Design of Connections in Steel Structures
Prof. Anil Agarwal
Department of Civil Engineering
Indian Institute of Technology - Hyderabad

Lecture - 21
Welded Flange Rigid Connection

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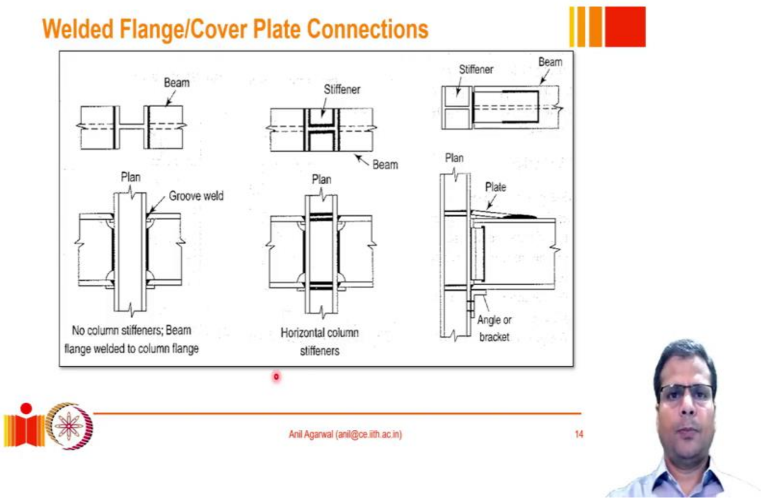

So, in the previous lecture we discussed the different types of end plate connections rigid connections a variety of rigid connections which is known as end plate connections and one and then also we solved one example on how to design an end plate connection. In this lecture we will discuss another variety of rigid connections which involves welding of the beam flange directly to the column.

So, these are typically known as welded flange or cover plate type of connections. So, here some of the examples are shown of such connections I will elaborate a bit. So, this is basically a column that is supporting 2 beams one on either side and this is the top view of the same connection here you might see that the web of the beam is welded to the column also the flanges are groove welded to the column.

So, the web here is fillet welded to the column and the flanges are groove welded. Now couple of things you might notice is that the top side there is a bevel in each flange which is group welded to the column in this particular diagram you do not see any continuity plates or any horizontal stiffness in the column but sometimes that may also be required depending on the local stresses that we are introducing in the column web.

And also what is the requirement of shear stresses in the column? One more detail that could be of interest to us is that it is not necessary to weld the beam's web to the column. So, even though we are welding the flange the flanges to the column, the web could be also bolted because the mechanism of force transfer for the 2 types of; part or the 2 parts of this connection are quite different and therefore, it is possible to mix welds with bolts.

Because bolts will be primarily designed to resist the shear force and these belts will be designed only to resist the building moment in all these joints you might also notice this excess hole that has been provided we will talk about that in a minute. The other example here is that some continuity plates could be provided and stiffners could be required. And in another example here what you see is a very different type of connection where the top flange of the beam is not directly welded to the column.

But there is another plate which is used and which is actually slanted plates with the reduced cross section thickness and that is welded to the beam and then other end of the plate is welded to the column and in addition to that separate arrangements are made to transfer the shear force. So, in this particular case, both a shear plate and an angle seat are provided. So, in this lecture, we will discuss the force transfer mechanism and design considerations for these types of connections and then we will do one example.

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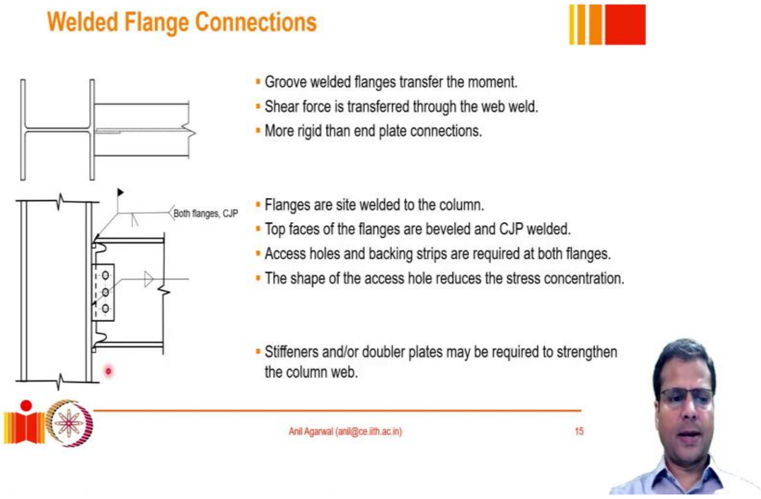

So, here you can see a typical rigid connection in this diagram wherein the beam flanges are directly welded to the column. So, as we have discussed the beam flanges are group welded and they are meant to transfer the design moment capacity design moment demands and the shear force is transferred through the web. So, the web could be welded to the column but also it is quite common to bolt it to the column.

So, typically what we do is the column and this shear tab or shear angle or any of the any alternative shear connecting element would be short welded to the column and then the column would be brought to the site with this component welded to it. And then the beam is also brought to the site wherein all these geometrical features have been added at the shop and brought to the site.

Then at the site first these the beam is aligned with this with these holes and then these bolts are placed. So, these bolts also not only resist shear they also act as support during the erection process. Once these bolts are in place then this beam is stable then welders can get there and they can weld they can provide a sight weld between the beam flange and the column flange. So, since in this connection the beam is directly welded to the column instead of an intermediate end plate like what we saw in the previous case.

Because of that this connection is more rigid in comparison to an end plate connection typically. So, as I mentioned, this weld is a side weld, which is the groove weld. So, generally site welds are to be avoided but in this type of a connection site field cannot be avoided and appropriate provisions should be made for proper inspection and quality control for any site weld in addition to increasing the factor of safety that we use for a welded connection.

So, here you might notice that only the top faces of the flange should be bevelled. Why the top is because that is where the welder can easily access and easily deposit the welded material the welding material. If it was bevelled from the bottom it would be very difficult to weld upside down because it is a side weld we cannot rotate the column upside down it has to be welded in this configuration only.

So, it is better to bevel them in the same direction. So, that sign is shown here representing the bevel in the bevel groove in the column in the beam flange. And typically such welds are complete joint penetration welds. Complete joint penetration weld is critical because it provides it the required level of ductility which is usually missing in a partial joint penetration well.

When we pull this flange apart from this flange if it is a partial joint penetration weld and there is no other component resisting opening of the route a PJP type of a weld would perform very poorly under this situation therefore in such situations complete joint penetration is very critical access holes are also required at both sides. So, here obviously we require the access hole.

So, that we can reach the full penetration, so, because of the presence of the web it is very difficult to take the electrode all the way from one side to the other side the web prevents moving of the weld from this side to other side therefore in order for us to develop a nice groove well a continuous weld we have to cut a small hole. So, that welder can first weld from this side all the way to the middle and go a little bit over to the other side on the middle from the middle and then complete the weld from the other side.

Here also an excess weld is required because again we need to provide in addition to in order to be able to provide this weld sometimes this gap is bit too much and we may have to provide a backing strip or a backing plate and when we need to provide a backing plate we also require to cut this portion. So, that backing plate can be tack welded to the beam. These access holes also play a very important role in separating the movements from shear. So, because of the presence of this particular geometry of this access hole.

And this geometry is also if you go to the detailing guidelines you will see that this geometry is very specific and one needs to satisfy that specific geometrical requirement in order to avoid any stress concentrations in this region. So, because of this access hole, any shear force demand does not get transferred to the flange. Because if we can imagine if we move this beam up or down, the flange can very easily deform.

And does not would not transfer a lot of shear force to the weld and most of the shear force will get transferred through these voltage joints in these connections as well if the stress concentration because after the groove weld is done this will become a monolithic kind of a component and any axial forces which are acting on these flanges will get transferred to the column flanges and then to the column web which may introduce local stresses which can cause web crippling and web local buckling etc.

In order to avoid that we have to put some we may have to provide the stiffners and if the shear capacity is not sufficient we may provide some kind of a doubler plate or a diagonal stiffener.

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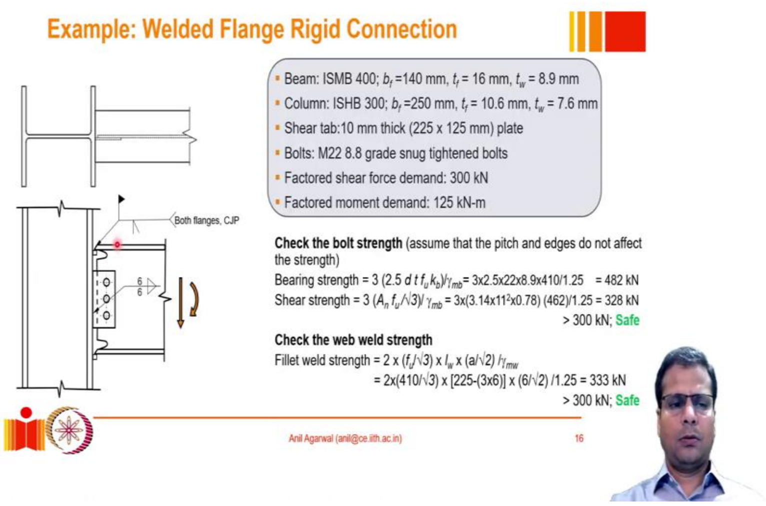

So, now we know the basic philosophy of these connections how they behave they are very simple for calculation point of view. The only challenge with these connections is that they require a site welding which is very critical for the performance of the connection but these are one of the very common types of connections especially when it comes to moment resisting frames. So, in this example the beam dimensions are given ISMB400.

The different useful parameters such as the thickness of the flange the width of the flange and thickness of the web are also mentioned here the column is an ISHB300 section the column of course is a not as deep as the beam but it is wider than the beam. So, columns width is 250 millimeters as compared to the width of the beam which is only 140 millimeters. Then there is a shear tab that has been provided shear type is just a plate which is a 10 millimeter thick plate of dimensions 225 in the vertical direction.

And 125 millimeters in the horizontal direction and then appropriate bolts are provided. So, three bolts of 22 millimeter diameter and 8.8 grid are snug tightened in these locations. The force demands are given as the shear force demand is 300 kilonewton and the factored moment demand is given as 125 kilonewton meter. So, there is a vertical force in this direction which is 300 kilonewton and there is a moment of 125 kilonewton meter.

So, we will go one component at a time first we will check the strength of the bolt then we will check the strength of the weld which is provided between the shear plate and the flange of the column. We are assuming here that the shear plate is safe for this for this force demand one may check however whether it is sufficient or not in this calculation we will not check the strength of the shear plate and then subsequently we will check will design the welds here and here.

So, since these are complete penetration joint weights we already know the dimensions, we can check whether they are sufficient or not. We will start with the bolt strength there are total three bolts there are three bearing surfaces between the bolt and the beam web and there are three bearing surfaces between the bolt and the shear plate. Now the shear weight thickness is given as 10 millimeters but the thickness of the web of the beam was only 8.9 millimeters.

And we are assuming here that all these sections are made of Fe410 type of steel its yield stress is 250 mPa and the ultimate stress is 410 mPa the bolts are 8.8 great volts. So, for the design of the bolts, let us first assume that the pitch and the end distances are sufficient that they do not start to control the strength of the bolt. So, first, let us look at the bearing strength of one bolt. So, the bearing strength for one bolt is given by 2.5 x d_t x f_u x k_b.

I hope you remember what these parameters represent divided by γ_mb, γ_ mb is the factor of safety. Now f_u was the ultimate strength of the bolt and the k_b factor controlled the effects of the pitch versus if the bolt is of a weaker bolt is stronger than the plate then we would use the f_u of the plate. The d is the diameter of the bolt and t is the thickness of the plate that we are designing for.

So, since the thickness of the web of the beam is smaller than the thickness of the shear plate. So, we will use t w of the beam since there are three bolts we will multiply the same values with three. So, three multiplied by 2.5 multiplied by diameter of the bolt multiplied by the thickness of the web multiplied by the f u of the of the beam web divided by a factor of safety of 1.25 and we get the bearing capacity of three bolts as 482 kilonewton shear strength of the bolt.

So, the bolt itself can undergo shearing type of failure we need to check for that the shearing shear strength again I hope you remember where it comes from and is given by a_n x f_u assuming that the shear plane passes through the threaded portion of the bolt. There is a conservative assumption a_n xf_u divided by root 3 divided by γ_ m b and multiplied by three bolts. So, three so area of cross section area of a bolt can be taken as pi r squared r is 11 millimeters multiplied by 0.78.

So, 0.78 basically is the factor that we can multiply the cross, cross section of the shank to calculate the net cross section of the threaded portion multiplied by f_u by root 3 divided by 1.25. So, here f u for this bolt is 800 mPa. So, 800 mpa divided by root 3 will give us 462 divided by 1.25 and we get the shear strength of the bolt as 328 kilonewton. Now the demand for the factored shear force demand was given as 300 kilonewton.

The shear strength of this voltage joint is more than 300 it is actually 328. So, it is safe. So, now let us check the strength of this weld that is connecting the shear tab with the column flange. So, as you can see, there are 2 welds on each side of the shear tab the size of the weld size is marked as six millimeters. So, the strength of this weld can be calculated easily we know the expression for the strength of affiliate weld we use f_u divided by root 3 as the material strength.

Because we assume that the strength of such a weld is governed by shear strength, as per the Indian standard code, IS800 l_w is the length of the world multiplied by the throat thickness. So, this gives l w multiplied by throat thickness and gives us the effective cross section area divided by gamma mw, which is the safety factor. Factor of safety in this particular case we will take it because this is this will be a shop welded connection at this location.

And this weld will be site welded but this is a shop welded connection therefore we can take gamma mw as 1.25. So, we substitute the values fu ultimate stress for the parent metal is 410 mPa length of the weld. So, since the plate depth or the plate length is 225 millimeters we will leave some at the ends we should terminate slightly short of that edge therefore we I am leaving three times the weld size at each end.

And that is the actual length that is available multiplied by a by root 2 that is the weld size divided by root 2 and the safety factor. We can calculate this strength to be 333 kilonewton and the shear force demand was given as 300 kilonewton. So, the shear strength of this joint is more than the shear demand therefore this plate this weld is also safe. So, now we have checked the safety of these bolts and safety of this weld.

Now let us check the safety of these welds that are between the beam flange and the column flange for the given bending moment demands.

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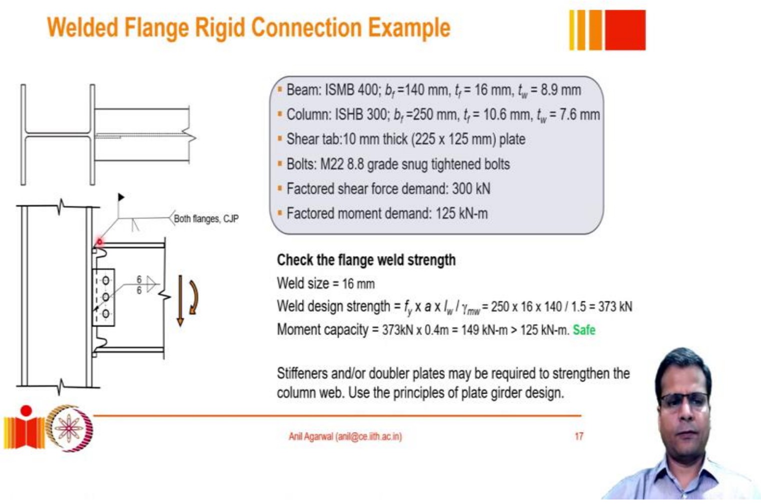

Now these welds of course they are given as con complete joint penetration welds and they are done on both flanges. So, the thickness of the weld or the throat size of this weld will be 16 millimeters that is the thickness of the flange of the beam. So, we will use 16 millimeters as the throat size and using that we can calculate the capacity of this weld under tension. So, now let us calculate the strength of these welds under this type of loading wherein this flange undergoes tension.

So, as you know groove welds are basically treated same as the parent metal and the yield strength governs the failure. So, we will use the yield value f y and the throat thickness that is a multiplied by l w. So, a would be 60 millimeters lw would be the length of the weld which is nothing but the same as the width of the flange. So, we will take that as 140 millimeters divided by the factor of safety partial factor of safety.

In this case since this is a field weld this symbol also represents that this is a field weld therefore gamma and w will be equal to 1.5 and we substitute these values and we get the weld strength as 373 kilonewton. Now for a weld of this strength how much moment can it resist since there are welds present at both ends. We can calculate the moment capacity of this weld couple by multiplying this strength with a with the lever arm.

So, lever arm between the 2 welds can be estimated as 400 millimeters which is the depth of the beam. To be more precise, you could have taken it as the distance between the flange center and the flange center, but here it is done approximately. And this strength turns out to be 149 kilonewton meter. The demand for factored bending moment demand was 125 kilonewton meter and the capacity is 149. So, which is significantly higher than the demand, this connection can be considered safe.

So, we have checked the safety of the boards, the weld between the shear tab and the column, and the welds between the beam flanges and the column. Now the remaining aspect of this connection would be to check the safety of this portion of the column. Now again we will not discuss that part in this course this will be should be should be covered separately in a plate girder section because it is more relevant there.

So, if you want to study that part in more detail please go ahead and read those sections in the Indian Standard Code and in the textbook. So, this concludes the discussion on non-ductile frame connections. Subsequently we will talk about the ductile frame connections for earthquake loading conditions.