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ABSTRACT

The stability in the down-aisle direction of high rise storage racks is usually provided by the beam-to-column and base-plate connections. In order to evaluate the effect of the beam-to-column connections, a complex test rig has been designed and developed at the University of Sydney to test rack sub-assemblages which are the lower levels of high rise rack systems. The rig is designed to apply up to 500 kN vertical load via gravity load simulators. Pins at the top and bottom of the uprights under test remove the effect of the base-plates on the lateral stability. To apply horizontal loads, a down-aisle controlled displacement is introduced at the top of the frame and a load cell is used to measured the force required to obtain this displacement. The corresponding vertical loads are then applied via 2 hydraulic actuators mounted in the gravity load simulators. The rig allows both the loading and unloading in the post-ultimate range to be followed.

The paper describes the development of this test rig, and preliminary testing performed using the test rig. The ability of the rig to follow the unloading curve is demonstrated.

INTRODUCTION

High rise storage racks are used throughout the world for storage of materials which are normally contained on timber pallets. These structures may be up to 30 metres tall, 100 metres wide and 150 metres long. Generally they are composed of cold-formed steel members which can be considered as light gauge. Racks are generally made up of a number of components which are the uprights, beams, bracing and connections.

The uprights, as the name suggests, are the vertical members. They are generally thin-walled mono-symmetric sections that can have a number of different cross-section shapes. These elements have circular, square or diamond-shaped slots along their length on the front face (web) of the uprights. The beams are the horizontal members that support the pallets and are generally formed by welding together 2 thin-walled lipped channels to form a closed section. On the ends of these beams are welded endplates which have steel teeth or tangs which slot into the front face of the uprights to form the beam to upright connection. This simple mechanical connection method allows beam levels to be adjusted if required.

Racks are braced in the transverse or cross-aisle direction between the uprights. In Australia and Europe these bracing members are connected to the uprights using the rear flange slots and a simple bolted connection. In the USA they are usually welded. Bracing in the longitudinal or down-aisle direction of high rise racks consists of spine towers at selected bays. These spine towers use bracing at the rear of the bay. However, in order to optimise the

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use of space, frames are often designed 2 bays deep so that pallets can be placed in a front or rear bay from any given aisle. Therefore, in many cases, no significant down-aisle bracing is present. Even when it is present in a high rise rack, it provides only partial down-aisle sway restraint.

Being storage structures, racks are subjected to very high gravity loads from the stored materials. The beams and beam-to-upright connections are also designed for an additional vertical impact load. As they are usually constructed inside massive warehouses, storage racks are normally not subjected to horizontal wind loads. Although there are no applied horizontal loads with the exception of earthquake loads in some regions, construction tolerances and connector looseness require an equivalent out-of-plumb horizontal load to be taken into account during design. An out-of-plumb frame undergoing vertical loading is statically equivalent to a vertical frame with both equivalent horizontal and vertical forces as shown in Figure 1. The magnitude of these out-of-plumb loads is specified in the appropriate design standards (AS4084 (1993), RMI (1997), FEM (2001)) and are typically around 0.5 to 1.5% of the vertical load. Further detail is given in Chapter 11 of Hancock, Murray and Ellifritt (2001).

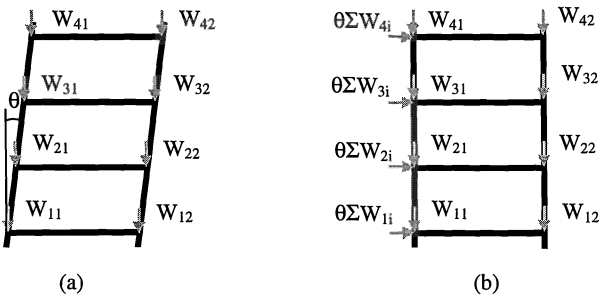


Figure 1 (a) Out-of-plumb frame; (b) Statically equivalent frame

Buckling and failure of storage racks occasionally occurs and often results in the need to replace all of the stored material which is invariably worth more than the structure itself. One of the common modes of failure occurs when the down-aisle rigidity is inadequate to prevent down-aisle sway and collapse can occur. Stability of frames unbraced in the down-aisle direction is heavily influenced by connection stiffness within the frame. The two main parameters determining lateral frame stiffness against down-aisle sway are the beam-to-upright connection and the upright-to-floor connection. The beam-to-upright connection is critical for much of the stability and load carrying capacity of pallet racking systems and is investigated more fully in this paper.

As mentioned previously, the beam has a welded endplate with tangs or pins which slot into the holes on the face of the uprights. A safety clip locked through one of the slots prevents accidental unlocking or knocking-out of the beam. The endplate also serves as a bearer on the upright. When a beam is loaded, the ends rotate outward and the connection tends to tighten up. Through distortion of the wall of the upright and distortion of the beam, a degree of beam to upright moment transfer occurs through this connection and so it is classified as semi-rigid.

There is currently only one test which can be used to determine the moment-rotation curve of a connection. This is the cantilever test (AS4084 (1993), RMI (1997), FEM (2001)) which gives a moment-rotation curve which becomes non-linear relatively early during static loading as is typical of semi-rigid connections. The joint stiffness can be obtained from this test but there is no general agreement on a method for determining an acceptable value. The alternate test is the portal sway test (AS4084 (1993), RMI (1997), FEM (2001)) which can only be used to get an initial linear stiffness of a connection and cannot be used to determine the moment-rotation curve.

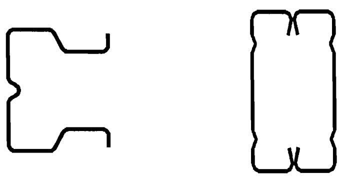
Correlation of joint stiffness between simple portal sway and cantilever tests and actual high-rise racks is one of the aims of full-scale testing. Results from full-scale tests can also be used to verify computer models, including simple 2-dimensional, 2nd order elastic models. Information can also be obtained on post-ultimate unloading behaviour of the racks and this can be correlated with a simple second order rigid-plastic model. This paper describes the development of a test rig and preliminary results of full-scale testing for the purposes just described.

DEVELOPMENT OF TEST RIG

Test frames

The test rig has been constructed to simulate the loading on the lower storeys of a high rise rack. Initially the effect of the beam-to-upright connections on lateral stability is the main focus of the project. To remove the effect of the base-plates, the sub-assemblage is connected to the ground at the bottom and to the loading frame at the top by specially designed pinned connections which do not allow the development of any moment in the down-aisle direction.

The test frames consist of either 3.54 in. (90 mm) or 4.33 in. (110 mm) deep uprights with a wall thickness of 0.094 in. (2.4 mm) and 5.12 in. (130 mm) deep, 8 ft 10 in. (2700mm) long beams with a wall thickness of 0.063 in. (1.6 mm). Typical upright and beam cross-sections can be seen in Figure 2. The test frames are 14 ft 9 in. (4500 mm) high, 1 bay deep with 3 levels of beams at equal spacings as in Figure 3.



(a) (b)

Figure 2 Typical member cross-sections: (a) Upright; (b) Beam

As shown in Figure 3 the racks are orientated such that the down-aisle direction is parallel to the strong floor beams. These strong floor beams are at 4 ft (1219 mm) spacings. In the cross-aisle direction, 2 rectangular hollow sections (RHS) at 8ft 10 in (2700mm) centre-to-centre are connected to 3 of the strong floor beams. Two of the pin connections are attached

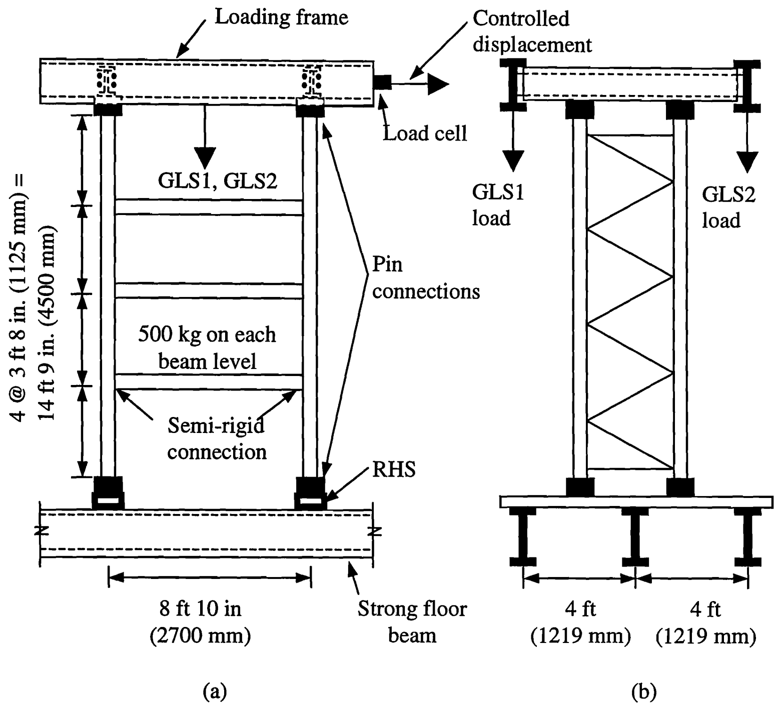


Figure 3 Test setup: (a) Down-aisle view; (b) Cross-aisle view

to each RHS at a spacing of 3ft 11in (1200 mm) and the uprights are then bolted to these connections using their standard base-plates. (See Figure 3). In this way the loads from the uprights can be transferred to the strong floor beams via the RHS.

Loads

Pallets sitting on the 3 levels of beams provide relatively small beam loads of 500 kg to each level. These loads act to lock-in the beam-to-upright connection.

The forces experienced by the lower sections of upright of a high rise rack are simulated by applying loads at the top of the uprights of a frame sub-assemblage. High vertical loads combined with a small component of horizontal load attempts to duplicate the effect of higher storeys with an initial imperfection.

With the exception of the beam loads, the loads are applied via a rectangular loading frame constructed from I-sections which is attached to the top of the uprights as can be seen in Figure 3. Connected to the loading frame both in front of and behind the rack, are two

250 kN hydraulic actuators. These are used for the vertical loading. To ensure the verticality of the downwards load, each actuator has been mounted in a gravity load simulator (GLS). The gravity load simulators sway with the frame, as shown in Figure 4, in order to keep each actuator in an upright position. To apply horizontal loads, a down-aisle controlled displacement is introduced by a pulley and threaded rod at the loading frame and a load cell is used to measure the force required to obtain this displacement, as shown in Figure 3.

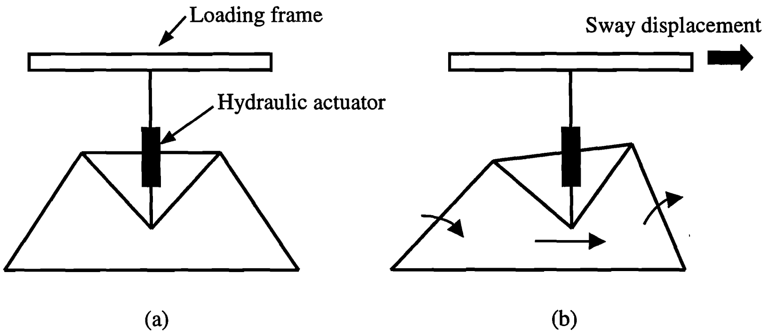


Figure 4 Gravity Load Simulator: (a) Without horizontal load; (b) With horizontal load

As the rack and hence the GLS sway, a small opposing horizontal stabilising force is introduced. This restraining force is due to the GLS moving away from its central, perfectly vertical position as is demonstrated in Figure 5. The restraining force results from the locus of the instantaneous centre of rotation (ICR) of the triangular frame of the GLS not being perfectly above the point of attachment (A') of the actuator to the GLS. Although of a small magnitude relative to the vertical load, it may be of the same order as the horizontal load commonly used in the design of racks and therefore cannot be ignored. The magnitude of this force is calculated based on the statics and kinematics of the GLS.

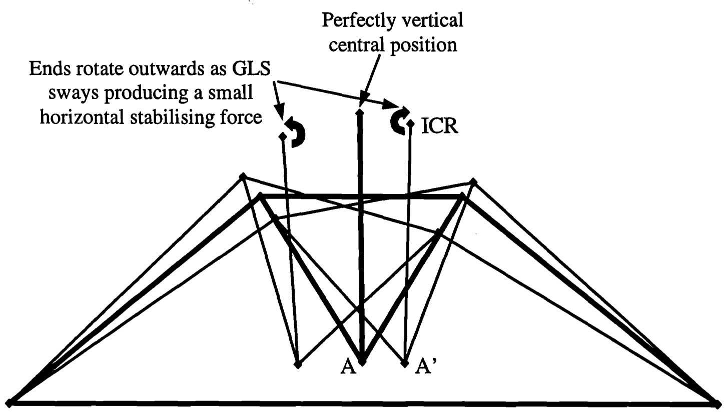


Figure 5 Restraining force being introduced by GLS

In order to load the frame, a horizontal controlled displacement is first applied by a wheel and threaded rod and the applied horizontal load determined from the load cell. The resultant load on the frame is therefore the applied load minus the computed restraining force from the GLS. Once the desired resultant horizontal load has been reached, a vertical load in a fixed proportion to the horizontal resultant load is then applied using the hydraulic actuators. Due to the non-verticality of the frame, the P-Δ effect will cause further horizontal displacement which must be applied by the wheel and threaded rod. Further, a reduction in the resultant horizontal force due to increasing restraint from the GLS requires additional controlled displacement. More horizontal displacement may then be wound on until the horizontal load to vertical load ratio is as desired and equilibrium is reached. Deflection readings are then taken.

Instrumentation

At the load frame level and at each of the beam levels, displacement transducers measure the deflection both at the front and back of the frame. A further pair of displacement transducers is used to determine the amount of twist in one of the uprights. Theodolites and levels are used to monitor the out-of-plane movement of the rack and the vertical deflection of the uprights respectively.

INITIAL TESTS

Procedure

As previously described, the focus is initially on the effect of the beam-to-upright connection on lateral stability and therefore pins are used to connect the rack at the base to the ground and at the top to the loading frame to prevent any moment transfer. At the commencement of testing, the only loads on the frames are the beam loads. The additional vertical and horizontal loads are then applied in regular increments and readings are taken at each increment. In the tests already undertaken it was planned that the horizontal load applied would be approximately 2.5 - 3% of the vertical load.

As the frames approach their maximum load, the horizontal deflection experienced at each increment increases dramatically. Close to the maximum load, the application of vertical load causes a large decrease in the resultant horizontal load and at this stage, the loading increments are decreased. Ultimately a point is reached where the required horizontal to vertical load ratio cannot be obtained and this signifies the start of the unloading curve. The unloading curve is then obtained by increasing the deflection in increments but still maintaining the required vertical to horizontal load ratio. Three frames have been tested to date and Figure 6 shows the graph obtained from Test 3 of vertical load vs horizontal deflection at the top of the frame. The unloading was successfully followed after the ultimate load.

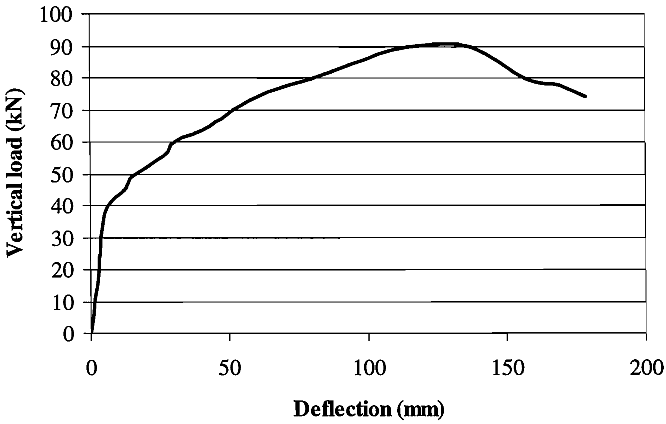


Figure 6 Sway load – deflection graph for Test 3

ANALYSIS AND RESULTS

Elastic analyses

A frame analysis computer program developed at The University of Sydney (PRFSA (CASE, 2000)) is being used to analyse high rise racks. Second order elastic analysis is being used to determine the apparent beam-to-upright joint stiffness of the test frames. The model is 2-dimensional with pinned bases and horizontal and vertical loads applied at the top of the uprights as shown in Figure 7. The semi-rigid beam-to-upright connections are represented using 30 mm long horizontal elements at both ends of each beam.

Joint stiffness is given by:

$$F = \frac{EI}{L} \tag{1}$$

where F = joint stiffness;
E = elastic modulus;
I = second moment of area;
L = connection length.

Using Equation 1, connection element properties can be adjusted for different values of joint stiffness. In the model being studied, L = 30 mm and I = second moment of area of beam connected to the uprights, therefore only the elastic modulus of the connection element needs to be adjusted to change the joint stiffness.

In order to determine the apparent stiffness of the test frames, the 2-D model is subjected to the same loading as the test frame under consideration. The elastic modulus of the connection

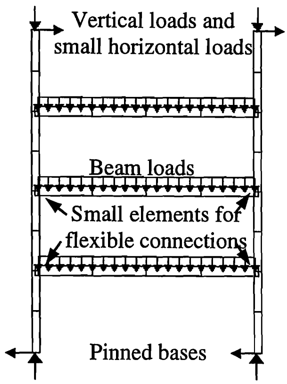


Figure 7 2-D Computer model of test frame

elements and hence the joint stiffness is varied until the 2[nd] order horizontal deflection at the top of the model is the same as the horizontal deflection at the top of the test frame. Once E is known, F can be calculated. A series of curves have been developed for different joint stiffnesses as shown in Figure 8.

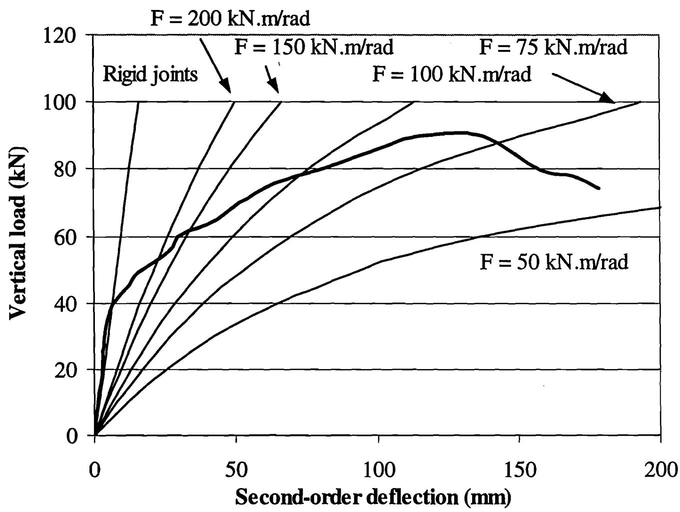


Figure 8 Sway – second-order deflection curves for various joint stiffnesses

From tests it appears that initially the connections act as rigid joints but then become increasingly flexible as the loads and the sway increase. For the tests carried out to date, the apparent joint stiffnesses at the maximum loads were 81, 83 and 78 kN.m/rad for Tests 1, 2 and 3 respectively. Beyond the ultimate load, the joint stiffnesses continued to decrease.

For cantilever tests conducted to AS4084 Clause 8.4.1.2 (RMI Clause 9.4.1.1), using the same member sizes as used in the testing of these sub-assemblages, the beam-to-upright joint stiffnesses obtained from AS4084 Commentary Clause C8.4.1.4 (RMI Commentary Clause 9.4.1.3) with R.F. = 1.0 were approximately 35 kN.m/rad. Figure 9 (CASE, (2000)) shows typical cantilever test results.

Plastic analyses

Investigation of the post-ultimate unloading of the frames is carried out using a simple second order rigid-plastic model. This rigid-plastic model is a single upright pinned at the base, loaded horizontally and vertically at the top and with applied loads moments along its length which represent the loads and constant moments being transferred through the beam-to-upright connections in the rack frame. The rigid-plastic model is shown in Figure 10.

From this simple model the following equation can be obtained:

$$3M = P_v \Delta + P_h H + 1.5 P_b \Delta \qquad (2)$$

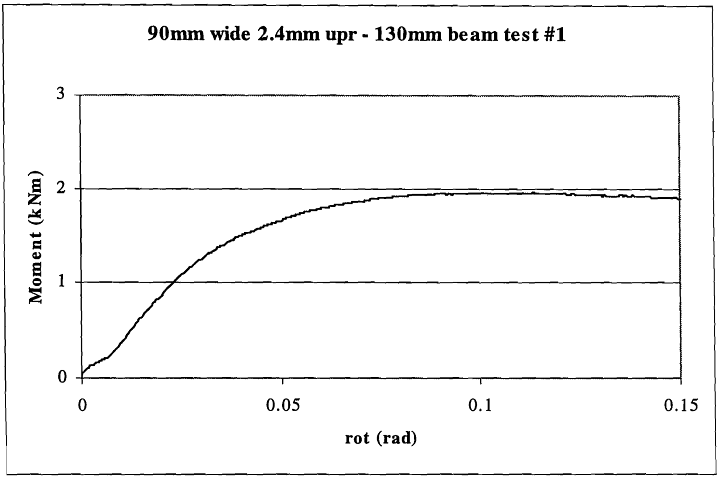
where M = moment transferred from beam to upright
P_v = vertical load on a single upright
Δ = horizontal displacement at top of upright
P_h = horizontal load on a single upright
P_b = vertical load transferred from beam to upright

P_v and P_h are in a fixed proportion to each other.

Different moment values (M) can be used to produce a series of falling curves on the load-deflection graph as shown in Figure 11. For the appropriate value of moment transfer, this provides information on the post-ultimate behaviour of high rise storage racks.

For the member sizes used in the tests carried out to date, the results from a typical cantilever test are given in Figure 9. The ultimate moment capacity of the beam-to-upright connection according to this cantilever test is approximately 1.9 kN.m in line with the falling curve in Figure 11.

For the 3 tests carried out to date, substantially different results for moment transfer at the ultimate load were obtained for each. The difference between the 3 is attributed to different deflections at the ultimate load for the 3 tests as the moment transfer itself is a function of deflection. A higher deflection gives a larger beam-to-upright joint rotation and as can be seen in the cantilever test results in Figure 9, a larger joint rotation gives a higher moment transfer value. When correlations between the cantilever tests and full-scale are made, an iterative process could be used to determine the ultimate load and the post-ultimate behaviour.



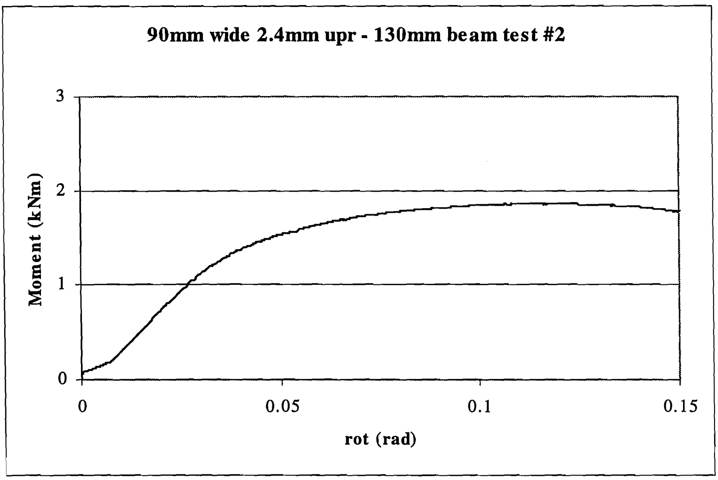


Figure 9 Typical moment-rotation curves from cantilever tests

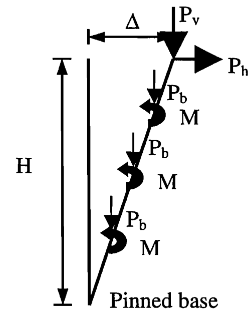


Figure 10 Second order rigid-plastic model

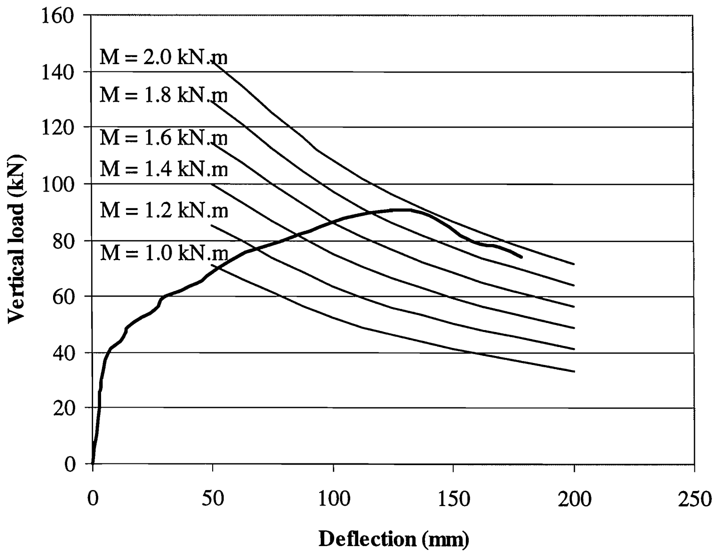


Figure 11 Sway – deflection curves for various values of beam-to-upright moment transfer

CONCLUSIONS

This paper provides preliminary results from a newly developed test rig for investigating the sway stability of high rise rack structures. The rig tests a sub-assemblage representing the lower levels of a high rise racking system. In particular, the influence of beam-to-upright connections is being investigated for both the loading and unloading (post-ultimate) conditions. Preliminary results show that the joints remain rigid to a point when they become semi-rigid. The values of joint stiffnesses back-calculated from second-order elastic analyses are much higher than those determined from simple cantilever tests. Further comparisons with portal sway tests to determine joint stiffness are ongoing, as are a much wider range of frame tests. It is intended that base-plate flexibility is also included in future tests.

APPENDICES

References

Centre for Advanced Structural Engineering (2000), "Beam to Column Connection Strength Tests of Steel Rack Frames – CT0031", Investigation Report S1285, Department of Civil Engineering, University of Sydney NSW, 2006.

Centre for Advanced Structural Engineering, (2000), PRFSA Version 4.0, A Computer Program for Structural Analysis of Plane Rigid Frames, Department of Civil Engineering, University of Sydney, NSW, 2006.

Federation Europeenne de la Manutention, (2001), "FEM 10.2.02 – The Design of Static Steel Pallet Racking", Birmingham, Great Britain.

Hancock, G.J., Murray, T.M. and Ellifritt, D.S., (2001), "Cold-Formed Steel Structures to the AISI Specification", Marcel Dekker Inc., New York, NY, USA.

RMI. (1997), "Specification for the Design, Testing and Utilization of Industrial Steel Storage Racks', Charlotte, NC, USA.

Standards Australia. (1993), "Steel Storage Racking – AS4084", Sydney, NSW, Australia.

Notation

E	Elastic modulus
F	Beam-to-upright joint stiffness
I	Second moment of area
L	Beam-to-upright connection element length
M	Moment transferred from beam to upright
P_b	Vertical load transferred from beam to upright
P_h	Horizontal load on a single upright
P_v	Vertical load on a single upright
Δ	Horizontal displacement at top of upright

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