ELASTIC PLASTIC FE ANALYSES OF SUB MODELS OF CONNECTIONS IN STEEL FRAMED MOMENT RESISTING BUILDINGS UNDER EARTHQUAKE LOADING

F.M. Burdekin and K. Kuntiyawichai
UMIST, Manchester (UK)

ABSTRACT

Finite element analyses have been carried out to investigate the factors that contributed to failure of the welded connections in moment resisting steel framed building as involved in the Northridge earthquake. The particular structure chosen for analysis was a four-storey frame typical of ones in which a significant number of brittle fractures had been reported. Analyses have been carried out under both full dynamic earthquake and equivalent Uniform Building Code (UBC) static loading. Five different geometries of sub model were incorporated into the global model, these being the basic beam to column connection with no strengthening, the case with added continuity plates between the column flanges, the case with the beam flange width reduced (dog bone) and the cases with an additional haunch or additional cover plates. The results confirmed the general increase in severity of conditions towards the centre of the beam flange width. The stress distribution also confirmed that the position of the maximum stress coincided with the location of the fracture initiation. Connections strengthened with a haunch or cover plates were found to show a very good performance and much better than the dog bone and “pre-Northridge” connection. The material tensile properties also affect the crack tip severity at the crack tip. The absolute values of applied CTOD obtained from the FE analyses are entirely consistent with the occurrence of fracture in materials with fracture toughness levels of the order of 0.1 mm to 0.2 mm CTOD.

IIW-Thesaurus keywords: *Buildings; Structural members; Columns; Girders>; Cover plates; Earthquakes; Design; Comparisons; Stress; Mechanical properties; Influencing factors; Welded joints; Finite element analysis; Structural analysis; COD; Practical investigations.*

1 INTRODUCTION

The 1994 Northridge and 1995 Kobe earthquakes caused widespread and unanticipated damage to welded steel moment resisting frame systems. Most of the damage occurred in or near the weld joints of girder bottom flanges and the supporting column flanges. Observation of damage sustained by buildings in the Northridge earthquake showed that brittle fractures had occurred in many cases. These failures have prompted the engineering community to investigate the reason and as part of this to explore alternative connection types. A great deal of research and laboratory testing has been conducted to identify better moment connections for new steel moment connection buildings. From laboratory testing carried out by the SAC phase I project, it has been shown that many different potential details can be used to modify the performance of the girder-column joints such as reinforcement at the connection. Amongst the analytical studies of connection behaviour by a number of researchers have been those of Chi et al [1] Popov [2], and Burdekin and co-workers [3, 4]. Finite element analyses have been carried out previously at UMIST to explore the stressing conditions in a typical beam to column connection from a Northridge building [3, 4]. Most of the analytical results were obtained from elastic analysis of selected connections (i.e. uncracked) from the global steel frame under equivalent static loading. Therefore, it is of interest to explore elastic plastic analysis of cracked connections situated in the complete frame subject to dynamic loading.

In this investigation, the finite element technique is used to study the local behaviour of connections with defects within the complete building frame. The purpose of the FEM analyses was to identify the effects of crack length, connection design, and material properties on the local behaviour of sub model connections located in the full steel frame building. The stress distribution in the region of the column and beam flange connection is also considered.

The sub model analyses consisted of five alternative designs of beam to column connections, each series being defined by the configuration of the connection as follows:

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• Basic connection as used at Northridge with no additional strengthening.
• Connection with continuity plates added between the flanges of the column.
• Connection with beam flange width reduced by a “dog bone” shape away from weld.
• Connection with additional haunch section added underneath the beam section.
• Connection with cover plates added to the beam flanges and welded to the column flange.

Each sub model series was subdivided into three different cases of analysis. Case one consisted of the sub model connection without initial defects; cases two and three consisted of the connection with 2.5 mm and 5 mm height crack like defects in the sub model across the underneath of the width of the bottom flange of the beam against the column face. Each series of sub model connections was attached to the global frame model. A sensitivity study concerning material properties was also investigated with effects of four different stress strain curves examined. The results of dynamic analyses for each series are also compared with pseudo static analyses, pushover analyses, using the Uniform Building Code (UBC)-1994 method [5].

2 STRUCTURE USED FOR GLOBAL FINITE ELEMENT MODEL

Four-storey buildings with welded steel moment resisting frame (WSMF) experienced many connection fractures in one of the levels. A post-earthquake inspection of the buildings revealed extensive connection fracturing at one of the N-S perimeter frames. Therefore a simplified two-dimensional model of an east moment frame in the north-south direction of the structure, i.e. frame D, was modelled in this study. This frame consists of four-storeys and three bays as shown in Figure 1 with all beams and columns having a nominal yield strength of 350 MPa. The columns of the 4-storey frame are considered as fixed at the base.

In order to simulate the behaviour of connection under real earthquake condition, a three-dimensional sub model was modelled at one of the connections as circled in Figure 1. Linear elastic analyses were conducted initially to provide an indication of forces used for design of the building, to indicate the stress concentrations at the connections, and to correlate to the damage pattern under the presumption that little inelastic action may have occurred prior to fracturing in the connections. More general elastic-plastic analyses were then carried out to provide more realistic values of forces, displacements, and ductilities by assuming three different material properties. Within each of these categories, both static analyses, i.e. pushover analyses, and transient dynamic time history analyses were performed on the global model.

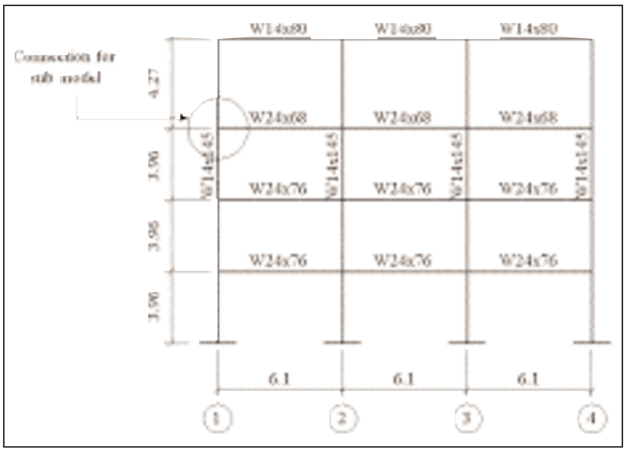


Fig. 1. Four-storey framed building.

The equivalent lateral load was calculated using the UBC-94 recommendations. Since there are two moment frames along the east-west axis of the building, the total base shear was divided by two and distributed vertically on the moment frame. The distribution of lateral forces over the height of the building was also obtained from UBC-94. Table 1 shows the resulting loads at each floor level.

For transient dynamic time history analyses, dynamic loading was applied as base acceleration to the structure using accelerogram record from the Canoga station. The record exhibits acceleration for a duration of approximately 56 seconds. Figure 2 shows the acceleration record from Canoga station and Figure 3 shows the frequency content obtained by Fourier transformation. The mass and dead load of the building were applied in terms of material density.

2.1 Description of sub models

This present study focused on the local behaviour of connections under full earthquake loading conditions. The three-dimensional sub model connections were modelled within the complete two-dimensional 4-storey building frame. Three different cases of defects were considered, i.e. no initial defect, 2.5 and 5 mm height of defect. Five different forms of joint taken from the global model were investigated in this study. The sizes of beams and columns were the same for all type of connections. The dimensions of beams and columns are shown in Figure 4. The finite element models for the five sub models analysed are shown in Figures 5(a) to (e).

2.1.1 Basic connection as used at Northridge with no additional strengthening

This type of connection is the original type of hybrid connection. Shear plates were welded to the column flange

Table 1. Equivalent lateral loads for four-storey building.

Floor level	Equivalent lateral load (kN)
1	277.9
2	555.8
3	833.7
Roof (R)	993.3

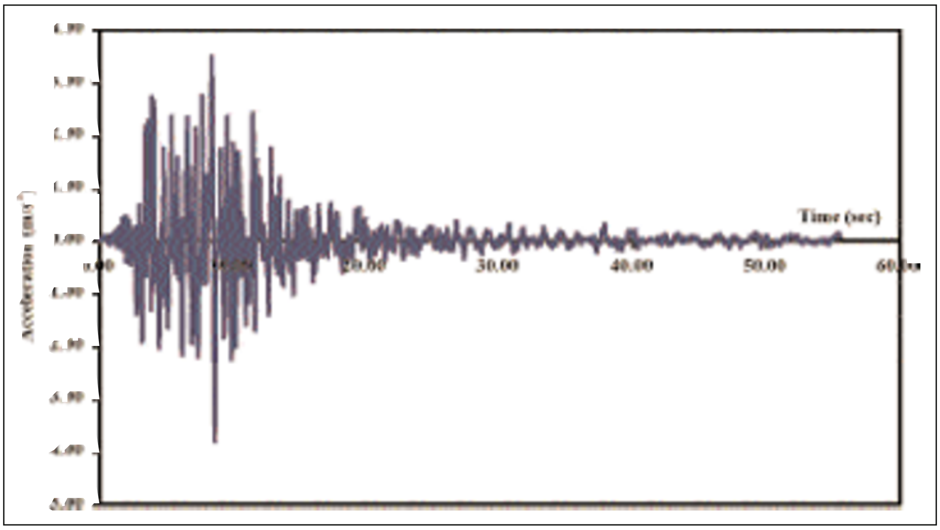


Fig. 2. Earthquake ground acceleration record (Canoga station).

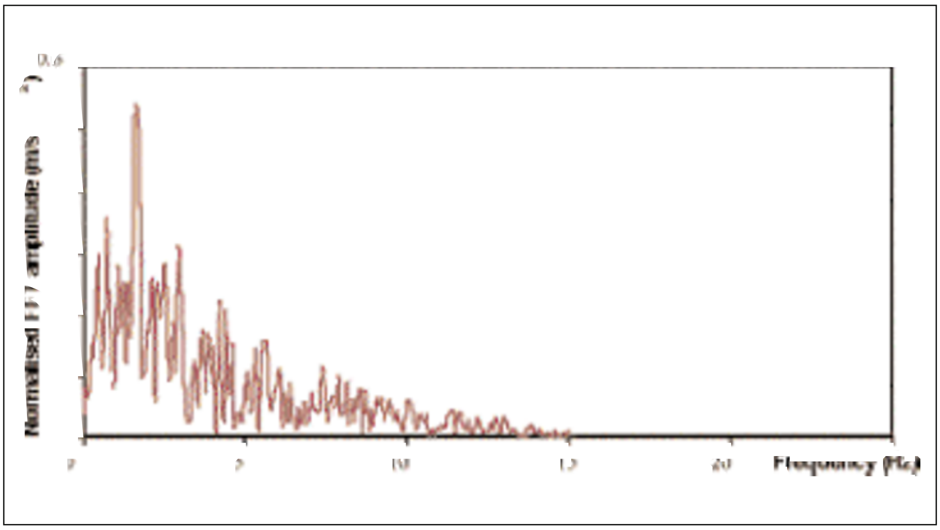


Fig. 3. Frequency spectrum of Canoga ground acceleration record.

in the shop and bolted to the beam web at the construction site. The beam flanges were welded to the column flange using full penetration butt groove welding. Continuity plates were not connected at the column flange. This type of connection is the worst case in which the design could not transmit the full moment capacity of the beam to the column. The analyses results of these connections were taken to model the base case. The

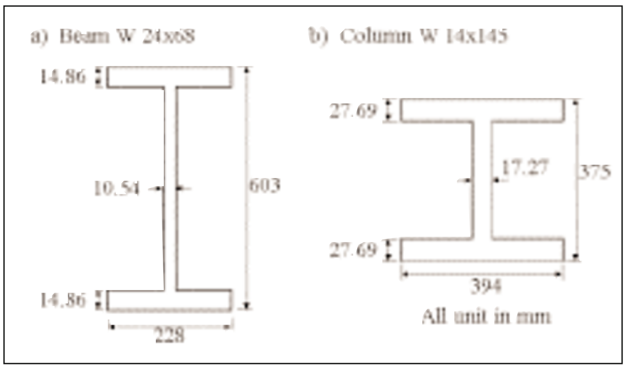


Fig. 4. Dimensions of beams and columns.

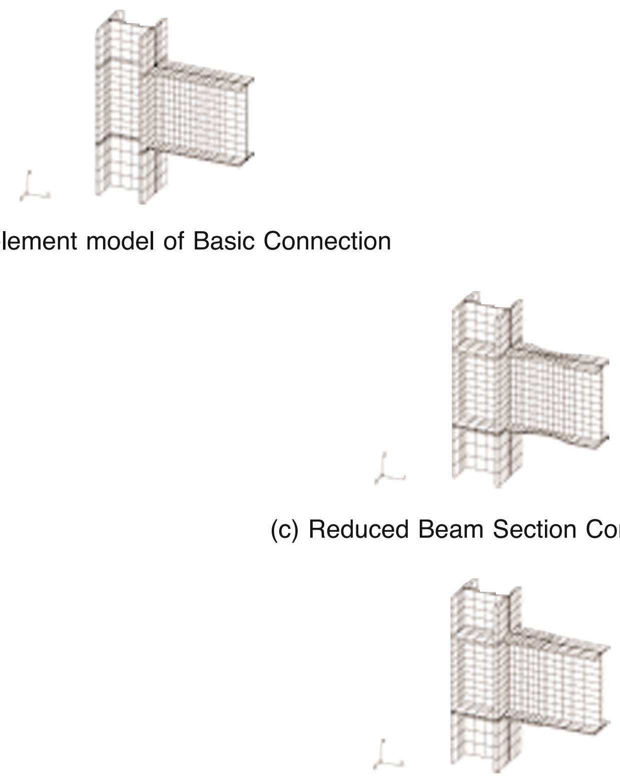
finite element model of the basic connection sub model is shown in Figure 5(a).

2.1.2 Connection with continuity plates added between the flanges of the beam

The connection in this series is the original basic connection with the addition of continuity plate connecting the beam flanges through to the column web and flanges. This type of connection is called the "Basic Connection with Continuity Plates". This connection was widely used in the US before the Northridge earthquake. It was presumed to have sufficient strength to permit either the beam to yield in flexure or the column panel zone to yield in shear. Figure 5(b) shows the finite element model of the original connection with the continuity plates added.

2.1.3 Connection with beam flange reduced by "dog bone" shape away from weld

Since the Northridge earthquake, extensive research and testing has been conducted to identify improved moment connections for new steel moment frame con-



(a) Finite element model of Basic Connection

(b) Continuity Plate Connection

(c) Reduced Beam Section Connection

(d) Bottom Haunch Connection

(e) Cover Plate Connection

Fig. 5. Finite element models of different connections analysed.

struction, as well as for repairing or upgrading of existing steel moment frames. Modifications of the connection design have been introduced to improve the performance of the connection under earthquake conditions. There are several ways of strengthening the connections including adding cover plates or adding haunch section as discussed later.

Another approach for improving the performance of the connection is to weaken the connecting beam thus limiting the maximum flange loads. This can be achieved by shaving off part of the beam flange. This connection is called the "reduced beam section connection (RBS)" or "dog bone connection". The finite element model of the reduced beam section given by SAC99-01 [6] is shown in Figure 5(c). RBS assemblies are intended to promote the formation of plastic hinges within the beam span by developing a segment of the beam with locally reduced section properties and strength.

2.1.4 Connection with additional haunch section added underneath the beam section

Adding haunch sections to the connection is one method that can strengthen the connection at the position of the welds to the column face. The finite element model of a connection with a haunch at the bottom beam flange is shown in Figure 5(d). The purpose of adding the extra haunch to the connection is the same as for the RBS connection, which is to shift the plastic hinge away from the column face.

From the basic configuration of this connection, it can be seen that the haunch creates a prop type support, beneath the beam bottom flange. Therefore this haunch section reduces the effective flexural stresses in the beam at the face of the support, and also reduces the shear, which will be transmitted to the column through the beam. The weld defect is assumed to be at the bottom of the beam flange before adding the haunch section.

2.1.5 Connection with cover plates added to the beam flanges and welded to the column flange

The use of cover plates has also been one of the most common connection reinforcing schemes used since the Northridge earthquake. The purpose of adding cover plates is to provide a connection that is stronger than the beam. The reinforcement is intended to move the position of any plastic hinge away from the face of the column, where premature fractures can occur due to potential weld defects, stress concentrations due to column flange bending or stress concentrations at weld access holes, etc.

Short cover plates are welded to the top and bottom flanges of the beam by using fillet welds adequate to transfer the cover plates forces to the beam flanges. The finite element model of added cover plate connections is shown in Figure 5(e).

2.2 Description of finite element model

The general purpose pre and post processing code PATRAN v9.0 was used to create the input data for the finite element models. The complete models then were analysed by the ABAQUS finite element package with facilities for linear and non-linear analyses. The two-dimensional beam elements with three nodes, i.e. B22 available in ABAQUS, were used for modelling all of the two-dimensional models. These elements have three degrees of freedom per node. The beams were rigidly attached to the columns.

For the sub models of the connections, three-dimensional solid elements incorporating initial cracks were used. Although the sub model with 3D solid elements within the full 2D model proved to be computationally expensive, it provided valuable understanding of the local behaviour of connections situated in the complete building frame. The elements used for the modelling of sub models were first-order (linear) interpolation three-dimensional linear brick elements, i.e. C3D8 available in ABAQUS. These elements have three degrees of freedom per node. With an adequately fine mesh, these elements are capable of providing accurate solutions even in complex structures. The cracks were modelled with a crack tip with initial root radius, which was assumed to be 0.5 mm in all cases in order to prevent the overlap between the crack faces. Figure 6 shows the typical crack tip mesh used. Table 2 shows the number of nodes and elements in each sub model.

Chi *et al* [7] concluded that the presence of the backing bar itself does not increase the fracture toughness demand or crack tip driving force beyond that, which would be caused by a small edge root defect. Rather, for any edge defect that extends into the weld root, the stress intensity factor is unaffected by the backing bar thickness. Therefore the backing bar has not been modelled separately in the finite element analysis of sub models. The effective crack depth is the height of the edge root defect as shown in Figure 7.

Finally, the sub models were placed into the global model by using distributing coupling elements (DCOUP3D) introduced with ABAQUS/Standard in the area of connection between the solid elements of the sub model and the beam elements of the global model. This special option offers general capabilities for transmitting loads and associating motions between one node and a collection of "coupling" nodes. The option associates the coupling nodes with a single node in a "rigid body" sense; translations and rotations of the node (the distributing coupling element node) are associated with the coupling node group as a whole.

Table 2. Number of nodes and elements used in sub models.

Type of connection	No. of nodes	No. of elements
No initial crack		
Basic Connection	2349	1332
Continuity Plate Connection	2421	1380
Reduced Beam Section Connection	2421	1380
Bottom Haunch Connection	2695	1546
Cover Plates Connection	2439	1436
Initial crack length = 2.5 mm.		
Basic Connection	5423	3836
Continuity Plate Connection	5567	3980
Reduced Beam Section Connection	5567	3980
Bottom Haunch Connection	5882	4162
Cover Plates Connection	5558	4012
Initial crack length = 5 mm.		
Basic Connection	5423	3836
Continuity Plate Connection	5567	3980
Reduced Beam Section Connection	5567	3980
Bottom Haunch Connection	5882	4162
Cover Plates Connection	5558	4012

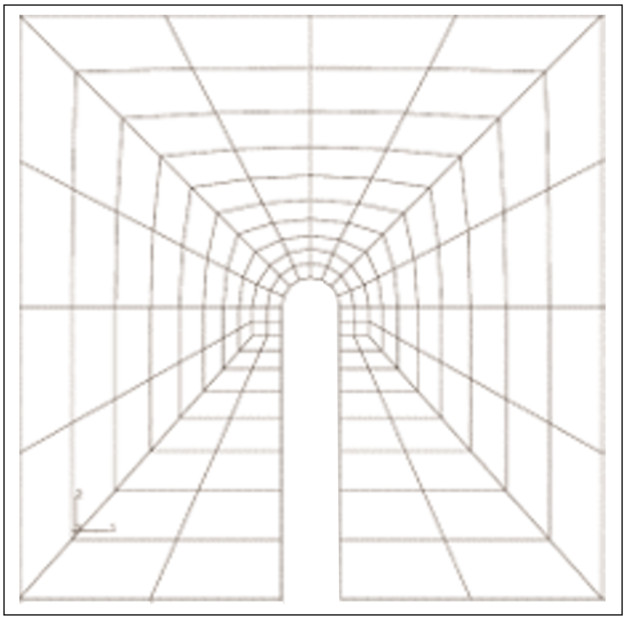


Fig. 6. Finite element mesh in the crack tip region.

2.3 Material properties

Both elastic and elastic-plastic material properties were considered in this study. The elastic-plastic material models for the analyses included bi-linear (mises) and Ramberg-Osgood models in which incremental theory of

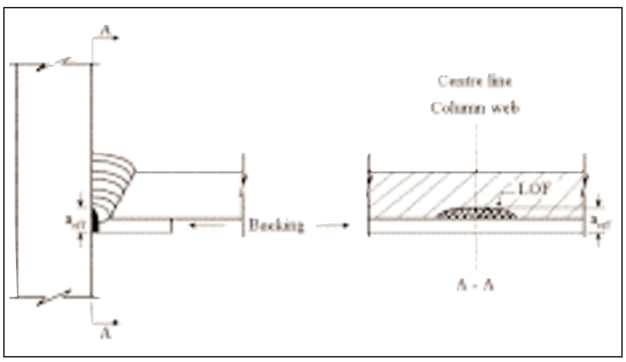


Fig. 7. Simple edge crack beneath the beam bottom flange.

Table 3. Material properties.

Material No.	Material	Young's Modulus (Mpa)	Yield stress (Mpa)	Work hardening Slope (Mpa)
M1	Elastic	210×10^3	–	–
M2	Elastic-plastic	210×10^3	350	21×10^3
M3	Elastic-plastic	210×10^3	350	210
M4	Elastic-plastic	210×10^3	350	σ_u 550 Mpa at 15 % strain

plasticity with (constant) isotropic hardening was employed with the von Mises yield surface expressed in terms of Cauchy (true) stress. The conventional engineering strain, σ_E and engineering (nominal) stress, ε_E values were converted to true strain and true stress for input to the FE analysis. Table 3 shows the four material properties used.

2.4 Applied loading

Two types of loading were applied, namely equivalent lateral load (pseudo static analyses) and dynamic time history analyses. The equivalent lateral load was based on the UBC-94 method [5]. The base shear value depends on local seismicity, subsurface profile, structural system type, dynamic properties of the structure, and the structure's importance. The results for the lateral load acting on the case study frame are shown in Table 1.

In the dynamic time history analyses, dynamic loading was applied as a base acceleration to the structure using the accelerogram record from the Canoga station as shown in Figs 1 and 2.

3 FINITE ELEMENT ANALYSES

The general direct time integration method, called the Hilber-Hugues-Taylor operator, provided in ABAQUS/standard was used in the dynamic response analyses. The set of simultaneous non-linear dynamic equilibrium equations is solved at each time increment. An automatic increment scheme in ABAQUS was also used in order to control the accuracy of the solution. Artificial damping, called the ALPHA parameter was also introduced in the model. A value of $\alpha = -0.05$ was used because this introduces just enough artificial damping in the system to allow the automatic time stepping procedure to work smoothly.

For the dynamic analyses, structural damping was assumed to be 2% of critical, and proportional to the structure's mass and stiffness.

4 ANALYSES RESULTS

4.1 General review

Because of uncertainties about J values under dynamic loading with unloading effects occurring, the applied crack tip severity was measured by the crack tip opening displacement (CTOD) concept. The relative displacements of the nodes along the crack faces were extracted and used to derive the crack tip opening displacement. The maximum values of these during the Northridge earthquake loading spectrum (Canoga station) were then plotted against the position across the width of the beam flange.

4.2 Applied CTOD distributions from dynamic sub model analyses

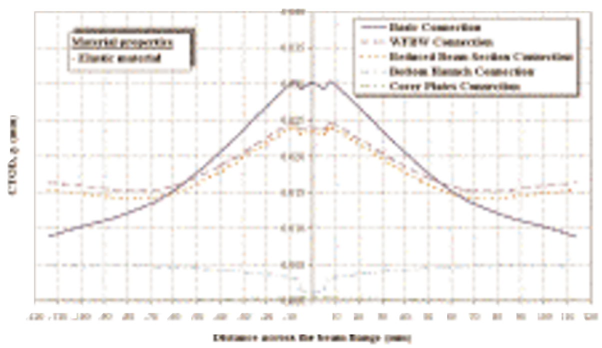
The results of the FE analyses for full earthquake dynamic loading are shown in Figure 8 for the 2.5 mm height flaw and Figure 9 for the 5 mm height flaw. The results for each of the four material types M1 to M4 are shown in the parts (a) to (d) respectively of each figure. Each part figure shows the results for the maximum applied CTOD distribution across the width of the flange for each of the five sub models at any time during the whole occurrence of the earthquake time history loading.

It can be seen that for both flaw sizes and each material case, the highest applied CTOD values occur for the basic model case with no additional strengthening. Furthermore, these highest values occur at the centre of the width of the beam flange with lower values occurring out towards the edges of the flange. This is consistent with the findings for general stress distributions across the flange width from previous investigations at UMIST in references 3 and 4.

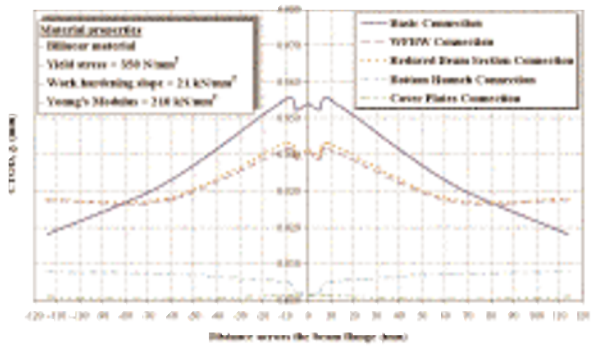
The inclusion of continuity plates produces some reduction in the peak values of CTOD for all material cases, as does the reduced beam width (dog bone) model. However much the greatest reduction in applied CTOD values is obtained by the use of either a haunch section or the addition of cover plates. In these cases the applied CTOD values were reduced to very low values for all material types.

The effect of the two different flaw size assumptions can be seen by comparing the corresponding part figure for the same material in Figures 8 and 9. It can be seen, as expected, that the peak applied CTOD values for the 5 mm flaw are greater than those for the 2.5 mm flaw.

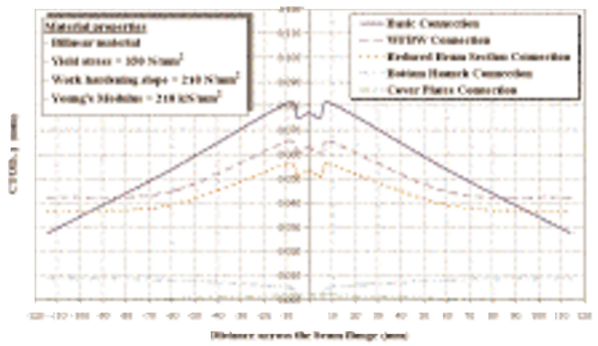
The effect of the different material assumptions can be seen by comparison of parts (a) to (d) of each of Figures 8 and 9. Significant differences occur as a result of the different material assumptions. The peak CTOD values for the basic model with the 2.5 mm high flaw



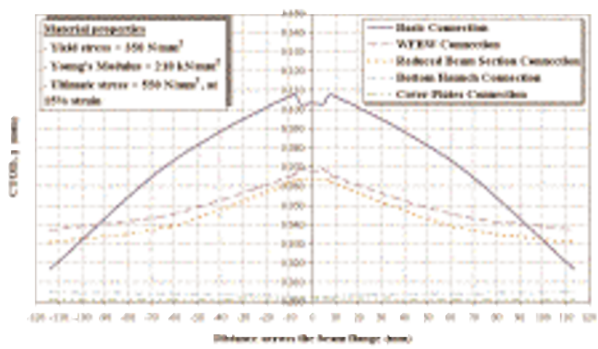
a) Material model M1



b) Material model M2

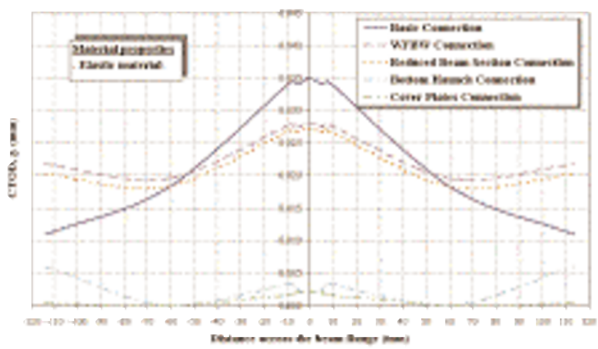


c) Material model M3

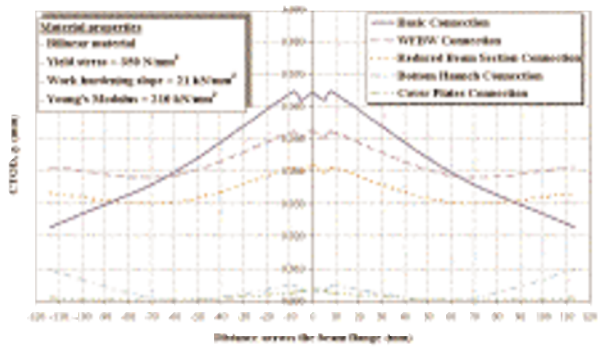


d) Material model M4

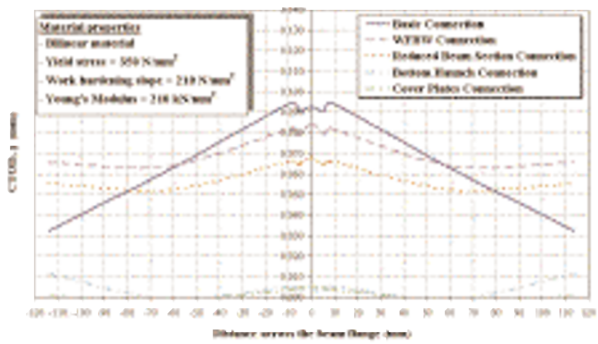
Fig. 8. Variation of CTOD across the bottom flange for all cases (crack height = 2.5 mm, dynamic).



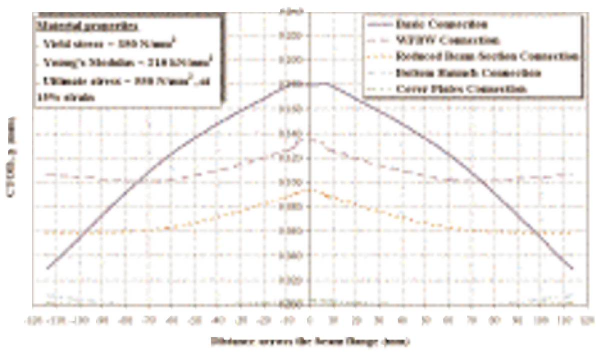
a) Material model M1



b) Material model M2



c) Material model M3



d) Material model M4

Fig. 9. Variation of CTOD across the bottom flange for all cases (crack height = 5 mm, dynamic).

vary from 0.03 mm for the elastic case (M1) to 0.11 mm for the Grade 355 steel (M4), whilst with the 5 mm height flaw the corresponding values are 0.035 mm for M1 and 0.18 mm for M4.

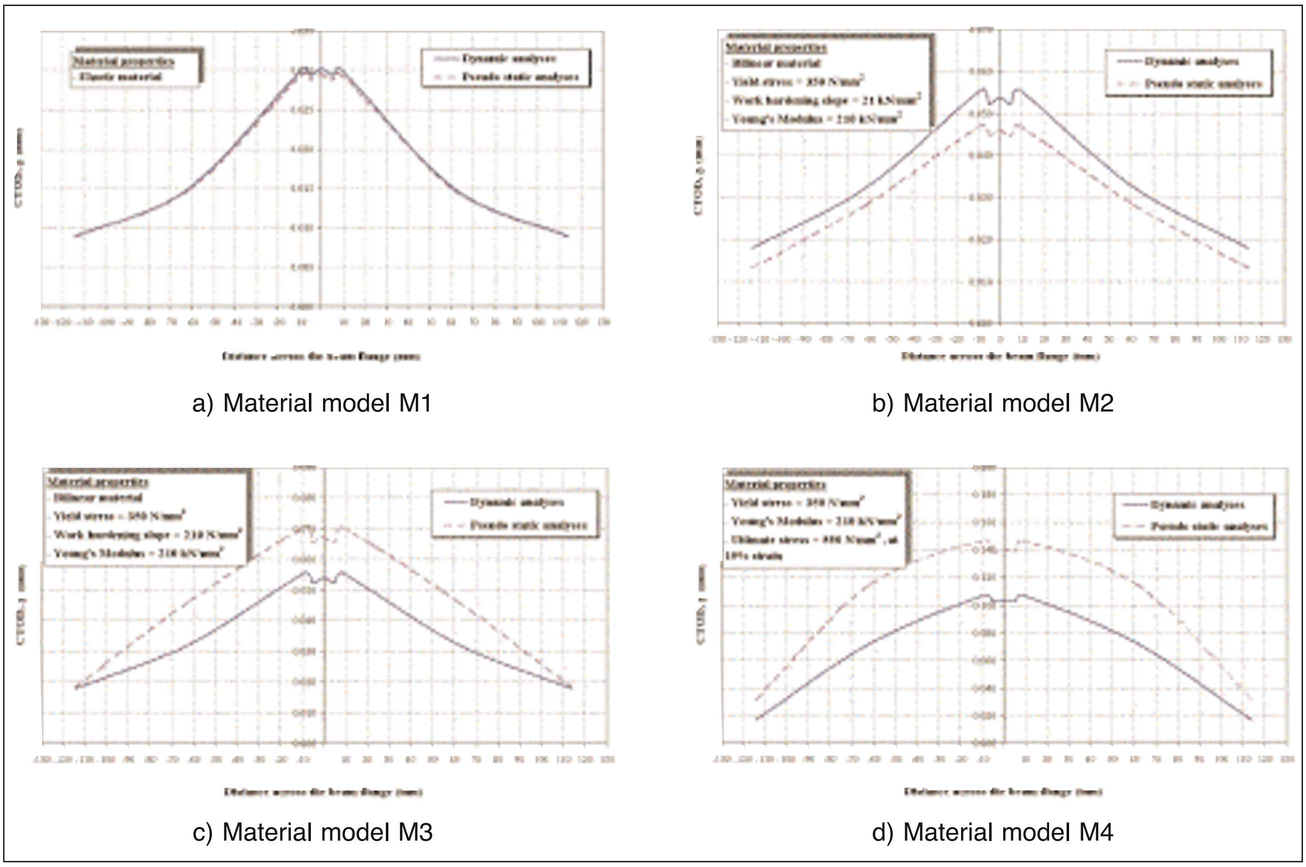
4.3 Applied CTOD distributions from UBC equivalent static loading sub model analyses

Each sub model connection was also analysed by using pseudo static analyses. The global model was applied by the equivalent lateral loads. It was noted that the mode shape of the deformed structure under the full dynamic earthquake loading did not necessarily correspond to that for the UBC equivalent static loading. The results of the FE analyses for applied CTOD are shown in Figures 10 and 11 for 2.5 mm and 5 mm height flaws respectively in the basic sub model. It can be seen that the static analyses produce the same type of distribution of applied CTOD across the width of the beam flange as the full dynamic loading. For the elastic material case (M1), the static and dynamic loads give good agreement. As more plasticity is allowed in the material cases however, differences occur between the dynamic and static results. It can be seen from Figures 10(d) and 11(d) for the Grade 355 steel (M4) that the UBC static loading gives increased estimates of the peak applied CTOD at 0.15 mm for the 2.5 mm height flaw and 0.22 mm for the 5 mm height flaw. For this particular material case it would be conservative to use the UBC loading in an analysis as it would over-estimate the applied CTOD compared to the full dynamic load case.

5 CONCLUSIONS

Finite element analyses have been carried out to investigate the factors that contributed to failure of the welded connections in moment resisting steel framed building as involved in the Northridge earthquake. The particular structure chosen for analysis was a four-storey frame typical of ones in which a significant number of brittle fractures had been reported. Analyses have been carried out under both full dynamic earthquake and equivalent UBC static loading. Five different geometries of sub model were incorporated into the global model, these being the basic beam to column connection with no strengthening, the case with added continuity plates between the column flanges, the case with the beam flange width reduced (dog bone) and the cases with an additional haunch or additional cover plates.

The results confirmed the general increase in severity of conditions towards the centre of the beam flange width. The stress distribution also confirmed that the position of the maximum stress coincided with the location of the fracture initiation. Connections strengthened with a haunch or cover plates were found to show a very good performance and much better than the dog bone and "pre-Northridge" connection. The material ten-



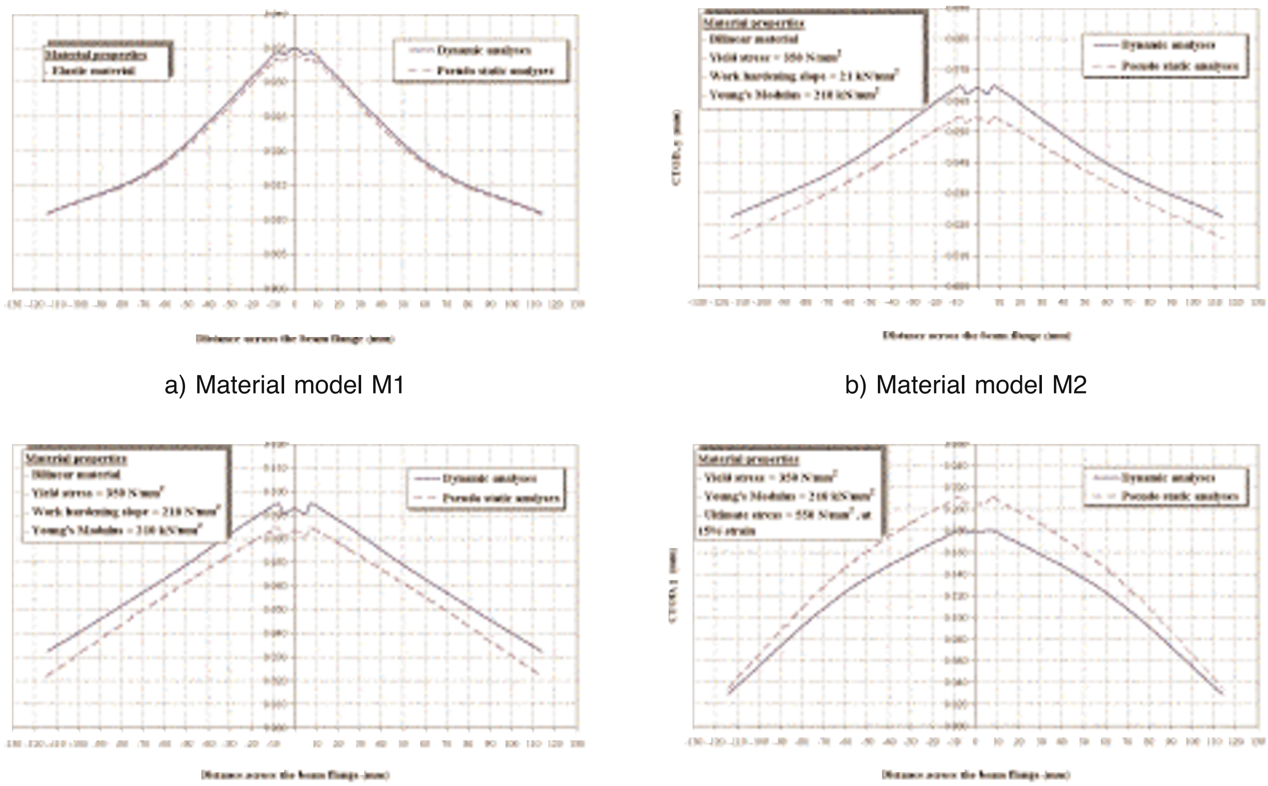
a) Material model M1

b) Material model M2

c) Material model M3

d) Material model M4

Fig. 10. Comparison of CTOD results for 2.5 mm crack height for full earthquake dynamic and UBC static loading (basic model).



a) Material model M1

b) Material model M2

c) Material model M3

d) Material model M4

Fig. 11. Comparison of CTOD results for 5 mm crack height for full earthquake dynamic and UBC static loading (basic model).

sile properties also affect the crack tip severity at the crack tip.

The absolute values of applied CTOD obtained from the FE analyses are entirely consistent with the occurrence of fracture in materials with fracture toughness levels of the order of 0.1 mm to 0.2 mm CTOD.

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