EXPERIMENTAL STUDIES OF BEHAVIOUR OF COMPOSITE BEAM-COLUMN FLUSH END PLATE CONNECTIONS SUBJECTED TO SEISMIC LOADING

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Abstract. *Steel-concrete composite systems have seen extensive use in recent decades because of the benefits of combining the two construction materials. Reinforced concrete is inexpensive, massive and stiff, while steel members are strong, lightweight and easy to assemble. The object of this paper is to provide deeper insight into the nonlinear seismic response on the behaviour of composite structures and how they are influenced by various loading conditions. The results of the experimental study enable the authors herein to develop a new and improved shear connectors and beam-column connection for composite steel-concrete structures under seismic loading. The experimental results showed a notable strength and ductile behaviour. An optimised and economic shear connector and blind bolts was obtained. The failure patterns, failure mechanism, ductility and stiffness degradation are discussed. Furthermore, the finite element analyses are in good agreement with the experimental study.*

1   INTRODUCTION

This paper provides an experimental study and finite element analysis of a novel joint system used in steel frames to withstand seismic load at a 1 in 2500 year occurrence. Despite existing research being conducted with regards to the development of robustness of composite connections by [1], [2] and [3], it is still imperative to further investigate the performance characteristics of these new and innovative composite connections when subjected severe earthquake loads. Figure 1 illustrates the connection uses blind bolts through a steel tube filled with concrete. The main compressive loads are therefore transferred in this zone and the longitudinal reinforcement in the slab is thus used to carry the major tension force.
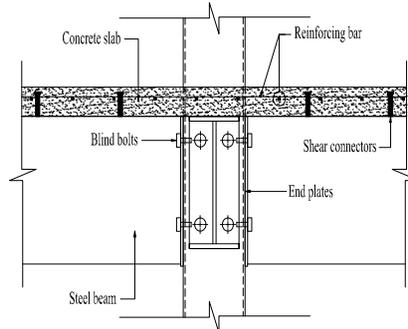
Figure 1: Typical connection detail.

Gibson and McCue [4] stated that the ignorance of earthquakes, tsunamis, cyclones and tornadoes received awareness in engineering design after the destruction caused by the Meckering earthquake in 1968. The authors also discovered that most of the structural failures were attributed to an omission of earthquake loads during these building's early planning stages. According to [5], due to the abovementioned failures, seismic loads using a 1 in 500 year return period is commonly considered by engineers. One should bear in mind that these scales vary according to geographic locations. Recently, [6] clarified that there are multiple benefits that would exist if the 1 in 2500 year return period calculations were to be formalised. They include reduction in human casualties from tragic seismic disaster, increase in insurance confidence, increase in structural life expectancies and minimised economic and commercial impacts if the structures were to be subjected to earthquake. Therefore, rigorous experimental studies are being undertaken in this paper to look at how these proposed composite connections would actually behave in the unlikely event of a high consequence earthquake.

2 EXPERIMENTAL TEST SPECIMENS

The response of composite beam-column end plate connections would be critical to the seismic performance. Therefore, tests were conducted in an attempt to determine the feasibility of using such connection. The main aim of these tests was to study the behaviour of blind bolts. Figure 2 is an illustration of the layout and dimensions of the composite test specimen.
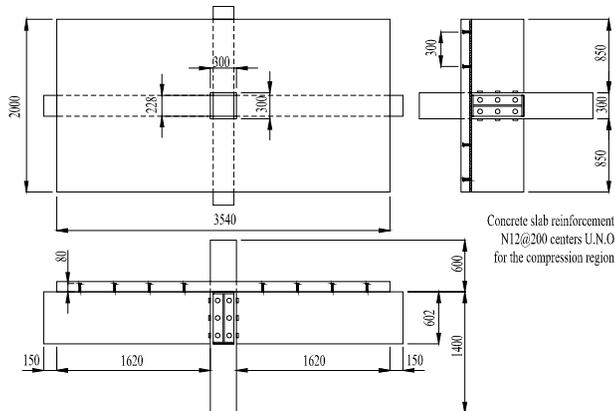
Figure 2: Detail of experiment specimens.

There were two experimental studies undertaken. The first specimen was subjected to monolithic static loading to determine the hogging moment magnitude. An initial load of 513 kN point load was applied to the specimen until the ultimate connection capacity was achieved. The specimen was loaded with a lever arm of 1620 mm. The second specimen was subjected to cyclic loading which is similar to earthquake loading. An initial load of 426 kN point load was applied to the specimen, and then reduced to 200 kN. The process was repeated for 30 cycles.

3 FINITE ELEMENT MODEL

3.1 Material properties

In general, constitutive laws are used to define the stress-strain characteristics of a material. The accuracy of the analysis is dependent on the constitutive laws used to define the mechanical behaviour. In materials such as concrete, structural steel and reinforcing steel, profiled steel sheeting and shear connectors, the constitutive laws are represented by the stress-strain relationships of the materials. In this

paper, the mechanical behaviour at static and seismic loading is considered. For static loading, the material properties are according to [7], and for seismic loading, the concrete and steel material are according to [8].

3.2 Finite element type, mesh and boundary conditions

Three-dimensional solid elements were used to model the push off test specimens in order to achieve an accurate result from the finite element analysis. For both the concrete slab and the structural steel beam, a three-dimensional eight-node element (C3D8R) was used because it is a solid element and it was also used to improve the rate of convergence. A three-dimensional thirty-node quadratic brick element (C3D20R) for blind bolts and shear connectors was used because it is a second order element consisting of 20 nodes and these provide higher accuracy when compared with the first order elements. They capture stress concentrations more effectively and better for modelling geometric features. A four-node doubly curved thin shell element (S4R) was used for the profiled steel sheeting because it is the most appropriate type of element to model thin walled steel structures. The S4R element has six degrees of freedom per node and proved to give accurate solutions and also permits quadratic deformation over four nodal coordinates, membrane action and plain strain behaviour. Finally, a two-node linear three dimensional truss element (T3D2) for steel reinforcing was adopted where the axial direction is released using an option of equation provided in ABAQUS.

Figure 3 illustrates the finite element mesh used to represent a quarter model of the experimental specimen. The aim of this was to reduce the simulation cost and computational time. The generated mesh was designed to give an optimal accuracy where the fine mesh surrounds the blind bolts and shear connectors and a coarse mesh was used elsewhere.
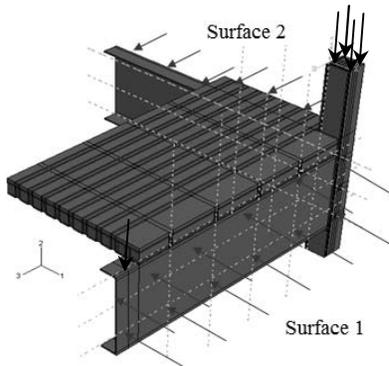
Figure 3: Finite element model layout.

All the nodes designated as Surface 1 are restricted to move in the y-direction while all the nodes in Surface 2, are restricted to move in the x-direction. For the application of load, uniform distributed axial load was applied to the centre of the reinforced concrete tubular column, whilst a concentrated load was applied at 150 mm from the tip of the end of structural steel beam. Both the loads were employed using the modified RIKS method which can be obtained through a series of iterations for each increment for a non-linear structure. The RIKS method was used for the nonlinear analysis to ensure that any unloading was captured.

4 RESULTS AND DISCUSSIONS

4.1 Monolithic static loading

Figure 4 illustrates a peak load of 365 kN was measured. During the experimental procedure, when the load reached 168 kN, there were signs of concrete cracking. The concrete cracked 150 mm from the column face which is shown in figure 5. During the experiment, it was observed that the column surface and the end plate buckled which caused the connection to fail. As the end hydraulic loads progressed, the concrete cracks expanded, encouraging the deformation in the endplates to take place, shown in figure 6. Further load progress eventually caused the headed stud shear connectors to pull out from the structural steel beam and severe concrete cracking, as shown in figure 7. This noticeable deformation in the endplates was a result of the overall stiffness within the connection depleting, and because of the excessive end beam deflection.
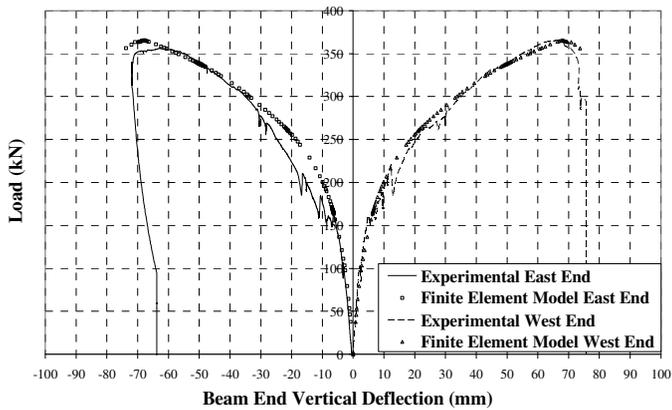
Figure 4: Load versus deflection for static loading.
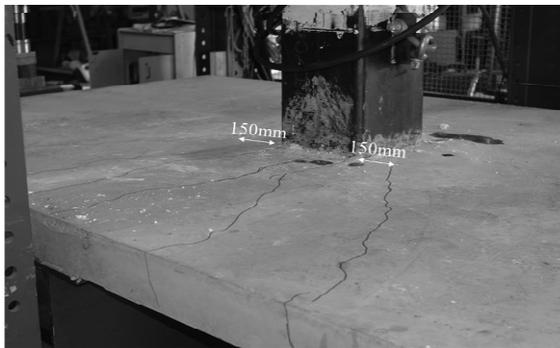
Figure 5: Concrete cracks at 150 mm from column face.

Figure 6: Blind bolts of the endplate displace 3 mm outwards.
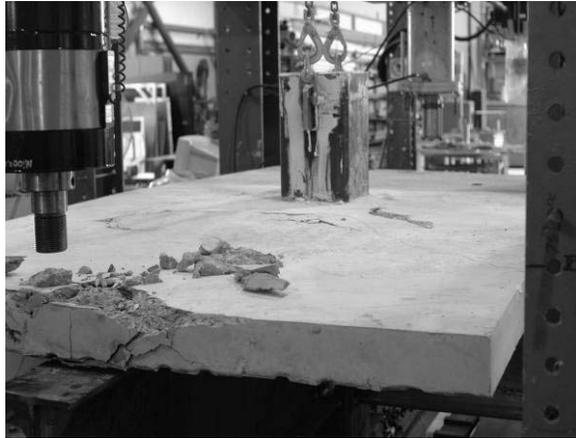
Figure 7: Shear connectors pull out and concrete failure.

Figure 4 proves that the finite element model is in good correlation with the experimental data. Initially, it was observed that the concrete element near the column face reached its cracking stress. This proved that the concrete started to crack near the column surface. With the continuation of the loading, it was observed that the end of concrete element started to crack followed by the headed stud shear connectors reaching their maximum stress, then pulling out from the structural steel element. Finally, the finite element illustrated that the column face and end plate exceed their yield stress and started to buckle which caused connections failure.

4.2 Seismic loading

Figure 8 illustrates that 140 kN peak load upward and 259 kN downward. During the first cycle, the upward load of 130 kN was reached. It was observed that the end of concrete slab started to crack shown in figure 9. When the downward load was applied in the first cycle, the load reached 225 kN. During the

first cycle of downward load, it was observed that the thin wall of the column started to buckle which is shown in figure 10. When the process was repeated, for the upward load, the failure in the headed stud shear connectors was observed in the composite steel-concrete slab, as in figure 11. However, for the downward load, the buckling of the thin column wall became severe which caused the connection failure in the blind bolt area, as shown in figure 12. At the same time, it also can be observed that the profiled sheeting started to buckle and tear, as depicted in figure 12. Figure 8 also demonstrates that the finite element model is in good agreement with the experimental data. The failure showed in finite element is similar to the experimental study.
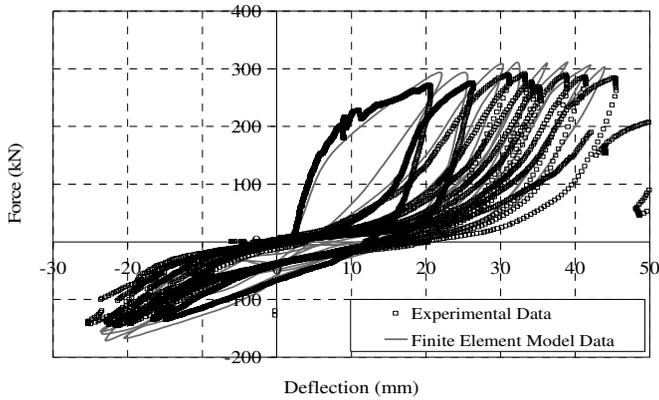
Figure 8: Load versus deflection for seismic loading.
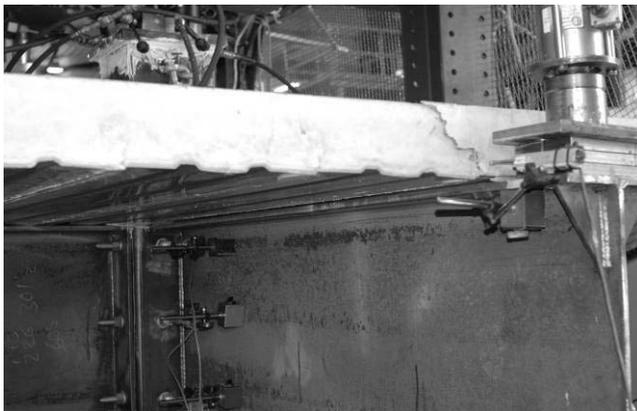
Figure 9: End concrete slab cracks.
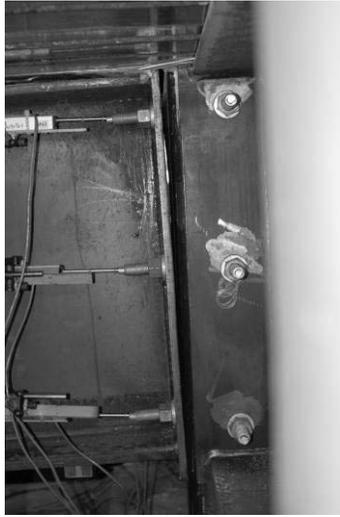
Figure 10: Column face and end plate buckle.
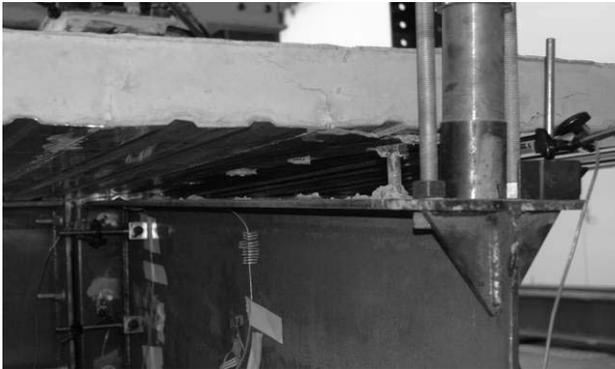
Figure 11: Headed stud shear connector failure.

Figure 12: Severe buckle at column face and end plate.

5   CONCLUSIONS

This paper discussed the behaviour of composite beam-column flush end plate connections subjected to low probability, high-consequences loading. It has been revealed that there is a need to cater for low probability, high-consequence earthquake loads such as those of a 1-in-2500 year return period to prevent further structural failures, catastrophic loss of lives, and economic blunders.

It has been asserted that medium-rise buildings are most likely to have an unenthusiastic behaviour towards an earthquake and that failure is most likely to occur at the connection of the structure. Hence the focus of this paper is the adequacy of standardising increased earthquake return periods within structural design connections. It is this earthquake return period that has been identified as being one of the main factors in governing the magnitude of an earthquake.

Given this fact, this paper has contributed to the broad spectrum of in-depth investigations that evaluate the practicalities of standardising earthquake return periods to 1-in-2500 years with the use of composite connections. The premature failure occurred in the cyclic test could be due to the low cycle fatigue. A reasonable accurate finite element model has been developed to investigate the behaviour of the shear connectors and end plate connections in composite steel-concrete beams. The comparison between the finite element models and experimental results are in good correlation.

ACKNOWLEDGEMENT

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