Design of Connections in Steel Structures
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Lecture - 20
End Plate Connection Design Example

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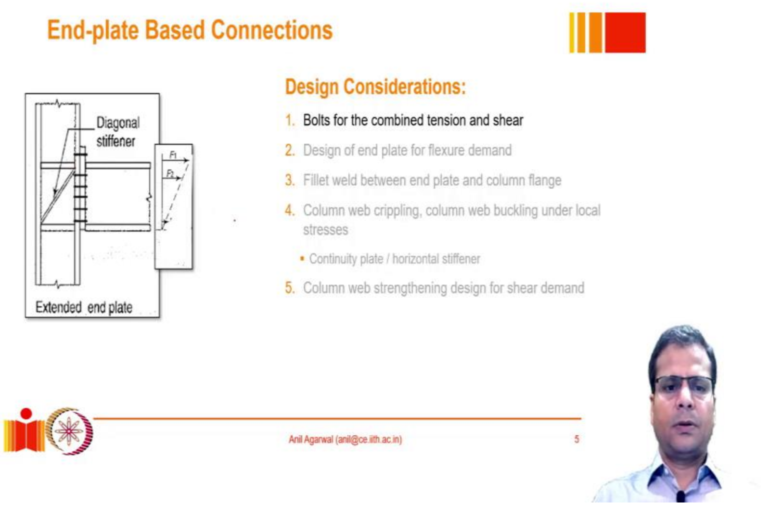

So, first we will start with the discussion on bolts that are subjected to combined tension and shear force these are those bolts which are supposed to combine tension and shear.

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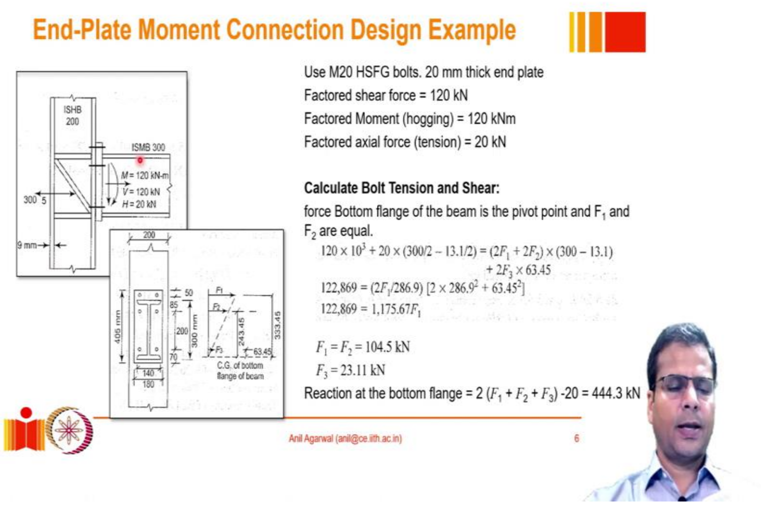

So, we have an example here again this example is taken from our textbook which is N. Subramanian's book this is a column ISHB 200 an ISMB 300 beam which has an end plate connection with the column right and here we can see that these three sets of bolts are provided. So, six bolts are provided overall to join the end plate with the column flange. There is also some requirements for horizontal stiffener and diagonal stiffness we will talk about them in a minute.

First let us discuss the basic dimensions. So, the column is ISHB200, beam is ISMB300 there is a moment demand of 120 kN-m in this direction there is an axial force demand which is marked as H that is 20 kilonewton and there is a shear force demand of 120 kilonewton. Additional information is provided these bolts are high strength friction grip boards M20 size and they are joining a 20 millimeter thick end plate.

So, this end plate thickness is 20 millimeters, the factored shear force and bending moment in the hogging direction and the actual force are also some more relevant details. The beam size was ISMB300. So, total beam depth was 300 millimeters and an end plate of 405 millimeter depth and 180 millimeter width and 20 millimeter thickness has been provided six bolts are located on this end plate as shown here.

So, the distance of the top layer of bolts from the edge is 50 millimeters then the second layer is 85 millimeters away from the first layer and then the third layer is 200 mm away from the second layer these are those distances that are marked. And the remaining distance is 70 millimeters from the bottom edge. The relative distances of these with respect to the flange of the beam are also given.

So, the top flange of the beam is in between the center of the distance between the top layer of the bolts and the second layer of the bolts. So, now we can start calculating some distances the beam bottom flange width is 140 millimeters we are assuming that the pivot point will be right at the centroid of the bottom flange. So, this will be the location of the bottom flange bottom flange thickness is 13.1 millimeters that we can look up in the catalog.

To find out what is the cross-section dimensions of ISMB300 which is given as 313.1 millimeters. So, basically from the bottom of the beam the centroid or center of gravity of this reaction would be 13.1 millimeters. Then that point onwards we would have bold forces increasing proportionally, theoretically speaking however since we know that there is a possibility of flexible behaviour in the end plate we can approximate it as that this F_1 or F_2 are not actually exactly be proportional to the distance from the pivot point.

But they will be equal and one simplifying way to do to handle that problem is to assume that they are both acting at the location of the top flange wherever the top flange is we will assume that that is where both F_1 and F_2 are acting and they are both equal. So, that is a simplifying assumption we can make to account for some level of flexibility in the end plate and then we can start balancing the forces.

So, first let us start to balance or try to balance the moment components. So, it is easy to calculate the moment components about a force whose values we do not know. So, we do not know the amount of compression that is acting here. So, let us try to start to do that we will calculate the moments about this point what are the total components that are acting force components which will introduce moment?

First of all there is an externally applied force at the centroid of the beam which is the value is given 20 kilonewton's. So, that will be inserting a force that force that moment will be equal to force multiplied by the distance from the pivot point so that force is 20 kilonewton multiplied by the distance and that distance is beam depth ,divided by 2 minus the flange thickness divided by 2.

So, that makes sense. So, this will be the lever arm that is what this distance is. So, this is multiplied and you will get a clockwise moment at the pivot point because of this force. The other component we should account for is the actual moment that is applied, which is 120 kilonewton meter. So, that gets added directly since I am using kilonewton millimeter as the units in this formula.

So, let me use 10^3 to convert this from kilonewton meter to kilonewton millimeter okay and both of them are in clockwise direction therefore an addition + sign.. Now these have to be balanced by the internal forces and the internal or internal moments and internal moments are produced by this F_1, F_2, F_3 forces. So, as I just mentioned, I have assumed that F_1 and F_2 are equal in magnitude and act at the location in the middle of the 2 forces that coincide with the top flange centroid.

So, and since there are 2 bolts at each layer there are 2x F_1 and 2x F_2. So, we will do 2xF_1 + 2x F_2. So, we are adding these forces multiplied by the lever arm and the lever arm would be basically the distance from the bottom flat centroid to the top flange centroid, equal to (300-13.1). So, this gives the moment because of the first 2 layers of bolts the third layer of bolt again is at a distance of 63.45 millimeters from the pivot point that has been calculated based on the dimensions that were given and 2xF_3 multiplied by 63.45 would be the moment contribution coming from the third layer of poles.

So, this is a simple equation we have got three unknowns F_1, F_2 and F_3 here we need to substitute them in some values for them. So, that we can get a single equation with a single unknown. So, the straightforward method would be to use the assumption between F_1 and F_2. So, we had assumed F_1 is equal to F_2. So, I can substitute that here. So,. Now I have got only 2 unknowns F_1 and F_3.

F_1 and F_3 can be related by the way this F_1 is now acting here and it is acting at a distance of 286.9 millimeters which is basically the distance between the centroid of the top flange and the center of the bottom flange. So, I can say that F_3 divided by its distance from the pivot point with assuming that the plane sections remain plane will be equal to F_1 divided by 286.9 right and now I can substitute that here this value goes on to that side 3.45.

Now my F_3 value can be substituted here to again write the entire equation in the form of F_1 that is what we do we write that equation in the form of F_1 and we can calculate for F_1 which turns out to be 104.5 kilonewton. F_2 is also the same. So, F_1, F_2 are both 104.5 kilonewton and F_3 is much smaller than that which is equal to 23.11 kilonewton okay.. Now if we do a force balance we can also calculate how much reaction would be acting at the bottom flange of the beam how do we do that?

We know the forces that are acting externally 20 kilonewton in this direction then these bolts are pulling the plate back in both of them are 104 and there are 2 of them 104.5 and two of them then at the second layer also 104.5 multiplied by 2 then at a 63 millimeter distance from the bottom we have another one which is equal to 23.11 multiplied by 2. So, for these forces we can calculate how much will be the force at the bottom end right and that is what we can calculate basically by balancing all these forces.

So, we add all the bold forces and subtract the external force that we are applying and we get the reaction at the bottom flange.. Now having done that. Now we should calculate whether what is the shear force demand on each of these bolts and then we should look for whether these bolts are safe under these loading conditions or not.

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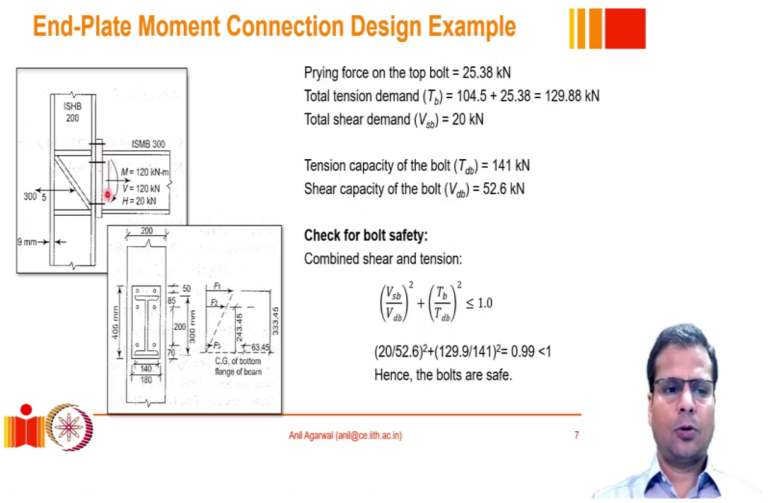

So, here we can see that there are total six bolts out of these the first layer and second layer is subjected to largest axial force demand because of the bending moment that was acting primarily. And the third layer was subjected to a very small axial force demand when it comes to shear force demand all these bolts can be assumed to resisting the same amount of load right. If we can assume that this plate is almost rigid in its plane then that assumption is not such a bad assumption.

So, obviously. Now we know from this understanding that shear force demand is the same in all the bolts, which are subjected to the largest axial force demand. Therefore the total force the net force on these bolts is much higher than these bolts therefore we need to worry about the design of these pair of bolts rather than this layer. If you are designing this bolt particularly, we must also be mindful of the possibility of a prying action developing right where we can imagine that this top flange is pulling away from this point.

This end plate may deform like so, and if it does deform like this it will introduce an additional force Q that we had discussed in the very first week of lectures that this additional force Q represents a prime action force which in results in an additional force demand in the bolt right.

So, this top layer of the bolt is likely to be subjected to an additional axial force demand tension force demand and let us first calculate that.

So, if we do that calculation I have not shown the calculation here because we have done enough examples before we can calculate that force and that turns out to be 25.38 kilonewton because this plate was stiff enough it does not it did not introduce a very large prying force. We need to add that force to the force that we had calculated directly from the movement, which had been calculated as 104 kilonewton.

So, added together the total force in the bolt becomes 129.88 kilonewton. As I mentioned, the shear force demand would be equally distributed among the six bolts that are present. The total shear force is 120 kilonewtons; therefore, the shear force demand in each bolt is calculated as 20 kilonewton.. Now I am not doing this part again, but we can easily calculate the tension capacity of the bolts provided here.

I have given the details these are HSFG high strength friction grip bolts tension capacity can be calculated and shear capacity can be calculated based on the material properties provided. So, if let us say we have calculated those.. Now we need to check whether these bolts are shear are safe under a combined axial tension and shear for that this interaction equation we would use we have discussed that also.

The shear force demand divided by the shear capacity and tension force demand you add better tension force capacity and their squares should be less than or equal to one we substitute the values here. These are already the design capacities right. So, the factors of safety are already inbuilt in these values. Once we do that we will get a value of 0.99 which is just less than 1 but that still makes still ensures that the bolts are safe under the combined axial force and shear capacity shear demand.

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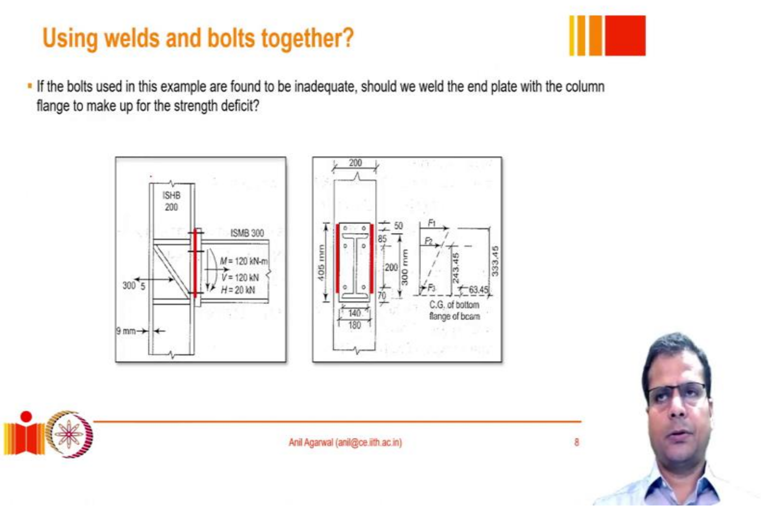

Now I have a question, let us say in this case the bolts turned out to be just safe to be able to resist the moment demand. There was a moment demand and this bolt we found was carrying the largest amount of force and it was just barely safe, then we got a factor of safety of 0.99 instead of with an upper limit of one. Let us see if they were not really safe and it had fallen short by let us say 20%. So, that factor instead of 0.99 let us say we had got 0.8.

Then would it have been wise to provide additional welds between the end plate and the column flange to resist the same force. So, basically these bolts are participating, these bolts are going into tension to resist the force and also shear to resist the applied force and let us say we are finding that that is falling short a little bit and we do not want to drill another set of holes. So, let us say these designs were done and there was some miscalculation we had let us ignore the prying action.

And when we do a recalculation at that by the time these holes have been drilled the material has been supplied to the site and we realize that the connection is unsafe by a small fraction. Now can we just add 2 welds ,we can add can we add welds between the end plate and the column flange at site and let us say we do enough number of welds. So, that it fullfills or it fills the deficit in the capacity would that be a wise choice?

I will give you a few seconds to think about it okay? So, I hope you have made up your mind you have an answer in your mind the correct answer is, no that would not have been a good choice and in fact not a wise choice. The reason for that is that we should never mix bolts and welds to resist the same load. So, these bolts were supposed to resist this bending moment and combine bending moment shear force and this axial force.

Either we should completely rely on bolted connections same types of bolts or we should completely rely on the welded connection. We should not mix them together the reason for that is that they have very different stiffnesses. In fact welds have much higher stiffness than bolts and if we provide welds along with bolts when we apply this force most of the force will be taken or will be resisted by the welds in the beginning and bolts will not be resisted resisting any force.

Because for the bolts to start resisting some force you they need more elastic deformation. So, what would happen eventually is that first all the force will be resisted by the welds and welds will be highly stressed and bolts will not be participating. And if the welds are not designed to resist the entire load, they will fail at some point. Once they fail they will not be able to offer any resistance and the entire load will then fall on the bolts.

So, the bolts and welds will not work together right they are often provided at the same time in a single connection. For example here you may see that we are we will discuss this also in a minute that this beam is welded to the end plate using welds but at the same time the end plate is welded to the column using bolts that is perfectly okay because then they are transferring different amounts or different types of forces right.

They are not participating to resist the same force they are not sharing the same force among each other and that is why that is perfectly okay I hope you got the answer.

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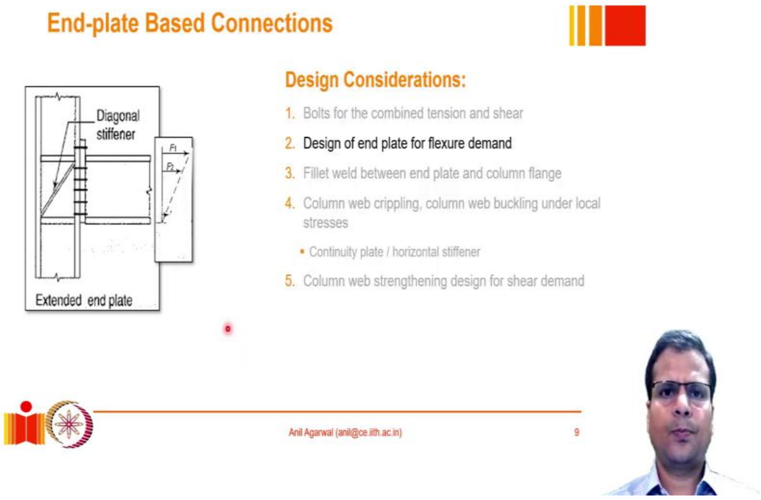

So, now let us also check whether the end plate has sufficient strength to resist the bending moment that will be acting at this interface because of this pull force.

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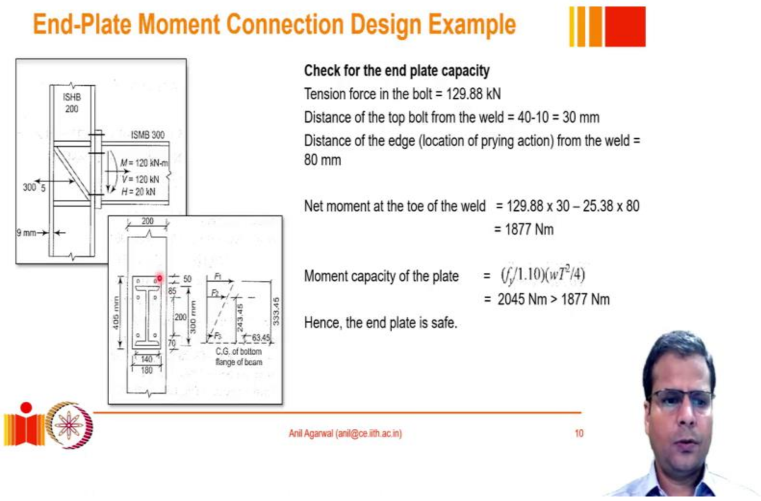

So, we all know that this bolt top bolt will be subjected to a maximum tension force of 129 kilonewton because of the prying action. The force in this stop top flange will be we can say that it is 129 minus the prying action. So, that is the actual net force demand but some of it will be enhanced because of the prying actions at this edge. So, if we can zoom into this area into this zone.

Let us see this is the top most edge of the end plate and the end plate is being pushed this way through prying action okay and that force we had calculated that had turned out to be 25.38 kilonewton. Next comes the point where the bolt is present and at the bolt is basically trying to pull the plate back in with a force of a total force of this is the top layer of the bolt therefore the total force is 129 point kilonewton.

And further down this is somewhere where the actual weld exists. So, we can approximate the location of the weld if we knew the size of the weld we could have actually calculated the toe of the weld but here since we have not calculated the weld size let us take the location of the bottom flange itself and let us try to calculate the moment demand there in the plate. So, let us take that location and that is where some force is acting which is actually the demand.

Now we need to calculate the moment demand in this plate at this location. So, first let us calculate these relative distances. So, it has been calculated the distance of the top bolt. So, the top bolt is this bolt from the weld this distance is equal to 30 millimeters and the distance of the edge of the plate from the location of the weld. So, basically the edge of the plate to the bolt is 50 millimeters that was given to us before.

And then another 30 millimeters is the distance from the bolt to the top flange; therefore, the total distance from here to here will become 80 millimeters. So, now if we know these lever arms we can calculate the moment demand here the moment demand here at this point of interest would be at the 2 of the well will be equal to 129.88 kilonewton multiplied by 30 that is the lever arm minus 25.38 kilonewton that is this force multiplied by 80 and that gives us a total moment demand of 1877 Newton meter.

Here you may notice that I have changed the units from the units in this expression was kilonewton millimeter and if I change the units from kilonewton millimeter to Newton meter I do not have to do any multiplication or division the same value would be the value would remain the same. Now this is the total moment demand at that location in the end plate. So, this end plate should be able to resist this much of moment.

So, since it is a plastic design, this is a ductile system. So, we can design this plate for the plastic moment capacity it is a compact section of course it is just a plate. So, if we look at this cross section of this plate and try to calculate its plastic section modulus for this section the plastic section modulus will be calculated with the assumption that half of the cross section goes into tension complete yielding and half of the cross section goes into yielding in complete compression.

And in such a situation the net forces will be acting at these locations which are the centroids of the each half and the moment capacity or we can say section modulus section plastic modulus would be equal to the areas of each half. So, which is basically the thickness of the plate multiplied by the width of the plate divided by 2; area of that half multiplied by the lever arm that is the distance between these 2 arrows and the distance is nothing but t divided by 2 again.

So, basically the section plastic section modulus can be calculated as w t square divided by 4. So, that is plastic section modulus if we multiply plastic section modulus with the with the yield stress of the material we can calculate the moment capacity of this plate. So, we do that we take the plastic section modulus that we have just calculated multiplied with the yield distance of the plate dividing by the factor of safety for resistance in yield conditions that is 1.1. When we do that here the width of the plate is taken as 180 millimeters thickness was taken as 20 millimeters and f y is taken as 250 mpm and then we substitute these values we substitute these values to calculate the moment capacity of the plate as 2045 Newton meter which is greater than the demand of 1877 therefore the plate is found to be safe in bending.

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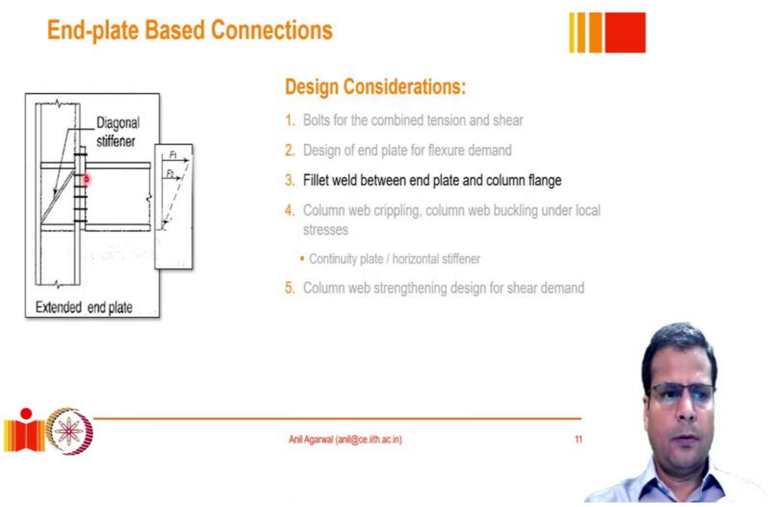

So, far we have checked the safety of the bolts and the safety of the end plate itself. Now let us also check the design of the welds that connect the beam to the end plate.

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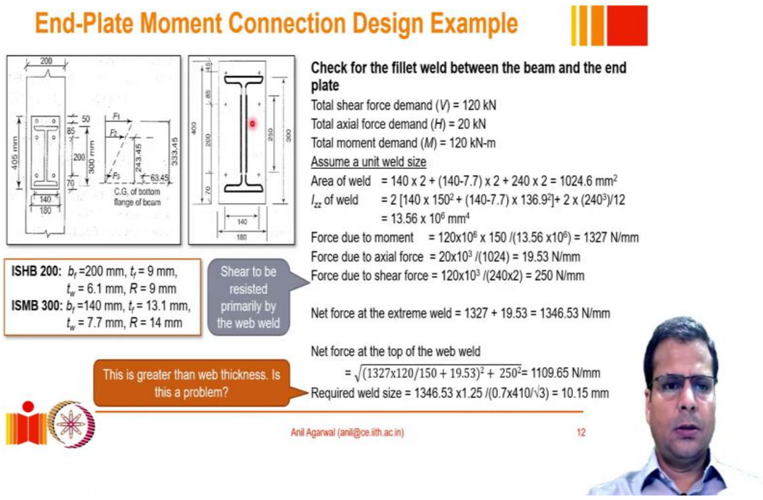

These are going to be fillet weld joint which is typically shop welded. So, here in this diagram you can see this is the end plate this is the beam and the edges of the beam will be fillet welded with the end plate and if we can look closely these are the locations where we will be providing the fillet welds. So, typically we will discontinue it between the shear and the portion of the weld which will resist the shear force and the portion of the weld which will resist the axial force.

So, again to reiterate the total force demands the shear force demand which is in this direction the axial force demand which is in out of the plane direction and total moment demand which is going to introduce tension in this fillet in out of plane direction. We need to calculate the weld size that will fulfill these loads' requirements. So, we will start with an assumption that the weld size is unit.

So, we will, so, that our calculations are easy and we will calculate the weld size which would be sufficient to resist this load. Some of the member dimensions are also listed here the thickness of the flange for the column thickness of the flange's width, and so on. We can refer to them whenever we need. Also, one idea we have to implement here is that the axial force or the effect of the bending moment will be resisted only by the welds provided around the flange.

Whereas the welds which are provided along the web will only partially resist the moment however shear force will not be resisted by the welds around the flange. And the reason for that is that when they if we assume that these welds are resisting shear that would also include an assumption that this flange has high stiffness in out of plane direction which is not really the case and therefore we cannot count down their ability to resist shear.

So, most likely the shear will be resisted by the welds which are provided along the web. So, to start the calculation, let us first try to calculate the total length of the weld and since we are assuming that the weld size is unit, the length will also be same as the area of the weld. By the way here there is a minor mistake here this dimension 300 that is marked here is actually the beam size it is not up to here this is 300 millimeters from the top of the beam to the bottom of the beam.

Now area of the weld the middle portion of the weld I am assuming it to be 240 millimeters long that is another minor correction here this is 240 millimeters long bend in the middle and the end welds are of course all the way around the flange and they stop because of the web in between. Now with these corrections let us continue the discussion. So, area of the weld can be calculated as 140 is the width of the flange.

So, 140 at both ends 140 x 2 + this much of width of the flange which is 140 minus 7.7 which is the thickness of the web of the beam. So, that we will deduct and the remaining we will keep that again 2 of those , that much length multiplied by 2 and then whatever is the additional at around the both sides of the web which is 240 multiplied by 2.. So, the total area of the weld for a unit size weld is 1024 millimeter square.

What is the moment of inertia of these welds? So, for the moment of inertia we can calculate easily this is a symmetric weld about this axis we will call it x axis about this axis we can calculate the moment of inertia by the way we are ignoring the part of the weld which is around the toe around this fillet in the beam. We are assuming that it is even though it is coming up to here we are ignoring that part.

Moment of inertia will be equal to first, we will calculate the moment of inertia of the top edge of this weld which will be 140 is the length or the area multiplied by the distance squared and distance from the centroid to this is equal to the half of the depth of the beam that is 150 squared. So, $A*R^2$ + the same thing would be repeated for the second layer which is this layer.

So, the area or the length there is slightly less than 140 because the beam web is present multiplied by the distance. So, this time the distance is not 150 but 150 minus the thickness of the flange that is 13.1 millimeter. So, we when we deduct that we get 136.9 squared. So, now we have calculated the moment of inertia of the drop top line and the second line of the weld. Now we will calculate the moment of inertia of the web portion.

So, moment of inertia of the web portion will be equal to b h^3/ 12 that is the typical formula we use. So, in this case, the h is 240, b is 1 but there are 2 such words. So, therefore we multiply with 2 divided by 12, b is 1 because we are assuming a unit size weld and calculating this value we get a moment of inertia of 13.56×10^6 mm^4. Now that we have calculated the weld's cross-sectional properties, we can go ahead with the force calculation.

So, first let us calculate the force demand in maximum force demand in any part of the weld because of the moment. So, maximum force would be equal to generally for a stress we do My/I, M multiplied by y divided by I. So, M divided by I multiplied by we will do a multiplication with y which will correspond to the distance. So, M moment demand is 120 into the 10^6 Newton millimeters multiplied by y, y is 150 from the neutral axis for the extreme fiber of the weld that is 150 divided by I.

I is this value that we have just calculated and from there we get the force demand on unit length of the weld as 1327 Newton per millimeter. Now the force demand will decrease as the size of the weld increases. Now similarly the force due to the axial force we can calculate. The actual force given is 20 kilonewton which is acting in out of the plane direction.

So, it is out of the plane direction we will take 20 and power 3 Newton’s divided by the total area because we can assume that the entire cross section of the weld or the beam is participating to release the axial load. So, that gives me a force demand of 19.53 Newton per millimeter length force due to shear force. Now because of the shear force we are assuming that only the middle portion of the weld is participating which is around the web and therefore we will take only the cross section area of that portion.

So, the total shear force demand was 120 kilonewton, which was given here divided by the weld length of the weld that is 240 multiplied by 2, which gives me a force demand of 250 Newton per millimeter length in the fillet. Now we need to be careful while designing it. So, we assume that this part of the fillet does not resist any of the shear force. The shear force which was acting in the vertical direction.

However this fillet will be resisting the moment effect and it will also be resisting the direct axial force effect which is acting in out of the plane direction. So, both of those moment and axial force will have the same consequence on the weld if we think about it. So, the same the force component because of them will be in the same direction, so we can directly add the 2 forces.

So, the force demand because of the moment is 1327 Newton per millimeter and the force demand due to the actual force is 19.53 Newton per millimeter we can add them and we get the total force demand on the extreme weld. Now that is of course likely to be the maximum force demand anywhere in the world but we should be careful that we did not account for the shear force here but there is a portion of the weld which is resisting shear force as well as the bending moment.

And for that portion we should check again whether the moment demand the force demand there is higher than what we have calculated here. So, for that portion let us calculate the force demand for this portion. So, in this case, we had calculated the force demand to be 1327 which was the extreme fiber, and the neutral axis is right in the middle. So, here at this location the force demand will be proportional to the distance from the neutral axis.

So, we can simply say 1327 that was the force demand in the extreme location multiplied by 120 which is the location of this point divided by 150 which was the location of the first point extreme point okay. So, if we do that we will get the force demand at the top end of the web weld and there is an effect of the axial force which is 19.53 which we had calculated for earlier well also that will be true here.

So, these two can be added directly because they have a resultant force in the same direction plus the effect of the shear force we have calculated before this is in this direction in downward direction the earlier 2 forces were in the out of the plane direction. Now they are at 90 degree from each other. So, we have to take the resultant by vector multiplication by resolving the components in the 2 directions.

And what we find is that that force is squared plus the out of the plane force squared that should give us the total force demand which is calculated as 1109 Newton per millimeter. Now we can compare the extreme weld and the weld at the web's top. So, the extreme weld was subjected to a forced demand of 1346 whereas this weld is only subjected to force demand of of 1109.

So therefore the extreme welds are more critical and let us design for that. So, we can calculate the weld size by simply using the available expression, which we have discussed again in the past. For a fillet weld we will take the f_u value for affiliate well we do not use the f_y value but we use the f_u the ultimate stress of the parent material as well as the weld material and typically.

The parent metal is the weaker of the 2 therefore we take the f_u value of the parent metal then we divide by $\sqrt{3}$ to correspond to the shear mode of failure 0.7 here represents the throat thickness. So, this gives us the total area and then we can calculate the size of the weld by this expression wherein the total force demand divided by the force demand of the unit weld of unit size weld which will give us the total force demand.

And we are using a factor of safety of 1.25 here because this is typically a shop welded joint and then we get the weld size of 10.15 millimeter. So, we can provide an 11 millimeter or 12 millimeter weld, which should suffice in such a scenario. The question arises if you look at this web of the beam the components that this weld is going to join are as follows. And there is an end plate that has a thickness of 20 millimeter.

Then there is a flange of the beam which has a thickness of 13 millimeter and then there is also in this case here I can show this is the web which only has a thickness of 7.7 millimeter. So, now for the weld that is running these 2 components 20 and this one is 13.1 or a web weld that is joining 2 components one of the which is 20 and the other one is 7.7. Can we use an 11 millimeter weld for joining these kind of components? Think about it is it.

To have a 10 millimeter weld that is welded to only a 7 millimeter thick plate, is there a problem with that? It is not a problem actually you some of you may recall that or might be trying to use the condition for fillet welds wherein we were welding 2 plates together let us say and we were required to provide a flat weld here and the requirement was that if it is a square cut surface we should leave at least 1.4 or 1.5 millimeter exposed.

That means the weld had to be smaller than the plate thickness right. Similarly there was a requirement for a curved toe also right both of them insisted that the weld should be smaller than the thickness of the plate by certain amount at least. Here that condition does not apply because we are not really welding the edge of a plate instead of instead what we are doing here is that edge of the plate is already at this end and the edge of the plate is already touching the other plate.

And what we are doing here is that this side here there is no such restriction that the weld has to be thinner than the plate thickness the weld can be thicker than the plate thickness which is perfectly acceptable and we can weld it this way.

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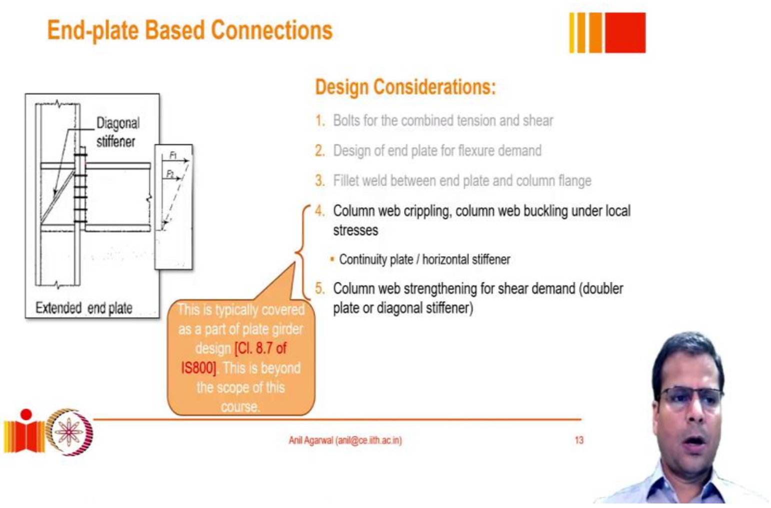

So, now in this connection, we have now discussed how to design this weld between the beam and the end plate welds here, here and in between. How to design the end plate itself for the moment capacity and then how to design the bolts? Now the question comes about designing this column whether any special provisions have to be provided or done to make the column resist this load effectively.

And generally of course when we have a shear force demand we should this shear force demand would be calculated using STAAD Pro itself or any analysis software that you are using that software will give you the shear demand and that column should be designed for that shear force demand. However in addition to that if these local forces may not be exactly calculated accurately calculated.

And the shear force demand may not be exactly accurate if you use the values calculated you calculated using only a software which assumes a center line as representative of the member size. In such a situation we should again do a hand calculation to calculate the shear force demand on this member and then make a provision for stiff appropriate stiffness or double up plate to resist that moment.

In addition to that any local stresses that develop in the web because of this concentrated forces coming out of the flange of the beam for such things we can always utilize the provisions for plate girders right. Since plate girders are not really a part of within the scope of this course I will not spend much time discussing this I would recommend you to go through clause number 8.7 to understand the requirements of these stiffener design etcetera.

And we can have a separate discussion on that later on but I would skip that discussion for this course and that would conclude basically as far as this course is concerned the designing of an end plate type of a connection.