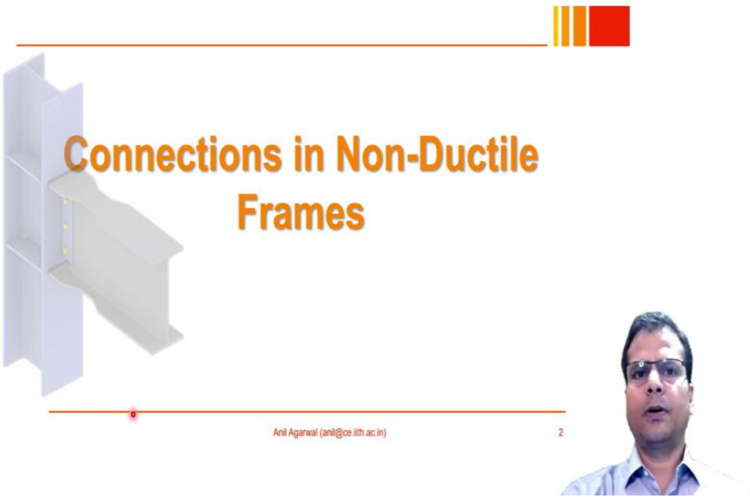
Design of Connections in Steel Structures

Prof. Anil Agarwal
Department of Civil Engineering
Indian Institute of Technology - Hyderabad

Lecture - 19
End-Plate Rigid Connection Introduction

Welcome back to the fourth week of this course on the design of Steel Connections. This week we will discuss different types of the moment and rigid connections. In the previous week, we discussed in detail the different types of simple or shear connections and the possibilities for moment connections. So, the basic configurations we discussed in this week we will go into the design examples and the design considerations for such connections.

(Refer Slide Time: 01:00)



So, the first part of this week's lectures will include non-ductile connections, which are part of a non-ductile frame. So, the non-ductile frames are typically the ones that are not required to be designed for earthquake loading conditions because, in earthquake loading conditions, we have to ensure that the connections behave in a ductile manner. Now to give you some background for regular steel structures, we follow IS800. Likewise, for regular concrete structures reinforcing concrete structures, we follow IS456.

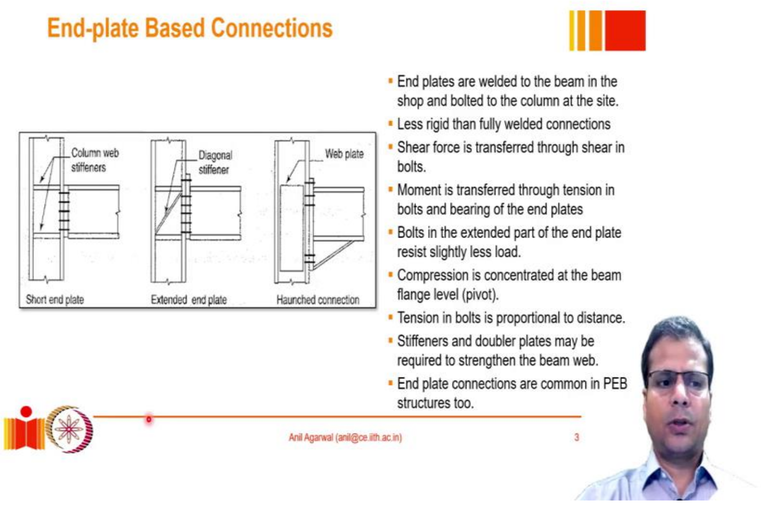
However, when it comes to the ductile detailing of reinforced concrete structures, which is again for designing of RC structures for earthquakes, we follow IS13920. And that particular document has been in practice in circulation for quite a while. However, there is no such code in practice for steel design of Steel structures and especially the detailing requirements for steel connections for ductile performance.

There is one code, one draft code which is being circulated among the stakeholders, and comments are invited. However, there is no such code in practice today, and IS 800 does provide us the design guidelines but does not provide us the detailed provisions for such a connection. So, we will talk about some of those aspects following some of the international codes, but for the time being, we will keep our focus on regular non-ductile framing requirements.

And when do we need non-ductile requirements? That we will discuss when we discuss the requirements of ductile frames. So, in the previous class, we had seen a couple of different types of rigid connections one configuration was where we used an end plate that was welded to the beam, and then it was bolted to the column flange. Another alternative was to use two T steps right.

Which also behave which would behave similarly as an end plate and then the completely different variety of such kind of connections was the one where we directly weld the flange of the beam to the column, or sometimes we may have we may have to use a cover plate to weld the beam to the column. So, the first variety we will discuss today is the one where we use an end plate.

(Refer Slide Time: 03:19)



So, there are several different possibilities within the domain within the area of end plate connection. So, here you see three different configurations in the first configuration; what you see here is the one where the end plate is relatively short. So, it is not extending beyond the beam depth and all the boards provided are only provided within the depth of the beam and in such a connection, the moment will be transferred through these bolts.

So, one of the ends will be under varying the other one will be pulling and these boards will be transferring the force and you have to make sure that this end plate is stiff enough and strong enough. So, that it can transfer the same amount of moment over a smaller lever arm another alternative is typically if the moment depending on the direction of the moment typically if the moment is of this type this type wherein it wants to bend in clockwise direction the beam wants to bend in clockwise direction.

Then the lower flange of the beam will be in compression and the top line will be in tension and if that is the case typically we need to count on the performance of bolts on the top end side. And therefore sometimes the end plate may be extended more to the top end side and we may provide a few extra bolts on that side to increase the lever arm. There can be another alternative where we can increase the length of the plate near the bottom flange of the beam.

However, in such a case we have to make sure that the end plate does not bend under that load and therefore we may have to provide a stiffener like the one shown here, typically known as haunched connection. When this portion is called a haunch which basically provides the stiffness the required stiffness to this plate which does not allow it to bend, we can increase the liver arm.

So, this increase in the liver arm may be required for that purpose. So, generally the idea is that in all these connections the end plate would be welded to the beam directly and this would be done at the shop and then the entire setup will be brought to the site. And then at the site, the beam and the end connection would be aligned with the column, and then these bolts will be placed.

Because of the presence of these bolts, bolts are more elastic or they have longer length than the welds and therefore these connections are typically more flexible as compared to the connection where the beam is directly welded into the column. But they provide this option that these connections can be opened and investigated later. In such fewer connections, the idea is that whatever the shear force demand will be transferred directly through the shear force that is acting at each bolt.

So, bolts will be participating in Shear to resist the shear force demand on the beam and the bolts will be going in tension to resist the bending moment demand. So, of course bolt cannot resist compression. So, whenever there is a compression demand that will be handled directly through bearing. So, for example in these cases the bottom flange of the beam which is the bottom most part that is the hunch will be bearing against the column.

And then all the bolts will be in tension, which is how a lever arm will develop. Whenever we extend the end plate exactly above the call above the top flange of the beam any bolts that we provide here since if we are not providing any stiffener such as haunches or anything like that, the end plate tends to bend because of this force that is acting on it because of the top bolt.

And because of the possible bending of this end plate, the force demand on this top bolt may not be as much as the force demand in the other bolts or it does not increase proportionally throughout. So, the assumption of plane section remaining plane for this connection may not be true and certain assumptions or certain approximate assumptions can be made to calculate the first demand in these bolts.

As I have mentioned before the compression is typically concentrated at the lowest point. So, it would be at the bottom flange in these two cases and the bottom of the haunch in this case. And if since that becomes the pivot point, we can assume if this is the pivot point, if the connection is tending to open in this way, and if this connection behaves or this end plate behaves like a rigid plate.

In that case we would have elongation in these bolts. So, all these bolts will be in tension but the elongation of each bolt will be proportional to the distance from the pivot point however that may not be the case for the boards that are provided beyond the top flange. So, in the extended part of the end plate that may not be the case because this portion of the end plate may bend a little bit because of the high action high bending moment demands.

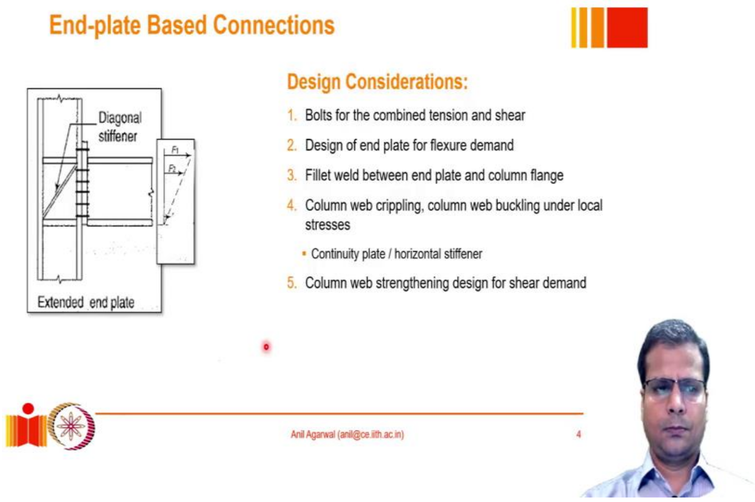
Another major aspect of these connections is the provision of these continuity plates or stiffeners horizontal diagonal stiffness or doubler plate in such connections. Whenever we have this kind of connections and if there is a point where the flange is going to bear against the column flange most likely it will introduce very large stresses in this neighborhood and there are different types of failures that are possible primary ones are the crippling failure of the beam web and the local buckling of the beam web.

These types of failures can be prevented by providing horizontal stiffness, also sometimes referred to as continuity plates. So, the idea here is that whatever force is being applied that can be transferred to the other end and if there is another connection another beam on the other side that force can be transferred all the way to the other beam instead of this web resisting the entire force.

Also, whenever we have this kind of loading acting on this, there is a large moment demand that requires a large shear resistance from the beam and the column web. Sometimes the column web may not have sufficient distance available and it can undergo plastic deformations or buckling. In order to prevent that we can either provide a diagonal stiffener which basically increases its shear resistance or sometimes we provide an additional doubler plate or the plate as shown here.

Such types of connections are quite common in PEB construction. So, PEB construction whenever we have Rafters in the structure typically, we have these end plates welded to the sections they are brought to the site and then they are bolted together using such bolts.

(Refer Slide Time: 10:31)



Now let us look at the different components that are part of a typical and plate connection and how these components are required to be designed. Here is a diagram of a typical and effect connection this is basically an extended endpoint connection but that hardly matters. So, as we have already understood the mechanism the pivot point will be somewhere here and individual bolts here will be resisting the force in proportional to their distance from the pivot point.

So, we can assume the pivot point to be at the bottom flange right at the center of the bottom flange and then we can calculate the strain developing in each of these poles. So, basically these boards first of all these bolts need to be designed for the combined shear force that would be acting on it because of the axial because of the shear force demand on the beam. And in addition to that they will also be resisting the tension force demand as shown here right.

So, we can we typically assume the shear force demand on each bolt to be equal because we can assume that these plates are much more stiff than the overall shear stiffness of all the bolts combined and as a result they can be considered as rigid planes and they can be considered to be not deforming while the bolts are deforming in shear. So, boards have to be designed for the combined tension and shear.

In addition, if it is an extended end plate, it is subjected to large bending moment demand and the end plate also needs to be designed for that demand if I may show this in another plane. So, if I take another view, I am looking from this side. So, I have got this end plate and then there is beam which is welded to the end plate. So, this weld is typically done in form of a fillet weld which runs all around the beam.

So, we will follow that the detailing requirements we can weld the flanges on all sides and then after some discontinuity we can weld the web and then also the flanges. So, this will be the locations of the fillet welds that we may need to provide and we need to make sure that these fillet welds which are welding which are joining the beam to the end plate are strong enough to transfer the force the required forces that is the third point here.

I miss the second point. So, in this particular scenario you might again notice that if I have any plate like this which is welded to the beam which is shown here and if we have some boards located here and maybe there are other boards along the way. And when this top flange is pulling out is moving in out of the plane direction outward these bolts are holding it back and again coming back to the plane what we may anticipate.

What we may expect is that because of this joining of this flanges of this beam this end plate may deform in this way right and these bolts are holding it back and this top flange of the beam is pulling it in this side and we may have to design this cross section of the end plate for the maximum bending moment demand which will be basically equal to the force in the bolt multiplied by this lever arm right.

That will be the bending moment demand and this plate needs to be designed for that. Now in addition to these; these are majorly a part of the connection but also we should be mindful that the column itself which might have been safe earlier because of this additional concentrated forces that are acting at the column where in this region this column may have to be strengthened around these areas.

There may be a pull force or there might be a push force and in either case we have to extend the network. So, the possibility of web crippling or column web buckling those are very much real and they have to be prevented and for that we need to provide lateral or horizontal stiffness. And in addition to that if there is a large shear force demand that is calculated on the column web and if the column web does not have enough capacity we may provide diagonal stiffners or doubler plates. So, these components need to be designed in a typical end plate rigid connection.