# DETAILED FEM ANALYSIS OF TWO DIFFERENT SPLICE STEEL CONNECTIONS

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OĞUZ YILMAZ

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**DETAILED FEM ANALYSIS OF TWO DIFFERENT SPLICE STEEL CONNECTIONS**

submitted by OĞUZ YILMAZ in partial fulfillment of the requirements for the degree of **Master of Science in Civil Engineering Department, Middle East Technical University** by,

Prof. Dr. Canan Özgen ____________________
Dean, Graduate School of **Natural and Applied Sciences**

Prof. Dr. Güney Özcebe ____________________
Head of Department, **Civil Engineering**

Asst. Prof. Dr. Alp Caner ____________________
Supervisor, **Civil Engineering Dept., METU**

Assoc. Prof. Dr. Cem Topkaya ____________________
Co-Supervisor, **Civil Engineering Dept., METU**

**Examining Committee Members:**

Asst. Prof. Dr. Özgür Kurç ____________________
**Civil Engineering Dept., METU**

Asst. Prof. Dr. Alp Caner ____________________
**Civil Engineering Dept., METU**

Assoc. Prof. Dr. Cem Topkaya ____________________
**Civil Engineering Dept., METU**

Inst. Dr. Afşin Sarıtaş ____________________
**Civil Engineering Dept., METU**

Syed Ateeq Ahmad ____________________
**General Manager, PROYA**

**Date :** September 05, 2008

**I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.**

Name, Last name: Oğuz YILMAZ

Signature:

# ABSTRACT

## DETAILED FEM ANALYSIS OF TWO DIFFERENT SPLICE STEEL CONNECTIONS

YILMAZ, Oğuz
M.Sc., Department of Civil Engineering
Supervisor: Asst. Prof. Dr. Alp CANER

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Beam splices are typically located at moment contraflexure points (where M=0). Most design specifications require these splices to develop a strength either to meet design forces or a minimum value set by specifications. The design forces are typically determined through elastic analysis, which does not include flexibility of splice connections. In this research, two types of splice connections, an extended end plate splice connection and a flange and web plate bolted splice connection, were tested and analyzed to investigate the effect of the partial strength splice connections on structural response. The splices were designed to resist 40% and 34% of connecting section capacities using current steel design codes, respectively. It has been observed from the experiments and FEM analysis results that splice connections designed under capacities of connecting steel members can result in changes in design moment diagrams obtained from analyses without splice connection simulation and can also significantly decrease the rigidity of the structure endangering serviceability. The differences in design moment diagrams can go up to 50 % of elastic analysis without connection flexibility. The vertical displacements can increase to 155% of values obtained from elastic analysis with no splice

connection simulation. Therefore, connection flexibility becomes very important to define in analysis.

Keywords: Beam splice, End plate splice, Flange and web plate bolted splice, Connection flexibility, Finite Element Modeling

# ÖZ

## İKİ FARKLI BAĞLANTILI ÇELİK BİRLEŞİMİNİN DETAYLI SONLU ELEMANLAR ANALİZİ

YILMAZ, Oğuz
Yüksek Lisans, İnşaat Mühendisliği Bölümü
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Kiriş bağlantıları tipik olarak moment sıfır noktalarına yerleştirilirler (M=0). Tasarım şartnamelerinin çoğu bu bağlantıların, ya tasarım kuvvetlerini ya da şartnamelerin belirlediği minimum bir değeri, karşılayacak dayanımı sağlamalarını zorunlu kılmaktadır. Tasarım kuvvetleri tipik olarak bağlantı birleşimlerinin esneklikleri düşünülmeden elastik analiz yöntemiyle elde edilirler. Bu araştırmada, genişletilmiş uç plakalı ve başlık ve gövde plakalı civatalı olmak uzere iki bağlantı tipi, kısmi dayanımlı bağlantılı birleşimlerin yapı davranışı üzerindeki etkisini araştırmak amacıyla test edilmiş ve analiz edilmiştir. Bu bağlantılar mevcut tasarım şartnameleri kullanılarak, bağlanan kesit kapasitelerinin 40% ve 34%`üne dayanabilmeleri için tasarlanmışlardır. Deneylerden ve sonlu elemanlar analiz sonuçlarından, bağlanan kesit eleman kapasitesinden daha zayıf tasarlanan bağlantılı birleşimlerin, bağlantı birleşiminin simule edilmediği analizlerden elde edilen tasarım moment şemalarının değişmesine yol açabildiği ve yapının rijiditesini düşürerek kullanılabilirliğini tehlikeye atabildiği gözlenmiştir. Tasarım moment şemalarındaki farklar, bağlantı esnekliğinin düşünülmediği elastik analiz sonuçlarının 50%`sine kadar çıkabilmektedir. Düşey deplasmanlar, bağlantı birleşiminin simule edilmediği elastik analiz sonuçlarının 155%`ine

çıkabilmektedir. Bu nedenle birleşim esnekliğinin analiz esnasında tanımlanması çok önemli olmaktadır.

Anahtar Kelimeler: Kiriş bağlantısı, Uç plakalı bağlantı, Başlık ve gövde plakalı civatalı bağlantı, Birleşim esnekliği, Sonlu elemanlar modellemesi

To my family

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**TABLE OF CONTENTS**

# CHAPTER 1

# INTRODUCTION

## 1.1 Beam Splices

Hot rolled beams and plate girders can have splices along their length due to construction or design requirements. These requirements can be referenced as: 1) beam or girder length can be less than span length, 2) shipping limitations, 3) camber and 4) design cross-section change [1]. Splices typically transfer shear and moment to the connecting members. These load effects are obtained from structural analysis, which are usually less than load carrying capacity of the spliced members since splices are placed at or close to moment contraflexure points of span. In any case these splices are desired to satisfy minimum requirements of a design code. Other than AISC-LRFD (2005) [2], Eurocode 3 [3] and AASHTO (1996) [4], some design specifications are claimed to require splices to transfer full strength of the connecting members [5]. In this research only major design codes such as AISC-LRFD, Eurocode 3, and AASHTO are evaluated. If a splice is designed for the load capacity of the member spliced, no special precautions may be necessary, since full continuity is maintained.

Some designers prefer to use only shear splices at points of moment contraflexure where moment is negligible and these shear splices can act as a real hinge at a point of zero moment. There are two main reasons not to use shear splices as follows: (1) the point of contraflexure under service load is not at the same location that it develops under factored loads and live load locations are unknown; (2) moments determined from continuous spans are invalid if real hinges, shear splices, are inserted along the continuous spans [1]. Hart and Milek demonstrated that in the presence of real hinges at splices, a given beam may perform as discontinuous when acted by a system of loads different from the system upon which the analysis was based [6].

When, rarely, the beam splice is placed at a point of maximum moment, splices shall have a rotation capacity that is consistent with the global analysis of the overall structure. When, as is usually the case the splice is located at a low moment zone, rotation capacity of splices is not to be consistent with the spliced members, which can soften the structural response.

AISC-LRFD requires groove-welded splices to “develop the nominal strength of the smaller spliced section”. ”Other types of splices in cross sections of plate girders and beams shall develop the strength required by the forces at the point of the splice.”

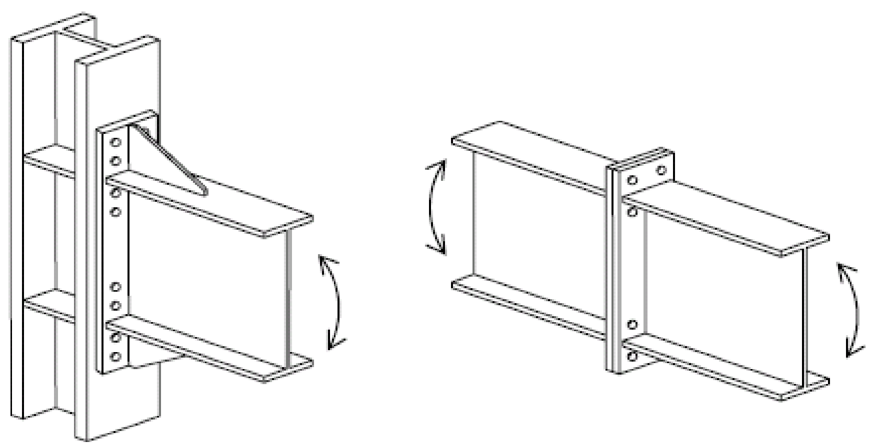
Eurocode 3 requires the internal forces at the splice location to be taken not less than a moment equal to 25% of the moment capacity of the weaker section about both axes and a shear force equal to 2.5% of the normal force capacity of the weaker section in the direction of both axes.

AASHTO requires splice capacity to be maximum of the following values: 1) average of the required design strength at the point of splice and the design strength of the member at the same point or 2) 75% of the design strength of the member.

Couple of splice types is available. Most common splice types are 1) end plate splice connections and 2) flange and web plate splice connections.

### 1.1.1 End-Plate Splices

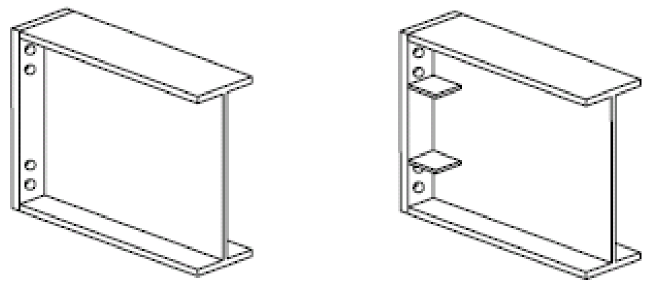
End-plate connections are typically used to connect a beam to a column or to splice two beams together. Moment end-plate connections can be made fully restrained, FR, or partially restrained, PR, based on type, configuration and end-plate stiffness [7]. The common uses of end-plate moment connections are illustrated in Figure 1.1.



a) Beam-to-Column Connection b) Beam Splice Connection

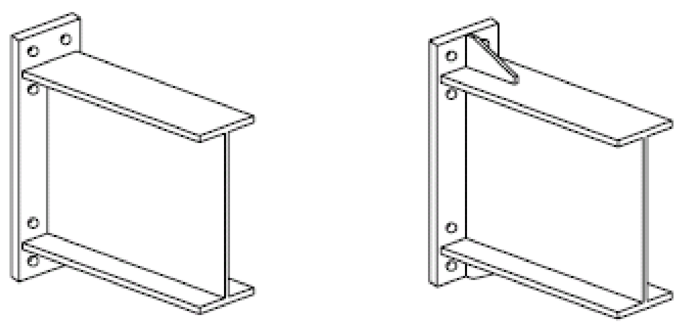
Figure 1.1: Typical uses for end-plate moment connections [7]

End-plate connections can be made either flush, as shown in Figure 1.2 or extended, as shown in Figure 1.3.



a) Unstiffened b) Stiffened

Figure 1.2: Examples of flush end-plate connections [7]



a) Unstiffened b) Stiffened

Figure 1.3: Examples of extended end-plate connections [7]

A standard column-tree system in which flush end-plate splice connections are used is shown in Figure 1.4. This connection configuration is commonly utilized in Turkish practice.

Figure 1.4: Flush end-plate splice connections used in a building frame (Turkish practice) (By courtesy of Caner)

Moment end-plate connections are further described by the number of bolts at the tension flange and the configuration of the bolt rows. Most common end plate connections are four bolt unstiffened and eight bolt stiffened connections.

Because moment end-plate connections do not require time consuming and costly field welds, they are easy to erect and comparatively more economical than other moment connections. Also field welds usually have low quality compared to a similar shop weld. However, these connections require precise beam length and bolt hole location tolerances. This problem may be reduced with the increased use of computer controlled fabrication equipment.

A design and analysis guide for flush and extended end-plate beam-column moment connections is developed by Murray and Shoemaker [8]. The guide includes provisions for the design of four different configurations of flush and five different configurations of extended end-plate connections. These provisions only cover connections subject to gravity, wind and low-seismic forces and a unified design procedure is employed for investigated connections. Design method is developed utilizing yield line analysis in determining end-plate thickness and utilizing the modified Kennedy method in determining bolt forces. Such a method may also be used for design of beam splices in absence of any other code requirement with some precautions.

A second guide for seismic design is developed by Murray and Sumner [9] for extended end-plate moment connections. The guide includes provisions for the design of three different extended end-plate connection configurations. Similar to Murray and Shoemaker guide, a unified design procedure is employed using similar assumptions.

Bahaari and Sherbourne (1994) [10] used ANSYS software to analyze 3-D finite element models to assess structural response of four bolt unstiffened end-plate moment connections instead of 2-D simple models. The new models utilized plate, brick, and truss elements with non-linear material properties. They were able to verify the results of investigated experiments and recommended that the 3-D models are a better option compared to 2-D models to generate analytical formulations to predict the behavior and strength of the connection components.

Choi and Chung (1996) [11] stated that membrane, plate, shell, and/or partially solid elements cannot detect the locally three-dimensional behavior in end plate connections. In their study they presented a refined finite element model using three-dimensional nonconforming solid elements with variable nodes. They took the effect of bolt pretensioning and the shapes of the bolt shank, head and nut into consideration during modeling. A simple but efficient contact algorithm with gap elements was used to employ the interaction between the end plate and the column flange. They verified the applicability of their approach through comparison of the numerical results with test results.

Bursi and Jaspart (1998) [12] provided a summary of developments for estimating the moment-rotation behavior of bolted end plate moment resisting connections. Inclusion of friction in the model had limited effect on the structural behavior of the connection compared to a model without friction simulation. Details of finite element analysis modeling of end-plate connections are presented in their study.

### 1.1.2 Flange and Web Plate Splices

Flange and web plate splices utilize flange plates and a web connection. Flange plates and web connection may be bolted or welded. A typical example of a bolted splice connection is shown in Figure 1.5.

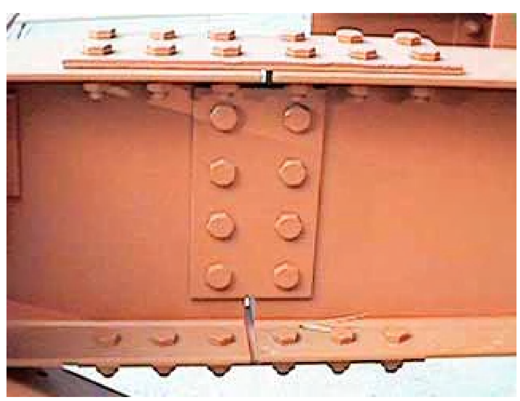


Figure 1.5: A flange and web plate bolted splice connection [13]

In this type of splice connection shear is primarily transferred thorough beam web connection and moment through coupling of axial forces at the top and bottom flange connections. This type of connections is commonly practiced in United States.

Firas I.Sheikh-Ibrahim and Karl H.Frank (1998) [14] conducted an extensive testing program for six full-scale symmetric bolted beam splices. Based on the results of these tests and on the tests available in the literature, following conclusions are presented; (1) web splice can be conservatively assumed to carry the total shear even if flange splices, if not fully yielded, carry a portion of the total shear. (2) the AISC approach is determined to be satisfactory except when the flanges alone are not able to resist the total moment, the AISC approach needs to be modified. In this exceptional case, web splice can be designed for the shear, moment due to eccentricity of the shear, and moment not resisted by the flanges.

## 1.2 Objectives and Scope

Beam splices are typically located in regions where internal load effects are low and they are designed for corresponding moment and shear values determined from analysis occurring at the location of the splice or some specification prescribed minimum strength. These splice connections are most of the time partial strength, partially restrained connections, but in structural analysis, the reduced stiffness at splice is typically ignored.

Two different splice connections are investigated in this research. An experimental program is conducted and finite element models of these connections are developed to investigate effect of splices providing partial strength on redistribution of internal forces.

The details of the experimental program along with the results are presented in Chapter 2. The finite element models of the connections, analysis results and comparison of the numerical analysis results with the experimental results are discussed in Chapter 3. A simple analysis method is presented in Chapter 4 and linear elastic analyses of the beams tested in the experimental program are carried out under their respective design loads once with modeling the beams as continuous and then with incorporating the flexibility of the splices either through the finite element model of the respective splice connection developed in Chapter 3 or through the proposed simple analysis method of defining a moment release at the splice location. The results of the study are discussed and conclusions are drawn in Chapter 5. Appendix A contains the design steps followed along with the design calculations for the end-plate bolted splice and the flange and web plate bolted splice as per AISC procedure.

# CHAPTER 2

# EXPERIMENTAL INVESTIGATION

## 2.1 Overview

The experimental program as a part of this research involved five loading tests. First test was conducted on an unspliced beam. Remaining four tests were on beams with two types of splice connections providing partial strength. Type 1: extended end-plate splice (from hereon called EEPS) and Type 2: flange & web plate bolted splice (from hereon called FWPBS) were selected for the experiments. Connections were tested under negative and positive moment regions of beams subjected to monotonic loading applied at mid span. These tests were conducted to investigate behavior of two types of partial strength splice connections, to determine redistribution of forces on structural system, and to provide experimental data to compare to the results of the finite element analysis which is conducted as a part of this research.

## 2.2 Test Specimens

For each test, two IPN140 beams of St 37 steel were spliced either by EEPS or FWPBS at either 20 cm from interior support or 20 cm from mid span except the first test having no splice. Tabulated beam geometric properties are listed in Table 2.1. All connections were designed weaker than the connecting beams as partial strength connections based on AISC design steps in such a way that the bolts were always the weakest link in the system. (EEPS connection was designed to have a nominal moment capacity of 40% of the nominal plastic moment capacity of the IPN140 member it connected while FWPBS was designed for a capacity of 34%, hence both splice types were designed as partial strength connections) Extended end-plate connection was designed with a thick plate with no consideration for the prying forces.

Table 2.1: Beam Properties

| | |
|---|---|
| Section | IPN140 |
| Depth, h | 140 mm |
| Flange Width, $b_f$ | 66 mm |
| Flange Thickness, $t_f$ | 8.6 mm |
| Web Thickness, $t_w$ | 5.7 mm |
| Moment of Inertia, $I_y$ | 573 x $10^4$ $mm^4$ |
| Elastic Section Modulus | 81.9 x $10^3$ $mm^3$ |
| Plastic Section Modulus | 95.4 x $10^3$ $mm^3$ |

The steel used for the end plates in EEPS and for the flange and web plates in FWPBS was St 37 with a minimum yield strength of 235 MPa. In all spliced beam tests, M6 8.8 bolts were used with washers with a minimum ultimate tensile strength of 800 MPa and tensile yield strength of 640 MPa.

The connection bolts were made snug-tight. End plate was welded to beam using complete joint penetration groove weld with Gas Metal Arc Welding process (GMAW).

Tensile coupon tests were conducted on the beam material used in the testing program. Standard tensile coupon specimens were prepared and tested. The yield strength, ultimate strength, and total elongation were determined.

Also bolt direct tension tests were conducted to find their ultimate strength. The material tests revealed that the beam specimens had a yield strength of 325 MPa, ultimate strength of 485 MPa, and bolts had an ultimate tensile strength of 830 MPa and ultimate shear strength of 450 MPa.

Four bolt extended unstiffened end plates with the same bolt layout pattern were used for both EEPS tests (Tests 2&4). End-plates of 6 mm thickness were used in Test 2 and 10 mm thickness were used in Test 4. The end-plate connection nominal geometric dimensions are given in Table 2.2.

Table 2.2: End-plate connection geometric dimensions

| Test # | Bolt Dia.& Grade | End-Plate Width, $b_p$ (mm) | End-Plate Thickness, $t_p$ (mm) | End-Plate Length,$L_p$ (mm) | End-Plate Extension $p_e$ (mm) | Pitch $p_{fi}$=$p_{fo}$ (mm) | Gage g (mm) |
|---|---|---|---|---|---|---|---|
| 2 | M6 8.8 | 100 | 6 | 210 | 35 | 20 | 50 |
| 4 | M6 8.8 | 100 | 10 | 210 | 35 | 20 | 50 |

The nominal connection geometric parameters shown in Table 2.2 are defined in Figure 2.1.

The same bolt layout pattern was used for both FWPBS tests (Tests 3 & 5). Flange plates of 6 mm thickness were used in Test 3 instead of 10 mm thickness plates in Test 5. Both tests used web plates of equal geometric dimensions (120mmx80mmx3mm). The gap between beams at the splice location was 10 mm in Test 3 while it was 5 mm in Test 5. The flange and web plate bolted splice connection geometric dimensions are shown in Figure 2.2.

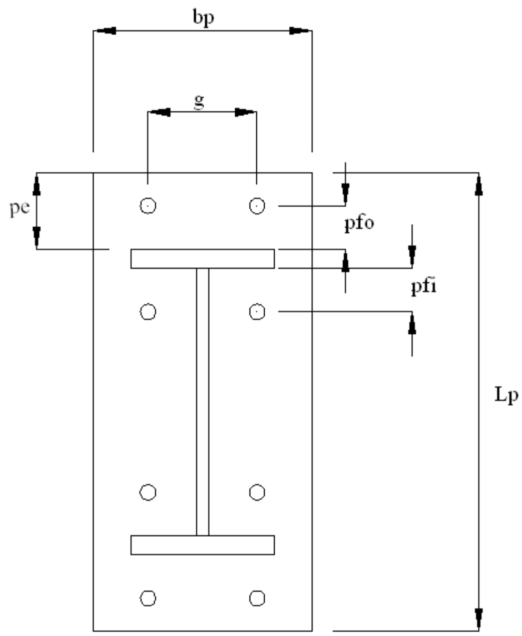



Figure 2.1: Four bolt end-plate connection geometry notation

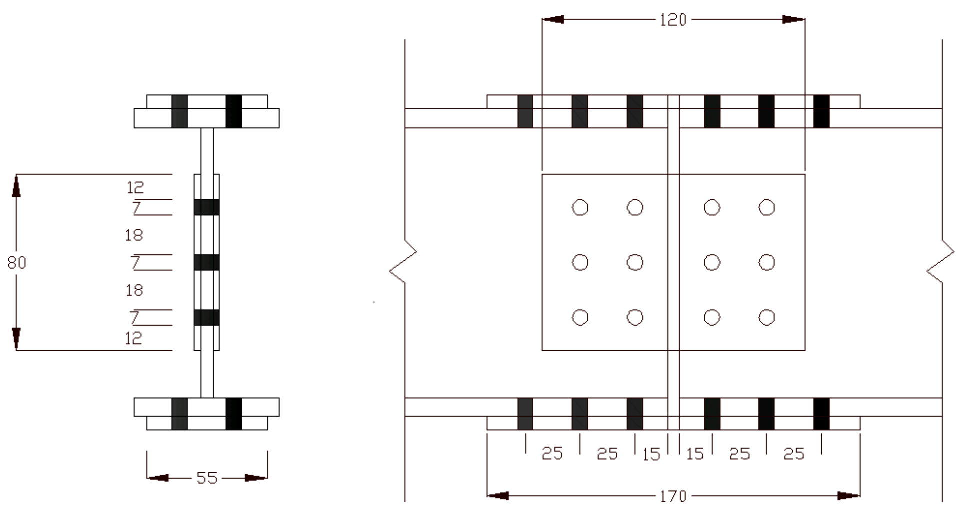



Figure 2.2: Flange & web plate bolted connection dimensions (mm)

In design of members always a margin is provided not to reach limiting requirements of the code. Most of these members are overdesigned by a factor. In such a case, splices are most of the time designed for forces on the members and not to capacity of the members. In investigated cases, the overdesign factors are determined by dividing unspliced member design load to spliced member design load. For instance, a 45 kN of design point load will develop 6.4 kN.m of design moment at the splice location, which is 20 cm from the south interior support. Member selection of IPN140 results in over design of member that can allow 60 kN of design point load in the absence of a splice. Therefore, the over design factor for the member is 60/45=1.33 as simulated in Test 2. Similar to this case, most of the splice connections are designed for developing a lower spliced member capacity than the unspliced member capacity.

As shown in Table 2.3, the overdesign factor is ranged from 1.33 to 2.61. The design details of splices are given in Appendix A.

Table 2.3: Overdesign factors for spliced members

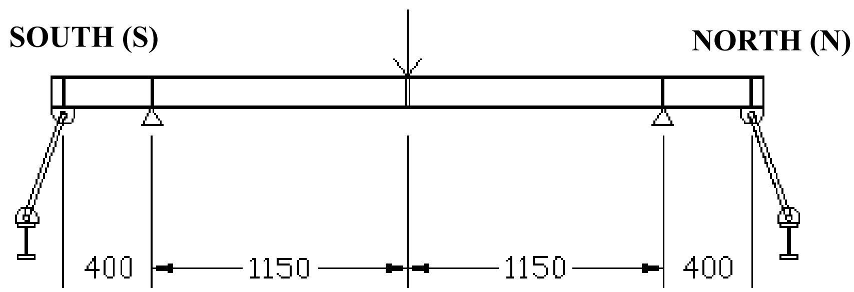
| Test | Member Over Design Factor<br>Unspliced Member Design Load/Spliced Member Design Load |
|---|---|
| 2 | 1.33 |
| 3 | 1.58 |
| 4 | 2.22 |
| 5 | 2.61 |

## 2.3 Test Setup

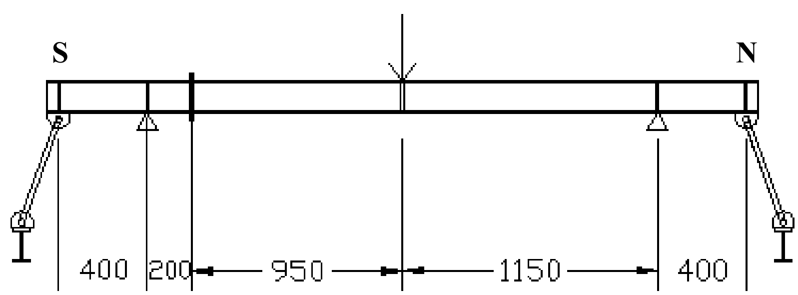
Mid span loading was applied using a hydraulic system with a loading capacity of 40 tons. The test beams were pin connected to 50 cm long, 24 mm diameter inclined rods at each end. The rods were connected by pins to 90 cm long IPN140 support beams which in turn were supported by the reaction floor. A photograph of a typical test setup is shown in Figure 2.3. The test setups for each of the tests are shown in Figure 2.4.


Figure 2.3: Test setup for a typical test

The applied load was measured by a load cell placed at point of load application at mid span. Also beam deflections were measured by LVDTs placed at mid span and splice locations. The instrumentation was calibrated prior to use and connected to a PC-based data acquisition system during testing. The load cell has a capacity of 15 tons and displacement transducers can measure up to 50 mm deflection.

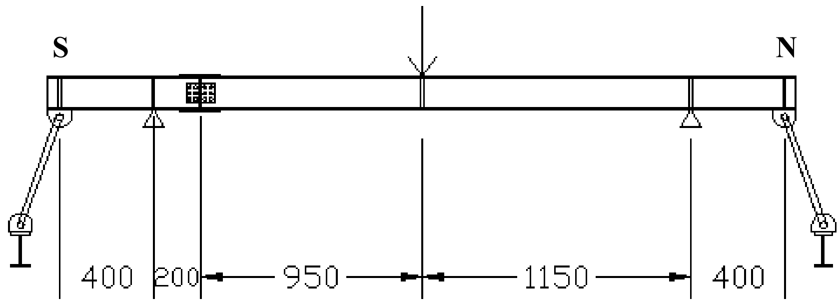



a) Test setup for Test 1

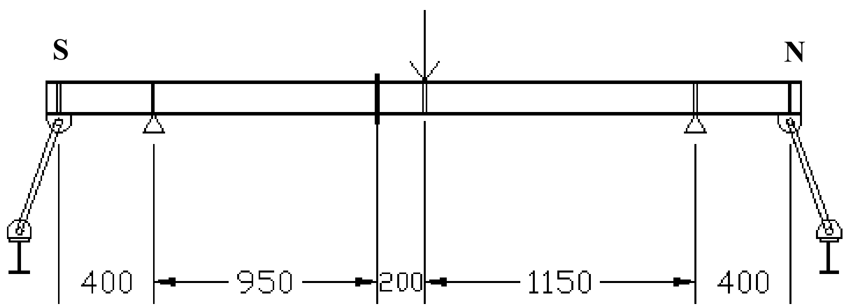



b) Test setup for Test 2

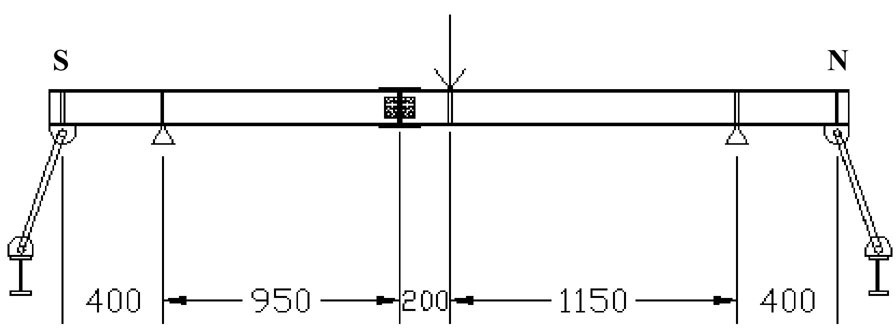
Figure 2.4: Test setups for all tests (dimensions in mm)




c) Test setup for Test 3




c) Test setup for Test 4




d) Test setup for Test 5

Figure 2.4 (cont`d): Test setups for all tests (dimensions in mm)

## 2.4 Testing Procedure

Once the test specimen was erected under the reaction frame, the displacement transducers and load cell were setup and connected to the data acquisition system. The instrumentation was then zeroed.

An initial zero reading was recorded and the test was begun with loading applied in increments of approximately 10 percent of the predicted failure load. Data points were recorded and the load steps were applied until the failure of the connection / beam was observed or until the capacity of testing instruments was reached.

## 2.5 Test Results

Load-deflection diagrams of the tests are shown in Figures 2.5 to 2.10. In Test 1, the data could not be retrieved after 120 kN because of a problem with the load cell. Yielding first happened at mid span followed by yielding of the beam at the internal supports.

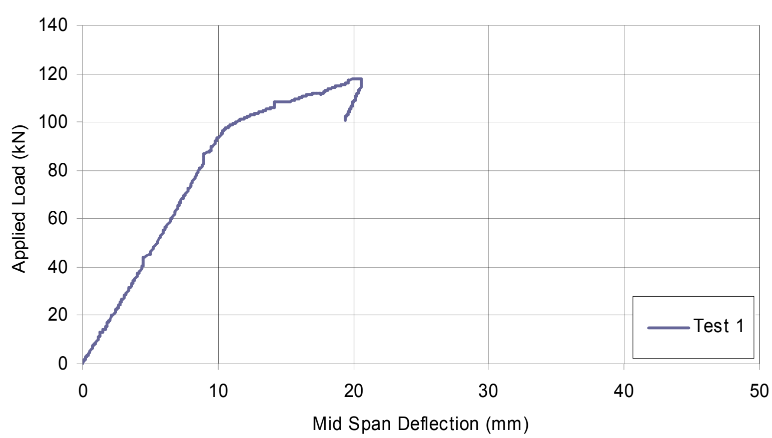



Figure 2.5: Applied load vs. mid span deflection response for Test 1

In Test 2, EEPS was located 20 cm from internal support as shown in Figure 2.4(b). The splice connection in this test like other splice connections in the remaining tests was designed to be weaker than the beam, taking into consideration the internal load effects determined from elastic analysis of the beam without splice flexibility. Test results indicated that the beam yielded first at mid-span following yielding over north support when mid-span deflection reached to 30 mm. This was due to the effect of the splice flexibility, increasing moment at mid span and north support and decreasing it at the splice location. The nuts were observed to slip from the body of bolts at 110 kN which was in agreement with the calculated capacity.

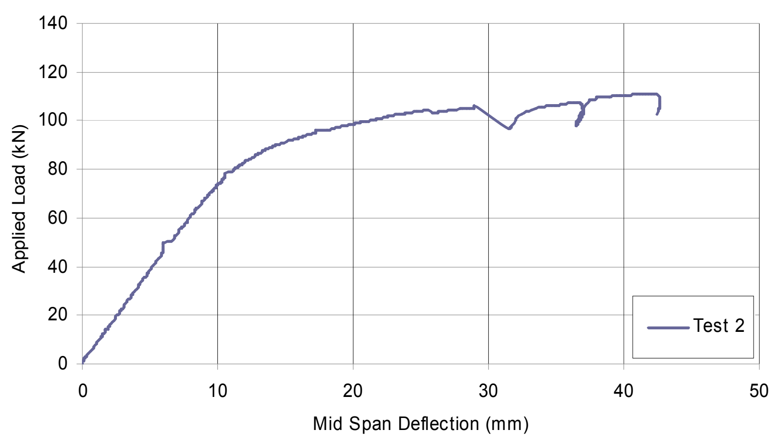



Figure 2.6: Applied load vs. mid span deflection response for Test 2

In Test 3, FWPBS was located 20 cm from internal support. Even if the load cell has 15 tons of capacity, the test was terminated a little bit over 10 tons due to safety reasons before reaching the calculated capacity of 118 kN of the specimen. Similar to Test 2, the beam yielded at the mid span and there was a significant bending on the top flange plate of the connection.

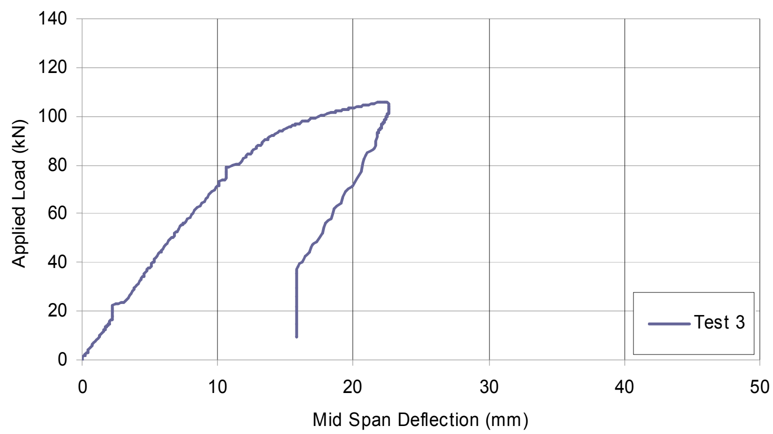



Figure 2.7: Applied load vs. mid span deflection response for Test 3

In test 4, the EEPS was located 20 cm from the mid span. The specimen had a calculated capacity of 50 kN. Despite the performance observed in Test 2, negative moment zone, the splice was subjected to more moment and nuts started to slip from body of bolts at 60 kN, which was the first sign of significant damage. The rod at the south end of the support is thought to have yielded and the load carrying system changed.

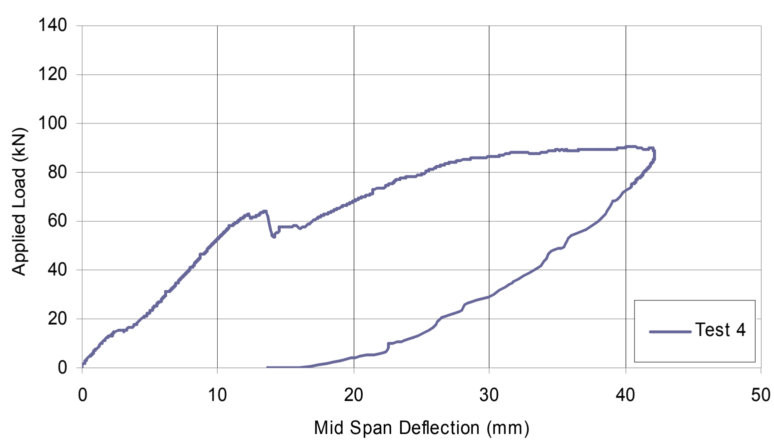



Figure 2.8: Applied load vs. mid span deflection response for Test 4

In Test 5, FWPBS was located 20 cm from the mid-span. Test 5 had a different boundary condition than Test 4 since south rod was not fully functioning and it was fully broken at applied load of 90 kN. North interior support yielded and the connection performed better than EEPS designed for similar capacity.

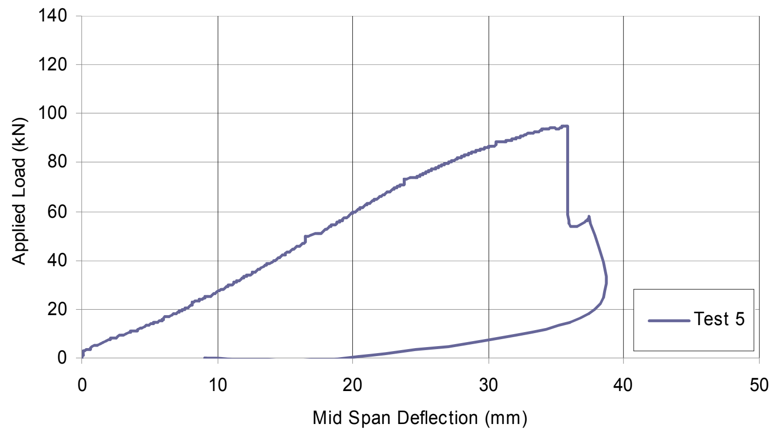



Figure 2.9: Applied load vs. mid span deflection response for Test 5

Superimposed test results are shown in Figure 2.10.

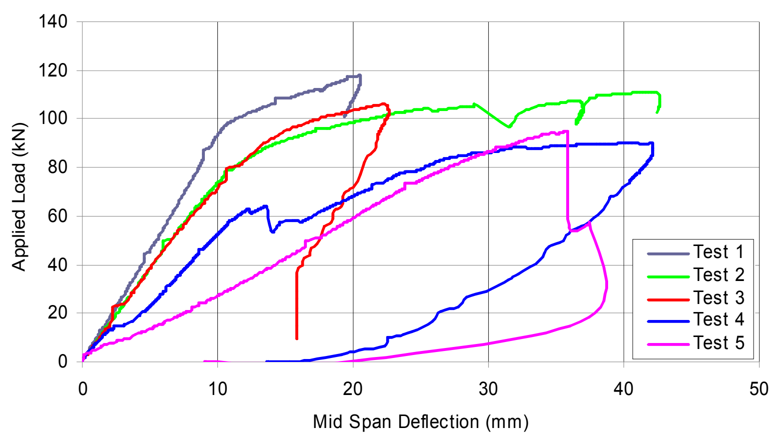



Figure 2.10: Applied load vs. mid span deflection response for all tests

As observed from Figure 2.10, demonstrating superimposed load – mid span deflection diagrams of all tests, using splices in the beams resulted in significant reduction in rigidity of the beams when compared with the unspliced beam in Test 1. Since the end plate splice was located in high moment region, close to mid span in Test 4, failure of the connection was observed early at 60 kN. Significant reduction of stiffness seen in Test 5 load deflection diagram is thought to be caused by the damaged rod at the south support. Photographs of specimens after testing for each of the tests are shown in Figures 2.11 to 2.15.


Figure 2.11: Photograph of Test 1 specimen after test

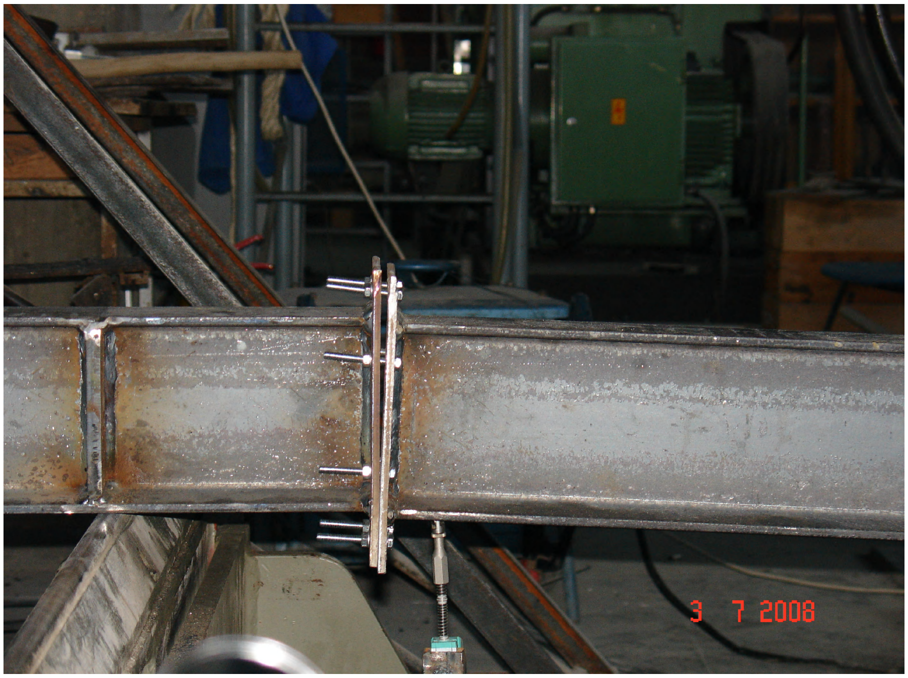



Figure 2.12: Photograph of Test 2 specimen after test

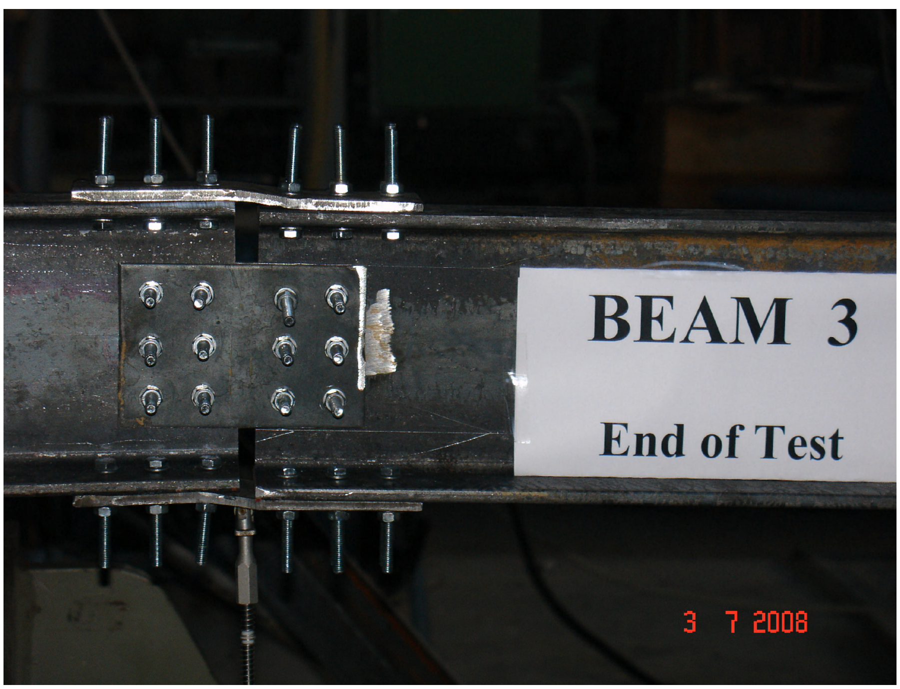



Figure 2.13: Photograph of Test 3 specimen after test

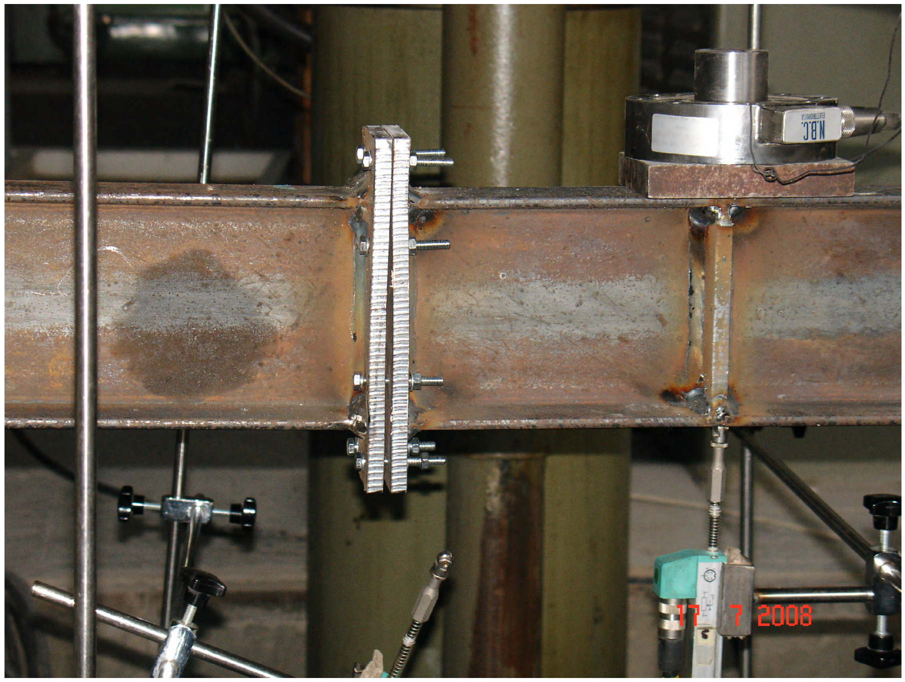



Figure 2.14: Photograph of Test 4 specimen after test

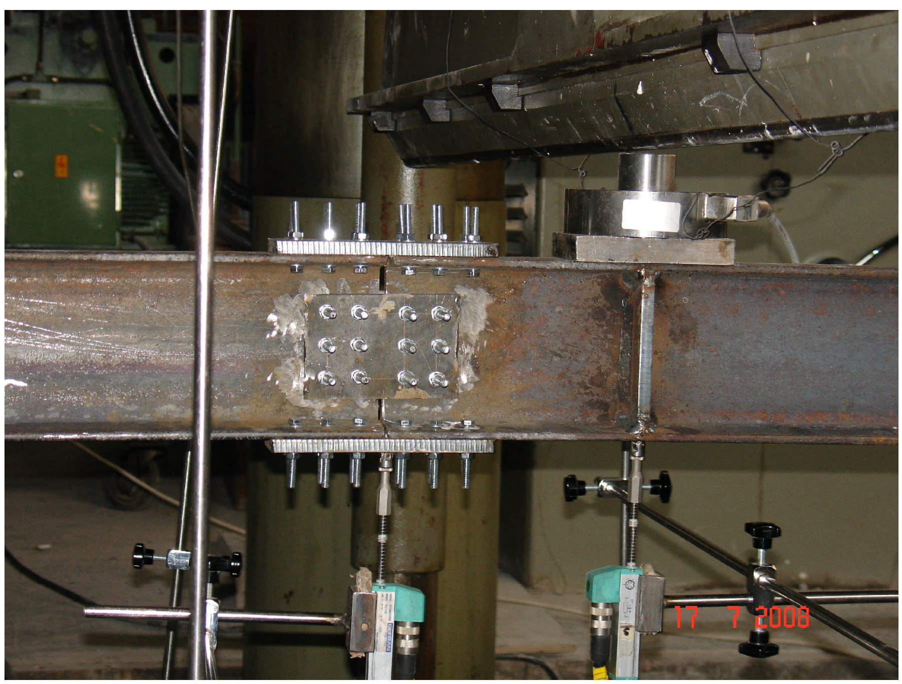



Figure 2.15: Photograph of Test 5 specimen after test

# CHAPTER 3

# FINITE ELEMENT STUDY AND COMPARISON WITH EXPERIMENTAL RESULTS

## 3.1 Finite Element Model

LARSA 4D, Structural and Earthquake Engineering Integrated Analysis and Design Software, was used to create and analyze finite element models of the two beam splice connections.

Numerical analysis of bolted connections depends heavily on constitutive relationships, step size, number of integration points, kinematic descriptions, element types and discretizations [11]. These items combined with complex non-linear phenomena which are commonly observed in connections make finite element modeling of connections difficult. In the research, three dimensional finite element models were used to analyze the two beam splice connection types in four tests. Rather than using highly complicated and varied element types available in literature, a simple modeling approach was utilized during preparation of finite element models of connections. In any case, whether the simple or advanced non-linear connection model is used in analysis, the residual stresses or locked in construction forces cannot be incorporated into model, which will result in inaccuracies in stress distribution and stress concentrations within the investigated elements.

Nonlinear translational spring elements instead of solid elements were used to model the bolts and again non-linear translational spring elements were used to model the interaction between end plates instead of contact elements to model the highly non-linear behavior. The comparisons between experimental and analytical results at the end of the chapter can be used to analyze the effectiveness of the approach followed. For global system checks, the proposed non-linear splice modeling is observed to be in good agreement with the test results even at post-yield zone. However, no completeness is claimed.

In the end plate type of spliced connections, the end plates were modeled using 4 node quadrilateral plate elements. The plate element in LARSA 4D is a planar element with constant thickness, with isotropic material properties. It is a linear element with linear material properties. These elements are designed to remain elastic.

The interaction of the end plates of the two spliced parts was modeled using compression only translational spring elements in the axial direction of the member placed on nodes of the plate elements. The load-deformation relationship of these springs as used in the analysis is shown in Figure 3.1. An arbitrarily large stiffness for these springs was selected to avoid penetration of plate elements modeling end plates into each other.

The contact problem between two bodies is highly non-linear with unknown boundary conditions. The friction between the end plates was modeled using translational spring elements in the transverse directions placed on the nodes of the plate elements which model the end plates. The load-deformation property of these spring elements is shown in Figure 3.2. These friction elements simulate an average friction on two end plate surfaces and friction coefficient was taken as 0.25 [15]. The average normal force was evaluated to be around 20% of yield strength of steel members. Static friction force was determined from

$$F_{fs} = \mu F_n \tag{3.1}$$

where $F_{fs}$ is the static friction force, $\mu$ is the static friction coefficient and $F_n$ is the average normal force. The relative displacement between two plates is supposed to be 0. However a small number has to be entered into the analysis program, representative of this situation. On static or kinematic friction coefficient of steel to steel, there is not much consensus due to uncertainties in determination of roughness of surfaces. In this study, kinematic coefficient was selected to be significantly smaller than static coefficient.

The analyses were carried out once with modeling the friction with these spring elements and once without considering friction since there is not much agreement in industry or academia on how to simulate friction. The bolts were modeled using spring elements with properties shown in Figures 3.3 & 3.4. The force-displacement relationships of the springs modeling the bolts reflected the coupon tests of the bolts.

FEM model of the end plate splice connection is rendered in Figure 3.5 and a skeleton model is shown in Figure 3.6.

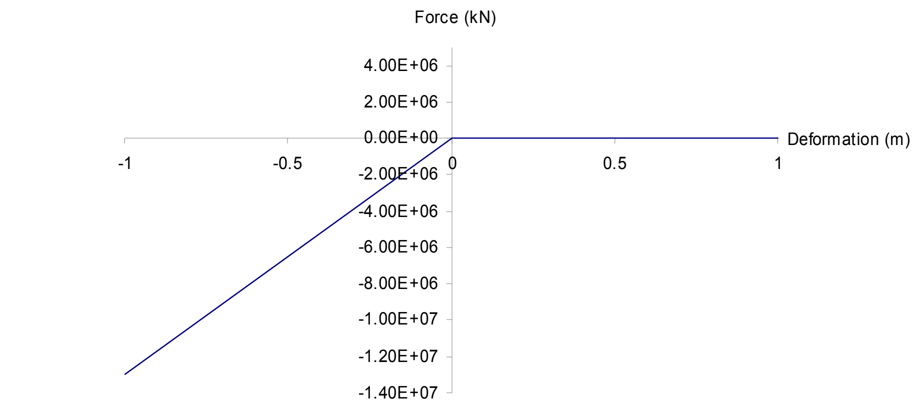



Figure 3.1: Force – deformation relationship of compression only spring elements modeling end-plate interaction

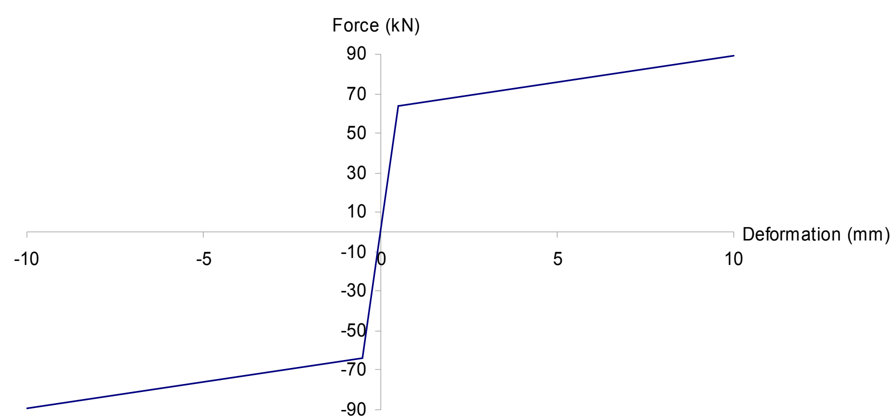



Figure 3.2: Force – deformation relationship of transverse direction spring elements modeling friction between end-plates

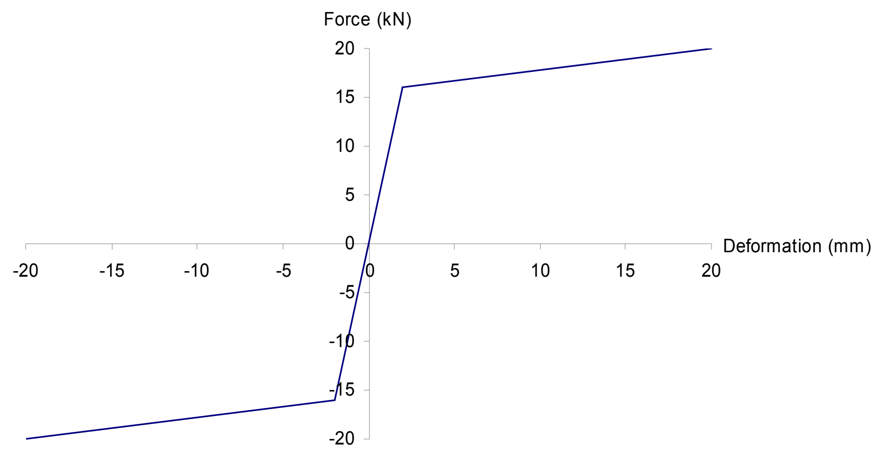



Figure 3.3: Force – deformation relationship of axial direction spring elements modeling bolt tensile action

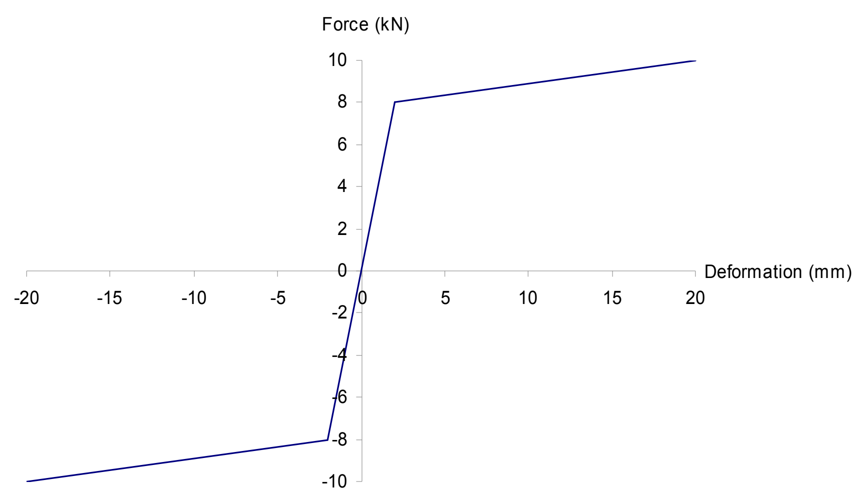



Figure 3.4: Force – deformation relationship of transverse direction spring elements modeling bolt shear action

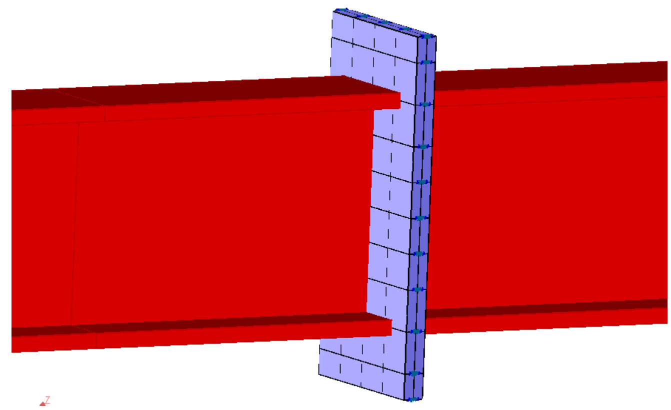


Figure 3.5: Rendered finite element model of the end-plate splice connection

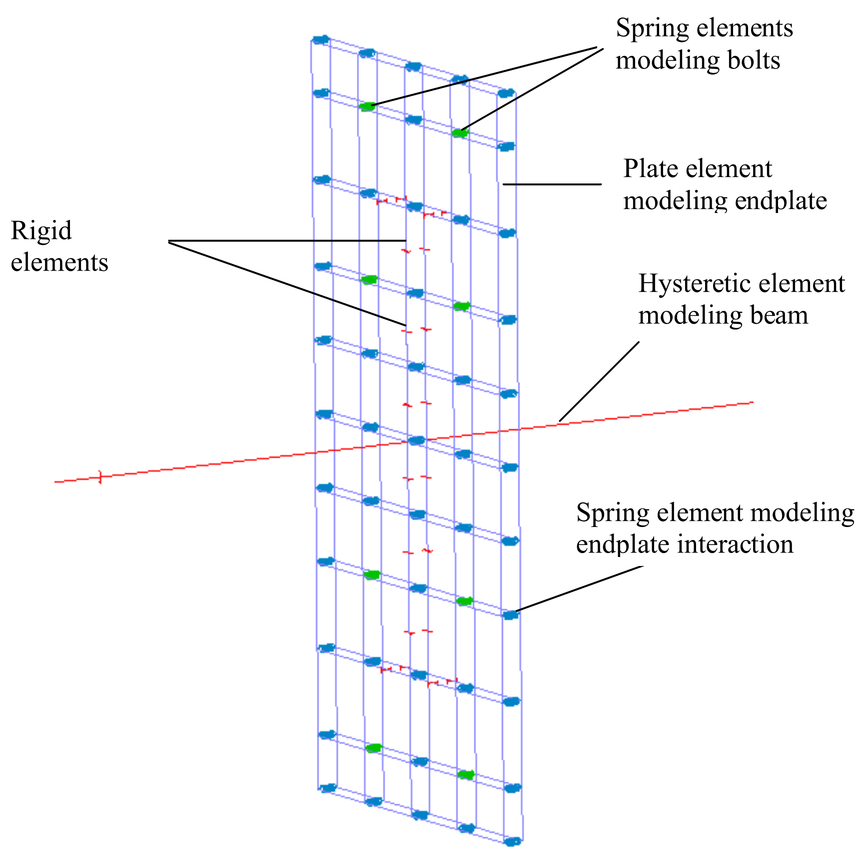



Figure 3.6: Skeleton finite element model of the end-plate splice connection

A bi-linear stress strain relationship was utilized for the structural steel hysteretic elements modeling the connected beam. The post yield to initial slope ratio of 0.02 was used. The yield strength, $F_y$, of 325 N/mm$^2$, which was determined from the coupon tests was used.

The hysteretic beam element was connected to the plate elements in such a way that the end of beam was modeled by rigid elements in depth to simulate the plane sections remain plane and normal sections remain normal condition to avoid stress concentrations at plate elements near neutral axis of the hysteretic beam element.

In the flange & web plate bolted splice connections the flange and web plates and the beam in the connection region were modeled again using the 4 node quadrilateral plate elements, which remain essentially elastic. A possible weak axis bending plastification at the gap can be ignored since the stiffness contribution of plate bending stiffness around its weak axis is insignificant compared to plate stiffness developed by its area around neutral axis of the connection. The bolts were modeled using the same spring elements used in the EEPS connection. Connection model of the flange and web plate splice is rendered in Figure 3.7 and a skeleton model is shown in Figure 3.8. The beam outside of connection region was modeled with hysteretic element with a yield strength of 325 MPa.

The hysteretic beam elements were connected to the plate elements in a similar way explained in the previous connection detail. Again plane sections remain plane and normals remain normal conditions are satisfied.

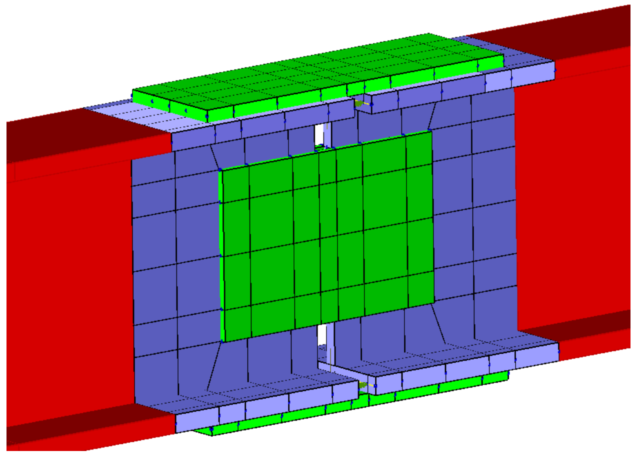


Figure 3.7: Rendered finite element model of the flange & web plate bolted splice connection

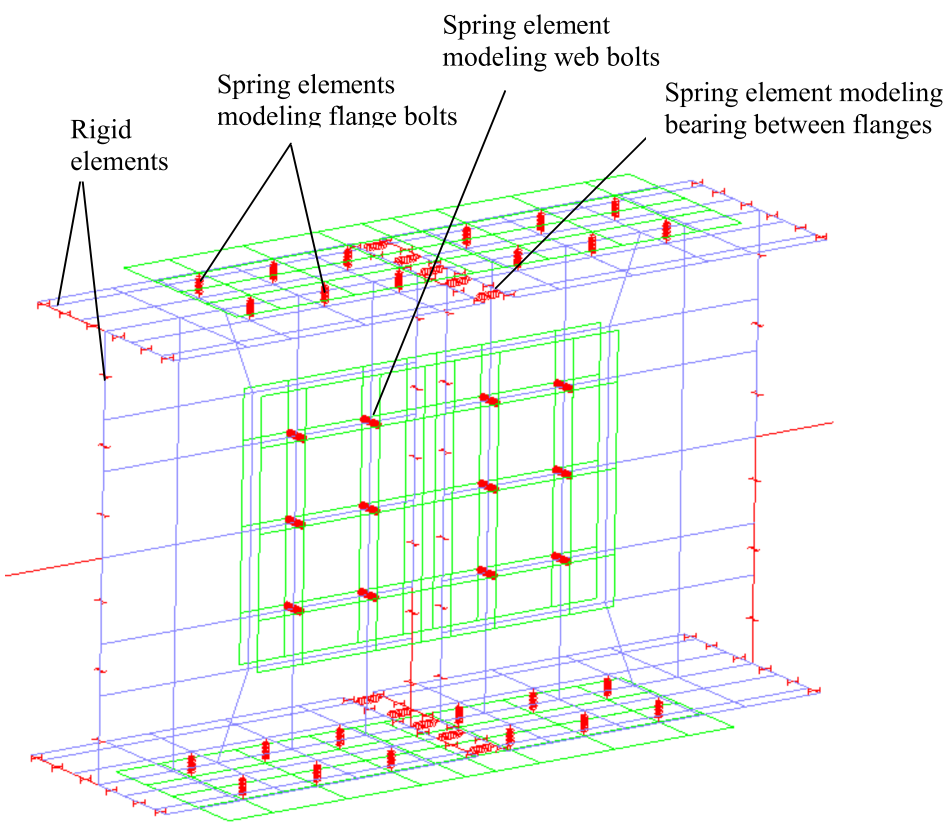



Figure 3.8: Skeleton finite element model of the flange & web plate bolted splice connection

In all models full boundary conditions were incorporated into model as shown in Figures 3.9 & 3.10. As stated in Chapter 2, the rod at the south support is believed to be damaged in Test 4, when the nuts slipped from the body of the bolt. So the south rod in the FEM analysis of Test 5 was modeled with reduced section properties.

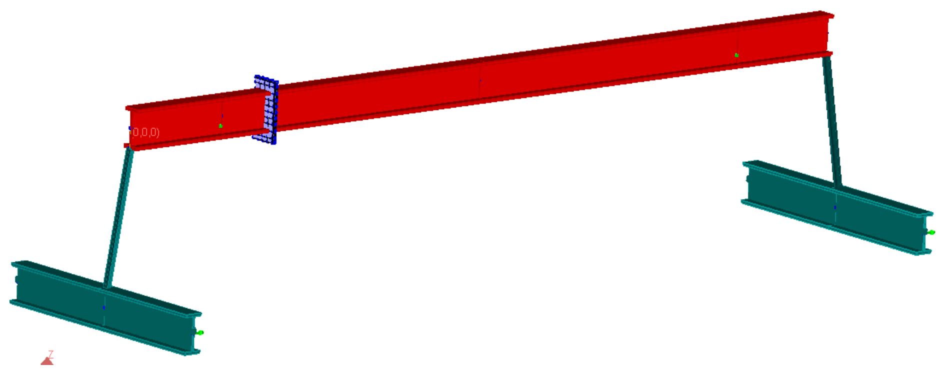


Figure 3.9: Modeled boundary conditions in end-plate splice

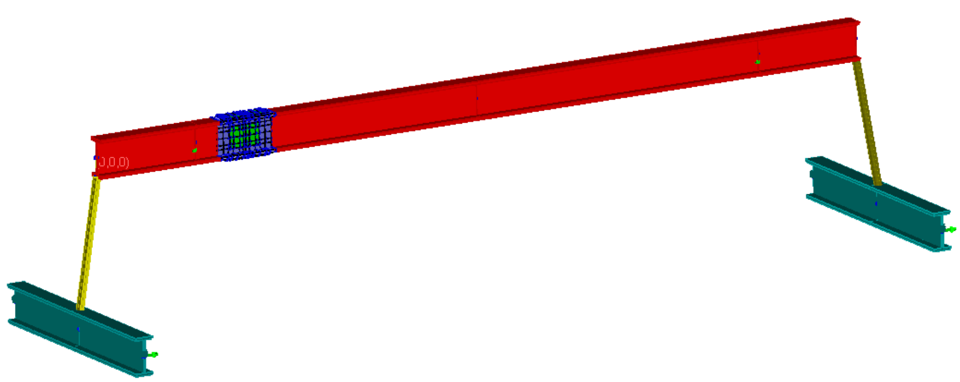


Figure 3.10: Modeled boundary conditions in flange & web plate bolted splice

## 3.2 Comparison of Results

All test results were able to be predicted both in elastic and plastic range with a good accuracy using the modeling described above as shown in Figures 3.11, 3.13, 3.15, 3.17 and 3.19. Deformed shapes of the finite element models of the test setups are shown in Figures 3.12, 3.14, 3.16, 3.18 and 3.20.

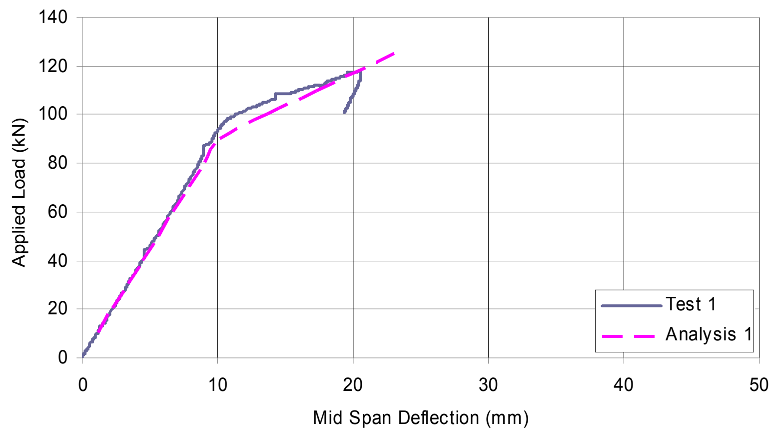



Figure 3.11: Test and analysis results compared for Test 1

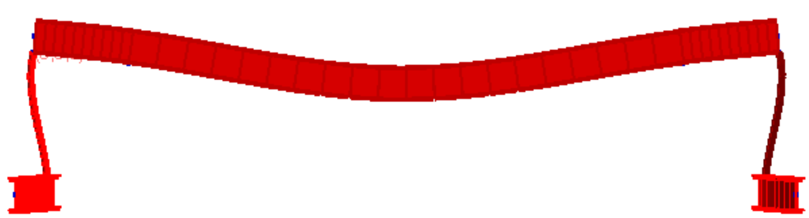


Figure 3.12: Deformed shape of the FEM of the Test 1 setup

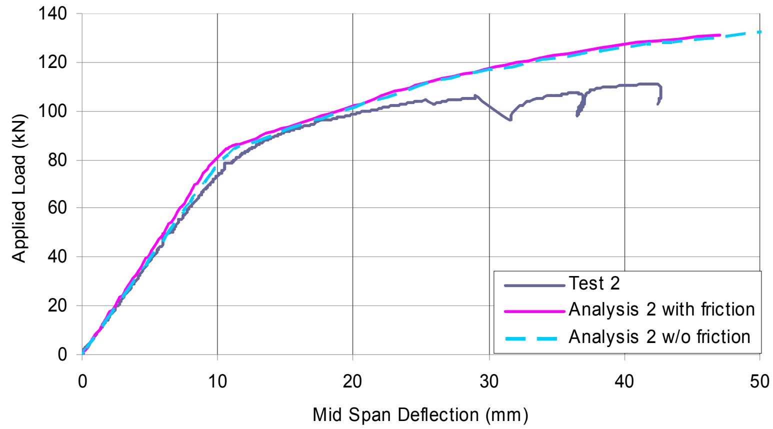



Figure 3.13: Test and analysis results compared for Test 2

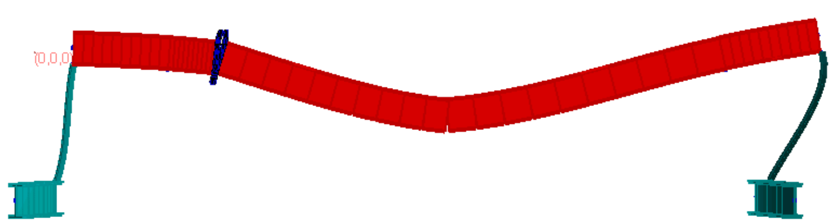


Figure 3.14: Deformed shape of the FEM of the Test 2 setup

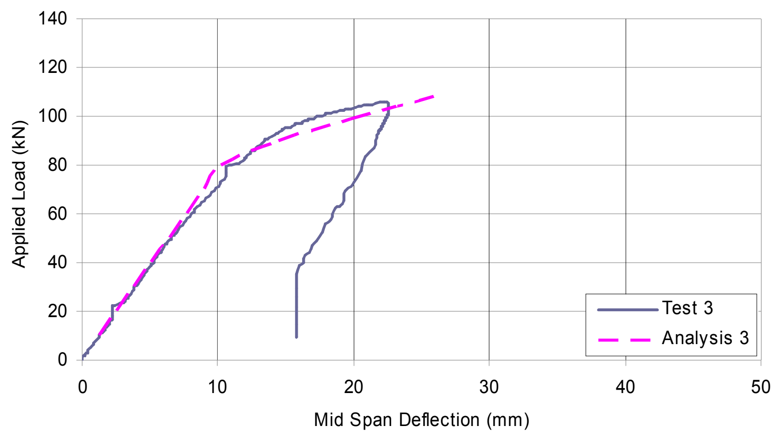



Figure 3.15: Test and analysis results compared for Test 3

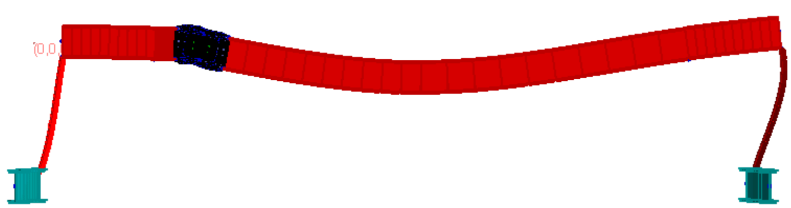


Figure 3.16: Deformed shape of the FEM of the Test 3 setup

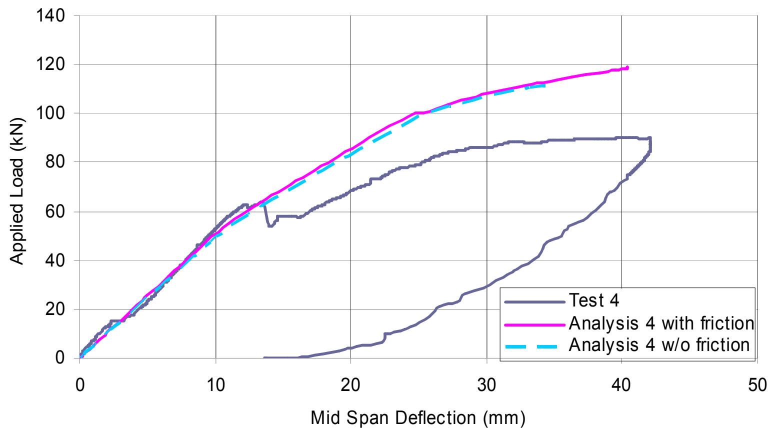



Figure 3.17: Test and analysis results compared for Test 4

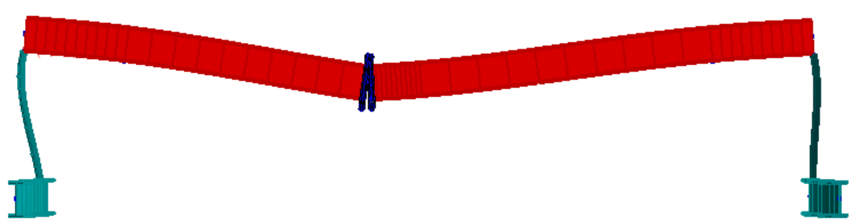


Figure 3.18: Deformed shape of the FEM of the Test 4 setup

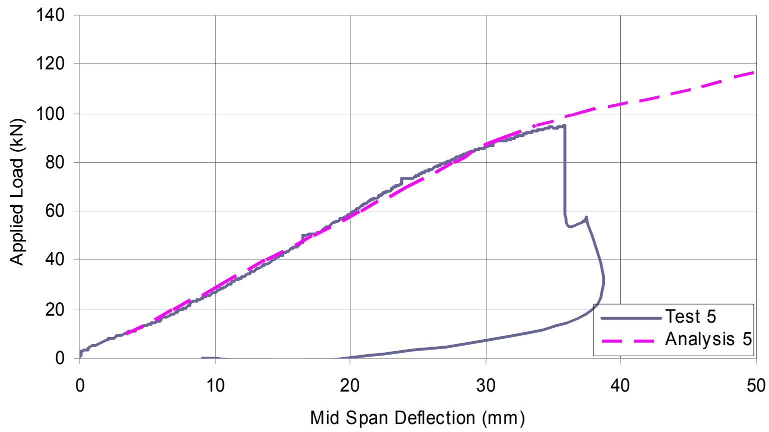



Figure 3.19: Test and analysis results compared for Test 5

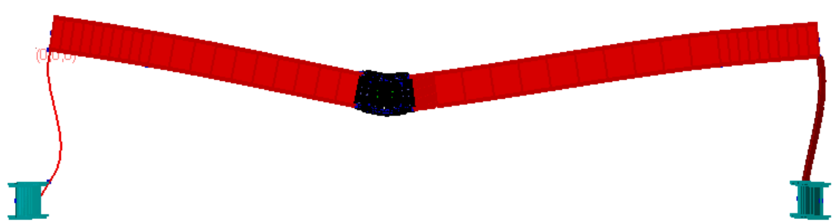


Figure 3.20: Deformed shape of the FEM of the Test 5 setup

Secant stiffness values obtained from experiments and FEM analyses are summarized in Table 3.1.

Table 3.1: Secant stiffness values obtained from tests and FEM analyses

| Test # | Secant Stiffness (kN/mm) | | | | | % of Softening of System Stiffness (Test) | % of Softening of System Stiffness (Analysis) | |
|---|---|---|---|---|---|---|---|---|
| | Test | FEM with friction | Diff. from test | FEM w/o frict. | Diff. from test | | with frict. | w/o frict. |
| 1 | 9.31 | 8.90 | 4.4% | - | - | | | |
| 2 | 7.59 | 8.31 | 9.5% | 7.84 | 3.3% | 18.5% | 6.6 | 11.9 |
| 3 | 7.47 | 7.80 | 4.4% | - | - | 19.8% | 12.4 | |
| 4 | 5.08 | 5.10 | 0.4% | 4.89 | 3.7% | 45.4% | 42.7 | 45 |
| 5 | 2.96 | 2.89 | 2.4% | - | - | 51.8% | 52.9 | |

As observed from Table 3.1, FEM analyses of the specimens resulted in secant stiffness values differing from the experiment results varying from 0.4% to 9.5%. Friction effects seem to be rather limited.

Percentage of softening in system stiffness, %S, was determined from the following equation:

$$\% S = (1 - K_{spl}/K_{unspl}) \text{ x } 100 \quad (3.2)$$

where $K_{spl}$ is the spliced structure secant stiffness and $K_{unspl}$ is the unspliced structure secant stiffness. In %S computation the $K_{unspl}$ term is taken equal to Test 1 secant stiffness, which is 9.31 kN/mm. However, due to the boundary condition change at Test 5, the $K_{unspl}$ is taken from analysis as 6.14 kN/mm.

In Figure 3.21, percentage of softening in system stiffness values of both splice connection types obtained from experiments and finite element method analyses for different over design factors are plotted.

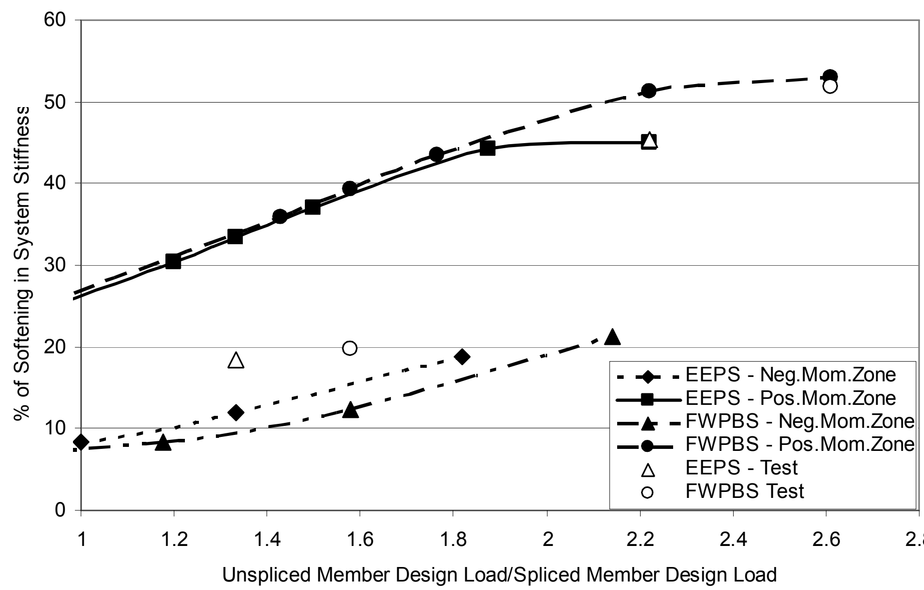



Figure 3.21: % of softening of system stiffness vs overdesign factors

As observed from the experimental data in Figure 3.21, over design factor (unspliced member design load/spliced member design load) has a significant effect on structural softening of the system. As the over design factor increases, softening of the system increases that means the weaker the connections are with respect to member, vulnerability of softening increases, endangering the serviceability of the system. Test results indicate that FWPBS have a better splice performance than EEPS in terms of percentage of softening in system.

From the analysis results, weaker splice connections are observed to be resulting in structural softening of the system. In terms of softening trend, the analytical results and test results are in good agreement. Except Test 5 results, analytical methods result in underestimation of softening by 3% to 12%.

The deformed shapes of the finite element models of the splice connections are seen in Figures 3.22, 3.24, 3.26 and 3.28. The photographs of the same connections after corresponding experiments are seen in Figures 3.23, 3.25, 3.27 and 3.29.

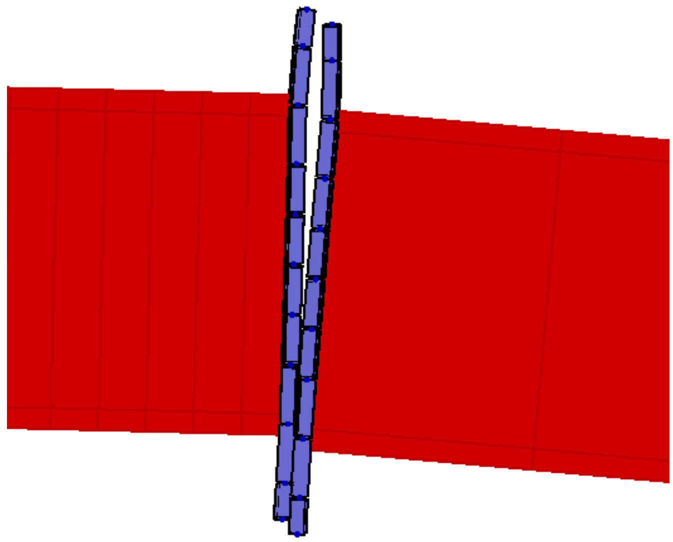


Figure 3.22: Deformed shape of the FEM of the Test 2 end plate splice

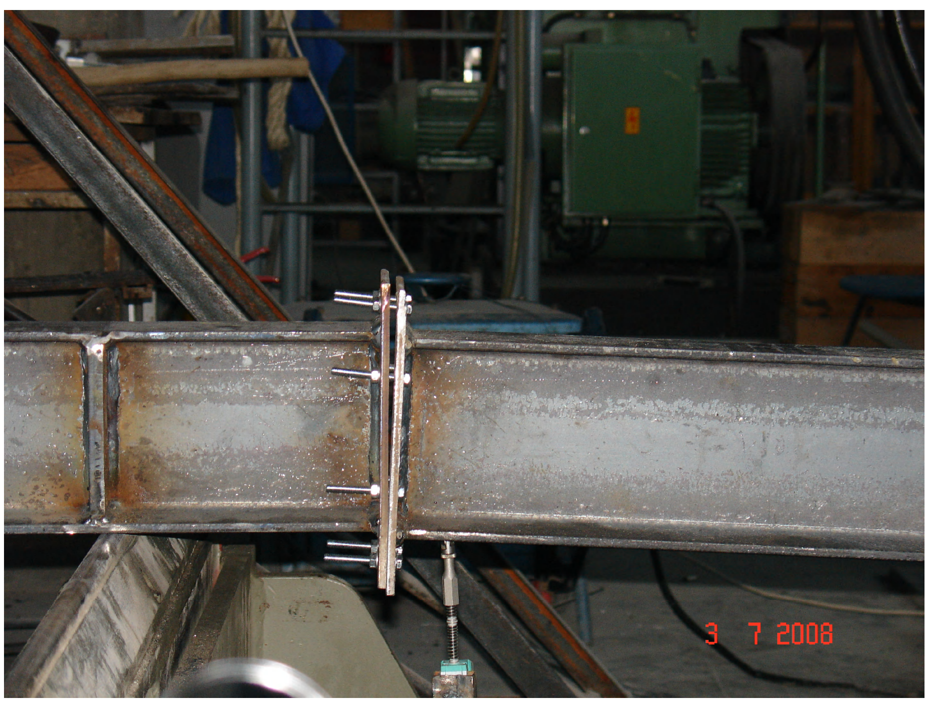



Figure 3.23: Photograph of Test 2 end plate splice after experiment

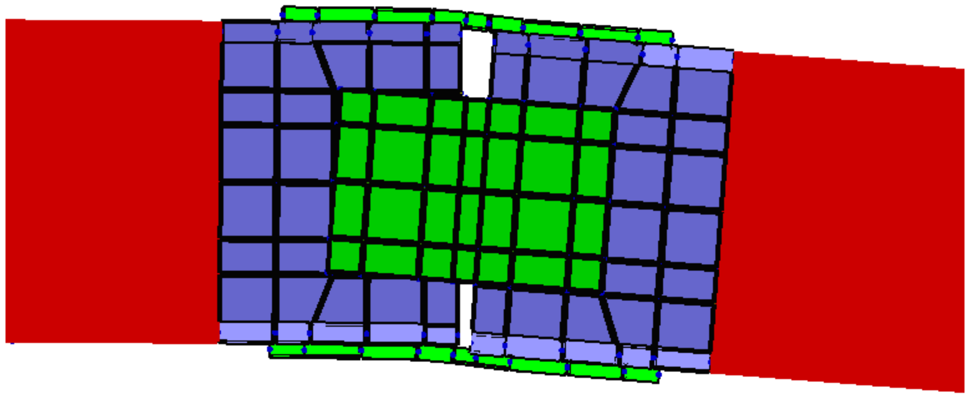


Figure 3.24: Deformed shape of the FEM of the Test 3 flange and web plate bolted splice


Figure 3.25: Photograph of Test 3 flange and web plate splice after experiment

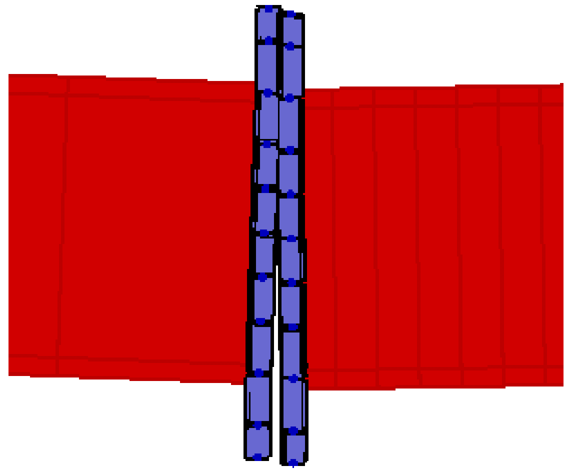


Figure 3.26: Deformed shape of the FEM of the Test 4 end plate splice

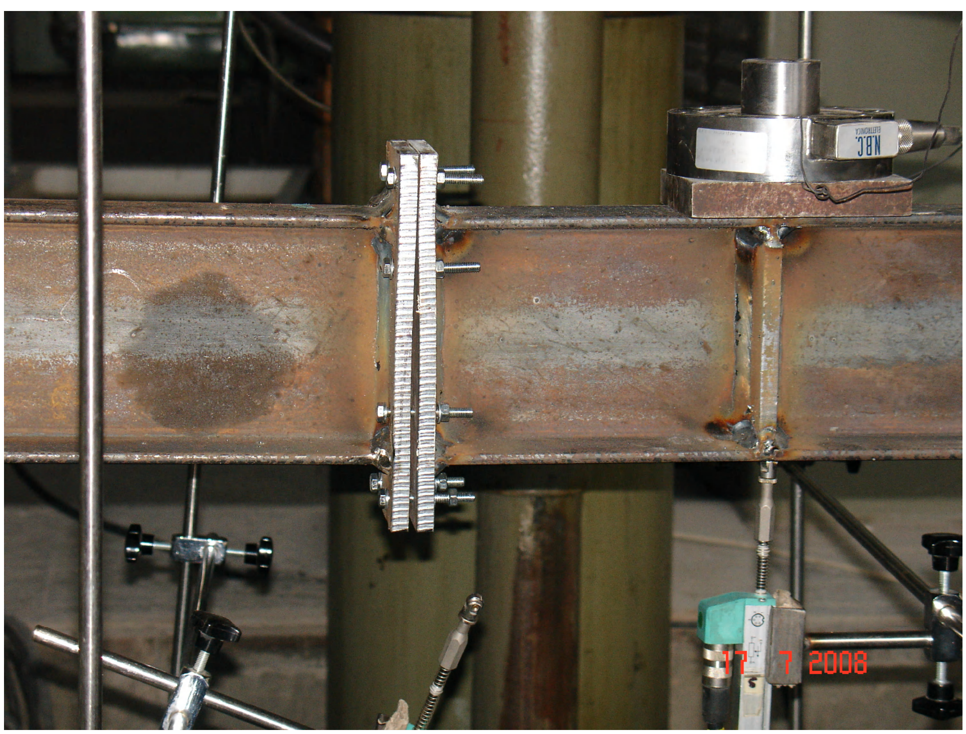



Figure 3.27: Photograph of Test 4 end plate splice after experiment

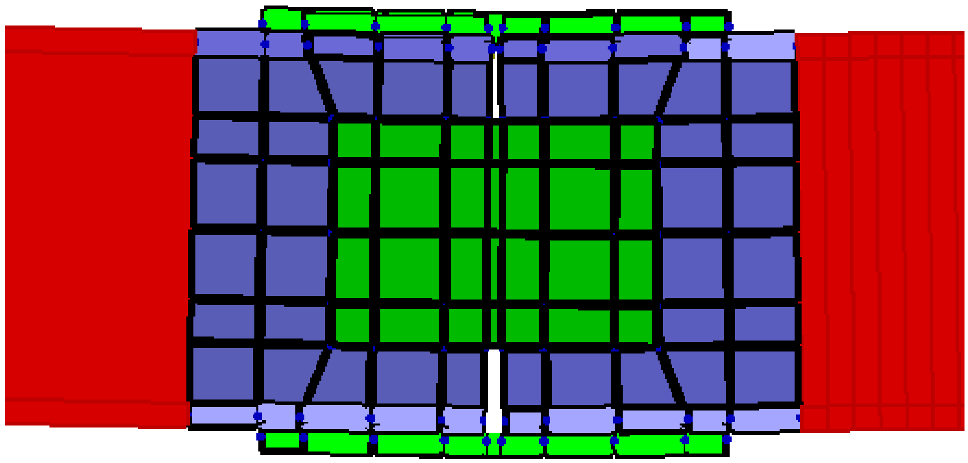


Figure 3.28: Deformed shape of the FEM of the Test 5 flange and web plate bolted splice

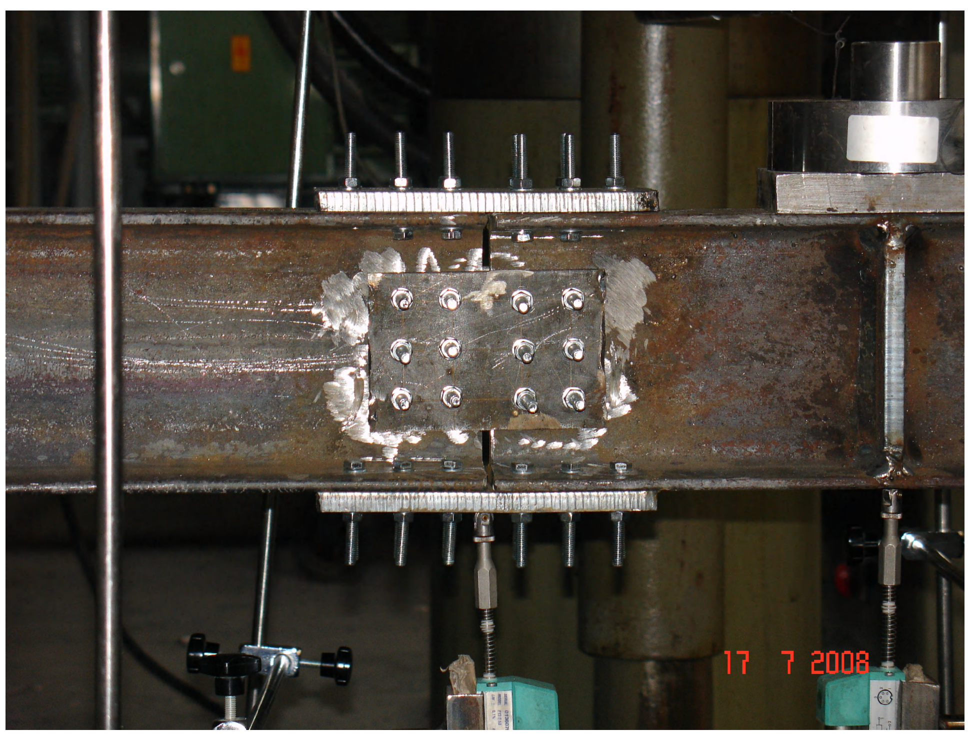



Figure 3.29: Photograph of Test 5 flange and web plate splice after experiment

# CHAPTER 4

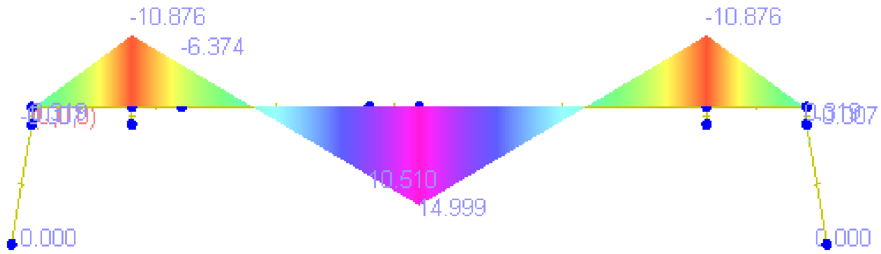
# COMPARISON OF ANALYSIS RESULTS WITH AND WITHOUT SPLICES

## 4.1 Verification of Experimental and Numerical Analysis Results With A Simple Analysis Method in Elastic Range

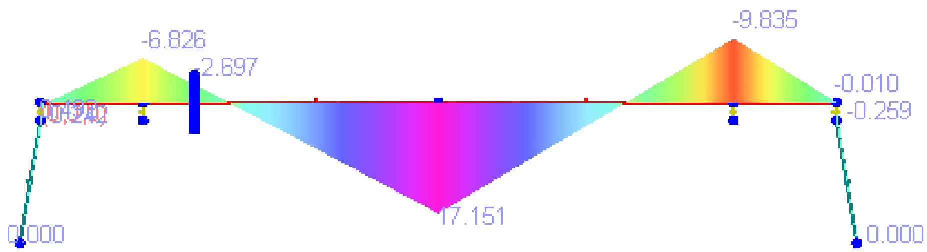
The spliced beams tested in the experiments are analyzed in this section under their design loads first without considering the flexibility of the splice connections. The moment diagrams obtained in this manner are shown in Figures 4.1(a), 4.2(a), 4.3(a) and 4.4(a).

The moment diagrams obtained from plastic analysis in the elastic range incorporating the FEM of the splice connection in the model resulted in the moment diagrams shown in Figures 4.1(b), 4.2(b), 4.3(b) and 4.4(b).

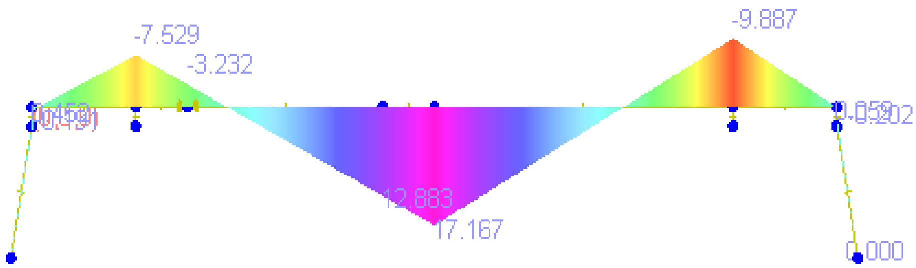
The moment diagrams obtained from elastic analysis in which 80% moment releases were placed at splice connection locations are shown in Figures 4.1(c), 4.2(c), 4.3(c) and 4.4(c).




a) Linear Elastic Analysis (P=45 kN)

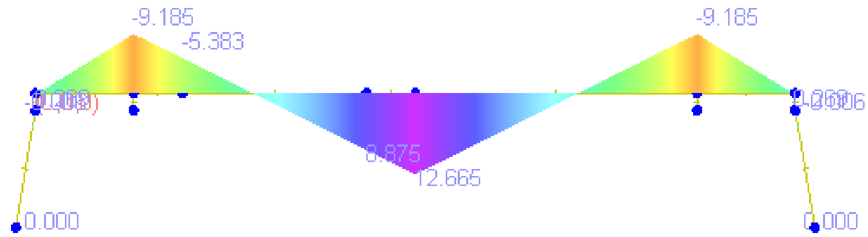



b) Plastic Analysis – in elastic range (P= 45 kN) – end plate connection modeled with FEM and non-linear springs

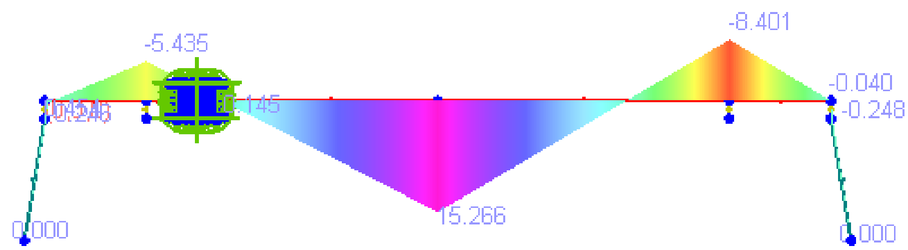



c) Linear Elastic Analysis (P= 45 kN) – moment released at end plate splice location by 80%

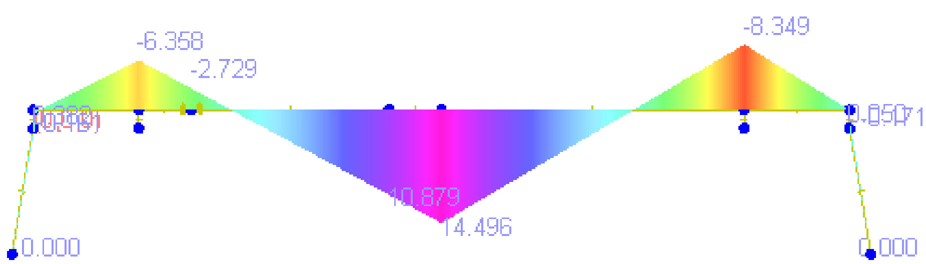
Figure 4.1: Moment Diagrams for Test 2 beam (kN.m)




a) Linear Elastic Analysis (P=38 kN)

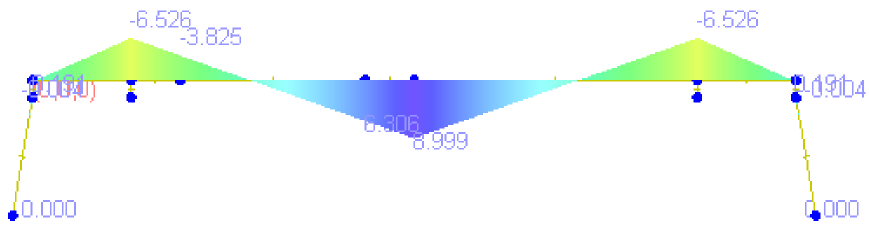



b) Plastic Analysis – in elastic range (P= 38 kN) – FWPBS modeled with FEM and non-linear springs

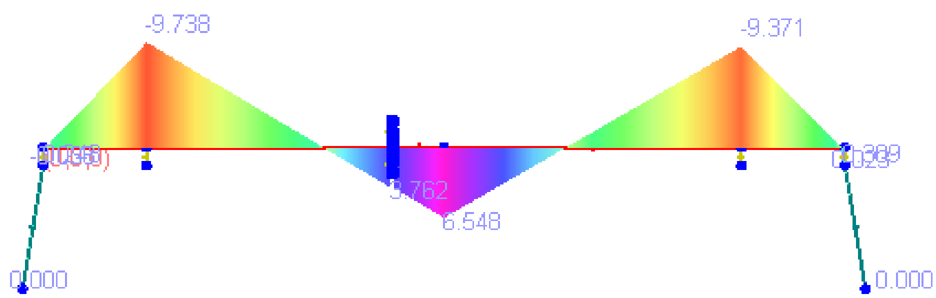



c) Linear Elastic Analysis (P= 38 kN) – moment released at FWPBS location by 80%

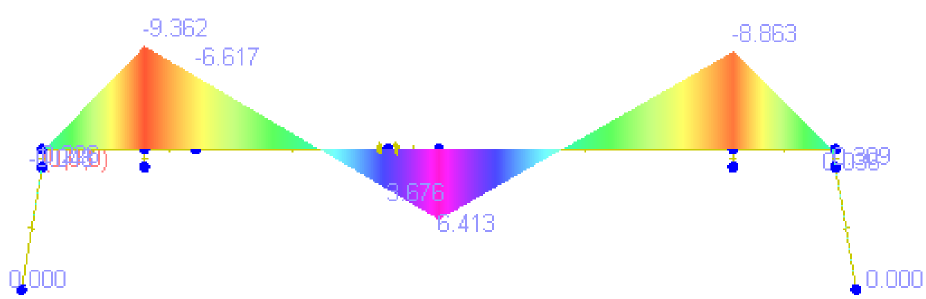
Figure 4.2: Moment Diagrams for Test 3 beam (kN.m)




a) Linear Elastic Analysis (P=27 kN)

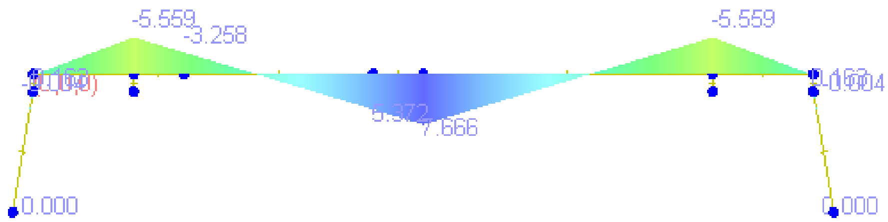



b) Plastic Analysis – in elastic range (P= 27 kN) – end plate connection modeled with FEM and non-linear springs

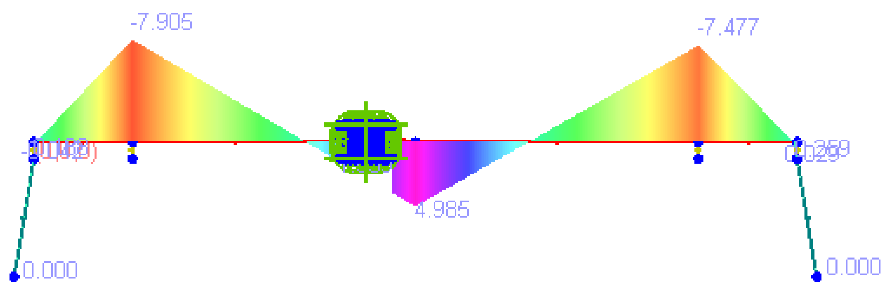



c) Linear Elastic Analysis (P= 27 kN) – moment released at end plate splice location by 80%

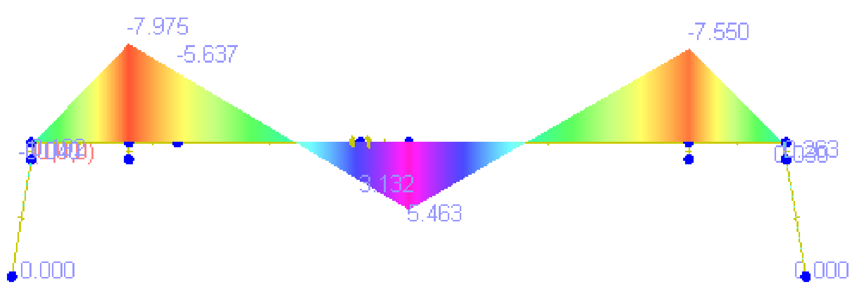
Figure 4.3: Moment Diagrams for Test 4 beam (kN.m)




a) Linear Elastic Analysis (P=23 kN)




b) Plastic Analysis – in elastic range (P= 23 kN) – FWPBS modeled with FEM and non-linear springs




c) Linear Elastic Analysis (P= 23 kN) – moment released at FWPBS location by 80%

Figure 4.4: Moment Diagrams for Test 5 beam (kN.m)

The moment values and mid span deflections obtained from these analyses and percent differences between the analyses results with and without splices are summarized in Table 4.1 and Table 4.2.

Table 4.1: Summary of analysis results for test cases

| Test # | Model Type | Moments (kN.m) | | | | | |
|---|---|---|---|---|---|---|---|
| | | Mid Span | Diff. | South Support | Diff. | North Support | Diff. |
| 2 | No Splice | 15 | - | 10.9 | - | 10.9 | - |
| | Splice - FEM | 17.2 | 14.7% | 6.8 | 37.6% | 9.8 | 10.1% |
| | Splice 80% Release | 17.2 | 14.7% | 7.5 | 31.2% | 9.9 | 9.2% |
| 3 | No Splice | 12.7 | - | 9.2 | - | 9.2 | - |
| | Splice-FEM | 15.3 | 20.5% | 5.4 | 41.3% | 8.4 | 8.7% |
| | Splice 80% Release | 14.5 | 14.2% | 6.4 | 30.4% | 8.3 | 9.8% |
| 4 | No Splice | 9 | - | 6.5 | - | 6.5 | - |
| | Splice-FEM | 6.5 | 27.8% | 9.7 | 49.2% | 9.4 | 44.6% |
| | Splice 80% Release | 6.4 | 28.9% | 9.4 | 44.6% | 8.9 | 36.9% |
| 5 | No Splice | 7.7 | - | 5.6 | - | 5.6 | - |
| | Splice-FEM | 5 | 35.1% | 7.9 | 41.1% | 7.5 | 33.9% |
| | Splice 80% Release | 5.5 | 28.6% | 8 | 42.9% | 7.5 | 33.9% |

As observed from Table 4.1, incorporation of splice flexibility at the splice location via the FEM of the connection which was shown to produce comparable results with experiments in the previous chapter, results in changes in design moment values varying from 9% to 49%. Thus the assumption of continuity at the splice location and the load effects obtained as a consequence of this assumption seems not to be reflecting the real situation.

Table 4.2: Summary of mid span displacements for test cases

| Test # | Model Type | Mid-Span Deflection (mm) | Difference |
|---|---|---|---|
| 2 | No Splice | 4.07 | |
| | Splice - FEM | 4.69 | 15% |
| | Splice-80%Release | 4.94 | 21% |
| 3 | No Splice | 3.43 | |
| | Splice-FEM | 4.15 | 21% |
| | Splice-80%Release | 4.17 | 22% |
| 4 | No Splice | 2.44 | |
| | Splice-FEM | 3.77 | 55% |
| | Splice-80%Release | 3.95 | 62% |
| 5 | No Splice | 2.08 | |
| | Splice-FEM | 2.95 | 42% |
| | Splice-80%Release | 3.36 | 62% |

As observed from Table 4.2, incorporation of splice flexibility at the splice location via the FEM of the connection, results in changes in mid span deflections varying from 15% to 55%.

Also it is observed from the analysis results that inserting the FEM of the specific splice type or an 80% moment release at the splice connection location produces comparable results.

# CHAPTER 5

# SUMMARY AND CONCLUSIONS

## 5.1 Summary

Beam splices, which can be used due to construction or design requirements, are typically located in regions where internal forces are low. Most design specifications require beam splices to be designed for moment and shear values determined from analysis occurring at the location of the splice or for some minimum strength. Hence, these splice connections are most of the time partial strength, partially restrained connections. In structural analysis the flexibility introduced by the splice at the splice location is typically ignored. The appropriateness of this approach of, neglecting the splices in the analysis and analyzing the structure assuming continuous members at splice locations, is investigated in this research thorough a combined experimental program and finite element model analysis.

In the experimental program, described in Chapter 2, the behavior of two types of partial strength splice connections and the effect of this behavior on the distribution of internal load effects in the test beams were investigated. Finite element models of the splice connections were prepared and analyses of finite element models of the test setups with complete boundary conditions were done following the experiments. The design elastic analyses of the test setups were done in Chapter 4 once without splices and then with splices which were incorporated into the structural system through either a FEM of the splice or an 80% moment release at the splice location. Comparisons were made between the analyses with splices and without splices.

## 5.2 Conclusions

The conclusions of this study can be summarized as follows:

- Splice connection flexibility is very important to define in analysis to simulate the accurate structural response.

- While the practice of locating the splices at low moment regions is an appropriate approach still it is important to define splice flexibility in the analysis of these structures. The moderate difference between the design moment diagrams and real behavior may become a high difference under unfavorable loading arrangements which increase the moment at the splice locations.

- The splice behavior can be incorporated into the analysis thorough

    - Conducting experiments on the specific splice connection type and inserting the observed moment rotation characteristics of the specific connection type into the analysis.

    - The FEMs developed in this research of two different specific splice types can be used as a starting point for modeling same type splices and with different geometries, and then these FEMs may be used to investigate the behavior of connections and may be used in the analysis of the system.

- The simplified approach of inserting an 80% stiffness moment release has been shown to produce similar results both to the results of FEM analysis and experiments. It should be mentioned that the splices were designed to be partial strength and to have 34-40% of the strength of the unspliced members in this study. So in the specific analyses to be conducted, the splice strengths must be evaluated with respect to the member strength and higher or lower percentages of moment release may be suitable.

- Experimental results indicate that beam without a splice performs better than a beam with a splice when structural responses are compared.

- Among the two investigated splices, extended end plate splice connection resulted in an early bolt connection failure compared to flange and web plate bolted splice connection designed per AISC.

- Each type of splice connection softens the rigidity of the structure and end plate splice connection is observed to be the softest one compared to flange and web plate bolted splice.

- The weaker the connections are with respect to member, vulnerability of softening increases, endangering the serviceability of the system

- A design based on an analysis without simulation of splice connections can significantly underestimate vertical deflections due to live load. In any case, partial strength splices soften the structural response endangering serviceability.

## 5.3 Suggestions

Splice connections designed to full strength of the members can be tested to understand their effect on flexibility of the system.

# REFERENCES


[1] Salmon C.G., Johnson J.E., 1996, "Steel Structures: Design and Behavior Emphasizing Load and Resistance Factor Design, 4th Edition", Prentice Hall, New Jersey

[2] American Institute of Steel Construction, Inc. (AISC), 2005, "Specification for Structural Steel Buildings", Chicago, IL.

[3] European Committee for Standardization, 2003, "Eurocode 3: Design of Steel Structures - Part 1-8: Design of Joints, prEN 1993-1-8-2003", Belgium

[4] American Association of State Highway and Transportation Officials, (AASHTO), 1996, "Standard Specifications for Highway Bridges", Washington D.C.

[5] American Institute of Steel Construction, Inc. (AISC), 1994, "Manual of Steel Construction - Load & Resistance Factor Design, 2nd Ed.", Chicago, IL.

[6] Hart W.H., Milek W.A., 1965, "Splices in Plastically Designed Continuous Structures", Engineering Journal, AISC, No.2., 33-37

[7] Sumner E.A., 2003, "Unified Design of Extended End-Plate Moment Connections Subject to Cyclic Loading", Phd. Thesis, Department of Civil Engineering, Virginia Polytechnic Institute and State University

[8] Murray T.M., Shoemaker W.L., 2002, "Steel Design Guide Series 16, Flush and Extended Multiple-Row Moment End-Plate Connections", American Institute of Steel Construction, Inc., Chicago, IL.

[9] Murray T.M., Sumner E.A., 2004, "Steel Design Guide 4, Extended End-Plate Moment Connections, Seismic and Wind Applications, 2nd Edition", American Institute of Steel Construction, Inc., Chicago, IL.

[10] Sherbourne A.N., Bahaari M.R., 1994, "3-D Simulation of End-Plate Bolted Connections", Journal of Structural Engineering, Vol.120, No.11, 3122-36

[11] Choi C.K., Chung G.T., 1996, "Refined Three-Dimensional Finite Element Model for End-Plate Connection", Journal of Structural Engineering, ASCE, 122(11), 1307-1316

[12] Bursi O.S., Jaspart J.P., 1998, "Basic Issues in the Finite Element Simulation of Extended End Plate Connections", Computers & Structures, Pergamon, 69(3), 361-382

[13] Beam Splice Moment Connection, University of Wyoming College of Engineering, 2008, "wwweng.uwyo.edu/connections/connection8.html"

[14] Sheikh-Ibrahim F.I., Frank K.H., 1998, "The Ultimate Strength of Symmetric Beam Bolted Splices", Engineering Journal, 3rd Quarter, 106-118

[15] Prestressed Concrete Institute, PCI Design Handbook, 3rd Edition, Chicago, Illinois

# APPENDIX A

# DESIGN OF TEST SETUPS

## 1. Design of Test Setup – End Plate Splice

In this section the recommended steps to design a bolted end plate moment connection as per AISC Design Guide 4 are summarized along with the design calculations performed in the design of Test 2 beam splice connection. IPN140 beams of St 37 steel used in the tests have the following tabulated geometric properties shown in Table A.1.1.

Table A.1.1: Beam Properties

| Section | IPN140 |
|---|---|
| Depth, h | 140 mm |
| Flange Width, $b_f$ | 66 mm |
| Flange Thickness, $t_f$ | 8.6 mm |
| Web Thickness, $t_w$ | 5.7 mm |
| Moment of Inertia, $I_y$ | 573 x $10^4$ $mm^4$ |
| Elastic Section Modulus | 81.9 x $10^3$ $mm^3$ |
| Plastic Section Modulus | 95.4 x $10^3$ $mm^3$ |

The design steps followed are as follows:

1. Determine the required connection design moment and design shear

From elastic analysis of the beam under 45 kN as shown in Section 4.1,

$$M_u = 6.4kN.m$$

$$V_u = 22.5kN$$

2. Select an end plate moment connection configuration and establish preliminary values for the connection geometry (g, $p_{fi}$, $p_{fo}$, $p_{e}$ etc) and bolt grade.

Four bolt extended end plate moment connection with following connection geometric properties is selected. 8.8 grade bolts are to be selected for the connection.

$b_p$ = 100 mm (end plate width) $L_p$ = 210 mm (end plate length)

$p_e$ = 35 mm (end plate extension) $p_{fi}=p_{fo}$ = 20 mm (inner & outer pitch)

g = 50 mm (gage)

3. Determine the required bolt diameter, $d_{b\ Req`d}$

$$d_{b\ Req`d} = \sqrt{\frac{2M_u}{\pi\phi F_t(h_o + h_1)}}$$

$M_u$ = 6.4 kN.m

$F_t$ = 600 MPa (reduced bolt tensile strength = 0.75 x 800)

$h_o$ = 155.7 mm

$h_1$ = 107.1 mm

$$d_{b\ Req`d} = \sqrt{\frac{2\times 6.4\times 10^6}{\pi\times 0.75\times 600\times(155.7+107.1)}} = 5.9 \text{ mm} \longrightarrow \text{Use M6}$$

4. Calculate the no prying bolt moment strength, $M_{np}$

$M_{np} = 2P_t(h_0 + h_1)$

$P_t$ = Bolt tensile strength = 16.1 kN

$M_{np} = 2\times 16.1\times(0.1557+0.1071) = 8.5$ kN.m

5. Determine the required end plate thickness,

$$t_{p\ Req`d} = \sqrt{\frac{1.11\phi M_{np}}{\phi_b F_{yp} Y_p}}$$

$F_{yp}$= the end plate material yield strength

$Y_p$= the end plate yield line mechanism parameter from Table 3.1 of AISC Design Guide 4

$$t_{p\ Req`d} = \sqrt{\frac{1.11\times 0.75\times 8.5\times 10^6}{0.9\times 235\times 952.934}} = 5.9 \text{ mm} \longrightarrow \text{Use 6 mm}$$

6. Calculate the factored beam flange force

$$F_{fu} = \frac{M_u}{d - t_{fb}} \quad ; \quad d = \text{depth of the beam}$$

$$F_{fu} = \frac{6.4\times 10^3}{140 - 8.6} = 48.7 \text{ kN}$$

7. Check shear yielding resistance of the extended portion of the four-bolt extended unstiffened end plate:

$$\frac{F_{fu}}{2} \langle \phi R_n = \phi 0.6 F_{yp} b_p t_p$$

$\phi = 0.9$

$b_p$ = width of the end plate

$$\frac{48.7}{2} = 24.35 \langle 0.9 \times 0.6 \times 235 \times 100 \times 6 / 1000 = 76.14 \longrightarrow \text{OK}$$

8. Check shear rupture resistance of the extended portion of the end plate

$$\frac{F_{fu}}{2} \langle \phi R_n = \phi 0.6 F_{up} A_n$$

$\phi = 0.75$

$F_{up}$ = minimum tensile strength of the end plate

$A_n$ = net area of the end plate

$$48.7/2 \langle (0.75 \times 0.6 \times 370 \times 504/1000) = 84$$

9. The bolt shear rupture strength of the connection is conservatively assumed to be provided by the bolts at compression flange, thus

$$V_u \langle \phi R_n = \phi n_b F_v A_b$$

$n_b$ = number of bolts at the compression flange

$F_v$ = nominal reduced shear strength of bolts

$$22.5 \langle 0.75 \times 4 \times (0.4 \times 800) \times 28.3/1000 = 27.14 \longrightarrow \text{OK}$$

10. Check bolt bearing / tear out failure of the end plate

$$V_u \langle \phi R_n = n_i \phi R_n + n_o \phi R_n$$

$n_i$ = number of inner bolts = 2

$n_o$ = number of outer bolts = 2

$R_n$ = $1.2 L_c t F_u \langle 2.4 d_b t F_u$

$L_c$ = clear distance, in the direction of force between the edge of hole and the edge of the adjacent hole or edge of the material

t = end plate thickness

$F_u$ = minimum tensile strength of end plate

$$22.5 \langle [(2 \times 0.75 \times (2.4 \times 6 \times 6 \times 370)) + (2 \times 0.75 \times (1.2 \times 11 \times 6 \times 370))] / 1000$$

OK.

The design steps for Test 4 beam which also utilizes an extended end plate splice connection is not listed here, since same splice connection with same geometric and material properties as summarized above is used in Test 4 except with an end plate thickness of 10 mm instead of a 6 mm end plate as is used in Test 2. Since in Test 4, the splice is located in a higher moment zone the design factored load which produces the design strength of the splice is 27 kN. The effect of increasing the end plate thickness is to increase the flexural yielding strength of the end plate, so again the design is for a thick end plate connection without prying forces considered.

## 2. Design of Test Setup – Flange and Web Plate Bolted Splice

In this section the recommended steps to design a flange and web plate bolted splice connection as per AISC LRFD Manual of Steel Construction, are summarized along with the design calculations performed in the design of Test 3 beam splice connection.

1. Determine the required connection design moment and design shear

From elastic analysis of the beam under 38 kN as shown in Section 4.1,

$$\mathrm{M}_u = 5.4 kN.m$$
$$V_u = 19 kN$$

Check the following limit states :

2. Flange Bolts Shear Rupture

$$M_u \langle \phi R_n = \phi n_b F_v A_b d$$

$n_b$ = number of bolts at one side of a flange plate
$F_v$ = nominal reduced shear strength of bolts (Use M6 8.8 bolts again)
$A_b$ = nominal gross area of bolt
d = beam depth

Assume $n_b$ as 6

$$5.4 \langle 0.75 \times 6 \times (0.4 \times 800) \times 28.3 \times 140 / 10^6 = 5.7 \longrightarrow \text{OK}$$

3. Tension Yielding of Flange Plate

$$F_{fu} = \frac{M_u}{d} \quad ; \quad F_{fu} \langle \phi R_n = \phi F_y A_g$$

$F_{fu} = \dfrac{5.4 \times 10^3}{140} = 38.6$ kN $\longrightarrow$ Factored beam flange force

Take dimensions of 170mm x 55 mm x 6 mm for the flange plate

$38.6 \langle 0.9 \times 235 \times 55 \times 6 / 1000 = 69.8 \longrightarrow$ OK

4. Tension Rupture of Flange Plate

$$F_{fu} \langle \phi R_n = \phi F_u A_n$$

$A_n = (55 \times 6) - (2 \times (6 + 2) \times 6) = 234 mm^2$

$38.6 \langle 0.75 \times 370 \times 234 / 1000 = 65 \longrightarrow$ OK

5. Flange Plate Bearing

$$F_{fu} \langle \phi R_n = n_i \phi R_n + n_o \phi R_n$$

$R_n = 1.2 L_c t F_u \langle 2.4 d_b t F_u$

$38.6 \langle [(4 \times 0.75 \times (2.4 \times 6 \times 6 \times 370)) + (2 \times 0.75 \times 1.2 \times 11 \times 6 \times 370)] / 1000 = 140$
OK

6. Beam Flange Bearing

Need not be checked since flange thickness is higher than flange plate thickness

7. Block Shear Rupture of Flange Plate

$F_{fu} \langle \phi R_n$

$R_n = 0.6 F_u A_{nv} + F_u A_{nt} \langle 0.6 F_y A_{gv} + F_u A_{nt}$

$R_n = 0.6 \times 370 \times 540 + 370 \times 102 \langle 0.6 \times 235 \times 780 + 370 \times 102 = 147 kN$

$38.6 \langle 0.75 \times 147 = 110 \longrightarrow$ OK

8. Compressive Strength of Flange Plate

Assume K=0.65 and l=40 mm

$$\frac{Kl}{r} = \frac{0.65 \times 40}{\sqrt{\frac{(55 \times 6^3)/12}{(55 \times 6)}}} = 1.25 \longrightarrow$$ ØF$_{cr}$=211 MPa from Table 3-36 of LRFD Manual

$\phi R_n = \phi F_{cr} A$

$\phi R_n = 211 \times 55 \times 6 / 1000 = 70 \rangle 38.6 \longrightarrow$ OK

9. Shear Rupture of Web Bolts

Web will carry shear and moment due to eccentricity of shear

Try 6 bolts in the web

$$\phi R_n = \phi F_{nv} A_b = \phi 0.4 F_u A_b = 0.75 \times 0.4 \times 800 \times 28.3 / 1000 = 6.8kN$$

$\downarrow$

Design shear rupture strength
of a single bolt

$V_u = \sqrt{(19/12 + 1.235)^2 + 2.47^2} = 3.74kN \langle 6.8kN$ $\longrightarrow$ OK

10. Shear Yielding of Web Plates

$$V_u \langle \phi R_n = \phi 0.6 F_y A_g m$$

Take dimensions of 120mm x 80 mm x 3 mm for the web plate

$19 \langle 0.9 \times 0.6 \times 235 \times 80 \times 3 \times 2 / 1000 = 60.9kN$ $\longrightarrow$ OK

11. Shear Rupture of Web Plates

$$V_u \langle \phi R_n = \phi 0.6 F_u A_{nv} m$$

$19 \langle 0.75 \times 0.6 \times 370 \times 168 \times 2 / 1000 = 55.9kN$ $\longrightarrow$ OK

12. Block Shear Rupture of Web Plate

$F_{fu} \langle \phi R_n$

$\mathrm{R_n} = 0.6F_u A_{nv} + F_u A_{nt} \langle 0.6F_y A_{gv} + F_u A_{nt}$

$R_n = 0.6 \times 370 \times 540 + 370 \times 102 \langle 0.6 \times 235 \times 780 + 370 \times 102 = 147kN$

$19 \langle 0.75 \times 147 \times 2 = 220 \longrightarrow$ OK

The design steps for Test 5 beam which also utilizes a flange and web plate bolted splice connection is not listed here, since same splice connection with same geometric and material properties as summarized above is used in Test 5 except with a flange plate thickness of 10 mm instead of a 6 mm flange plate as is used in Test 3. Since in Test 5, the splice is located in a higher moment zone the design factored load which produces the design strength of the splice is 23 kN.