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Testing of cold-formed steel framed domestic structures

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ABSTRACT: In common with many regions of the world which experience low to moderate levels of seismicity, the majority of domestic houses in Australia are not designed specifically to resist earthquakes. The Australian earthquake standard, introduced in 1993, includes domestic structures which are mostly light framed construction. However, the existing knowledge on seismic vulnerability of such structures and particularly for new framing systems which involve high strength components and innovative connections is limited. The present paper addresses this urgent and important issue by giving key results from an extensive recent programme of research comprising non-destructive dynamic field tests on real typical single storey steel-framed brick veneer houses. Also presented is a review of findings from an earlier laboratory experimental study to determine the static and dynamic performance of such structures and components. The results from both phases of research indicate that cold-formed steel framed domestic structures as built in Australia perform well. The non-structural components particularly the plasterboard interior lining contribute significantly to the lateral strength and stiffness of wall frames.

1 AUSTRALIAN SEISMICITY

Although Australia lies in the middle of one of the World’s largest tectonic plates, it is not immune from earthquakes. The Newcastle earthquake (1989) with a magnitude of only 5.6 caused 13 deaths and insurance claims estimated at A$1.5 billions, making it the most expensive Australian natural disaster in the last 200 years for the insurance industry.

On average an earthquake the size of the Newcastle earthquake, or larger, occurs once per year in Australia. Such earthquakes are called “bullseye events” and seriously affect a limited area. It has been estimated that the occurrence of an earthquake of equivalent severity in Melbourne or Sydney would result in approximately 500 deaths and A$8 billion damage to domestic structures alone (Blong, 1993). Most recently, an earthquake measuring 6.3 on the Richter scale struck a remote area of the Kimberley region in Western Australia in August 1997. This earthquake released in the order of five times the energy of the 1989 Newcastle earthquake.

In 1990 it was estimated that in the previous 20 years one million houses and dwellings were destroyed by earthquakes causing 5 million people to be homeless worldwide (Walker, 1990). This illustrates the possible and evident devastation resulting from inadequate earthquake provisions in domestic structures in Australia and worldwide.

Recently, earthquake design procedures in Australia have undergone major changes with the introduction of a new Australian earthquake loading standard (Standards Association of Australia AS 1170.4, 1993) which not only introduced a new seismic design philosophy in Australia but also included domestic structures. The new earthquake code reflects the realities of the earthquake hazard in Australia and the consideration of domestic structures recognises the potential severity of earthquake damage to these structures.

2 STEEL DOMESTIC CONSTRUCTION

The majority of domestic structures in Australia are separate (detached) framed houses which consist of steel or timber frames, plasterboard interior lining, brick veneer exterior cladding and steel or tile roof cladding. Conventionally, the frames are laterally braced by diagonal strap braces. The plasterboard and brick veneer are generally considered to be non-structural components. Approximately 120,000 new houses are built each year with a total value of more than A$9 billions.

Australia is one of the pioneer countries in the use of cold-formed steel frames for domestic construction. In the past, steel frames were not competitive with traditional timber frames when contrasted on the bases of cost, versatility, manufacturing capability and trades acceptance. However, this picture is rapidly changing due to a number of factors, but mainly due to the more effective use of steel.

The increasing use of cold-formed steel for domestic structures is not only evident in Australia but worldwide. In the United States, there were 500 homes built using cold-formed steel (often referred to as light gauge steel in the USA) in 1992. This number rose to 15,000 in 1993 and 75,000 in 1994 (Peköz, 1995). This trend is continuing and it is expected that approximately 325,000 new homes will be framed using steel in the year 2000 (Tri-Steel Structures Inc, 1997, www site).

In Japan extensive research programs and marketing strategies are rapidly progressing to increase the market share of cold-formed steel in residential structures. To promote steel framed houses, major steel companies, through Japan's steel industry association (Kozai Club), are pursuing changes in the Japanese building regulations to expand the use of cold-formed steel into three-story houses, multi-unit dwellings and two-story homes in areas with heavy snowfall.

In Europe, intense research is being carried out on various aspects of cold-formed steel to expand its use in residential construction. Testing and monitoring of a light steel framed demonstration building are taking place at Oxford Brookes University (UK). The building forms a part of SCI's (Steel Construction Institute) and British Steel's involvement in the European MeagProject 5, a large multi-partner project that has addressed the use of steel in housing, refurbishment and temporary structures. Further, innovative systems and fabrication and construction techniques are continually being developed in different parts of Europe including Denmark, Finland and Sweden (Anderson and Hamrebjörk, 1994).

3 EARTHQUAKE RESEARCH ON STEEL DOMESTIC STRUCTURES IN AUSTRALIA

The constant development of steel frames and the inclusion of domestic structures in the Australian earthquake code (Standards Association of Australia AS 1170.4, 1993) necessitated research into the performance of cold-formed domestic steel frames when subjected to earthquakes. A research team involving The University of Melbourne, BHP, CSIRO (Commonwealth Scientific and Industrial Research Organisation) and the Australian Research Council (ARC) was formed in 1993 in Australia to investigate the performance of steel framed domestic structures when subjected to earthquake loading. The research project has considered the influence of both the structural and non-structural components present in domestic structures. The research has involved laboratory based experiments (on two- and three-dimensional specimens) and tests on field full sized houses. Further, it has developed predictive analytical models on which sensitivity studies have been conducted. In this paper attention is given to the findings from the experimental program, particularly the tests on the full sized houses. Some of the findings from the analytical modelling have been presented in Gad et al. (1997a,b). The research to-date has

- established the safety of the framing system when subjected to earthquake loads;
- examined the performance of the framing systems, including their ductility and energy absorption capability;
- assessed the sensitivity of the framing behaviour to differing geometrical configurations and connection types; and

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- optimised the structural form and considered the effects of the interior lining and exterior cladding.

The following sections in this paper briefly outline some of the specific findings from the various phases of the research project.

4 LABORATORY BASED TESTS

4.1 *Tests on two-dimensional isolated wall frames*

Cyclic racking, swept sine wave, pluck and earthquake tests were conducted on isolated unlined wall frames with simulated roof mass (equivalent to steel clad roof) as shown in Figure 1. Frames with tab-in-slot (essentially pinned) and welded connections were investigated to examine the influence of such connections on their dynamic performance. Both types of frames survived simulated 100% and 200% El-Centro earthquake records (the 200% record has twice the acceleration intensity of the original record). It was found that the type of framing connection does not seem to have a significant influence on the dynamic response of such frames. The governing component in unlined frames is the strap bracing system which includes the strap, end connections and tensioner unit (Barton et al., 1994). The initial tension in the strap bracing system affects directly the initial stiffness and alters the shape of the load-deflection curve.

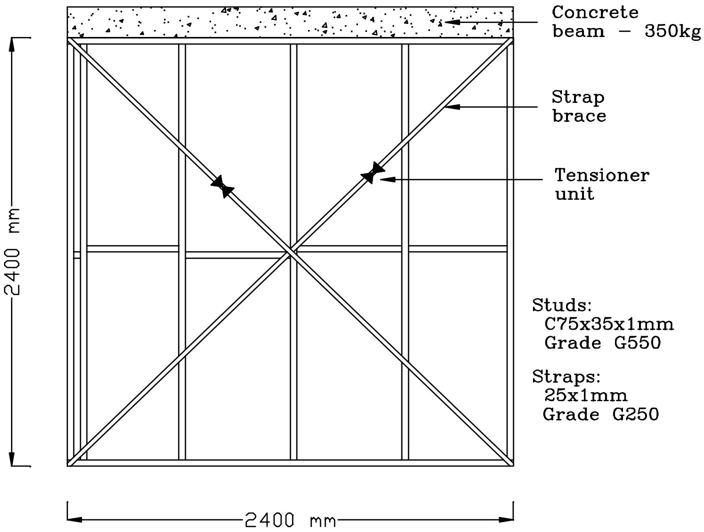

Figure 1. Typical tested isolated unlined wall frame with simulated roof mass.

4.2 *Tests on three-dimensional structure*

In order to study the interaction between the structural and non-structural components and correctly incorporate the relevant boundary conditions, a one-room "test house" was adopted. The test house measured 2.3m x 2.4m x 2.4m high and was made out of full scale components as shown in Figure 2. The test house simulates a section of rectangular house with plasterboard interior lining and brick veneer exterior cladding. A concrete slab weighing approximately 2400kg was placed on the top of the house to simulate a tiled roof mass. The properties of the framing members were identical to those of the isolated frames with tab-in-slot connections. The strap braces were conventionally fixed at each plate-stud junction by a single self-tapping screw and at each intermediate stud to prevent rattling of the strap under service conditions. The tensioner units were fitted to the strap braces as per common practice to square and plum the walls during the construction phase. However, all the strap braces in all construction stages were tensioned to the same load to enable comparisons of behaviour. The plasterboard was fixed using one of the industry recommended methods where the plasterboard is screwed at the vertical edges at 200mm centres, the top and bottom horizontal edges screwed at 600mm centres, and along the intermediate studs screwed at 400mm centres. Detailed description of the test house is presented in Gad et al. (1995).

The house was tested at five stages of construction, as shown in Table 1, to investigate the behaviour and interaction of the various components of the structure. Swept sine wave, double amplitude cyclic racking and simulated earthquake tests were conducted on the house to reveal as many characteristics as possible. From these tests the natural periods of vibration, mode shapes, damping ratios, load-deflection curves, ultimate deflection and load capacities, load sharing characteristics, ductility and energy absorption capacity were established.

Some of the findings from the experimental program on the test house are listed below, further findings and conclusions have been presented in Barton (1997) and Gad et al. (1998a).

- For unclad frames the connections between perpendicular wall frames resist very little racking load, however they do contribute significantly to damping the framing.
- The strap bracing system governs the lateral behaviour and performance of unclad frames.

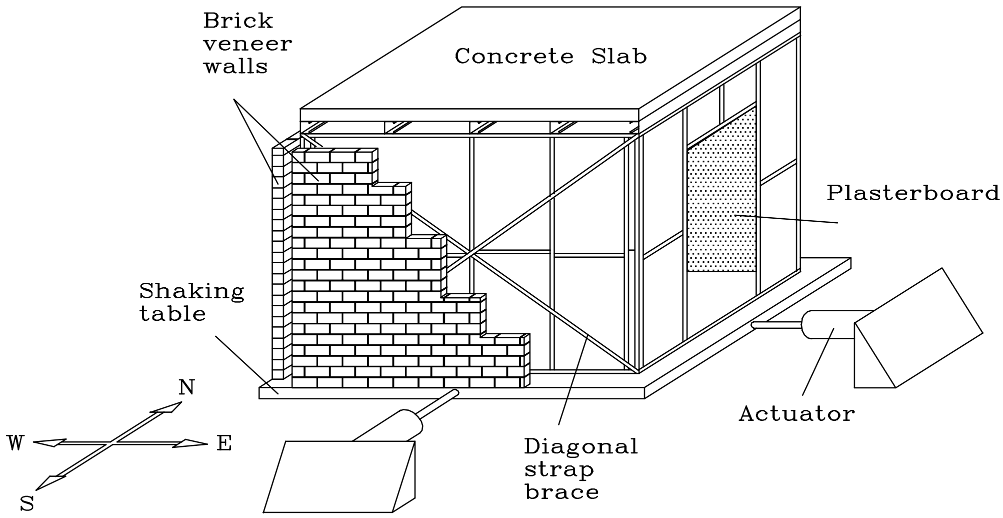

Figure 2. A schematic diagram of the test house and the shaking table.

Table 1. Summary of the construction stages of laboratory test house.

Stage of Construction	Description
0	Unclad wall frames with no strap bracing.
1	Unclad wall frames with strap bracing on all four walls.
2	Plasterboard-clad wall frames and ceiling with skirting-boards and ceiling cornices, without strap bracing.
3	Plasterboard-clad wall frames and ceiling with skirting-boards and ceiling cornices, with strap bracing on all four walls.
4	Plasterboard-clad wall frames and ceiling with skirting-boards and ceiling cornices, with strap bracing on the East-West walls only. Brick veneer external walls on all four walls but not connected at the corners.

- The connection between the end of the strap and the corner of the wall frame was found to be the first point of failure of the wall frame system. Close to the failure load of the end connections, the straps also show strong signs of failure at the tensioner units as the straps have a reduced cross sectional area due to the presence of a bolt which is used to tighten the brace.
- The initial tension in the strap bracing has a significant effect on the racking behaviour of the unclad wall frames, in particular the ultimate deformation capacity and consequently the frame ductility.
- Plasterboard lining with return corners, cornices and skirting-board and fixed with a nominal number of screws was proven to be a very effective bracing medium. It provides higher strength and stiffness than conventional strap braces.
- The plasterboard lining exhibits a high level of damping and energy absorption capacity.
- The strength and stiffness degradation of plasterboard under four repeated cycles with the same amplitude was found to be acceptable (less than 20%).
- The failure mode of the plasterboard lining with end return walls is not just tearing of the plasterboard at the screws connecting it to the frame but also crushing of its vertical edges through bearing action on the perpendicular walls.
- The contributions of the plasterboard and the strap braces to the strength and stiffness are additive. In the test house, the plasterboard resisted 60% to 70% of the lateral loads compared to 30 to 40% resisted by the strap braces.
- For wall frames with strap braces and plasterboard, the deformation capacity of the strap brace is the governing variable as the braces fail at a lower displacement than the plasterboard. To fully utilise the capacity of plasterboard, strap braces should be improved (at the end connections and tensioner unit) so that their failure displacement is compatible with that of the plasterboard.
- The test house survived the simulated 100% and 200% El-Centro earthquake records. It failed during the 300% El-Centro record. The strap braces failed first which was followed by failure of the plasterboard lining which led to large drifts and subsequently the out-of-plane brick veneer walls broke along their mid-height with the top halves collapsing. In addition, the in-plane veneer walls slid along the damp proof course with many brick ties disengaging from the frame.
- The failure mechanisms of the plasterboard and strap braces in the simulated earthquake tests were identical to those observed in the racking tests. In other words, the load carrying mechanisms were the same for both loading types.

5 FIELD TESTS ON FULL SIZED HOUSES

A number of real full size houses have been tested to provide a calibration between the laboratory based work and typical full sized houses. This has been established through testing of a damaged house that was subjected to severe racking loads at the Cyclone Testing Station in Townsville (Queensland, Australia) and a number of new houses at various stages of construction in Sydney.

These full sized houses were subjected to modal testing through the use of an impact hammer and a portable data acquisition system. Modal testing has been utilised in many civil applications particularly in dynamic assessment of bridges (Haritos, 1994). Modal testing provides accurate estimates of natural frequencies, damping ratios and mode shapes. For the damaged Nu-Steel house, these dynamic parameters have been used to estimate the possible decay in stiffness and the implications on the earthquake design criteria. For the new houses in Sydney these parameters have been used to assess the contributions from the various components. This paper presents the results from the new houses tested in Sydney.

5.1 *Experimental procedure*

The impact hammer used to excite the full sized houses has a load cell to measure the input pulse force and a soft tip which lengthens the duration of the pulse. The pulses induced on the houses

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were in the range between 1 kN and 4 kN. The hammer was able to produce an almost constant energy in the frequency range between 0 Hz and 50 Hz.

To accurately assess the dynamic behaviour of each house, the response to the excitation was measured using accelerometers at approximately 40 different locations along the house. Since the data acquisition system used can accommodate 16 channels only, the accelerometers were moved systematically within the house to measure the response at various locations. Typically, 15 accelerometers had to be moved three times to cover all the required measuring points as depicted in Figure 3. For each position of accelerometers the house was excited by one hammer blow (i.e. one pulse) with the pulse force and the response at the accelerometers recorded to one data file. This process was repeated approximately 20 times. Hence for each position of accelerometers the house was excited approximately 20 times. In the analysis of the data, the 20 records (from the 20 excitations) were compared to ensure that the results are consistent and then averaged to eliminate the background noise from the signals.

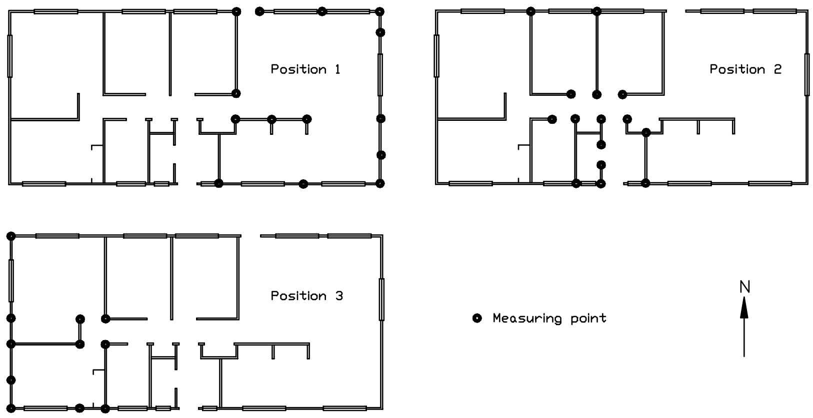

Figure 3. A typical house floor plan showing how the accelerometers are moved into three positions to cover different measuring points for excitation in the North-South direction.

For each house more than one excitation point (where the impact is applied) was utilised. To ensure that all modes of vibrations were revealed one excitation point was selected close to the middle of the house to show the racking (sway) mode and another point close to one of the corners of the house to reveal torsional modes (if any). When sufficient wind was blowing the vibrations of the houses due to wind was measured (ambient vibration). The ambient vibration was compared with that caused by the impact hammer and in all cases the response of the houses due these two sources of excitation was identical. This comparison ensured that the impact hammer was able to excite all the relevant modes of vibration.

5.2 *Analytical procedure*

The signals from the accelerometers and load cell were gathered using a 16 channel data acquisition system which incorporates a spectral analysis software TSPECTRA (Chalko, 1991). Using TSPECTRA the response at every location was divided by the input excitation (pulse force) to obtain a transfer function (TRF) for every point of measurement. Each function was constructed from a series of digital points. Having obtained all the TRFs the results from the repeats (approximately 20 repeats for each measuring position) were averaged using TSPECTRA. The averaging is valuable in reducing the impact of noise on the quality of the records.

A modal analysis software package DSMA (Direct Simultaneous Modal Approximation, Chalko, 1996) was then used to perform a modal analysis on the structure using these TRFs obtained from the TSPECTRA module. DSMA attempts to fit the best linear dynamic system to the data. The TRFs relate the input force (from the impact hammer) to the acceleration measured at the various locations on the structure. DSMA is not just a curve-fitting software, it aims to approximate simultaneously the entire set of TRFs with suitable functions to obtain the optimal values for common modal parameters (eigenvalues and eigenvectors) of the structure. An interactive optimisation of the eigenvalues and eigenvectors is essential to obtain the best possible modal fit in the selected frequency band.

The resulting experimental model from the modal analysis is represented as a set of natural frequencies (eigenvalues), modal damping ratios and corresponding modal shapes (complex eigenvectors). DSMA provides scaled animation of the calculated mode shapes in two and three dimensions. The whole modal testing procedure is graphically summarised in Figure 4.

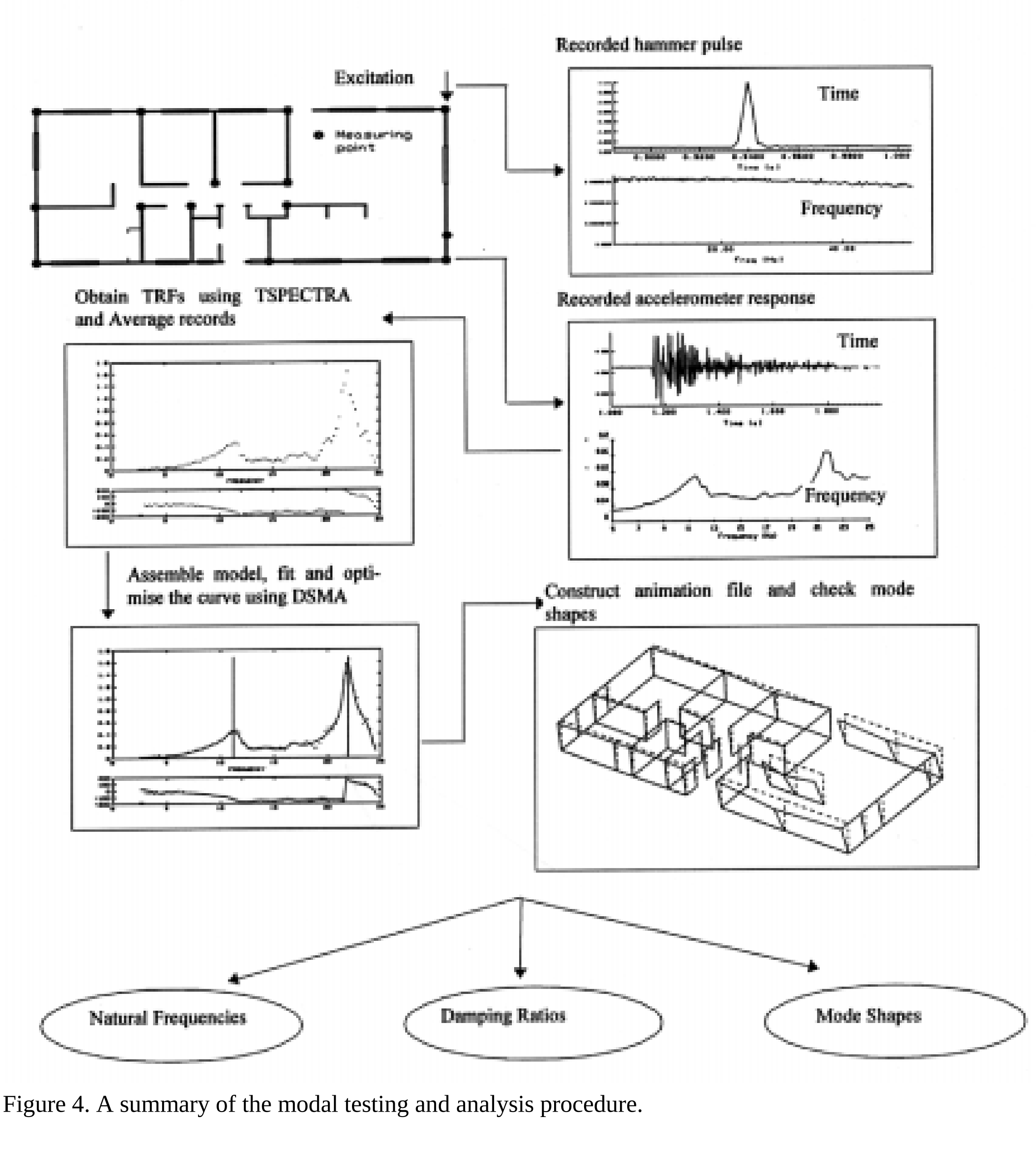

Figure 4. A summary of the modal testing and analysis procedure.

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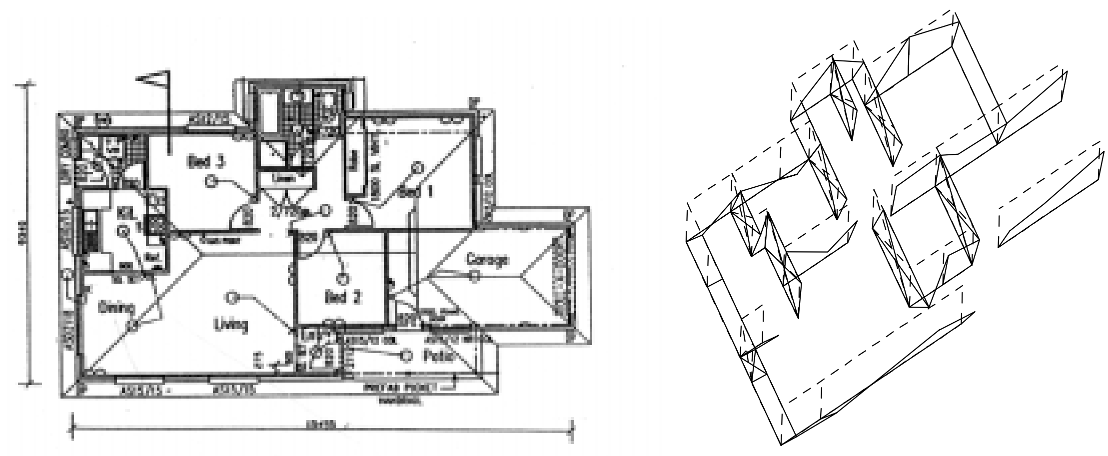
5.3 *Testing stages and results*

All houses tested were single-story brick-veneer-style with tiled roofs. These houses were part of a new development in Sydney. The construction details in the full sized houses were very similar to those used in the laboratory. In this paper the results of two representative houses are presented. The first house was a three-bedroom house “Long Homes Jabaru floor plan” and was tested at five stages of construction as shown in Table 2. Figure 5 shows the Jabaru floor plan and a snap shot of the animated sway mode. The second house “Long Homes Currawong floor plan” was only tested at Stages 2 and 5 as described in Table 3. Figure 6 depicts the Currawong floor plan and a snap shot of the animated sway mode shape. Stage 2 represents the house with all of its structural elements and its dead loads (essentially the roof tiles). Stage 5 represents the complete structure with all the so-called non-structural components (plasterboard lining, ceiling cornices and brick veneer cladding). It should be noted that Stage 2 represents the “as designed” structure (i.e. the house includes only the elements that are considered in the design phase), whereas Stage 5 represents the “real” structure.

Table 2. Stages of construction at which the Jabaru house was tested and natural periods of vibration corresponding to the sway mode.

Construction Stage	Description	Natural Period (sec)
1	Frame with strap braces	0.17
2	Frame with strap braces + roof tiles	0.33
3	Frame with strap braces + roof tiles + brick veneer	0.21
4	Frame with strap braces + roof tiles + brick veneer + plasterboard lining	0.13
5	Frame with strap braces + roof tiles + brick veneer + plasterboard lining + ceiling cornices	0.08

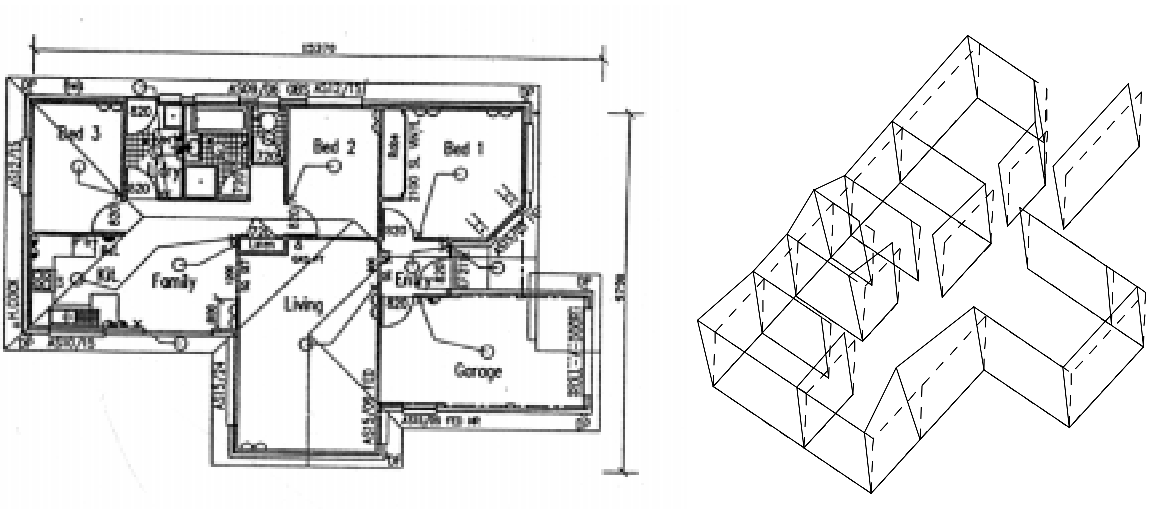


(a) (b)

Figure 5. The Jabaru floor plan and a snap shot of the animated sway mode shape

Table 3. Stages of construction at which the Currawong house was tested and natural periods of vibration corresponding to the sway mode.

Construction Stage	Description	Natural Period (sec)
2	Frame with strap braces + roof tiles	0.37
5	Frame with strap braces + roof tiles + brick veneer + plasterboard lining + ceiling cornices	0.11



(a) (b)

Figure 6. The Currawong floor plan and snap shot of the animated sway mode shape

It should be noted that the sway mode of vibration was the dominant mode for both the Jabaru and Currawong houses. It clear from Tables 2 & 3 that the addition of the non-structural components has a dramatic effect on the natural period of vibration. The addition of the brick veneer walls affects both the mass and stiffness characteristics (compare Stages 2 and 3). The addition of the plasterboard lining increases the stiffness dramatically as the wall frames act as shear diaphragms. The plasterboard lining does not change the mass of the house as the weight of the plasterboard is negligible when compared to that of the roof tiles. The addition of the ceiling cornices increases the stiffness even further as they connect the internal walls to the ceiling diaphragm and hence they become active in resisting the lateral loads. By comparing Stages 2 and 5 for both the Jabaru and Currawong houses, the inclusion of the non-structural components decreased the natural period of vibration by 70 to 75%. This decrease in natural period is attributed to a large increase in the lateral stiffness. The results obtained from these vibration tests confirm earlier findings obtained from static tests on a full sized steel framed house (Reardon, 1990).

From the results obtained from the Jabaru and Currawong houses, the natural period of vibration of such complete structures would be very low and would fall on the plateau of the Australian design earthquake response spectrum (Standards Association of Australia: AS 1170.4, 1993) as shown in Figure 7. This is also applicable to other earthquake codes such as Eurocode 8 (European Standard: EC8, 1998) and American Uniform Building Code (International Conference of Building Officials, 1994). From the tests on the laboratory test house and vibration tests on a damaged full sized house, it was found that the residual stiffness close to failure is about 10 to 20% of the initial stiffness (Gad et al., 1998b). Considering an approximate initial period of vibration to be 0.1 sec for a typical single story brick veneer house with tiled roof, the natural period of a corresponding dam-

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aged house would be between 0.2 sec and 0.3 sec. This implies that the structure would remain on the plateau of the design response spectrum for even after loss of some stiffness due to damage.

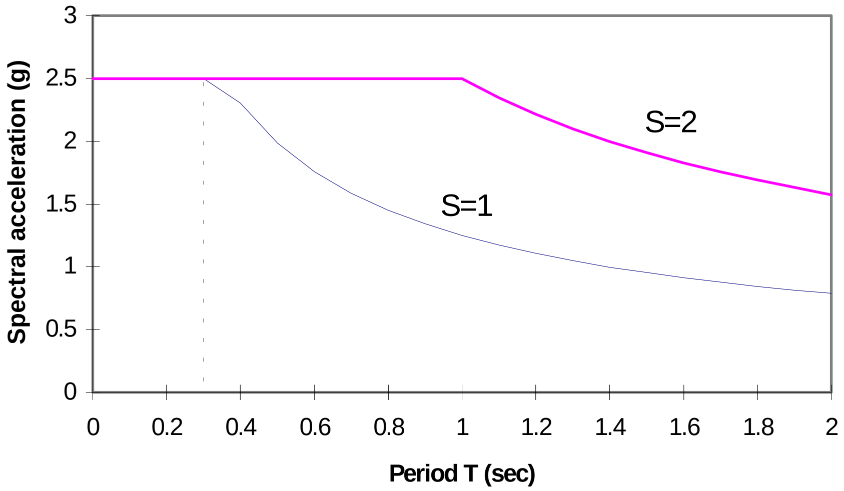

Figure 7. Normalised response spectra according to AS1170.4 showing a maximum plateau to periods up to 0.3 sec. (S=1 refers to a normal soil and S=2 refers to a soft soil).

6 CONCLUSIONS

This paper has introduced the need for rigorous investigation of domestic structures when subjected to earthquake loads in Australia and has highlighted the international interest in the use of steel framed structures. The research presented has reported the outcomes from laboratory based and field testing of steel framed structures in Australia. Complimentary modelling work has been undertaken and reported separately. Significant conclusions that can be drawn from this experimental programme include:

- the overall lateral performance of steel framed domestic structures is governed by the cumulative contribution of steel strap bracing, internal cladding (plasterboard) and boundary conditions such as corner returns, ceiling diaphragm action, cornices and skirting boards;
- the structural bracing system limits the ultimate deformation capacity of the structure;
- the so-called non-structural components such as plasterboard significantly increase the level of damping and energy absorption capacity the structural system;
- the observed fundamental mode shape of vibration of full sized houses was consistent with those observed from the laboratory tests for both new and damaged structures;
- the natural period of vibration for complete structures ranged from approximately 0.1 sec for new structures to 0.3 sec for damaged structures. Hence, using the normalised response spectra provided in AS 1170.4 steel framed domestic structures can be designed for the maximum spectral acceleration without the need for adjustment due to the increase in the natural period of vibration due to earthquake induced damage.

In conclusion, for accurate seismic evaluation and design of domestic steel framed structures, it is essential to incorporate appropriate allowances for the effects of non-structural components on the overall system stiffness, strength, ductility and damping.

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