Verification of Raker Shores using New Zealand Timber

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2006 NZSEE Conference

ABSTRACT: This paper investigates the capacity of full triangle (fixed) raker shores using New Zealand Timber, through analytical analyses and experimental tests. Full triangle (fixed) raker shores, or simply raker shores, are a temporary structure used to support collapsed or damaged buildings. They are used extensively by Urban Search and Rescue (USAR) teams around the world to allow the safe location and rescue of victims of collapsed or damaged buildings following an earthquake event. The shores used by New Zealand USAR are similar to those used in the United States; however they are made from Radiata Pine which has different mechanical properties than the timber used in the U.S. Hence, the need to verify the shores constructed in New Zealand still provide the required design strength. The analytical verification suggests that raker shores are unsafe, according to the New Zealand Timber Structures Standard NZS 3603:1993. However, the experimental results on full-scale specimens indicate that the performance of the raker shores satisfies the required demands, with a safety factor of 2 with regard to the design load, and a ductile type of failure. Suggestions to increase the capacity of the shores are also included.

1 INTRODUCTION

The full triangle (fixed) raker shores (herein described simply as raker shores) are used by New Zealand Urban Search and Rescue (NZ USAR) to stabilise collapsing walls. NZ USAR is involved in the locating and rescuing of people trapped by structural collapse (Welcome to USAR, undated). It is essential that the damaged building be prevented from further collapse to ensure the safety of the rescue operation. There are many different types of shores that are used to stabilise the different parts of the damaged buildings, but for the purposes of this study the sole focus is on the raker shore.

The development of NZ USAR was initiated after a team from the New Zealand Society for Earthquake Engineering (NZSEE) visited Los Angeles after the Northridge Earthquake in 1994, where they observed the United States USAR teams at work (USAR History, undated). As a result the practise of the NZ USAR is based on what has been developed by the Federal Emergency Management Agency (FEMA) which runs the United States USAR. This includes the shore construction modules which are produced by FEMA. Many countries including New Zealand, Australia, and Singapore are using these modules in their own USAR teams (Professor D. Bull, pers. comm.). The significance of this is that the shores specified in these modules were designed using American timber properties which are quite different from New Zealand timber properties. It is important to determine how the variations in timber properties will affect the performance of the raker shores.

The performance of the raker shores are analysed in two different ways; firstly they were checked against the New Zealand Timber Structures Standard NZS 3603:1993, and secondly a number of full scale tests were performed. The raker shores where built by the NZ USAR Task Force 2 to ensure the authenticity of construction. The results of these analyses will provide useful information to the NZ USAR about the performance of the raker shores and the necessary amendments that may need to be made.

2 **RESEARCH**

2.1 *FEMA Modules*

The raker shore is a simple timber truss system which transfers loads from the damaged building to the ground in order to stabilise it. As illustrated in Figure 1 the raker shore comprises three key components: the wall plate, sole plate and raker. Each individual shore also has a mid-point brace, cleats, gussets, spikes, wedges and an anchor plate. These individual shores are usually erected in pairs and joined by lateral braces (Fig. 2). The shores can be braced in a variety of ways, provided they resist lateral flexural buckling at the midpoint of the raker and any lateral forces. The capacity of an individual raker shore is in the range of 2400 lbs; or 10.7kN (Module 2a, undated).

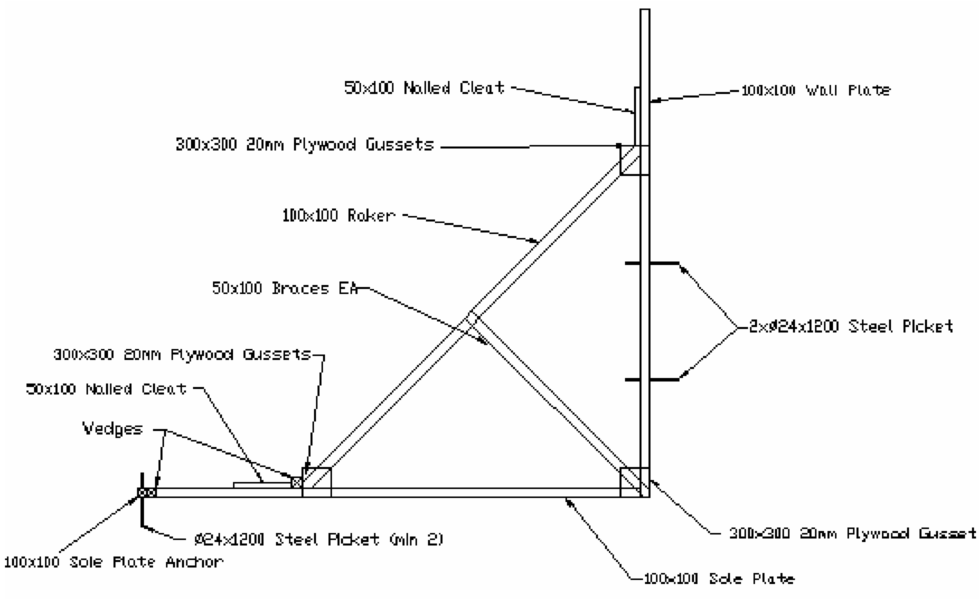

Figure 1 Elevation of raker shore verified

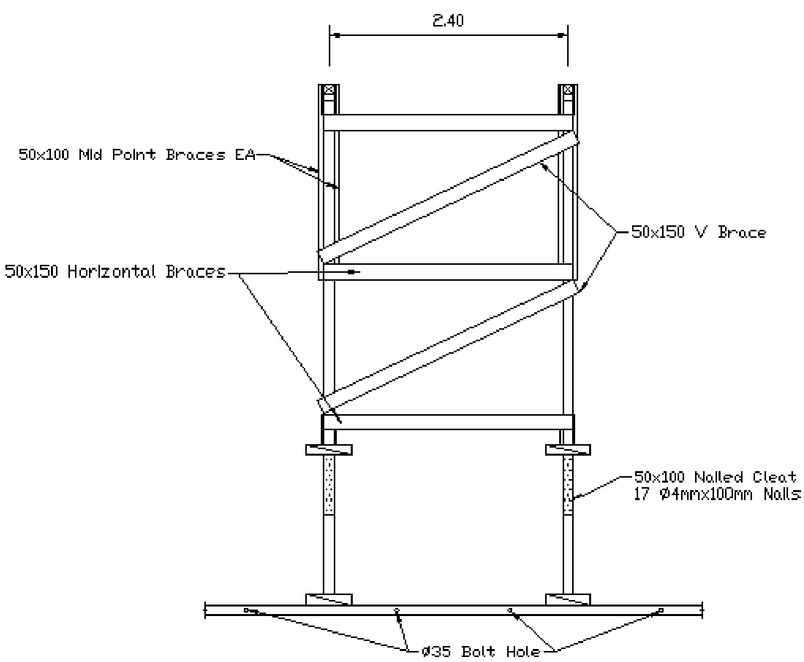

Figure 2 Plan of raker shore verified

2.2 *Comparison of Mechanical Timber Properties*

The underlying reason for this project is the variation in timber properties between the American and New Zealand timbers. The most common wood used in the U.S. is Southern Pine and Douglas Fir. It is the average properties of these species that FEMA used for the design of the raker shores (Module 2a, undated). However, the most common timber species used in New Zealand is Radiata Pine, which has different properties (Table 1). The characteristic values for Radiata Pine come from NZS 3603, and the averaged characteristic values for Southern Pine and Douglas Fir are specified by FEMA (Module 2a).

Table 1 Comparison of Timber Properties

Characteristic Strengths	Bending (MPa)	Comp. (Par.) (MPa)	Comp. (Perp.) (MPa)	Tension (MPa)	Shear (MPa)	M.O.E. (MPa)
NZ - Radiata Pine*	7.5	11.0	5.3	3.0	2.4	4800
US – South.Pine and Douglas F.	10.3	7.6	4.1	5.0**	0.6	11030

*Radiata Pine is green (m/c = 25%) and No. 1 Framing

**Tension Strength of US timber from AITC (1985)

Table 1 indicates that one is not a distinctly stronger species than the other. From this information alone it is difficult to determine whether the shores constructed out of New Zealand timber will be weaker or stronger than the U.S. counterpart. The density of the timber is another important property which determines the strength of the fastener connections. Data provided by Douglas Gaunt at Ensis, gave a characteristic dry density ($\rho_{0,k}$) of Radiata Pine, No. 1 Framing of 380kg/m^3, whereas the characteristic dry densities of Southern Pine and Douglas Fir are 550kg/m^3 and 510kg/m^3 respectively (AITC 1985). As a result, connections in New Zealand timber are expected to be weaker than in the U.S. timbers. Clearly, shores constructed from New Zealand timber will behave differently than those constructed using U.S. timber. For this reason it is important to verify that the behaviour of the New Zealand shores is adequate.

2.3 *NZ USAR Modifications*

There are a few practices of NZ USAR that differ from FEMA recommendations. Where there are differences, the current practices of NZ USAR were used in this research to ensure realistic and applicable results. There are three variations of the NZ USAR practices from the FEMA modules: the use of a 50mm gap between the wall and wall plate, 3.15mm diameter nails instead of 4.1mm, and 16mm reinforcing bars (rebar's) as spikes instead of ½ inch drill-in anchors, lag screws or rebar's.

3 RAKER SHORE VERIFIED

There are a variety of sizes and styles of the raker shore. For the results to be as practical as possible, it was important to choose the setup and materials most likely to be used in an emergency event. Where this was not so clear, the worst case scenario was used. The most important parameters of the shores to be tested are listed below:

- Load application height of 3.6m; a common inter-floor height in New Zealand
- Raker angle of 45 degrees; best combination of efficiency and practicality
- No spikes through the sole plate into the ground i.e. all load through the anchor plate
- V bracing used between the individual shores
- No. 1 Framing timber used; H4 treated, rough sawn for 100x100 members, and untreated, planer gauged for other members

Figures 1 and 2 show the shore used in analysis and testing.

4 ANALYTICAL VERIFICATION

The raker shores described above were first verified against the New Zealand Timber Structures Standard NZS 3603:1993. Some aspects of the shore were not covered clearly by NZ 3603, in which case the Eurocode 5 (CEN 2003) was used. Firstly, the shore was analysed to find the load paths through the shore and the actions in the various elements (Fig. 3).

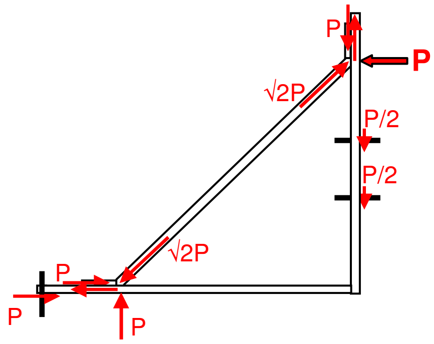

Figure 3 Load paths through the shore

Two key assumptions were made when analysing the loads through the shore. Firstly, that the entire load in the raker parallel to the wall plate or sole plate was transferred to the respective members through the nailed cleat. Friction was ignored as it was deemed insignificant and is unreliable in terms of its effects. The second assumption is that the spikes in the sole plate will not carry load. Although not shown in the following figures, spikes are often put into the ground through the sole plate as well as the anchor plate. It was assumed that the horizontal load applied from the cleats would be transferred to the anchor plate, and not to any spikes put in the sole plate. Each individual shore is designed to resist approximately 10kN; this is the horizontal load, P. This corresponds to a design load of 14.14kN in the raker, 10kN in the wall plate, sole plate, and cleats, and 5kN in the wall spikes.

With these design loads, the three elements of the shore capacity were then verified: the member capacities, bearing capacities and joint capacities. Since the timber that is used by NZ USAR during an emergency event may vary significantly in moisture content, the verification of the shores was done assuming green timber as this is the worst case. The capacities of the three key members (raker shores, sole plate, and wall plate) were verified using NZS 3603 and found to have sufficient strength (Pettigrew 2005). The bearing capacities of one timber member to another were also verified and all found to be sufficient.

The critical elements of the shore are the capacity of the two joints: nailed cleats and spikes. There are two cleats on each individual shore but as the raker is at 45 degrees, both will carry the same load. The nailed cleats are 600mm long with 17 nails evenly spaced in the 5-star pattern. The nails that the NZ USAR uses are 89mm long with a diameter of 3.15mm, different from that specified by FEMA. The strength capacity of nailed connections was found to be 6.43kN, significantly less than the demand of 10kN. This result is important as it indicates the current nailed cleat joint that NZ USAR uses is not sufficient according to NZS 3603 and experimental tests need to be done to further analyse the shore.

The spikes put through the wall plate into the collapsing wall, are often a rebar with no nut screwed on the end; as such, they need to be designed as a dowelled connection. In the spike connection used by NZ USAR, there is a 50mm gap between the wall plate and collapsing wall, but the dowel connections in Eurocode 5 (and bolted connections in NZS 3603) do not allow for such a gap. Verification of the spike connection was done using first principles; this was found to be the most accurate solution. The capacity of the spike connection (2 spikes) was found to be 8.69kN and, again, this is not an adequate strength. This gives further reason to perform a number of experimental tests on the shores.

Expected failure loads for the different components can then be calculated from the capacities found in the analytical verification. The design values calculated using NZS 3603 are characteristic strengths with reduction factors and require modification to find the expected (mean) failure load (R_e). There are also other modification factors that may need to be adjusted to account for the laboratory testing conditions i.e. duration factor. In order to find the expected failure loads, the design strengths (ΦR_n) need adjusting for the strength reduction factor Φ, the variation factor k, which accounts for the natural variation in material properties, and the duration factor. The expected (mean) failure loads and factors of safety (given by the ratio between expected strength and the design action) for the first six failure mechanisms in the hierarchy are shown in Table 2.

Table 2 Predicted Analytical Results

Rank of Failure	Element	Design Strength ΦRn (kN)	Reduction Factor	Variation Factor	Duration Factor	Expected Strength Re (kN)	Factor of Safety
1	Spikes (1st Principles)	8.69	0.7	0.918	1.0	13.52	1.35
2	Nailed Cleat	6.43	0.8	0.685	0.8	14.67	1.47
3	Wall Plate (Tension)	15.30	0.8	0.685	0.8	34.90	3.49
4	Wedge (from Raker Shore)	15.65	0.8	0.685	0.8	35.70	3.57
4	Wedge (from Cleat)	15.65	0.8	0.685	0.8	35.70	3.57
6	Raker Shore (Compression)	24.43	0.8	0.685	0.8	55.73	3.94

The predicted results suggest the shores will fail in the spikes just before failure of the nailed cleats. The factor of safety for each of these failure loads is less than the desired value of 2. This reiterates the expected weakness of the connection and the need for experimental tests to be performed.

5 TEST SETUP

Figures 4 and 5 demonstrate the test setup. A rigid test frame was designed to apply two point loads to the individual shores, and a test wall to represent the overturning wall by preventing uplift of the shores. Load cells were placed between the wall plate and test frame to measure the load in the shores, strain gauges were placed on the rakers midway between the brace and the wall plate, and displacement potentiometers were placed at the top of the wall plates, at each cleat, and at the base of the wall plate.

The analytical verification showed that the failure of the shore would be in the joints, which are easily modified. Once a joint had failed it would be replaced with a stronger one so that under retesting, failure would no longer occur at that joint, but at another place in the structure. This would establish a hierarchy of failure in the shores. This also allowed multiple tests to be performed on each set of shores until permanent damage occurred to the main structure.

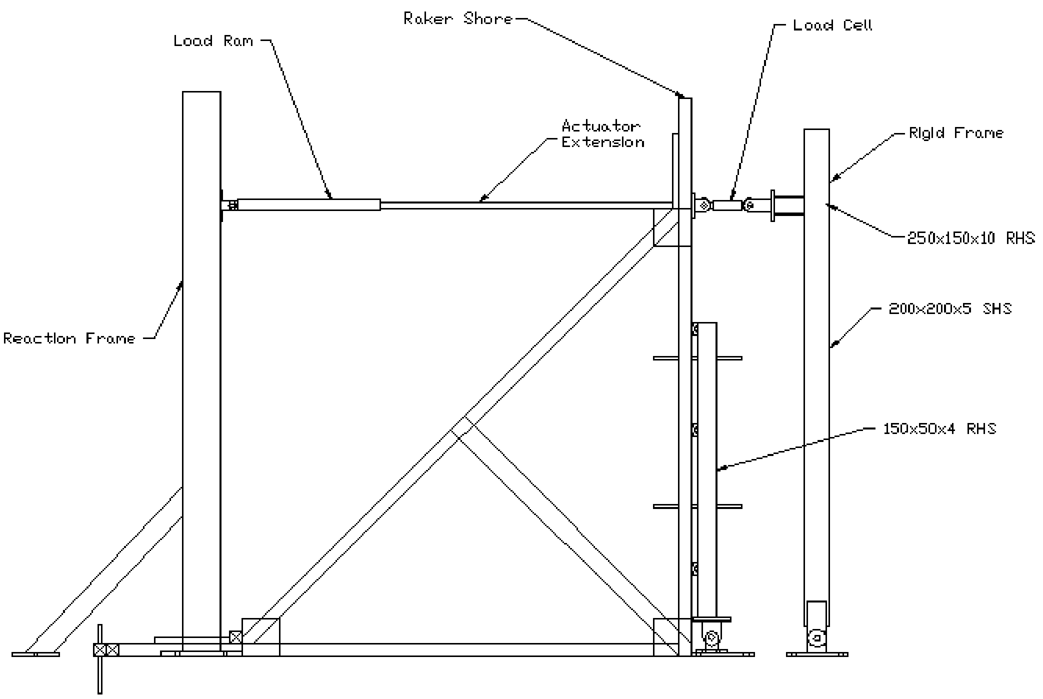

Figure 4 Elevation of test setup

Figure 5 Photo of test setup

6 RESULTS AND DISCUSSIONS

A total of five tests were performed on three sets of shores. The first two sets had two tests performed on each. The first test used the typical NZ USAR setup until failure occurred in the wall plate spikes. The second test had the spike connection strengthened sufficiently to move to the next failure mechanism in the hierarchy. On the third set of rakers, a test was performed to observe the effect of closing the gap between the wall and the wall plate using the same number of spikes (two) as that employed by NZ USAR. In this section, the shores are referred to as Raker A and Raker B. Raker A refers to the left hand shore when looking at the shores from the anchor plate, and Raker B to the right hand shore.

6.1 *Typical Setup used by NZ USAR*

Two tests were done on a typical set of shores used by NZ USAR (Test 1 and Test 3). A unidirectional load was applied to the shores until a load of approximately 20kN was achieved; this gives a satisfactory safety factor of 2. The graph of load-displacement for the two tests is shown in Figure 6. Test 1 shows a steady increase in load until approximately 14kN to 16kN where yielding of the steel spikes occurs, shown by the graph levelling off. This is slightly higher than the predicted plastic yield force of 13.5kN. Strain-hardening of the steel caused the load to slowly increase to a value of 19.1kN and 17.1kN when the test was stopped to minimise damage to the shores. Test 3 shows no evidence of spike yielding. This different behaviour with respect to Test 1 can be justified on the basis of a modification adopted for the setup in Test 3, where the overturning test wall was connected through steel bars to the load cells (Fig. 6). This modification was made in order to keep the distance between the point of application of the load and the hinge of the overturning test wall constant during the whole loading process, so as to better represent a real collapsing wall. Because of such a modification, another load path took place in the test setup, and part of the load was transmitted through the steel bars connecting the overturning wall with the load cells resulting in a decrease of the shear forces in the spike connection, and in a delay of their yielding (Pettigrew 2005).

The results from these tests are very encouraging for NZ USAR. The individual shores where able to carry a load in excess of 20kN which gives a safety factor of 2 for the desired capacity of 10kN. The bending of the spikes is a ductile failure which is highly desirable as it gives warning of failure. The rescue teams will be able to see the spikes bending and evacuate the building before collapse occurs.

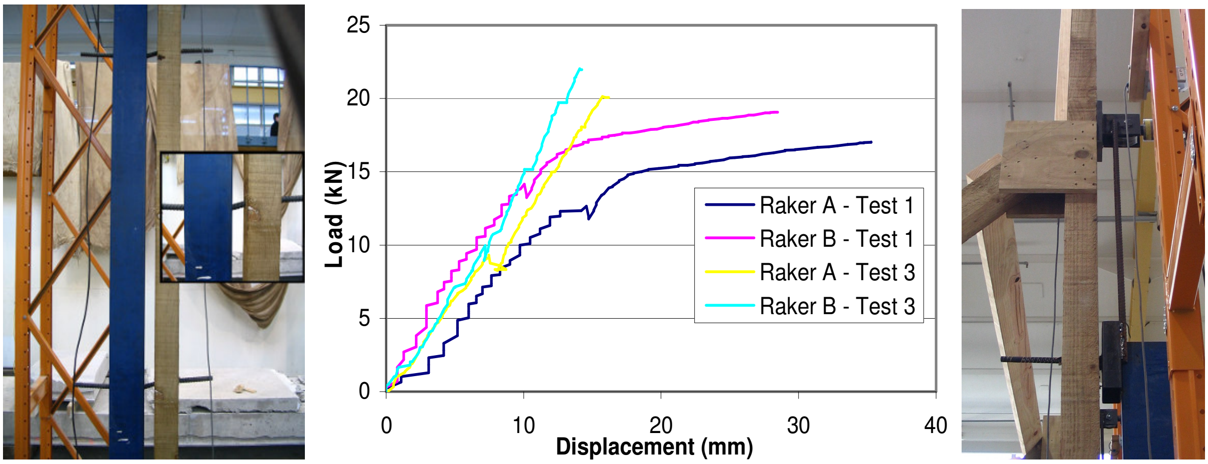

Figure 6 Test 1 spike connection failure (left), load-disp. graph (centre), setup modification of Test 3 (right)

The strains in the raker members were measured in both tests and plotted against the load, the plots for Raker B in both tests are shown in Figure 7. The linearities of these plots show that the raker was still in the elastic range and no flexural buckling had occurred.

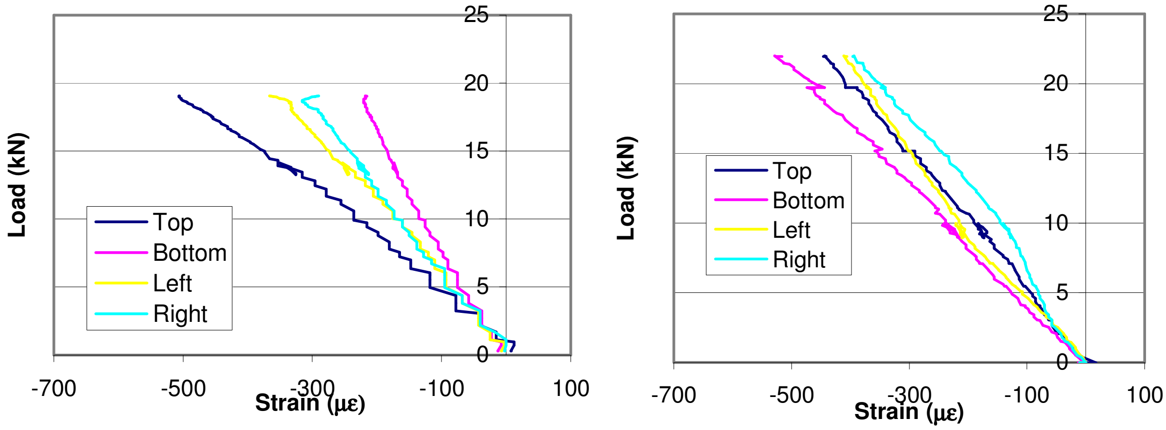

Figure 7 Strains in Raker B of Test 1 (left) and Test 3 (right)

6.2 *Strengthened Spike Connection*

The second tests (Test 2 and Test 4) on the first two rakers were performed using strengthened spike connections to identify the next failure mechanism in the hierarchy. The spike connection was strengthened by increasing the number of spikes to five and increasing the yield strength of the spikes to Grade 500, as illustrated in Figure 8. It is acknowledged that the connection would not be set up like this in practice however it was done purely to observe the next failure mechanism.

The load-displacement graphs for these tests are displayed in Figure 8. The loads initially increased quite slowly as the residual deformations from the previous tests (Test 1 and Test 3) were recovered. The load then increased relatively linearly until the raker members began buckling where the stiffness decreased. The load reached a maximum value when flexural buckling of the shore was in full development. The load then gradually decreased and displacements increased while the raker continued to buckle until complete rupture of the raker occurred (Fig. 9). The maximum loads in the shores in test 2 were 61.2kN and 66.1kN, and in test 4 they were 66.6kN and 74.2kN. These differences were expected to be due to natural variations in the timber. Buckling of the raker members was a brittle failure which occurred at approximately the midpoint of the brace and end of the member.

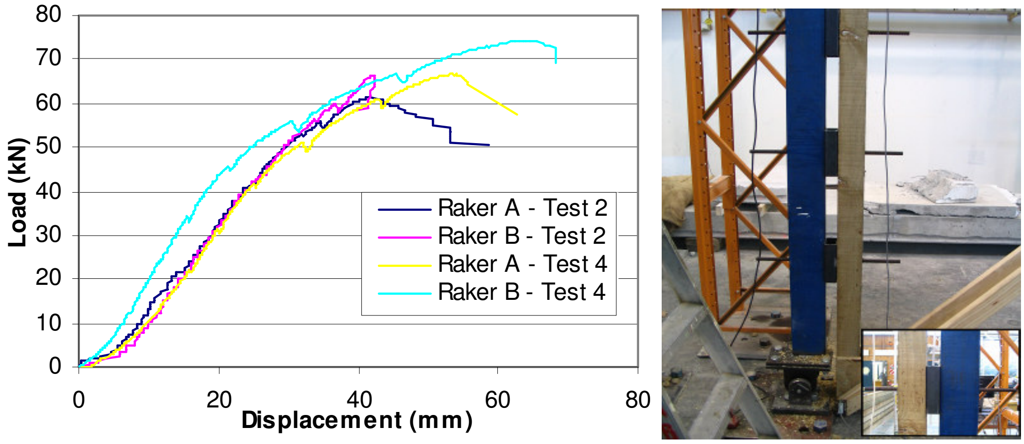

Figure 8 Load-displacement graph (left) and strengthened spike connection (right) - Test 2 and 4

This is an unexpected result as the behaviour is different than predicted. After spike failure, the next failure mechanism was expected in the cleats at a load of 14.7kN, followed by the wall plate in tension at a load of 34.9kN, then the bearing of the wedge from the cleat and raker at 35.7kN, and lastly the observed flexural buckling failure at an expected load of 55.7kN. Three predicted failure loads were exceeded; these can be explained by some inaccuracies in the assumptions made in the analytical verifications, such as no load transfer through friction or the gussets in the cleats, etc.

Figure 9 shows the flexural buckling in the raker members. The strain remained relatively linear until the onset of flexural buckling and the strains on opposite faces diverged. The load-strain graph reached an ultimate load then the load dropped off while the strains increased until rupture of the member.

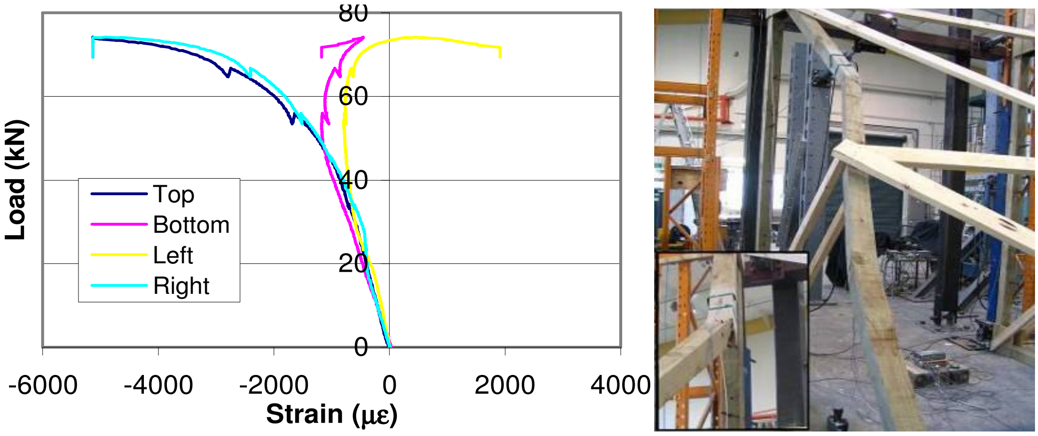

Figure 9 Strains in Raker B (left) and raker shores at collapse (right) - Test 4

Timber samples were collected from the rakers at the point of failure and the average moisture content was 76.0 and 81.8 percent for tests 2 and 4 respectively; these are very high for construction timber. The results of these tests show the shores performed adequately under very wet conditions. This is important as the shores need to perform well in all weather conditions. Test 4 was the only test with any significant cleat displacements recorded. The nails began yielding at approximately 74kN.

6.3 *Closing of the Gap using two Spikes only*

The final test performed was to determine the effect of closing the 50mm gap between the collapsing wall and wall plate using only 2 spikes (Grade 300). This is a possible small improvement NZ USAR may implement to increase the strength of the raker shore. A higher collapse load is expected by closing this gap. The load-displacement plot is illustrated in Figure 10.

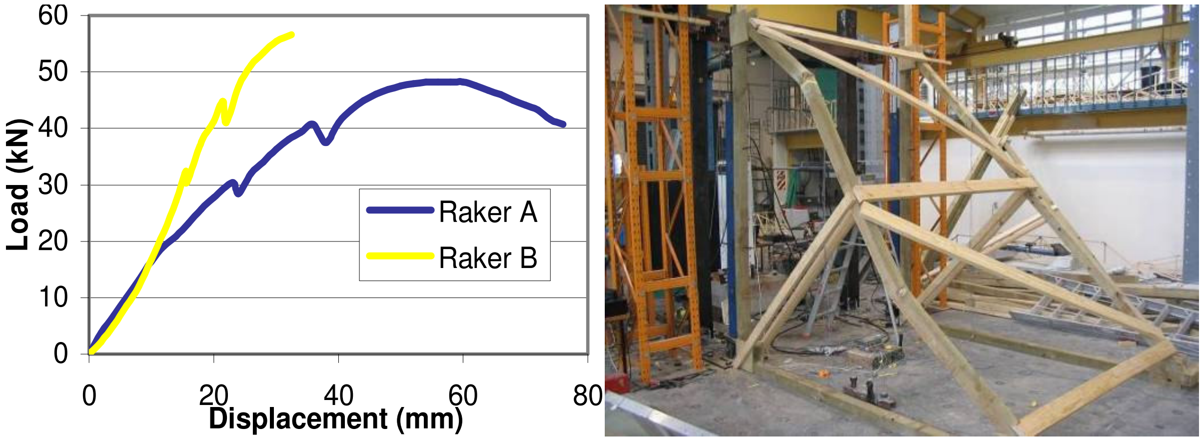

Figure 10 Load displacement curve (left) and failure of Test 5 (right)

The key finding in this test was that by closing the gap the spike connection was strengthened sufficiently such that flexural buckling failure occurred, as in Test 2 and Test 4. The maximum loads reached in the shores were 48.6kN and 57.0kN, less than that for the other flexural buckling failures of Test 2 and Test 4, because of the presence of a substantial knot. This result has mixed outcomes for NZ USAR. On one hand the closing of the gap significantly increased the strength, but on the other, a more brittle failure was imposed. To gain a better understanding of the exact performance of the spikes and rakers with a closed gap, more tests should be done to improve the statistical credibility of these results.

6.4 *Summary of Results*

Table 3 allows for easy comparison to be made of the data from these tests. The top block of the table shows the tests done with the typical setup used by NZ USAR only loaded to approximately 20kN. The second block compares the tests with strengthened spike connections which caused flexural buckling failure. A number of tests were done on each set of shores; the corresponding shores are aligned vertically.

Table 3 Summary of test results

		Test 1 - Shore 1		Test 3 - Shore 2	
		Raker A	Raker B	Raker A	Raker B
Ultimate Load	kN	18.21	20.36	20.08	21.97
Max Displacement	mm	22.48	35.29	16.19	14.24
Shore Stiffness	$kNmm^{-1}$	1.348	0.973	1.279	1.563
Raker Stiffness	$kNmm^{-1}$	14.67	15.9	8.7	14.37
Max Cleat Displacement	mm	0.3		0.6	

		Test 2 - Shore 1		Test 4 - Shore 2		Test 5 - Shore 3	
		Raker A	Raker B	Raker A	Raker B	Raker A	Raker B
Ultimate Load	kN	61.2	66.14	66.61	74.16	48.6	57
Max Displacement	mm	41.72	42.26	53.66	64.08	57	32
Shore Stiffness	$kNmm^{-1}$	1.96	2.04	2.07	2.54	1.64	2.11
Raker Stiffness	$kNmm^{-1}$	13.64	16.37	8.97	13.64	-	-
Max Cleat Displacement	mm	3.3		11.6		-	
Moisture Content	%	76.03		81.81		91.83	

7 CONCLUSIONS

As a result of the differences between U.S. and New Zealand timber properties, and due to modifications NZ USAR makes to the FEMA modules, it was necessary to verify the capacity of the New Zealand raker shores. The verification was done analytically and experimentally, with the results providing valuable information on the actual performance of the shores.

The analytical verification using NZS 3603 found that a majority of the shore was satisfactory, with the exception of the nailed cleat and the spike connection. The design strength of the nailed cleat and spike connection were 6.43kN and 8.69kN respectively; both of which are lower than the required design load of 10kN. From the design strengths, the expected mean failure load was predicted, a factor of safety calculated and then an expected hierarchy of failure obtained. The expected failure of the shores was at 13.52kN, due to bending in the spikes.

A total of five full-scale tests were performed on three sets of shores. The first key result was that shores currently used by NZ USAR are sufficient. Yielding of the spikes occurred at approximately 15kN, which correlates well to the predicted result of 13.5kN. The strength continued to increase due to strain-hardening of the spikes. The tests done on the current NZ USAR setup were tested to a factor of safety of 2, corresponding to a load of 20kN, which indicates there are no changes needed to the shores. The failure was ductile which is highly desirable as it gives warning of failure and allows time for evacuation of the structure it is supporting.

Preventing failure of the spike connections by increasing their strength will significantly increase the shores capacity. Thus the failure mechanism becomes flexural buckling in the raker member. The applied loads for which flexural buckling occurred ranged from 48.6kN to 74.2kN, which is much higher than the required load. The disadvantage of flexural buckling is brittle failure. By simply closing the 50mm gap between the collapsing wall and wall plate, the strength of the spike connections increased sufficiently to cause flexural buckling failure. Therefore, a simple change to the shore setup can significantly increase the capacity of the shores if it is required by NZ USAR. The nailed cleats did not fail as predicted due to the assumption that no load would be transferred through the gusset or through friction. It is now clear that these factors will significantly increase the strength of the shores. However, it is difficult to determine the exact contribution of these elements.

The overall conclusion is that raker shores constructed by NZ USAR provide the required resistance, despite being made from a different timber species and including the NZ USAR modifications.

ACKNOWLEDGEMENTS

We express our appreciation and thanks for the time and efforts of the members of Task Force 2, NZ Urban Search and Rescue, Christchurch and the efforts and input of the Department's technicians, particularly John Maley and Russell McConchie.

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