Experimental Moment-Rotation Behavior of Semi-Rigid Beam-to-Column Connections

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ABSTRACT

An experimental research was accomplished to examine the rotational capability of extended end-plate connections and stiffened fin plate connections. Four extended end plates and four stiffened fin plate connections configurations were tested under monotonic loading histories suggested by SAC. In addition, the connections suitability for use in seismic resisting moment frames (SRMF) were examined for ductility to find that all the connections achieved more than 0.03 rad rotation which is minimum rotation capacity required by Eurocode for high ductility class structures. Experimental results for maximum moment resisted by the connections were compared with analytical predictions. All the tested connections except one, F200x200, achieved higher moments than the predicted moments.

The experimental results demonstrated that, the extended end plate moment connections can be detailed and designed by using the web plates and stiffeners in certain locations so as to be reliable for use in seismic regions. However, the stiffened fin plate connections have very high rotational capacity with very low moment capacities unless three or more bolts in a row is used.

Keywords: Connection tests, Moment-rotation curves, Moment frame, Extended end plate, Stiffened Fin plate, Monotonic loading.

ÖZ

Genişletilmiş uç plaka ve güçlendirilmiş fin plaka bağlantılarının dönme kapasitlerini ölçme amaçlı bir deneysel çalışma gerçekleştirilmiştir. Bu nedenle dört adet genişletilmiş uç plaka ve 4 adet de güçlendirilmiş fin plaka bağlantıları SAC tarafından önerilen şekilde monoton statik yükleme uygulayarak test edilmiştir. İlaveten, bu bağlantıların depreme dayanıklı rijit çerçevelerde kullanımının süneklik açısından uygunluğu incelenmiş ve tüm bağlantıların Eurocode tasarım standardında depreme dayanıklı ve yüksek süneklik sınıfında olan yapıların bağlantıları için öngörülen 0.03 rad minimum dönme kapasitesini aştığı saptanmıştır. Deneylerden elde edilen kolon-kiriş bağlantılarının taşıyabileceği maksimum moment değerleri analitik olarak tahmin edilen değerlerle kıyaslanmıştır. Buna göre deneyi yapılan tüm bağlantılar, biri dışında F200x200, analitik olarak hesaplanan moment değerlerinden daha yüksek moment taşıyabildiler.

Deneysel çalışmanın sonuçları genişletilmiş uç plakalı moment bağlantılarının detaylandırılıp gövde plakası ve bayraklar kullanılarak deprem bölgeleri için güvenilir hale getirilebileceğini göstermiştir. Fakat güçlendirilmiş fin plaka bağlantılarının dönme kapasitelerinin çok yüksek olması yanında moment kapasitelerinin de çok düşük olduğu gözlemlenmiş ve bağlantılarda her sırada en az 3 civata kullanılması durumunda bu bağlantıların da daha uygun olabileceği sonucuna varılmıştır.

Keywords: Bağlantı deneyleri, Moment-dönme eğrileri, Rijit çerçeve, Genişletilmiş uç plaka, Güçlendirilmiş fin plaka, Monoton statik yükleme

DEDICATION

I would like to dedicate all of the time spent, knowledge, hard work and effort on this work sincerely to ALLAH and hope he accepts it from me, I would like to thank my parents Assoc. Prof. Dr Nimir Salih, MS.Intisar Elbasher and my brother Dr.Abazer Ali my sisters Razan Ali and Nihad Ali and all my family and friends for their support and guidance.

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LIST OF SYMBOLS

M	Moment [kN-m]
Φ	Rotation [rad]
A	Inclination angle [deg]
h_c	Height of the column section [mm]
h_b	Height of the beam section [mm]
b_{fc}	Width of the column section [mm]
b_f	Width of the beam section [mm]
t_{wc}	Web Thickness of the column section [mm]
t_{wb}	Web Thickness of the beam section [mm]
t_{fc}	Flange Thickness of the column section [mm]
t_{fb}	Flange Thickness of the beam section [mm]
r_c	Radius of column section fillet [mm]
r_b	Radius of beam section fillet [mm]
A_c	Cross-sectional area of the column [mm^2]
A_b	Cross-sectional area of the beam [mm^2]
I_{xc}	Moment of inertia of the column section [mm^4]
I_{xb}	Moment of inertia of the beam section [mm^4]
$M_{j,Rd}$	Connection resistance for bending [kN-m]
$S_{j,ini}$	Initial rotational stiffness [kN-m/rad]
S_j	Final rotational stiffness [kN-m/rad]
$M_{pl,Rd,lim}$	Full resistance connection Strength [kN-m]

LIST OF ABBREVIATIONS

SRMF	Seismic Resistance Moment Frame
CWPS	Column Web Plate in Shear
BFWS	Beam Flange and Web in Shear
LVDT	Linear Variable Differential Transformers
ASTM	American Society for Testing and Materials

Chapter 1

INTRODUCTION

1.1 General

A broad experimental examination on the behavior and design of beam-column semi-rigid connections has been accomplished over the past decades. The investigation was started by the unpredicted failure of many welded beam-column connections in 1994 Northridge, California earthquake. A substantial number of study was financed through the SAC Joint Venture, which was separated into two phases. The preliminary phase aimed on identifying the source of welded connection failure. The second phase aimed on coming up with unconventional connections for use in steel moment resisting frames.

The extended end-plate moment connection is a substitute that has been considered throughout the second phase of the investigation. The study involves empirical, analytical modeling and experimental testing to define if the extended end-plate is suitable for the use as a moment resisting frames or not. [1]

1.2 Classification of Connection Types

If the structural system is planned to dissipate energy in the beams, the beam-column connections should be designed for the necessary level of over strength, to categorize whether the effects of connection performance on the analysis need be taken under consideration, as in Table 1.1 or a distinction may be made between three simplified joint models are as follows: [2]

- simple, in which the connection may be assumed not to transmit any bending moments;
- continuous, in which the performance of the joint may be assumed to have no effect on the analysis;
- Semi-continuous, in which the performance of the joint needs to be taken into account in the analysis.

Table 1.1. Classification of joints based on global analysis

Method of global analysis	Classification of joints		
Elastic	Nominally pinned	Rigid	Semi-rigid
Rigid-plastic	Nominally pinned	Full-strength	Partial-strength
Elastic-plastic	Nominally pinned	Rigid and full-strength	Semi-rigid and Partial-strength Semi-rigid and full-strength rigid and Partial-strength
Type of joint model	simple	continuous	Semi-continuous

1.3 Seismic Design Requirements

The seismic design specifications for steel resisting frames have considerably altered since 1994. The necessity provisions oblige that beam-column moment connections deliberately designed with satisfactory strength to force improvement of plastic hinges away from the column face to an approximately known location within the beam. Structures in seismic regions shall be designed and assembled with a sufficient degree of reliability under some requirements. [3]

- Fundamental requirements
  - No collapse requirement: the structure shall be intended to resist seismic action without local or global failure.
  - Damage limitation: the structure shall be intended to resist earthquake movement having the higher expectancy of happening.
- Compliance criteria
  - Ultimate limit states: are those associated with collapse or other forms of failure which might endanger the safety of people.
  - Damage limitation states: are those associated with damage beyond service requirements.

1.4 Significance of Research

After Northridge earthquake SAC carried out numerous research and produced reports on ductility levels of connections and how to test and use them. Since then many more research has been carried out around the world towards improving the post-Northridge connections. As a result of all these research there are pre-qualified connections with American steel sections, suggested for use in seismic regions. However, considering the vast number of connection types and steel sections produced by using standards from different countries, there is still need to increase the number of pre-qualified connections both in USA, Europe and in the rest of world. This significance of this research is as such that the test results reported in this thesis and possible parametric work results that can be carried out in future using these test results will help in introducing new pre-qualified connections for use in European seismic regions. In order to achieve pre-qualified connections there is need for experimental test results on rotational capacity and classification of the connections using Eurocode. This approach is a must to achieve efficiency, cost effectiveness and safety for structures.

1.5 Research Objectives

The first study objective was to conclude the reliability of extended end plate as semi-rigid connection and stiffened fin plate as pinned connection for use moment frames in seismic regions.

The prime purpose of this investigation is to examine four extended end plate and four stiffened fin plate configurations shown in Figure1.1 and Figure 1.2 under monotonic loading through failure to define the rotational capability of these connections. Experimental connection strength is compared to the expected strengths. The actual test moment-rotation curves deliver the origin for categorizing connections.

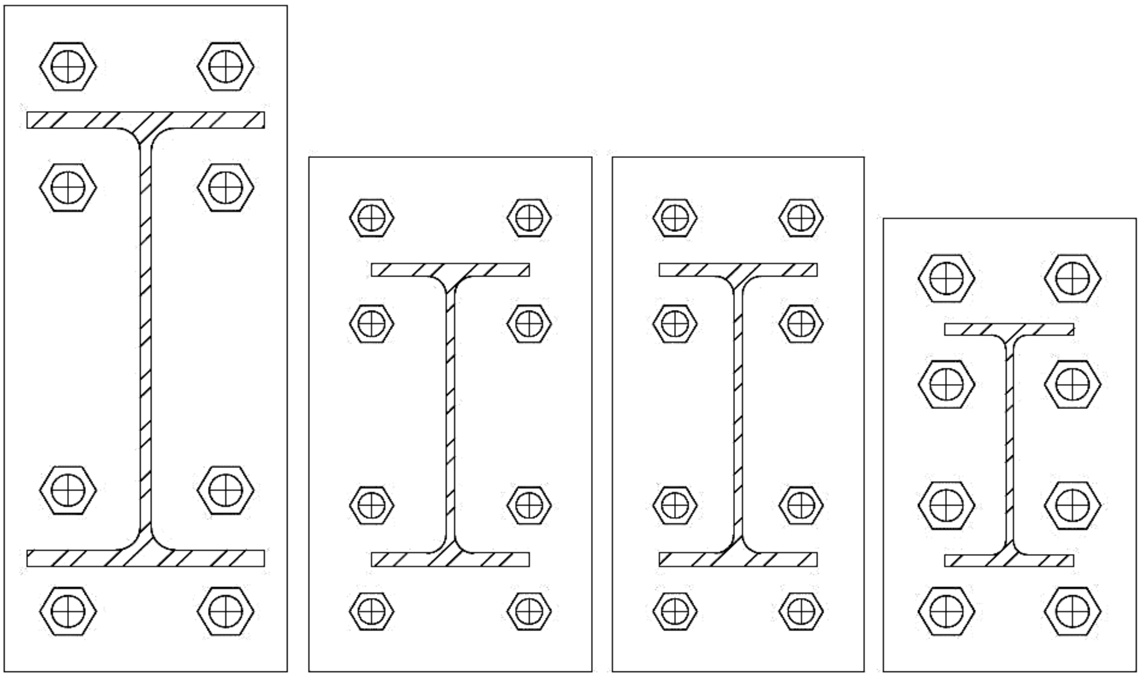


Figure 1.1. Extended end plate configurations

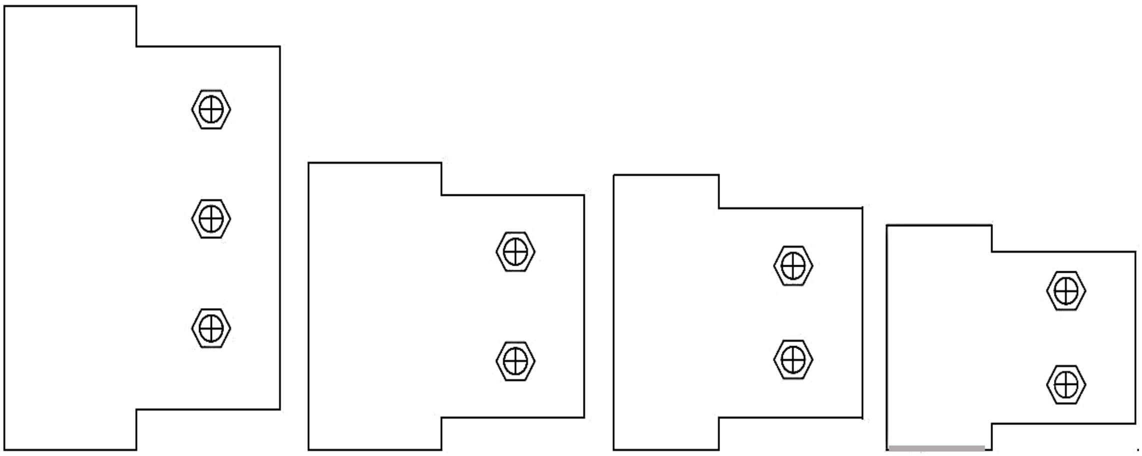


Figure 1.2. Fin plate configurations

Literature review is revised in Chapter 2. The experimental program and testing results are discussed in Chapters 3 and 4. Chapter 5 contains the conclusion and the design proposals.

Chapter 2

LITERATURE REVIEW

2.1 Introduction

A steel frame is a flat structural system made up of linear members, such as beams and columns, linked together by connections. From an economic point of view, the expenses for design and manufacture covers the major part of the total costs of the structure. From the designing point of view, mainly the connection parameters have a huge effect on the reactions of the structure. Especially, in the design and analysis of steel structures, it is presumed that all structural beam-to-column connections behave either as:

i. Simply pinned, which designates that, no moment will be transferred between the beam and the column and therefore the connection is only capable of transferring shear and axial force. As far as rotation is concerned, the column and the beam that are linked together by a pin will behave separately.

ii. Fully rigid, which specifies no relative rotation will follow between the attached members and the beam end-moment is transferred to the column. The angle between the beam and the column stays unaffected as the frame deforms.

Current techniques of analysis and design of steel structures still subject to these basic unrealistic models, regardless of the fact that, it has been acknowledged for more than a century that the joint behavior lies someplace between the two extreme idealizations.

Researches have been done during the last few decades revealed that the performance of real bolted connections is neither rigid nor pinned; rather, they hold some degree of rotational limitation which depends on the sort of joints used. so The term "Semi-rigid" is used to describe such connections [4].

2.1.1 Origin and Background of Steel Frames with Semi-Rigid Connections

Beam-to-column connections are essential element in steel structures and have been a focus of research since the 19th century. In their first experimental research conducted in 1917, Wilson and Moore who defined the rigidity of riveted joints in steel frames. In the 1930's, more testing of beam-to-column connections were carried out in United Kingdom, Canada and the United States. Ever since, it has been required to recognize the performance and behavior of the connections and its influence on the whole structure. Since then, many tests on riveted, bolted and welded connections have been carried out.

From the early studies into joint performance it was recognized that, economy was attainable by lowering the moment and deflection at the midpoint of a beam. Hence, smaller sections can be used with the same applied forces which lead to less cost in production. This is in contrast to the use of rigid connections which are unpopular due to being costly to fabricate [5].

The effect of semi-rigid end restraint on column behavior has only been investigated in recent years by Marino and De Falco. An alteration of the relative stiffness's of beams with semi-rigid connections was used by Driscoll in defining effective lengths of columns Since then, many researches have been done. As a result of this approach, there is a possible economy in column design since effective lengths can be decreased due to the restraints provided by the semi-rigid connections.

For beam-to-column connections, the most valuable feature to describe is the relationship of moment to rotation, M-Φ. In order to include the influence of semi-rigid connection behavior in any kind of frame analysis, an understanding of the M-Φ response or at least, the capacity to estimate the initial stiffness, ultimate moment capacity and deformation of the M-Φ curve effectively is a must.

2.1.2 Types of Steel Frames

Steel frame is a structural system with a "skeleton frame" of horizontal beams and vertical columns, erected as much as possible in a rectangular grid to lift the floors, roof and walls of a building which are all fixed to the frame. The development of this procedure made the creation of the high-rise buildings possible.

The structural method of the skyscrapers is designed to handle lateral loads produced by wind or seismic forces and the vertical gravity loads. The structural design involves the members that were designed to carry the loads only, and the remaining members are stated as non-structural [5].

A building stability is a critical design element for structures. Structures typically rely on two main systems for lateral stability

- Rigid frame
- Braced frame and shear-walled frame

2.1.2.1 Rigid frame

Moment-resisting frames are straight-lined assemblies of beams and columns, with the beams rigidly linked to the columns via rigid connections, as shown in Figure 2.1.

Lateral forces resistance is mostly result of rigid frame action through transfer of shear forces and bending moments from one frame member to another via the rigid joints. In moment frames lateral forces are resisted by bending in beams and columns. Therefore, ductile connections are needed for the beam-to-column connections to resist the forces and moments without fracture. Therefore, the bending strength and rigidity of the structural members is the major cause of lateral stiffness and strength for the entire structure.

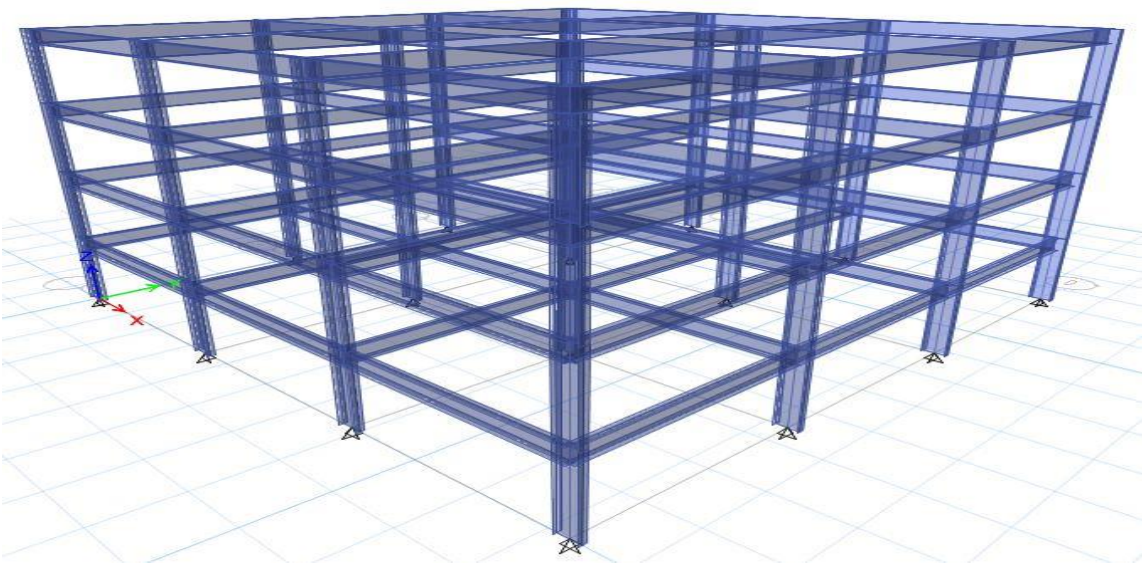


Figure 2.1. Moment frame system

2.1.2.2 Braced frame

Braced frame is a structural arrangement which is designed mainly to resist gravity, wind and earthquake forces. Bracing members are mainly designed to resist tension and compression as in the case of a truss. Bracing members are made of steel. An alternative method to bracing members is the use of shear walls to resist the lateral forces as shown in Figure 2.2, and the building has to be symmetrical to avoid lateral torsional buckling.

Figure 2.2. Braced frame and shear wall systems

2.2 Semi-Rigid Connections

2.2.1 Connection Classifications

Significant parameters of steel are its stiffness, strength and deformability. These factors can be validated in a tensile test, see Figure 2.3. A well designed steel structure should have the best of those factors [9].

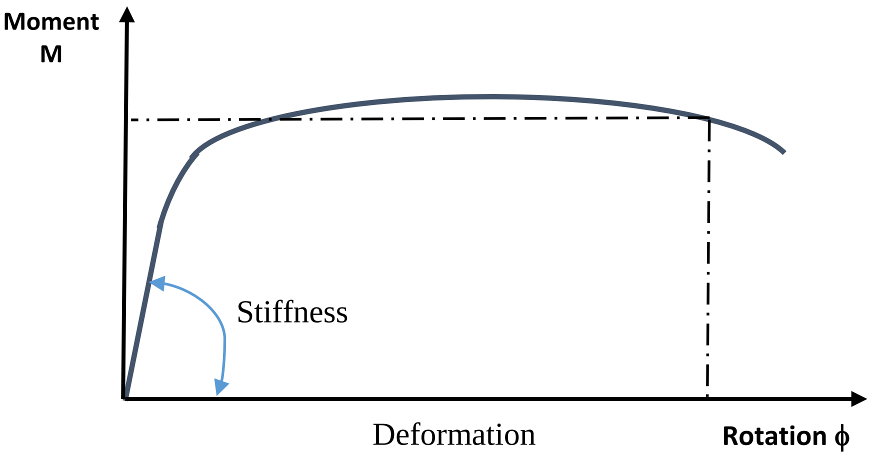


Figure 2.3. Moment-rotation diagram

Connections are categorized by stiffness as, rigid, semi-rigid and pinned, as in Figure 2.4. by strength as full strength, partial strength or nominally pinned. The stiffness characteristic is significant for elastic analysis of frames; where the strength characteristic of frames is analyzed plastically. Depending on the stiffness and strength requirement of connection by the design, the standard suggests joint models as simple, semi-continuous and continuous [10].

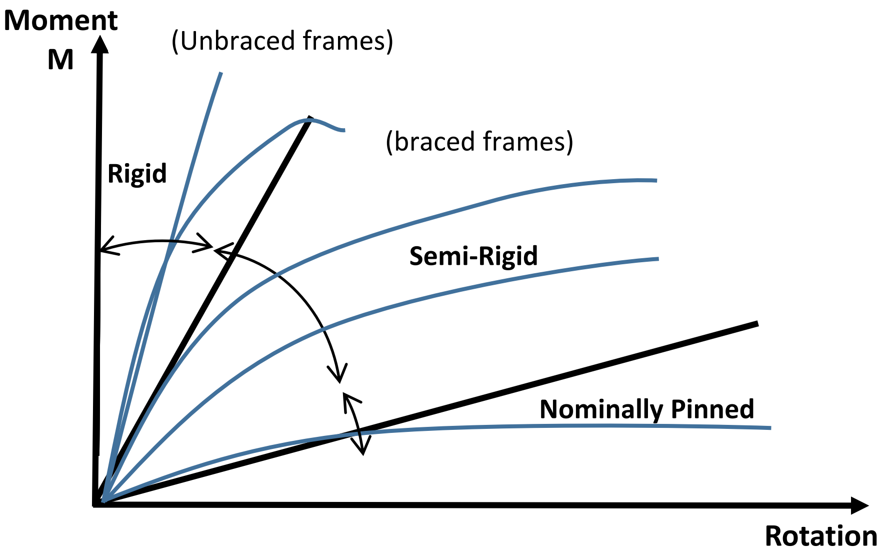


Figure 2.4. Moment-rotation curve showing connection classifications by stiffness

2.2.1.1 Strength

To estimate the joint forces, analysis must contain the purpose of the design loads on the structure, see Figure 2.5. When representing the structure in an organize way, the stiffness of the joint is an essential element. Connections can be presumed as rigid, pinned or having stiffness somewhere between the two. The deformability of the sections and of the joints has an essential part in the final delivery of the forces, such classification behaves either as:

i. Nominally pinned connections, where the connection can transfer the forces suggested in the design, without evolving substantial moments which could undesirably have an effect on members of the structure.

ii. Full-strength connections, in which the design endurance of a full strength connection should be at least equivalent to that of the member joined.
iii. Partial-strength connections, in which the design endurance of the connection may be less than that of the members attached [11].

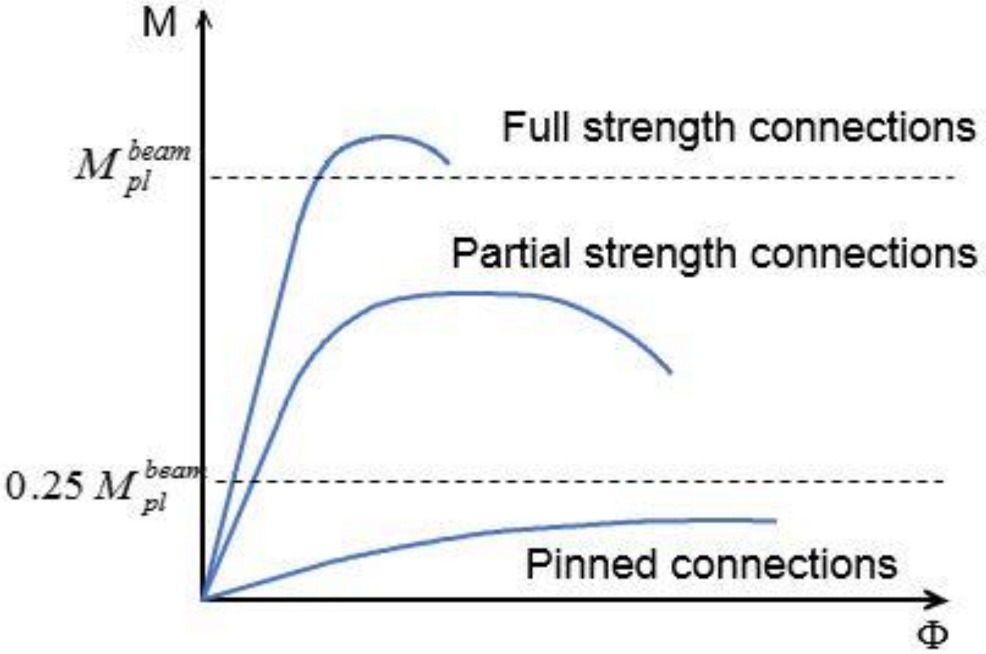


Figure 2.5. Moment-rotation curve showing connection classifications by strength [11]

2.2.1.2 Stiffness

The stiffness of the joint influences the loading capability of conveying loads. A joint of low rotational stiffness cannot handle high bending moments and for that reason it might be presumed as a pinned connection, of course, the stiffness of a joint can influence beam deformation. Particularly, in non-braced frames, the stiffness can have a big influence on the stability and deflection of the building.

If the joints are rigid in the structure, then accordingly the connections should be considered that their deformations have an insignificant impact on the load distribution and the distortions of the structure. Connections can be classified as showed in Figure 2.6 as:

i. Nominally pinned joints, must be able of transferring the internal loads without evolving substantial moments, which can influence the entire structure.

ii. Rigid joints, expected to have adequate rotational stiffness to validate the analysis.

iii. Semi-rigid joints, does not meet the criteria for nominally pinned joint or rigid joint should be categorized as a semi-rigid joint.

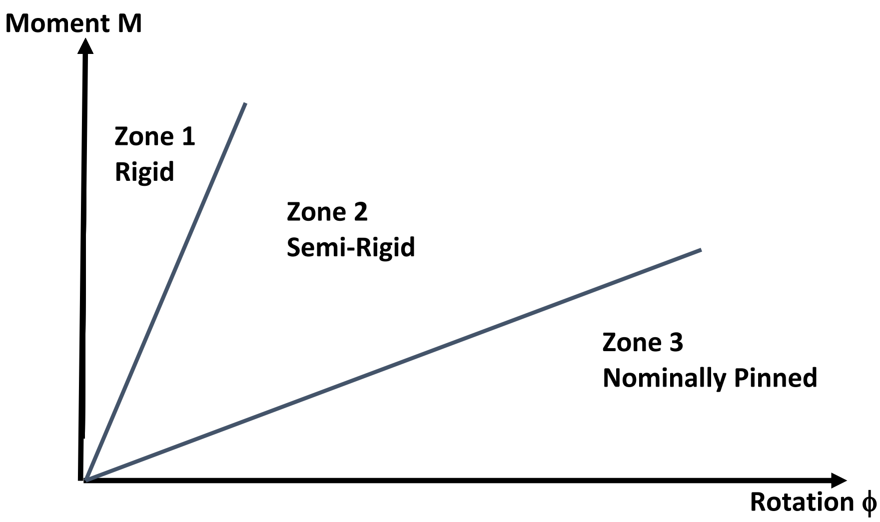


Figure 2.6. Classification of Stiffness [11]

Zone 1: Rigid, if $S_{j,ini} \geq K_b \, EI_b / L_b$

Zone 2: Semi-rigid $K_b \, EI_b / L_b \leq S_{j,ini} \leq 0.5 \, EI_b / L_b$

Zone 3: Nominally pinned, if $S_{j,ini} \leq 0.5 \, EI_b / L_b$

2.2.2 Welding and Bolting of Connections

Connections are like glue that held the structural steel members together. Several types are used depends upon the type of connecting elements, magnitude and nature of the forces (bolting, welding) [12].

2.2.2.1 Welding of Connection

Advantages:

1. Eliminates the need for drilling or punching
2. Simplifies complicated connections

Disadvantages:

1. Require greater level of skill
2. Expensive
3. Weld inspection is required and is expensive
4. In earthquake areas it may be more prone to fatigue

2.2.2.2 Bolting of Connection

Advantages:

1. Easy to joining members on site
2. bolting is cheaper than welding in Field

Disadvantages:

1. Requires punching or drilling in members

2.2.3 Extended End Plate Connection

Bolted extended end-plate connections have been a conventional form of linking beams to columns in rigid-frame construction. Joints can be designed to fulfill the essential measures of stiffness, strength and rotation capacity, yet still remain cost-effective and easy to produce. In such connections, the end-plate can be extended either on the tension side only or on both sides. Due to providing balance the second option is favored when the frame is exposed to reversal of loadings, such as earthquake or wind loading. Both forms of the extended end-plate can be either with or without column web stiffeners; these are used to stop the flexural deformation of the column flange. Different level of moment-transfer is expected from these connections. For an unstiffened assembly, normally the flush end-plate joint conveys almost 30% and the extended end-plate connection transfers about 60% of the beam's yield moment.

Because of the additional row of tension bolts above the beam flange, the active lever arm of the corresponding tensile force will be improved in contrast to the flush end-plate connections and accordingly better joint stiffness and strength can be reached. As the active lever arm to the bolts rises the end-plate gets extended and the rotation normally occurs about the beam compression flange. This leads to smaller bolt sizes with a lower resultant tensile forces and compressive forces on the column. Thinner end-plates are normally related with this preparation, resulting in the extended end-plate system being commonly favored in the industry.

The extended end-plate is realized to be far stiffer at all load degrees and holds a critical moment of about 1.6 times larger than the flush end-plate, regardless of bolt preload. Originally, the moment-rotation curve is nearly linear, followed by a large knee, during which significant unstiffening of the connection occurs, leading to a last phase of about 5 to 10 per cent of its initial stiffness. The shape and the extension of the curve in the later range influenced by the connection part which mostly affected by yielding.

2.2.4 Stiffened Fin Plate Connection

Fin plate is a common pinned connection type used worldwide, it consists of one set or more of bolts along the beam web with single or double plates. The bolts eccentricity from the column flange is what makes this connection type a simple connection, the more the eccentricity increase the more the connection gets weaker. Fin plate connections develop their rotational capacity from the whole distortion within the fin plate as well as beam web and from the shear deformation of the bolts.

Because of this weakness to moments, fin plate connection has been modified with stiffeners, welded to bottom and top of the plate to resist the moment. Those two stiffeners are welded to the column web and flanges. The rotational capacity and stiffness of this modified connection has not been acquired yet.

2.3 Connection Behavior

2.3.1 Moment-Rotation Characteristic (Ductility)

To assess a connection stiffness or strength a connection may be symbolized by a spring linking the center lines at the intersection point. The spring properties can be stated as a design moment-rotational characteristic that defines the association between bending moment $M_{j,ed}$ on the connection and the resultant rotation ϕ_{ed} among the members, Figure 2.7 shows the structural properties:

i) Design moment resistance $M_{j,rd}$, which is equal to maximum design moment-rotation characteristic

ii) Rotational stiffness S_j, which is the secant stiffness specified in Figure 2.7 as the rotation ϕ_{xd} at which $M_{j,\,ed}$ first reaches $M_{j,\,rd}$, but not for larger rotation. While the initial rotational stiffness $S_{j,ini}$ is the slope of the elastic range of the design moment-rotational characteristic

$$S_j = \frac{EZ^2}{\mu \sum \frac{1}{k}}$$

iii) Rotation capacity ϕ_{cd}, which is equal to the maximum rotation of the design moment-rotation characteristic.

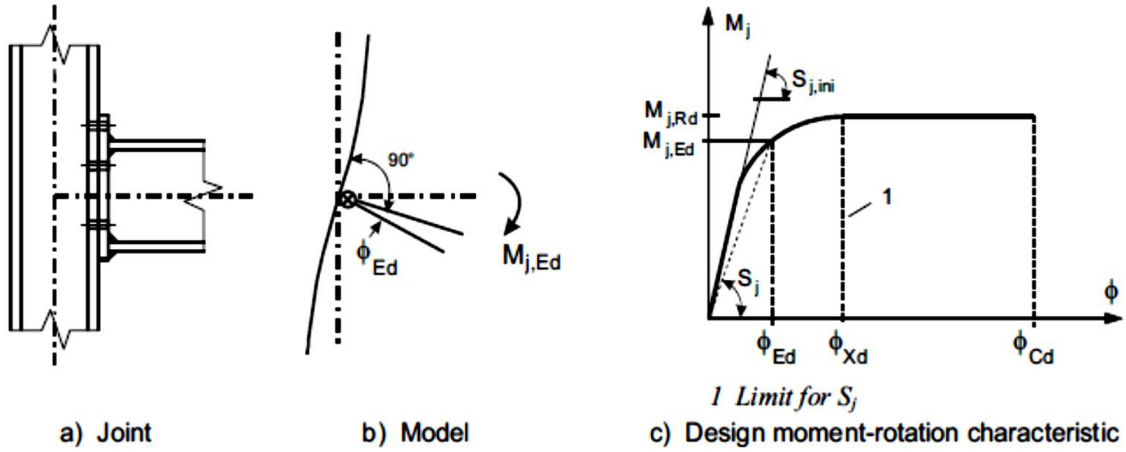


Figure 2.7. Moment-rotation properties [11]

Classic moment-rotation M-ϕ curves for the most frequently used beam-to-column connections are shown in Figure 2.8. In this graph, the vertical axis (M) symbolizes the ideally rigid connection and the horizontal axis (ϕ) symbolizes the ideally pinned connection. Real connections fall between the two axes within the quadrant. The nearer the curve is to the horizontal axis, the more flexible is the connection whilst those which approach to the vertical axis are perform almost as a rigid joint. The definition of some of the characteristics of semi-rigid connections can be found below.

i. The moment-rotation curve is distinct as a curve stating the moment, M. conveyed by the connection, from beam to column, as a role of the relative rotation ϕ of the two members linked by the connection.

ii. In the case of the rigid connection, under the application of load, both the column and the end of the beam will rotate through an angle θ. However, in the semi-rigid connection, the column will rotate through an angle θ while the beam rotates through an angle,φ. The relative rotation, $\varphi - \theta = \phi$

is distinct as the connection rotation. Therefore, care must be taken to guarantee that true connection rotation values are measured in experiments when relating experimental moment-rotation curves obtained from different sources.

iii. Connection moment capacity, M_c, is equivalent to the ultimate value of the moment-rotation characteristic.

iv. The rotation capacity, ϕ_c, of a beam-column connection shall be taken as the rotation realized at the moment resistance of the connection.

v. Each connection has its own moment-rotation distinctive curve and the slope, k, of the M-ϕ curve is a measure of the rigidity or stiffness of the connection at any specific value of rotation. A typical moment-rotation curve is shown in Figure 2.8.

All familiar forms of beam-to-column connection reveal non-linear M-ϕ behavior over nearly the whole range of loading and the connection stiffness drops as rotational deformation rises.

The sorting of a connection's rigidity relies on connection type and limitations. As quoted before, all connections in structural steelwork may be categorized as rigid, semi-rigid or pinned, this is a suitable categorizing from both the structural frame analysis and the connection design point of views. The vital distinctive that determines the classification is the moment-rotation characteristic [3].

In Eurocode 3 the classification of beam-column connections is founded on either the moment resistance or on the rotational stiffness of the connection. The categorizing with respect to rotational stiffness is given in a non-dimensionalised M-ϕ diagram.

Connections which are classified as rigid or nominally pinned may freely be treated as semi-rigid.

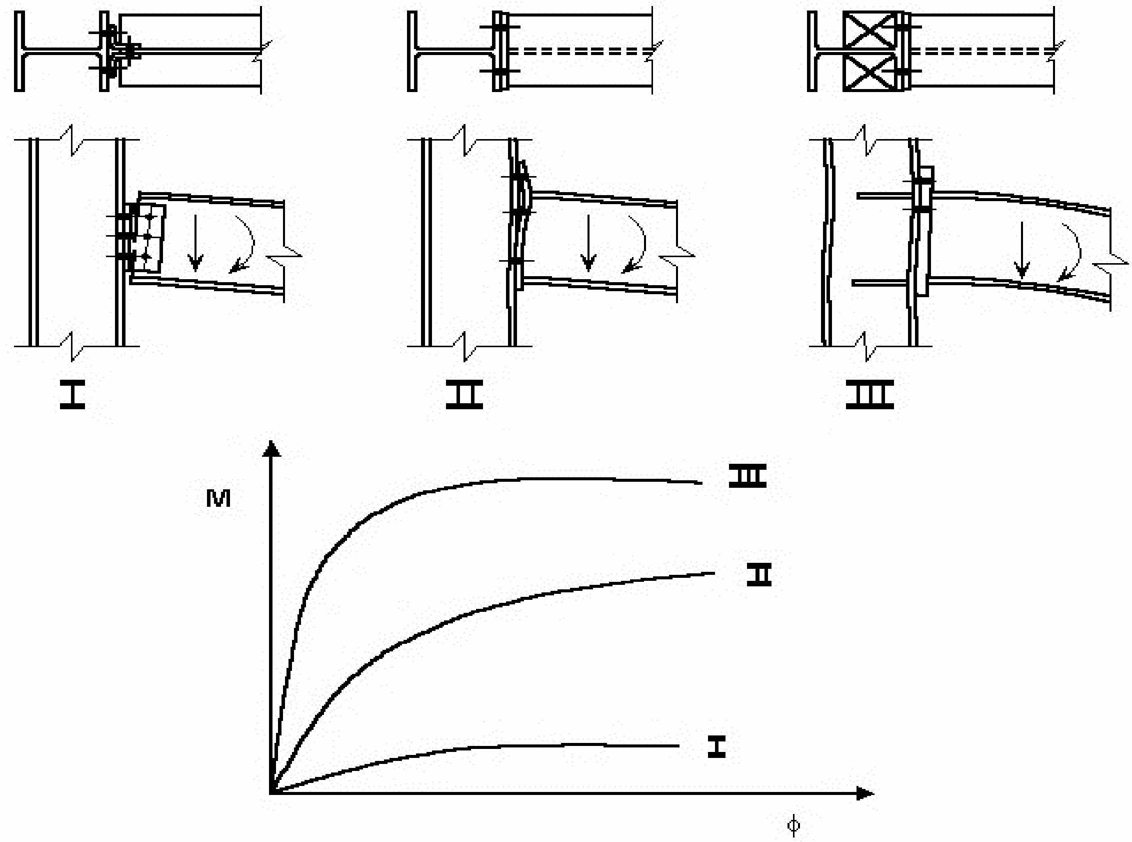


Figure 2.8. Typical moment-rotation curves [9]

The characteristics of the moment-rotation, M-ϕ, given in Figure 2.8 can be defined as follows:

I. The moment resistance and the rotational stiffness are insignificant and may rationally be ignored, leading to the impression of a pinned connection. It is essential to convey Vertical shear, with the beam end reaction being equivalent to resist the beam loading.

II. Behavior in-between M-ϕ **I** and **III**, the joint have a moment capacity lower than the plastic resistance of the beam and a rotational stiffness that allows for some partial rotation. Yet it is possible to have joints that are fully-rigid but only partial strength or full-strength and semi-rigid.)

III. The rotational stiffness is great and the connection's plastic resistance is at least that of the beam. Accordingly, stability is conserved with no relative beam- column rotation.

All three types of connection classifications can be useful in multi-story buildings. Type **III** can be used in unbraced and braced structures; Type **I** is just appropriate for braced systems. Type **II** can be used for unbraced and braced frames, but in the final situation the impact of M-ϕ on structural performance has to be measured.

For everyday use, it’s essential to overstate the M-ϕ curves. For design purposes a non-linear idealization or linear idealizations, like the bi-linear or tri-linear M-ϕ relationship can be used (Figure 2.9).

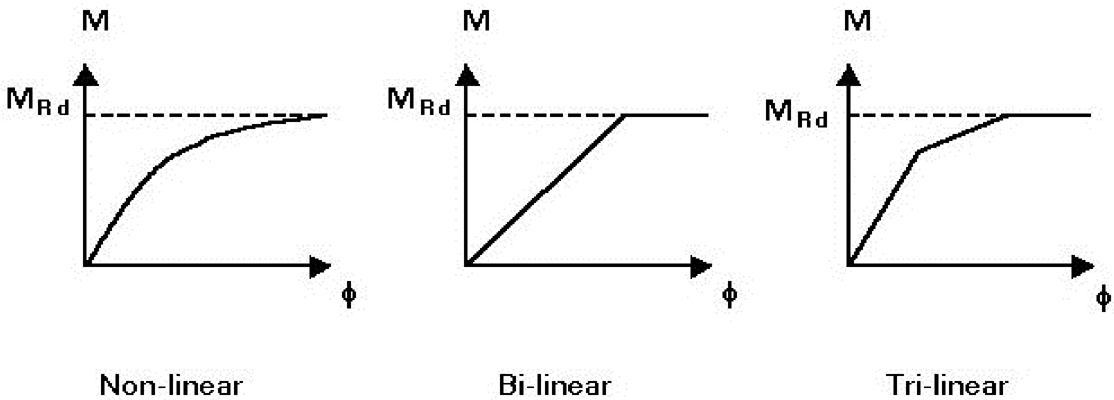


Figure 2.9. Idealization of moment-rotation curves [9]

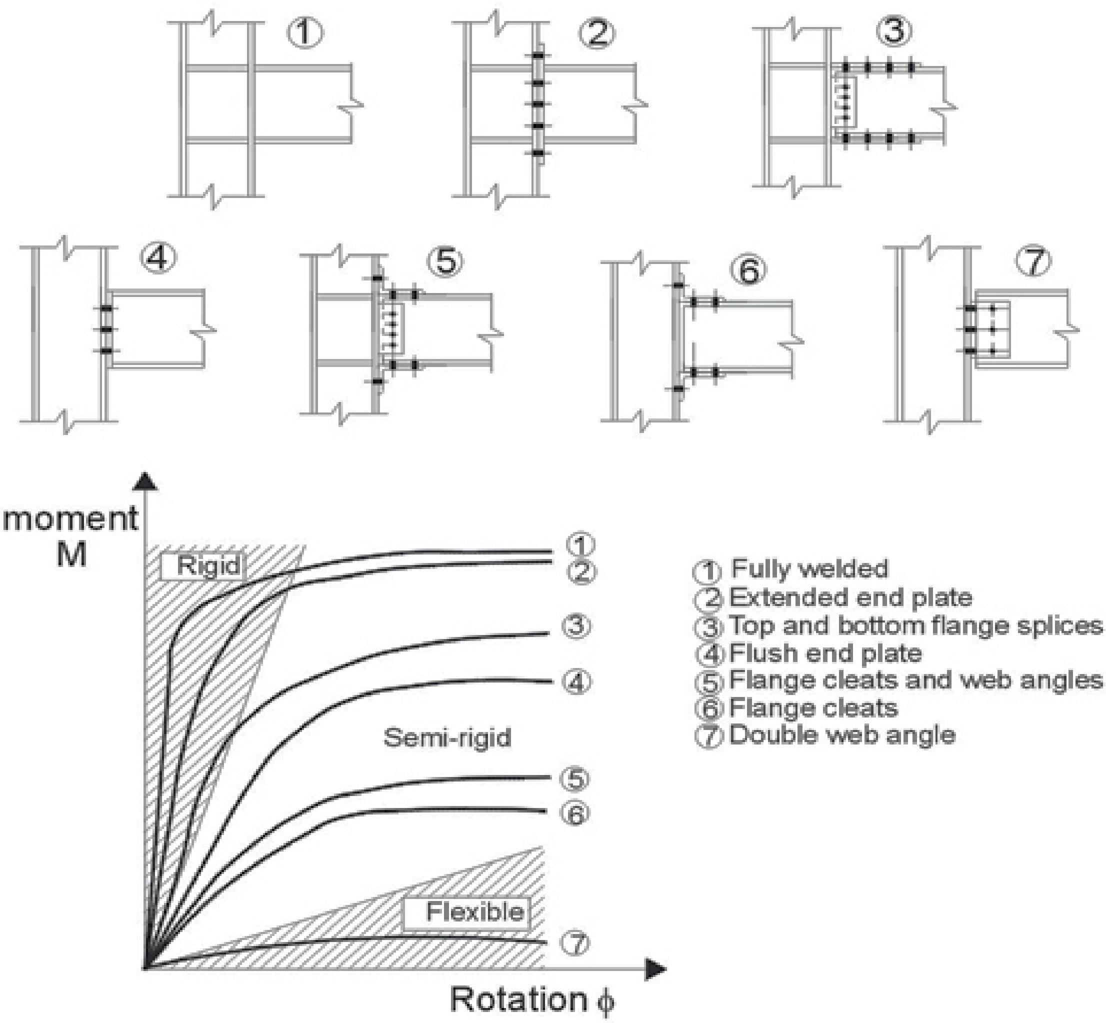


Figure 2.10. Moment rotation curves for common connections [13]

2.3.2 Modelling of the Moment – Rotation Curves

The main difficulty with the analysis of semi-rigid connections is the great number of design variables. Each connection has its own specific moment-rotation curve which is subject to geometrical size, material properties, bolt sizes, types and locations, bolt pretension etc.

A change of one factor will lead to a change of moment-rotation curve and characteristic. This curve must be either acquired experimentally or analytically, before one can continue to the analysis of semi-rigid steel frames. The creation of moment-rotation curves from real tests is time-consuming and expensive to cover the complete applied range for all types of semi-rigid connections but still necessary. However, the other economical alternative is to develop an M-ϕ estimated formula which can replicate the performance of the semi-rigid connection established on a database covering a sufficient range of values or based on mechanical models.

Several researchers have developed their own exceptional models that were used to define the nonlinear behavior of their own connection investigations. Generally, all the research into modelling of moment-rotation curves falls into one of the following categories:

i) Empirical curve fitting model:

   This is based on collecting data on moment-rotation curves of connections and engaging in curve fitting and regression procedures to develop estimate formulas. This technique normally uses size and shape limitations. Curve fitting practices are normally working to transform experimental data into curves which are identified algebraically. Such procedures have been used frequently over the last two decades. These powerful approaches of curve fitting of experimental data are appropriate to represent at high precision level all the range of connection behavior applied interest.

ii) Analytical models:

   These semi-empirical models (analytical models) are established in two isolated stages. Firstly, the basic models for the main parameters defining the M-ϕ curve, such as initial elastic stiffness, major sources of connection, ultimate moment capacity deformation etc.,. In the second phase, a curve fitting procedure is used to develop the full M-ϕ relationship.

iii) Mechanical models:

Unlike the previous defined models produced purely (i) or partially (ii) from empirical curve fitting, the mechanical models are based on the thought of the deformation behavior of the connection components. Mechanical models with comprehensive interaction obtainable between the separate parts creating the connection are very difficult to develop. So far, such method rarely used by a very few investigators for connections with rather simple physical behavior.

iv) Finite element models:

Lately the finite element technique has been used to study the global behavior of semi-rigid connections, in which elastic, elastic-plastic and collapse behavior of the joint can be realized. Such models are very challenging, complex and require the modelling of a very large number of parameters and conditions such as: various components of the members and of the connection, nonlinear element material capability, bolt behavior, weld representation, preload and hole clearance, interaction forces, initial imperfections, etc.

2.4 Review of Previous Research

Thirty specimens modeled in ABAQUS and tested with monotonic loading to understand the behavior of semi-rigid/partial-strength I beam to tubular column connection via reverse channel connection (RCC). The importance of this research was on the strength, stiffness, rotational capacity, sources of deformability and failure mechanisms of the. This research was concluded that the channel wall thickness to the flush end-plate thickness ratio is critical factor affecting the rotational capacity of

RCC. RCC has confirmed itself as a reliable connection in terms of ductility and rotational capacity and to be more than satisfactory for full plastic design [14].

A series of circular holes introduced in the panel zone of a column to upgrade the total plastic rotation capacity of post-Northridge connection. Analytical results showed that an optimum ratio of 0.81 to increase the plastic rotation characteristic by increasing the contribution of the panel zone [15].

New analytical model to estimate the moment-rotation behavior for stiffened and extended beam-column end plate connections has been developed in this. Founded on an exact description of the end-plate connection rotation, the end-plate connection is divided into its components, including the column flange, end-plate, panel zone and bolt. The deformation development of each component is then analyzed. To conclude, the deformation development for the whole connection is acquired by overlaying the behavior of each component [16].

Eight full sized beam-column end-plate connection samples were examined under cyclic loading, to investigate the end-plate thickness, end plate extended stiffener, column stiffener. The test outcomes show that extended end-plate connections have acceptable ductility, rotational stiffness, strength and energy dissipation capacity necessary for use in earthquake regions [17].

An attempt to simulate the effect of beam in a coupled girder moment–resisting system under horizontal loading, through the use of four moment connections tested under cyclic loading with vertical and horizontal displacements. After the plastic rotation exceed 0.03, the connection responds in a ductile manner without major descent in capacity, and the attendance of horizontal stiffener aided in conveying the axial forces and delayed deformation of the web local buckling [18]

This research studies the effect of semi-rigid performance of the connections in finite element analysis of steel frame to acquire the real beam-column connections behavior in steel frame, presuming that resistance of members and the loads are variables, then the Monte Carlo replication procedure is used for finest prediction of chance of failure. The total reliability of the total system is illustrated by the significance of the outcome of semi-rigid behavior [19].

The development of the structural analysis methods for a steel frame under statically applied loads have been studied, using advance analysis methods considering connection behavior and its influence of the overall response of frames [20].

The plastic moment strength of connecting beams often is larger than that expected. The amplified beam strength must be measured in joint design [21].

Weld access holes are not needed for end-plate connections. Stress concentrations in the flange are possible reasons for rupture to occur [22].

Five full scale specimens were tested to failure, three tested statically and two tested dynamically to report the outcome of strain rate on the brittle fracture of steel moment frame connection after Northridge. It was found that dynamically loaded connections have lower plastic rotation and energy dissipation capacities, and the failure mode is in the column flange not like statically loaded connections which indicated that the level of strain rate increased the force demand in the beam flange [23].

Shims were be used between the end-plate and column flanges to reduce prying forces on the bolt [24].

Energy dissipation could be affected by Yielding of Panel zone (PZ), inelastic deformation of the PZ can be controlled by the end-plate aiding (Ghobarah et al., 1992. Analytical results in a study done by Hedayat and Celikag in 2008 showed that the presence of a circular hole with the right diameter to height ratio in PZ can increase the influence of the PZ leading to increase in the total plastic rotation of the connection by up to 10% [25,26].

Design techniques for monotonic loading are not enough for seismic forces. Explicitly, the end-plate and bolts must be planned for more than the plastic capability of the beam to avoid loss of end-plate strength [27].

Connections intended for less than the beam capability may not offer enough ductility [28].

Chapter 3

TESTING PROGRAM

3.1 Overview

The necessity for further experimental study into the performance of extended end plate moment connection has been realized by the literature review. eight beam-column connections tests were performed as part of this study. Four of the tests done were beam-column extended end plate connections and four were stiffened fin plate connections all under monotonic loading. The aim of the experiments is to examine the rotation capacity and the total strength of the connection configurations and to identify if stiffened fin plate as pin connections and extended end plate as moment connections were proper for the use in SRMF. The aim of testing the stiffened fin plate was to examine the behavior and strength to see its suitability for braced frames in seismic regions.

3.2 Prototype Model

The structure used as a prototype in this research is a two story residential building. The 3D model, plan and elevation are shown in Figure 3.1, 3.2 and 3.3 respectively. The prototype was a residential building with two by two bays, moment in one direction and braced in the other direction designed by using ETABS [40]. The structure modeled in this study is designed by using EUROCODE where frame ductility class high moment resisting frame (DCH MRF) on one side and a braced frame on the other, in a frames were designed for earthquake loading. Moment connections and the members are designed to undergo significant deformation while dissipating energy. Connections in the frames must be ductile enough to deform with the lateral force resisting system without attracting huge forces. In the laterally restrained frame the horizontal forces are resisted by the bracing sections. Therefore, all the connections were intended to carry gravity loads only with pinned connection, such as, stiffened fin plate [29].

The sections used were European sections as they are widely available in N.Cyprus. Grade S275JR hot-rolled I-sections with 275 MPa yield strength. [30].

Structural parameters were taken from Eurocode BS EN 1993-1-1 for general requirement, BS EN 1993-1-8 for design of connections, BS EN 1993-1 for rules for earthquake resistant structures and UK National Annex for Eurocode 3.

The design of columns and beams were taken from the model that was the result of a linear analysis for the structure subjected to 5 kN/m^2 dead load, 2.5 kN/m^2 live load on floors and 6 kN/m super dead load on walls as gravity loads. Seismic forces with a stratigraphic profile of Nicosia with sand, sandstone, gravels, limestone and silts, with the absence of deep geological effects Type 1 spectra have been used with soil factor of 1.2 for ground type B [31,32,33,34]. In this study the following assumptions were included in the design:

- No axial force was acting on the column. In reality, the column was exposed to axial force due to the dead and live loads and also due to the lateral seismic forces. In general, the axial force helps develop the performance of the joint and joint shear strength.

- The floor system was not modeled in these tests. In the prototype building, the flooring system was considered to be composite with galvanized metal deck and shear studs. Slab was spanning in one way using secondary beams between the perimeter frames and interior frames as shown in Figure 3.1.

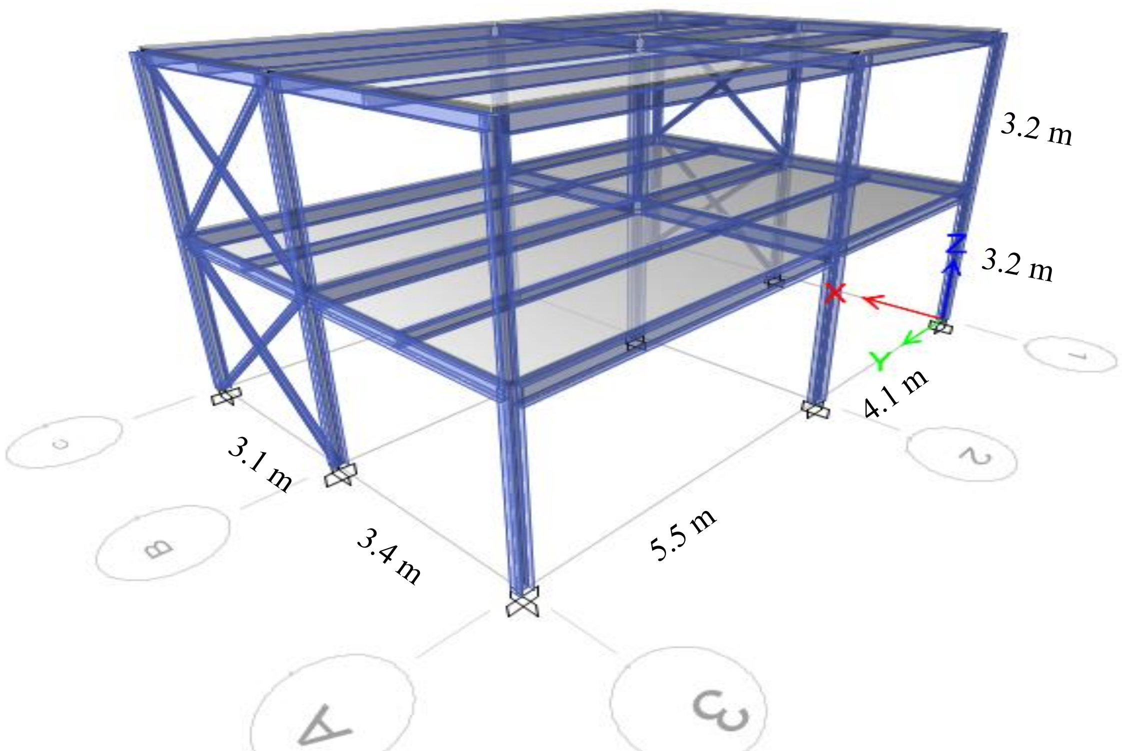


Figure 3.1. 3D view of the prototype structure

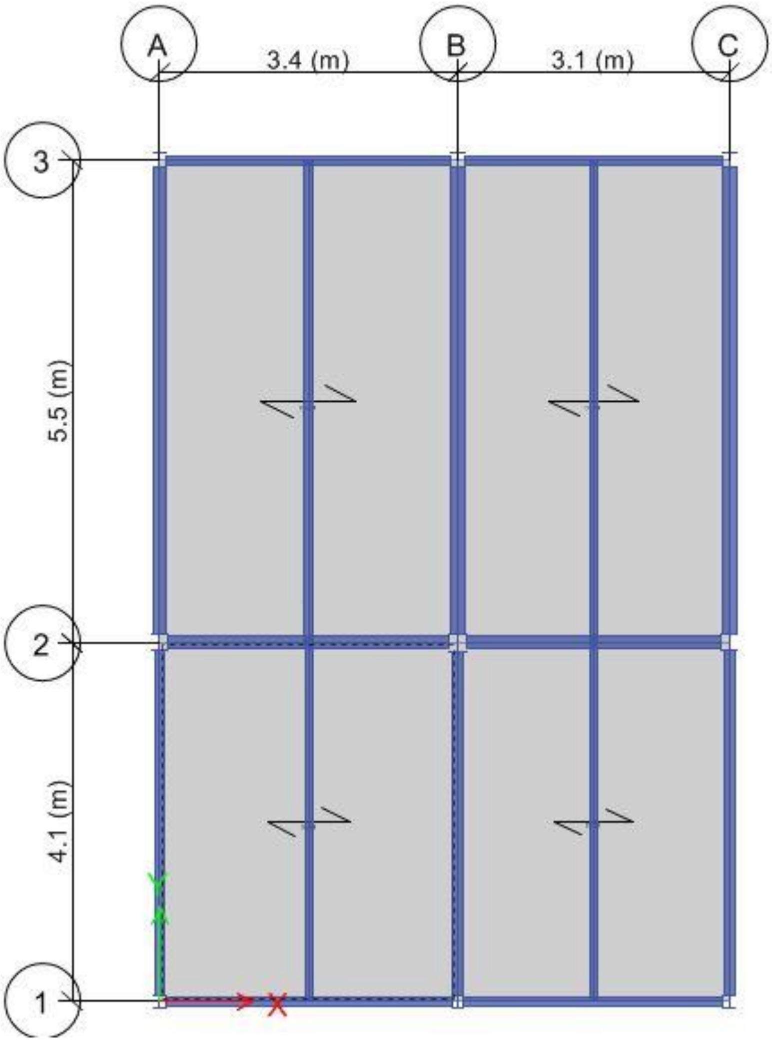


Figure 3.2. Floor plan of the prototype structure

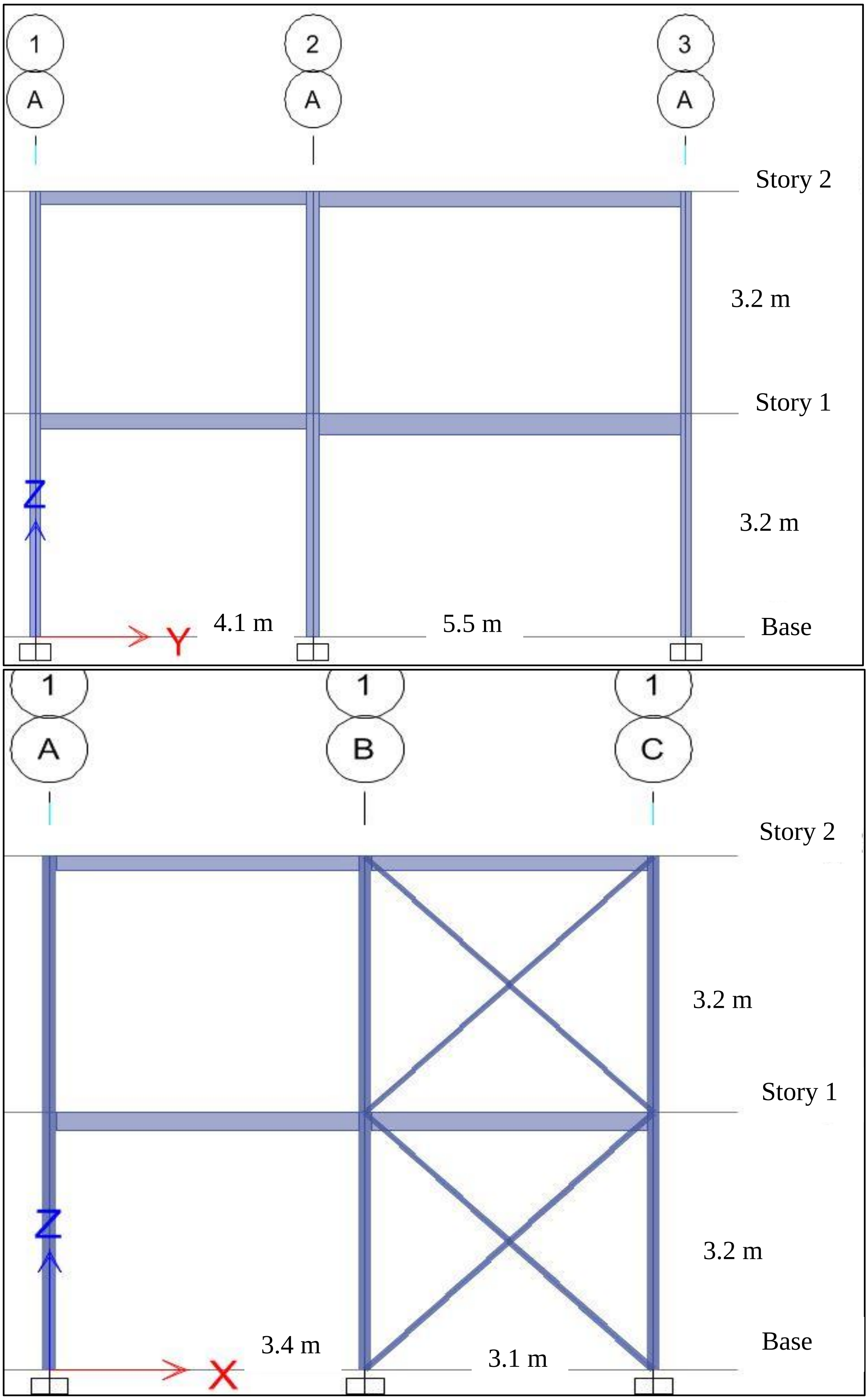


Figure 3.3. Elevations of the prototype structure

3.3 Test Specimens

Eight cantilevered form connection tests were conducted in this research using monotonic loading. The tests were sponsored by Eastern Mediterranean University. Four of the beam-column connection tests were extended end plate and the other four stiffened fin plate connections with different beam-column combinations. HE200B column with IPE 300 beam, HE200B column with IPE 200 beam, HE160B column with IPE 200 beam, HE160B column with IPE 160 beam were used, all the beams are 1200 mm and all columns are 1300 mm in length, M16 and M20 diameter grade 8.8 non preloaded bolts were used for connections as in Table 3.1.

The test specimen matrix is shown in Table 3.1. The connection geometric parameters shown in Table 3.2 are defined in Figure 3.4. Specimen ID for connections is a combination of the beam and column names and the symbols used for extended end plate (F) and for stiffened fin plate is (W). The test specimens were structural steel grade S275 for beams, columns, extended end plate and fin plate. Stiffeners were grade S235 and grade 8.8 bolt of M16 and M20 non preloaded hexagon bolts were used [35].

The connections design was done using the sections and forces from the model by taking the most critical joints in the moment frame and the braced frame. Then using Autodesk Robot Structural Analysis Professional to complete the connections design [41].

Table 3.1. Connection test specimen matrix

Specimen ID	Column	Beam	No. of bolts and diameter	Plate Thickness (mm)
F200x300	HE 200B	IPE 300	8 M20 grade 8.8	15
F200x200	HE 200B	IPE 200	8 M16 grade 8.8	10
F160x200	HE 160B	IPE 200	8 M16 grade 8.8	10
F160x160	HE 160B	IPE 160	8 M20 grade 8.8	15
W200x300	HE 200B	IPE 300	3 M16 grade 8.8	10
W200x200	HE 200B	IPE 200	2 M16 grade 8.8	10
W160x200	HE 160B	IPE 200	2 M16 grade 8.8	10
W160x160	HE 160B	IPE 160	2 M16 grade 8.8	10

Different sizes of fillet welds were used for the beam-column connections of web, flange and stiffeners of the test specimens.

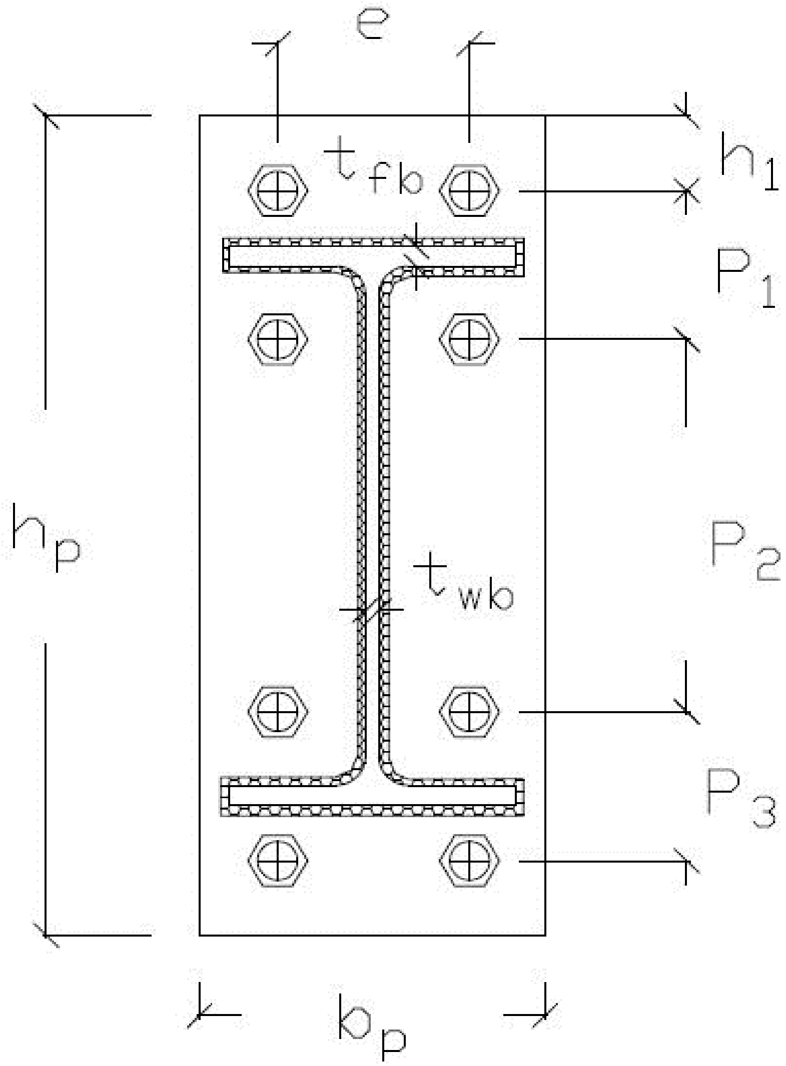


Figure 3.4. Notations for connection geometry

Table 3.2. Nominal connection geometric parameters

Test	Column	Beam	No. of Bolts	Connection Plate Dimensions (mm)									
				h_p	b_p	t_p	t_{fb}	t_{wb}	h_1	p_1	p_2	p_3	e
F(200x300)	HE 200B	IPE 300	8	440	180	15	11	7	40	80	200	80	100
F(200x200)	HE 200B	IPE 200	8	340	180	10	9	6	40	70	120	70	100
F(160x200)	HE 160B	IPE 200	8	340	160	10	9	6	40	70	120	70	80
F(160x160)	HE 160B	IPE 160	8	300	160	15	7	5	40	70	80	70	80
W(200x300)	HE 200B	IPE 300	3	220	100	10	11	7	40	70	70		
W(200x200)	HE 200B	IPE 200	2	150	100	10	9	6	40	70			
W(160x200)	HE 160B	IPE 200	2	130	100	10	9	6	35	60			
W(160x160)	HE 160B	IPE 160	2	130	100	10	7	5	35	60			

3.4 Test Setup

Connection tests can be done in a vertical or horizontal configuration. Structures laboratory set up allows a better use of the vertical position as described in Figure 3.5. The connections were tested under loading at the tip of the beam. The test specimen was free from one end and connected to the column at the other end. The column on the other hand was fixed to the base in two places from the bottom flange in the extended end plate connection and fixed from both flanges in stiffened fin plate connection as shown in Figure 3.6 and 3.7 respectively. Loading was applied using horizontal hydraulic jack supported from the reaction wall. The test specimens were supported by stiffeners under the loading point, at the fixation area with the base and at the panel zone to prevent any local buckling that could happen to the beam or to the column. A sample test setup is shown in Figure 3.5.

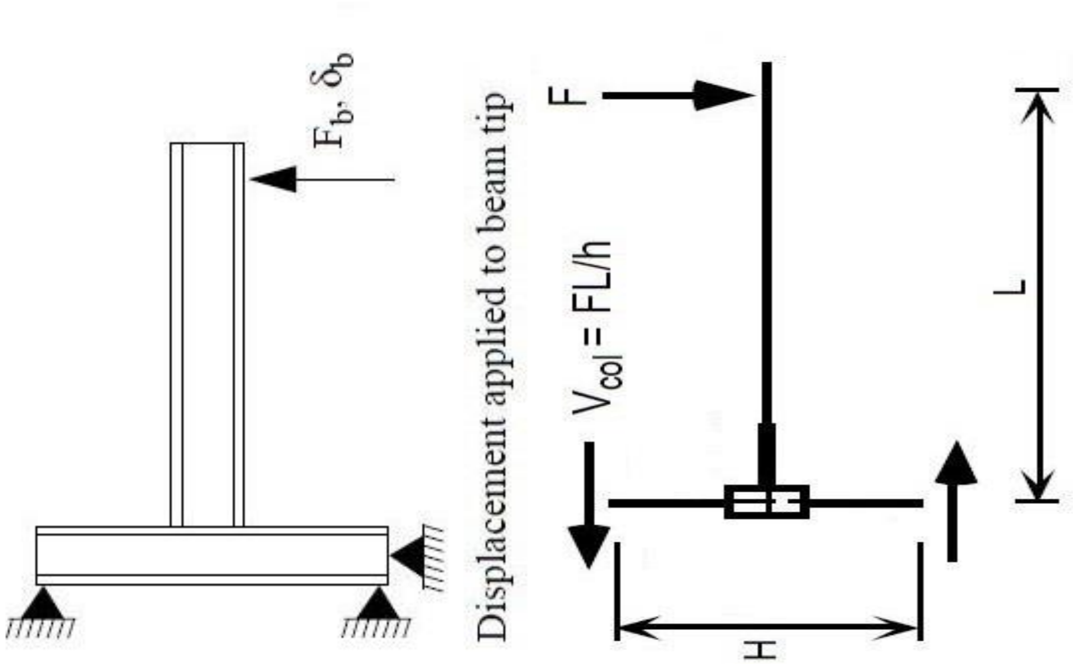


Figure 3.5. Test setup

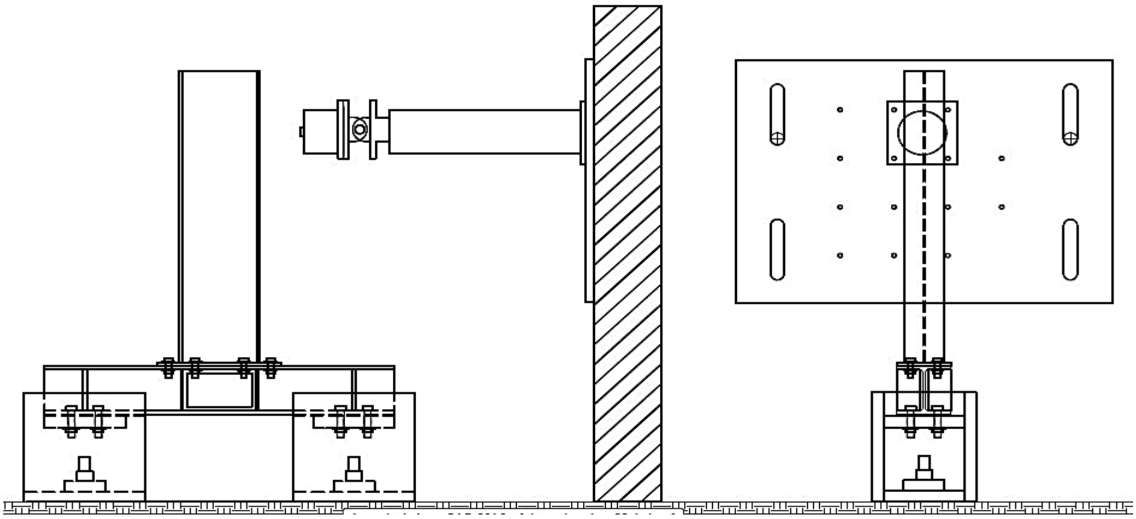


Figure 3.6. Extended end plate connection test configuration

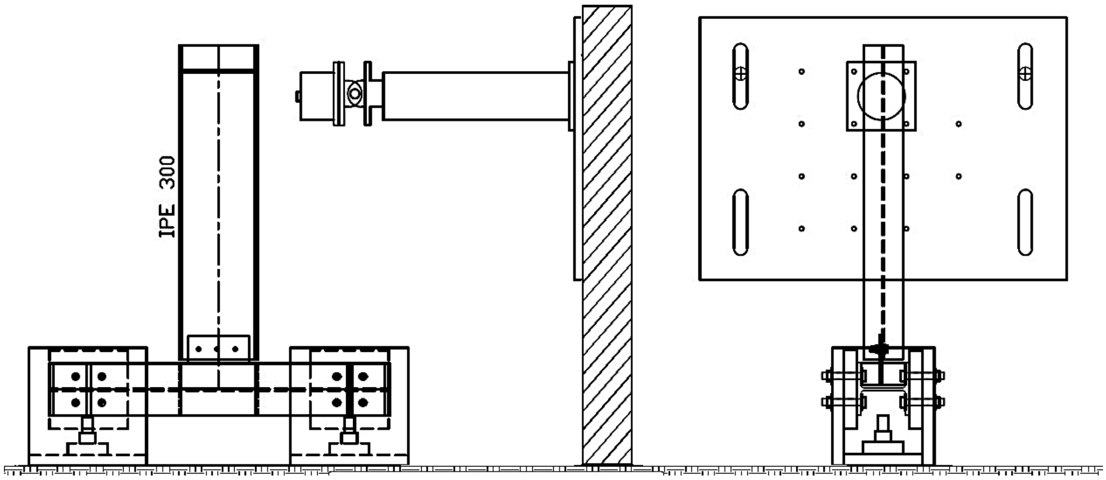


Figure 3.7. Stiffened fin plate connection test configuration

The loading applied on the beam for connection test specimens was monitored by load cell and Linear Variable Differential Transformers (LVDT) were used to measure the displacements at various locations on the beam and column. The locations of LVDTs are given in Figure 3.8. These LVDTs were calibrated according to TDS-303 portable data logger by inputting the right calibration coefficients before use and then linked to a data acquirement system throughout loading [35]. The nominal geometry for extended end plate and stiffened fin plate connections tested are shown in Figure 3.9 and 3.10 respectively.

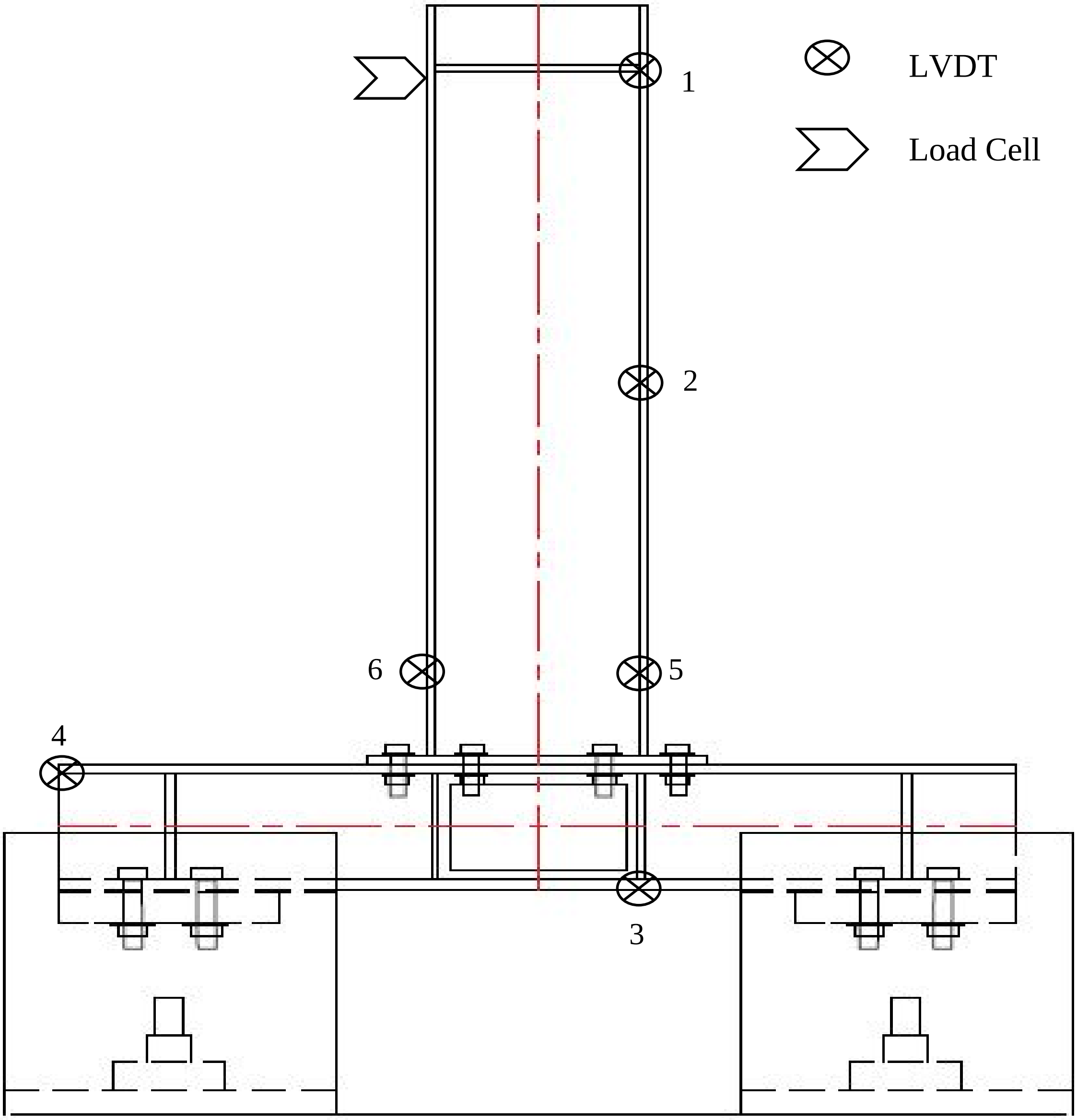
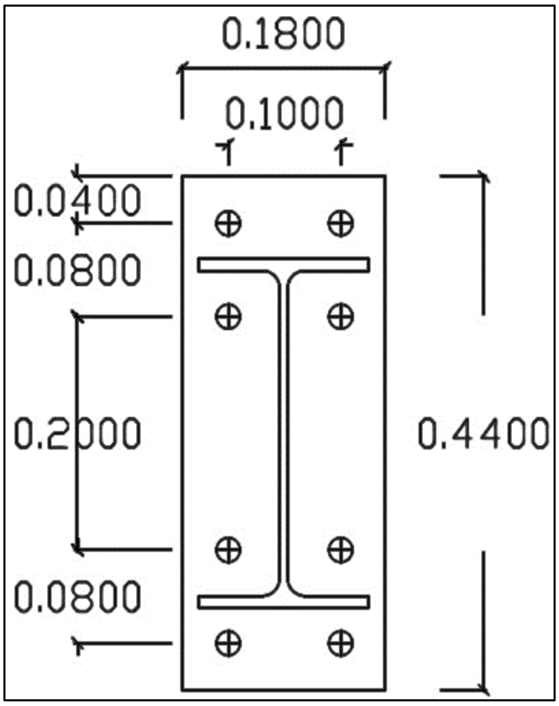
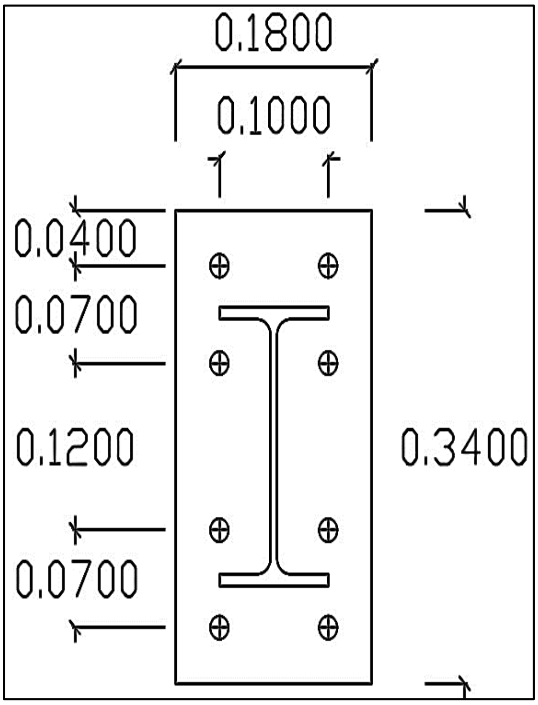


Figure 3.8. Locations for LVDTs and load cell

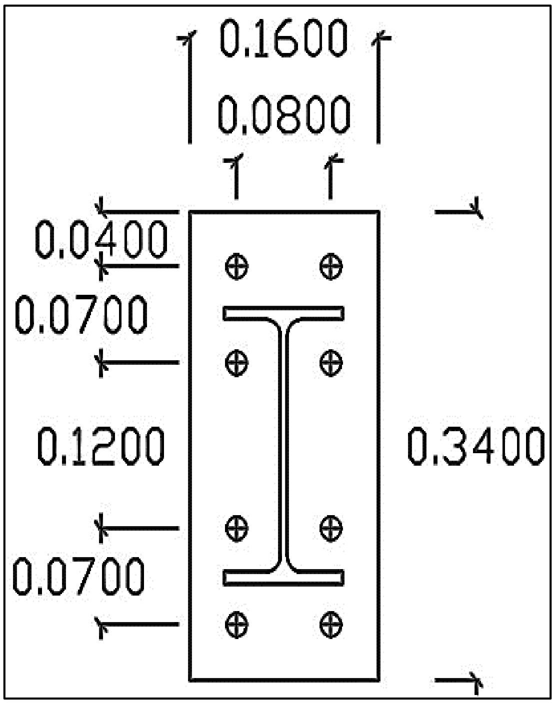
Connections are classified as major axis (Extended end plate) where the beam is connected to the column flange with higher flexural bending moment, and minor axis (Stiffened fin plate) where the beam is connected to the column web. For example, in order to calculate the connection rotation at LVDT 2, divide the displacement of LVDT 2 by the distance of LVDT 2 from the face of the column flange for major axis, and for minor axis divide the displacement of LVDT 2 by the distance of LVDT 2 from the face of the column web.



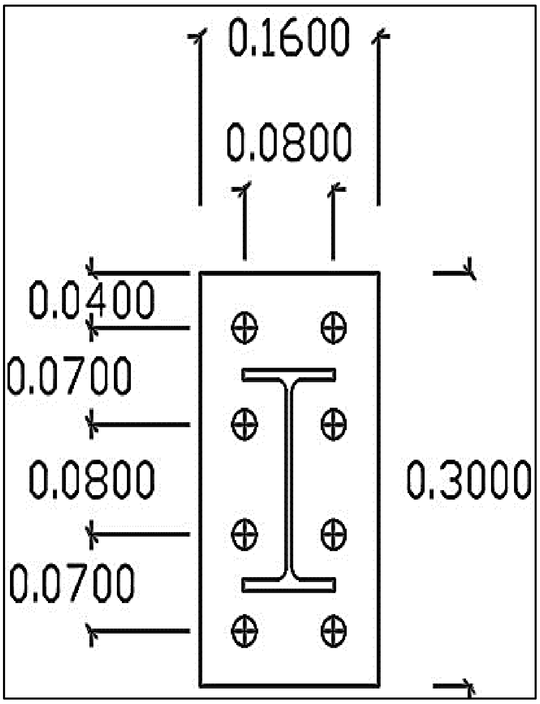
F 200x300



F 200X200



F 160X200



F 160X160

Figure 3.9. Extended end plate dimensional details

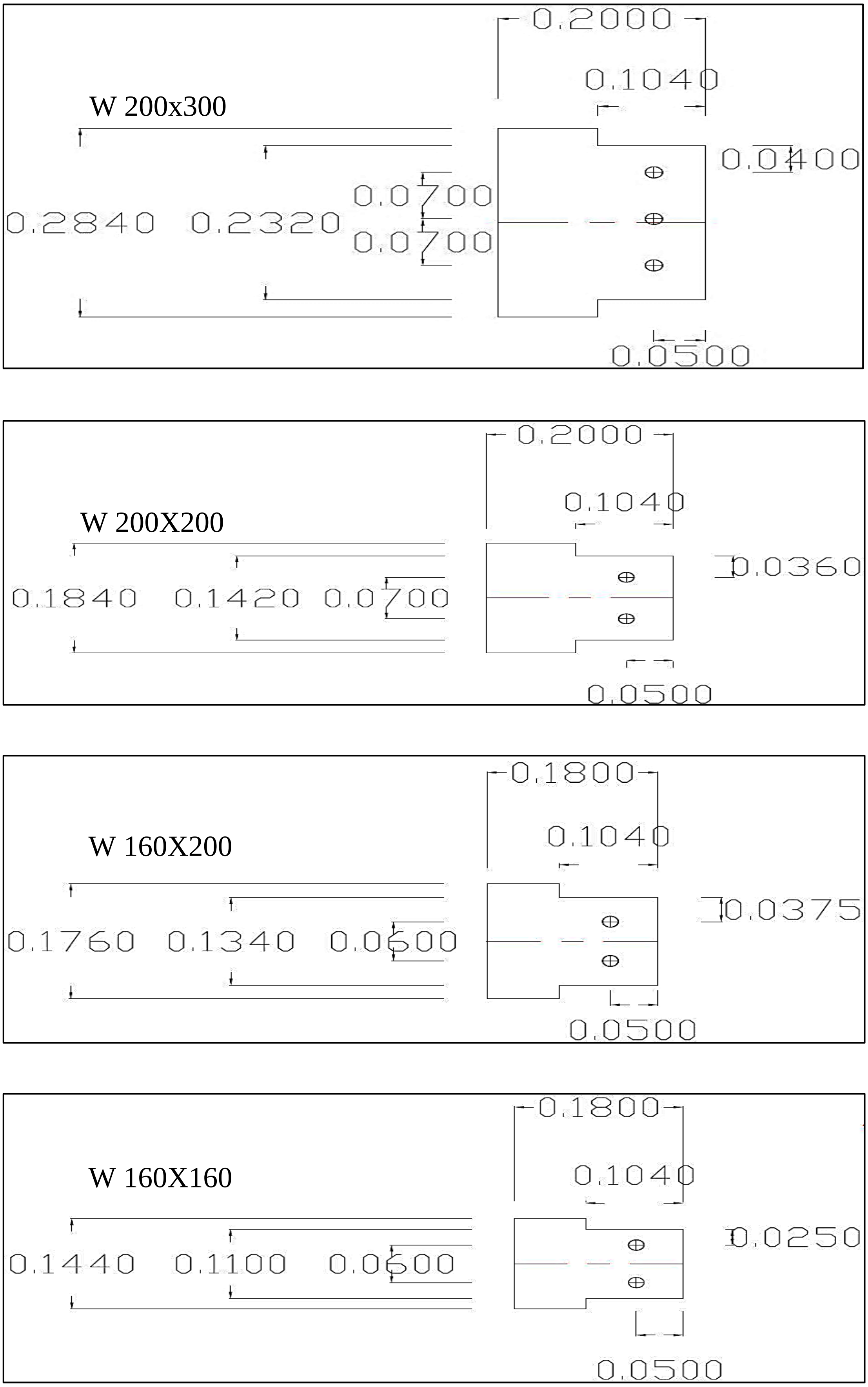


Figure 3.10. Stiffened fin plate dimensional details

3.5 Test Procedure

When the test specimen was setup in place, the sensors were calibrated and installed, the instrumentation were connected to a data logger. The hydraulic pressure transducers have been conducted using the loading protocol as the jack keeps loading the specimen as it started to yield, specified by levelling of the load rotation axis. The loading was continued up until any kind of failure can be clearly seen, as outlined in SAC/BD-97/2 Appendix E [36,37].

Tensile coupon tests were accompanied the specimen's components. The mean of two standard coupon specimens was taken from the web and flanges of each column and beam as shown in Appendix A. The specimens were arranged in agreement to ASTM A370 "Standard test methods and definitions for mechanical testing of steel products" and tested in accordance with Appendix D "Guidelines for tension testing of rolled shapes used in SAC testing program" of SAC protocol (SAC, 1997) [38,39].

Chapter 4

EXPERIMENTAL RESULTS AND DISCUSSION

4.1 General

The elements used for the assessment of the specimen behavior were the highest moment applied and the highest resultant beam to column rotation angle. The moment applied was acquired by multiplying the load by the distance of loading point from the face of the column. For a classic frame, the rotation is acquired by dividing the upper story displacement by the story height. For the tested assemblies, the rotation was acquired by dividing the loading point displacement by the distance to the column centerline for the minor axis tests and to the face of the column flange in the major axis tests.

Material properties from coupon tests can be found in Appendix A. Detailed summary sheets for all tests can be found in Appendices B and C. The test summaries include all the data gathered. Appendix D contains a connection design sample using robot structural.

4.2 Extended End Plate Test Results

4.2.1 Test F200x300

Figure 4.1 shows that, the final failure mode was the compression flange failure with a noticeable bending in the plate due to prying force that was working as a pull up force in the compression flange. The maximum applied load to the beam at connection failure was 181.4 kN. This maximum load gave a maximum bending moment of 197.7 kN-m (181.4x1.09 m) in the connection, where 1.09 m is the distance from the loading point to the face of plate connection. Figures 4.2 show the deformed shape of the connection and the condition of the bolts. The beam flanges and web has experienced yield mechanism before fracture.

Figure 4.1. Test F200x300 failure mode

Figure 4.2. Bolts condition

Figure 4.3 illustrates the recorded moment-rotation curve; the rotation was calculated as reading from LVDT2 divided by the length 615 mm. This is established on the hypothesis that, the beam deformed in a straight line and the vertical deformations of the column and the base movement were insignificant relative to the deformation of the beam as illustrated in Appendix B. The maximum applied bending moment was 197.7 kN-m at a rotation of 0.072 rad for LVDT 1 and 0.052 rad for LVDT 2, as shown in Figure 4.3. The difference in rotation under same loading was due to torsional behavior of the section. The fluctuation at 0.006 rad was due to a glitch in between the load cell and the beam. It’s clearly that the connection has reached the maximum capacity and started failing.

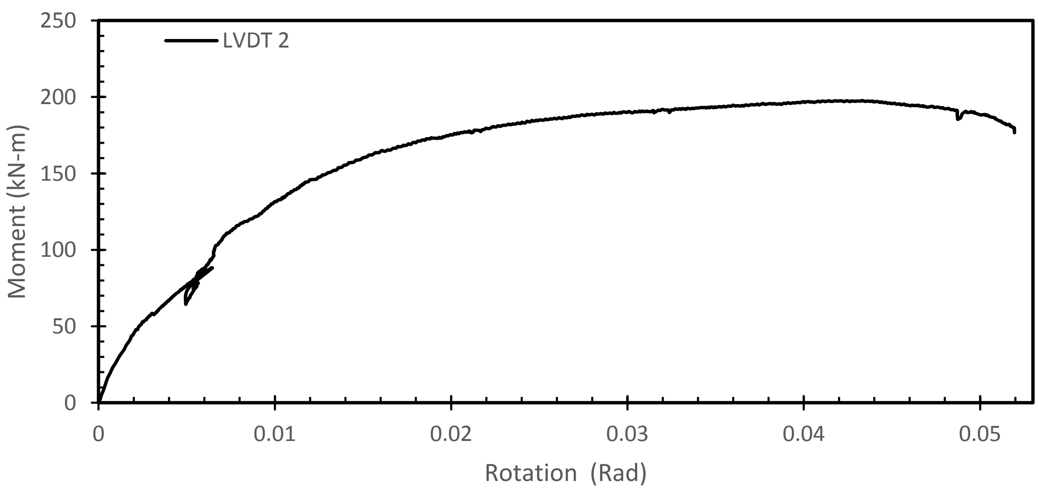


Figure 4.3. Moment-rotation curve for test F200x300

4.2.2 Test F200x200

The final failure mode was connection plate failure and local buckling shown in Figure 4.4 and the maximum applied load to the beam at connection failure was 50 kN. This maximum load produced a maximum bending moment of 53.5 kN-m (50x1.07 m) in the connection, where 1.07 m is the distance from loading point to the face of plate connection. Figure 4.5 display the deformed shape of the connection and the state of bolts, the same failure mode as in F200x300, it has a bit less significance in this connection. Clearly, the extended end plate has experienced yield before the fracture.

Figure 4.4. Test F200x200 failure modes

Figure 4.5. F200x200 connection deformation

Figure 4.6 presents the documented moment-rotation. the rotation was calculated as the reading from LVDT2 divided by 610 mm. The maximum bending moment applied was 53.5 kN-m and the corresponding rotation was 0.071 rad for LVDT 1. The difference between LVDT 1 and LVDT 2 due to very small eccentricity of the load cell to the center of the beam where causing a torsion.

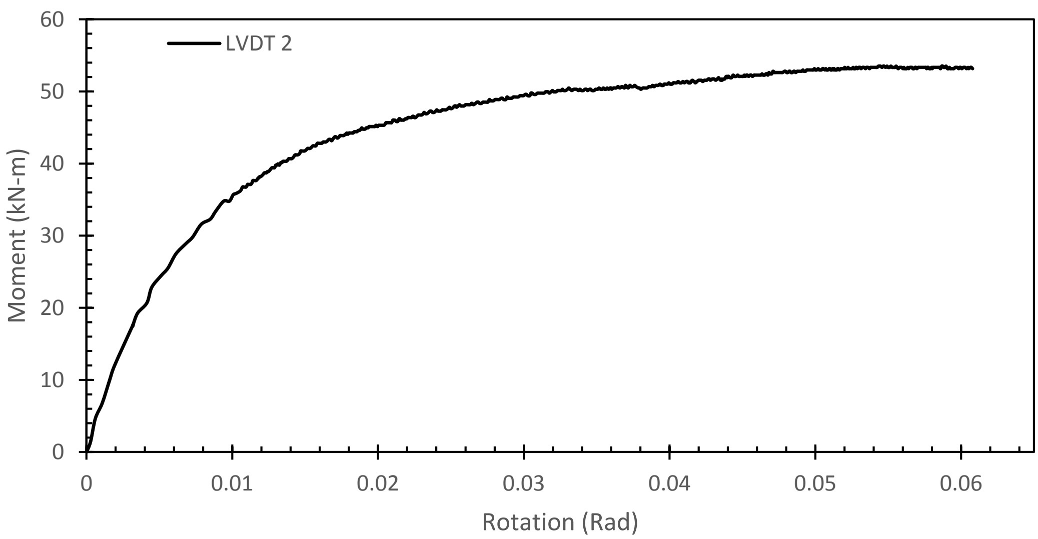


Figure 4.6. Moment-rotation curve for test F200x200

4.2.3 Test F160x200

Figure 4.7 presents the final failure mode as a connection plate failure with torsion. The maximum applied load to the beam at connection failure was 59.5 kN. This maximum load gave a maximum bending moment of 65.4 kN-m (59.5x1.1 m) on the connection, where 1.1 m is the distance from loading point to the face of plate connection. Figure 4.8 shows the test plate deformed shape and the condition of bolts. It is clear that, the extended end plate has undergone yield mechanism before fracture.

Figure 4.7. Test F160x200 failure mode

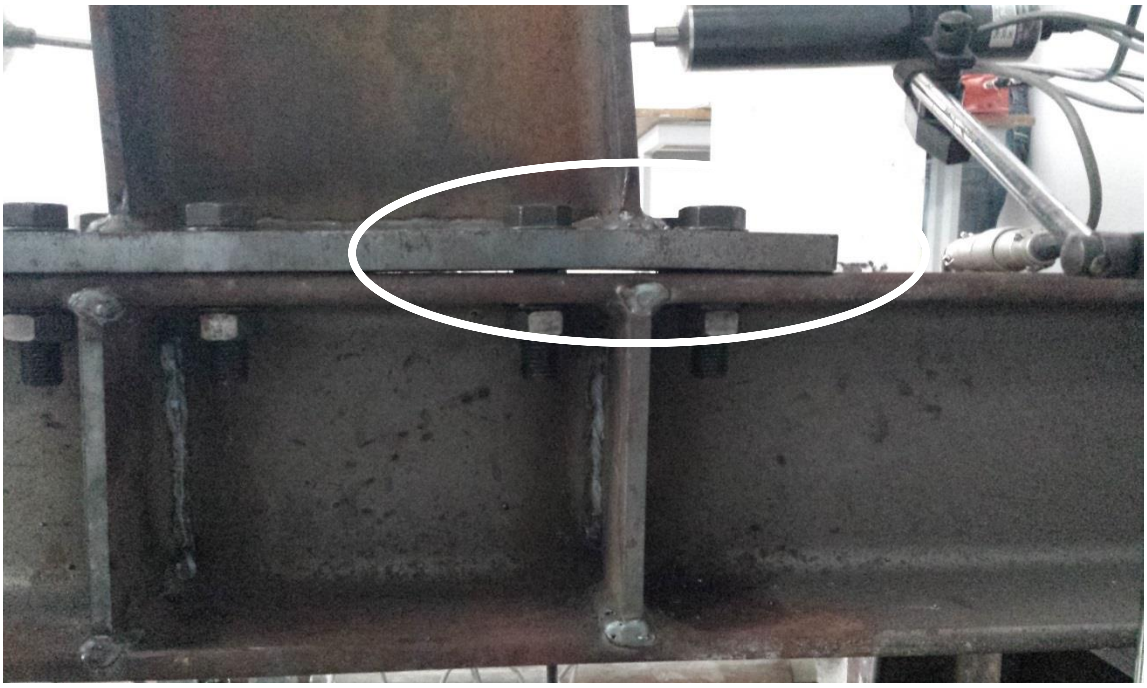


Figure 4.8. F160x200 connection deformation

Figure 4.9 shows the detailed moment-rotation curve; the rotation was calculated by the reading from LVDT2 divided by 610 mm. The maximum bending moment applied was 65.4 kN-m and the corresponding rotation was 0.0732 rad. The step down at 0.011 rad was due to the yielding mechanism within the plate.

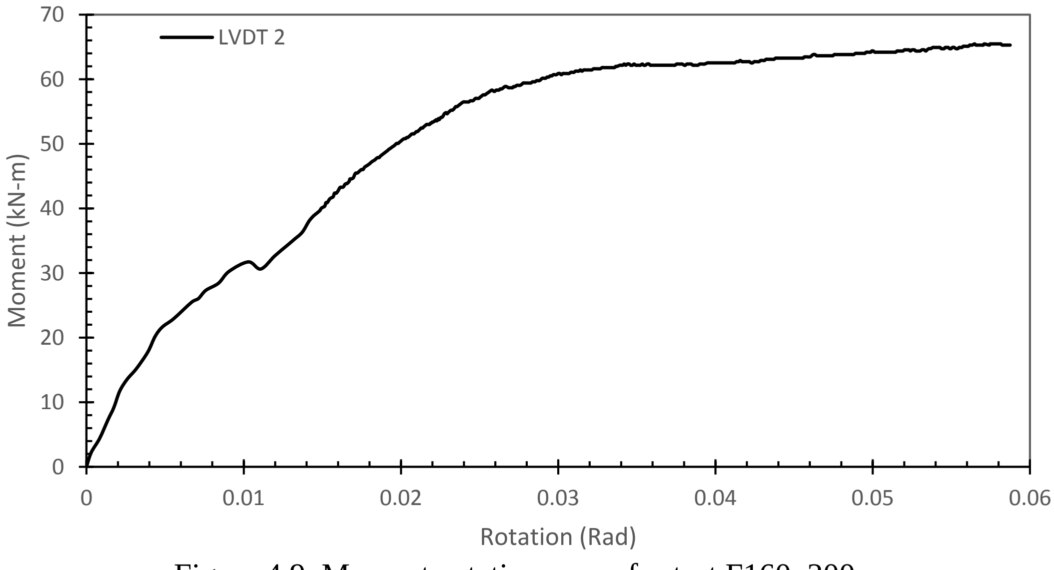


Figure 4.9. Moment-rotation curve for test F160x200

4.2.4 Test F160x160

The maximum applied load to the beam at failure was 37.7 kN. This maximum load gave a maximum bending moment of 41.8 kN-m (37.7x1.11 m) in the connection, where 1.11 m is the distance from loading point to the face of plate connection. Figure 4.10 shows that the final failure mode was beam torsion. Figures 4.11 Show the deformed shape of the connection and the condition of bolts. The beam flanges have experienced yield before fracture.

Figure 4.10. Test F160x160 failure modes

Figure 4.11. F160x160 connection deformation

Figure 4.12 introduces the logged moment-rotation curve; the rotation was calculated by the reading from LVDT2 divided by 615 mm. The maximum bending moment applied was 41.8kN-m at a rotation 0.056 rad for LVDT 1.

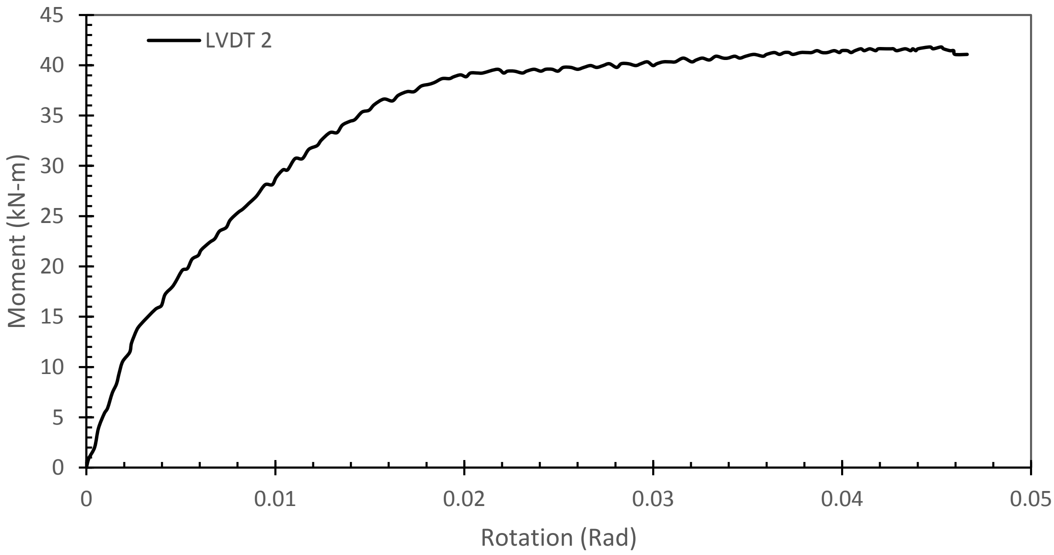


Figure 4.12. Moment-rotation curve for test F160x160

4.2.5 Classification of Extended End Plate Connection Test Results by Strength

Predicted capacities of the connections using Autodesk Robot Structural Analysis Professional and verified by hand calculations are in Table 4.1. The predicted failure loads match the test results reasonably well for most cases Table 4.2. Accuracy of the calculation results may be improved by better estimating the lever arm, bolt force distribution and to ensure that the beam deformed in a straight line.

Table 4.1. Extended end plate predicted connections capacity [41]

Test	$S_{j.ini}$ (kN-m)	S_j (kN-m)	$M_{j.rd}$ (kN-m)	$M_{b.pi.rd}$ (kN-m)	$M_{j.ult}$ (kN-m)	Predicted Failure Mode
F(200x300)	41074.98	20537.49	132.40	172.00	353.65	CWPS
F(200x200)	15732.53	5284.82	49.87	60.77	-	BFWC
F(160x200)	13447.59	6723.80	46.90	60.77	194.70	CWPS
F(160x160)	11108.94	5554.47	40.64	34.10	194.70	BFWC

Table 4.2. Classification of major axis connections by strength

Connection	Maximum Moment (kN-m)		Predicted / Tested	f_{yp} MPa	Classification
	Predicted strength $M_{b,pi,rd}$	Test Strength M_{app}			
F(200x300)	172.00	197.69	0.870	280.0	Partial Resistance
F(200x200)	60.77	53.51	1.135	263.8	Pinned
F(160x200)	60.77	65.46	0.930	263.8	Pinned
F(160x160)	40.64	41.82	0.972	302.5	Pinned

When classifying by strength a joint may be categorized as full-strength if it meets the conditions in BS EN 1993-1-8

$$M_{j,Rd} \geq M_{b,pi,Rd}$$

A joint may be categorized as nominally pinned if its moment resistance is less than 25% of strength of a full resistance connection.

$$M_{j,Rd} < 0.25 M_{b,pi,Rd,lim}$$

A connection which doesn't satisfy the conditions for a full-strength or a nominally pinned should be categorized as a partial-strength joint.

4.2.6 Classification of Extended End Plate Connection Test Results by Stiffness

The results in Table 4.2 and Figure 4.13 show the moment-rotation at failure for different connections. The connections were classified by strength and stiffness. Connection F200x300 showed the highest moment capacity around 200kN-m compared to the rest of the connections (F200x200, F160x200 and F160x160) where they had similar behavior in terms of moment capacity achieving 53kN-m as an average moment value. Rotation values for major axis tests were varied between 0.047 and 0.061 rad. The detailed rotation values for the extended end plate tests are summarized in Table 4.3.

The connection F200x300 gave a much higher moment capacity compared to the rest of the connections due to the huge difference in plate depth, that's giving more effective tension\compression resistance area which are providing more stability and resistance to the connection.

It can be clearly seen that the difference in plate depth from 440 mm to 300 mm can have a significant change in moment capacity for connections.

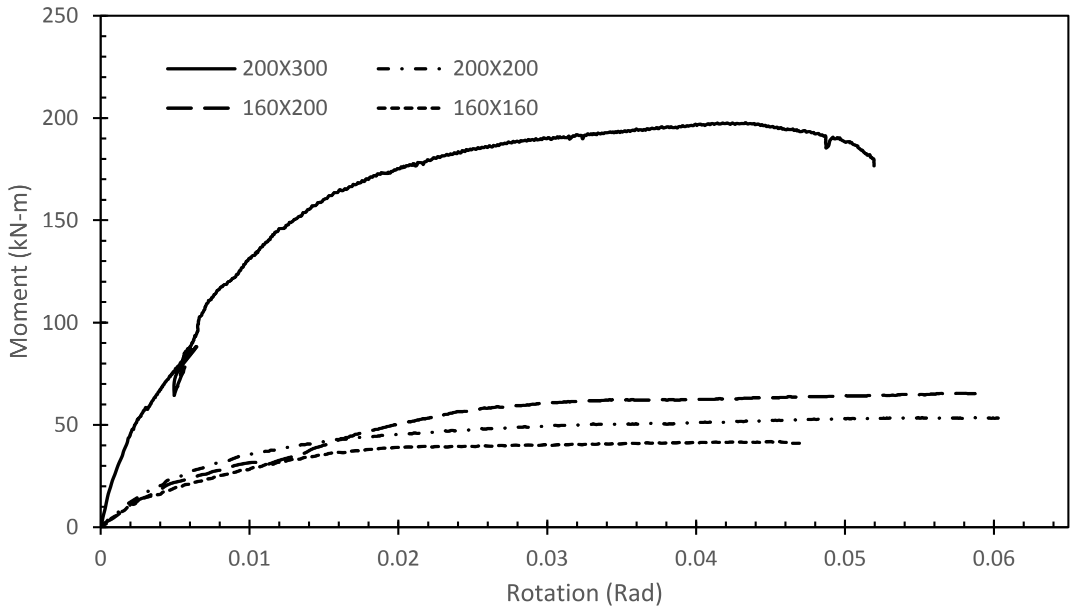


Figure 4.13. Comparison of the extended end plate moment-rotation curves

Table 4.3. Classification of major axis connections by stiffness

Connection	Maximum Applied Moment M_{max} (kN-m)	Maximum rotation (rad)	$S_{j.ini\ pinned}$ (kN-m)	Predicted $S_{j.ini}$ (kN-m)	$S_{j.ini\ rigid}$ (kN-m)	Failure Mode	Classification
F(200x300)	197.6933	0.0720	7311.50	41074.98	116984	Torsion	Semi-Rigid
F(200x200)	53.5107	0.0710	1700.25	15732.53	27202	Plate yielding	Semi-Rigid
F(160x200)	65.46309	0.0732	1700.25	13447.59	27202	Plate yielding	Semi-Rigid
F(160x160)	41.81836	0.0563	760.375	11108.94	12166	Torsion	Semi-Rigid

4.3 Stiffened Fin Plate Test Results

4.3.1 Test W200x300

The maximum applied load to the beam at connection failure was 24.4 kN. This maximum load gave a maximum bending moment of 29.33 kN-m (24.4x1.205 m) in the connection, where 1.205 m is the distance from loading point to the face of column web. Figures 4.14 show the deformed shape of the connection and the condition of bolts. The fin plate has experienced no damage but the beam holes has experienced plate shear as a yielding failure not a sudden fracture.

Figure 4.14. W300x200 Connection deformation as enlargement of bolt holes

Figure 4.15 shows the recorded moment-rotation; the rotation was calculated by dividing LVDT2 by 707.5 mm. The maximum bending moment applied was 29.33kN-m at a rotation of 0.079265 rad. Between 0-0.034 rad there was a slip due to the difference in size between the designed and the actual test bolt hole sizes. Connection was designed to have M16 bolts with 18 mm diameter holes but in actual test specimen bolt holes were 20 mm therefore M18 bolts were used.

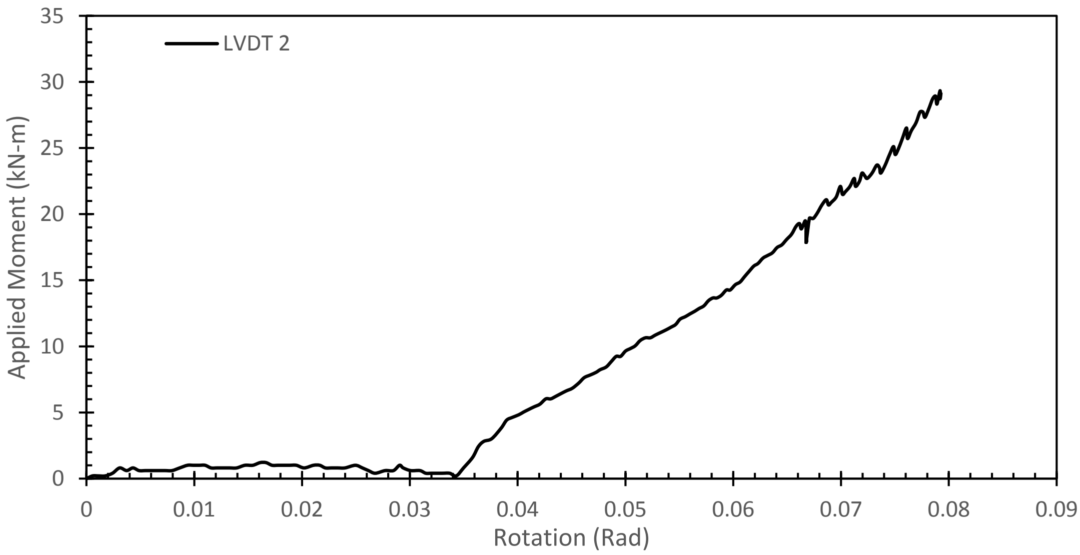


Figure 4.15. Moment-rotation curve for test W300x200

4.3.2 Test W200x200

The concentrated applied load to the beam at connection failure was 8 kN. This maximum load gave a maximum bending moment of 9.8 kN-m (8 x1.2255 m) in the connection, where 1.2255 m is the distance from loading point to the face of column web. Figures 4.16 show the deformed shape of the connection and the condition of bolts. Clearly, the fin plate has endured no damage but the beam holes has experienced plate shear.


Figure 4.16. W200x200 connection deformation

Figure 4.17 Presents the recorded moment-rotation, the rotation was calculated by dividing the LVDT2 by 709.5 mm. The maximum bending moment applied was 9.8 kN-m at a rotation 0.098069 rad. There was a small slip at the beginning of the test and then the first bolt row started resisting the applied force. The extra resistance recorded at 0.086 rad was due to the full contribution of the second bolt.

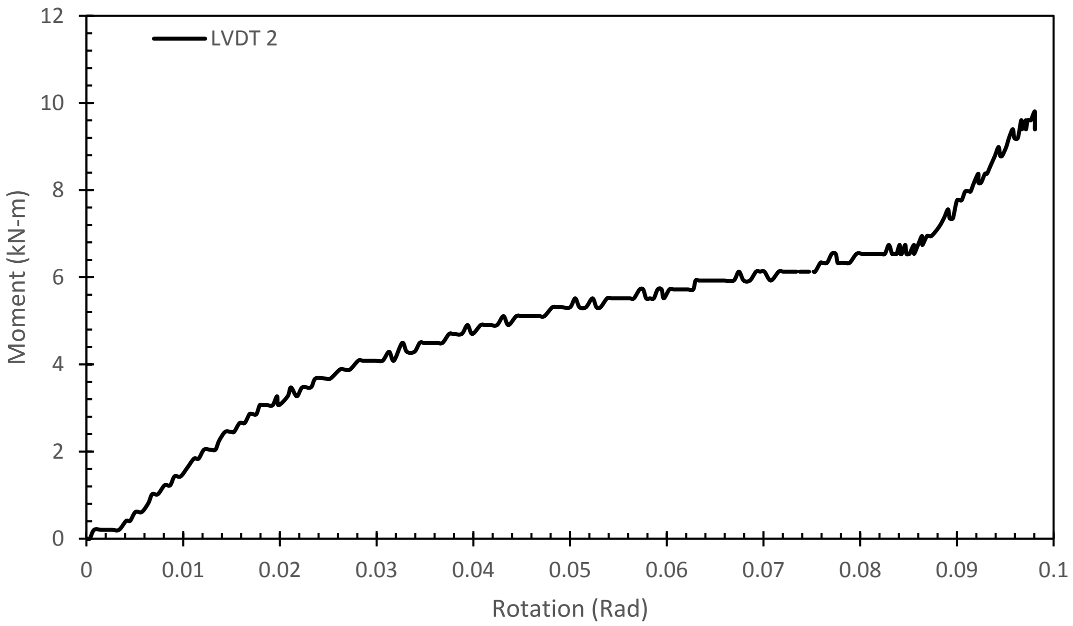


Figure 4.17. Moment-rotation curve for test W200x200

4.3.3 Test W160x200

The maximum applied load to the beam at connection failure was 3.75 kN. This maximum load gave a maximum bending moment of 4.56 kN-m (3.75x1.216) in the connection, where 1.216 m is the distance from loading point to the face of column web. Figures 4.18 show the deformed shape of the connection and the condition of bolts. The fin plate has experienced no damage but the beam holes has experienced web shear before fracture.

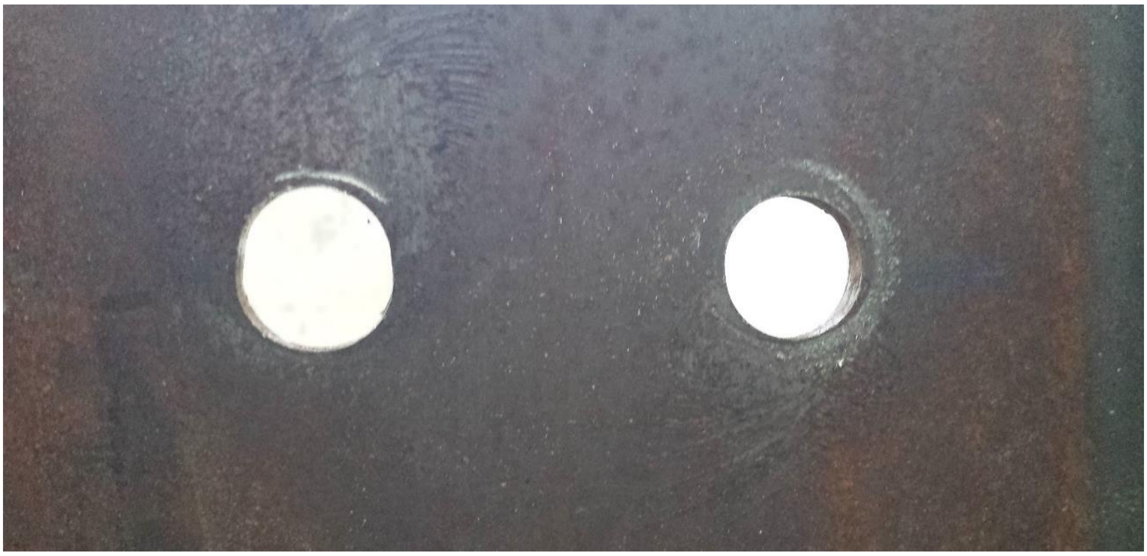


Figure 4.18. W160x200 Connection deformation

Figure 4.19 presents the recorded moment-rotation, the rotation was calculated by dividing LVDT 2 by 676 mm. The maximum bending moment applied was 4.56 kN-m at a rotation 0.101376 rad. There was a slip between the rotation values of 0 to 0.033 rad due to the difference between the designed and actual bolt and bolt hole sizes. The fluctuation of the M-Ø curve at a rotation of 0.078 rad was due to a small glitch of the load cell because of the constant rotation of loading point.

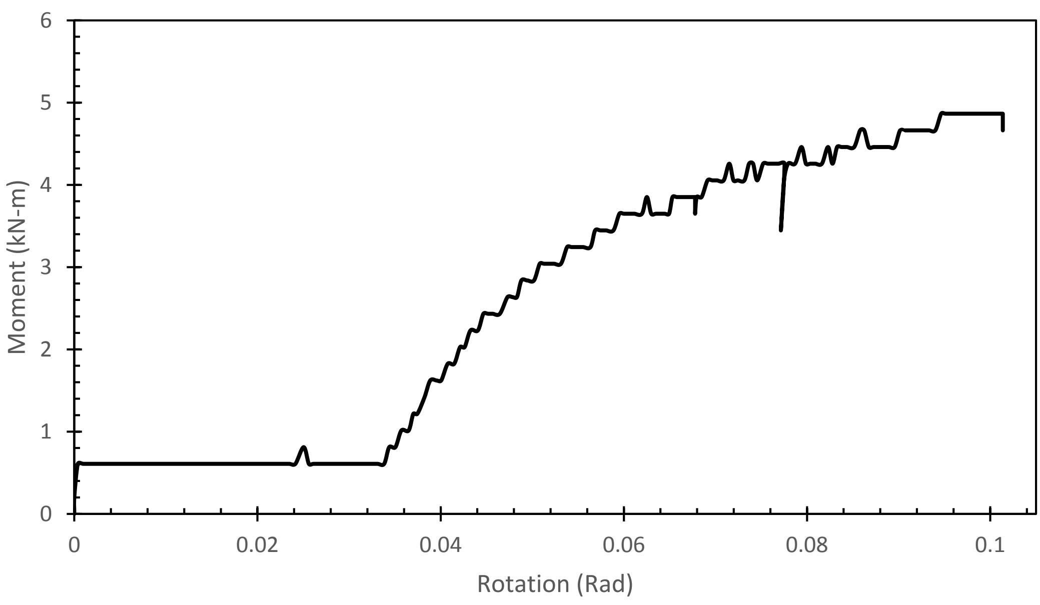


Figure 4.19. Moment-rotation curve for test W160x200

4.3.4 Test W160x160

The maximum applied load to the beam at connection failure was 4.67 kN. This maximum load gave a maximum bending moment of 5.72 kN-m (4.67x1.226 m) in the connection, where 1.226 m is the distance from loading point to the face of column web. Figures 4.20 show the deformed shape of the connection as a bolt hole deformation. The fin plate has experienced no damage but the beam holes has experienced yield before fracture.

Figure 4.20. W160x160 Connection deformation

Figure 4.21 presents the recorded moment-rotation, where the rotation was calculated as LVDT2/686mm. The maximum bending moment applied was 5.72 kN-m at a rotation 0.139985 rad. The connection did not reach the maximum moment capacity because of the torsional behavior but it has clearly more than enough rotational capacity to be adequate for use in earth quake regions.

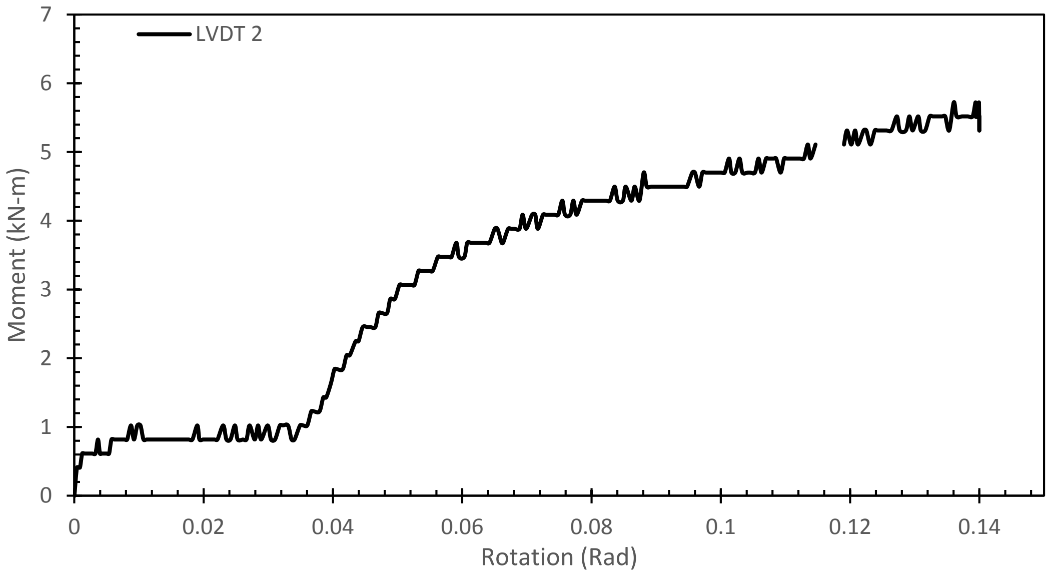


Figure 4.21. Moment-rotation curve for test W160x160

4.3.5 Classification of Stiffened Fin Plate Connection Test Results by Strength

The predicted failure loads don't match the test results in table 4.4. Accuracy of the calculation results may be improved by better estimating the bolts slip, bolt force distribution and by ensuring that the beam deformed in a straight line.

Table 4.4. Classification of minor axis connections by strength

Connection	Maximum Moment (kN-m)		$\boldsymbol{M_{j,rd}}/\boldsymbol{M_{app}}$	f_{yp} MPa	Classification
	Predicted strength $M_{j,rd}$	Applied Strength M_{app}			
W(200x300)	10.00	29.327	0.34	300	Pinned
W(200x200)	3.95	9.805	0.40	175.6	Pinned
W(160x200)	3.40	4.864	0.7	175.6	Pinned
W(160x160)	3.02	5.722	0.527	341.8	Pinned

4.3.6 Classification of Stiffened Fin Plate Connection Test Results by Stiffness

The results in Table 4.4 and Figure 4.22 show the moment-rotation at failure for different connections. The connection W200x300 showed the highest moment capacity around 30 kN-m comparing to the rest of the connections (W200x200, W160x200 and W160x160) were they behaved in similar manner in terms of moment capacity where they have got 9.8kN-m, 4.8kN-m and 5.7 kN-m respectively. All sections showed close rotation values which were 0.079, 0.098, 0.10 and 0.14 rad respectively. The detailed rotation results of the stiffened fin-plate tests are summarized in Table 4.5.

The connection W200x300 gave a sudden and much higher moment capacity compared to the rest of the connections due to the stability of the three bolt rows geometry used rather than 2 rows as the rest of the tests. Using this kind of geometry, connection is having a much like ductile behavior rather than flexible as the rest of the minor axis connection configurations.

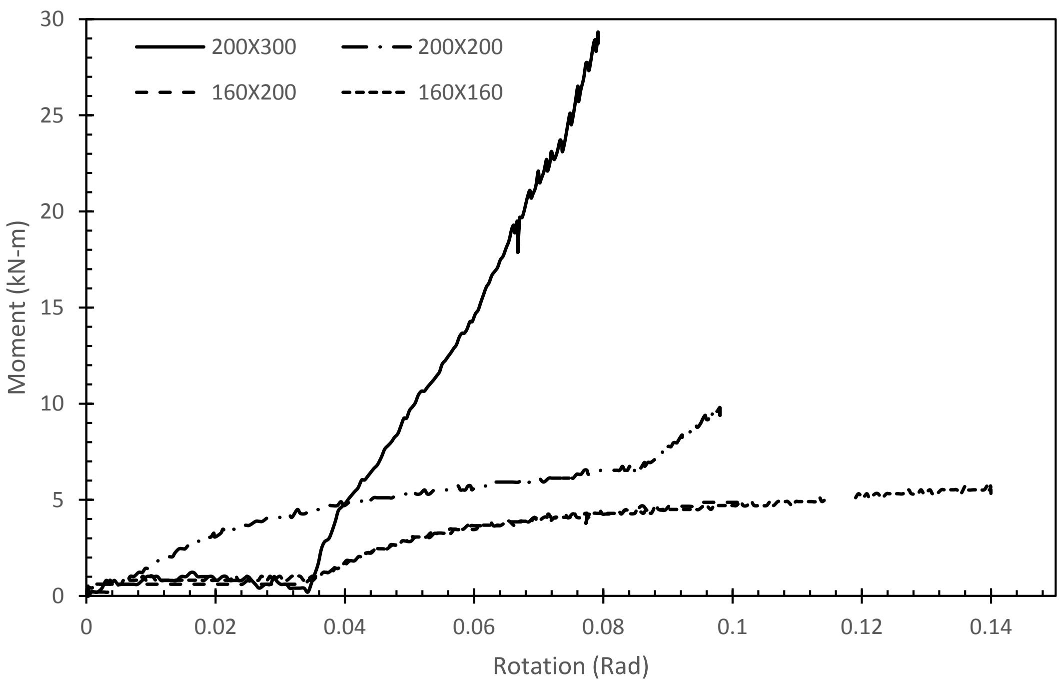


Figure 4.22. Minor axis moment-rotation tests comparison

Table 4.5. Classification of minor axis connections by stiffness

Connection	Maximum Applied Moment M_{max} (kN-m)	Maximum Rotation (rad)	Failure Mode	Classification
W(200x300)	29.32753	0.079265	Plate shear	Pinned
W(200x200)	9.805961	0.098069	Plate shear	Pinned
W(160x200)	4.864973	0.101376	Plate shear	Pinned
W(160x160)	5.722478	0.139985	Plate shear	Pinned

Chapter 5

SUMMARY, CONLUSION AND RECOMMENDATION

Summary

The main purposes of this study was to

- study the applicability of selected extended end plate and stiffened fin plate connections for seismic regions.

- classify and study the stiffness of these connections.

To achieve these goals, an investigation that include a wide-range background literature review, experimental testing and comparison of test results with analytical results were executed. Four extended end plate moment connection configurations and four stiffened fin plate connection outlines were investigated in this study.

The first step of the investigation was a widespread background literature review to detect the knowledge and the previous studies. From the literature review it was recognized that the necessity for such study so as to have better understanding of the behavior of selected common connections.

The second step of investigation was the experimental testing of extended end plate and stiffened fin plate connections subjected to monotonic loading. The experimental results showed that, extended end-plate connections can be effectively intended to resist monotonic loading. To get the adequate amount of moment-rotation capacity out of stiffened fin plate the designed geometry has to use three or more bolt rows.

5.1 Conclusions

1. The moment-rotation characteristics of the stiffened fin plate connections can be changed by changing connection details. For example, number of bolt rows appears to be important and changes in number would affect the behavior. Since the plate is connected to the column web, reducing the lever arm of this plate can increase the connection moment capacity. However, all the tested connections were able to reach a rotation capacity of 0.03 rad, commonly considered to be sufficient for plastic design and the connection to have adequate ductility and therefore resistance to earthquake loading.

2. Enlargement of holes due to plate shear was the most common failure observation for the minor axis connections. The connection may have higher moment resistance by increasing the web thickness. Changing connections geometry may have substantial outcome on rotational capacity and moment capacity.

3. Observations from the tests indicate that, the moment-rotation characteristics of the extended end plate connections may change with changes in connection details. Among these changes, the plate total height and the number of bolt rows in tension side is the most important component. Since the plate is connected to the column flange, reducing the beam length can be used to increases the reliability of monotonic testing. However, all the tested connections were able to reach a rotational capacity higher than 0.03 rad, commonly considered to be sufficient for plastic design.

4. End plate yielding along the tension side was observed to be a failure mode due to prying force in major axis connections.

5. Connections with two bolt rows are not reliable since they show slip and holes expanding before connection failure or section yielding.

6. Sections lower than IPE 300 can be very flexible. Giving higher rotation but lower moment capacity.

5.2 Recommendations for future work

1. The testing procedure reported in this study is recommended to be used but the beam length should be kept short, as much as possible, to avoid torsional buckling. Using the standing column-beam configuration is reliable when applying the load at the tip of the beam.

2. Use of three or more bolts in a row in any stiffened fin plate connection allows the connection to reach the plastic region.

3. Moment-rotation curve data obtained from tests can be used in analysis and design programs to compare the theoretical behavior of structures with the experimental one.

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APPENDICES

Appendix A: Material Properties Obtained from Coupon Tests

Table A1. Material properties obtained from coupon tests

Coupon	Yield Stress MPa	Ultimate Stress MPa	Ultimate Strain %
HE200 Flange	328.95	475.17	32.10
HE200 Web	313.88	472.54	34.00
HE160 Flange	321.62	492.15	33.13
HE160 Web	333.33	490.32	28.75
IPE300 Flange	280.00	393.00	35.00
IPE300 Web	300.00	383.33	33.75
IPE200 Flange	263.87	345.46	35.00
IPE200 Web	175.60	251.19	33.75
IPE160 flange	302.47	423.56	28.75
IPE160 Web	341.84	447.96	31.25
Plate 15 mm	276.26	384.74	35.63
Plate 10 mm	358.94	410.96	31.25

Table A2. Actual material geometry

LOCATION	LOADING	MIDDLE	CONNECTION
HE200 FLANGE			
HE200 WEB			
HE160 FLANGE			
HE160 WEB			
IPE300 FLANGE			
IPE300 WEB			
IPE200 FLANGE			
IPE200 WEB			
IPE160 FLANGE			
IPE160 WEB			

Appendix B: Monotonic Extended End Plate Connection Test summary sheets

Test Name: F(200x300)

Test Date: 8.7.2015

Column

Section: HEB 200

$a =$	-90.0	[Deg]	Inclination angle
$h_c =$	200	[mm]	Height of column section
$b_{fc} =$	200	[mm]	Width of column section
$t_{wc} =$	9	[mm]	Thickness of the web of column section
$t_{fc} =$	15	[mm]	Thickness of the flange of column section
$r_c =$	18	[mm]	Radius of column section fillet
$A_c =$	7810	[mm^2]	Cross-sectional area of a column
$I_{xc} =$	56960000	[mm^4]	Moment of inertia of the column section

Material: S275

$f_{yc} =$ 275.00 [MPa] Resistance

Beam

Section: IPE 300

$a =$	0.0	[Deg]	Inclination angle
$h_b =$	300	[mm]	Height of beam section
$b_f =$	150	[mm]	Width of beam section
$t_{wb} =$	7	[mm]	Thickness of the web of beam section
$t_{fb} =$	11	[mm]	Thickness of the flange of beam section

a =	0.0	[Deg]	Inclination angle
r_b =	15	[mm]	Radius of beam section fillet
r_b =	15	[mm]	Radius of beam section fillet
A_b =	5380	[mm^2]	Cross-sectional area of a beam
I_{xb} =	83560000	[mm^4]	Moment of inertia of the beam section
Material:	S275		
f_{yb} =	275.00	[MPa]	Resistance

Bolts

The shear plane passes through the UNTHREADED portion of the bolt.

d =	20	[mm]	Bolt diameter
Class =	8.8		Bolt class
F_{tRd} =	141.12	[kN]	Tensile resistance of a bolt
n_h =	2		Number of bolt columns
n_v =	4		Number of bolt rows
h_1 =	40	[mm]	Distance between first bolt and upper edge of front plate

Horizontal spacing e_i = 100 [mm]

Vertical spacing p_i = 80; 200; 80 [mm]

Plate

h_p =	440	[mm]	Plate height
b_p =	180	[mm]	Plate width
t_p =	15	[mm]	Plate thickness
Material:	S275		
f_{yp} =	275.00	[MPa]	Resistance

Predicted strengths

$M_{b,pl,Rd}$ = 172.70 [kN-m] Plastic resistance of the section for bending (without stiffeners)

$F_{t,Rd}$ = 141.12 [kN] Bolt resistance for tension

$M_{j,Rd}$ = 132.40 [kN-m]Connection resistance for bending

$S_{j,ini}$ = 41074.98 [kN-m]Initial rotational stiffness

S_j = 20537.49 [kN-m]Final rotational stiffness

$M_{pl,Rd,lim}$ = 353.65 [kN-m]Strength of a full resistance connection

Figure B.1. F200x300 Test setup

Rotation

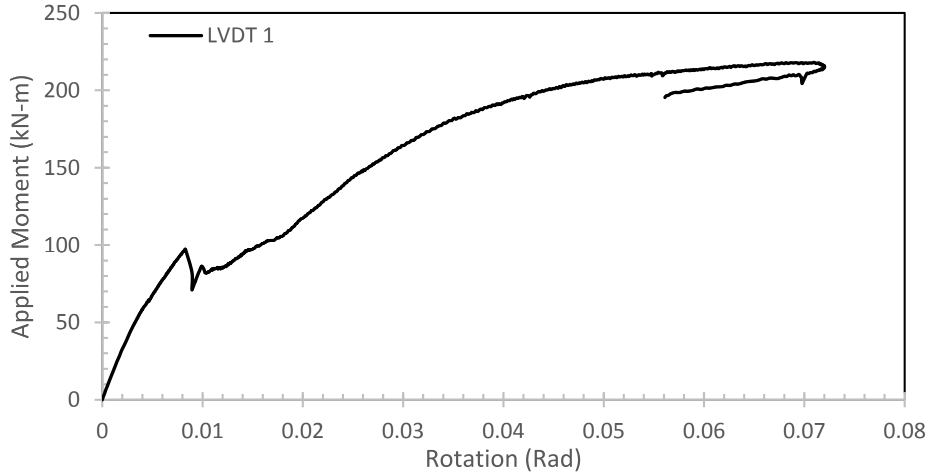


Figure B.2. F200x300 Moment-Rotation of LVDT 1

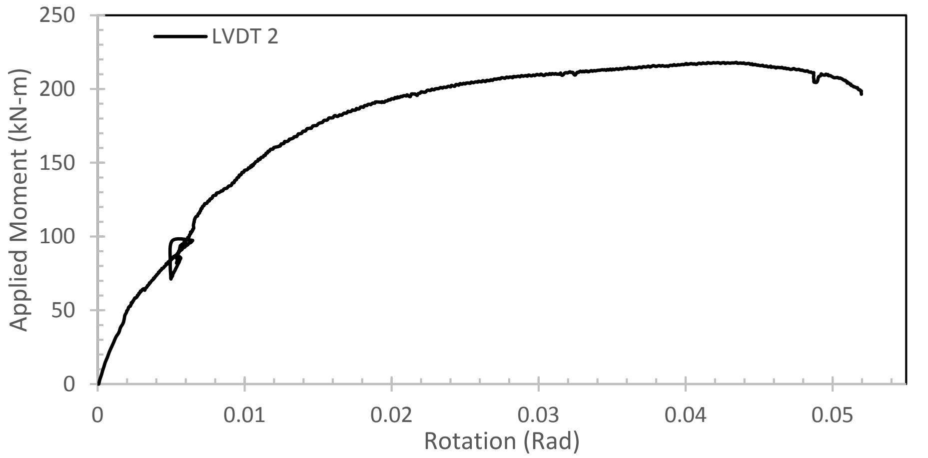


Figure B.3. F200x300 Moment-Rotation of LVDT 2

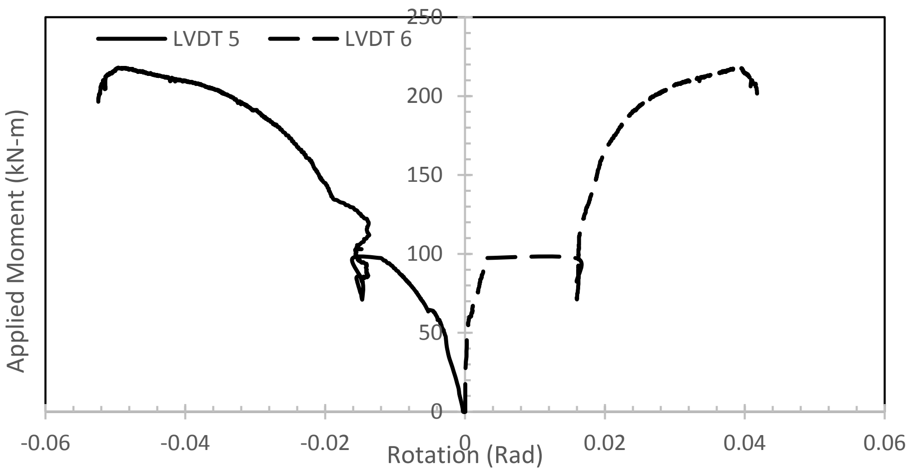


Figure B.4. F200x300 Moment-Rotation of LVDT 5 and 6

Displacement

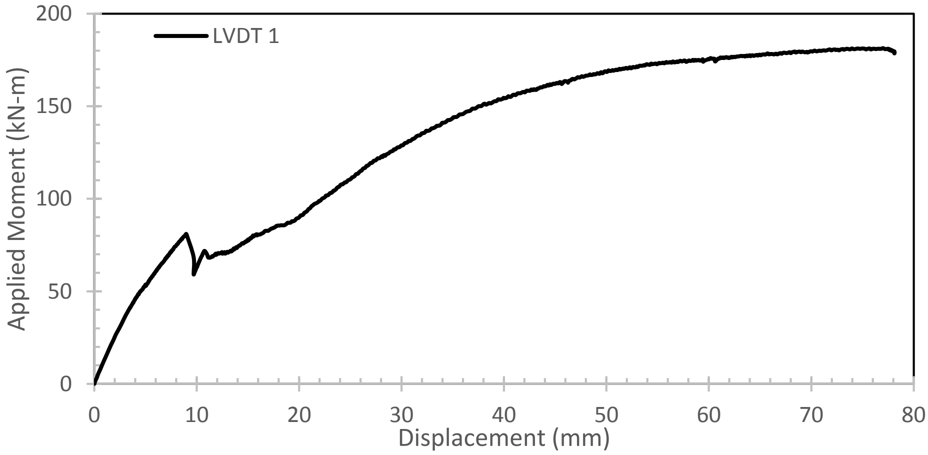


Figure B.5. F200x300 LVDT 1 displacement

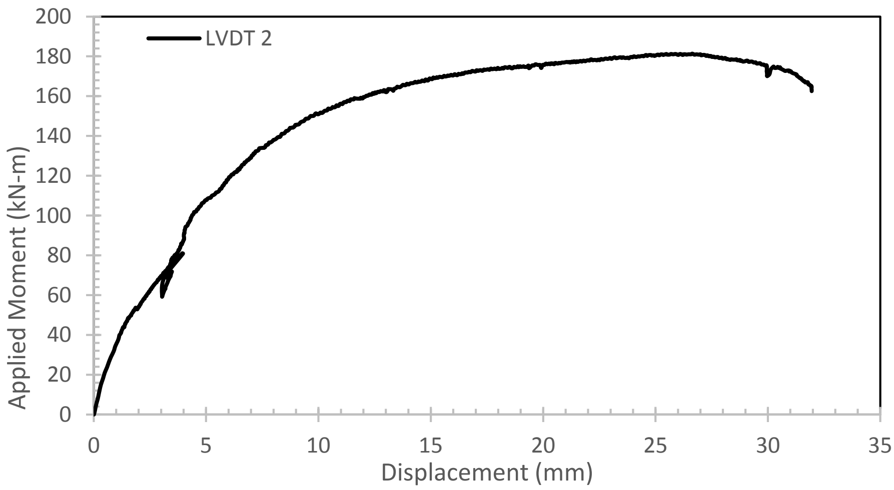


Figure B.6. F200x300 LVDT 2 displacement

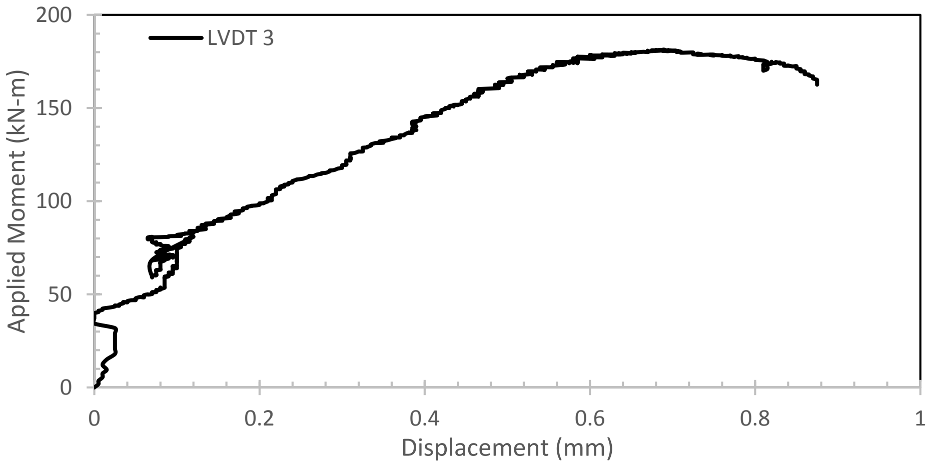


Figure B.7. F200x300 LVDT 3 displacement

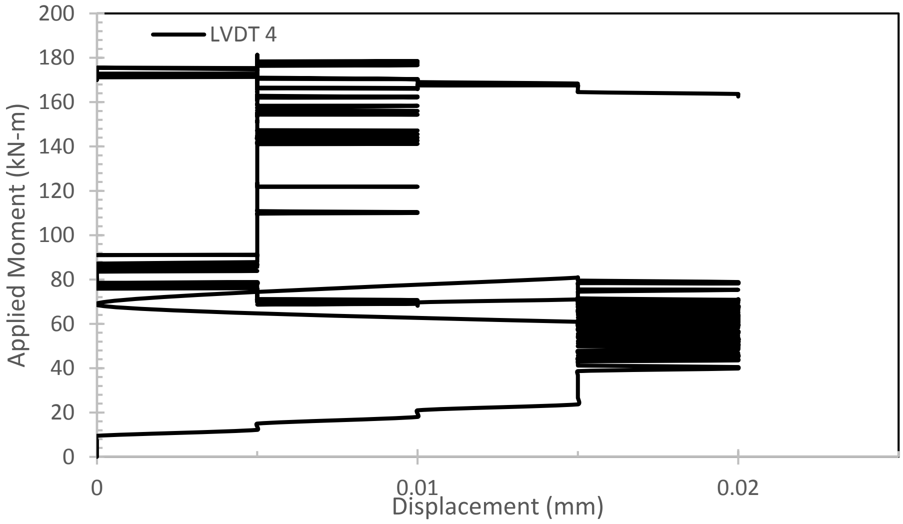


Figure B.8. F200x300 LVDT 4 displacement

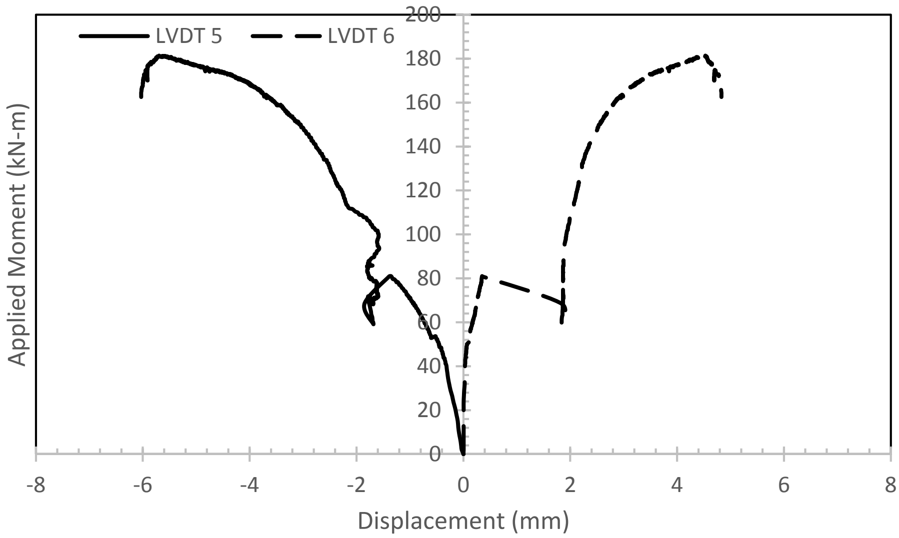


Figure B.9. F200x300 LVDT 5 and 6 displacements

Test Name: F(200x200)

Test Date: **9.7.2015**

COLUMN

Section: HEB 200

a =	-90.0	[Deg]	Inclination angle
hc =	200	[mm]	Height of column section
b_{fc} =	200	[mm]	Width of column section
t_{wc} =	9	[mm]	Thickness of the web of column section
t_{fc} =	15	[mm]	Thickness of the flange of column section
r_c =	18	[mm]	Radius of column section fillet
A_c =	7810	[mm^2]	Cross-sectional area of a column
I_{xc} =	56960000	[mm^4]	Moment of inertia of the column section
Material:	S275		
f_{yc} =	275.00	[MPa]	Resistance

BEAM

Section: IPE 200

a =	0.0	[Deg]	Inclination angle
h_b =	200	[mm]	Height of beam section
bf =	100	[mm]	Width of beam section
t_{wb} =	6	[mm]	Thickness of the web of beam section
t_{fb} =	9	[mm]	Thickness of the flange of beam section
rb =	12	[mm]	Radius of beam section fillet
A_b =	2850	[mm^2]	Cross-sectional area of a beam
I_{xb} =	19430000	[mm^4]	Moment of inertia of the beam section
Material:	S275		

f_{yb} = 275.00 [MPa] Resistance

BOLTS

The shear plane passes through the UNTHREADED portion of the bolt.

d = 16 [mm] Bolt diameter

Class = 8.8 Bolt class

F_{tRd} = 90.43 [kN] Tensile resistance of a bolt

n_h = 2 Number of bolt columns

nv = 4 Number of bolt rows

h1 = 40 [mm] Distance between first bolt and upper edge of front plate

Horizontal spacing ei = 100 [mm]

Vertical spacing pi = 70; 120; 70 [mm]

PLATE

h_p = 340 [mm] Plate height

b_p = 180 [mm] Plate width

t_p = 10 [mm] Plate thickness

Material: S275

f_{yp} = 275.00 [MPa] Resistance

Predicted strengths

$M_{b,pl,Rd}$ = 60.77 [kN-m] Plastic resistance of the section for bending (without stiffeners)

$F_{t,Rd}$ = 90.43 [kN] Bolt resistance for tension

$M_{j,Rd}$ = 49.87 [kN-m] Connection resistance for bending

$S_{j,ini}$ = 15732.53 [kN-m] Initial rotational stiffness

S_j = 5284.82 [kN-m] Final rotational stiffness

Figure B.10. F200x200 Test Setup

Rotation

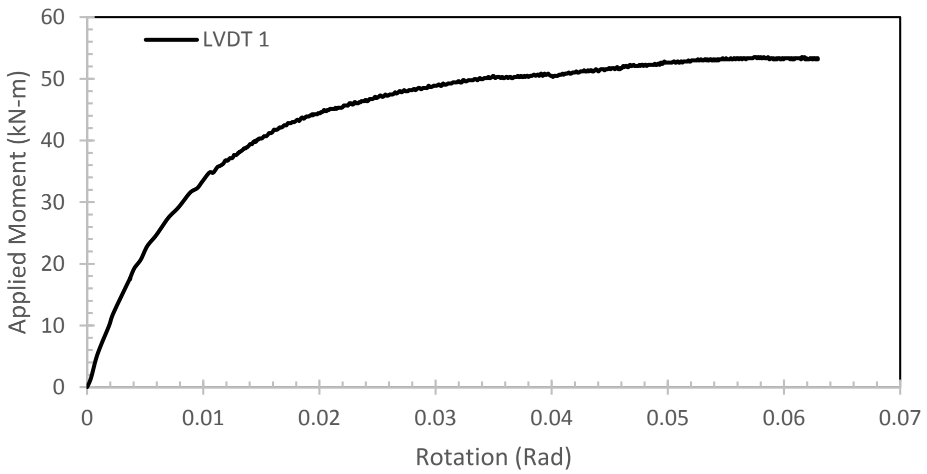


Figure B.11. F200x200 Moment-Rotation of LVDT 1

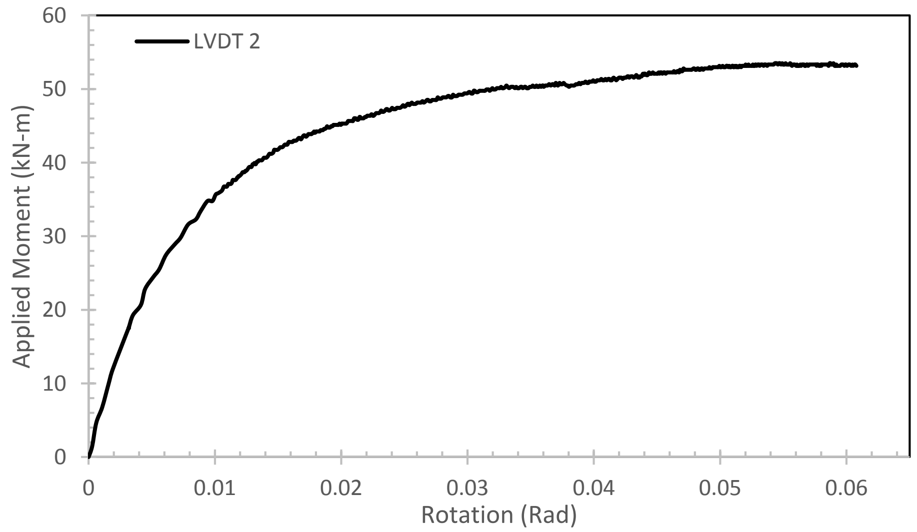


Figure B.12. F200x200 Moment-Rotation of LVDT 2

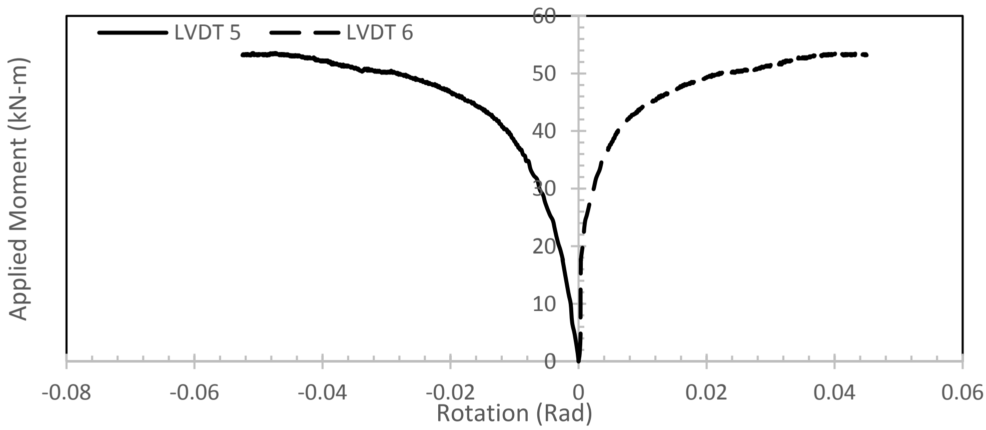


Figure B.13. F200x200 moment-rotation of LVDT 5 and 6

Displacement

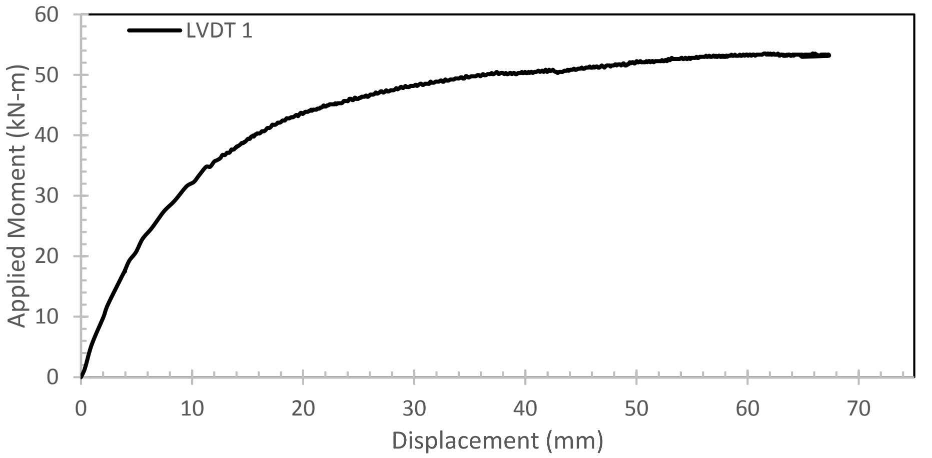


Figure B.14. F200x200 LVDT 1 displacement

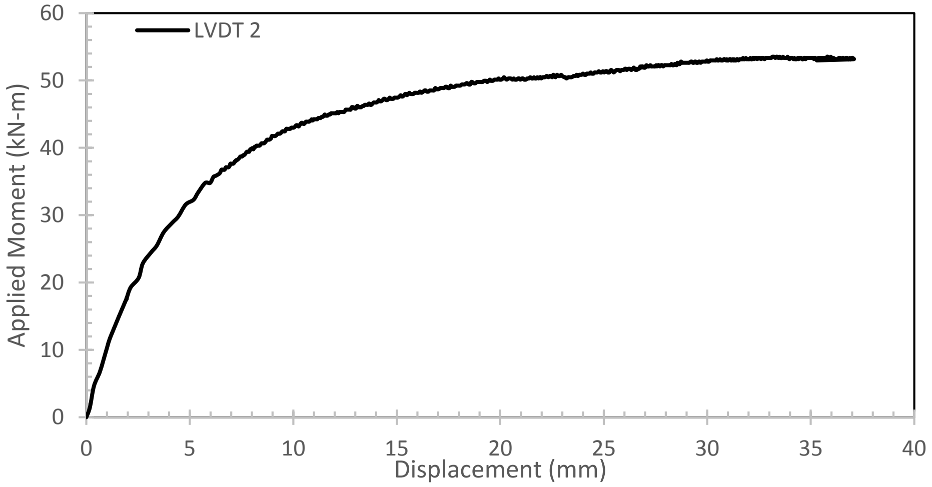


Figure B.15. F200x200 LVDT 2 displacement

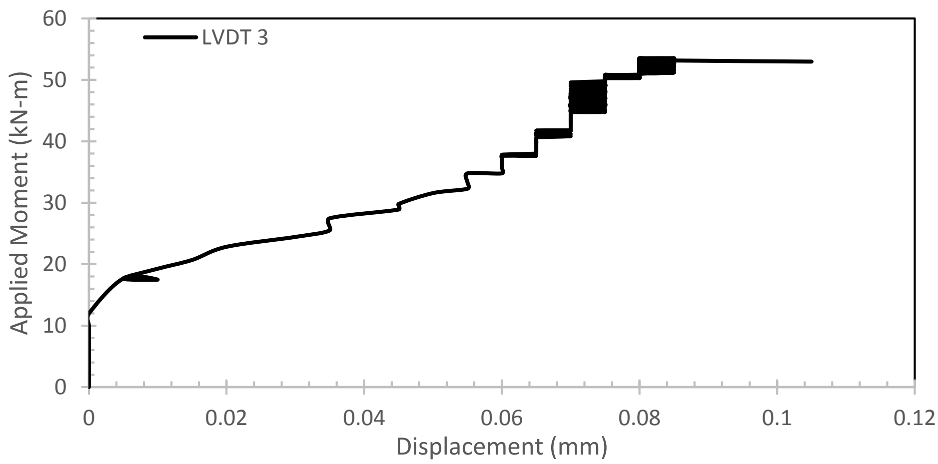


Figure B.16. F200x200 LVDT 3 displacement

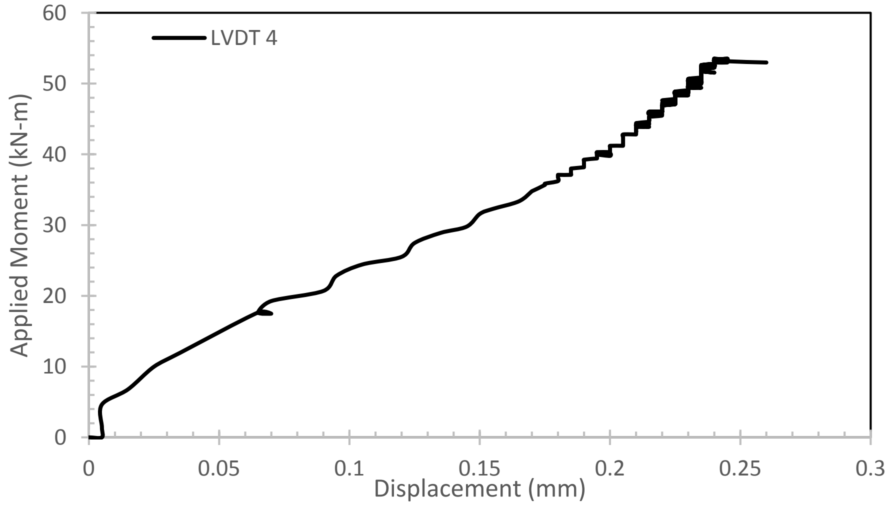


Figure B.17. F200x200 LVDT 4 displacement

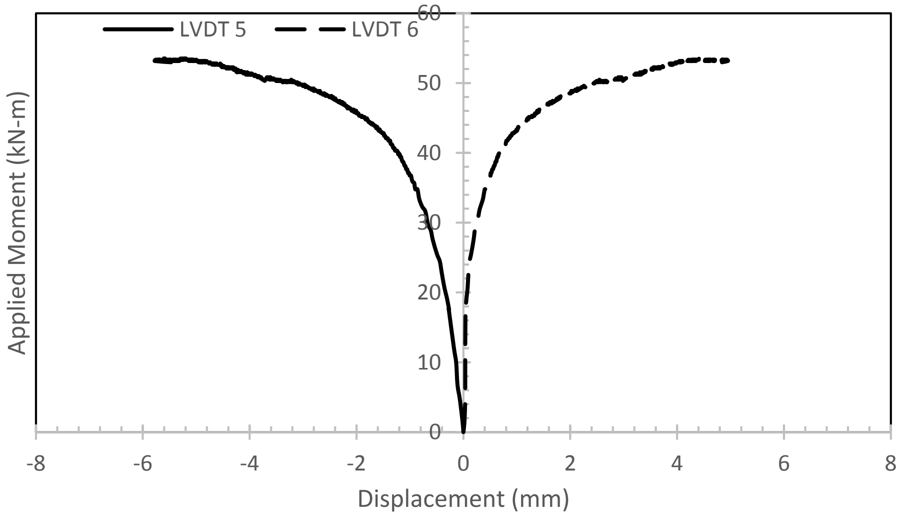


Figure B.18. F200x200 LVDT 5 and 6 displacements

<u>Test Name: F(160x200)</u>

<u>Test Date:</u> 9.7.2015

<u>COLUMN</u>

Section:	HEB 160		
a =-90.0	[Deg]	Inclination angle	
h_c =	160	[mm]	Height of column section
b_{fc} =	160	[mm]	Width of column section
t_{wc} =	8	[mm]	Thickness of the web of column section
t_{fc} =	13	[mm]	Thickness of the flange of column section
r_c =	15	[mm]	Radius of column section fillet
A_c =	5430	[mm^2]	Cross-sectional area of a column
I_{xc} =	24920000	[mm^4]	Moment of inertia of the column section
Material:	S275		
f_{yc} =	275.00	[MPa]	Resistance

<u>BEAM</u>

Section:	IPE 200		
a =0.0	[Deg]	Inclination angle	
hb =	200	[mm]	Height of beam section
bf =	100	[mm]	Width of beam section
t_{wb} =	6	[mm]	Thickness of the web of beam section
t_{fb} =	9	[mm]	Thickness of the flange of beam section
rb =	12	[mm]	Radius of beam section fillet
A_b =	2850	[mm^2]	Cross-sectional area of a beam
I_{xb} =	19430000	[mm^4]	Moment of inertia of the beam section
Material:	S275		

f_{yb} = 275.00 [MPa] Resistance

BOLTS

The shear plane passes through the UNTHREADED portion of the bolt.

d = 16 [mm] Bolt diameter

Class = 8.8 Bolt class

F_{tRd} = 90.43 [kN] Tensile resistance of a bolt

nh = 2 Number of bolt columns

nv = 4 Number of bolt rows

h1 = 40 [mm] Distance between first bolt and upper edge of front plate

Horizontal spacing ei = 80 [mm]

Vertical spacing pi = 70; 120; 70 [mm]

PLATE

h_p = 340 [mm] Plate height

b_p = 160 [mm] Plate width

t_p = 10 [mm] Plate thickness

Material: S275

f_{yp} = 275.00 [MPa] Resistance

Predicted strengths

$M_{b,pl,Rd}$ = 60.77 [kN-m] Plastic resistance of the section for bending (without stiffeners)

$F_{t,Rd}$ = 90.43 [kN] Bolt resistance for tension

$M_{j,Rd}$ = 46.9 [kN-m] Connection resistance for bending

$S_{j,ini}$ = 13447.59 [kN-m] Initial rotational stiffness

S_j = 6723.8 [kN-m] Final rotational stiffness

$M_{pl,Rd,lim}$ = 194.7 [kN-m] Strength of a full resistance connection

Figure B.19. F160x200 test setup

Rotation

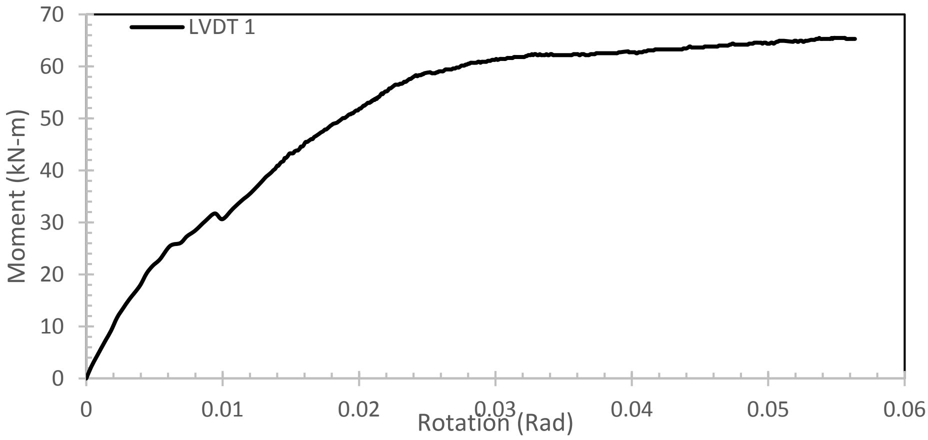


Figure B.20. F160x200 moment-rotation of LVDT 1

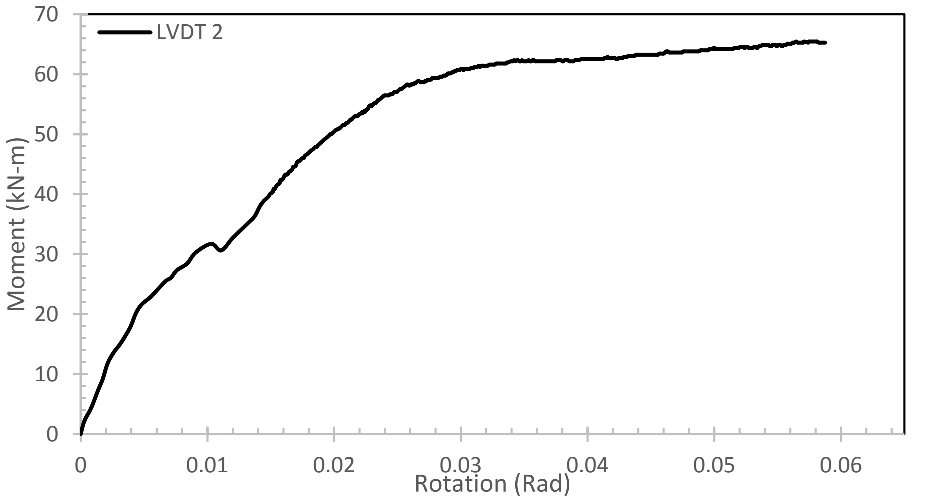


Figure B.21. F160x200 moment-rotation of LVDT 2

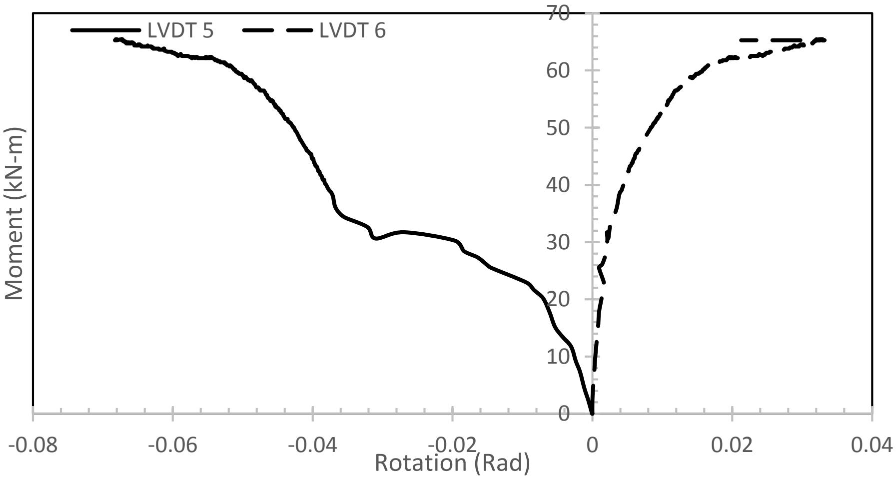


Figure B.22. F160x200 moment-rotation of LVDT 5 and 6

Displacement

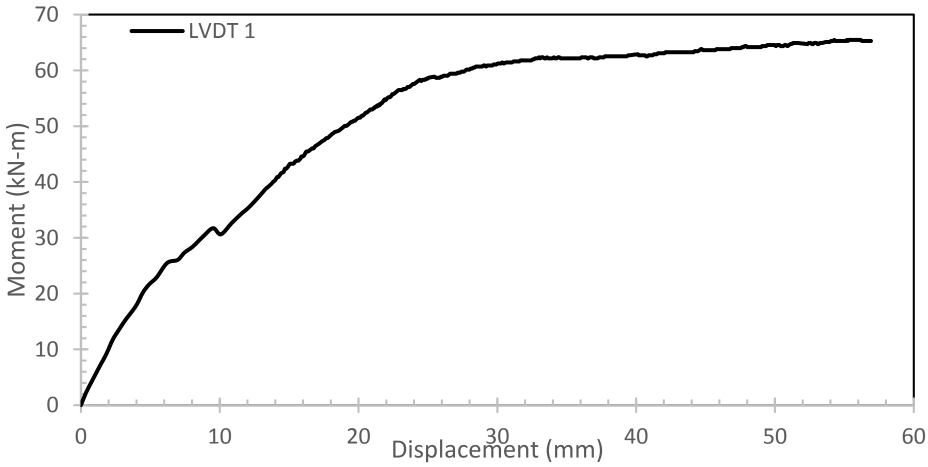


Figure B.23. F160x200 LVDT 1 displacement

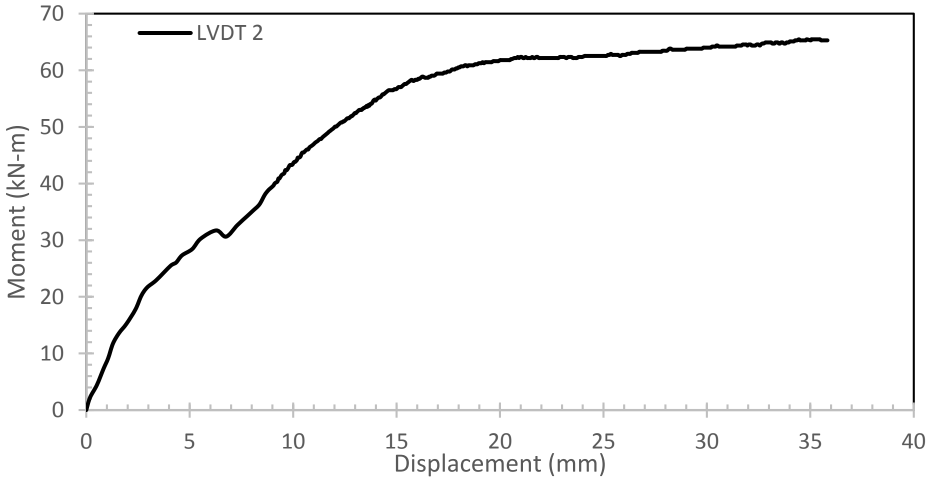


Figure B.24. F160x200 LVDT 2 displacement

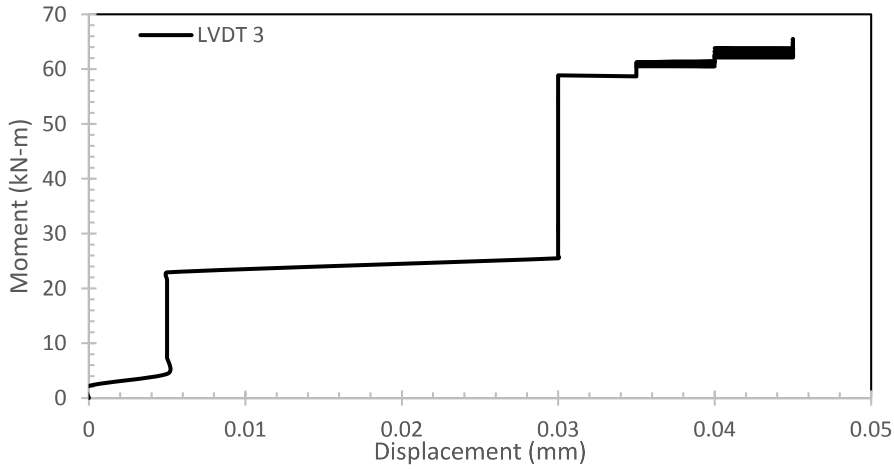


Figure B.25. F160x200 LVDT 3 displacement

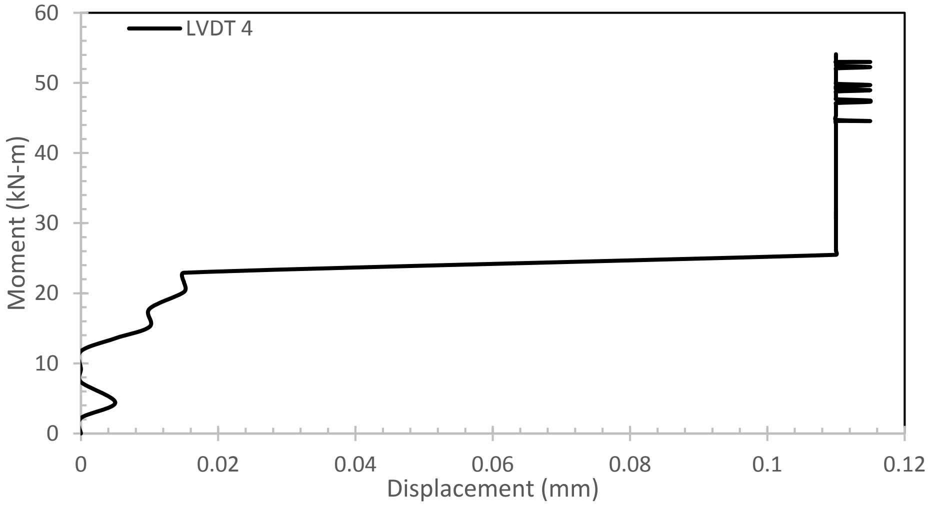


Figure B.26. F160x200 LVDT 4 displacement

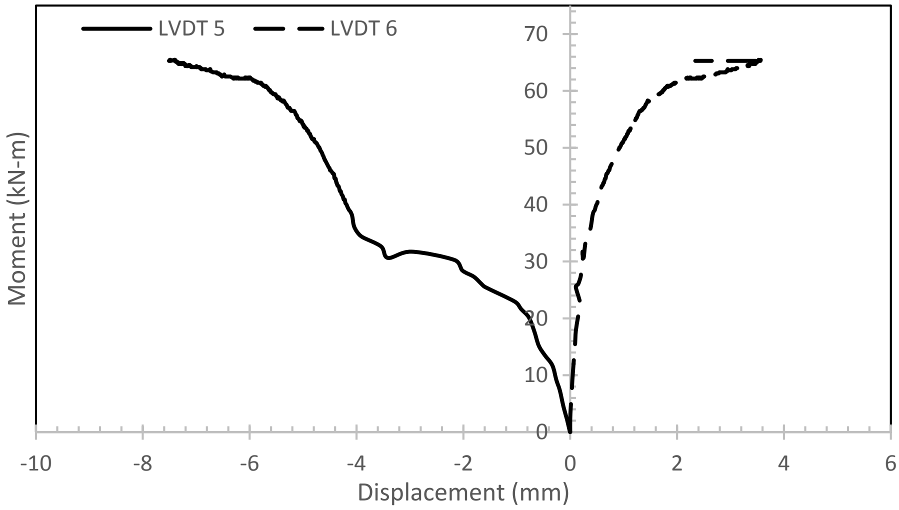


Figure B.27. F160x200 LVDT 5 and 6 displacements

Test Name: F(160x160)

Test Date: **9.7.2015**

COLUMN

Section:	HEB 160		
a =-90.0	[Deg]	Inclination angle	
h^c =	160	[mm]	Height of column section
b_{fc} =	160	[mm]	Width of column section
t_{wc} =	8	[mm]	Thickness of the web of column section
t_{fc} =	13	[mm]	Thickness of the flange of column section
r_c =	15	[mm]	Radius of column section fillet
A_c =	5430	[mm^2]	Cross-sectional area of a column
I_{xc} =	24920000	[mm^4]	Moment of inertia of the column section
Material:	S275		
f_{yc} =	275.00	[MPa]	Resistance

BEAM

Section:	IPE 160		
a =0.0	[Deg]	Inclination angle	
h_b =	160	[mm]	Height of beam section
b_f =	82	[mm]	Width of beam section
t_{wb} =	5	[mm]	Thickness of the web of beam section
t_{fb} =	7	[mm]	Thickness of the flange of beam section
r_b =	9	[mm]	Radius of beam section fillet
A_b =	2010	[mm^2]	Cross-sectional area of a beam
I_{xb} =	8690000	[mm^4]	Moment of inertia of the beam section
Material:	S275		

f_{yb} = 275.00 [MPa] Resistance

BOLTS

The shear plane passes through the UNTHREADED portion of the bolt.

d = 20 [mm] Bolt diameter

Class = 8.8 Bolt class

F_{tRd} = 141.12 [kN] Tensile resistance of a bolt

n_h = 2 Number of bolt columns

n_v = 4 Number of bolt rows

h_1 = 40 [mm] Distance between first bolt and upper edge of front plate

Horizontal spacing ei = 80 [mm]

Vertical spacing pi = 70; 80; 70 [mm]

PLATE

h_p = 300 [mm] Plate height

b_p = 160 [mm] Plate width

t_p = 15 [mm] Plate thickness

Material: S275

f_{yp} = 275.00 [MPa] Resistance

Predicted strengths

$M_{b,pl,Rd}$ = 34.1 [kN-m] Plastic resistance of the section for bending (without stiffeners)

$F_{t,Rd}$ = 141.12 [kN] Bolt resistance for tension

$M_{j,Rd}$ = 40.64 [kN-m] Connection resistance for bending

$S_{j,ini}$ = 11108.94 [kN-m] Initial rotational stiffness

S_j = 5554.47 [kN-m] Final rotational stiffness

$M_{pl,Rd,lim}$ = 194.7 [kN-m] Strength of a full resistance connection

Figure B.28. F160x160 test setup

Rotation

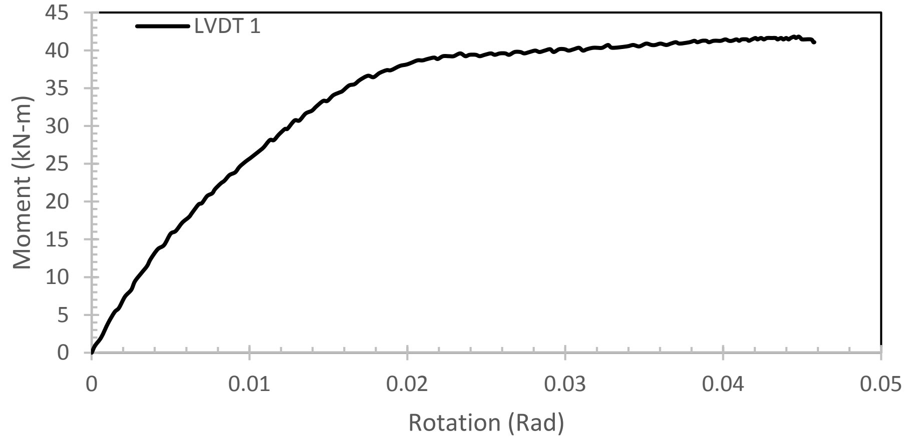


Figure B.29. F160x160 moment-rotation of LVDT 1

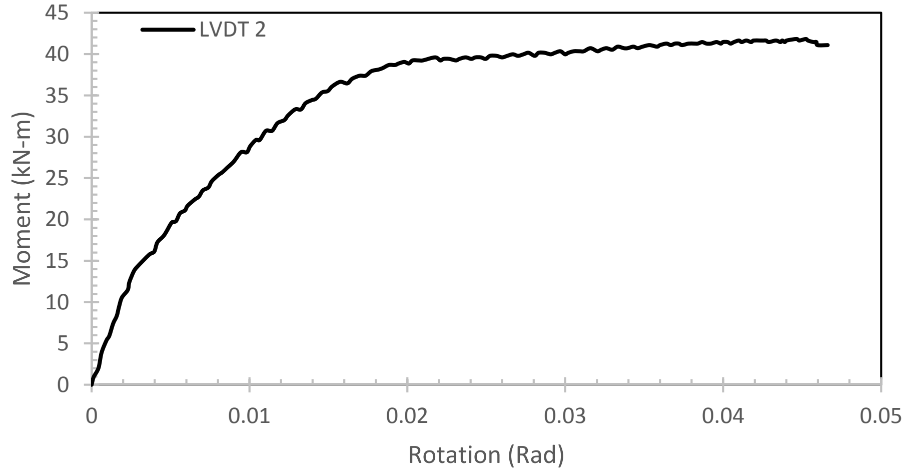


Figure B.30. F160x160 moment-rotation of LVDT 2

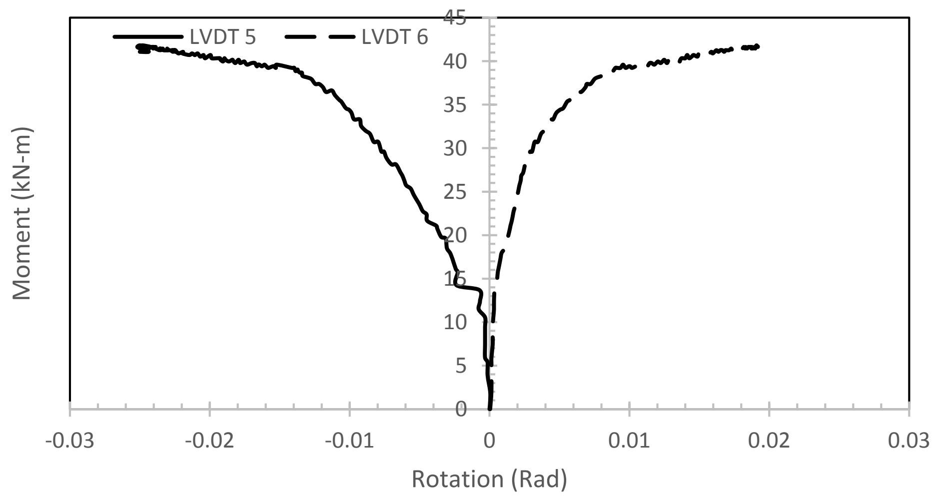


Figure B.31. F160x160 Moment-Rotation of LVDT 5 and 6

Displacement

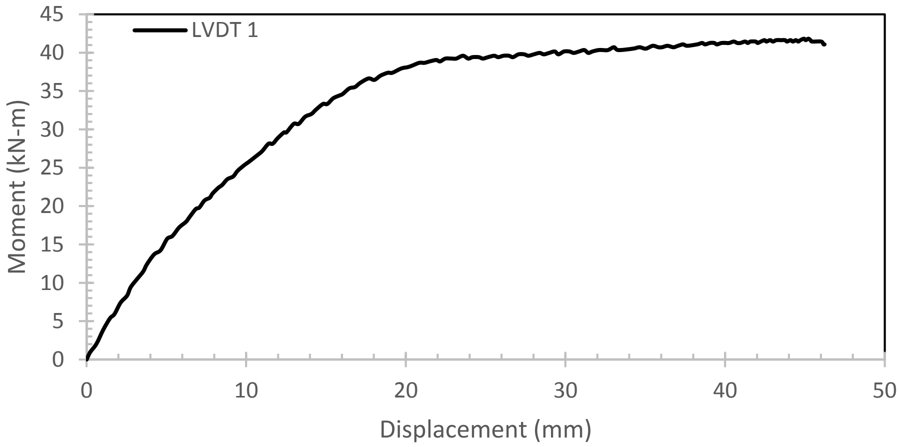


Figure B.32. F160x160 LVDT 1 displacement

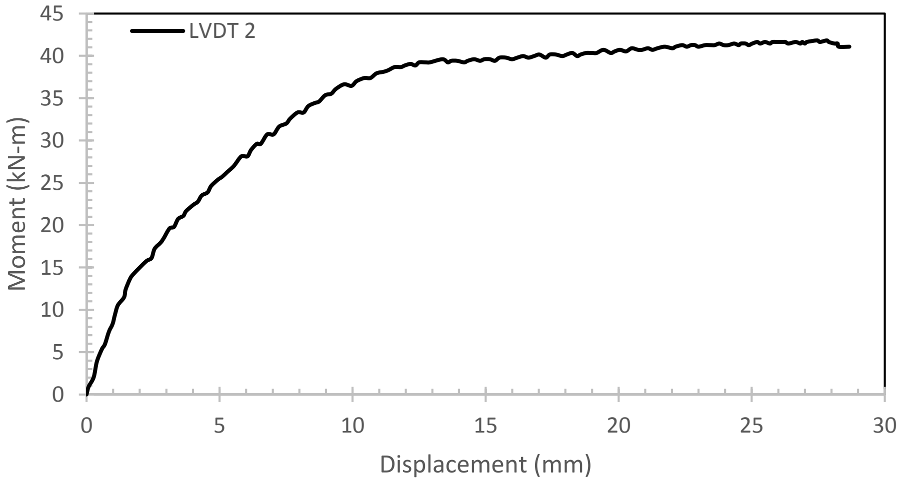


Figure B.33. F160x160 LVDT 2 displacement

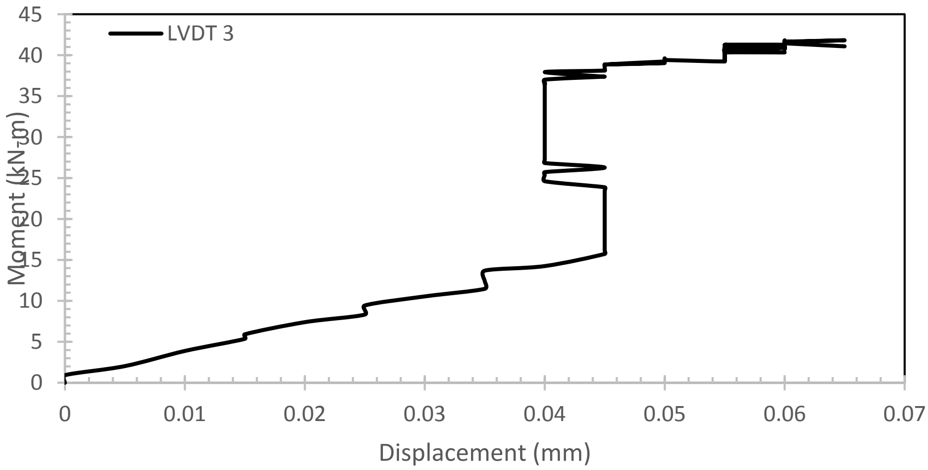


Figure B.34. F160x160 LVDT 3 displacement

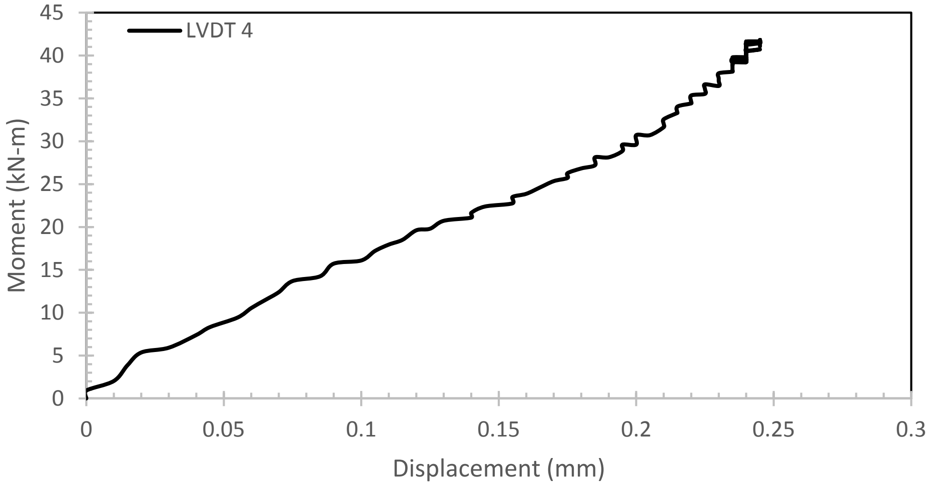


Figure B.35. F160x160 LVDT 4 displacement

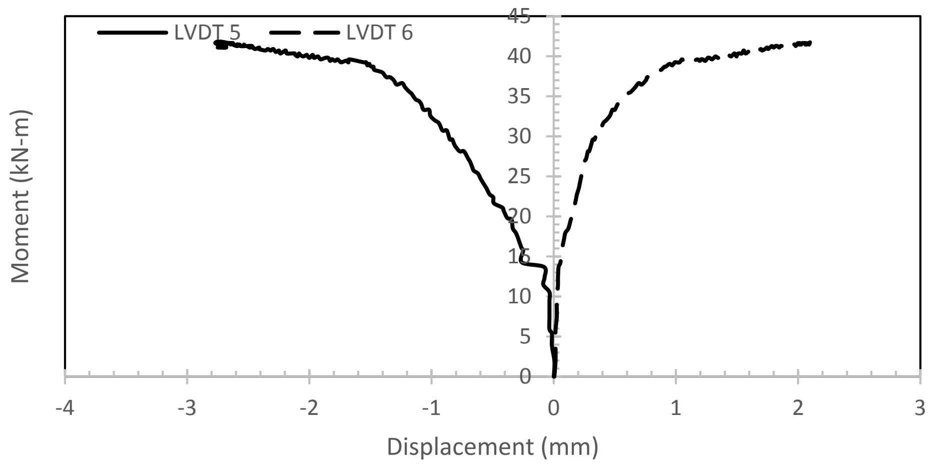


Figure B.36. F160x160 LVDT 5 and 6 displacements

Appendix C: Monotonic Stiffened Fin Plate Connection Test summary sheets

<u>Test Name: W(200x300)</u>

<u>Test Date</u>: 6.7.2015

<u>Column</u>

Section:	HEB 200		
a =	-90.0	[Deg]	Inclination angle
h_c =	200	[mm]	Height of column section
b_{fc} =	200	[mm]	Width of column section
t_{wc} =	9	[mm]	Thickness of the web of column section
t_{fc} =	15	[mm]	Thickness of the flange of column section
r_c =	18	[mm]	Radius of column section fillet
A_c =	7810	[mm^2]	Cross-sectional area of a column
I_{xc} =	56960000	[mm^4]	Moment of inertia of the column section
Material:	S275		
f_{yc} =	275.00	[MPa]	Resistance

<u>Beam</u>

Section:	IPE 300		
a =	0.0	[Deg]	Inclination angle
h_b =	300	[mm]	Height of beam section
b_f =	150	[mm]	Width of beam section
t_{wb} =	7	[mm]	Thickness of the web of beam section
t_{fb} =	11	[mm]	Thickness of the flange of beam section

a = 0.0 [Deg] Inclination angle

r_b = 15 [mm] Radius of beam section fillet

A_b = 5380 [mm^2] Cross-sectional area of a beam

I_{xb} = 83560000 [mm^4] Moment of inertia of the beam section

Material: S275

f_{yb} = 275.00 [MPa] Resistance

Bolts

The shear plane passes through the UNTHREADED portion of the bolt.

d = 16 [mm] Bolt diameter

Class = 8.8 Bolt class

F_{tRd} = 141.12 [kN] Tensile resistance of a bolt

n_h = 1 Number of bolt columns

n_v = 3 Number of bolt rows

h_1 = 40 [mm] Distance between first bolt and upper edge of front plate

Vertical spacing p_i = 70; 70 [mm]

Plate

h_p = 220 [mm] Plate height

b_p = 100 [mm] Plate width

t_p = 10 [mm] Plate thickness

Material: S275

f_{yp} = 275.00 [MPa] Resistance

Predicted strengths

$M_{b,pl,Rd}$ = 172.70 [kN-m] Plastic resistance of the section for bending (without stiffeners)

$F_{t,Rd}$ = 141.12 [kN] Bolt resistance for tension

$M_{j,Rd}$ = 132.40 [kN-m] Connection resistance for bending

$S_{j,ini}$ = 41074.98 [kN-m] Initial rotational stiffness

S_j = 20537.49 [kN-m] Final rotational stiffness

$M_{pl,Rd,lim}$ = 353.65 [kN-m] Strength of a full resistance connection

Figure C.1. F200x300 connection Setup

Rotation

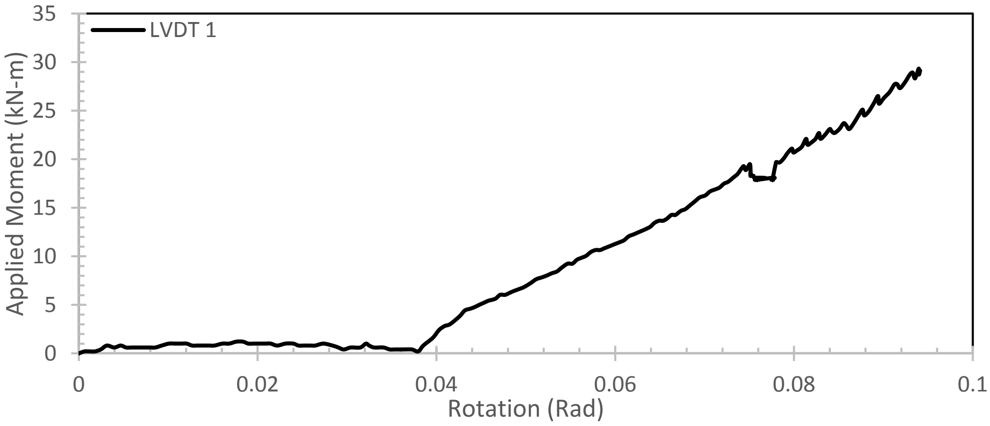


Figure C.2. F200x300 moment-rotation of LVDT 1

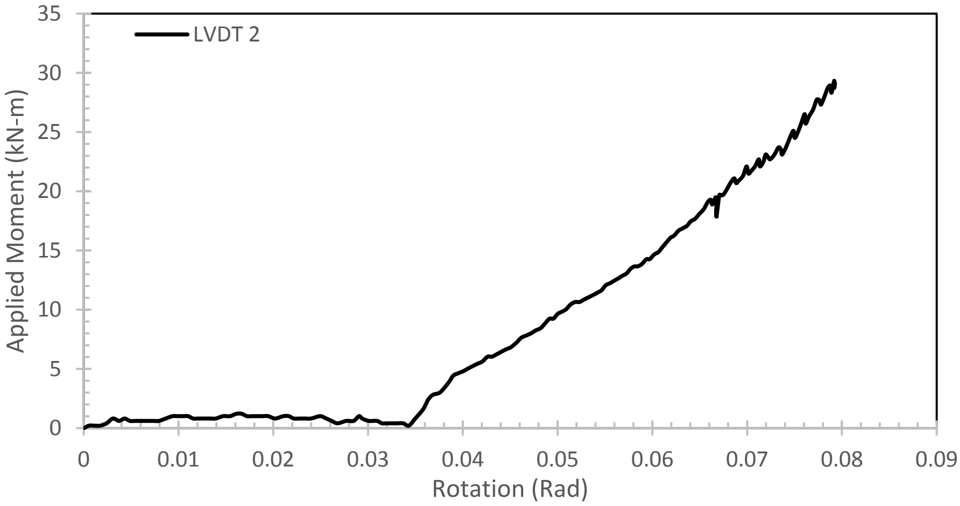


Figure C.3. F200x300 Moment-Rotation of LVDT 2

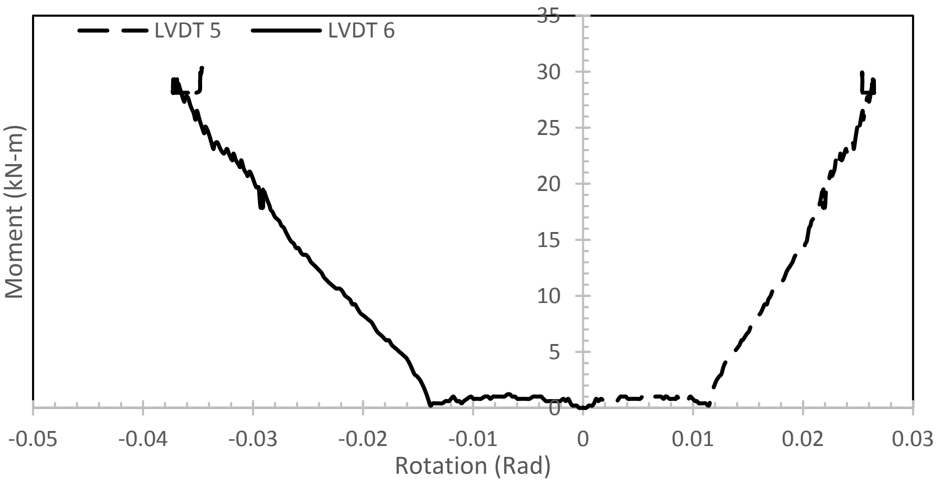


Figure C.4. F200x300 moment-rotation of LVDT 5 and 6

Displacement

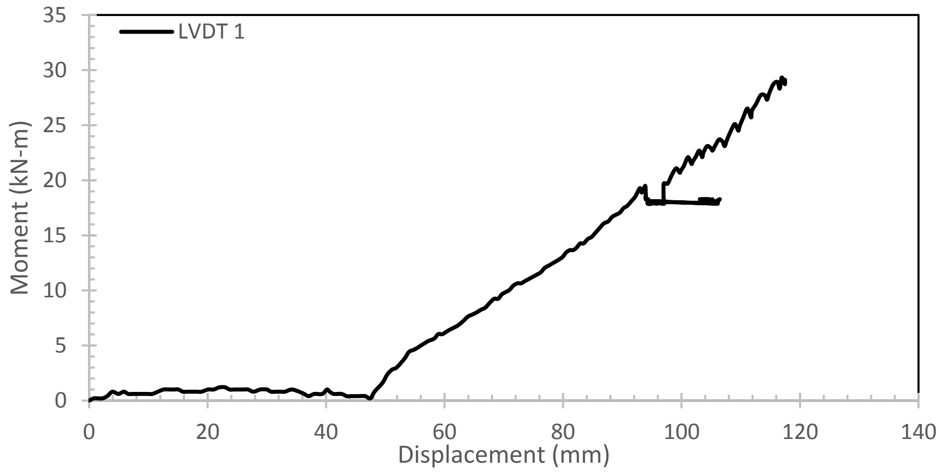


Figure C.5. F200x300 LVDT 1 displacement

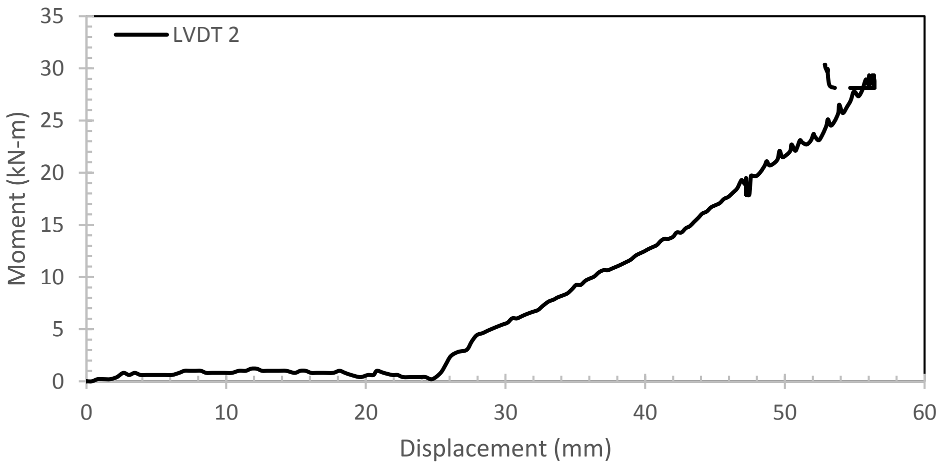


Figure C.6. F200x300 LVDT 2 displacement

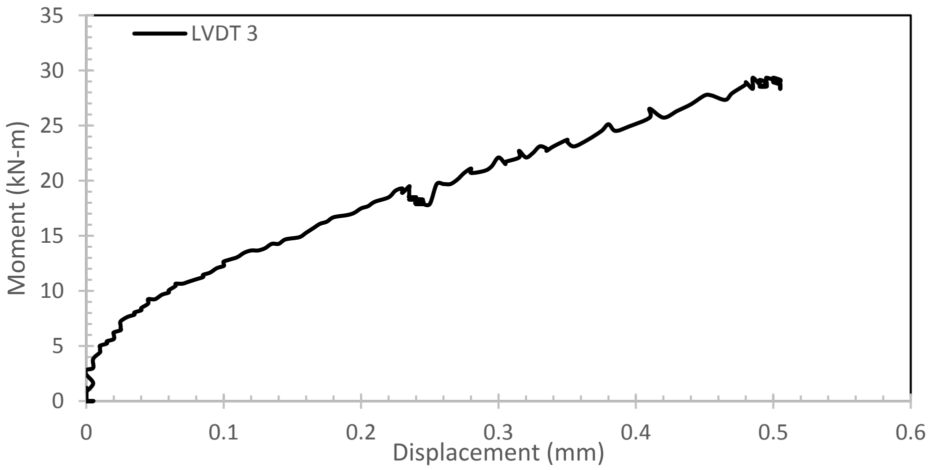


Figure C.7. F200x300 LVDT 3 displacement

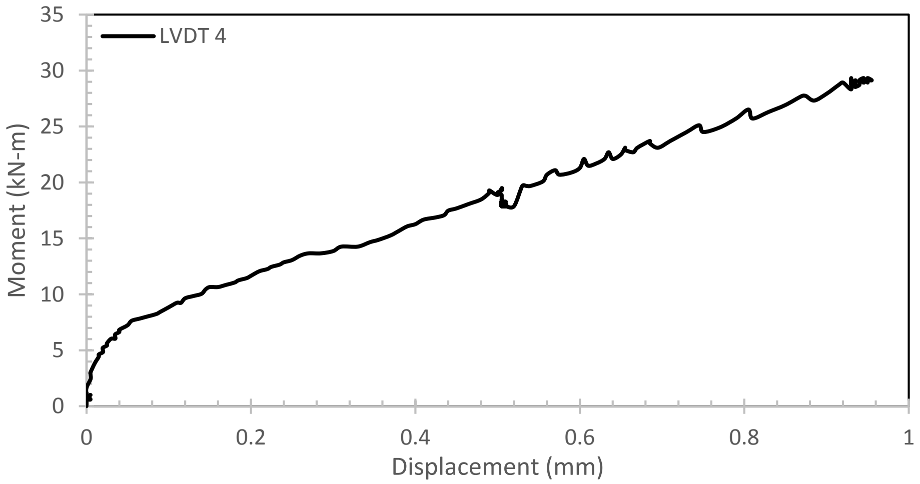


Figure C.8. F200x300 LVDT 4 displacement

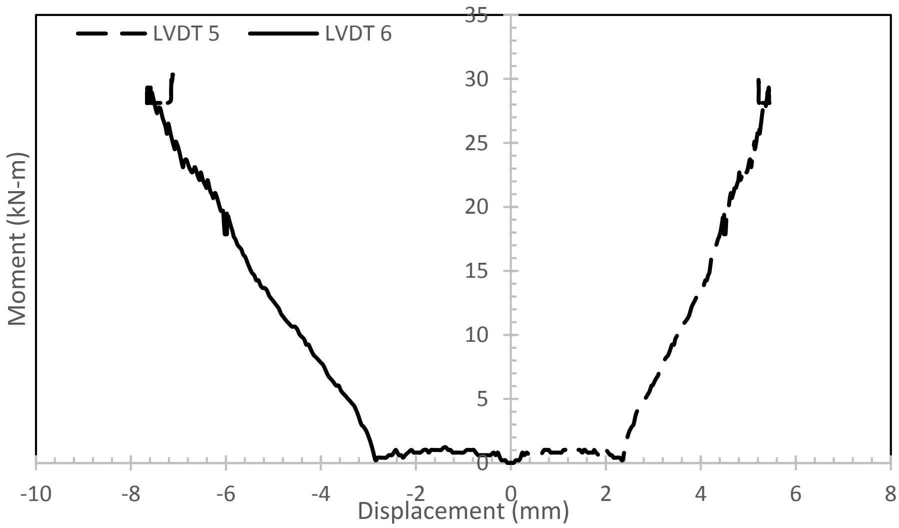


Figure C.9. F200x300 LVDT 5 and 6 displacement

Test Name: W(200x200)

Test Date: **6.7.2015**

COLUMN

Section:		HEB 200	
a =	-90.0	[Deg]	Inclination angle
h_c =	200	[mm]	Height of column section
b_{fc} =	200	[mm]	Width of column section
t_{wc} =	9	[mm]	Thickness of the web of column section
t_{fc} =	15	[mm]	Thickness of the flange of column section
r_c =	18	[mm]	Radius of column section fillet
A_c =	7810	[mm^2]	Cross-sectional area of a column
I_{xc} =	56960000		[mm4] Moment of inertia of the column section
Material:		S275	
f_{yc} =		275.00 [MPa]	Resistance

BEAM

Section:		IPE 200	
a =	0.0	[Deg]	Inclination angle
h_b =	200	[mm]	Height of beam section
b_f =	100	[mm]	Width of beam section
t_{wb} =	6	[mm]	Thickness of the web of beam section
t_{fb} =	9	[mm]	Thickness of the flange of beam section
r_b =	12	[mm]	Radius of beam section fillet
A_b =	2850	[mm^2]	Cross-sectional area of a beam
I_{xb} =	19430000		[mm^4] Moment of inertia of the beam section
Material:		S275	

f_{yb} = 275.00 [MPa] Resistance

Bolts

The shear plane passes through the UNTHREADED portion of the bolt.

d = 16 [mm] Bolt diameter

Class = 8.8 Bolt class

F_{tRd} = 141.12 [kN] Tensile resistance of a bolt

n_h = 1 Number of bolt columns

n_v = 2 Number of bolt rows

h_1 = 40 [mm] Distance between first bolt and upper edge of front plate

Vertical spacing p_i = 70 [mm]

Plate

h_p = 150 [mm] Plate height

b_p = 100 [mm] Plate width

t_p = 10 [mm] Plate thickness

Material: S275

f_{yp} = 275.00 [MPa] Resistance

Predicted strengths

$M_{b,pl,Rd}$ = 60.77 [kN-m] Plastic resistance of the section for bending (without stiffeners)

$F_{t,Rd}$ = 90.43 [kN] Bolt resistance for tension

$M_{j,Rd}$ = 49.87 [kN-m] Connection resistance for bending

$S_{j,ini}$ = 15732.53 [kN-.m] Initial rotational stiffness

S_j = 5284.82 [kN.-m] Final rotational stiffness

Figure C.10. W200x200 connection Setup

Rotation

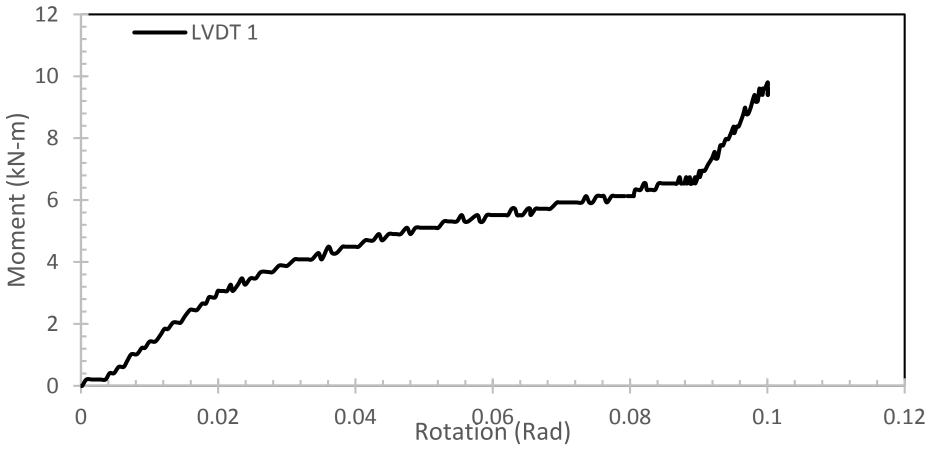


Figure C.11. W200x200 moment-rotation of LVDT 1

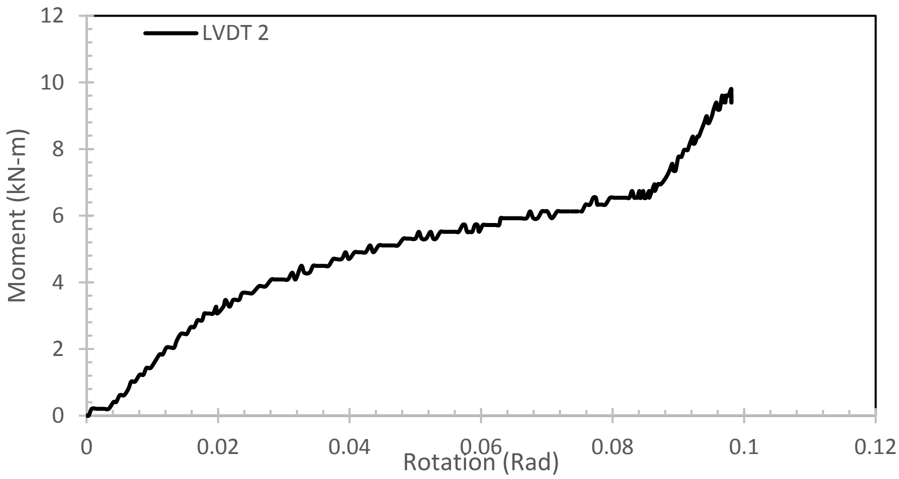


Figure C.12. W200x200 moment-rotation of LVDT 2

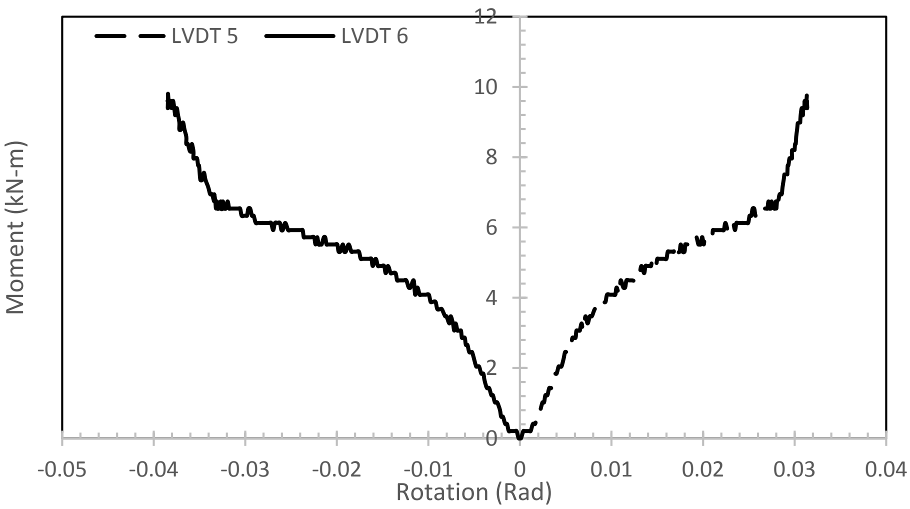


Figure C.13. W200x200 moment-rotation of LVDT 5 and 6

Displacement

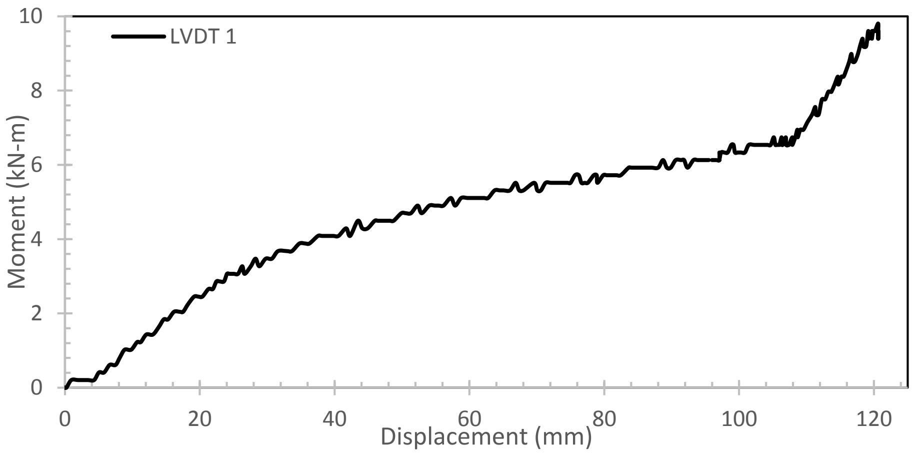


Figure C.14. W200x200 LVDT 1 displacement

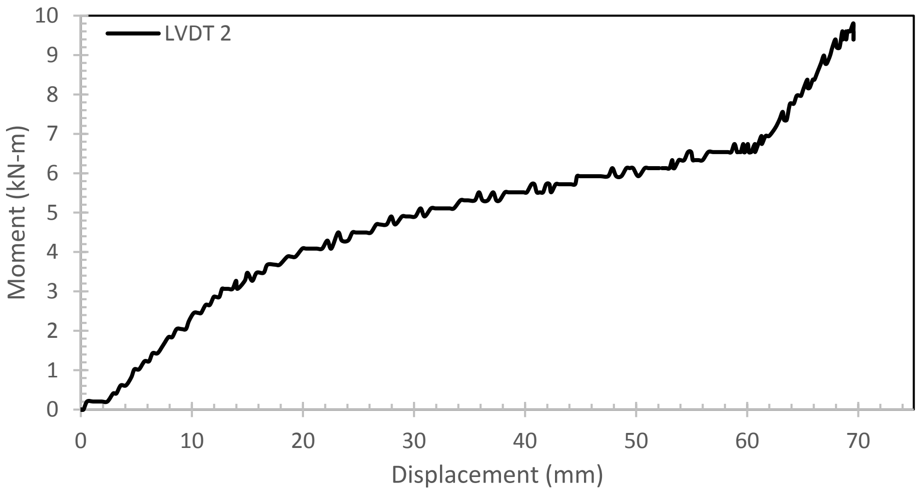


Figure C.15. W200x200 LVDT 2 displacement

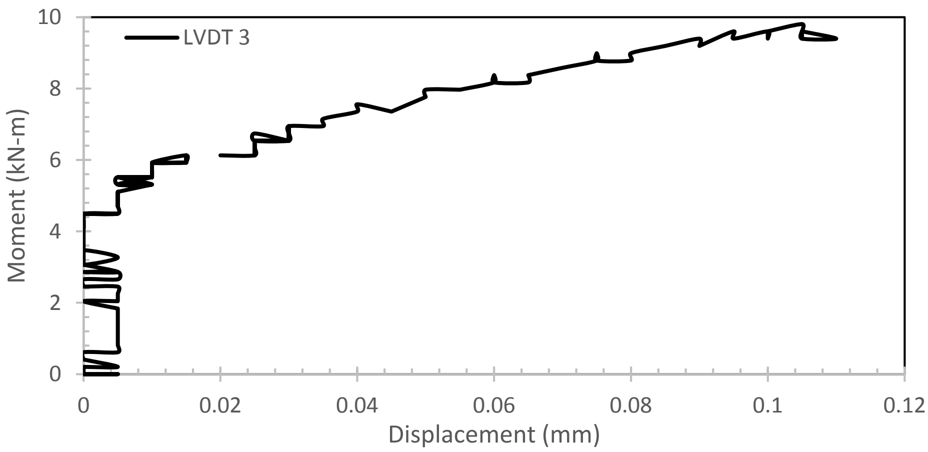


Figure C.16. W200x200 LVDT 3 displacement

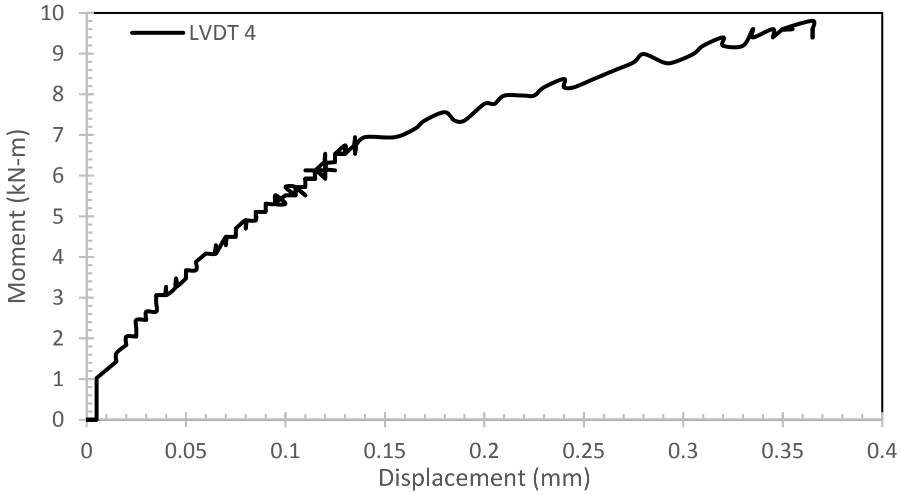


Figure C.17. W200x200 LVDT 4 displacement

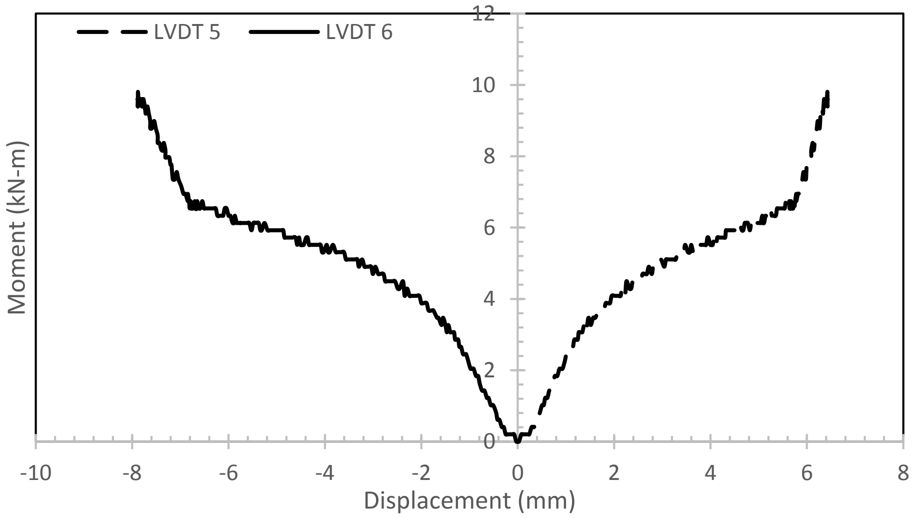


Figure C.18. W200x200 LVDT 5 and 6 displacement

Test Name: W(160x200)

Test Date: **7.7.2015**

COLUMN

Section:		HEB 160	
a =	-90.0	[Deg]	Inclination angle
h_c =	160	[mm]	Height of column section
b_{fc} =	160	[mm]	Width of column section
t_{wc} =	8	[mm]	Thickness of the web of column section
t_{fc} =	13	[mm]	Thickness of the flange of column section
r_c =	15	[mm]	Radius of column section fillet
A_c =	5430	[mm^2]	Cross-sectional area of a column
I_{xc} =	24920000	[mm^4]	Moment of inertia of the column section
Material:		S275	
f_{yc} =	275.00	[MPa]	Resistance

BEAM

Section:		IPE 200	
a =	0.0	[Deg]	Inclination angle
h_b =	200	[mm]	Height of beam section
b_f =	100	[mm]	Width of beam section
t_{wb} =	6	[mm]	Thickness of the web of beam section
t_{fb} =	9	[mm]	Thickness of the flange of beam section
r_b =	12	[mm]	Radius of beam section fillet
A_b =	2850	[mm^2]	Cross-sectional area of a beam
Ixb =	19430000	[mm^4]	Moment of inertia of the beam section
Material:		S275	

f_{yb} = 275.00 [MPa] Resistance

Bolts

The shear plane passes through the UNTHREADED portion of the bolt.

d = 16 [mm] Bolt diameter

Class = 8.8 Bolt class

F_{tRd} = 141.12 [kN] Tensile resistance of a bolt

n_h = 1 Number of bolt columns

n_v = 2 Number of bolt rows

h_1 = 35 [mm] Distance between first bolt and upper edge of front plate

Vertical spacing p_i = 60 [mm]

Plate

h_p = 130 [mm] Plate height

b_p = 100 [mm] Plate width

t_p = 10 [mm] Plate thickness

Material: S275

f_{yp} = 275.00 [MPa] Resistance

Predicted strengths

$M_{b,pl,Rd}$ = 60.77 [kN-m] Plastic resistance of the section for bending (without stiffeners)

$F_{t,Rd}$ = 90.43 [kN] Bolt resistance for tension

$M_{j,Rd}$ = 46.9 [kN-m] Connection resistance for bending

$S_{j,ini}$ = 13447.59 [kN-m] Initial rotational stiffness

S_j = 6723.8 [kN-m] Final rotational stiffness

$M_{pl,Rd,lim}$ = 194.7 [kN-m] Strength of a full resistance connection


Figure C.19. W160x200 connection setup

Rotation

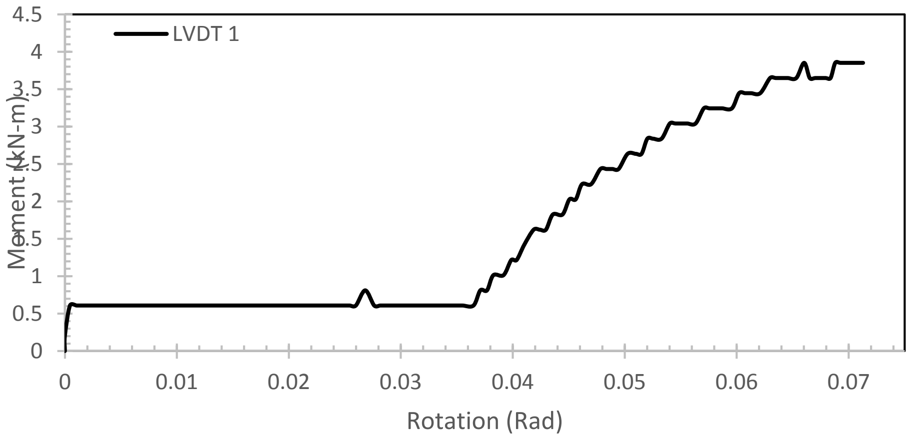


Figure C.20. W160x200 moment-rotation of LVDT 1

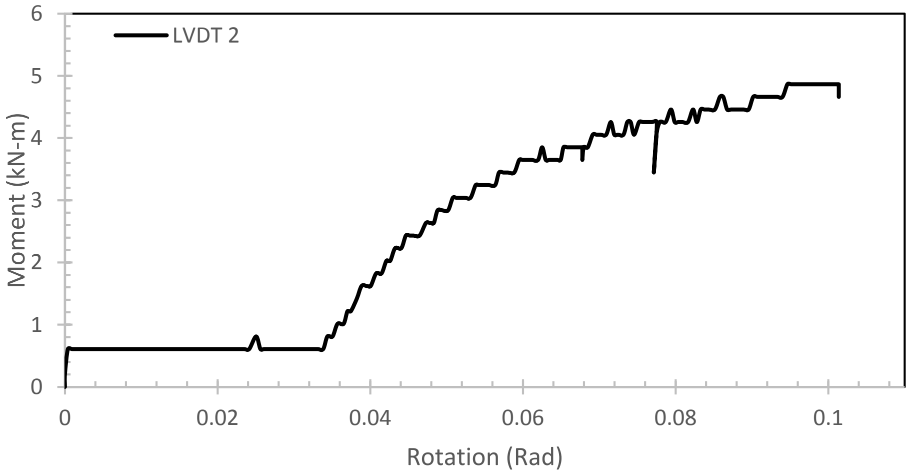


Figure C.21. W160x200 moment-rotation of LVDT 2

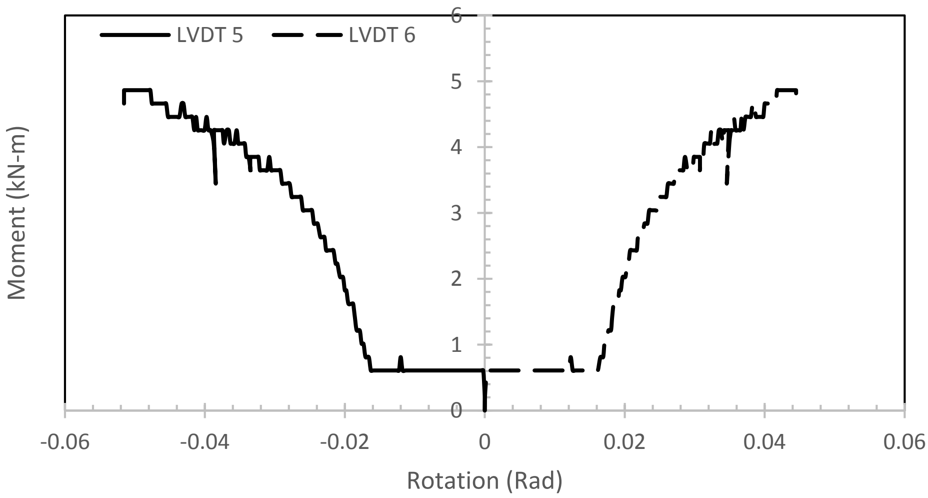


Figure C.22. W160x200 moment-rotation of LVDT 5 and 6

Displacement

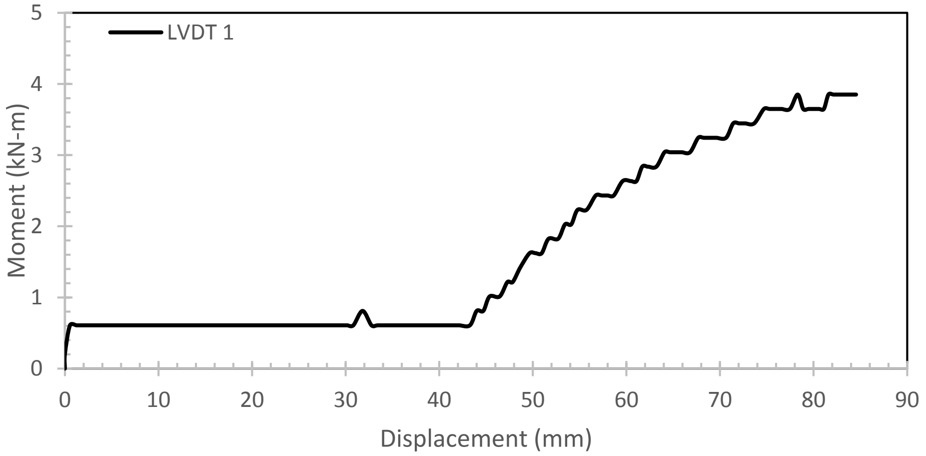


Figure C.23. W160x200 LVDT 1 displacement

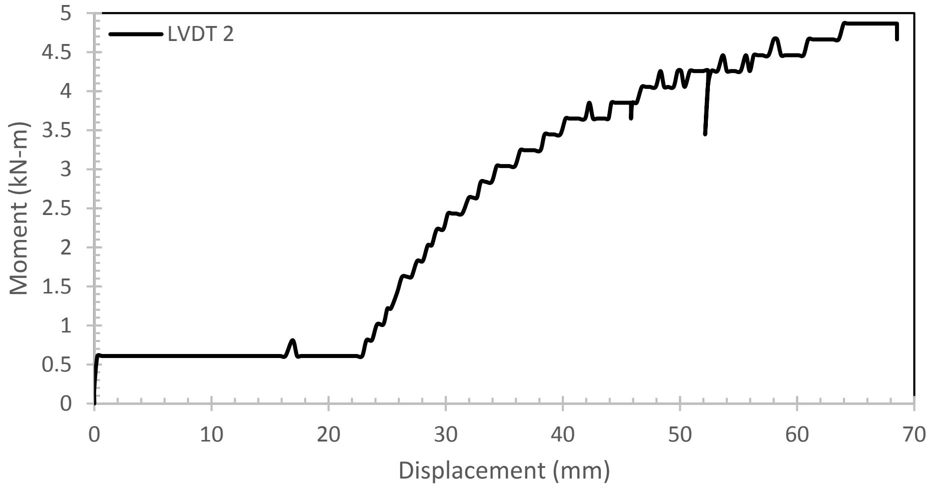


Figure C.24. W160x200 LVDT 2 displacement

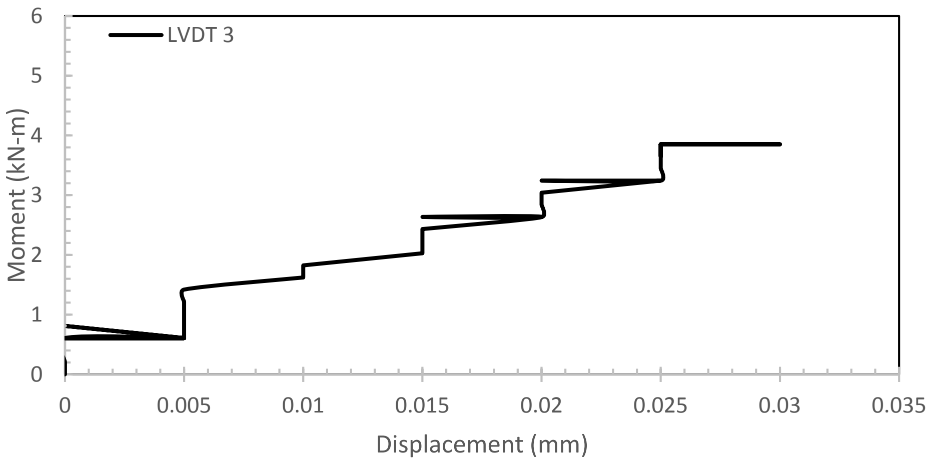


Figure C.25. W160x200 LVDT 3 displacement

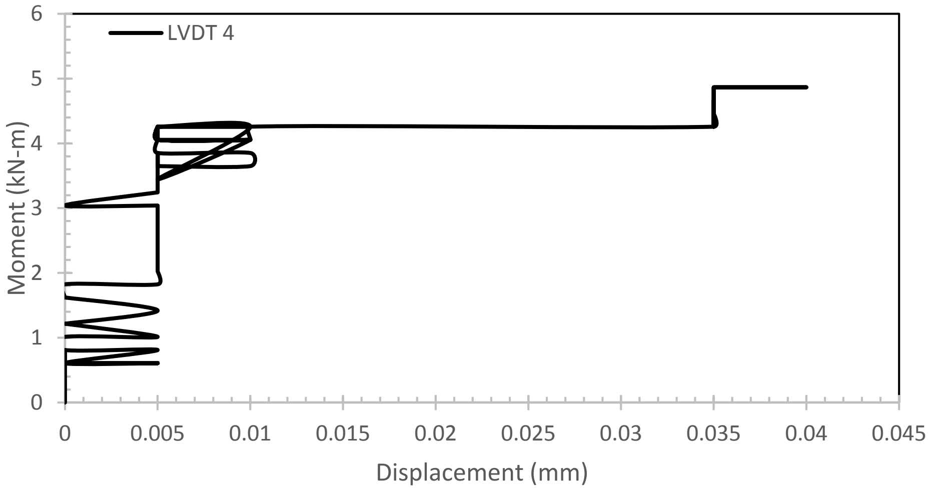


Figure C.26. W160x200 LVDT 4 displacement

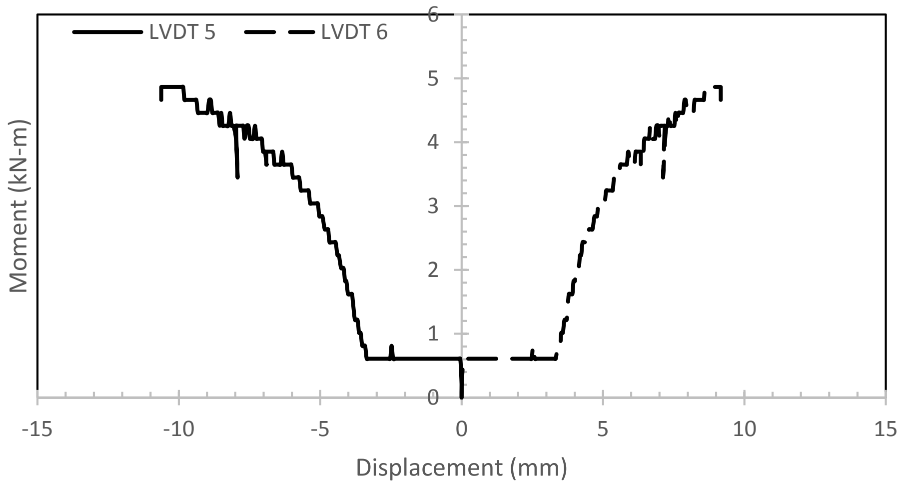


Figure C.27. W160x200 LVDT 5 and 6 displacements

Test Name: W(160x160)

Test Date: **7.7.2015**

COLUMN

Section:		HEB 160	
a =	-90.0	[Deg]	Inclination angle
h_c =	160	[mm]	Height of column section
b_{fc} =	160	[mm]	Width of column section
t_{wc} =	8	[mm]	Thickness of the web of column section
t_{fc} =	13	[mm]	Thickness of the flange of column section
r_c =	15	[mm]	Radius of column section fillet
A_c =	5430	[mm^2]	Cross-sectional area of a column
I_{xc} =	24920000	[mm^4]	Moment of inertia of the column section
Material:		S275	
f_{yc} =		275.00 [MPa]	Resistance

BEAM

Section:		IPE 160	
a =	0.0	[Deg]	Inclination angle
h_b =	160	[mm]	Height of beam section
b_f =	82	[mm]	Width of beam section
t_{wb} =	5	[mm]	Thickness of the web of beam section
t_{fb} =	7	[mm]	Thickness of the flange of beam section
r_b =	9	[mm]	Radius of beam section fillet
A_b =	2010	[mm^2]	Cross-sectional area of a beam
I_{xb} =	8690000	[mm^4]	Moment of inertia of the beam section
Material:		S275	

f_{yb} = 275.00 [MPa] Resistance

Bolts

The shear plane passes through the UNTHREADED portion of the bolt.

d = 16 [mm] Bolt diameter

Class = 8.8 Bolt class

F_{tRd} = 141.12 [kN] Tensile resistance of a bolt

n_h = 1 Number of bolt columns

n_v = 2 Number of bolt rows

h_1 = 35 [mm] Distance between first bolt and upper edge of front plate

Vertical spacing p_i = 60 [mm]

Plate

h_p = 130 [mm] Plate height

b_p = 100 [mm] Plate width

t_p = 10 [mm] Plate thickness

Material: S275

f_{yp} = 275.00 [MPa] Resistance

Predicted strengths

$M_{b,pl,Rd}$ = 34.1 [kN-m] Plastic resistance of the section for bending (without stiffeners)

$F_{t,Rd}$ = 141.12 [kN] Bolt resistance for tension

$M_{j,Rd}$ = 40.64 [kN-m] Connection resistance for bending

$S_{j,ini}$ = 11108.94 [kN-m] Initial rotational stiffness

S_j = 5554.47 [kN-m] Final rotational stiffness

$M_{pl,Rd,lim}$ = 194.7 [kN-m] Strength of a full resistance connection

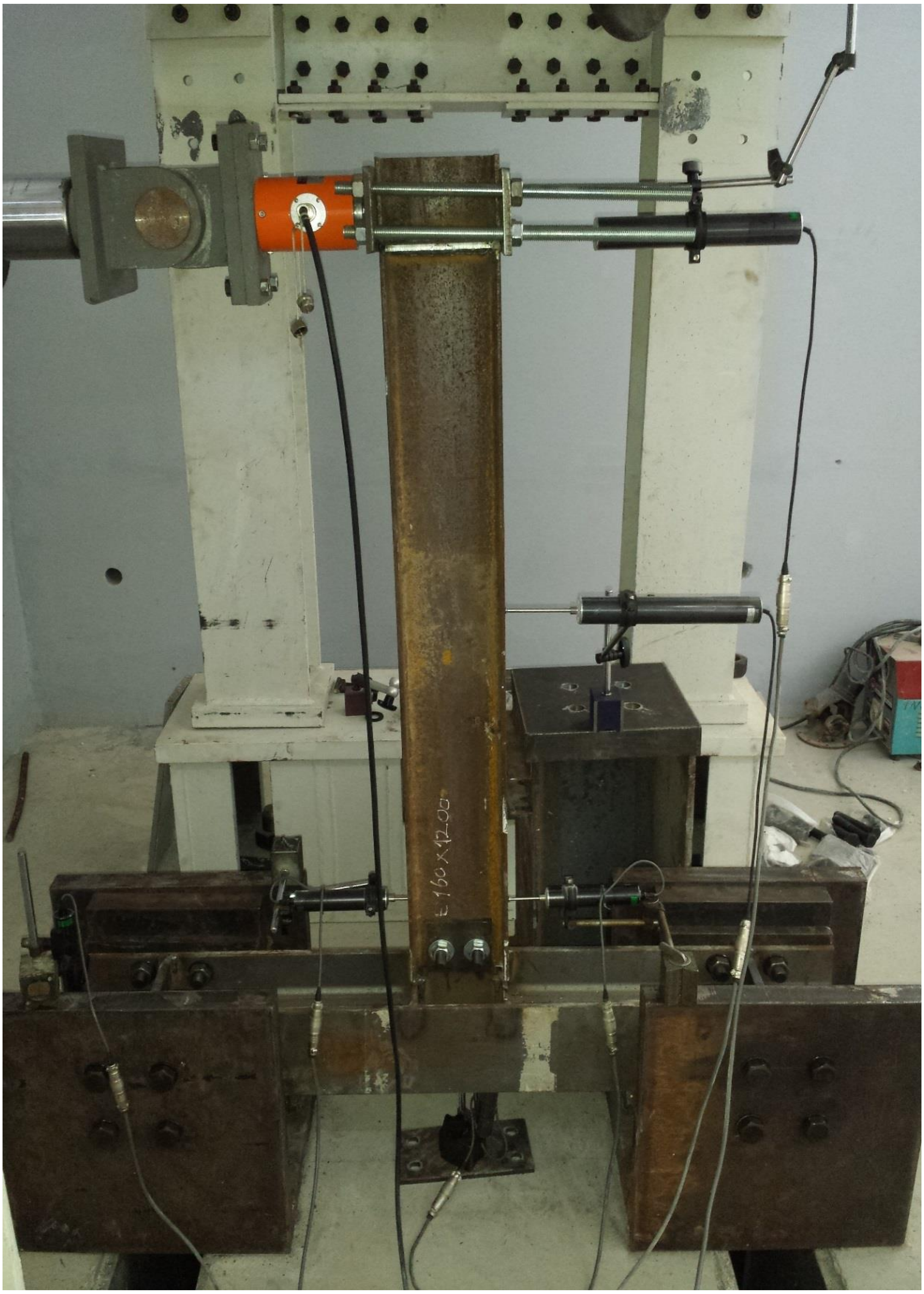


Figure C.28. W160x160 connection setup

Rotatio

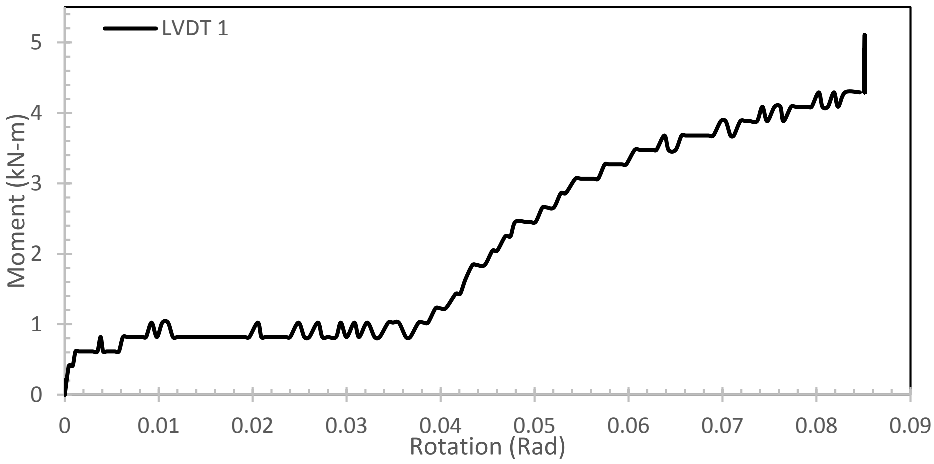


Figure C.29. W160x160 moment-rotation of LVDT 1

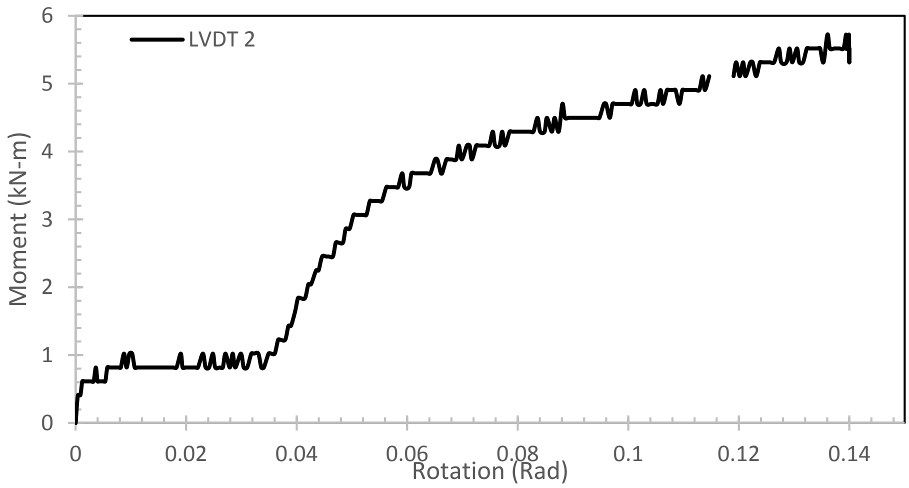


Figure C.30. W160x160 moment-rotation of LVDT 2

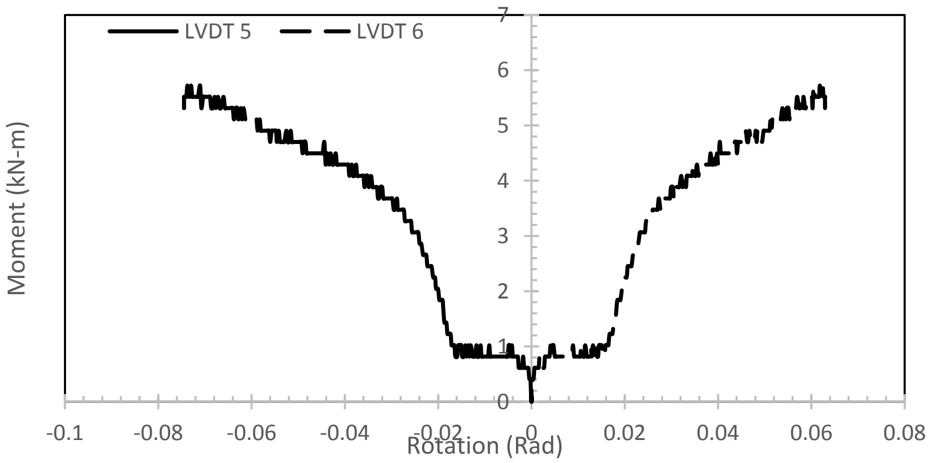


Figure C.31. W160x160 moment-rotation of LVDT 5 and 6

Displacement

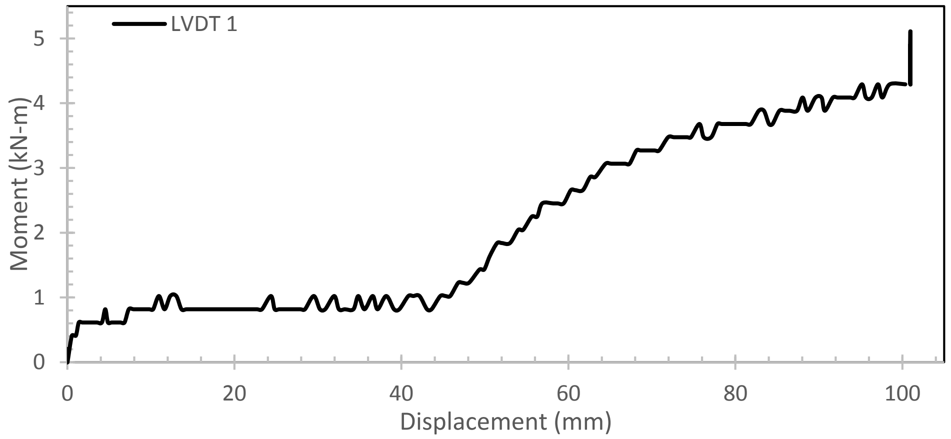


Figure C.32. W160x160 LVDT 1 displacement

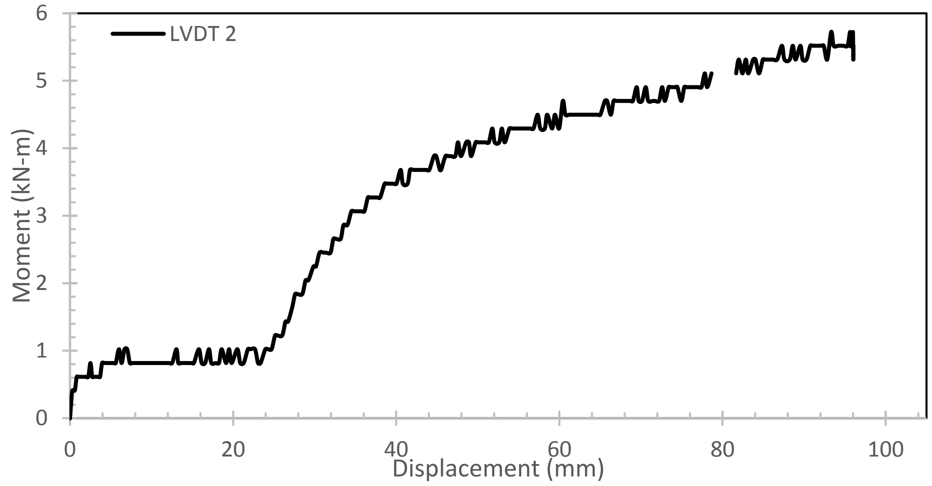


Figure C.33. W160x160 LVDT 2 displacement

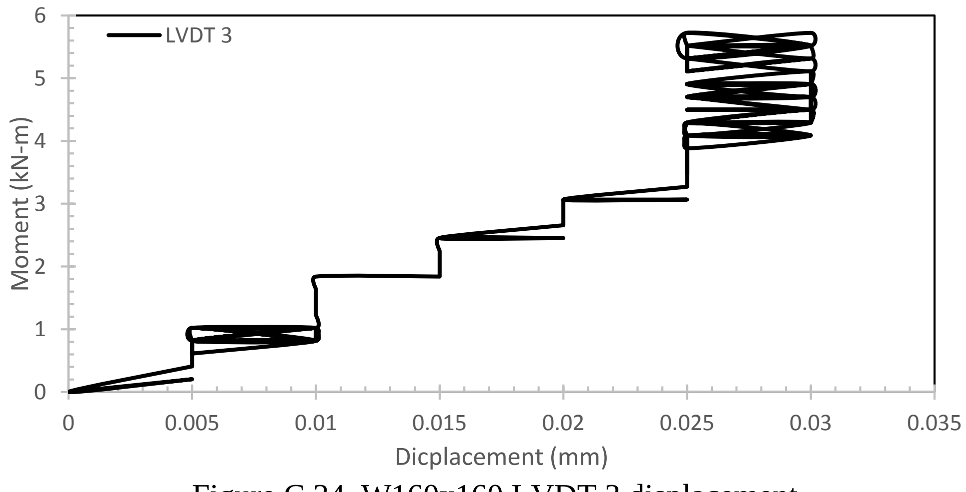


Figure C.34. W160x160 LVDT 3 displacement

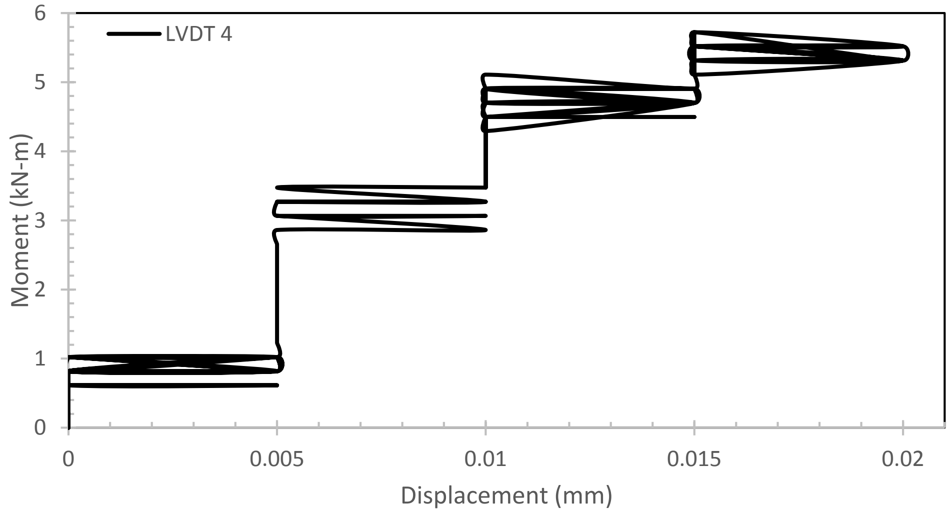


Figure C.35. W160x160 LVDT 4 displacement

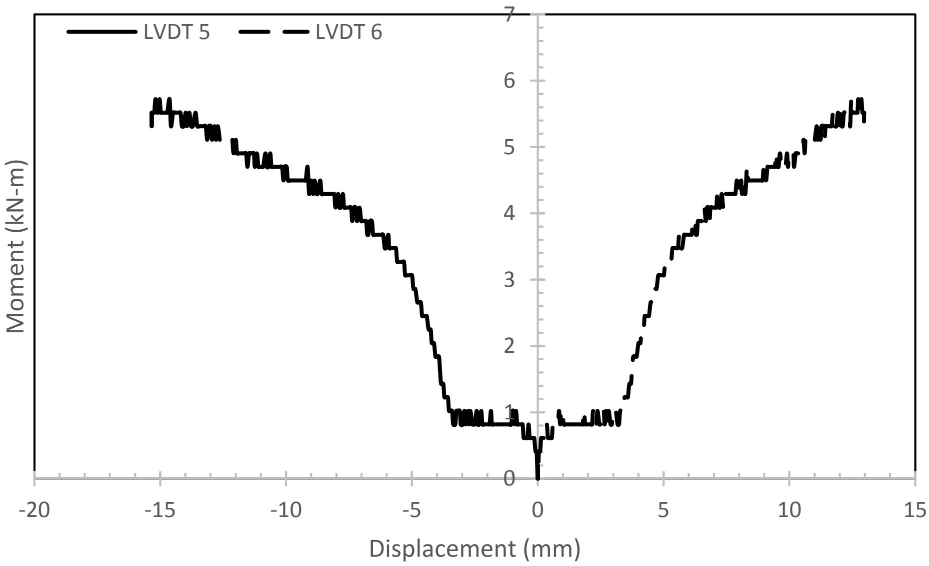
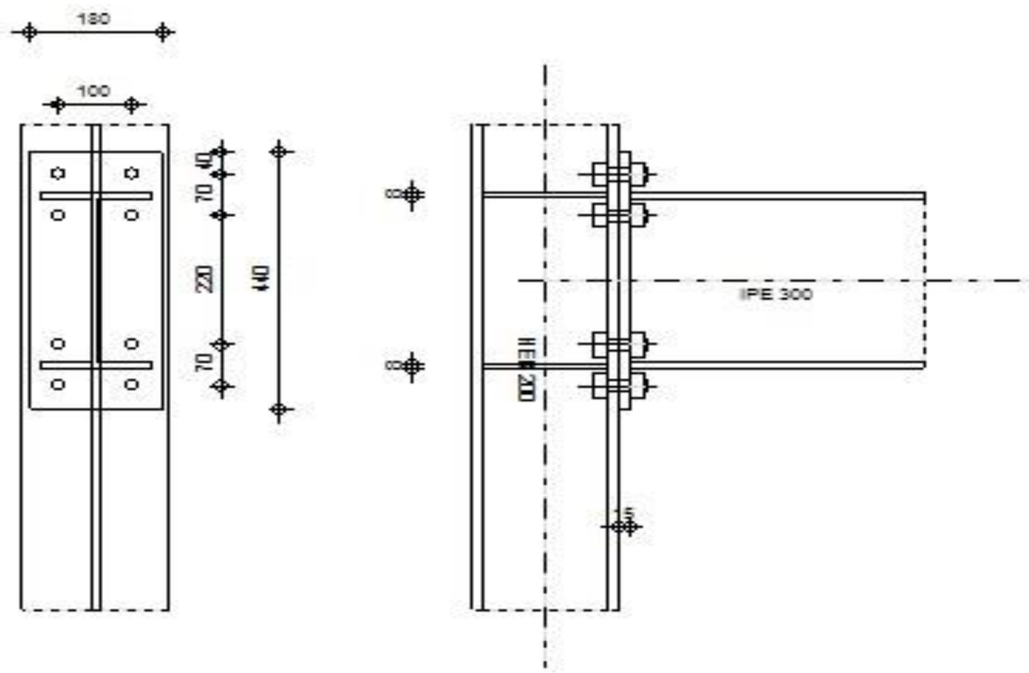


Figure C.36. W160x160 LVDT 5 and 6 displacements

Appendix D: Sample of Connection Design by Robot Structural

	**Autodesk Robot Structural Analysis Professional 2015** **Design of fixed beam-to-column connection** **EN 1993-1-8:2005/AC:2009**	OK Ratio **0.99**



General

Connection no.: 1

Connection name: Column-Beam

Geometry

Column

Section: HEB 200

α = -90.0 [Deg] Inclination angle

h_c = 200 [mm] Height of column section

Section: HEB 200

α =	-90.0	[Deg]	Inclination angle
b_{fc} =	200	[mm]	Width of column section
t_{wc} =	9	[mm]	Thickness of the web of column section
t_{fc} =	15	[mm]	Thickness of the flange of column section
r_c =	18	[mm]	Radius of column section fillet
A_c =	7810	[mm^2]	Cross-sectional area of a column
I_{xc} =	56960000	[mm^4]	Moment of inertia of the column section

Material: S275

f_{yc} = 275.00 [MPa] Resistance

Beam

Section: IPE 300

α =	0.0	[Deg]	Inclination angle
h_b =	300	[mm]	Height of beam section
b_f =	150	[mm]	Width of beam section
t_{wb} =	7	[mm]	Thickness of the web of beam section
t_{fb} =	11	[mm]	Thickness of the flange of beam section
r_b =	15	[mm]	Radius of beam section fillet
r_b =	15	[mm]	Radius of beam section fillet
A_b =	5380	[mm^2]	Cross-sectional area of a beam
I_{xb} =	83560000	[mm^4]	Moment of inertia of the beam section

Material: S275

f_{yb} = 275.00 [MPa] Resistance

Bolts

The shear plane passes through the UNTHREADED portion of the bolt.

d = 20 [mm] Bolt diameter

Class = 8.8 Bolt class

F_{tRd} = 141.12 [kN] Tensile resistance of a bolt

n_h = 2 Number of bolt columns

n_v = 4 Number of bolt rows

h_1 = 40 [mm] Distance between first bolt and upper edge of front plate

Horizontal spacing e_i = 100 [mm]

Vertical spacing p_i = 70;220;70 [mm]

Plate

h_p = 440 [mm] Plate height

b_p = 180 [mm] Plate width

t_p = 15 [mm] Plate thickness

Material: S275

f_{yp} = 275.00 [MPa] Resistance

Column stiffener

Upper

h_{su} = 170 [mm] Stiffener height

b_{su} = 96 [mm] Stiffener width

t_{hu} = 8 [mm] Stiffener thickness

Material: S275

f_{ysu} = 275.00 [MPa] Resistance

Lower

h_{sd} = 170 [mm] Stiffener height

b_{sd} = 96 [mm] Stiffener width

h_{sd} = 170 [mm] Stiffener height

t_{hd} = 8 [mm] Stiffener thickness

Material: S275

f_{ysu} = 275.00 [MPa] Resistance

Fillet welds

a_w = 8 [mm] Web weld

a_f = 8 [mm] Flange weld

a_s = 8 [mm] Stiffener weld

Material factors

γ_{M0} =	1.00	Partial safety factor	[2.2]
γ_{M1} =	1.00	Partial safety factor	[2.2]
γ_{M2} =	1.25	Partial safety factor	[2.2]
γ_{M3} =	1.25	Partial safety factor	[2.2]

Loads

Ultimate limit state

Case: Manual calculations.

$M_{b1,Ed}$ = 110.00 [kN*m] Bending moment in the right beam

$V_{b1,Ed}$ = 72.00 [kN] Shear force in the right beam

Results

Beam resistances

SHEAR

A_{vb} = 2567 [mm^2] Shear area EN1993-1-1:[6.2.6.(3)]

$V_{cb,Rd} = A_{vb} (f_{yb} / \sqrt{3}) / \gamma_{M0}$

$V_{cb,Rd}$ = 407.56 [kN] Design sectional resistance for shear EN1993-1-1:[6.2.6.(2)]

$V_{b1,Ed} / V_{cb,Rd} \leq 1,0$	0.18 < 1.00	verified	(0.18)

BENDING - PLASTIC MOMENT (WITHOUT BRACKETS)

W_{plb} = 628000 [mm³] Plastic section modulus EN1993-1-1:[6.2.5.(2)]

$M_{b,pl,Rd} = W_{plb}\, f_{yb} / \gamma_{M0}$

$M_{b,pl,Rd}$ = 172.70 [kN*m] Plastic resistance of the section for bending (without stiffeners) EN1993-1-1:[6.2.5.(2)]

BENDING ON THE CONTACT SURFACE WITH PLATE OR CONNECTED ELEMENT

W_{pl} = 628000 [mm³] Plastic section modulus EN1993-1-1:[6.2.5]

$M_{cb,Rd} = W_{pl}\, f_{yb} / \gamma_{M0}$

$M_{cb,Rd}$ = 172.70 [kN*m] Design resistance of the section for bending EN1993-1-1:[6.2.5]

FLANGE AND WEB - COMPRESSION

$M_{cb,Rd}$ = 172.70 [kN*m] Design resistance of the section for bending EN1993-1-1:[6.2.5]

h_f = 289 [mm] Distance between the centroids of flanges [6.2.6.7.(1)]

$F_{c,fb,Rd} = M_{cb,Rd} / h_f$

$F_{c,fb,Rd}$ = 596.96 [kN] Resistance of the compressed flange and web [6.2.6.7.(1)]

Column resistances

WEB PANEL - SHEAR

$M_{b1,Ed}$ = 110.00 [kN*m] Bending moment (right beam) [5.3.(3)]

$M_{b2,Ed}$ = 0.00 [kN*m] Bending moment (left beam) [5.3.(3)]

$V_{c1,Ed}$ = 0.00 [kN] Shear force (lower column) [5.3.(3)]

WEB PANEL - SHEAR

$M_{b1,Ed}$ =	110.00	[kN*m]	Bending moment (right beam)	[5.3.(3)]
$V_{c2,Ed}$ =	0.00	[kN]	Shear force (upper column)	[5.3.(3)]
z =	290	[mm]	Lever arm	[6.2.5]

$V_{wp,Ed} = (M_{b1,Ed} - M_{b2,Ed}) / z - (V_{c1,Ed} - V_{c2,Ed}) / 2$

$V_{wp,Ed}$ =	379.77	[kN]	Shear force acting on the web panel	[5.3.(3)]

A_{vs} =	2485	[mm^2]	Shear area of the column web	EN1993-1-1: [6.2.6.(3)]
A_{vc} =	2485	[mm^2]	Shear area	EN1993-1-1: [6.2.6.(3)]
d_s =	292	[mm]	Distance between the centroids of stiffeners	[6.2.6.1.(4)]
$M_{pl,fc,Rd}$ =	3.09	[kN*m]	Plastic resistance of the column flange for bending	[6.2.6.1.(4)]
$M_{pl,stu,Rd}$ =	0.88	[kN*m]	Plastic resistance of the upper transverse stiffener for bending	[6.2.6.1.(4)]
$M_{pl,stl,Rd}$ =	0.88	[kN*m]	Plastic resistance of the lower transverse stiffener for bending	[6.2.6.1.(4)]

$V_{wp,Rd} = 0.9\ (A_{vs}*f_{y,wc}) / (\sqrt{3}\ \gamma_{M0}) + Min(4\ M_{pl,fc,Rd} / d_s\ ,\ (2\ M_{pl,fc,Rd} + M_{pl,stu,Rd} + M_{pl,stl,Rd}) / d_s)$

$V_{wp,Rd}$ =	382.3	[kN]	Resistance of the column web panel for shear	[6.2.6.1]

$V_{wp,Ed} / V_{wp,Rd} \le 1,0$	0.99 < 1.00	verified (0.99)

WEB - TRANSVERSE COMPRESSION - LEVEL OF THE BEAM BOTTOM FLANGE

Bearing:

t_{wc} =	9	[mm]	Effective thickness of the column web	[6.2.6.2.(6)]
$b_{eff,c,wc}$ =	228	[mm]	Effective width of the web for compression	[6.2.6.2.(1)]
A_{vc} =	2485	[mm^2]	Shear area	EN1993-1-1 :[6.2.6.(3)]
ω =	0.73		Reduction factor for interaction with shear	[6.2.6.2.(1)]
$\sigma_{com,Ed}$ =	0.00	[MPa]	Maximum compressive stress in web	[6.2.6.2.(2)]
k_{wc} =	1.00		Reduction factor conditioned by compressive stresses	[6.2.6.2.(2)]
A_s =	1528	[mm^2]	Area of the web stiffener	EN1993-1-1 :[6.2.4]

$F_{c,wc,Rd1} = \omega\, k_{wc}\, b_{eff,c,wc}\, t_{wc}\, f_{yc} / \gamma_{M0} + A_s\, f_{ys} / \gamma_{M0}$

$F_{c,wc,Rd1}$ =	831.37	[kN]	Column web resistance	[6.2.6.2.(1)]

Buckling:

d_{wc} =	134	[mm]	Height of compressed web	[6.2.6.2.(1)]
λ_p =	0.66		Plate slenderness of an element	[6.2.6.2.(1)]
ρ =	1.00		Reduction factor for element buckling	[6.2.6.2.(1)]
λ_s =	2.27		Stiffener slenderness	EN1993-1-1:[6.3.1.2]
χ_s =	1.00		Buckling coefficient of the stiffener	EN1993-1-1:[6.3.1.2]

$F_{c,wc,Rd2} = \omega\ k_{wc}\ \rho\ b_{eff,c,wc}\ t_{wc}\ f_{yc} / \gamma_{M1} + A_s\ \chi_s\ f_{ys} / \gamma_{M1}$

$F_{c,wc,Rd2} =$ 831.37 [kN] Column web resistance [6.2.6.2.(1)]

Final resistance:

$F_{c,wc,Rd,low} = \mathrm{Min}\ (F_{c,wc,Rd1}\ ,\ F_{c,wc,Rd2})$

$F_{c,wc,Rd} =$ 831.37 [kN] Column web resistance [6.2.6.2.(1)]

Geometrical parameters of a connection

EFFECTIVE LENGTHS AND PARAMETERS - COLUMN FLANGE

Nr	m	m_x	e	e_x	p	$l_{eff,cp}$	$l_{eff,nc}$	$l_{eff,1}$	$l_{eff,2}$	$l_{eff,cp,g}$	$l_{eff,nc,g}$	$l_{eff,1,g}$	$l_{eff,2,g}$
1	31	-	50	-	71	195	237	195	237	168	179	168	179
2	31	-	50	-	220	195	239	195	239	318	256	256	256
3	31	-	50	-	220	195	239	195	239	318	256	256	256
4	31	-	50	-	71	195	237	195	237	168	179	168	179

EFFECTIVE LENGTHS AND PARAMETERS - FRONT PLATE

Nr	m	m_x	e	e_x	p	$l_{eff,cp}$	$l_{eff,nc}$	$l_{eff,1}$	$l_{eff,2}$	$l_{eff,cp,g}$	$l_{eff,nc,g}$	$l_{eff,1,g}$	$l_{eff,2,g}$
1	37	21	40	40	71	132	90	90	90	-	-	-	-
2	37	-	40	-	220	235	252	235	252	337	263	263	263
3	37	-	40	-	220	235	200	200	200	337	210	210	210
4	37	21	40	40	71	132	90	90	90	-	-	-	-

m – Bolt distance from the web

m_x – Bolt distance from the beam flange

e – Bolt distance from the outer edge

e_x – Bolt distance from the horizontal outer edge

p – Distance between bolts

$l_{eff,cp}$ – Effective length for a single bolt in the circular failure mode

$l_{eff,nc}$ – Effective length for a single bolt in the non-circular failure mode

$l_{eff,1}$ – Effective length for a single bolt for mode 1

$l_{eff,2}$ – Effective length for a single bolt for mode 2

$l_{eff,cp,g}$ – Effective length for a group of bolts in the circular failure mode

$l_{eff,nc,g}$ – Effective length for a group of bolts in the non-circular failure mode

$l_{eff,1,g}$ – Effective length for a group of bolts for mode 1

$l_{eff,2,g}$ – Effective length for a group of bolts for mode 2

Connection resistance for bending

$F_{t,Rd} =$ 141.12 [kN] Bolt resistance for tension [Table 3.4]

$B_{p,Rd} =$ 291.79 [kN] Punching shear resistance of a bolt [Table 3.4]

$F_{t,fc,Rd}$ – column flange resistance due to bending

$F_{t,wc,Rd}$ – column web resistance due to tension

$F_{t,ep,Rd}$ – resistance of the front plate due to bending

$F_{t,wb,Rd}$ – resistance of the web in tension

$F_{t,fc,Rd}$ = Min ($F_{T,1,fc,Rd}$, $F_{T,2,fc,Rd}$, $F_{T,3,fc,Rd}$) [6.2.6.4] , [Tab.6.2]

$F_{t,wc,Rd} = \omega\ b_{eff,t,wc}\ t_{wc}\ f_{yc} / \gamma_{M0}$ [6.2.6.3.(1)]

$F_{t,ep,Rd}$ = Min ($F_{T,1,ep,Rd}$, $F_{T,2,ep,Rd}$, $F_{T,3,ep,Rd}$) [6.2.6.5] , [Tab.6.2]

$F_{t,wb,Rd} = b_{eff,t,wb}\ t_{wb}\ f_{yb} / \gamma_{M0}$ [6.2.6.8.(1)]

RESISTANCE OF THE BOLT ROW NO. 1

$F_{t1,Rd,comp}$ - Formula	$F_{t1,Rd,comp}$	Component
$F_{t1,Rd}$ = Min ($F_{t1,Rd,comp}$)	215.87	Bolt row resistance
$F_{t,fc,Rd(1)}$ = 261.58	261.58	Column flange - tension
$F_{t,wc,Rd(1)}$ = 376.38	376.38	Column web - tension
$F_{t,ep,Rd(1)}$ = 215.87	215.87	Front plate - tension
$B_{p,Rd}$ = 583.58	583.58	Bolts due to shear punching
$V_{wp,Rd}/\beta$ = 382.31	382.31	Web panel - shear
$F_{c,wc,Rd}$ = 831.37	831.37	Column web - compression
$F_{c,fb,Rd}$ = 596.96	596.96	Beam flange - compression

RESISTANCE OF THE BOLT ROW NO. 2

$F_{t2,Rd,comp}$ - Formula	$F_{t2,Rd,comp}$	Component
$F_{t2,Rd}$ = Min ($F_{t2,Rd,comp}$)	166.44	Bolt row resistance
$F_{t,fc,Rd(2)}$ = 262.66	262.66	Column flange - tension
$F_{t,wc,Rd(2)}$ = 376.38	376.38	Column web - tension

$F_{t2,Rd,comp}$ - Formula	$F_{t2,Rd,comp}$	Component
$F_{t,ep,Rd(2)} = 246.76$	246.76	Front plate - tension
$F_{t,wb,Rd(2)} = 458.81$	458.81	Beam web - tension
$B_{p,Rd} = 583.58$	583.58	Bolts due to shear punching
$V_{wp,Rd}/\beta - \sum_1^1 F_{ti,Rd} = 382.31 - 215.87$	166.44	Web panel - shear
$F_{c,wc,Rd} - \sum_1^1 F_{tj,Rd} = 831.37 - 215.87$	615.50	Column web - compression
$F_{c,fb,Rd} - \sum_1^1 F_{tj,Rd} = 596.96 - 215.87$	381.09	Beam flange - compression

RESISTANCE OF THE BOLT ROW NO. 3

$F_{t3,Rd,comp}$ - Formula	$F_{t3,Rd,comp}$	Component
$F_{t3,Rd} = \mathrm{Min}\,(F_{t3,Rd,comp})$	0.00	Bolt row resistance
$F_{t,fc,Rd(3)} = 262.66$	262.66	Column flange - tension
$F_{t,wc,Rd(3)} = 376.38$	376.38	Column web - tension
$F_{t,ep,Rd(3)} = 225.64$	225.64	Front plate - tension
$F_{t,wb,Rd(3)} = 389.71$	389.71	Beam web - tension
$B_{p,Rd} = 583.58$	583.58	Bolts due to shear punching
$V_{wp,Rd}/\beta - \sum_1^2 F_{ti,Rd} = 382.31 - 382.31$	0.00	Web panel - shear
$F_{c,wc,Rd} - \sum_1^2 F_{tj,Rd} = 831.37 - 382.31$	449.06	Column web - compression
$F_{c,fb,Rd} - \sum_1^2 F_{tj,Rd} = 596.96 - 382.31$	214.65	Beam flange - compression
$F_{t,fc,Rd(3+2)} - \sum_2^2 F_{tj,Rd} = 539.94 - 166.44$	373.51	Column flange - tension - group
$F_{t,wc,Rd(3+2)} - \sum_2^2 F_{tj,Rd} = 541.80 - 166.44$	375.36	Column web - tension - group
$F_{t,ep,Rd(3+2)} - \sum_2^2 F_{tj,Rd} = 480.56 - 166.44$	314.12	Front plate - tension - group
$F_{t,wb,Rd(3+2)} - \sum_2^2 F_{tj,Rd} = 922.40 - 166.44$	755.96	Beam web - tension - group

The remaining bolts are inactive (they do not carry loads) because resistance of one of the connection components has been used up or these bolts are positioned below the center of rotation.

SUMMARY TABLE OF FORCES

Nr	h_j	$F_{tj,Rd}$	$F_{t,fc,Rd}$	$F_{t,wc,Rd}$	$F_{t,ep,Rd}$	$F_{t,wb,Rd}$	$F_{t,Rd}$	$B_{p,Rd}$
1	325	215.87	261.58	376.38	215.87	-	282.24	583.58
2	255	166.44	262.66	376.38	246.76	458.81	282.24	583.58
3	35	-	262.66	376.38	225.64	389.71	282.24	583.58
4	-35	-	261.58	376.38	215.87	-	282.24	583.58

CONNECTION RESISTANCE FOR BENDING $M_{j,Rd}$

$M_{j,Rd} = \sum h_j F_{tj,Rd}$

$M_{j,Rd} = 112.47$ [kN*m] Connection resistance for bending [6.2]

$M_{b1,Ed} / M_{j,Rd} \leq 1,0$	$0.98 < 1.00$	verified (0.98)

Connection resistance for shear

$\alpha_v =$	0.60	Coefficient for calculation of $F_{v,Rd}$	[Table 3.4]
$\beta_{Lf} =$	0.98	Reduction factor for long connections	[3.8]
$F_{v,Rd} =$	118.83 [kN]	Shear resistance of a single bolt	[Table 3.4]
$F_{t,Rd,max} =$	141.12 [kN]	Tensile resistance of a single bolt	[Table 3.4]
$F_{b,Rd,int} =$	209.14 [kN]	Bearing resistance of an intermediate bolt	[Table 3.4]

α_v = 0.60 Coefficient for calculation of $F_{v,Rd}$ [Table 3.4]

$F_{b,Rd,ext}$ = 156.36 [kN] Bearing resistance of an outermost bolt [Table 3.4]

Nr	$F_{tj,Rd,N}$	$F_{tj,Ed,N}$	$F_{tj,Rd,M}$	$F_{tj,Ed,M}$	$F_{tj,Ed}$	$F_{vj,Rd}$
1	282.24	0.00	215.87	211.14	211.14	110.67
2	282.24	0.00	166.44	162.79	162.79	139.75
3	282.24	0.00	0.00	0.00	0.00	237.66
4	282.24	0.00	0.00	0.00	0.00	237.66

$F_{tj,Rd,N}$ – Bolt row resistance for simple tension

$F_{tj,Ed,N}$ – Force due to axial force in a bolt row

$F_{tj,Rd,M}$ – Bolt row resistance for simple bending

$F_{tj,Ed,M}$ – Force due to moment in a bolt row

$F_{tj,Ed}$ – Maximum tensile force in a bolt row

$F_{vj,Rd}$ – Reduced bolt row resistance

$F_{tj,Ed,N} = N_{j,Ed} F_{tj,Rd,N} / N_{j,Rd}$

$F_{tj,Ed,M} = M_{j,Ed} F_{tj,Rd,M} / M_{j,Rd}$

$F_{tj,Ed} = F_{tj,Ed,N} + F_{tj,Ed,M}$

$F_{vj,Rd} = \mathrm{Min}\ (n_h F_{v,Ed} (1 - F_{tj,Ed}/ (1.4\ n_h F_{t,Rd,max}), n_h F_{v,Rd}\ , n_h F_{b,Rd}))$

$V_{j,Rd} = n_h \sum_1^n F_{vj,Rd}$ [Table 3.4]

$V_{j,Rd}$ = 725.72 [kN] Connection resistance for shear [Table 3.4]

$V_{b1,Ed} / V_{j,Rd} \leq 1,0$ 0.10 < 1.00 verified (0.10)

Weld resistance

A_w =	8184	[mm^2]	Area of all welds	[4.5.3.2(2)]
A_{wy} =	4206	[mm^2]	Area of horizontal welds	[4.5.3.2(2)]
A_{wz} =	3978	[mm^2]	Area of vertical welds	[4.5.3.2(2)]
I_{wy} =	110494245	[mm^4]	Moment of inertia of the weld arrangement with respect to the hor. axis	[4.5.3.2(5)]
$\sigma_{\perp max}=\tau_{\perp max}$ =	108.41	[MPa]	Normal stress in a weld	[4.5.3.2(5)]
$\sigma_{\perp}=\tau_{\perp}$ =	87.50	[MPa]	Stress in a vertical weld	[4.5.3.2(5)]
τ_{II} =	18.10	[MPa]	Tangent stress	[4.5.3.2(5)]
β_W =	0.85		Correlation coefficient	[4.5.3.2(7)]

$\sqrt{[\sigma_{\perp max}^2 + 3*(\tau_{\perp max}^2)]} \le f_u/(\beta_W*\gamma_{M2})$	216.81 < 404.71	verified (0.54)
$\sqrt{[\sigma_{\perp}^2 + 3*(\tau_{\perp}^2+\tau_{II}^2)]} \le f_u/(\beta_W*\gamma_{M2})$	177.79 < 404.71	verified (0.44)
$\sigma_{\perp} \le 0.9*f_u/\gamma_{M2}$	108.41 < 309.60	verified (0.35)

Connection stiffness

t_{wash} =	4	[mm]	Washer thickness	[6.2.6.3.(2)]
h_{head} =	14	[mm]	Bolt head height	[6.2.6.3.(2)]
h_{nut} =	20	[mm]	Bolt nut height	[6.2.6.3.(2)]
L_b =	55	[mm]	Bolt length	[6.2.6.3.(2)]
k_{10} =	7	[mm]	Stiffness coefficient of bolts	[6.3.2.(1)]

STIFFNESSES OF BOLT ROWS

Nr	hj	k_3	k_4	k_5	$k_{eff,j}$	$k_{eff,j}\ h_j$	$k_{eff,j}\ h_j^2$
					Sum	1542	432258
1	325	6	17	30	3	826	268203
2	255	7	20	14	2	633	161164
3	35	7	20	12	2	83	2891

$k_{eff,j} = 1 / (\sum_3^5 (1 / k_{i,j}))$ [6.3.3.1.(2)]

$z_{eq} = \sum_j k_{eff,j} h_j^2 / \sum_j k_{eff,j} h_j$

$z_{eq} =$ 280 [mm] Equivalent force arm [6.3.3.1.(3)]

$k_{eq} = \sum_j k_{eff,j} h_j / z_{eq}$

$k_{eq} =$ 6 [mm] Equivalent stiffness coefficient of a bolt arrangement [6.3.3.1.(1)]

$A_{vc} =$ 2485 [mm^2] Shear area EN1993-1-1:[6.2.6.(3)]

$\beta =$ 1.00 Transformation parameter [5.3.(7)]

$z =$ 280 [mm] Lever arm [6.2.5]

$k_1 =$ 3 [mm] Stiffness coefficient of the column web panel subjected to shear [6.3.2.(1)]

$k_2 =$ Stiffness coefficient of the compressed column web [6.3.2.(1)]

$S_{j,ini} = E\ z_{eq}^2 / \sum_i (1 / k_1 + 1 / k_2 + 1 / k_{eq})$ [6.3.1.(4)]

$S_{j,ini} =$ 33649.38 [kN*m] Initial rotational stiffness [6.3.1.(4)]

$\mu =$ 2.81 Stiffness coefficient of a connection [6.3.1.(6)]

$S_j = S_{j,ini} / \mu$ [6.3.1.(4)]

$S_j =$ 11954.45 [kN*m] Final rotational stiffness [6.3.1.(4)]

Connection classification due to stiffness.

$S_{j,rig} =$ 27407.68 [kN*m] Stiffness of a rigid connection [5.2.2.5]

$S_{j,pin} =$ 1712.98 [kN*m] Stiffness of a pinned connection [5.2.2.5]

$S_{j,ini}$ B $S_{j,rig}$ RIGID

Weakest component:

COLUMN WEB PANEL - SHEAR

Remarks

Bolts vertical spacing is too large. 220 [mm] > 200 [mm]

Connection conforms to the code	Ratio	0.99