Design of Offshore Structures
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Module - 4
Lecture - 11
Tubuler Joints Design for Static and Cyclic Loads XI

So, yesterday we were looking at the members subjected to tension so you can see here as I emphasis yesterday the difference between LFRD and ASD does not exist.

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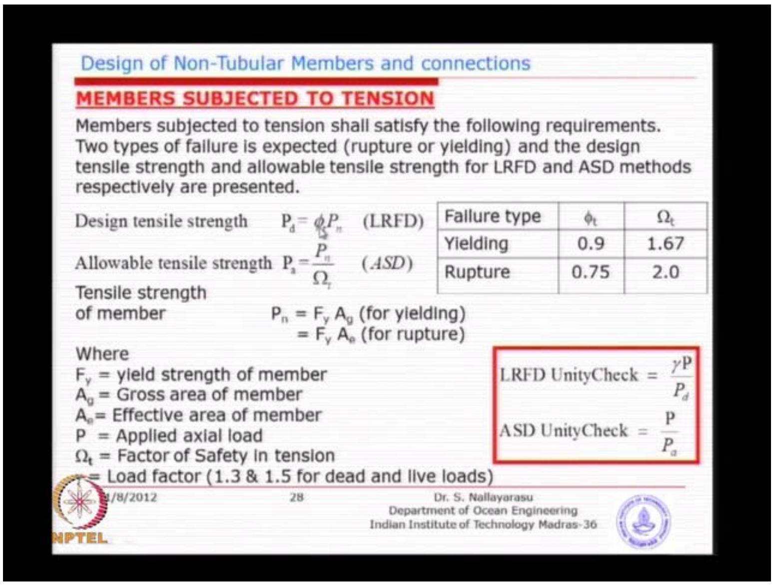

This is useing 1.5 as a load factor divided by 0.9 which is basically the material factor coming from here. So, because the P d is at the bottom, so 1.5 divided by 0.9 will also become 1.67 is almost you will see that there is no big difference as longest, the consist values are used. So, here the capacity is taken basically as a tensile strength of the member, which is gross area, which is multiplied by the yield strength of material F y times the cross sectional area of a member.

If it is a pipe section, you will use a circular area calculation or if its high section flange area plus the web area. The next the rupture only will happen once you have for bolted connection, for example if beam is connected, so if I take a cross section across the bolted section, then you will be having a net total area. You will have a area where minus d volts and that will cover by rupture which is basically the a longest of whole and then starting to break rather than to failure by yielding. So, that is why there is a slight different of this factor 0.9 for yielding without any holes, there as there is a purposeful wholes made and then the rupture is going to happened when there is a reduced factor of 0.75.

So, calculation for tension because very easier because area times yield will give you the yield capacity no other forms of failure is going to happen. You imagine if you have an member in full tension, definitely either the global buckling or local buckling is going to happened this is under tension. So, that makes calculation for tension is very simple, at the same time if you go back to compression, you will do the same thing, only thing is the stress is not going to be limited to yield value.

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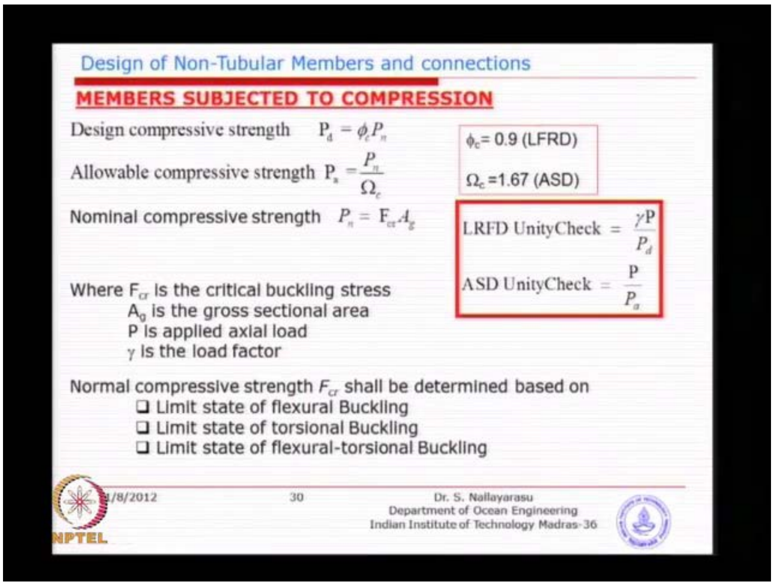

You would not be able to reach yield unless this member is very strong quite big wall thickness big enough to avoid all local failure. For example, local buckling I think we did talk about during the tubular section, you know the local buckling because of the d by d ratio global buckling because of KLBRS.

So, same thing is going to happen plus you are going to have additional problem, what we saw yesterday is the torsion buckling because of the instability due to longer length are smaller section. So, this news to be checking that is what we are seeing here you have to verify whether the section is susceptible to anyone of this failure. That means you will not reach the yielding and you failed earlier by one of them and you limit the stress to that that is basically the FCR, which is a buckling stress. It could be local, it could be global or it could be lateral torsion buckling and that is what we are going to see each one case. We will just see what will be the limits and what are the parameters involving.

Once you find this FCR value, then you calculate the nominal strength, which is basically again multiplying area times the buckling stress and divide by factor of safety. This will give you the allowable strength multiplied by the material factor will give you the design strength for LRF and allowable. The format is almost similar, only additional things that we are trying to do here is basically the buckling failures due to insufficient section is too long and boundary conditions are different. Again, the unity check is similar gamma times the load divide by the design strength are low divided by allowable strength, which is the again the similar idea for the attention.

So, the FCR only defers in terms of calculation for pipe section, what we were having we were having d by ratio, we were having KL by ratio and we were looking at D by ratio for basically leading strength not for compression always you are using your KL for your allowable axial stress, but of course D by ratio have some influence, whereas here we are going to look at the flanges open flange, where this also partly stiffen connected to the flanges.

So, what we are going to see whether the flange and web are the slender or they are non slender. So, that means whether they are not adequately stiffened, you have few cases whether you can have non-slender are we can have slender with stiffen slender without stiffness. You will see three or four cases which you will take into account the effect of that on the dad buckling stress that will come out here instead of F y we are going to calculate this.

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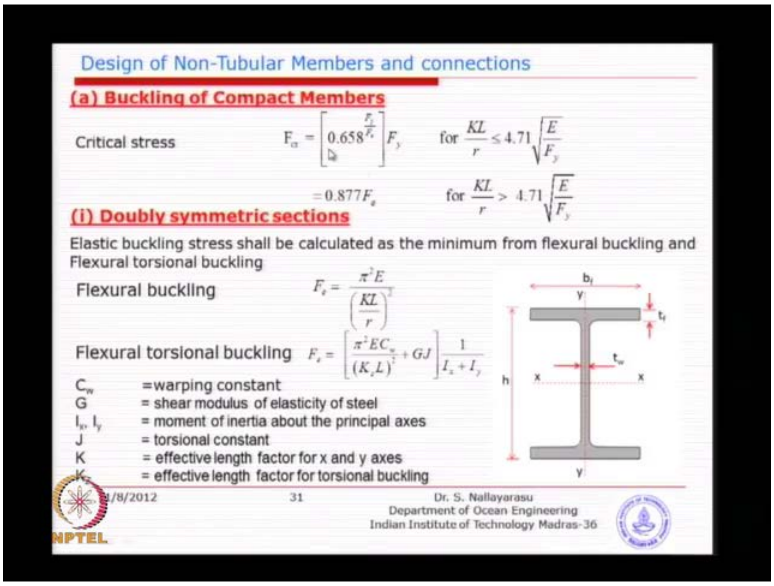

So, buckling of compact members basic idea is if the members are quite compact that means there is no slender less on. Then, you straight away calculated in this empirical formula, you can see here is not something very easy to the remember is a empirical formula from test. So, they have conducted several test, so 0.658 raised to the power F y by F e, F e is the oiler buckling stress, I think most of you will remember this formula pi square e by k l by r square depending on the boundary conditions. You know this k will change whether it is a pin condition fix conditions cantilever.

So, you calculates is buckling stress and then substitute in this formula to get the value of the FCR, now this FCR depends on the KLB ratio like this. I think you might remember the formula that we introduced for tubular system quite complicated to remember listened is a little bit of long form, whereas here quite simple state away a factor multiplied by yield strength this factor is basically the calibration involving F e and when the KLR ratios greater than this value, you will be using straight away 8,7 percent of F e itself not the F y.

So, that is the difference that we are trying to make, this will be a case governing by a buckling, this will be a case governing by almost close to yielding. So, that is that is exactly the idea and what I have written here is double symmetry section because these are the simplest cases. You will go into single asymmetric, you may go into unsymmetrical cases, which are also part of the ASE which I have not repeated here, but procedure is slightly longer. This means the symmetric will create different forms of equation that is all the difference, but the procedure is same.

So, I have given to symmetric sections double symmetric the flanges symmetric can the web is symmetric about the y axis. So, this is basically the flexural buckling, which we saw about it earlier that torsion buckling the formal is given little bit different. Basically, what I have explained yesterday regarding the load apply at the centre, but at the same time when the length becomes longer starts to rotate and create at torsion failure. That is why this is sometimes called flexural torsion buckling are sometime lateral torsion buckling one and the same.

So, you do not need to worry about this that means even before global buckling occurs because global buckling is because of slenderness. This is basically KL by R value K and L will represent the lender the member are represented reduce the direction corresponding to the section. So, KLBR will be global buckling phenomena, whereas the flexural torsion buckling almost is mixed to boundary conditions basically global as well as the local flange effects is going to affect the larger flange is going to fail early. So, that formula also a empirical formula, so you can see it is a derived form for from oiler buckling plus the effect of torsion is taking into account.

So, pi square ECW warping constant depending on type of section and k z will be with respect to torsion effect not respect to bending about AXRY. I think I have given this access here, x and y is perpendicular the access of the member. So, the jet axis will be along the member length, so you need to find out how they are restrained unless you are restriction like yesterday, I have some pictures something like this something like this if you have restriction of the member about rotation.

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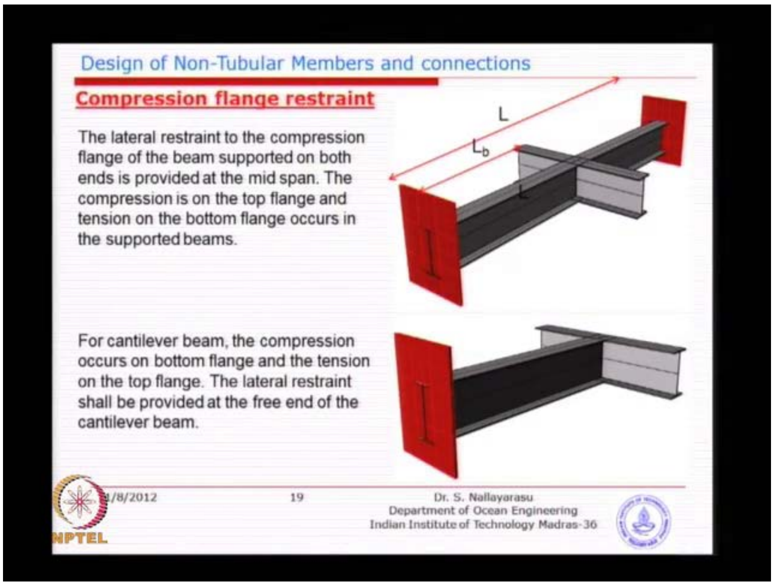

Then, you can specify that length instead of the full length, so basically this formula well involved k z times l and then here you have anyway conventional i x and i y. I think might remember you know in your applied mechanics calculation polar moment of inertia calculation of x and y, so above the principal access. Then, you can find out the polar moment of inertia or if it is a simple section it is easy, otherwise you find x and y convert into the polar moment of inertia, so all this values are knows for the particular section.

So, i x, i y, k z and modules of elasticizes is known, you can calculate back the sheer modules simple relationship from elastic theory. So, you can calculate that also are you can rewrite this equation in terms of e just simply put e divided by 2, 1 plus mu. I think you can calculate that also, either you governed by this, you have to find out which is values smaller and then calculate. Now, single symmetric the difference between the double symmetric and single symmetric.

Basically, the flange is going to be different, the top flange and bottom flange may be smaller bigger many situations you will get into this type. You know in ship hull construction, you will get very often this you will have a ship hull plate, whereas stiffness will be t section. So, when you want to take a equivalent section, you will always find that the stiffener will be thicker and the self plate will be smaller.

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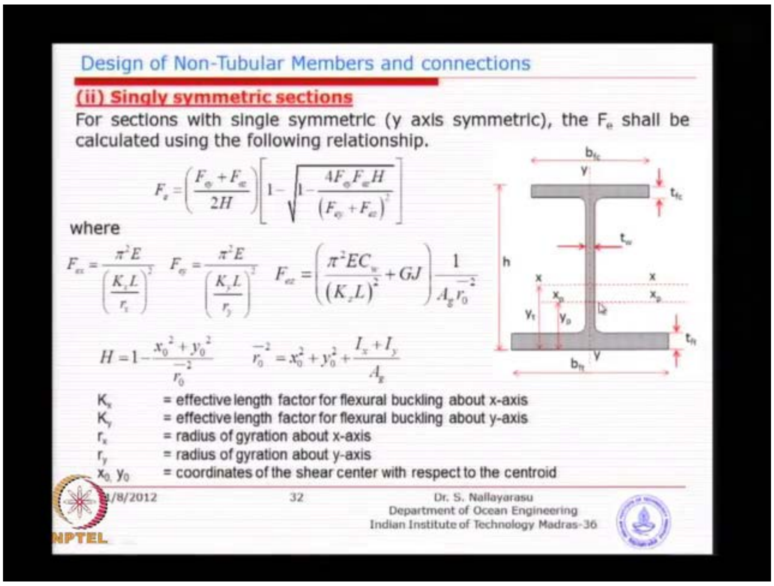

So, when you find out equivalent cross section, you will something like this most of the hull bulkheads will be like this. When you are doing building construction maybe not this type because we always preferred a symmetric section with runner beam and top of with you will put the deck plane, whereas ship construction, we do not do this like. You will have a hull plate and this stiffened by t section sometimes angle bars, similar session, so in that case, the idea is similar. Whatever we were looking at, only differences you see a formula getting slightly complicated because of the asymmetric of the sections.

So, F e calculated using this formula and corresponding values of a F x a, F z, F y and F z is to be calculated is this formula. So, you can see here slightly complicated, so imagine if you have a doubly asymmetric section, so it will become even difficult and basically those are channels and angle bars are fabricated section. You will see that these are only just matter of doing calculation, but the procedure is similar, so the values will be replaced by a complicated formula rather than the procedure is not changing.

So, double symmetric are singly asymmetric, only the values is to be calculated is to corresponding formulas. So, in here is just explained k s, k y you know very well is flexural buckling about x and y axis like oiler buckling and r x, r y is radius of gyration x naught, y naught is a sheer centre symmetric x, at least once single symmetric. It will be along the web line, it is double symmetric, double asymmetric, and then you have to calculate c a centre in the sheer flow theory. This will be slightly difficult for this type of section c a centre will be the centered of the section.

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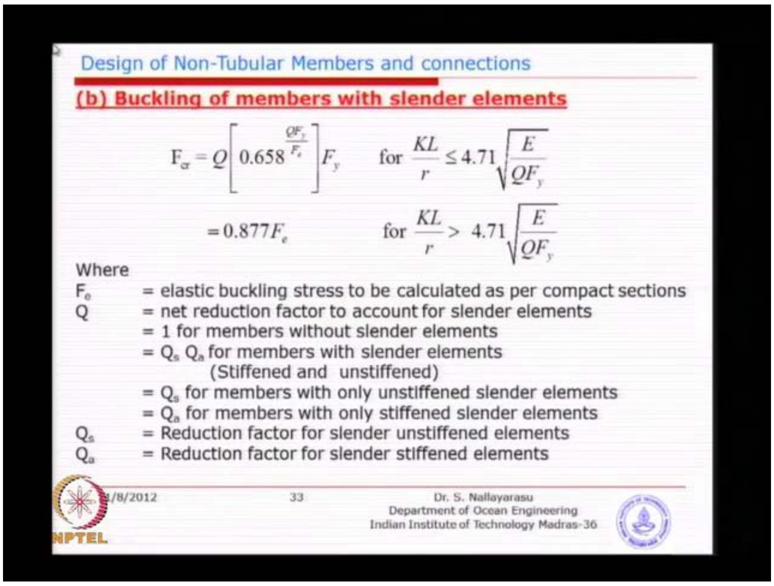

Now, whatever we saw basically buckling of compact member, no slender elements that means you classified you saw one table here just go back here for clarity member classification. For compression elements we had flange we had web that means non-slender.

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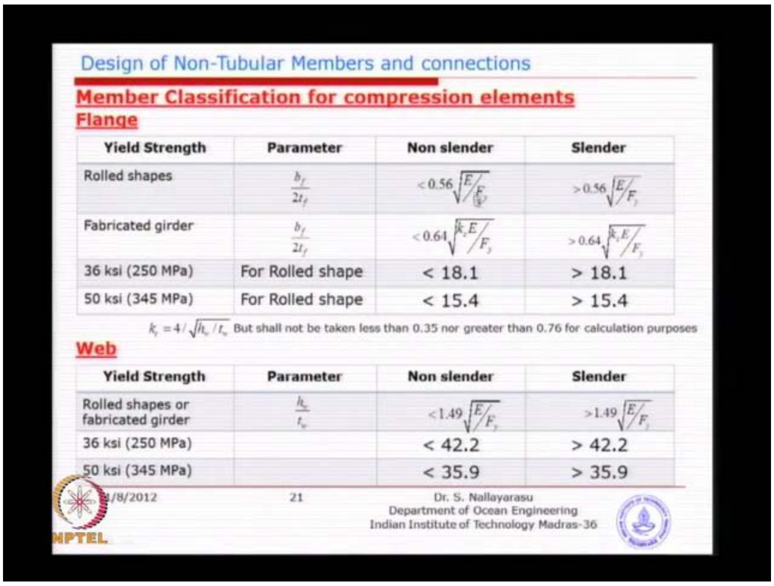

We were all looking at you look at the web and look at the flange, find out the thickness to with ratio are with thickness ratio as longest. They fall within the category, they are non slender elements, which is basically a compact that is what we were looking at this now that is why the subclass a, is far buckling of compact are non slender.

I would put whether non slender, then if you look at slender elements, what we are trying to do is follow exactly the same procedure, but because of its slenderness, we want to have a reduction factor. This is basically a factor called q, it must be less than 1 because that is what I was trying to emphasize early as first few classes. When you are designing structures, it is not that because your intelligent you want to reduce thickness.

You are going to pay the price somewhere else as longest, you make them slender thinner trying to reduce weight, that is what idea know, but then allowable stress will come drastically down either the increasing ratio are increased D by ratio. This means it is going to be becomes slender and then allowable stress will be drastically reduce, which is not a good idea because ultimately we were trying to reduce, but then allowable is reduced, then you will go back and then make it bigger.

So, that means as longest you keep reasonable D by ratio gale by ratio in here width to thickness ratio. For example, when a flange within limits of compactness, you will get full allowable stress once you make slender automatically, the allowable is down that is what the idea the q value is to account for the slenderness of the flange and the web. So, that the idea behind again you will have a similar slenderness bifurcations KL by R less than 4.7 times e by F y and then greater than.

So, when it is greater, the buckling is governing the slenderness does not come into picture, but the slenderness does come into picture here, basically multiplied by q and also on the super script of the 0.658. So, how do we calculate the q is the simple matter, so q will be a multiplication is too component 1 is q s and q a. Now, you can see here when you select, you have two things can happen, the flange compact web maybe slender or web maybe compact the flange.

So, that means they could be a mixed of compact non compact elements is not both of them if both of them compact, then no problem, because the multiplication becomes just 1. So, the idea you can have suppose if you have a fabricated many times, if you do this we have flanges and then maybe three are four webs many cases. We use in after construction because restriction of debt will given to you it is not there you can infinitely increase the depth increase the mode inertia. In such cases, you will have you know many webs in cases of very heavy sheer force coming web will be able to take it.

So, you may put one more than one web so in here one is for members with no slender elements q s multiplied by q a member with slender elements q s and q a stiffened and an stiffened. So, basically you will calculate separately q s for members with the elements a stiffened q a member only stiffened. So, you have slender stiffened, but still you would not be able to achieve 100 percent because there could be a reduction, so these values.

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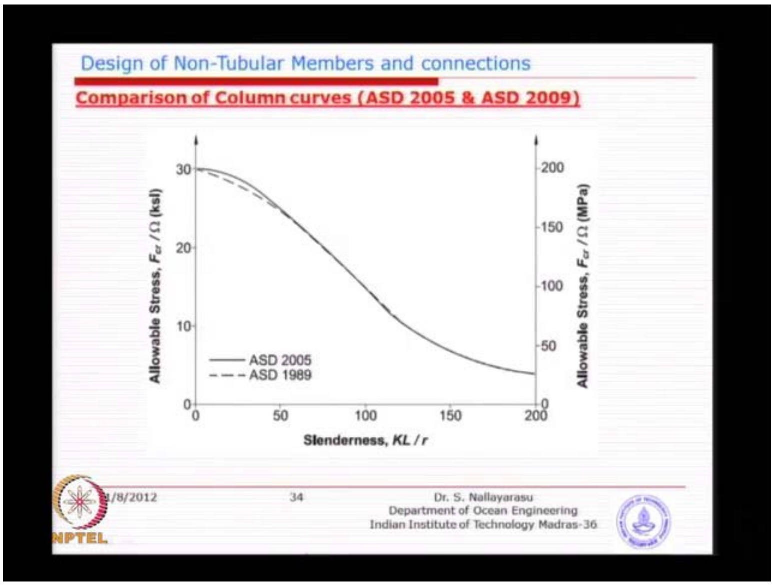

This is to be calculated again, they are empirical formulas given for that, I think this coming here. We can see the relationship between KL by ratio which depicts this equation basically this equation the relationship between allowable stress, which is what basically sheer by how. Basically in terms of mega Pascal, this is your k psi value horizontal axis; this is your KL by r. Remember we were having similar graph for tubular section, almost similar parabolic type.

So, you can see here is longest, you keep less than 100 KL by ratio somewhere around here, you will reach typically about 100 mega Pascal and if you have a KL by ratio larger and that drastic reduction in you know allowable stress. This is not very good and that is why most of the codes especially gale by ratio restricted to 120 only. Their cases go beyond and you have to limit that and in case of some of them big structure, even rested to gale by ratio less than 100, especially primary structures.

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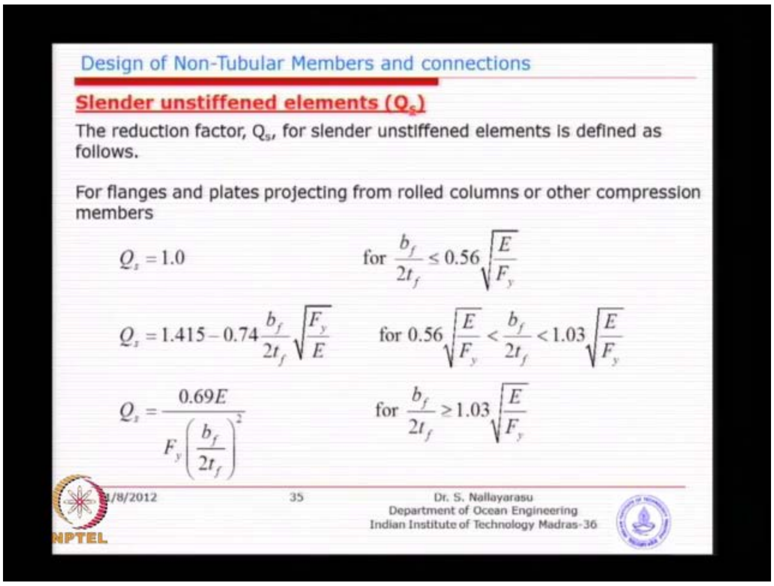

So, q s there is a formula, wherever you have b f by 2 t f less than 0.56 e by F y, then the value of q for an stiffened elements is 1. That means is already compact, if you go back to the compactness table, just let us just quickly go back is in here that the value we were looking at. As long as you keep less than 0.56, then you are value becomes just 1, so no reduction in the allowable stress. So, if you are between this and this, there is reduction factor and then slender stiffened to basically that is what we are going to see. So, between 0.56 and 1.03, the values of e by f is constant anyway, which is going to be produce around 28 depending on what is the yield strength material.

The q s value is take this kind of form is an empirical form, so you have to careful, but what happened in the recent days, comparing the previous addition of IAC manual. So, ninth addition thirteenth addition one good thing as happened this all of them as become non dimensional does not matter whether you are using metric units are you using English units the earlier version off course highly dependent on units.

You have to use only the English units convert back all the values to your metric units though you are using the designing using you know metric units are the Newton SI units are those equation, the older court than ninth addition. Before everything, you have to follow their American units, otherwise those this numbers are all corresponding to such type of units. So, because of the difficulty faced by designers, now they have come up modified form everything as in non dimensionlized.

Here, F y by e is does not matter as longest you used consistent units, do not go use in one unit one and the other work different, and then it becomes a problem. So, as long as you use consistent units, so you do not have a problem b t, it cancel as you can see here b by 2 t f greater than 1.03. Then, the value becomes another form, so q values have to be calculated as longest you have slender elements.

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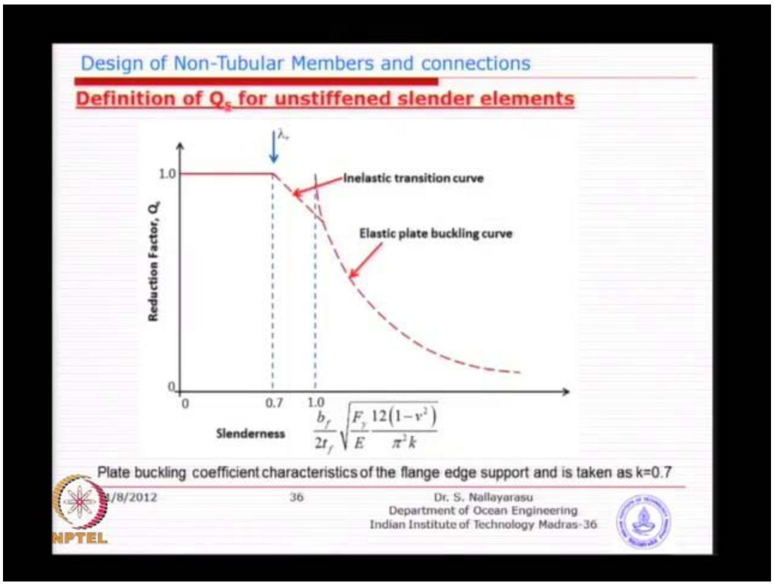

Especially, the flange and they are not stiffened that means left as such.

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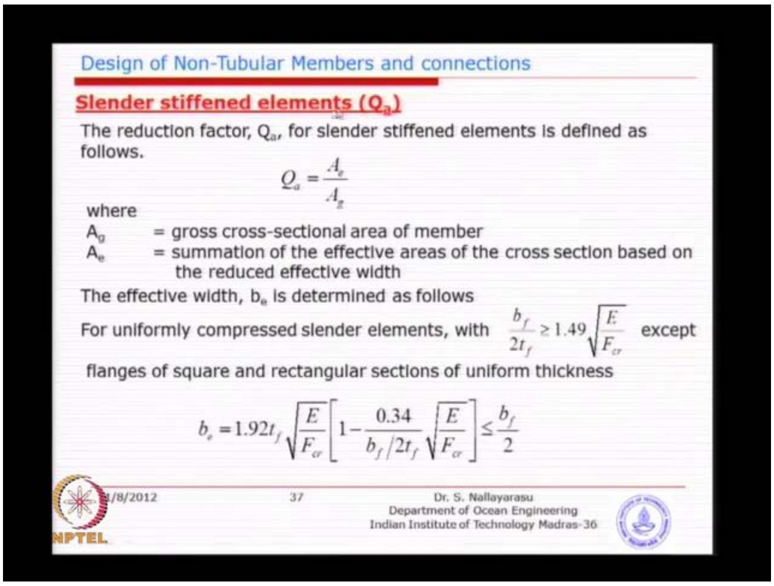

Similarly, before seeing this for slender, but they are stiffened, you could see here as longest you have such elements some stiffening. For example, many times what happens is you want to make the very deep, because we want get more moment of inertia thinking that by Shipley simply shifting the flange away will get more moment of inertia? That will be correct provided you have sufficient thickness for your web, otherwise the web becomes slender, so even before you achieved the moment of inertia is going to fail by local buckling of the web itself.

So, what we normally do is, we do that and also gone provide the intermediate stiffness is stiffened the web itself. So, it is not coming free by simply making the web height higher and higher, you are not getting free moment of inertia, you are going to spend more yield on stiffening them. So, the buckling does not govern the design, so basically here q a is defined as the area of you know the summation of the effective area of the cross section based on the reduced effective width.

So, whatever is the width that you are originally provided, because it is becoming slender, so you reduced with you calculate using this formula and the cross sectional area of member, which is basically the summation of the web and flange. So, that is the ratio of that we are going to get as longest you have this reductions smaller the Q A value will be become one means to close to the full area as longest the reduction value is larger. Then, you will have less value of which will have correspondingly reduced allowable stress, both have them that the graph, how it varies as longest as slenderness increases. You can see here the value from one drastically reduces, so that is one for the stiffened elements similarly, for stiffened elements.

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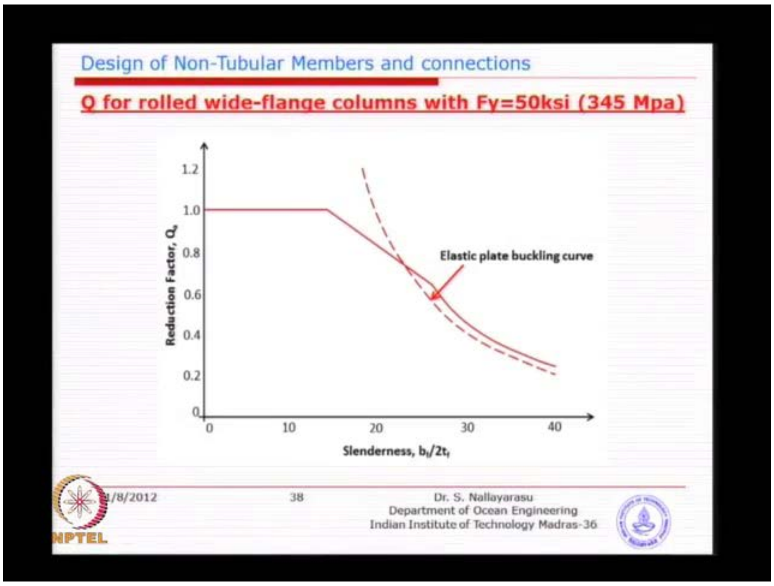

You can see here just for one particular value A plotted its one for up to slenderness of above 11, 12 after that start to reduce drastically below 0.5, which is not very good. So, ultimately you have 0.5 times 0.5, it will become very small allowable stress, so as well go back and then instead of putting a lot of stiffening, you make the section less than the value of slender.

So, that the idea beyond, so if you go back here for slender elements, we do have a provision that you calculate the reduction factor in terms of stiffened element and un stiffened element. The next one what we want to look at is the bending, so we have here tension, we have look at compression. Both of them involves the area cross-sectional area and yield strength are buckling strength buckling strength is governing by KL by r plus the stiffened, un stiffened.

Basically, the compactness of the members and flanges and web, the next one is we are going to look at is the major axis bending. I think for the pipe section. So, we did not talk about major axis, minor axis, which has circular section, which uniform property all are around principal axis same. Here, we do have a problem because is not uniform throughout and basically the horizontal axis.

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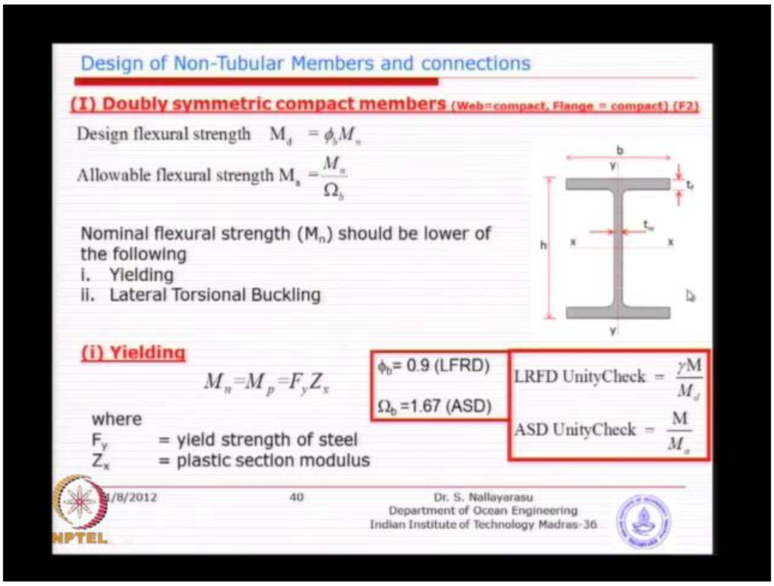

We normally notation given as a x and y, does not matter and basically the major axis bending the vertical axis is the minor axis as you can see. When you calculate the property itself, you will find the flanges are kept away, so you will get a higher moment of inertia. You can give a notation as major axis bending and predominantly you know this means placed for this nobody would not turn. So, major axis bending and minor axis bending, what is the difference suppose if you go put one flange or one or one more web on this sides, then they become actually does not matter whether this is a major axis that is minor axis.

It becomes almost similar, but most of the idea behind which we are talking about yesterday we wanted to increase the bending one plane large bending is there the other plain bending is not their you know horizontal banding for fixed structures is very small maybe a very little. So, that is why you maximize the binding and what we are worried about is the major axis trend. So, if you look at here again, similar idea allowable structural strength and design strength, this is for LRFD.

This is for the allowable stress design and the nominal strength is given by yield strength multiplied by the modules of the structure. So, this is where one major difference has been made. As long as I look at the IA state contend or any other chords, including the ninth edition of ASV chords, this has always has been elastic section modulus, but in the recent times, a lot of studies shows that in the form of yield it takes higher strength than actually elastic. That is where you see this plastic suction as long as suction is governed by yield; you can go up to plastic suction modules.

That means you are having highest trend, so by making the members compact, this is what we are going to achieve, you are going to get higher strength than what we are going to get. When you make it slender or non compact, now you see here yield strength multiplied by the section modulus will give you the nominal moment capacity.

This is basically that we are looking at divided by a factor of safety will give you allowable movement strength and multiplied by the material factor will give you the design moment capacity. The unity check is exactly the same as what we are looking at gamma times m, m is the applied moment external courses divided by design moment or the design capacity. Similarly, for allowable stress design m divide, so you see here the material factor is 0.9 and basically the allowable stress factor is 1.67 is almost same. Now, the matter is how to find out this m n if yielding is governing, which is the first one is very simple, because you have five multiplied by section modulus.

Section modulus calculation is simply using plastic theory which we learned about the other day you can calculate. If it is a complicated ship; it may take a little bit more time. If it is a pipe which is very easy to derive i section, which is symmetric, which I have given you I think in the previous slides. So, that is not a problem, so it can be governed by yielding. This is easy or if it is governed by torsional buckling, then we are going to look at what is the value of this F y will be replaced by buckling stress that is exactly the difference.

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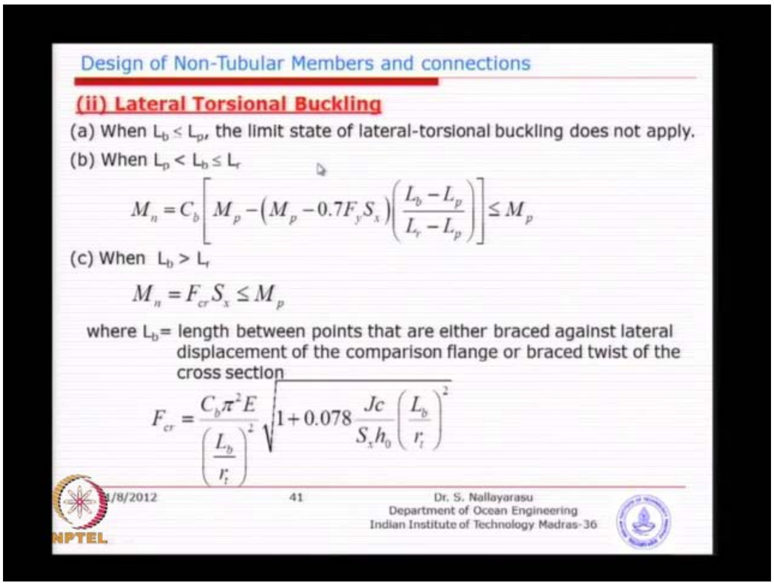

So, what we are looking at is the torsional buckling may govern as long as either the section a smaller compared to the length is a relative this is not just obsolete. You can have longer length as long as the section is bigger and you are going to torsional buckling. So, that is where you will see that the section property coming to picture length of the member from support point from the point where it is restricted from rational buckling. That means a rotation is prevented, so you have three cases while is basically the length of the compression plans restricted against rotation which is called L b.

So, the definition of L b, I have already shown one pictured the length of the member is full from support to support. We have got one lateral member which is preventing the member from rotation which is basically the L b so if L b is less than the L p which is going to be calibrated using some empirical formula in the next page as long as you keep your length of that unsupported compression flange, then you do not have torsional buckling happening that means if the flange is reflected then it is good that will be the case most of deck structure in offshore you put one big deck plate no like you have a floor plate and weld the floor plate to the beams. So what will happen to floor plate will not allow the beams to rotate so most of the cases you will see that this will be very small where it'll be bigger in case where you have open sections there is no floor but, maybe there may be open grating you know the to open deck floor then you will have such kind of problems.

When you have two cases L b is between L p and L r and L b is greater than L r, basically the relative term and these are moderate case this is an extreme case. So, basically please two cases you will calculate the instead of F y multiplying by F y. Here, you will be replacing that F y with this value of calculated stresses, basically in this year this whole term is replaced by this, whereas in the case last one L b greater than L r, you see here FCR multiplied by elastic section modulus. It is the difference that you find, so you go back here you have full yield stress multiplied by the plastic modulus is going to give your very large moment capacity when you made the section very small.

When you made the member longer, you see here the moment capacity is reduced by almost you are reducing by a 30 percent here and the case where it is becoming too large. Then, you actually go into buckling at all, so you have the model is replaced by plastic to elastic you will see that for I sections, I think we remember what is the ratio of plastic to elastic somewhere around 1.2 to 1.3 or 1.5 depending.

So, another 10 to 20 percent reduction, so you can see here allowing torsional buckling to happen is not a good idea. You know just going to reduce it and again you will see a large formulas for again the form of the formulae is very similar to oiler buckling. You can see here only the additional term is added on the right hand side which is empirical just to fit you know the test results. So, FCR is calculated using phi squared e and here you see slightly different form of L b because it is not any more KL by r.

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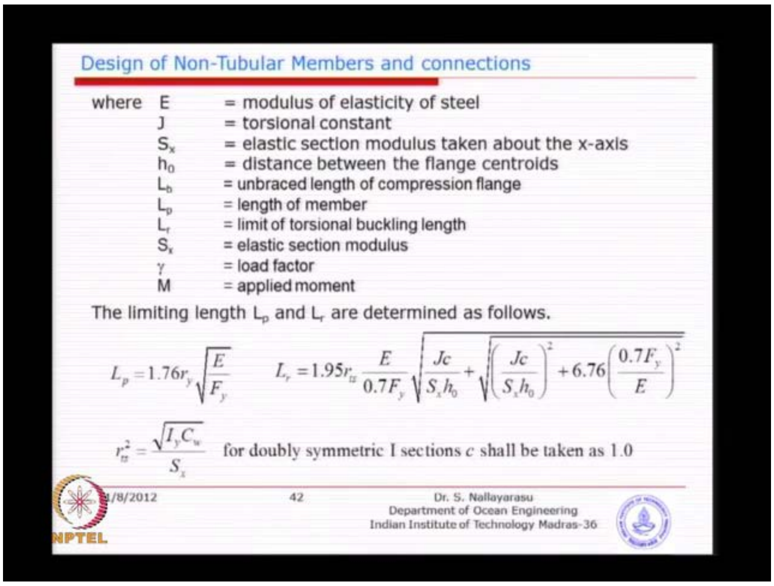

Basically, the torsion prevention by means of lateral compression restriction and the L p values and L r values which are used here you see your L p and L r are again calculated using this formula. This formula is to be very simple before the thirteenth addition because it was based on k SI units, but when they try to do non dimensional this formulas as become quite big very hard to remember, but does not matter. So, when use the codes to refer most of the time because memorizing this is no use, so the L p values and L r values are the limit values where the bifurcations takes place from this equation to the equation for calculation of m n. It is a typical graph which described what we were talking about the slenderness.

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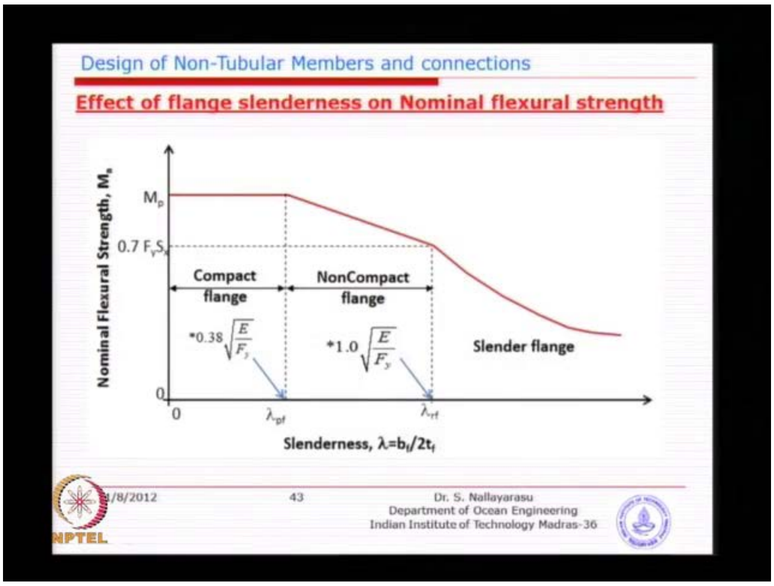

You know the a nominal flexural strength which is basically the moment capacity of the section. So, you can see here as long as you keep the the flange compact which is basically the compactness parameter. This is b divided by 2 t f because total flange divided by half divided by the thickness will give you the compactness of the continuous session of the flanges one side which is this will be around 9.2. So, as long as you keep it less ten you could see here becomes compact flange to achieve full capacity which is F y times the plastic modulus and then in the transition region.

Basically, you get a reduction of about seventy percent this is described by the second formula. This formula is basically the last one which is basically here will reduce less than 50 percent which is not very good having the section and make the plans bigger just because we wanted to be increase in the moment of inertia. You can increase the moment inertia, but then the allowable stress decrease not a very good idea most of the time it is designed, we try to keep it within this limit if possible if it is not possible and you want to still than go for between, but never ever come into slender flange.

The section will not be able to achieve that capacity, so that is give an idea of how the variation of the moment capacity with respect to compactness of flange. Most of the predefined section described by the course whether it is an American courts are Indian course mostly. They make it compact flange and you do not it really worry if you check it will be compact only fabricated are fabricated beam, which we are making from plates we will have verify this is basically the relationship between the unbrace.

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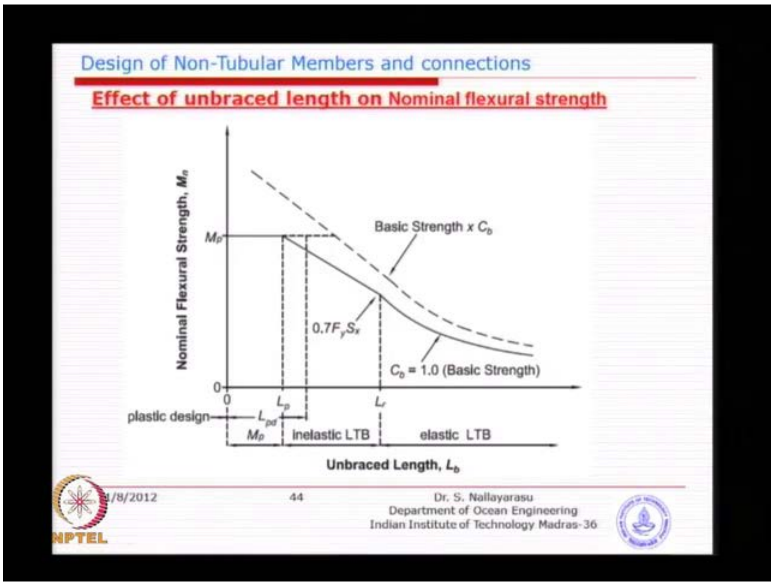

The length with respect to flexural strength, so you see here the m n and b r is basically the elastic torsion buckling almost somewhere between here to here. The equation that what we are seeing inelastic and the plastic capacity this is only used for plastic capacity design where we use for you know the accidental cases. So, it is exactly opposite, so most of the cases will be using in this form the value of c b which has been appearing in many of the equations we will see here c b, here is depending on the moment the distribution across.

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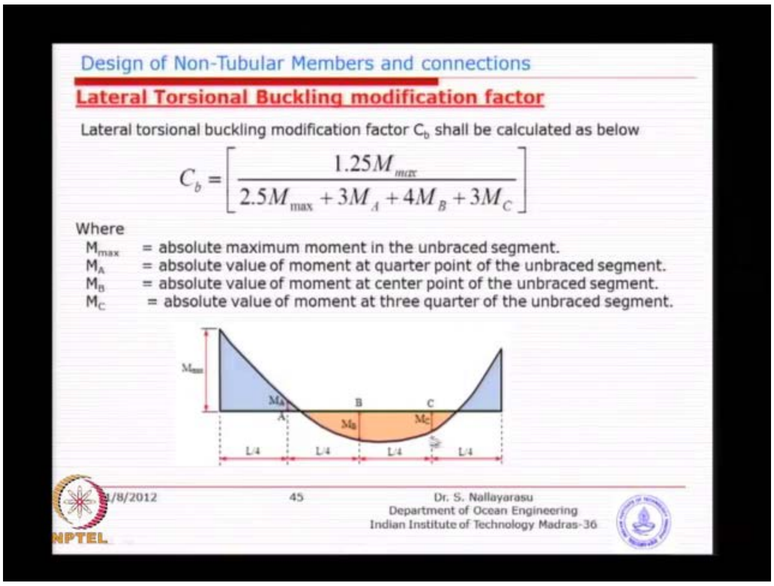

The member very similar to the values of c m which we talked about during the design of tubular member is depending on the basically the distribution or the profiler the moment along member length. The typical case I have just given you an idea of the moment variation you know ABC and the maximum moment absolute maximum and then the moment at the locations ABC. Basically, A is one quarter middle and the three quarter and will give you an idea of the reduction factor.

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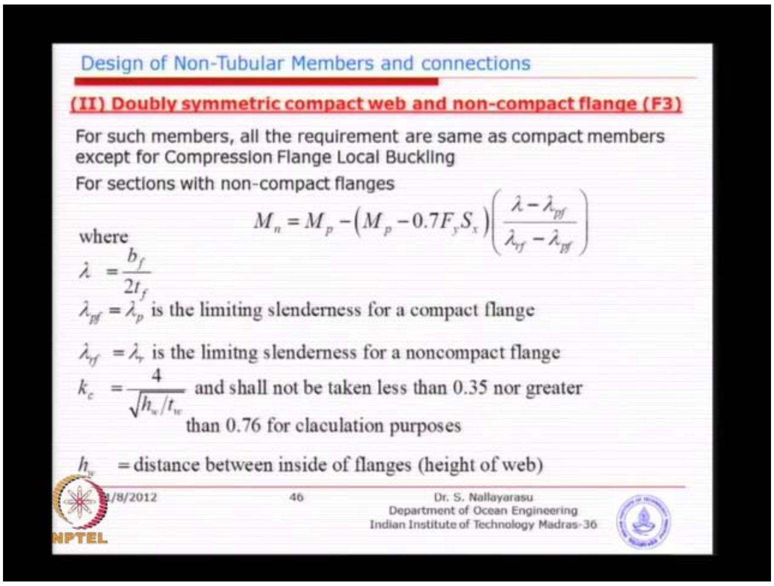

For the torsion buckling which is C M, we will talk about the torsional buckling in tubular members in here it is C B and you calculate and apply this value to reduce the buckling similar procedure can be followed. I have just described also has have just given you the reference number. In case you some of the parameter are missing, you can go back because there is almost eight or nine cases starting from compact, web compact flange is all the way up to the slender web and slender flange. I have picked up three cases for this percentage in basically and this number represents is section number in the AFC code.

So, let you do not get confused in fact some time you get confused, so basically this one doubly symmetric compact web non-compact flange what we have. So, just now compact web compact flange that what we saw just basically A, one of the case. So, when you go to doubly symmetric again compact web at the non-compact flange, so flange is going into the transition region.

You will see a similar approach only the formulas will be different and you will need to calculate the nominal moment capacity M n go back your design equation substitute. Then, there the next one, so in this basically a need to high light the difference for compact web. When the flange becomes non compact the other equation is replaced only for m n and remaining is almost same four doubly symmetric compact web.

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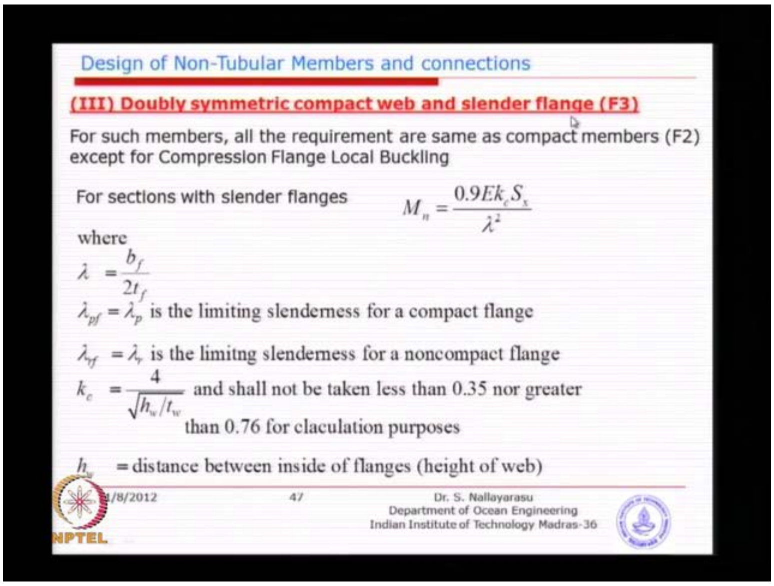

The slender flange, so web is kept compact, but the flange is coming from compact non compact and becomes slender. So, slender you could see the form of equation takes place here it becomes elastic there is no more plastic section models going to come. So, it becomes reduced capacity, you will be in the tail section of you know the fog you will be somewhere here. So, what we did was we did here and then here and then in here the last one is basically the just a typical case where plate gutter are going to be designed most of the time when we design plate gutters you going to make the web very big that is what the thing.

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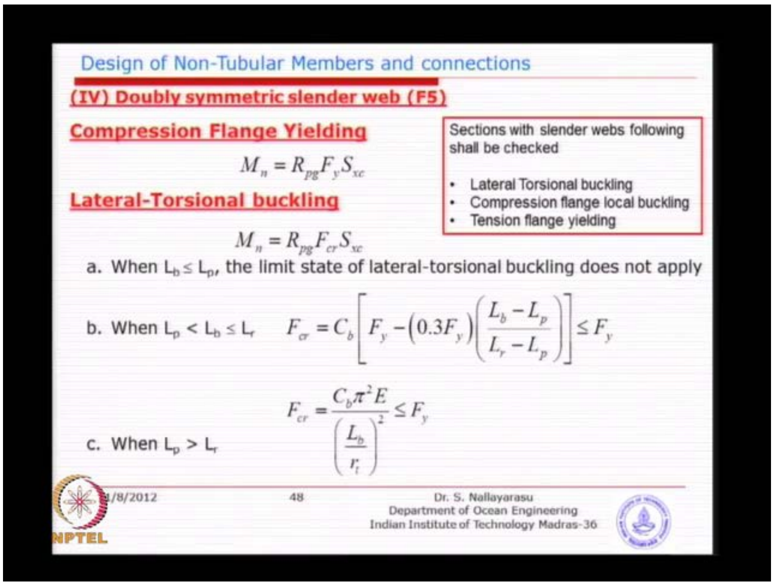

Immediately, we come to our mind will make the depth of the sections, so large to increase amount. So, the web become slender and once the web become slender, it becomes almost capacitor actually is very drastic. So, that is where you will see double symmetric, but web is slender ones the web is become slender. It does not matter whether you have compact flange or non compact flange are slender flange because the capacity is going to be governed by the web buckling itself.

So, that is why it does not matter whether you have the flange in which category, so is automatically governed by the web buckling and you have the check all these three lateral torsional buckling compression flange local buckling. Then, tension plans yielding and the formulas are almost similar you see here in case of yielding for compact web compact flange F y multiplied by plastic modulus we had. So, once the web becomes slender you see here there is a multiplication factor which is basically a notation is applied gutter notation or P G and multiplied by F y.

Then, it is not anymore a plastic section modulus is a elastic section modulus of the section we need to just follow wherever the multiplication factors. You have to calculate if it is yielding is F y if it is buckling it is a f c r just the equations are getting replaced because the web has become slender and again for torsion buckling. You have every classes with less than L p and between L p and L r and then greater than L r the formulas are instead of 0.7.

It became 0.3 because itself has become slender, otherwise if you look at the form of the equation is exactly same. So, the reduction is even further and then here the buckling is similar equation what we seen higher buckling pi square e divided by L by r square multiplied by A C B factor. This is very similar to the previous case where we were computing by bending moment distribution and then the values of L p and L r repeated again.

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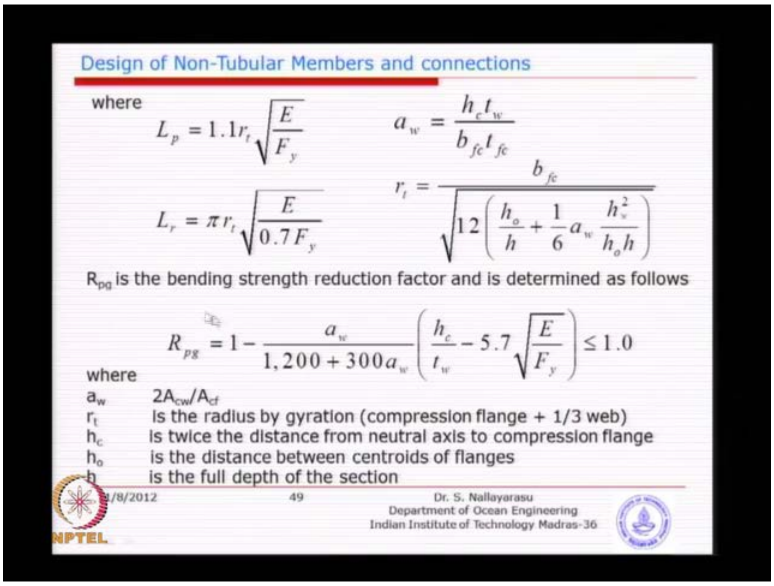

The parameters for reducing the building strength because this web is slender is taking this kind of form. So, what is this parameter basically this parameter is to account for slender web of the gutter which is that is why the p g is basically most of the time. This is applicable for plate gutter in fact if you have seen the AFC code, previously there was a separate section for plate gutter design even in is 800. You will see, but now the new code there is no separate section for plate gutter, because they have merge the section together.

They have made several cases one of the cases a plate gutters design where plate gutter means the web depth is made very large. So, that is the idea behind there is no separate are exclusive section for design applied gutters as such is the only para meters. That will be differing from when it is a short gutter or a long depth plate gutter, so you just have a multiplication factor now if you look at this parameter a w is the ratio of the compression flange. A part of the web and the compression flange two times the compression website divide divided by the compression flange.

That is the a w and h is the full depth and h naught is the centroids of the flanges basically between centre the h is that distance from neutral axis to the compression flange. You multiply by two sister definition earth is radius by gyration, so basically you will substitute this parameters. Just you see because whether the compression side of the web is buckling or not that is why you see here the RPG always will be less. If it more than one something must be wrong, then you have compression flange local buckling. Here, three things is to be check one is the lateral torsional buckling and then the compression flange local buckling because the web itself is trying to bend and you also have tension flange yielding.

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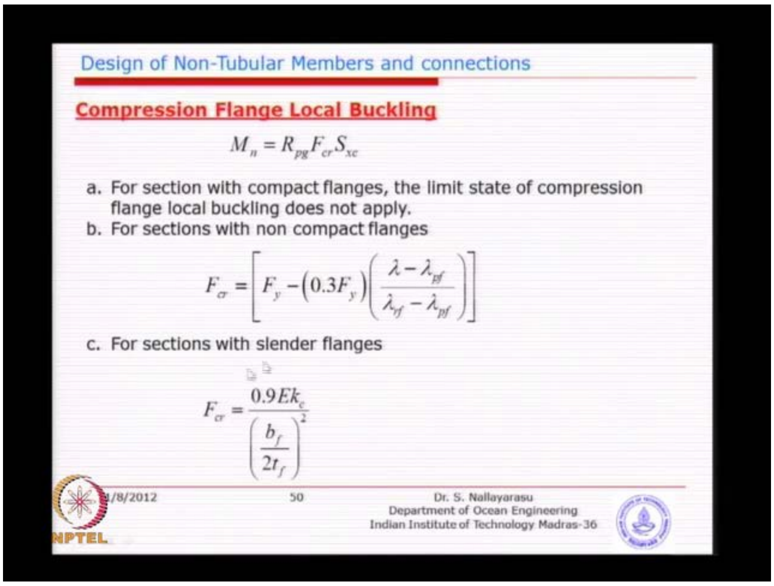

So, similar to the previous case exactly except this will be 0.7 and says to 0.3 and again RPG is the same value and you will calculate the section modulus corresponding to the compression flange only. Then, for three cases you how compact flange non-compact flange and slender flange slender flange is similar to the last formula what we saw and then the yielding case you will see that F y multiplied by elastic modulus.

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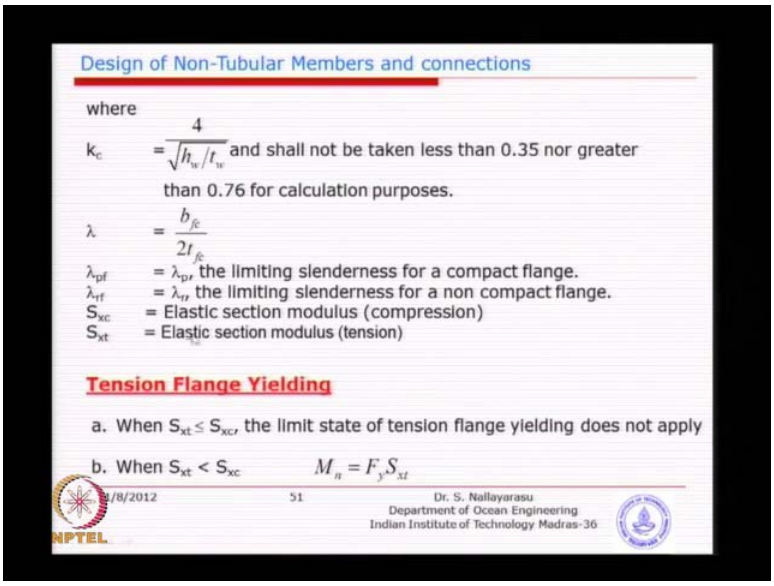

So, the difference between last time we were seeing only a F y S x writing most of you first three cases. So, here you have a two different cases compression side and tension side if you have a symmetric. You know basically the flanges hope you remember you can calculate the neutral axis from bottom which will be your compression side a tension side depending on the bending. Then, calculate the top side, top side and divided by corresponding neutral axis distances will give you the S x c or S x t if it is a symmetric section both will become equal.

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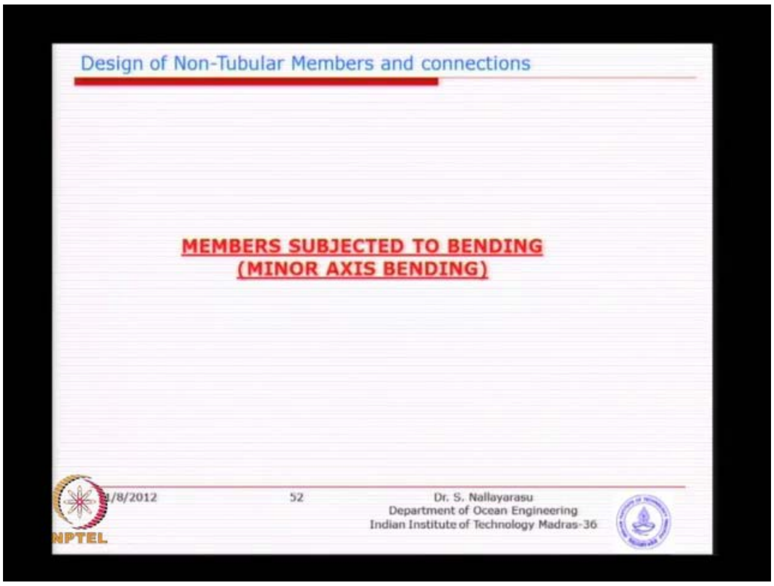

So, the minor axis bending, so what we saw was all the time is bending is along the measure dimension. So, also can bending the horizontal direction if you have the load perpendicular to the web of the section you know basically from the lateral to the only load that they will get from our offshore or anywhere. For that matter will be the wind loading which is going to be predominantly smaller is not going to be very big load.

The second thing what can also happen not only direct loading on the member, but also loading elsewhere can make the beam to bend horizontally. It is not that the beam is going to bend only purely on the loading on the member itself sometime it may happen, but very small loading. So, that goes hands in hands with our idea because we have made it very weak, if you look at the moment of a calculation of an high section, it will drastically smaller compare to the major axis bending which is good.

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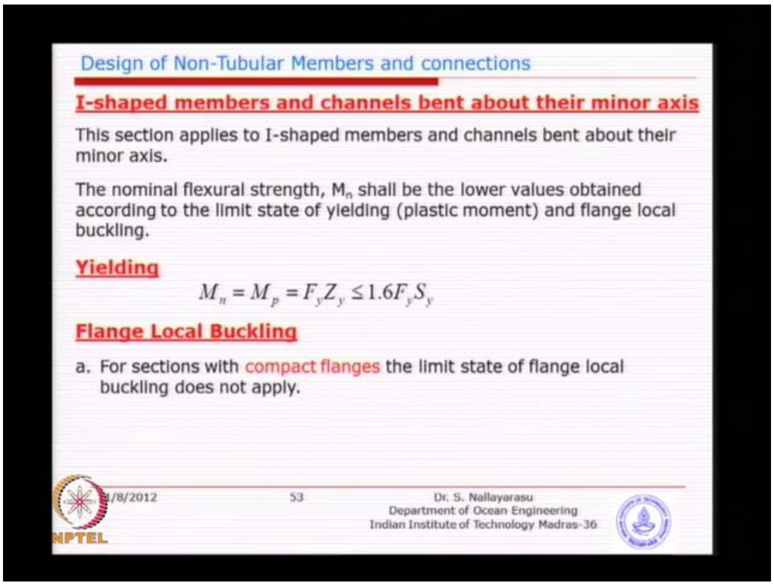

So, the idea behind here is very simple, I shaped the member and channels bent about their minor axis your yielding can govern basically z y is taken that means the buckling is not going to happen here because there is no such phenomena. There is no to flanges, for example if you keep the i beam turned 90 degree, then buckling will happen, whereas here when you keep the section primary section loaded in perpendicular direction. The lateral direction there is no buckling because there is only one web is there is nobody else.

So, unless you have two flanges are two webs if you make a box section, then buckling will come at here there is no such thing that is why you still go back to the very good capacity, but of course z y. If you calculate for such a section will be very small, so yielding is F y time Z y, but limited to 1.6 times the F y time's S y. So, you can see here the ratio between Z y and S y matters a lot as long as you keep the ratio larger than it is good, but unfortunately for i section the ratio is about 20 percent. So, even still see a reduction there so for local buckling compact flanges, you do not need to change anything that means buckling does not apply when you sections of non compact or slender, you apply this formula is very similar.

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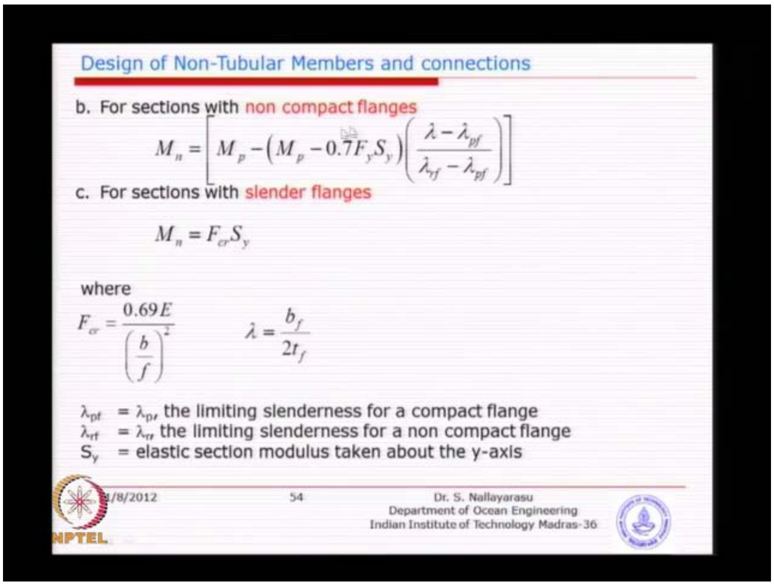

What we did for the major axis bending, so you can see here minor axis bending one great advantages the lateral torsional buckling is not going to happen, because you have only one web. Otherwise, only it is because of this compactness of the flange if it is compact or slender. You apply the reductions basically when it becomes slender you take the buckling capacity based on this formula is very similar formula to our major axis bending. So, in summary what we have done is tension compression and you know basically the major and minor axis bending major axis bending for classes compact web compact flange. Then, we had compact web non-compact flange compact web slender flange and then slender web with compact non compact or slender flanges.

So, all the cases doubly symmetric section we have seen, so you can have single symmetric sections, you can also have doubly symmetric sections or you can also have sections. So, you can see each one what you neat understand is the form of equation compactness affects KL by r affects the allowable compression is stress, whereas the compactness allows the bending strength to be drastically reduced. So, we will see hear tomorrow and followed by I think few examples.