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RELIABILITY OF MOMENT RESISTING BEAM COLUMN BOLTED CONNECTIONS

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Abstract

Connections are more complex than members to analyse and discrepancy between analysis and actual behaviour is large. Further, in case of overloading, we prefer failure confined to an individual member rather than in a connection, which could affect many members. Connections account for more than half the cost of structural steelwork and so their design and detailing are of primary importance for the economy of the structure. A structure is only as strong as its weakest link. Unless properly designed, the connections joining the members may be weaker than the members being joined. Thus designing for adequacy in strength, stiffness and ductility of connections ensure deflection control during service load and larger deflection and ductile failure under over load. Hence, a good understanding of the behaviour and design of joints or connections in steel structures is an important pre-requisite for any good design engineer.

This study presents the study of reliability of moment resisting bolted beam column connections. Flush end plate type connections were designed, fabricated and tested. Behaviour and failure pattern of different components of a connection like bolt, flush end plate, column member, beam member and stiffeners have been studied both analytically and experimentally.

Keywords: Flush end plate, bolted connection, lever arm, column flange stiffeners and web doublers.

1. INTRODUCTION

Now a day's use of structural steel in building construction has been increased due to its aesthetic appearance, ease of fabrication and faster erection time. The main usage of steel structures includes industrial structures (such as buildings, conveyors and pipe racks etc.), transmission towers, bridges etc.

In steel construction it is important to note that various members or elements in a structure are to be joined together by means of a joints or connections to transfer various loads from one member to the other.

Connections are used for joining different members of a structural steel framework. Any steel structure is an assemblage of different members such as beam, column and tension members, which are fastened or connected to one another, usually at the member ends. Many members in steel structures may themselves be made of different components such as plates, angles, I-beams or channels. These different components have to be connected properly by means of fasteners, so that they will act together as a single composite unit.

The joints or connections play a significant role in transfer of load from one member to the other member (for example beam to column or bracing to column or column to base plate etc.) at the same time they hold the total space frame in position. The selection and design of joints in steel construction plays a significant role which governs the safety and serviceability of the structure.

Connections form an important part of any structure and are designed more conservatively than the members. This is because, connections are more complex than members to analyse and the discrepancy between analysis and actual behaviour is large. Further, in case of overloading, we prefer the failure confined to an individual member rather than in any connection which could affect many members. Connections account for more than half the cost of structural steel work and so their design and detailing are of primary importance for the economy of the structure.

The type of connection designed has an influence on member design and so must be decided even prior to the design of the structural system and design of members.

1.1 Philosophy of Connections

A structure is only as strong as its weakest link. Unless properly designed, the connections joining the members may be weaker than the members being joined. However, it is desirable to avoid connection failure before failure of a member due to the following reasons:

- To achieve an economical design, usually it is important that the connections develop full strength of the members.
- Usually connection failure is catastrophic type of failure. Hence, it is desirable to avoid connection failure before failure of any member.

Therefore, design of connections is an integral and important part of design of a steel structure. They are also critical components of steel structures, since

- they have the potential for greater variability in behaviour and strength,
- they are more complex to design than the design of members and
- they are usually the most vulnerable components, failure of which may lead to the failure of whole structure.

Thus, designing for adequacy of strength, stiffness and ductility of connections will ensure deflection control during service load and larger deflection and ductile failure under overload. Hence, a good understanding of the behaviour and design of joints or connections in steel structure is an important pre-requisite for any good design engineer.

1.2 Classification of Connections

The connections used in steel structures are classified according to the internal forces that are to be transmitted by them and are given below:

1. Rigid connection, where no relative rotation occurs between adjacent beams and columns and bending moment can be transferred fully from a beam to the neighbouring column,
2. Pinned connection, where relative rotation occur between beams and columns and bending moment cannot be transferred at all.
3. Semi-rigid connection, where relative rotation occurs between beams and columns and bending moment can be transferred partially.

The basic assumption of conventional structural analysis is that joints are either perfectly rigid or perfectly hinged. Therefore, when analysing a structure, joints are idealised as either fixed or hinged. However, in actual structures, typical connections do not behave in either a perfectly rigid or a perfectly hinged manner.

The fact is that most simple connections do have some degree of rotational rigidity, was recognised and efforts were made to utilise it which finally led to the development of the semi-rigid connections. Similarly, rigid connections do experience some degree of joint deformation and this can be utilised to reduce design moments at the joints.

To achieve a more accurate analysis of a structure, it would be advantageous to include the true behaviour of joints. For example, there is a substantial variation in the distribution of bending moment for a beam with hinged, semi-rigid and fixed joints.

If we consider connections as pinned and are at 'l' distant apart, the beam is loaded with uniformly distributed load of 'w', resulting bending moment diagram is parabolic in shape having zero ordinates at the ends and $wl^2/8$ at the mid span, which results in designed section of beam to be somewhat uneconomical. So, to economise the section, designers usually go for connections to be rigid, due to which bending moment gets distributed resulting again in parabolic shape but having sagging ordinates of $wl^2/24$ at mid span and hogging ordinates of $wl^2/12$ at the ends. However in actual structures, resulting joints are neither fully fixed nor fully pinned, they are basically semi-rigid connections allowing some rotation as well as transfer some amount of moment to neighbouring member, which results in reduction of difference in magnitudes of moments at mid span and the ends. So, economical designs can be produced by balancing positive and negative moments in the beams.

1.3 Classification of Connections as per IS: 800-2007

Connections are classified according to their ultimate strength or in terms of their initial elastic stiffness. The classification is based on the non-dimensional moment parameter ($m^1 = M_u/M_{pb}$) and non-dimensional rotation ($\theta^1 = \theta_r/\theta_p$) parameter, θ_p is the plastic rotation. Where, rotation capacity $m^1 = (5.4 - 2.5\theta^1)/3$. This classification is based on a reference length of the beam equal to 5 times the depth of the beam. The limits used for connections classification are shown in Table 1. The moment-rotation relationship of a connection has to be determined based on experiments conducted for specific design.

Table 1 Connections classification limits

nature of the connection	in terms of strength	in terms of rotation
rigid connection	$m^1 \geq 0.7$	$m^1 \geq 2.5\,\theta^1$
semi-rigid connection	$0.7 > m^1 > 0.2$	$2.5\,\theta^1 > m^1 > 0.5\,\theta^1$
flexible connection	$m^1 \leq 0.2$	$m^1 \leq 0.5\,\theta^1$

Therefore, aim of this study is to design moment resisting connections and test them in laboratory in order to plot moment rotation relationship, to check reliability of bolted moment resisting beam column connections.

2. LITERATURE REVIEW

An experimental investigation on full scale frames was carried out by Miklo [1] to determine the effect of flexibility in beam column connections and column bases. He briefly explained

the complexities arising from three-dimensional (3-D) nature of the tests. A large number of tests were performed on isolated semi-rigid connections and flexibly connected sub frames. Tests on full scale 3-D frames were somewhat less numerous. However, experimental data on full scale frame behaviour is more important. First, it enables the effect of column continuity through a loading level to be investigated- a parameter not present in many sub-frame tests, and second, it confirms whether the experimentally observed performance of isolated joints and sub-frames is indeed representative of their behaviour when they form part of an extensive frame. This latter point is of particular importance if the extensive work on isolated specimens is to be incorporated into universally accepted methods of semi-rigid and partial strength frame design. He also carried out comparative study based on second order plastic analysis and non-linear shakedown analysis.

In this work, behaviour of semi-rigid connections on four full scale 3-D multi-storey frames was studied. He also explained how to address complexities arisen from 3-D nature of tests.

Aribert et. al. [2] carried out six laboratory tests to study the behaviour and classification of simple beam-to-column joints. Some of these tests concerned bending of column about its major axis, the others bending of column about its minor axis. The interpretation of test results deal with the main characteristics of rotational stiffness, moment resistance, rotational capacity, mode of failure and classification according to Eurocode 3 - Part 1.8. Based on gathered information they also briefly discussed some aspects of the design of simple joints and their consequence on the global structural analysis, are also illustrated using three worked examples, due to sensitivity to buckling, it appears that the usual assumption of pinned joints may be un-conservative, however in very specific cases.

They concluded that Eurocode 3 – Part 1.8 and numerous rules which it contains do not necessarily solve all problems. Eurocode 3 most probably allows for a rather good estimation of the characteristics of joints in terms of stiffness and strength. While their real influence on the behaviour of structures, their intervention in the analysis of structures and on the design verification are not necessarily adequately accounted for. One bay single storey frame, joints may behave in different manners. The choice which is accounted for in calculations may be either over safe or unsafe, strength, stiffness and also ductility is the parameters that must be considered altogether. Some specific joints show a rather high stiffness though their resistance is very small. Although, they should be classified as semi-rigid, in usual situations they can be considered as normally pinned, and lead to safe design, except in few cases related to

significant column buckling risk, but probably usual in practice, where the conception might be marginal safe. On the other hand, it is evident that advantage should be taken from the consideration of the stiffness of such joints.

Composite structures exhibit higher stiffness than steel structures with similar ductility and also show better performance to fire as explained by Gracia et. al. [3]. These benefits have led to an increase in the number of composite structures built in high seismic areas. The main objective was to obtain useful data about the seismic behaviour of a composite semi-rigid joint with a double sided extended end plate. A series of monotonic and cyclic quasi-static tests were performed to characterize the behaviour of this type of joint against seismic actions. The tests were performed on interior and on exterior joints and they have provided useful data about the ductility of the joint, the amount of energy dissipated and the degradation effects. This data was used to elaborate and calibrate a component based model of the joint that was composed of two rotational springs, one for the shear panel behaviour and other for the connection behaviour. The model is capable of simulating the joint behaviour and damage under cyclic loads with sufficient accuracy.

They have carried out various full scale tests to study seismic behaviour of a proposed semi-rigid composite joint with a double sided extended end plate coupled to two different types of slab. All prototype tests showed a very satisfactory behaviour. They concluded that the tests performed on exterior joints showed a very ductile behaviour. The connection exhibited pinching effects due to contact interaction of the connection elements during each load cycle. This phenomenon is more evident in prefabricated slab option. Otherwise, the behaviour of shear panel, with both slab solutions is quite regular with a large amount of energy dissipated. Exterior joint based on both slab solution demonstrated rotations close to 40 m rad.

A very ductile behaviour was noted during tests on interior joints. However, the contribution of left and right connections to the overall behaviour of the joint is smaller than that of the exterior joint, with a limited amount of energy dissipated. The shear panels showed large deformations without significant resistance reduction. The rotation of shear panel represents 65% of the overall joint rotation. The rotations reached 50 m rad under both hogging and sagging moments. The exterior joint based on prefabricated slab solution developed more resistance than the steel sheeting solution, both in shear panel (about 30%) and in connection (about 20%).

An experimental research program on end plate beam column composite joints under monotonically loading is presented by Simo˜es da Silva et. al [4]. Their major focus related to the identification of contribution of concrete confinement in composite columns to the behaviour of joint, coupled with a thorough assessment of various loading possibilities, ranging from symmetric and anti-symmetric loading on internal nodes to external nodes under hogging and sagging moments, typical in seismic regions.

The monotonic static loads were performed under force control in the elastic phase and displacement control in the plastic phase. The comparative analysis of the test results has been done in the paper.

France et. al. [5] reported test results conducted on joints made between universal beams and tubular columns using end plates bolted directly to the column face with ordinary bolts screwed into threaded holes formed using the flow drill process. Results obtained from the tests conducted on concrete filled tubes and compared these with those from a parallel series of tests in which tubes were unfilled. They observed significant difference in strength and stiffness between the two.

Cabrero and Bayo [6] conducted experimental investigation of statically loaded extended end-plate connections in both major and minor column axes. The aim was to provide insight into the behaviour of these joints when a proportional load is applied to both axes (3-D loading). The rotational stiffness of the joints increases with this type of 3-D loading. The findings also show that an increase in the end-plate thickness results an increase in the connection's flexural strength and stiffness.

3. EXPERIMENTAL INVESTIGATION

Moment connections are designed to transfer bending moment, shear force and sometimes normal force also. The design strength and stiffness of a moment connection are defined in relation to the strength and stiffness of the connected members. There are various options available to make a moment resistant connection. Various types of moment connections are:

1. Bolted Moment End Plate - Beam on one or both sides of column, with or without stiffeners or column web doubler plate.
2. Bolted Moment End Plate – Matching mitred end plates or apex connection in rigid frame.
3. Haunched Beam End Plate – Rafter horizontal or inclined. Beam on one or both sides of column. With or without stiffeners or column web doubler plate.

4. Flush Moment End Plate – One or two rows of bolts inside tension flange, available for restricted range of I-sections. May be used as splice.
5. Extended Moment End Plate – One, two or three rows of bolts inside tension flange – stiffeners available with one and three interior rows. May be used as splice.

All above mentioned configurations of connections are configured to resist moments as per their stiffness and geometric configurations. Connections that are commonly used are Extended Moment End Plate and Haunched Beam End Plate because of their various advantages.

Flush End Plate connections are comparatively less famous therefore there is deficiency of referral data about their working behaviour with different geometric configuration. Therefore, we tried to gather some information regarding various geometric possibilities of Flush End Plate type beam column connection, both by analytical and experimental methods to add some more data to already deficient database.

Flush End Plate connections have a single plate welded to end of the beam and bolted to the column flange or web using two or more bolts arranged in pairs. Where necessary, adjustments have been done by slotted holes between end plate and the column. Flush End Plate provides a neat detail and requires a greater number of bolts due to its smaller lever arm.

3.1 Details of Components of Specimens

Two moment resisting bolted Flush End Plate connections were designed [7 - 12] and three specimens for each type were fabricated accordingly which were tested in flexure and moment-rotation relations were plotted.

In view of testing facility available in the laboratory, ISMB 150 @ 15 kg/m conforming to IS: 2062 - 2006 [13] was used as beam member and ISHB 150 @ 34 kg/m was used as column member.

For ISMB 150 @ 15 kg/m, b_f = 75 mm, t_f = 8.5 mm, t_w = 5 mm, D = 150 mm and Z_p = 110487 mm^3. According to IS: 2062 the section is of grade E250 (Fe410W). Therefore,

Ultimate strength of beam = f_u = 410 N/mm^2 Plastic moment capacity of beam = f_u x Z_p

= 410 x 110487 = 45300000 Nmm

Flush End Plate bolted connections were designed for 75% and 60% of plastic moment capacity of the beam i.e. for 33.975 kNm and 27.18 kNm respectively.

High Strength Friction Grip (HSFG) bolts were used, which provide extremely efficient connections and perform well under fluctuating/ fatigue load conditions. These bolts tightened to their proof load and require hardened washer to distribute the load under the bolt head/nut. The washers are usually tapered when used on rolled steel sections. The tension in the bolt ensures that no slip takes place under working conditions and so the load transmission from plate to the bolts is through friction and not by bearing. However at ultimate load, the friction may be overcome leading to slip and so bearing governs the design. HSFG bolts are made from quenched and tempered alloy steels with grade from 8.8 to 10.9 conforming to IS: 3757-1985 [14] and IS: 4000 – 1992 [15]. Bolts of 16 mm diameter and length 55 mm were used, and were tightened to snug tight conditions. Snug tight is defined as full effort of a man on a standard podger spanner, or by a few impacts of an impact wrench. Podger spanners are graded in length in relation to bolt size and strength, for example, of order of 450 mm long for M20 high strength structure bolts and 600 mm long for M24 high strength structural bolts.

For a 16 mm diameter 10.9 grade bolt minimum strength in tension in snug tight condition is 146.25 kN and in shear is 54.75 kN.

The joints were tested under two point loads at a distance of 200 mm centre to centre on a simply supported span of 2200mm placed symmetrically about its mid span.

Connection for 33.975 kNm bending moment

Flush End Plate connections were designed and geometrically configured as per Figs. 1 and 2. Two end plates of size 150 X 125 X 8 mm were welded to the beams ends. Four 16 mm diameter HSFG bolts were used at distances of 29 mm and 28 mm from inner faces of top and bottom flanges and 30 mm from face of web, providing a lever arm of 76 mm and 65 mm centre to centre distance horizontally.

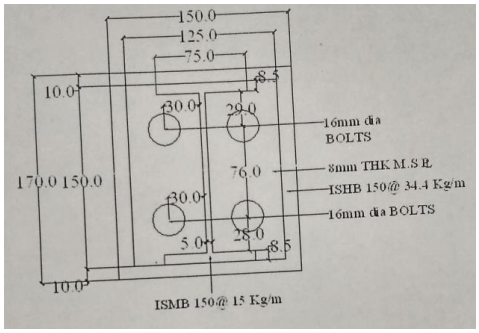


Fig. 1 side view of the connection

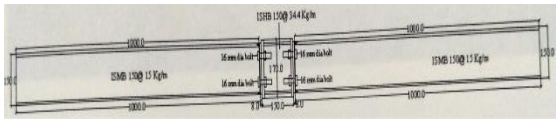


Fig. 2 front view of the connection

Connection for 27.18 kNm bending moment

Flush End Plate connections were designed and geometrically configured. Two end plates of size 150 X 125 X 8 mm were welded to the beams ends. Four 16 mm diameter HSFG bolts were used at distances of 29 mm and 56 mm from inner faces of top and bottom flanges and 30 mm from face of web, providing a lever arm of 48 mm and 65 mm centre to centre distance horizontally.

Inclinometer is an instrument for measuring angles of slope (or tilt), elevation or inclination of an object with respect to gravity. Rotation of beam members was measured using inclinometers by mounting them on beam members as shown in pictures by using those measurements, moment rotation relations are plotted. Least count of the inclinometers used was 1'.

4. TEST RESULTS AND DISCUSSION

Specimen designed for bending moment of 27.18 kNm was tested first on flexure testing machine as shown in Fig. 3. Loading was increased monotonically. The premature failure of the specimen was at about 13.8 kN means at 13.8 kNm i.e. at 50.77% of designed moment. It's moment rotation relationships is plotted in Fig. 4. Next, a specimen which was designed for bending moment of 33.975 kNm was tested on flexure testing machine. Loading was again increased monotonically. The premature failure of this specimen was at about 19 kN means at 19 kNm i.e. at 55.92% of design moment. It's moment rotation relationship is plotted in Fig. 5.

Initially while designing the flush end plate connection we designed the bolts but we did not design the flush plate because data from reference was not available. So, we provided plate with nominal thickness of 8mm. Stiffeners of about 25 mm width were provided in the column on both sides to stiffen the flanges of columns.

The specimens were removed from testing machine and were observed to investigate the reason for its premature failure. It was noticed that there was an excessive bending in flush end plate. So, basically it was flush end plate which was to transfer forces due to bending to the connection bolts in the form of tension force. It was concluded that there was some inadequacy in plate and its important to check its behaviour.



Fig. 3 specimen under flexural testing

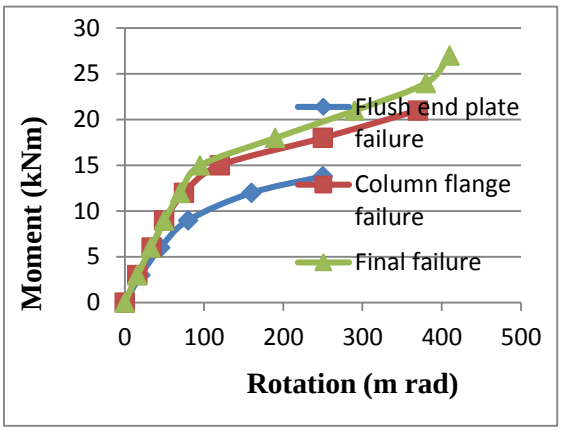


Fig. 4 Moment versus rotation relationship for 27.18 kNm connection

So, to validate the observation we reanalysed the connection using ANSYS software to understand behaviour of the plates. Both types of connections were modelled and from the analysis it is found that two factors were behind the failure.

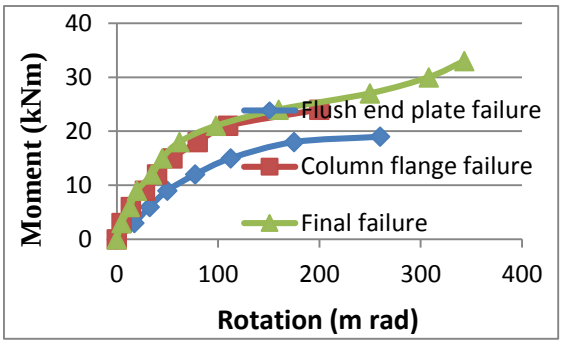


Fig. 5 Moment versus rotation relationship for beyond 33.975 kNm connection

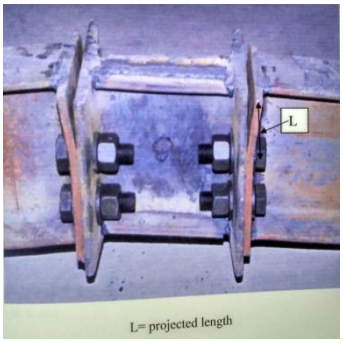


Fig. 6 projected length of flush end plate critical bolt line

As the behaviour of flush end plate in both cases (i.e. designed for bending moments of 27.18 kNm and 33.975 kNm) was different which was also observed physically after checking of the failed specimens and confirmed from the analysis: (1) first reason was inadequate thickness of the plate and (2) second reason was projected length of plate beyond critical bolts as shown in Fig. 6.

Projected length of the plate in specimen designed for bending moment of 27.18 kNm was more than that was designed for bending moment of 33.975 kNm. Because of difference in projected lengths beyond centre line of last bolt line on tension side, behaviour of bending was different as shown in Fig. 7 and 8. The plate is bending in vertical direction as its projected length is more (i.e. specimen designed for bending moment of 27.18 kNm). The specimen designed for bending moment of 33.975 kNm, the plate is bending in horizontal direction as its projected length is less.

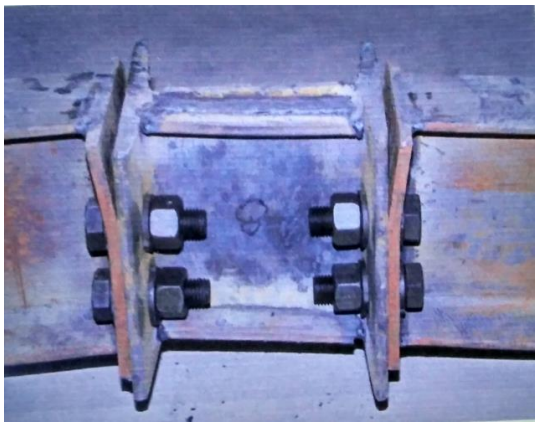


Fig. 7 large projected length of flush end plate plate beyond critical bolt line



Fig. 8 small projected length of flush end beyond critical bolt line

It was found from the analysis that flush end plates were failed due to excessive stresses, so it was decided to reanalyse the connections by increasing the thickness of flush plates. After performing a number of trials, it was found that the specimens designed for 27.18 kNm being less in magnitude requires 24 mm thick flush end plate i.e. the same as that is designed for 33.975 kNm. Reason for this type of behaviour is difference in projected length beyond last bolt line on tension leg. With this option, stresses in flush end plates were found within limit except surrounding holes i.e. due to stress concentration in that region because of drilling of holes for bolting. Additional plates of 16 mm thickness were welded to achieve a total thickness of 24 mm. Those specimens with thickened plate were tested on flexure testing machine and improvement in results was observed as the two specimens which were designed for bending moment of 27.18 kNm failed at 27 kNm and 19kNm and other two specimens which were designed for bending moment of 33.975 kNm failed at 33 kNm and 24 kNm. The

moment curvature relationships of these specimens are also plotted in Fig. 4 and 5 respectively. We still did not get the desired results because the flanges of ISHB150 @ 34 kg/m used as column were showing deformation.

On physically observing the specimens, it was found that column flanges were showing bending which can be the reason that after strengthening of flush plate, now column were not supporting the connection which was also shown by ANSYS in the analysis that stresses in column were exceeding its ultimate strength.

But, we could not ascertain that earlier because it was needed to stiffen the flanges of column by providing plates which creates difficulty in tightening of bolts due to constraint of availability of very less space.

Stiffeners for full width of column flanges and of 9 mm thickness were provided on columns to prevent buckling of column flanges. It was checked by further analysis on ANSYS that stiffening in that way could be used to strengthen the columns for those deformations but cannot be confirmed experimentally due to lack of time and resources.

CONCLUSIONS

1. End flush plate connecting beam member to column member through bolts is very critical part of connection as it transfers tension force to bolts, therefore it needs to be analysed and designed carefully.
2. Projection of end flush plate beyond the last bolt line, should be kept as minimum as possible for economical design of the plate.
3. Effective span of flush plate between two adjacent bolts should be optimized for the required number of bolts.
4. There is need to stiffen column flanges also, to prevent their bending.

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