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SEISMIC PERFORMANCE OF CONCRETE ENCASED STEEL BUILDINGS WITH RIVETED JOINTS

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ABSTRACT

In the early 20th century, steel frame buildings were built to much lower earthquake standards and utilizing different materials from those used in modern construction. Riveted, built-up members were used instead of rolled sections, with joints and members encased in concrete for fire protection. These early steel buildings were designed based upon observations of past building performance rather than through detailed calculations and predictions of structural behavior. The walls were infill masonry and floors were typically reinforced concrete. The earthquake resisting strength and stiffness of the semi-rigid connections and masonry infill as well as the effect of floor slabs integral with their supporting beams are not well documented. Although riveted stiffened seat angle connections are not designed to resist moments, they can develop an appreciable moment capacity and exhibit a relatively ductile hysteretic behaviour which could be beneficially considered when evaluating frames built of these connections and subjected to small and moderate earthquakes. Structural engineers have found it challenging to make realistic predictions of the seismic performance of these buildings, many which are quite prestigious, in full service and often enjoying heritage protection. Examples in New Zealand include Auckland's Britomart Station and Guardian Trust Building, and Wellington's Tower Corporation, Prudential Assurance and Hope-Gibbons Buildings. To understand seismic behavior of these riveted connections, experimental testing and finite element modelling (FEM) of a typical component was first undertaken, followed by experimental testing and finite element modelling of a half-scale beam column interior joint of a building built in 1925 in Wellington, New Zealand. This work is ongoing and the final stage involving the retrofit measures to be taken to enhance the moment-rotation capacity of theses kind of connection.

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1. Introduction

For much of the first half of the 20 century, earthquake loads on buildings were either not considered or were significantly lower than are required by current design standards and the resulting gravity load-dominated structures tended to be constructed using moment-resisting steel frames with what would be considered today as semi-rigid or flexible types of connections. Fortunately the concrete encasement that was often used for fire protection purposes probably bestowed additional strength and stiffness on the joints and members of these older buildings that make up a small but significant part of our built environment. Historically, these buildings have performed reasonably well in severe earthquakes. Assessing the seismic adequacy of such buildings using a particular example is the aim of the research reported in this paper. Accurate assessment of the strength and hysteretic capabilities of these riveted and encased joints may show that they make a significant contribution to the seismic performance of a building. This could significantly lessen the extent and intrusiveness of any retrofitting required, particularly in less active seismic zones. Furthermore, there is a need for localized seismic retrofit strategies that could be implemented with minimum disturbance to the occupants, preferably without the need to remove the concrete encasement to expose the connections. Although many studies have experimentally and analytically investigated the seismic adequacy of some types of riveted joints, there exists little literature on the cyclic behavior and seismic retrofit of riveted connections.

2. History of Riveted Joints

Prior to the 1920s, steel frames commonly were constructed as complex built-up members with gusset plates and built-up connections. In riveted structures, the entire steel frame was normally encased in concrete for fire protection. Few if any of these structures were designed for seismic loading, only wind load was considered prior 1930. In some older buildings, even wind load received minimal consideration. These buildings invariably included many stiff, strong un-reinforced or minimally reinforced masonry or cast in place concrete external walls and partitions. Structural engineers relied upon these walls and partitions to help resist lateral loads, but there is little evidence of calculation of the stiffness and resistance provided by these walls. Seismic design forces were considered structures in 1930-1955, but the seismic design forces were simplified and often much smaller than those used today. These early structures were highly redundant in that every beam-column connection had some strength and stiffness and many of these used two way moment frames. This meant that lateral resistance came from a moment-resistant connection plus a large but uncalculated stiffness provided by non-structural elements such as masonry walls and the concrete encasement.

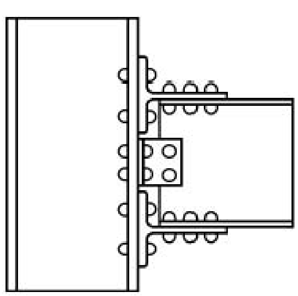

Fig1. Typical Riveted T-stub Connection

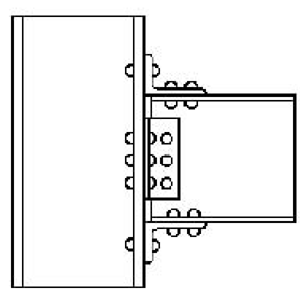

Fig2. Typical Riveted Clip-Angle connection

The connections illustrated in Figures 1 and 2 were used until the mid-1950s or early 1960s. Riveted T-Stub connections such as that shown in Figure 1 were used in the lower floors of taller buildings, and riveted Clip-Angle connections such as illustrated in Figure 2 were used in the top stories of tall buildings. The seismic behavior of these older riveted connections is quite complex (Roeder et al., 1996), but the connection design used in these older buildings remained basic and simple, because of redundancy in the system, conservatism in the design, and the reliance upon satisfactory past performance. After about 1960, high strength bolts began to replace the rivets in the T-stub and Clip-Angle connections but the connection details and geometry remained essentially the same as those used for riveted construction. Concrete encasement was also discontinued in favor of lighter fire protection materials. By this time, the seismic design procedures had evolved into methods similar to those used in modern seismic design.

3. Earthquake Performance History for Riveted Steel Structures

This type of building has performed reasonably well in past severe earthquakes, such as the 1925 Santa Barbara earthquake in USA and 1906 San Francisco earthquake (M= 7.9) and also in the Great Kanto earthquake in Japan(M=7.9) Most of the steel framed buildings survived, many with minimal damage and a number are still in service.

Furthermore modern moment-resisting steel frames with intended rigid connections subjected to the Northridge earthquake of 1994 performed well in an overall sense even though the rigid beam to column connections suffered large numbers of weld failures between the beam bottom flange and the column flange, effectively turning these into semi-rigid buildings with strong columns and weak connections. This shows that moment resisting steel frames tend to dependably remain standing in earthquakes well in excess of their calculated design capacity provided that the following three factors apply:

(1) The beam to column connections retain their integrity, with regard to carry shear and axial force, when their moment capacity is exceeded
(2) Inelastic demand is minimised in the columns, both demand due to general plastic hinging and, especially, demand due to local buckling or crippling failure
(3) The inelastic response is essentially symmetrical in nature and does not lead to a progressive movement of the building in one direction.

4. Past Experimental Results

A number of past tests (Roeder 1994 & Batho1934) on T-stub connections have been undertaken. Most of tests were on riveted connections subjected to monotonic behavior, but some tests examine cyclic inelastic behavior and high-strength bolts. Many of the riveted connections tested were concrete encased following the common practice of encasing for fire protection reasons. Tests (Roeder 1994) on concrete encased connections show that the concrete encasement may increase the strength and stiffness of the connection by as much as 30-50%, but the general behavior, including modes of failure and deformation limits, remains unchanged. However most of these tests did not include a floor slab, as is typically used.

5. Heritage Steel Structures with Riveted Joints in NZ

As the first stage of this research, the as-built drawings of a number of heritage steel buildings in Auckland and Wellington were examined and used to define the geometric parameters of the three FE joint models reported herein below.

Fig 3. Glass-enclosed riveted joint at Britomart Transport, Auckland Fig 4. Hope Gibbons building

Jean Batten State Building in Auckland Central and Hope Gibbons building, Wellington were chosen because of having a complete set of drawings.

6. Finite Element Analysis

6.1 FE models of two common riveted joints in NZ

In the first instance, two exterior beam-column joint models including "model A" (clip angle) and "model B" (T-stub) have been modeled using the general finite element analysis program, ABAQUS. The columns above and below the joint carried no axial load and were fixed at mid-storey level. Monotonic axial and shear loading was applied at the free end of the beam. The model included detailed modeling of rivet contact using reduced 20 node quadratic brick elements and an isotropic elasto-plastic material model with a Mises yield surface, rate independent plasticity, kinematic hardening and a fracture criterion based on limiting plastic strain. Element type C3D20R (Reduced 20-node quadratic brick) was used. Although this element type provided high accuracy and prevented any unpredicted errors such as shear locking, it increased time and cost of running significantly compared with simpler alternatives. Model B was based on drawings of Auckland's Jean Batten building and Model A based on drawings of the Hope-Gibbons building in Wellington. (7th floor).

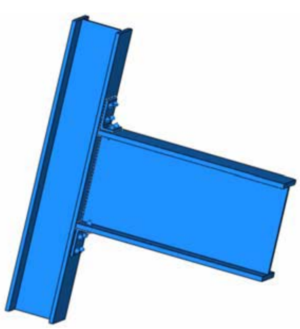

Fig 5. Clip Angle joint (model A)

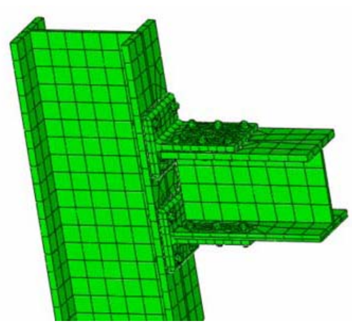

Fig 6. T-Stub joint (model B)

The "*contact pair, finite sliding*" option was chosen and also "*surface behaviour, no separation*" options were used to simulate the full contact condition between rivet shank and hole surface.

6.2 Mechanical Properties of Structural Steel

For the Abaqus model currently being developed, the material is modeled as non-linear elastoplastic and fracture is assumed to occur when the plastic strain exceeds a pre-set limit, with fracture denoted by the material stress falling from the ultimate tensile strength to a nominal minimum value over a short increase in strain.

6.3 Fasteners

Kulak(1987) recommended for rivet material ultimate tensile strength of 414 N/mm . He also recommended for rivet material shear strength of 310-410 N/mm which must increase 10-20% in the driven(installed) condition.

6.4 Analysis Result for these Type of connection

The failure mechanism of "model A" involved fracture of the rivets connecting top angle and column flange. The development of plastic strain in the most critical part of "model A" under tension load occurred in the middle of rivets, where plastic strain reaching the pre-set limit (which means moving beyond the strain hardening region and into the necking area); meanwhile other parts of the joint remain elastic. This failure mode is not desirable as it has been observed (Roeader at al 1994) that tensile yielding of rivets generally leads to less rotational capacity and lower moment capacity than flexural yield of beam flange or clip angles. In this model, the large plastic strains in the rivets occur at relatively small deformation levels. The failure mode of "model A" under shear load occurred in the stem of the bottom angle because of high plastic strain associated with flexural yielding. Flexural yielding of the angle stems results in greater ductility and rotational capacity than shear yield of rivets as occurred under tension loading. The failure modes of the T-stub connection "model B" are more complex and are influenced by the uncertainty in the tensile forces in the rivets due to prying action and the potential for panel zone yielding of the column.

Under tension load, tension failure of rivets is known to be likely to be the governing failure mode. The distribution of plastic strain in "model B" spreads from the middle of the rivet shank towards the both ends. The behaviour of the model B sub-assemblage under shear loading was strongly influenced by both the response of the T-Stub stem connection and by the shear yielding of the rivets between column flange and T-Stub angle. A combination of shear and flexural failure of rivets was the main failure mode due to shear load.

6.5 Verification of ABAQUS data

To validate the ABAQUS model, two experimental tee stub tension tests were undertaken. The test setup is shown in Figure 7. The tee stub representing the attachment to the beam flange is shown oriented vertically. The aim of the test was to measure the separation between the stem of the loaded tee stub and the base column to which it is bolted. The tension load is applied vertically upwards and the bottom section represents the typical column section to which the tee stub is attached There was a good compatibility between the results of the experimental tests and the ABAQUS model, as shown in Figure 8 except initial stiffness of numerical results is higher than experimental test resulting from high clamping force applied to both end of bolts in Abaqus environment.

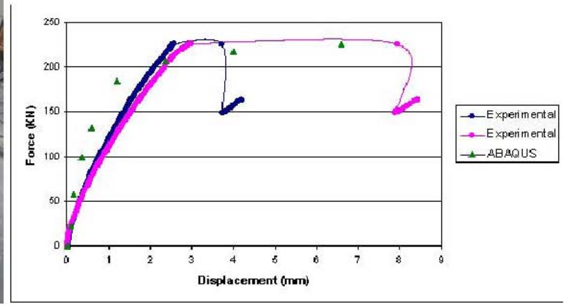

Fig 7. Tee stub experimental test Fig 8 . Experimental against FE result

7. FE Analysis of Half-Scale Model of Joint from Hope Gibbons Building

The next stage has been experimental testing and analytical modeling of a typical interior beam to column joint from the Hope Gibbons building, which is an 8 story building located in Wellington which was completed in 1925 and is now used for both commercial and residential purposes. It has an A grade historic places category. The joints use mild steel bolts and rivets as fasteners.

Due to limitations on testing space in the University laboratories, the experimental testing was conducted on a half scale model of a typical internal beam to column joint. Figure 11 shows one such joint from the building, with some of the encasing concrete removed to expose the steelwork. The half scale joint has been designed and built to represent as closely as practicable the subassemblage setup and load paths through the connection region, in order that the moment rotation characteristics can be determined experimentally and data acquired for validation of the FE model. Particular attention has been paid to duplicating in the model the observed clearances between joint components in the actual building.

The first experimental test on the “as built” assemblage was completed in mid-November 2009, providing data for comparison with the FE study and confirming the overall behavior of the joint. As seen in Figure 14, the initial agreement is quite satisfactory however the model will be further modified to improve the prediction of experimental performance and pattern of inelastic action observed.

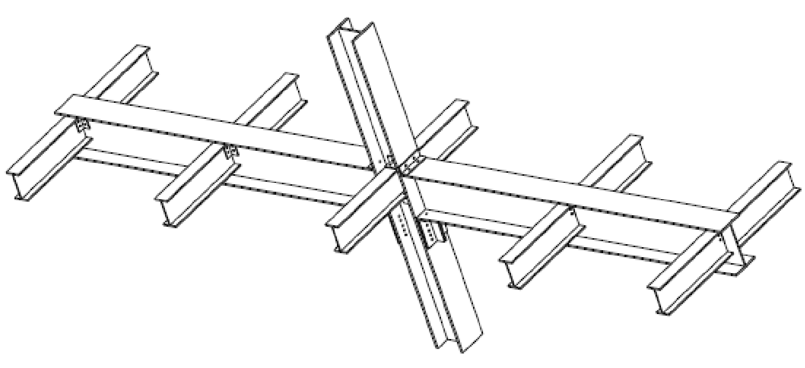

Fig 9. Half Scale model of HG Building joint (floor slab and encasing concrete not shown)

The “concrete damage plasticity” model in Abaqus was chosen to simulate concrete encasement and also slab behavior in tension and compression. Because of development of concrete cracking under load, the option of Abaqus/standard could not be used so Abaqus/explicit was used to simulate the behavior of assemblage. This led to long running times and has restricted the modelling to that under monotonic loading only, thus generating a monotonic predicted curve to compare with the spine curve obtained from the cyclic experimental testing.

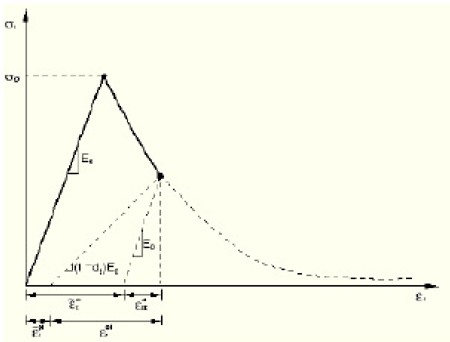

Fig 10. Post-cracking behavior of concrete

Fig 11. Exposed joint for taking samples

The post-cracking behavior for direct straining is modeled with tension stiffening, which allows the definition of strain-softening behavior for cracked concrete. This behavior also allows for the effects of reinforcement interaction with the concrete to be simulated in a simple manner. Tension stiffening is required in the concrete damaged plasticity model. from on-site samples, the compressive strength of the original slab concrete (21 MPa) was measured and tensile strength and stress-strain curves for post-cracked concrete defined according to Abaqus requirements. An elastic-plastic hardening behavior with respect to the strain rate was selected for the structural steel of the model frame and also for the reinforcing bars.

Time scaling was used to reduce the running times to acceptable lengths. This can lead to inaccurate results and it is recommended in the Abaqus manuals that the proportion of kinetic energy to internal energy should be kept under 10% during the run to ensure acceptable output. For both monotonic and cyclic loading, several time scaling values were tried and finally time scaling of 5E-6 was selected. In monotonic loading this resulted in an average energy ratio of about 1.4 % for monotonic loading and about 0.1% for cyclic loading.

8. Experimental Testing

The experimental test of the half-scale model representing the “as-built” conditions was completed early November and the data is still being analyzed. Cyclic testing was in accordance with the SAC protocol with 3 cycles of joint rotation at intestorey drifts of 0.3%, 0.5%, .75% , 1% , 1.5% , 2% , 3% applied by rotating the column which was pinned at top and bottom and loaded by a horizontal actuator at the bottom pin. The column ends coincided with the mid-storey heights above and below the joint being tested. The beams were also pin-supported at their ends, with the pin locations corresponding to mid-span in the actual building.

The joint displayed high flexibility with little tension capacity in the beam bottom flange to column flange connection, where the two connecting bolts soon failed. The slab provided greater tension capacity at top flange level, developing the cracks shown in Fig. 13, but with no fracture of reinforcing bars. Compressive capacity by bearing of the beam flanges against the column flanges together with the tensile capacity available through the slab reinforcement generated the moment capacity at larger rotations.

Fig 14 shows the experimental moment-rotation plot with the preliminary Abaqus monotonic result overlaid. It can be seen that there is good agreement between the two graphs although the Abaqus results still require correcting to the measured concrete compressive strength and stress-strain behavior of the concrete.

As expected from previous FE analyses of typical riveted joints, the connection behavior falls into the flexible category and had a maximum moment capacity of 30 kNm at a rotation of 0.015 radians. This means that for a full-scale connection the expected maximum moment capacity provided by as built joint would be about 120 kNm.

The minimum required moment capacity for HG building has been estimated as 550 kNm, underlining that this category of structure was not designed in the 1920s to resist lateral loading by frame action.

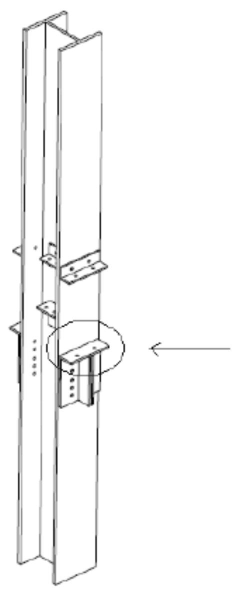

Fig 12 . The Location of weakest point of connection

Fig 13. Crack Pattern across the slab

Visible inelastic action in the joint first developed in the bolts holding the beams onto the bearing seats, with these bolts yielding in shear, then fracturing in shear with increased column rotation. From a rotation of approx 0.01 radians and beyond, the rotating column pushed the two beams apart, putting the reinforcement running past the column into the slab on each side into yield in tension. This reinforcement which comprised plan bars was able to slip in the concrete and so accommodate the increasing opening of the gap by tension yielding over an increasing length of bar without suffering necking and fracture close to the rotating column. There was no fracture of the reinforcement even at a column rotation of over 0.06 radians.

From fig 14 , it can be seen that cyclic loops for riveted joint are pinched and repeat of cycle

leads to considerably drop in stiffness. The results indicate that joint strength is limited by the strength of the tension load path through the slab in the as-built model. .
Fig 16 taken during the final step (.03 rad) of cyclic loading and show governing failure mode, which is shear failure of rivets between seat angle as it was expected besides the separation of seat angle and beam can be seen as well. This in itself is an undesirable failure mode and this non-ductile behavior must be modified to develop higher strength and more ductile modes of failure enhanced with appropriate retrofitting measures.

The method being considered to achieve this is to greatly enhance the tension load path through the slab so that the rotating column cannot push the two halves of the slab apart, but instead will generate compression yielding of the beam flange in contact with the column flange. Development o f this enhanced tension load path through the use of carbon fibre wrap laid onto the top of the concrete slab and extending past the column is the first option being considered.

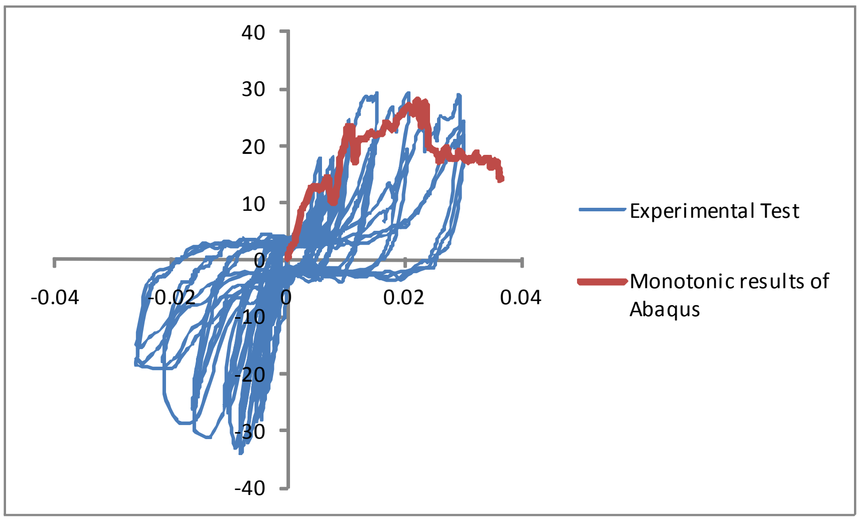

Fig 14. Moment-rotation curves for half-scale model (KN-m)

Fig 15. Test Setup

Fig 16. Fracture of bolts between beam and angle

9. CONCLUSIONS

From experimental testing and analytical modeling described herein the following conclusions can be made:

10.1 In these early buildings, engineers relied on the strength of unreinforced masonry or lightly reinforced concrete walls to help resist lateral loads, so joints were designed just to carry gravity load only. The Hope Gibbons model is a good example of this kind of design method in 1925.

10.2 To have a better seismic behavior of riveted joints like higher ductility and moment capacity , it is important to understand the possible failure modes to prevent undesirable failure modes like shear yielding of rivets.

10.3 Experimental test of a representative as-built half scale model show pinched cyclic loop and low energy dissipation for this kind of joints prevalent in 1920s.

10.4 Many riveted joints have a undesirable failure mode which shear failure of rivets or bolts group in top and seating angle lead to non-ductile behavior and need to be considered while redesign for retrofit measures.

11.5. Hope Gibbons building with combination of bolts and rivets as fasteners fall into flexible category and need to be retrofitted to a higher level of stiffness.

11.6 The strength, stiffness and ductility contribution of the perimeter walls is also important in this building and this is being determined in an associated research project.

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