**Design of Steel Structures**
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**Module - 6**
**Flexural Members**
**Lecture – 5**

Hello today I am going to deliver the lecture on built-up beams. In last lectures, we have discussed about the design of beams, with laterally supported and unsupported cases. Here we will see the built-up beams how to design, first of all, why built-up sections are required. As we know when, the magnitude of the load is becoming high then the available section, available steel rolled section may not be sufficient to carry that much load that is why we may have to go for some sort of built-up section. It is not only because of that some other reasons are there also like, if the span length is very high length of the span is very high, then deflection will become very high, because that is some KWl cube by EI.

So, l cube or in case of UDL load l to the power four. So, if the span length becomes very high, then delta is going to become very high, delta means deflection. So, to change the deflection, but we know codal provisions that l by t 25 has to be there limiting deflection. So, to check the deflection to keep the deflection within the codal provisions we have to, make selection of beam in such a way that deflection be restricted. How do we make that by increasing I because Wl cube by EI is fixed because steel section, we are going to use. So, only option is I moment of inertia can be increased to decrease the deflection. Now, moment of inertia how to increase we can increase only by configuration of certain section means, the beam section has to be made in such a way that, the I of that section Ixx not Iyy, Ixx of that section will become high in that case the deflection will be less.

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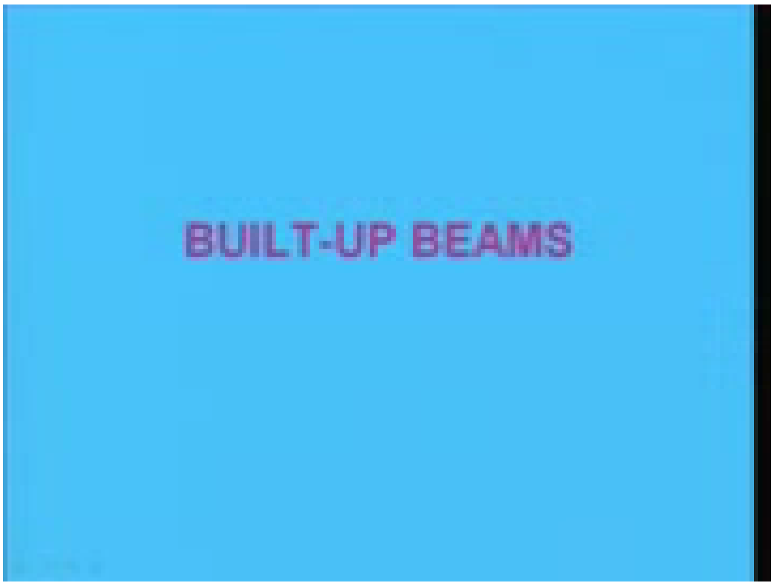


Now, see suppose there is a say beam with a load; now how the moment is going to act. We know say for simply supported it is say moment is happening like this. Now, if we increase the load more, then what will happen? The moment is going to become more again; if we increase the load then this is also increase the load. So, in the same way the as we know that Z is M by sigma bc; that means, the section modulus, if M is going to increase. So, what will happen? The section modulus if we see section modulus is if it is first case, the second case it will be this is first case, if I see the section modulus it is coming.

In second case, we have to go for this. And in third case maybe, we have to go for this, but how long it will go. We may not have availability of that higher section. So, in that case, if still more load are there then what will happen? Because of high magnitude of moment, we have to go with some built-up section; that means, we can increase the moment of inertia and other properties by addition of some plate addition of the plate.

So, plate can be added or some other built-ups means rolled section can be added like channel can be added or I sections can be added in this way we can increase the moment of inertia as a whole or section modulus as a whole. So, in this way we can make it. However, before going to details of the built-up sections, first I would like to complete quickly the earlier remaining portion of the lecture. In earlier lecture, I was discussing about the design procedure of unequal beam or plate sections. This unequal beam unequal flanges of beam or plate sections comes mainly when it is made built-up sections. So, the design procedure which I have not discussed; I could not discuss because of shortage of time, first let me give an overview in very shortly, then I will come to the details of the built-up beams.

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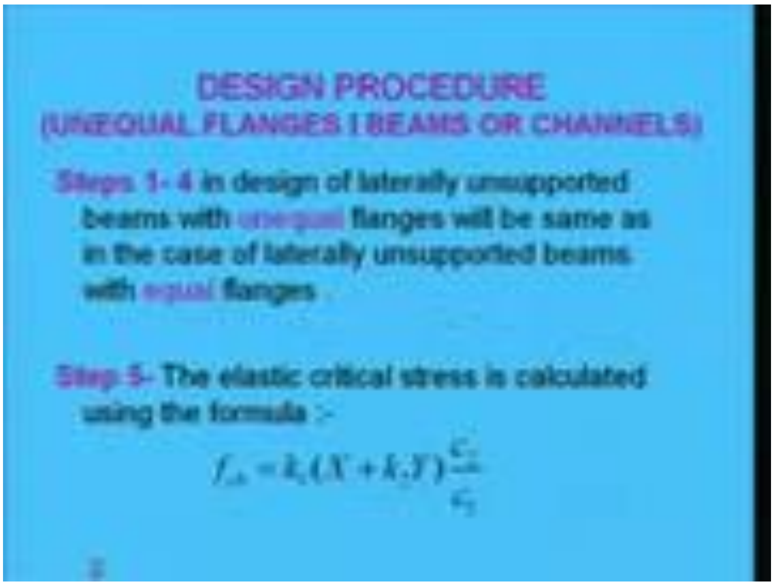


So, in last lecture I was telling that design procedure of unequal flanges, I beams or channels. So, theory we have already told that how to find out the sigma bc; that means, bending stress in compression, that how to find out that I have discussed and other aspects also I have discussed that how trial and error method we have to find out. What are the parameters, what are the expressions to find out fcb elastic critical stress, then for finding our fcb what is the different parameter like X Y which can be find out from expression or from the table, which is given in 6.345; In different tables, different values are given coefficients are given.

So, from that we can make, so now we will just have a look what are the procedure we used to follow for design of such sections. So, first is that from steps 1 to 4 in design of laterally unsupported beams with unequal flanges, will be same as in the case of laterally supported beams with equal flanges. So, steps 1 to 4 whatever we have discussed in case of laterally unsupported equal flanges, will be followed for the case of laterally unsupported unequal flanges; that means, step1 to 4 will be provided for laterally unsupported unequal flanges also.

Now, step 5 will be differ from the laterally unsupported equal flanges with this. What is step 5, In step 5 we will find out the elastic critical stresses, using this formula that is fcb is equal to k1 into X plus k2 into Y into C2 by C1. Now, the coefficient of k1 k2 X Y all either can be found from the tabular form given in code in IS 819, 84 in table 6.345; Otherwise we can find out from the expressions which was shown in earlier lecture. And also given in the code at corresponding closures. So, from there also directly we can find out.

As I was telling if we go for development of a software program to design such beams, we need to calculate the values by the use of expression not by the use of table, because in table certain values are given from this equation. Now, if we put directly these equations, we can find out the exact value for a particular case which we do not need to extract from the table. So, if we can develop software properly, we can utilize our time we can save lot of our time for designing such beams. When in a design office a designer design engineer is going to design such type of things they can follow, they can use such software to save the time. And by using such software they can make the design economize means, they can find out a very suitable section which will be economy from cost point of view.

So, in that way they can do. Because of manual calculation which is very much tedious generally, design engineer is reluctant not to do the iteration; that means, again and again and again they are not calculating. But if he has the software then the computer itself will through search through iterative they will find out the appropriate section, which will be most economy. So, I will suggest the students those who are learning this course to develop their own software. So, that it will be useful for their own purpose as well as for others.

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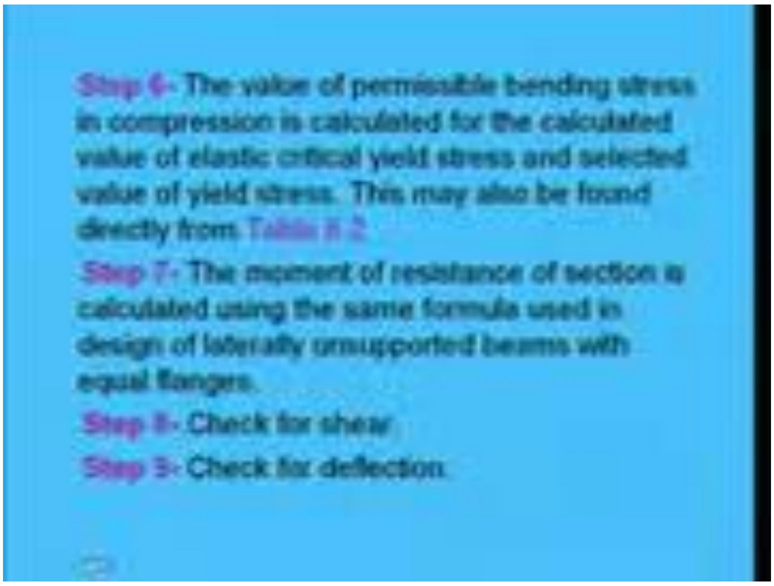


In step 6 what we will do. The value of permissible bending stress in compression is calculated for the calculated value of elastic critical yield stress and selected value of yield stress this may also be found directly from table 6.2. So, from table 6.2 with those also we can find directly the elastic critical stress and sigma bc that is also possible. Now, in step 7 the moment of resistance of the section is calculated using the same formula used in design of laterally unsupported beams with equal flanges.

So, similarly we can find out the moment of resistance; that means, Z into sigma bc. And step 8 will be as usual checking for shear whether the developed shear stress is exceeding the allowable shear stress or not if it is exceeding then we have to redesign otherwise it is and then finally, check for deflection the deflection of the beam whether it is coming within the limit of the codal provision or not that we have to check. So, in this way we can design an unequal flange of beam and plates which are unsupported laterally. So, this was the things which I could not finish in last lecture we have discussed here.

Now, you can start with some sort of problem some example, for this kind of problem you can do at your own and it is very simple because in table 6.2. You will get the data and table 6.345 you will get all other necessary data to find out. And the procedure will be exactly same as we have done in case of equal flanges unsupported flange.

So, procedure will be exactly same only the value of sigma bc will be changes other things will be remaining same. So, I am not going to solve another example for this rather, I will start some other things.

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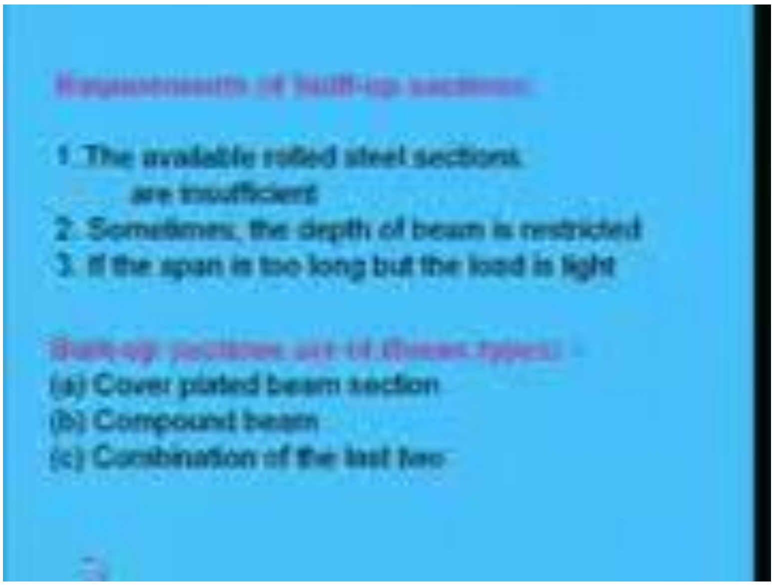


So, what I was discussing in the beginning for today's lecture that is the, built-up section. So, the list of the requirement of the built-up sections can be told in this way that the available rolled steel sections are insufficient to carry the high magnitude of the load. If load is becoming very high then the available sections may not be sufficient to carry. So, in that case, we have to use some built-up section, where the additional plate or other members will be used sometimes the depth of beam is restricted.

Suppose, we cannot go beyond certain depth. So, if we use a higher section of say, I section or some other section that may exceed the limiting depth of the beam due to certain architectural or other purpose restrictions. So, in that case the built-up section means with lesser depth with some additional plate we may have to go. Another is if the span is too long, but the load is light. So, in that case also you may have to go for built-up section, because unnecessary we can provide heavier section because this that will be uneconomic.

So, to make economic we can provide 2 sections with a proper configuration. So, that the section means, the moment of inertia of the compound section becomes very high. So, built-up sections are basically of 3 types. One is generally, what we used to do that, I section or any section with some cover plates. So, cover plated beam section, I sections it may be some channel section it may be. So, like this we can make. Another is compound beam; that means, I section with channel section. So, it will be like this or channel section with another channel section. So, such type of things may come into picture. Another is combination of the last 2; that means, combination of this 2; that means, maybe this I section along with plate section and channel section. So, different type of built-up sections can be made, some of the built-up section let me draw here.

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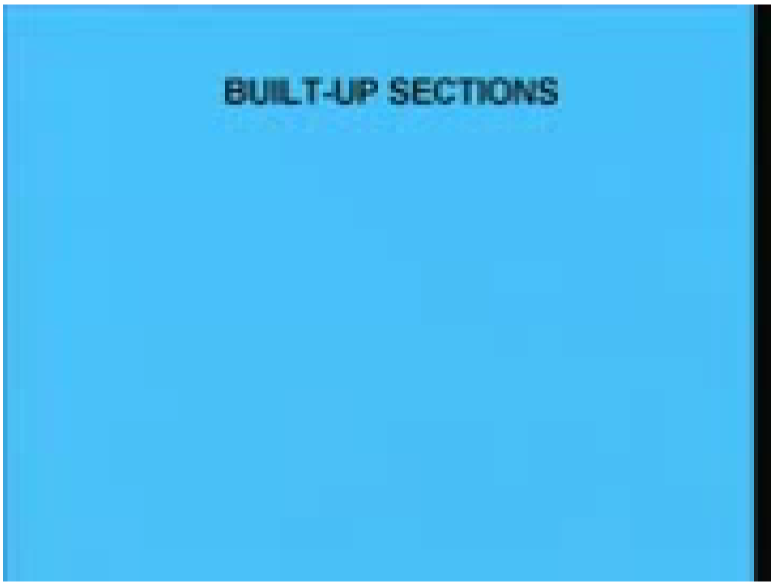


One is generally which is the most common is that I section we used to make with some additional plate and this maybe riveted or wielded this can be done. Another section we use is I am just showing here, those which are commonly used in practice say I section with some channel section at the top. Other is say maybe 2 channel sections with some plate maybe at the top, maybe at the top and bottom whatever it is. Another is saying 2 channel sections and with another 2 channel section another is say I section 2i sections we are providing then we are providing plate. So, all these things depend on how the load is coming, what is the magnitude of load is coming in what way it is coming.

So, according to that we have to find out the configuration of the built-up section. This is another section say only we are providing at the top. Another section is say 2i sections are provided in this way just to increase the depth. So, you see. So, 2 small I sections can be placed 1 in top another in bottom. So, that the moment of inertia of the section becomes I and this is basically to care this case 3 that if span is too long, but the load is light. So, for this case generally such type of sections will be suitable.

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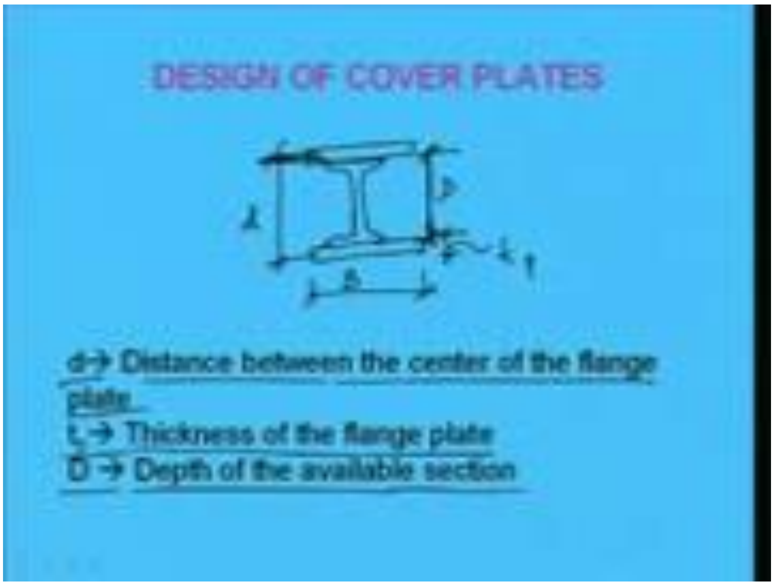


Now, we will see how to design a cover plate means as we are seeing that, built-up section means the some rolled section along with some other additional section. Suppose if we use a cover plate how we will calculate the properties let us see. Say 1 I section is given. So, this is a I section and this is a cover plate. Now, the thickness of the plate say let us have tp thickness of the plate and width of the plate let us make B and say center to center distance of the plate is say d and as we know, this is D the depth of the section rolled section.

So, if we rename the parameters of this configuration in this way that d is distance between the centers of the flange plate. And tp is the thickness of the flange plate and D is the depth of the available section; that means, this 1 D. If it is… So, then how do we find out the cover plate dimension?

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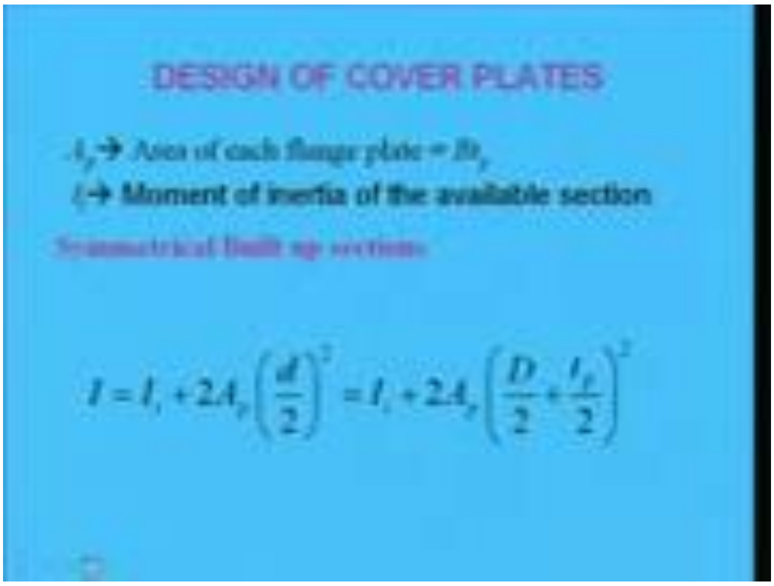


Now, Ap can be written the area of each flange plate which will become B into tp because B is the width of the plate and tp is the thickness of the plate. So, B into tp. Now, Ii we are assuming as moment of inertia of the available section; that means, moment of inertia of this I section. So, with this parametric symbol we can write that, I is equal to Ii plus 2 Ap into d by 2 whole square, this is we are assuming that symmetrical built-up section; that means, the plate of this and plate of this top plate and bottom plate are same area is same thickness is same everything is same.

So, with these assumptions we can write, I will become Ii plus 2 Ap because if this is I. So, when we are making means, moment of inertia of the rolled section. So, this is I, this is I section. Now, when we are going to make some plate here, this is I section. So, I will become Ii; Ii means I of this plus 2 Ap 2 into area of plate into d by 2 whole square; because if this is d then from neutral axis to here it will be d by 2. So, d by 2 whole square. So, we can rewrite this as Ii plus 2 Ap into D by 2 means that, D by 2 plus tp by 2; this is this is total D. So, this will become D by 2 and this will become tp by 2. So, and this total is basically d by 2. So, d by 2 is nothing but, d plus tp by 2 D by 2 plus tp by 2. So, Ar square means 2 Ap into this square.

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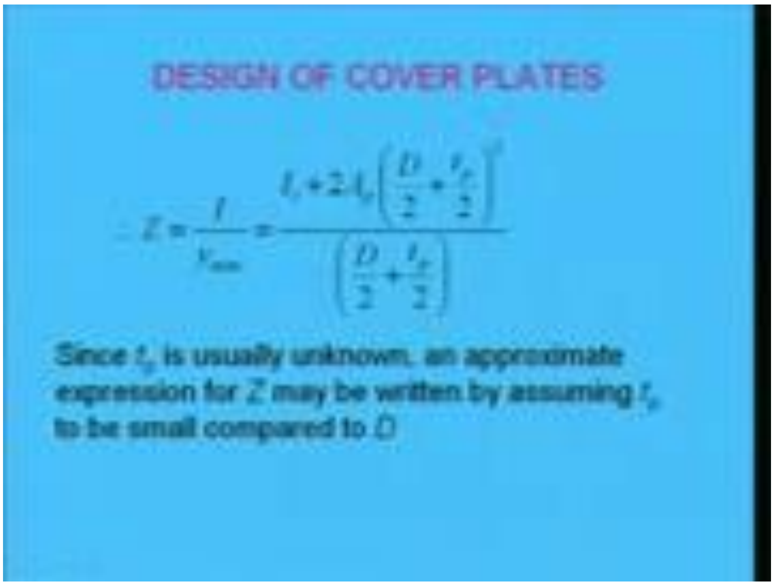


Now, Z will become I by Y max. So, I means; this 1 which we have found here I. So, I by Y max is this 1; D by 2 plus tp by 2 because D plus tp is the total right. If I draw here, it will be clear say I is this 1 and this is plate. Now, here will be the maximum stress. So, I can find out, this distance will be D by 2 plus tp by 2 at this right. And we are making actually it should be D by 2 plus tp, but for dividing these 2 we are approximately making like this. Since you see tp is usually unknown, an approximate expression for Z maybe written. By assuming tp to be small compared to D. So, we are taking some approximation to get the value of Z appropriately.

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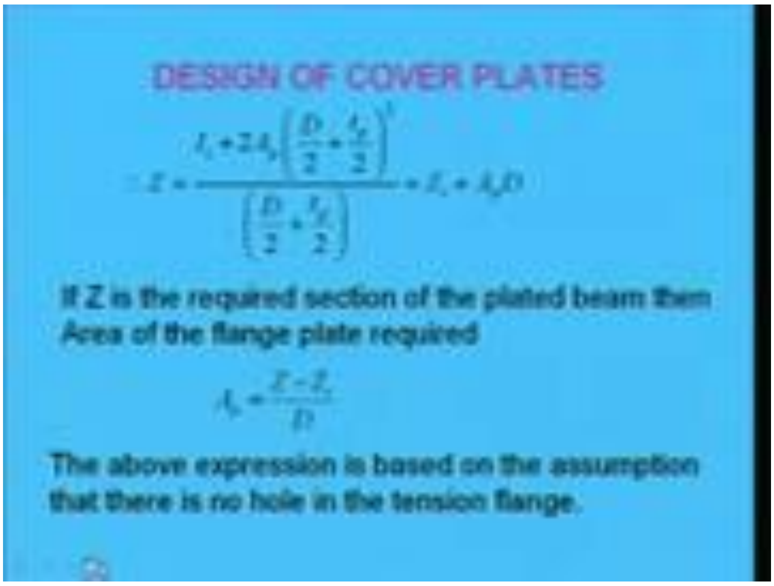

So, what we are doing? So, Z we are going to get in this way and if we make I by this as Zi plus Ap into D. If we derive this equation we will find out finally, that Z will be approximately Zi plus Ap into D by neglecting the tp in some places, because D is much much greater than tp; otherwise we have to calculate as a whole. So, Z now, if Z is required section of the plate beam, then area of the flange plate required will be this because Z minus Zi by D from this equation. I can find out Ap is equal to Z minus Zi by D.

So, the above expression is based on the assumptions that, there is no hole in the tension flange, area of plate we are going to get approximately it is not exactly Z minus Zi by D; where Z is the total section modulus and Zi is the section modulus of the rolled section means, available section and D is the overall depth of the section. So, from this we can find out Ap. Now, again we are assuming that there is no hole and in this basis we are making, but we know when we are going to make an built-up section, either we have to connect through rivet or weld joint. So, in case of a rivet joint we have to make some hole. So, the net area will be going to reduce, net area of the plate is going to reduce.

So, the required area of plate Ap needs to increase up to certain percentage. So, that after deduction of hole, this Ap is will become sufficient to carry that much load. So, for designing purpose we will increase some percentage of Ap to find out the proper dimension of the plate.

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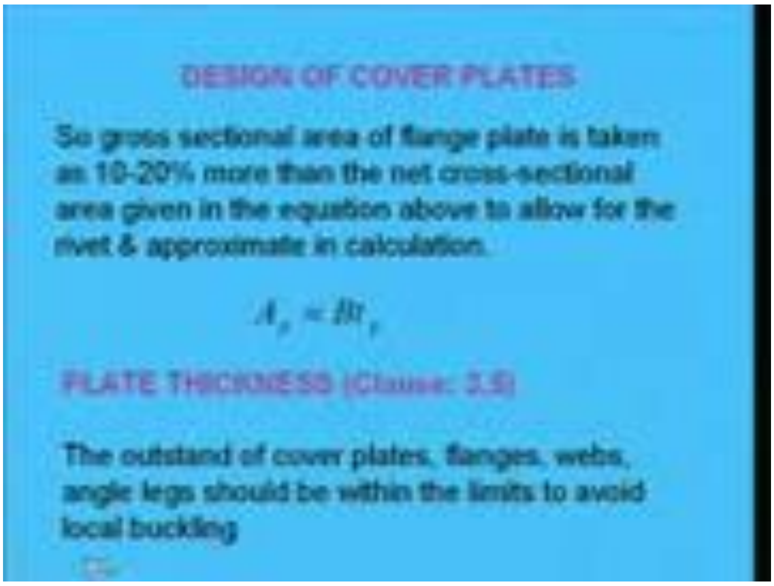

So, gross sectional area of the flange plate is generally, taken ten to 20 percent more than the net cross sectional area given in the equation above; to allow for rivet and approximate calculation. First of all, we have made approximate by neglecting some places tp compared to D; another thing is because of the hole, because of providing hole due to rivet we have to find out the net area which is less than the gross area.

So, we need to increase the percentage of some percentage for the gross area. So, that the net area become equal to more or less that Ap. So, that it can withstand that much load. So, as we know Ap is equal to B into tp. So, that say 1.2 into Ap we will find out finally. Now, the moment we are going to get Ap area of the plate now we have to decide, what will be the thickness of the plate and what will be the width of the plate. Some guidelines has been given by the code IS 894; In clause 3.5 the plate thickness has been described, plate thickness of the section means the additional plate whatever it is that has been told there.

It is told that the outstand of cover plates, flanges, webs angle legs should be within the limits to avoid local buckling, because plate cannot be made abruptly because local buckling may happen. So, outstanding cover plate’s, flanges, webs angles etcetera has to be within limits and that limits has been specified by the code.

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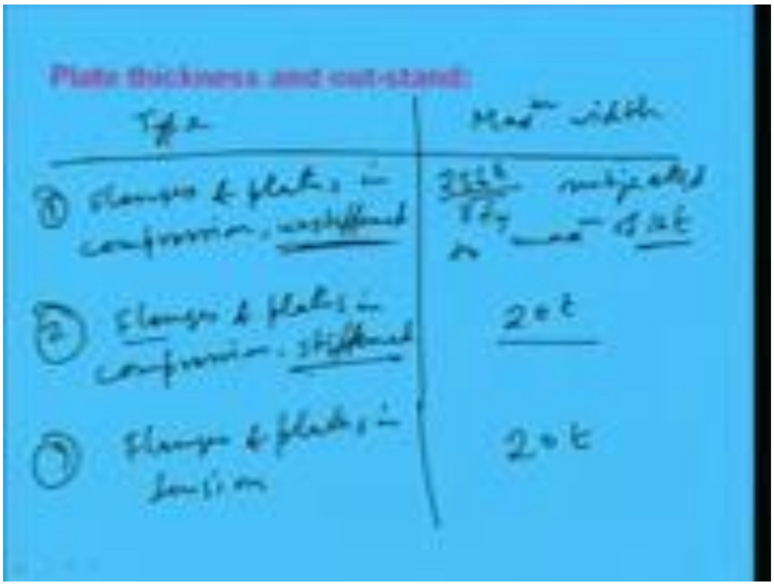


Now, code has given some provisions which I am going to write here. So that, we should have some idea before going to design a built-up section. So, in code, in clause 3.5 it is been told that it, it depends on the type of plate and, then accordingly the maximum width for different type of plate's maximum width has been decided; say, like if the flanges and plates in compression un stiffened, then the maximum width will be 256t by root over fy and that is subjected to maximum of 16t.Where t is equal to thickness, thickness of single plate or the total thickness of the all plates. So, this is 1 case may happen. Second case, is flanges and plates in compression, but stiffened. So, in this case this will be 20t.

So, flanges and plates in compression with un stiffened this will be maximum 16t. And flanges and plates in compression with stiffened plate it will be 20t. And flanges and plates in tension this will be the maximum width will be 20t; where t is the thickness of a single plate or the total thickness of 2 or more plates effectively tacked together.

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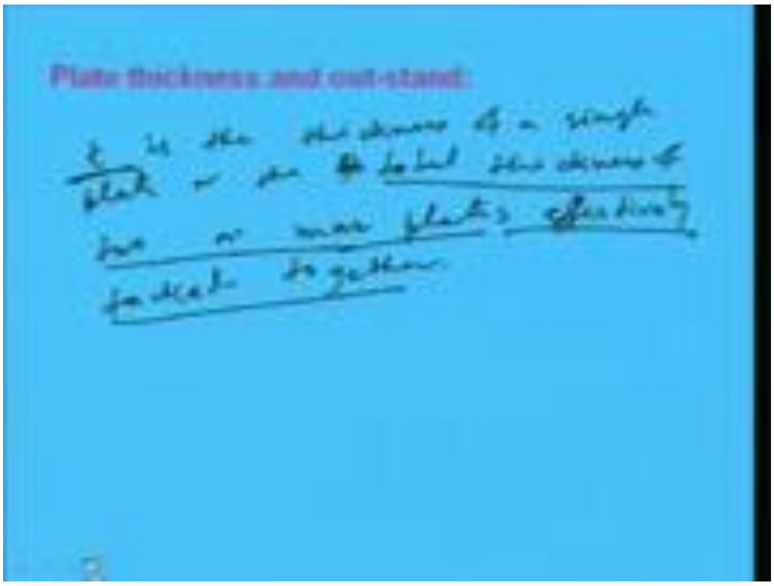


So, this is the t is the thickness of a single plate or the total plate total thickness of 2 or more plates effectively tacked together. So, this is the t.

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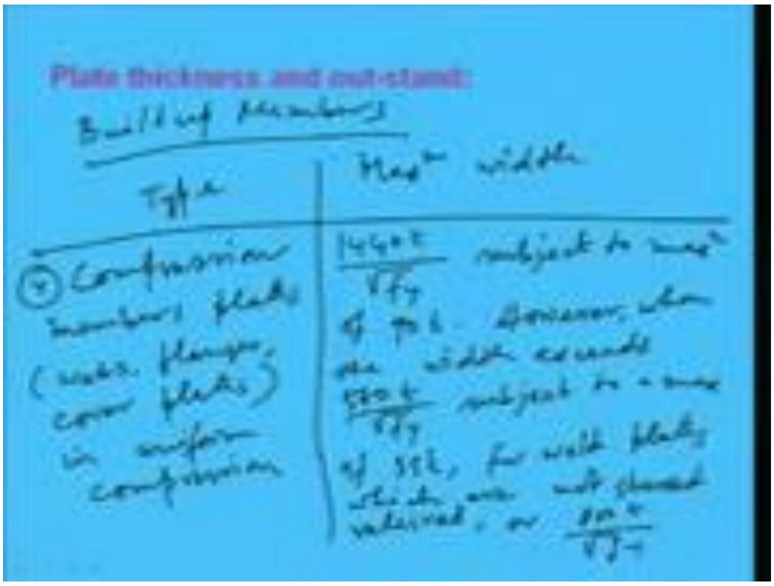


Now, for built-up members say for built-up members let us see how the thickness and maximum width are defined by the code; say, type and say maximum width. Say case 4 now, say compression member this is for built-up member, compression member, plates like webs, flanges cover, plates in uniform compression. For this the maximum width will be 1440t by root over fy subject to maximum of 90t. However, when the width exceeds 560 t by root over fy subject to a maximum of 35t for weld plates, which are not stressed relieved or 800t by root over fy. This place is type and this is maximum width.

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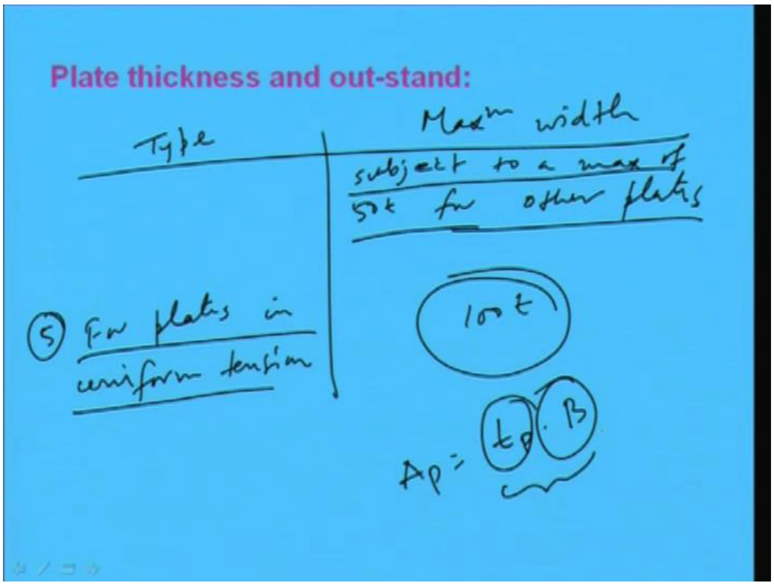

So, it was writing that 800t by root over fy subject to a maximum of 50t for other plates. And number 5, which is last 1 for plates in uniform tension, for this case this will be hundred t. So, plate thickness and outstand are depending on the type of plate. And for plates in uniform tension the maximum width can be taken as hundred t. So, in this basis means with this codal provision we have to decide the thickness and width of the plate because already we can decide the area of plate now, the moment we decided area of plate we have to find out what should be the thickness and what should be the width. So, that area of plate becomes that that constant. That means Ap will be equal to tp into B.

So, how it will be distributed it depends on the codal provision, As per the codal provisions we have to adjust, we have to suitably decide what should be the width and then what should be the thickness.

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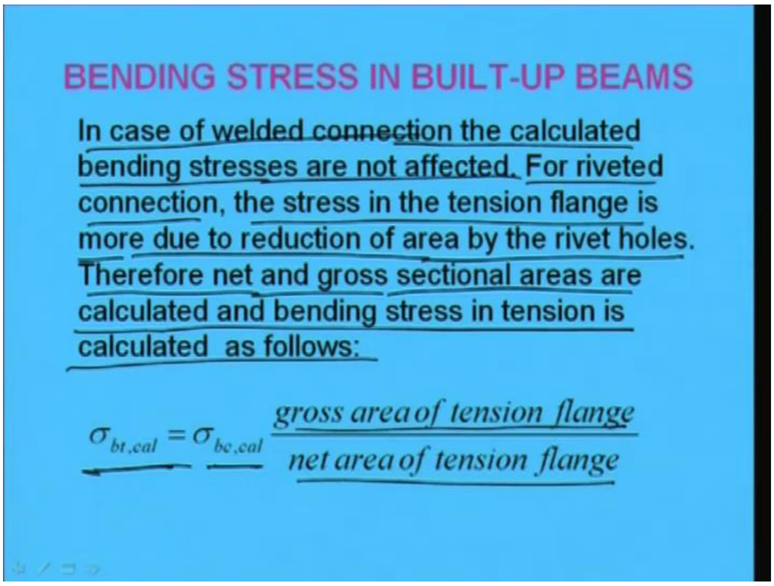


Now, we will discuss about the bending stress in built-up members, how the stresses are going to develop in case of built-up members. As we know, the built-up members means either the connections will be riveted or welded. In case of, welded connection the calculated bending stress are not affected. So, calculated bending stresses are not affected. But for riveted connection the stress in the tension flange is more due to reduction of area by the rivet holes. So, stress will be more. Therefore, net and gross sectional areas are calculated and bending stress in tension is calculated as follows.

So, bending stress in tension will be calculated like this, sigma bt calculated is equal to sigma bc calculated into gross area of tension flange by net area of tension flange. That means; we know what the gross area of tension flange is and we know because of the number of holes due to the rivet what the net area of the tension flange is. So, we have to multiply that ratio with the sigma bc cal. So, that we can find out the sigma bt

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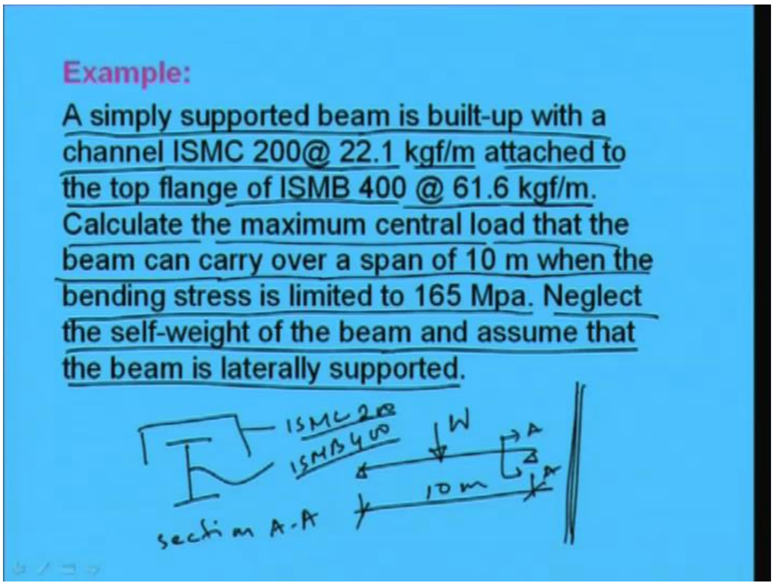


Now, we go through 1 example, through which we will be getting some idea about how to handle the built-up sections. In fact, first example whatever I am going to show here, is a part of; that means, it is not the end of built-up sections because other aspects like curtailment, like how to design the connections from shear point of view etcetera those things I will come later. But before that, whatever we have discussed here on that basis let us try to work out 1 example.

A simply supported beam is built-up with a channel ISMC 200 at 22.1 kg force per meter attached to the top flange of ISMB 400 at 61.6 kg force per meter. Calculate the maximum central load that, the beam can carry over a span of 10 meter when, the bending stress is limited to 165 Mpa. Neglect the self weight of the beam and assume that, the beam is laterally supported.

So, we are going to neglect the self weight of the beam; that means, as it is less. So, let us neglect then with with neglecting the self weight of the beam we will try to find out what will be the load carrying capacity of that beams means what will be the maximum load centrally placed for this particular beam. So, what are the things are given, that this is a first of all simply supported beam and carrying a central load say W and the length is given it is ten meter and if we see the section. So, section will be looking like this. So, this is ISMB 400 and this is ISMC 200. So, sections are placed like this and loading conditions are like this and end conditions are like this. So, from this we have to find out what will be the maximum central load that beam can carry.

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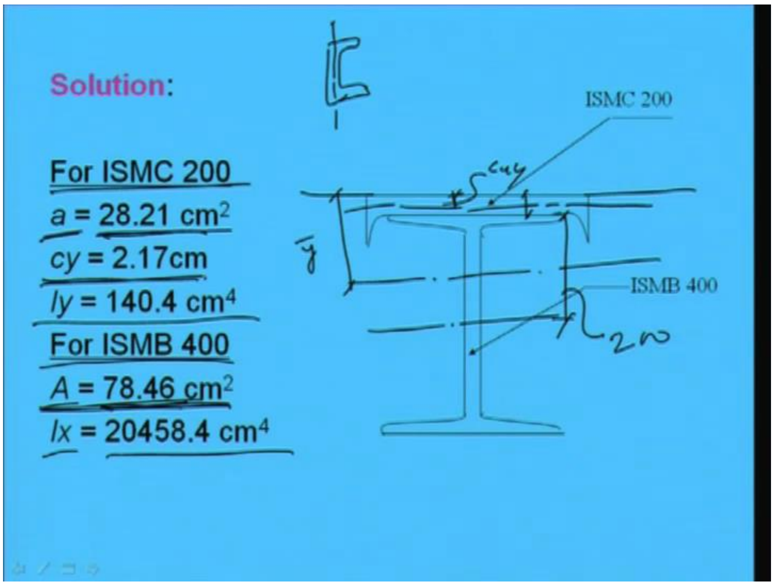


So, what we will do? So, we have to find out the relevant properties from handbook that SP 6 from SP 6 we can find out the given properties of ISMC 200 and ISMB 400. Say for ISMC 200 what are the things required is the area, cross sectional area is given as 28.21 centimeter square. Similarly, cy means from this will be the cy that is 2.17 centimeter and Iy is 140.4 centimeter to the power 4; Iy means, if we see the channel section like this will be Iy because here Iy is will be this 1 and will be required for us. Similarly, for ISMB 400 we need area is given 78.46 centimeter square. And Ix is given as 20458.4 centimeter to the power 4 the moment of inertia is given.

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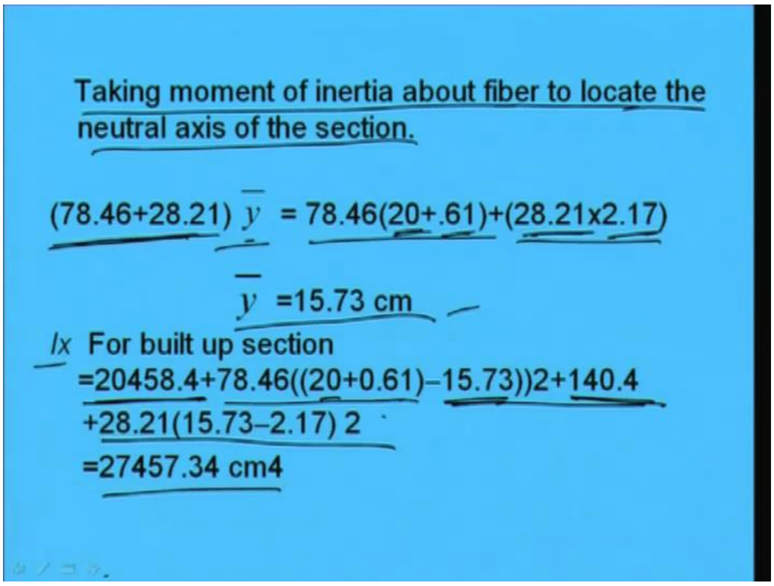


So, with this value what you can do; we can take the moment of inertia about the fiber to locate the neutral axis of the section about which 1 the top fiber. So, if take moment of inertia about this, to locate the neutral axis we can say that total area into y bar. Total area is this 78.46 plus 28.21.So, total area into y bar; y bar means say if neutral axis is acting here. So, this is y bar distance between extreme fibers at the top to the center to the neutral axis here. So, this is equal to 78.46 this is the area of the I section. So, 78.46 into distance will be its individual neutral axis is here. So, from here to this will be 200 because this is ISMB 400. So, this will be 200 plus the thickness; this thickness was given.

So, 200 means 20 centimeter and thickness was given 6.1 millimeter. So, that is tw thickness of wave of the channel section. So, 20 plus 6.1 and similarly area of the channel section that is 28.2 1 and then 2.17 cy is 2.17. So, in this way we can find out. So, y bar is becoming 15.73 centimeter. So, neutral axis depth from top fiber we can find out. Now, we have to find out the Ix means, moment of inertia of the built-up section about this. So, Ix will become Ix of the individual section of the I section plus Ar square because of shifting; Ar square means, this distance we know because 200 minus this y bar 200 minus y bar or we can simply can find out what is the distance, then we can provide you see this plus point 6 minus y bar.

So, we can find out the shifting means moment of inertia due to shifting of the neutral axis then neutral axis of the channel section. So, finally, we will get 27457.34 centimeter to the power 4. The viewers can calculate the moment of inertia about x axis at their own, because in other way also you can find out whichever you feel easy you can calculate in that way; finally the result has to come same.

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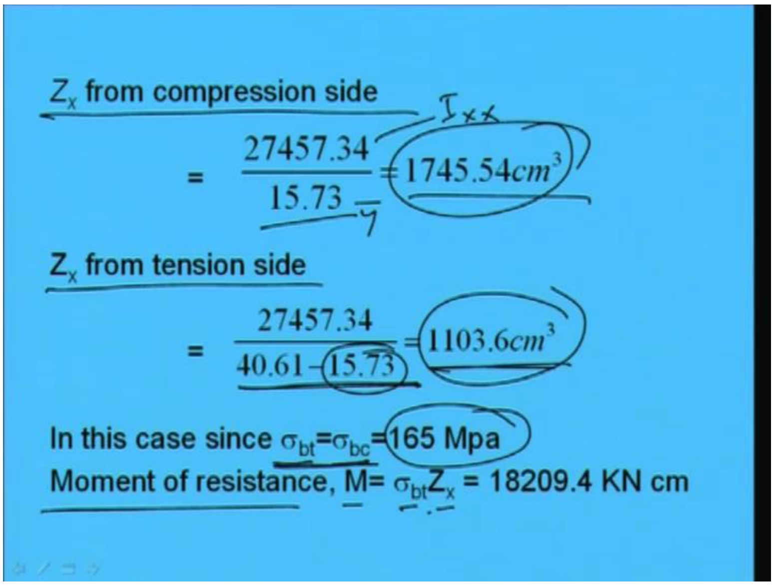


Next what we will do? Next we will find out Zx section modulus from compression side. From compression side, we can find out this is y bar this is I. So, Ixx by y bar. So, we are getting Zx is like this. And Zx from tension side we are getting this is total depth minus y bar; that means, from bottom side. So, this we are getting like this 1, 103.6. So, in this case as it has told that sigma bt is equal to sigma bc means, because of laterally restrained means at the question itself it is told that the sigma bc and sigma bt can be can be assumed as 165. So, moment of resistance will become M is equal to sigma bt into Zx because this is less, this is more. So, failure will be at the bottom first due to tension. So, M will become this after multiplying this we will get like this. 18209.4 kilo Newton centimeter or 182.0 94 kilo Newton meter.

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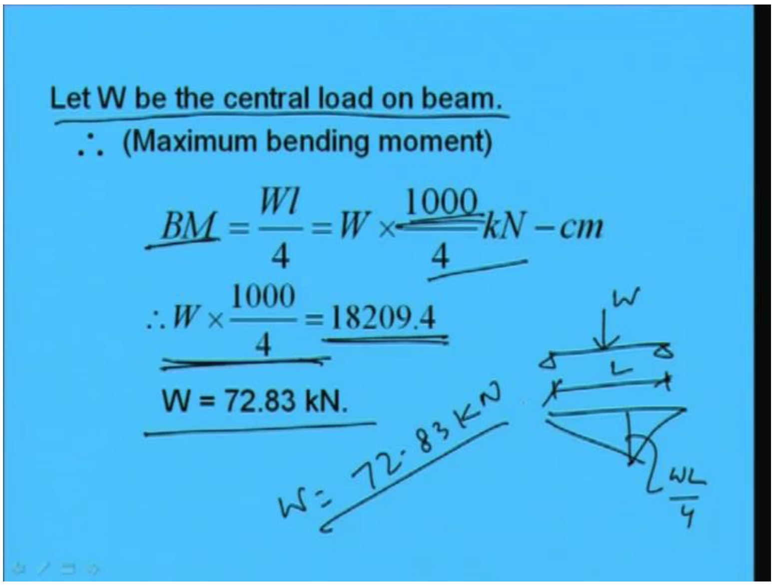


Now, if the W is the central load on the beam then maximum bending moment we can find out Wl by 4 because for a simple supported beam maximum bending moment will develop like this that is Wl by 4 if this is W and this is l. So, Wl by 4; that means, l is 1000; 10 meter it was. So, thousand centimeters Wl by 4 is coming this. So, now, if we make equal. Equal means, the moment developed due to load and the moment resistance capacity. So, from this 2 we can find out the load carrying capacity as 72.83 kilo Newton. So, W we are getting the load carrying capacity central concentrate load for this particular beam is 72.83 kilo Newton.

So, this is how for a compound section for a built-up section we can find out the load carrying capacity of the beam. Here you can observe that we have not given means, we have not gone to details of the curtailment connection riveting or wielding other things we have not gone which we will go step by step in details. Just to freshen up our study means, whatever up to whatever we have done on that basis we just solve this problem.

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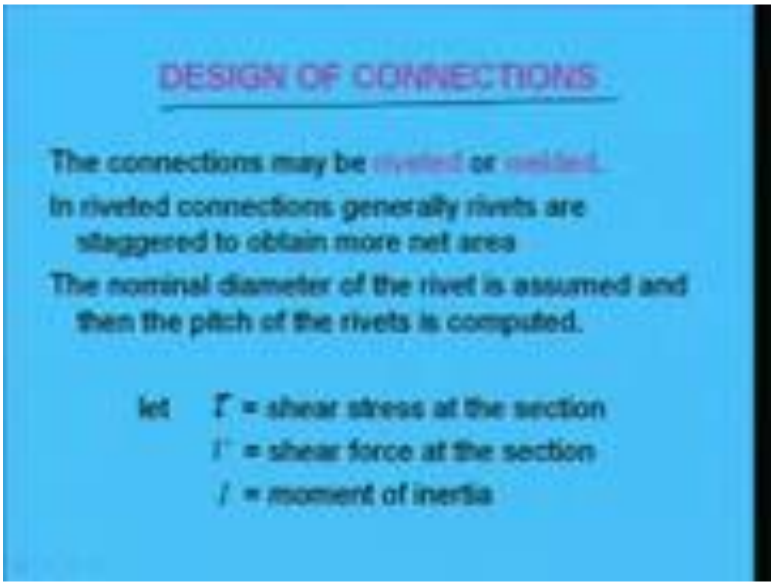


Next we will discuss about the design of connections. Now, as we know connections means connection between the plate and the main section. That means, if we have a I section along with additional plate, then the flange of I section and the plate has to be connected properly. So, that it acts monolithically and the connections maybe done through riveting or wielding. In rivet connections generally, rivets are staggered to obtain more net area

So, when we are going to connect say, if see from top if this is the plate which is connected with the say this is the flange from top if I see. So, it should be some sought of stagger not in same line. So, that the net area can be obtained more. So, in this case failure will be either here or here or here means 1 hole only we are getting. So, net area will be more. Otherwise the failure may happen like this, like this, like this; where also the net affective area will be more means, in place of 2 in same line it will be very less.

Now, the nominal diameter of the rivet is assumed and then the pitch of the rivets can be computed. So, while designing for connections what we can do we can find out some nominal diameter mean, finding out means we can assume some nominal diameter then as per the codal provision we can find out some minimum pitch. So, pitch also you can find out.

Now, to find out the shear stress; let us, denote this tau as shear stress at the section and let us assume V as the shear force at that section and I is the moment of inertia of the whole section. Tau is the shear stress at the section.V is the shear force at the section. And I is the moment of inertia.

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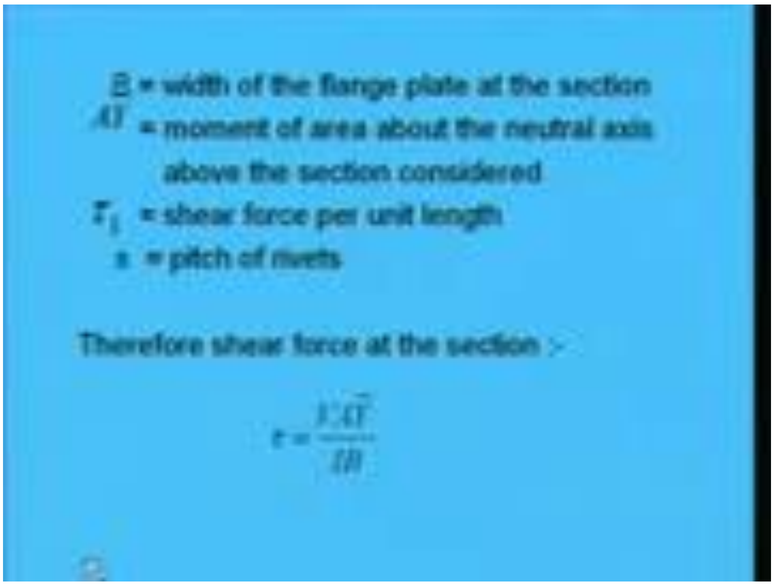


And other things like B are the width of flange, plate at the section width of flange plate at the section. And AY bar is the moment of area about the neutral axis, above the section considered; that means, Q. VQ by IB we used to provide for getting the value of tau VQ by Ib; this Q is nothing but, A into Y bar. And let us assume that, tau 1 is the shear force per unit length and s is the pitch of rivets.

So, with this parameter we can write the shear force at the section means, shear stress at the section tau is equal to VQ by IB. Where B is the width of the flange of the plate and V is the shear force at the particular section, I is the moment of the inertia of the whole section and Q is nothing but, AY that is moment of area about the neutral axis above the section considered where we are going to consider that 1.

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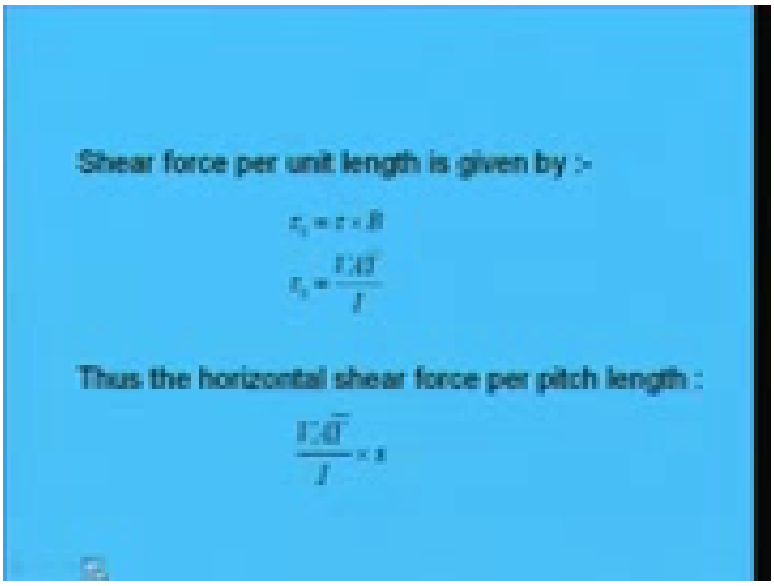


So, shear force per unit length we can find out that is will become tau 1 is tau into B, where B is the width of the section. So, tau 1 can be find out as like this, because this tau was VAY bar by IB and into B. So, this will cancel. So, we can write tau 1 as VAY bar by I. Thus the horizontal shear force per pitch can be calculated. Now, shear stress per unit length is this means shear force per unit length. Now, so shear force per pitch length can be find out if we multiply with the pitch where s is the pitch. So, this is the horizontal shear force per pitch length VAY bar by I into s.

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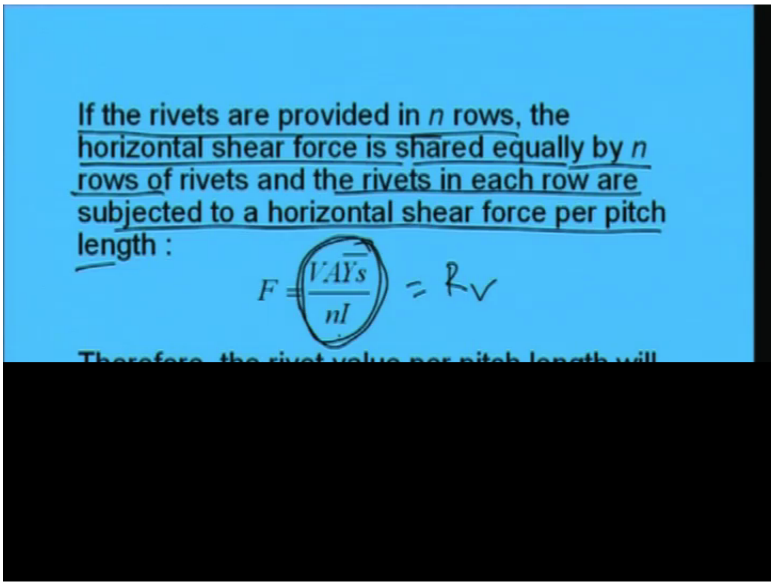

Now, how many rivets are there per pitch. We can assume that, if the rivets are provided in n rows means how many rows are there. Say let us assume the rivets are provided in n rows, then the horizontal shear force is shared by equally n rows of rivets and the rivets in each row are subjected to a horizontal shear force per pitch length. That means F will become total by n because n rows of rivets are there. So, shear force is going to distribute to that n number of rows.So, finally, F is going to become VAY bar s by n into I.

So, R pitch means per rivet in every rivet this much force shear force is developing this much shear force, is developing in every rivet and the rivet has to carry this much force. So, we can say this has to becoming as rivet value; that means, the rivet value per pitch length can be written as Rv is equal to this 1 right VAY bar s by nI. Now, if number of row is 2 we can write Rv is equal to VAY bar s by 2 I. If number is 1 then simple I. So, it depends on that, For n is equal to 2 Rv can be written like this.

So, in this way the rivet value per pitch length can be find out. Now, we know for a particular diameter of rivet what is the rivet value and the value whatever it is coming here we know. So, from that, we can find out that whether the rivet whatever we have provided with a particular pitch and with a particular number is safe or not.

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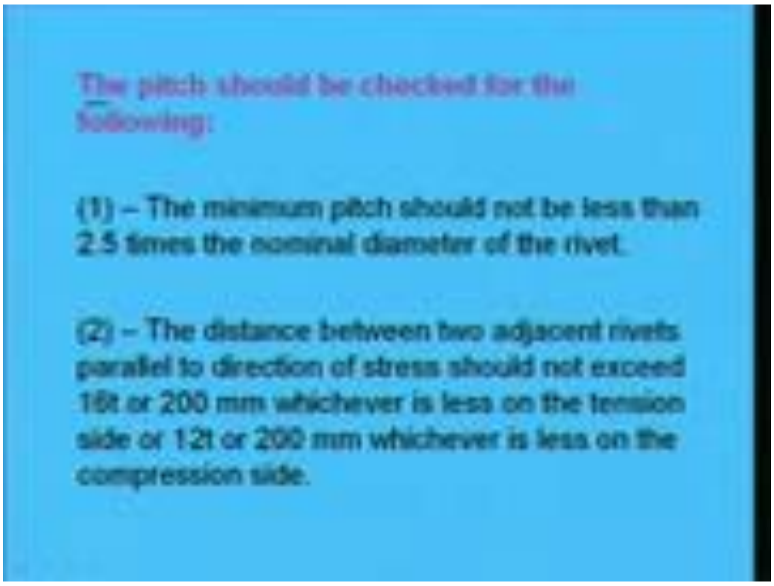


Now, the code has given some provision for providing pitch, that the pitch should be checked for the following, because we cannot provide arbitrarily the pitch we have to maintain some codal provisions.

So, what are the things let us see; first is the minimum pitch should not be less than 2.5 times the nominal diameter of the rivet minimum pitch should not be less than 2.5 times the nominal diameter of the rivet this is 1.Second is the distance between 2 adjacent rivets parallel to direction of stress should not exceed 16 t or 200 mm whichever is less on the tension side or 12t or 200mm whichever is less on the compression side. So, while providing the pitch distance we must take care of all this codal provisions.

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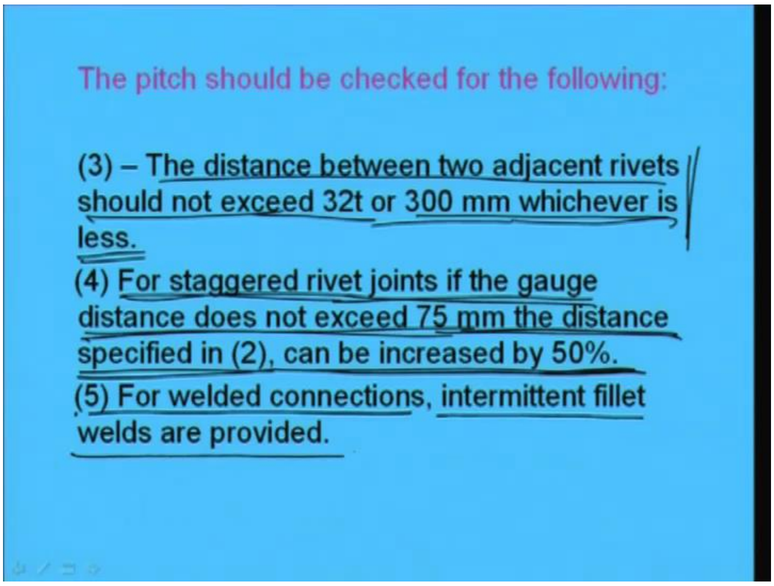


What are the other codal provisions? Other is like the distance between 2 adjacent rivets should not exceed 32t or 300mm whichever is less the distance between 2 adjacent rivets should not exceed 32t or 300mm whichever is less. Another is for staggered rivet joints if the gauge distance does not exceed 75 millimeter, the distance specified in 2 can be increased by 50 percent; specified in 2 means this 1. Here we specified the distance. So, that can be increased by 50 percent that is for staggered rivet joints if the gauge distance does not exceed 75 mm the distance specified in 2 can be increased by 50 percent. Then another option is that for wielded connections intermittent fillet welds are provided for welded connection intermittent fillet welds are provided.

So, these are the provisions which have been defined in code we have to remember before deciding the pitch distance for connecting plate with the flanges. Other things are curtailment. That sometimes we have to see that the, plate whatever we are using additional plate may not be required throughout the length of the span, unnecessarily we do not need because moment is going to reduce as per the end condition and loading condition. So, where the moment is less, we can provide less section; that means, and we can reduce we can omit the additional plate and where the bending moment is more we can introduce some additional plates with length. So, that also has to be taken care properly, how it will be we will see in next class. So, today with this I like to conclude and tomorrow, we will start the remaining part.

Thank you very much.