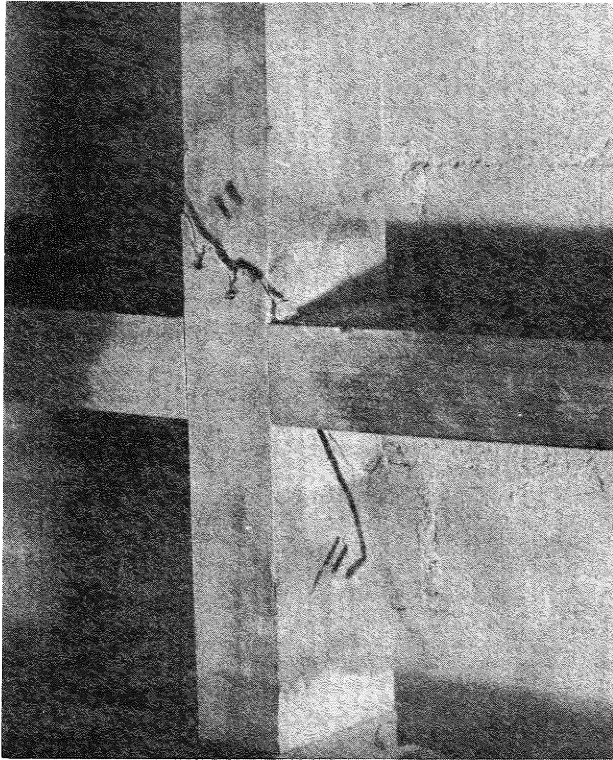


FIG. 10 FLEXURAL FAILURE OF STUD

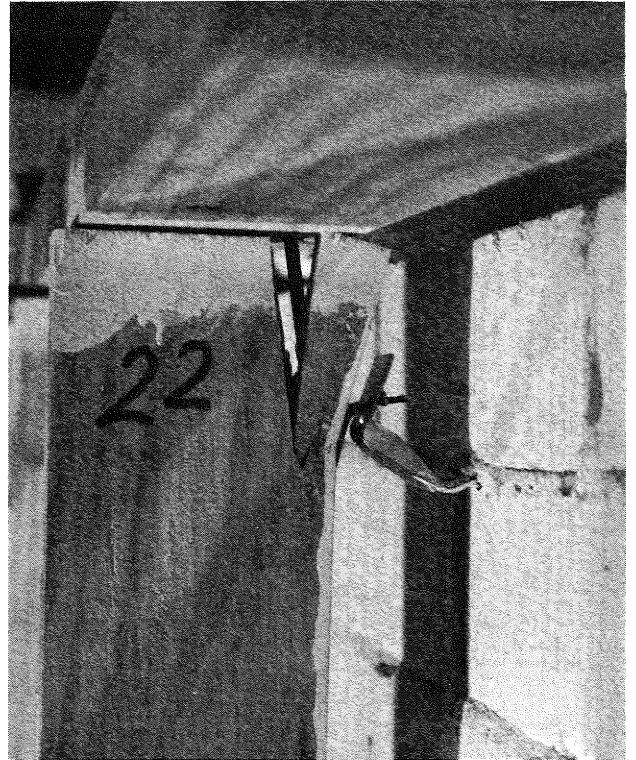


FIG. 11 STUD/PLATE FAILURE

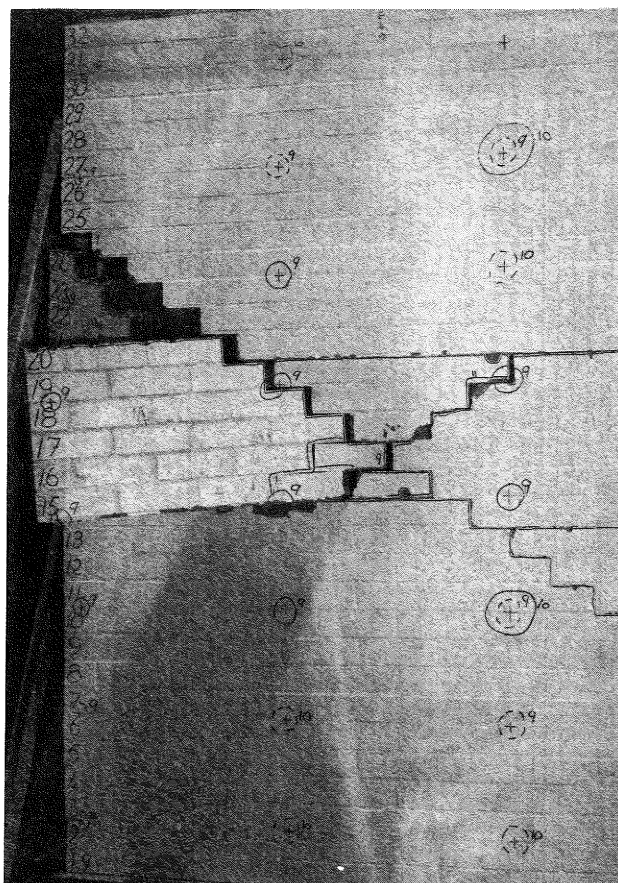


FIG. 12 MASONRY SHEDDING, VENEER UVR5 AT 2.4 g RESPONSE

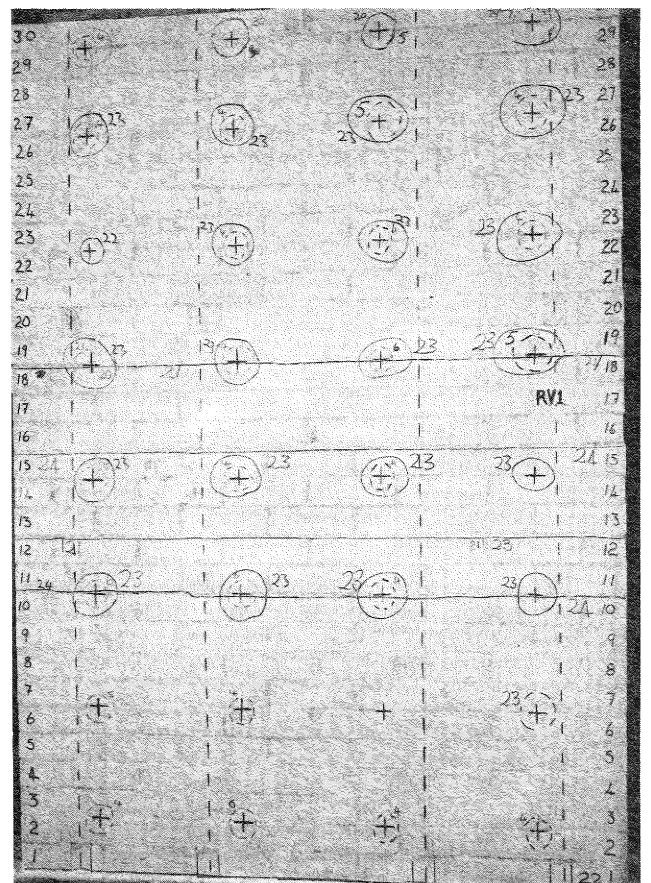


FIG. 13 REINFORCED VENEER RV1 AT END OF TEST

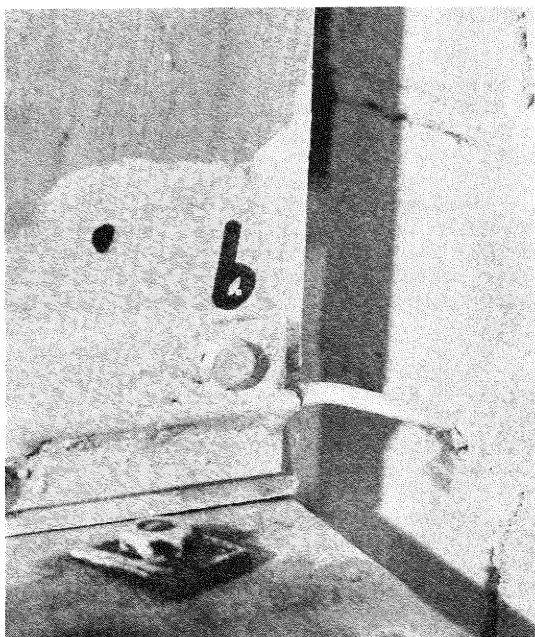


FIG. 14 FRACTURE OF UV SERIES SPECIAL TIE

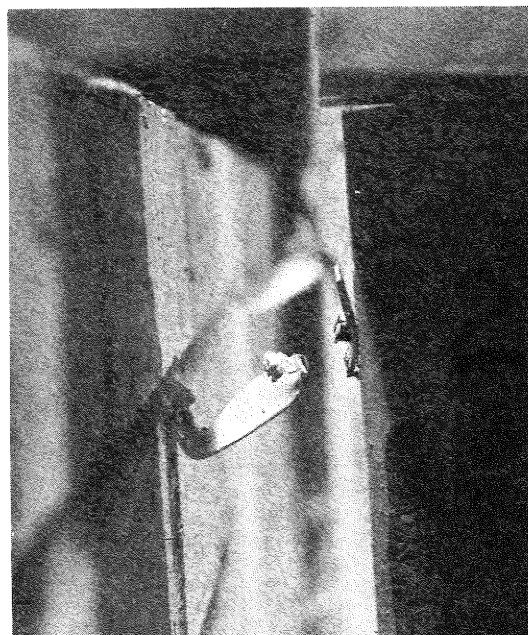


FIG.15 FRACTURE OF CONVENTIONAL FACE-NAILED TIE

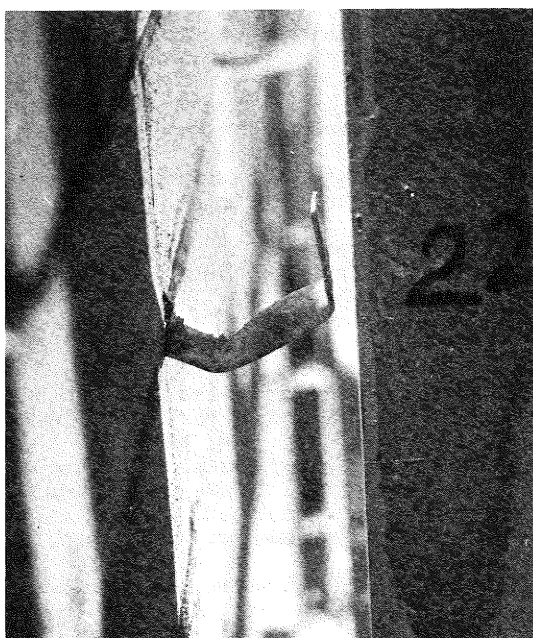


FIG. 16 NAIL WITHDRAWAL, CONVENTIONAL TIE

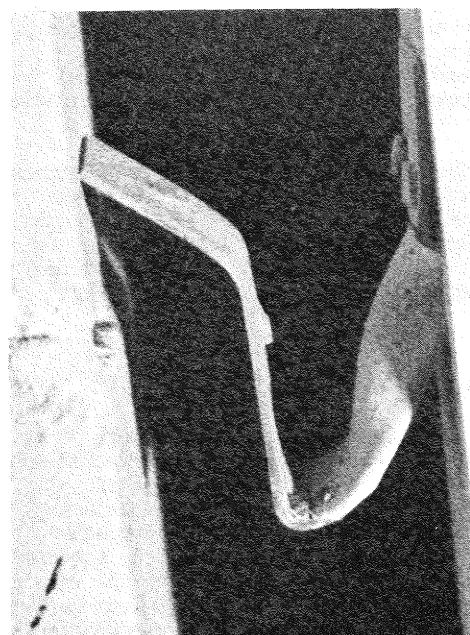


FIG. 17 TIE WITHDRAWAL, CONVENTIONAL TIE

between 0.75g and 0.93g response acceleration, damage was limited to minor movement of ties, both at the face nailing connection, and due to movement in the mortar bed.

Failure, in the form of masonry shedding, was preceded by stud failure at midheight, or at the stud/plate connection (fig. 11), and by failure of the veneer ties by fracture (fig. 15), face-nail withdrawal (fig. 16) or withdrawal of tie from mortar bed (fig. 17). Fig. 12 shows panel UV5R at the end of testing, when a portion of brickwork bounded one side by the preformed diagonal crack has been dislodged.

Initial response of walls UV3R, UV5R, and UV8R is in good agreement with the theoretical uncracked-veneer curve in fig. 8. UV4R, with three zerobond courses conforms reasonably well to the theoretical cracked-veneer curve from the start. Response of the panels has been extrapolated past the final measured accelerations, by extending the curves parallel to the theoretical cracked-veneer relationship, up to the maximum excitation acceleration. This should be very conservative, as the curves from the UV series indicate that actual response in the cracked state exceeds the theoretical response.

Reinforced Veneers (fig. 9)

Damage to the reinforced veneers was limited to veneer-tie failure, generally by face-nail withdrawal, and relative displacement of the stud/plate connection. The vertical reinforcement provided sufficient integrity to the panels to avoid masonry being shed subsequent to veneer tie failure, though RV3 eventually separated as a unit from the frame after continuous excitation at up to 1.7g peak acceleration. Fig. 13 shows RV1 at a late stage of the testing.

From Table 2 it may be seen that, taking masonry shedding as the failure criterion, all veneer panels exceeded the design level earthquake response with a substantial factor of safety. The presence and type of preformed cracking did not have any apparent effect on failure acceleration or mode.

Table 2 also compares response accelerations at first cracking with theoretical values based on the measured brick/mortar tension bond strength. Moderate agreement is obtained, though scatter is considerable, with measured cracking acceleration exceeding predicted values for all except panel UV8R.

3. CONCLUSIONS

Dynamic tests on brick masonry veneer panels indicated that unreinforced veneers constructed to accepted specifications (3,10) are capable of surviving response accelerations well in excess of those expected under design level earthquake for Zone A⁽¹⁾. This was despite the very severe testing regime which in effect subjected the veneers to a series of simulated earthquakes of ever increasing intensity until failure resulted. Performance under a single event at the design level or higher can be expected to be better than indicated by these tests.

At Zone A design level attack, damage in all veneers was limited to minor cracking of the veneer in flexure and slight movement of veneer tie

connection to stud and veneer.

Preformed horizontal or diagonal cracking had no apparent influence on the ultimate performance of the veneers. Performance was also insensitive to variations in stud and veneer.

Failure of the veneer panels was normally preceded by failure of the studs in flexure, generally initiated by a defect (such as a knot) or by a nailed or bolted connection. Flexural stress in the timber at stud failure was at least 15 MPa.

Natural frequencies and response accelerations agreed well with values predicted using normal dynamic theory. Damping of the unreinforced veneers decreased at onset of cracking and stabilized at a minimum value of 4 - 5% critical.

The reinforced veneers indicated higher damping in the cracked state than the unreinforced veneers, and as such are likely to have reduced response under seismic attack. Very high response was obtained from the reinforced veneers without causing failure. It was not possible to dislodge bricks from the reinforced veneers at the limits of excitation acceleration imposed by the experimental equipment.

ACKNOWLEDGEMENTS

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Panel No.	Symbol	Stud Spacing (mm)	Veneer Width (mm)	Type of Zero Bond	Connection Details	Brick/Mortar Bond Strength (kPa)	Grout Comp. Strength (MPa)
UV3		600	1880	None	Special	290	-
UV3R					Conventional	430	-
UV4R		600	1880	Horizontal at 9th, 16th and 23rd mortar beds	Conventional	700	-
UV5		600	1880	Diagonal	Special	290	-
UV5R					Conventional	270	-
UV8		400	1460	Diagonal	Special	290	-
UV8R					Conventional	710	-
RV1		450	1670	None	Conventional 4-D10 vertical reinforcing bars	380	15.7
RV3		600	1910	Horizontal at 12th, 15th, 18th mortar beds	Conventional 4-D10 vertical reinforcing bars	735	12.8

TABLE 1 – CONSTRUCTION DETAILS OF VENEER PANELS

Wall No.	Symbol	Design Earthquake Response a_d (xg)	Predicted cracking Acceleration a (xg)	PEAK ACCELERATION RESPONSE			$\frac{a_m}{a_d}$	$\frac{a_c}{a_p}$
				Mid-height cracking a_c (xg)	Stud Failure a_s (xg)	Masonry Shed a_m (xg)		
UV3		1.0	0.46	0.6	1.7	1.7	1.7	1.30
UV5		1.0	0.46	0.6	1.3	>2.3	2.3	1.30
UV8		1.0	0.46	0.5	>2.5	>3.3	3.3	1.09
UV3R		0.93	0.66	0.75	>2.6	>3.3	3.4	1.14
UV4R		0.93	N.A.	N.A.	>1.1	>3.2	3.4	N.A.
UV5R		0.93	0.43	0.73	>1.5	>2.4	2.6	1.70
UV8R		0.93	1.07	0.86	N.A.	>2.1	2.3	0.80
RV1		0.8	0.59	1.7	N.A.	N.A.	>2.0	2.88
RV3		0.8	N.A.	N.A.	N.A.	N.A.	>3.0	N.A.

TABLE 2 – SUMMARY OF PANEL PERFORMANCE