Design of Offshore Structures
Prof. Dr. S Nallayarasu
Department of Ocean Engineering
Indian Institute of Technology, Madras

Module - 04
Lecture - 10
Non-Tubular Members and Connection

So, we have last time covered the tubular members in full, I think we were looking at actual shear bending and combined action. I think we spend sufficient time to understand how the design works, so what we are going to now look at is the design of non tubular or open sections which I think is as important as tubular members becomes the sub structure. We normally use tubular members as I mentioned several advantages when you are trying to use tubular section for jacket. When you come to topsides v g exactly opposite, you know tubular members do offer several good properties when it comes to submerged hydrodynamic loading buoyancy and so on.

For topsides structure above water predominantly subjected to bending loads, you know that the floor is composed of floor beams and floor slab like concrete structure. Now, in this case, you have floor slab is replaced by the steel plate, you know most of the offer platforms pulse, you will have floor beams. Now, if you look at the loading on the topsides, predominantly gravity loading which is going to be producing simple beam bending and to some extent, you might have some been loading. You know horizontal loads, but that will be considerably low compared to what you normally have as a bending loads in substructures.

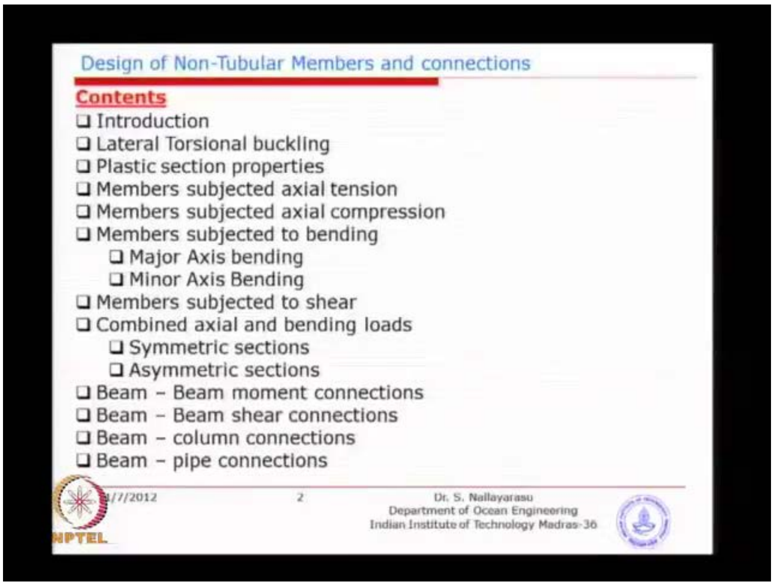
You know if you look at the percentage of wave loading versus the vertical loading, the wave loading is substantially larger. So, you could see exactly opposite, so we need to change our mindset how we optimize the structure. So, that is why this is called a non tubular sections, of course it is not the first time we are using in offshore. You know if you look at onshore building structures, you use this kind of open sections heavily in industrial buildings, in residential buildings, we normally use concrete structures and not very much useful because we cannot replicate.

The industrial structure mostly use open sections, so that is what we are going to see the use of non tubular members, why we changed, why not we use tubular, we would justify, of course there are several advantages. Then, you move onto some design aspects since you already have a clear idea of tubular member design, I think going through this is very easy because it is a same thing only section is replaced and section characteristics going to be changing. The behavior and allowable stresses going to change, otherwise the procedure what we were looking at the unity check allowable bending stress allowable actual stress allowable shear stress combining them unity check ratio.

All those procedures are almost same except section size and there allowable characteristics are going to slightly change. Then, we look at also the connections since we already had a very detailed discussion on tubular connections and their static behavior and their cycle behavior dynamic loading. We also have to have quick look at, but it is not going to be so complicated here because the connections are quite simple and the design also quite simple.

The way they get connected is easy, you know basically look at the mechanics of the load transfer, whereas comparing the tubular connections, we spend a lot of time trying to understand. Then, we come up with a design procedure which is so complicated that you could use only empirical procedure, whereas here I think mostly you will use a basic mechanics of load transfer by shear bending and things like that. So, in that sense is quite simple in design of connection design of members also quite simple because you already have a clear idea.

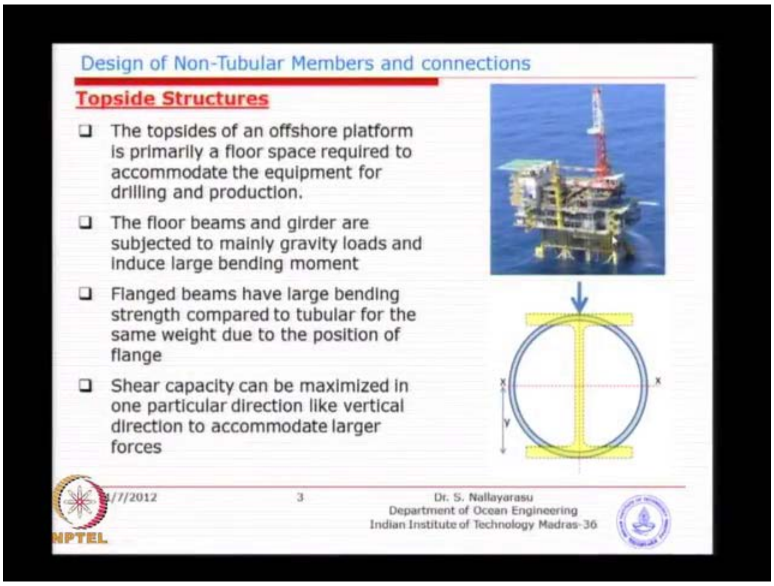
(Refer Slide Time: 03:48)



So, we will just spend a next three classes on this particular topic to various areas of discussion is going to be something like this. We will just introduce this sections and the special properties which make them better use or which makes them actually more troublesome. So, we will look at that torsional buckling and then the sectional properties, major axis bending, minor axis bending and then will also look at the combined actions of you know basically the actual and bending. Then, we look at how they behave in terms of symmetric sections versus asymmetric sections, we have you know both forms, whereas in tubular sections you do not have much.

You do not use normally ellipse, you use only circular sections so that here we have got various forms of manipulation possible. So, symmetric and asymmetric sections we will that and then will also look at the connections as I mentioned beam connection, beam column connections. Basically, beam and pipe connections some several times, we use it you know if you look at it structure in half floor construction, we will have columns as pipes. Then, beams are open sections, so you will see that there is a combination of them coming in so that they will give some idea so that you can understand.

(Refer Slide Time: 05:01)



So, you can see on the right hand side, see a photograph which is a large topside comprise of several horizontal levels. So, you can see here if you do not get the clear picture the first level what you see one line is actually a one floor, like a building floor and you got 1, 2, 3, I think three major floors. Then, on the sides, you have several secondary floors going above, so you see this is the second floor third floor and then you got several floors on the living side as well as on the other side. So, you can see is just a horizontal floor made up steel pancake connected by columns and ultimately supporting facilities which you will produce oil and gas supported on the substructure.

This is at 4 or 8 point in this particular case, we got eight columns which is jacket legs, so it is very similar to any building construction. You got columns, you got beams, and only difference is its going to be made up of steel instead of concrete. So, the topsides structures basically an offshore platform is to support equipment and facilities very similar to any building or industrial structures. This means predominantly gravity type of loading, so means you need to support the floor the floor beams and the floor plate or floor slab.

In this case, the floor beams are normally made of open sections because we know very well predominant loading is pure simple bending. That means we need to have sufficient in plane bending section or the property called movement of inertia or section modulus. So, that is what we are planning to have it, so if you look at the right side, you see here as you take a tubular section and compare the moment of inertia about the bending axis x, you can optimize. You can see here the movement of inertia is going to be highly proportional to the flange area and the distance away from the neutral axis.

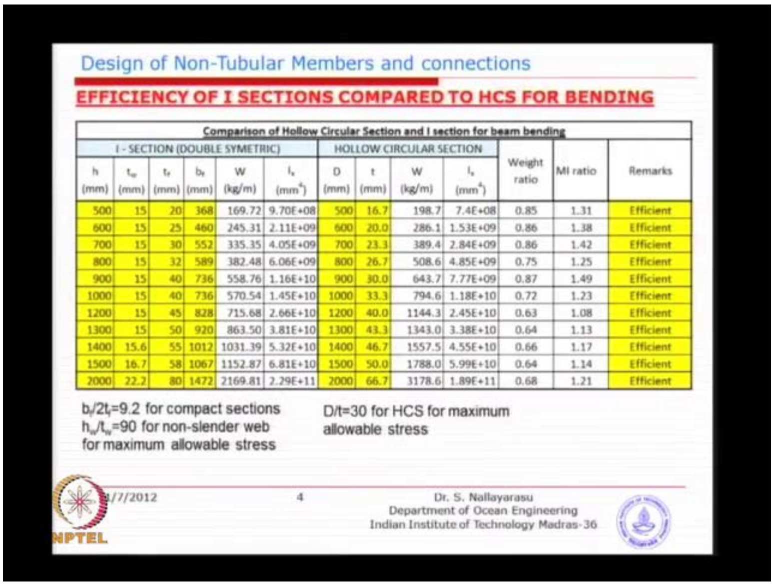
If you do a simple movement of inertia calculation, the more that you go away from the neutral axis, you are in plane moment of inertia is going to be higher because this area times square of the distance. So, that is where we try to play around, if you look at this tubular lot of area is wasted at the middle because is not serving any purpose for us by keeping the flanges away. So, for the same circular section if you just rearrange the three wall thickness, you can get more moment of inertia of course be vice versa to get the same moment of inertia as the pipe can reduce the weight considerably.

So, you just need to do quick calculations to see whether that is true or not, so that is one of the advantages. In the other advantage in case of vertical loading for example, gravity loading you seem to have large shear because you have too much of weight, So, you want to manipulate the shear in vertical direction, if you look at the pipe the shear in any direction is same sheer capacity. If you look at the beam the sheer capacity in vertical direction is predominantly large because we arranged web which is going to take large amount of vertical shear.

So, you try to achieve what we wanted, so that is why you from a circular section, you manipulate the section into getting maximum amount of inertia, maximum shear capacity. So, that is why because the loading is like that and its gravity loading is very well known because you know what facilities are going to put uncertainties very small is not that. Tomorrow, the weight of the equipment is going to act horizontally, whereas the wave loading you know the uncertainty associated with it the wave direction, the wave heights and the kind of wave load it will produce.

You are not very sure that is why uniform cross section is essentially in the substructure rather than open sections. So, that is why we can see a great advantage of manipulating the circular section into a open section which can offer higher resistance. We can reduce the weight for a same work to be done, so that is the idea behind why we use substantially all the floor beams, but of course when it comes to columns, we still see a big problem with the open sections, you see the column one direction the bending is very good, the other direction is very weak. So, that is why we take columns most of the columns in offshore platforms, we make use of pipe sections and all the floor beams we make use of this kind of open sections.

(Refer Slide Time: 09:35)


Design of Non-Tubular Members and connections

EFFICIENCY OF I SECTIONS COMPARED TO HCS FOR BENDING

Comparison of Hollow Circular Section and I section for beam bending

I - SECTION (DOUBLE SYMETRIC)						HOLLOW CIRCULAR SECTION				Weight ratio	MI ratio	Remarks
h (mm)	t_w (mm)	t_f (mm)	b_f (mm)	W (kg/m)	I_x (mm^4)	D (mm)	t (mm)	W (kg/m)	I_x (mm^4)			
500	15	20	368	169.72	9.70E+08	500	16.7	198.7	7.4E+08	0.85	1.31	Efficient
600	15	25	460	245.31	2.11E+09	600	20.0	286.1	1.53E+09	0.86	1.38	Efficient
700	15	30	552	335.35	4.05E+09	700	23.3	389.4	2.84E+09	0.86	1.42	Efficient
800	15	32	589	382.48	6.06E+09	800	26.7	508.6	4.85E+09	0.75	1.25	Efficient
900	15	40	736	558.76	1.16E+10	900	30.0	643.7	7.77E+09	0.87	1.49	Efficient
1000	15	40	736	570.54	1.45E+10	1000	33.3	794.6	1.18E+10	0.72	1.23	Efficient
1200	15	45	828	715.68	2.66E+10	1200	40.0	1144.3	2.45E+10	0.63	1.08	Efficient
1300	15	50	920	863.50	3.81E+10	1300	43.3	1343.0	3.38E+10	0.64	1.13	Efficient
1400	15.6	55	1012	1031.39	5.32E+10	1400	46.7	1557.5	4.55E+10	0.66	1.17	Efficient
1500	16.7	58	1067	1152.87	6.81E+10	1500	50.0	1788.0	5.99E+10	0.64	1.14	Efficient
2000	22.2	80	1472	2169.81	2.29E+11	2000	66.7	3178.6	1.89E+11	0.68	1.21	Efficient

$b_f/2t_f$=9.2 for compact sections
h_w/t_w=90 for non-slender web
for maximum allowable stress

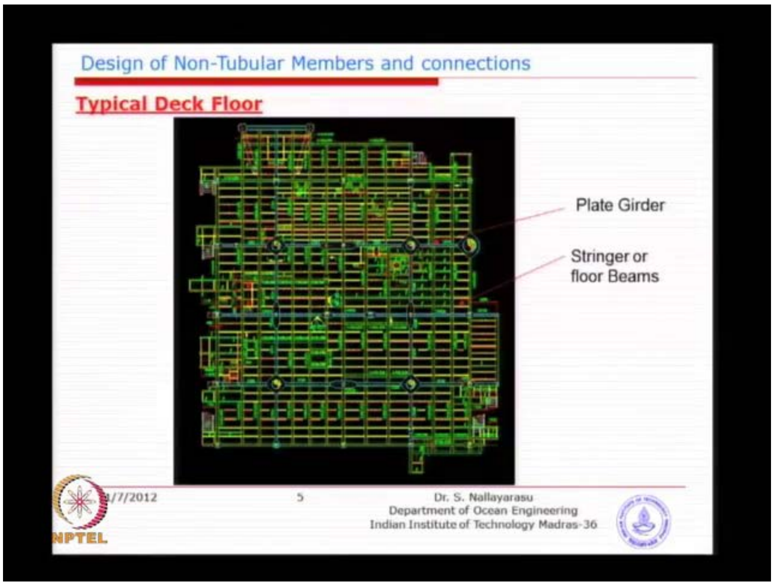
D/t=30 for HCS for maximum
allowable stress

I just made a simple comparison just to show you that most of the sections if you manipulate, you will see that the high sections are efficient than the pipe sections for a given you know depth of the beams. For example, I just utilize the depth of the beam and the depth of the tubular section are the diameter of the tubular section, I made it same and you can see comparison of weight and weight of high sections as well as moment of inertia and moment of inertia. Both of them is in advantages position weight, it is lower moment of inertia is higher of course, what I did to make a comparison properly, I made the d by t ratio of the pipe section to be equal to 30.

I think you might remember when you look at the bending stress variation, I think you can recollect your memory the bending stress is maximum 0.66 f y up to a d by t ratio of thirty after that only starts to reduce same thing I have done here for the open sections. We are going to learn that as long as you keep the sections compact the bending stress the allowable bending stress will be maximum. So, I am just going to compare both in a similar platform because if you do not do that then the comparison becomes no use because you may actually manipulate the section can be made weaker, but then the allowable stress can be substantially smaller.

So, that is why both the cases I am keeping the allowable stress same that means 0.66 f y and try to just see the advantage of the depth and manipulated weight. Manipulated moment of inertia is always going to be higher as you can see from the previous picture what you see here you are just rearranging the same material in a right place for right job that is the idea behind. So, that is why you will see that pipe sections we should not call here as a deficient section for this purpose, it is not very useful, so that is why we are changing to high sections.

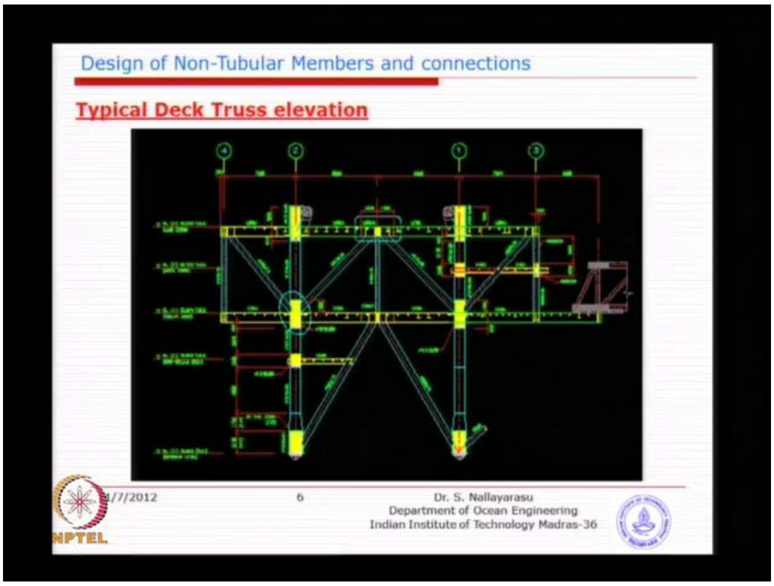
(Refer Slide Time: 11:35)



Typically, if you see this drawing this is how offshore platform floor will look like you know you this is particularly four columns structure, four column is basically one column here, another column, another column, another column. Then, I do not know whether see the colors, the blue colure is the grid beams where it should be deeper and you see other yellow color beams which are floor beams or a runner beams. You know basically supporting the equipment floor plate and facilities.

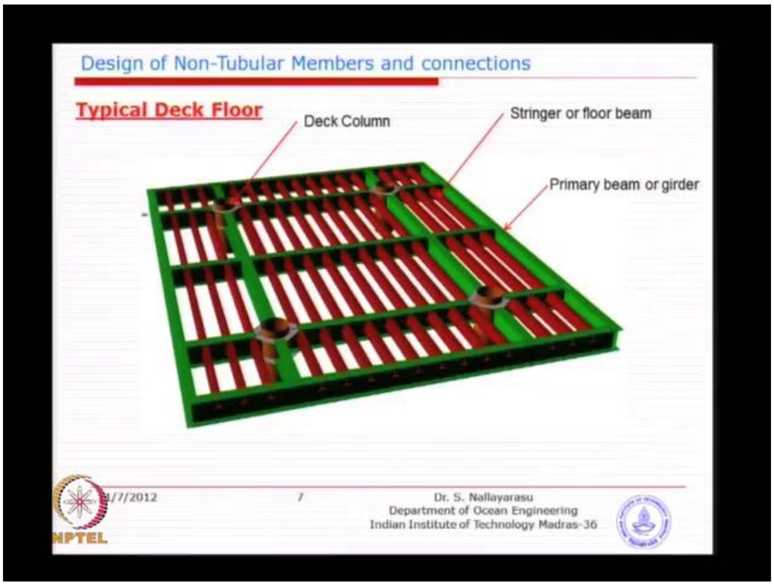
So, you can see here these this, this circular columns carry all the load from elsewhere all around carry all the way down to the sub structure which is supported on the ground by pile foundation. So, basically this columns are very important to carry all the loads, collect all loads each floor from floor one, floor two, floor three and then make them to go down to the jacket structure. Typically, elevation of a particular grid deck you know you got grid one, grid two, grid three.

(Refer Slide Time: 12:41)



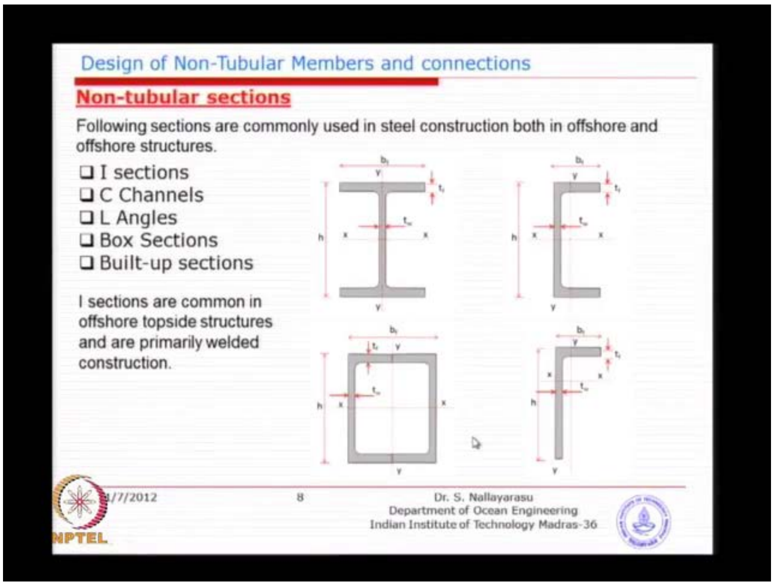
So, I just took one grid, so you how it looks like this, so you can see here this is one of the floor which you see in that horizontal parallel lines and another floor here and connected by two columns and some inclined braces for stability purpose. So, it does not sway, we will look at the sway non sway frames, I think you might have studied in your applied mechanics the different in a sway frame and non sway frame, you will see that buckling will be a problem. So, basic idea is this is how a typical super structure of a offshore platform looks like all those small high shades drawn here shows that the floor beams are running in that the particular direction and that is the idea.

(Refer Slide Time: 13:28)



So, if you would like to know little bit more how they are constructed, you can see here the four columns are here 2, 3 and 4 and the green color is the main girders and the red color is the floor beams. I purposely did not put the floor plate because otherwise you cannot see, so basic idea is this is how you construct typical floor and you can duplicate it and then just stack up one by one. So, typical deck floor will consist of columns major floor beams and secondary floor beam or runner beams and deck filling plate support members, so we are going to see the design of these floor beams how they are going to help us.

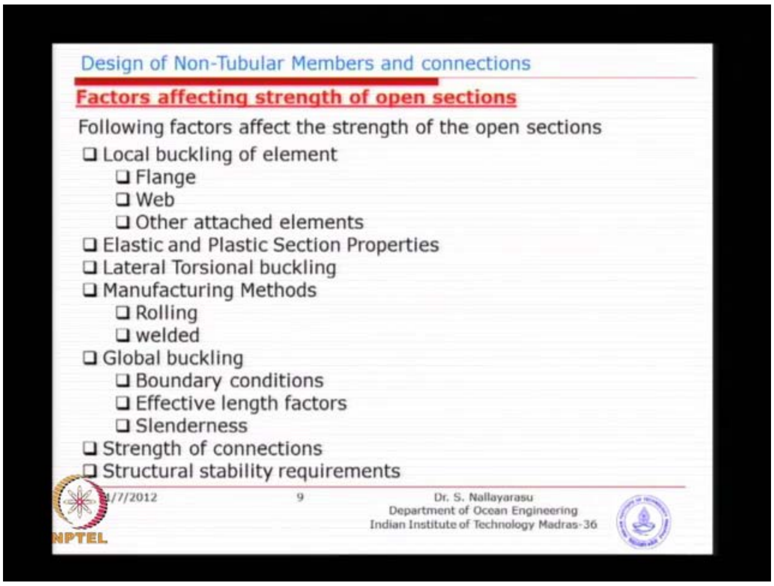
(Refer Slide Time: 14:08)



So, some cases we will use I sections some cases we will use channel depending on the availability and the load category and angle bars or maybe box sections. You see the box sections is almost similar to circular section or a kind of if you if you find out equally. So, wherever there is a out of plane bending is predominant you go for box sections what you are trying to do is just manipulate the circular section into you know the top and bottom flange and two side webs many times.

You do not prefer this because fabrication of this is very difficult, but high sections is readily available you can go and buy from the milk you do not need to fabricate. So, we will see the design of these one in the next two three classes, but of course the procedure is similar only thing is because of their symmetry in vertical direction. The calculation becomes slightly exhaustive, so the idea is very similar, similarly for angle behavior is going to be slightly different than this kind of symmetric sections.

(Refer Slide Time: 15:19)

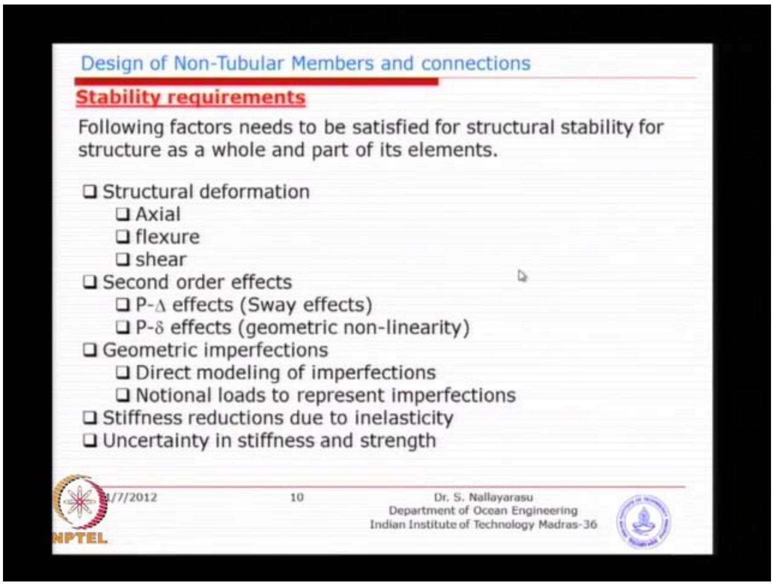


So, what we will be looking at is basically a different characteristic compare to circular section that is what we are going to see what I mentioned in during the tubular design. The tubular section does not have a property called a torsional buckling because it is close. So, it is not going to get torsion there, so that is the predominantly one difference, the open section behavior is going to be different in the shear floor, so that we are going to see and else remaining is almost same. So, the local buckling contains because is open section, you got flange you got web, whereas in the circular section is a continuous section connected to together.

So, such separation is not there and in case of open sections, we are also attached some stiffness. So, stiffness also may contribute to local strength and we need to see whether they stiffener is sufficient enough to support for the purpose for which you have actually placed a stiffener. It should so become that is a stiffener is so weak instead of stiffener supporting remember supports the stiffener. So, you need to make sure that the stiffener is having sufficient thickness sufficient rigidity and then we look at the elastic plastic properties which I think for tubular sections.

We derived, remember we spend lot of time in going back to the basics, so we will just spend some time and then will look at the l t b which is the most important characteristics among the difference between open section and the a circular section. Then, we also look at the manufacturing methods not in detail was just to see how we can actually get the beams manufactured global buckling and a strength consideration. The last one is the stability method or how you actually going to establish the relationship between the stability and the design method.

(Refer Slide Time: 17:10)



So, this stability requirement is what exactly means you might recollect portal frame you know simple portal frame and apply a horizontal load, apply a vertical load what happens is trying to sway with a large differential placement in horizontal direction. If you go and do a brazing pattern which may prevent the sway action, so that is what we are going to see, that is why many times when you design a whether it is onshore or offshore structure or building structure. You know you try to brace so that the unnecessary sway action comes into picture additional secondary movements will develop which is not going to be of any use.

So, as long you can try to brace it the sway action can be reduced, so basic idea is the second order effects, there are two things to be considered, one is the p delta effect which is basically a sway and the delta effect. The beam bending are local deformation of the beam which produces additional moment because when you do a linear elastic analysis you ignore this. In reality, if it is going to happen how you can incorporate because in olden days like 1960s and 70s, people used to do a simple calculations for bending moment column. Forces try to design as the finite element concepts came into picture, everyone does the f e analysis of a frame, but only linear analysis.

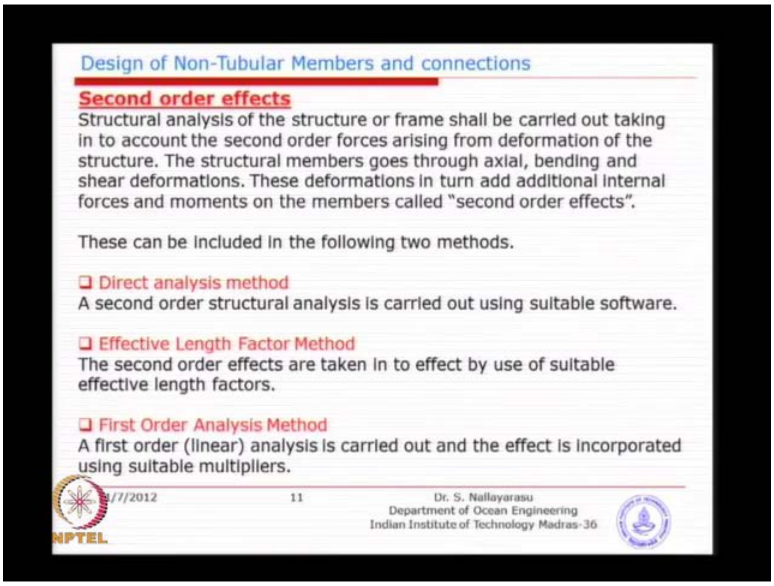
When you are doing linear analysis, you do not incorporate these two effects, so we have always been thinking about incorporating in a indirect manner like the effective length method, we try to incorporate the column buckling by means of buckling coefficient. This is comparing with the oiler buckling of any column with the different boundary conditions. That is what we normally do it for a cantilever for a fixed beam we have a different effective length factor, but how accurate is it? It is not very accurate, so that is why the new design.

I hope I have explained what AAC is, you know the American steel construction which is acceptable code for all the offshore structures for topsides. So, this I am going to discuss, of course you could use IS 800, I think some of you might have studied in your college days, but substantially the offshore industry is using AAC codes. Basically, for certification purposes, many of the certification agency is do not recognized regional codes, so they will always asks for an American codes. So, that is one of the reason why though you can use IS 800 which is the definitely a similar code and you already have some knowledge during your college days. So, I am just going to introduce this AAC codes, this will be very useful for practice.

Then, we look at the geometric imperfection very similar to tubular sections. I think we were looking at out of ship which is 1 percent, 2 percent of diameter or out of verticality you know 5 mm, 6 mm we were talking about. So, how do we incorporate that in the design and basically the codes take care of that stiffness reduction due to in elasticity whether we want to go to elastic stage or inelastic.

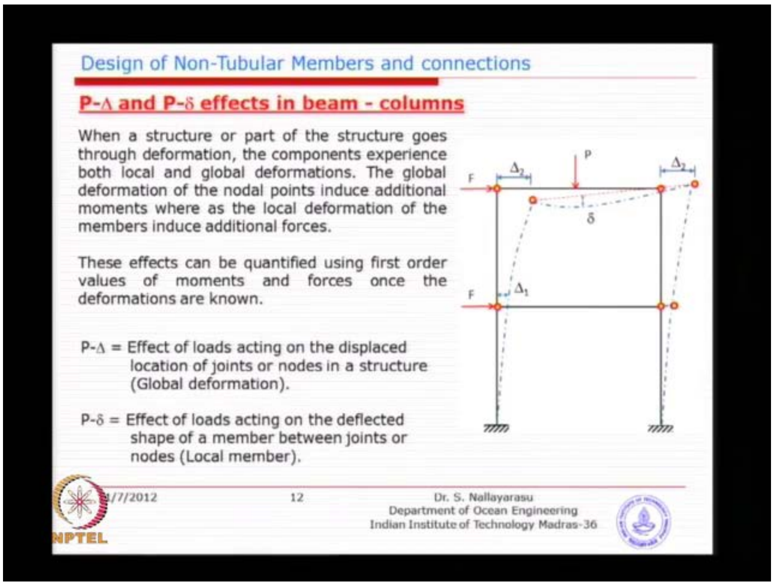
Then, the uncertainty in the stiffness and modeling for strength, so these are some of the characteristics AAC code requires you to address when you are doing design earlier ninth edition code. They do not talk about so many parameters you talk about elastic deformation and design it. So, the new code requires you to address this in some way or other.

(Refer Slide Time: 20:45)



So, the basic methods of addressing is very simple the direct analysis method we choose a method which you can incorporate the second order effects. That means you will do a analysis in step by procedure updated Lagrangean method, I think some of you might do a course on FE analysis next time.

(Refer Slide Time: 21:05)



I can just quickly look at this picture to explain what second order effects are if I have a column and carry only actual load. There is no moment and if it is, there is a column on the carrier's horizontal load the moment is only due to the horizontal load, but if it carries both, what happens is there is a coupling effect. The horizontal load produces a horizontal displacement of delta or x whatever, but because of the delta and the actual load acting on the column is shifted horizontally because of the column moment at the top produces additional moment.

This is basically the vertical load multiplied by the delta so this is called the second-order effects which is normally not covered in any of the linear elastic analysis software. By theory, you might have studied in your basic mechanics applied mechanics, we normally ignore such kind of coupling effects we superimpose the response of the structures without coupling effect which is what is called a the linear superposition. That is why it is called a linear analysis when you do not take into account such interaction.

Then, it is slightly approximate, but the idea of that linear method is assuming that this delta is considerably small. That is the assumption, we make the displacement of the structure is so small that we can ignore, but whenever it is not so small according to your thinking or according to the codes justification. When the displacement are larger enough to produce secondary movements which can cause substantial stresses to the columns are beam, then we cannot ignore. So, that is exactly the idea, so this p delta b delta analysis some of the software's can handle not all of them.

That means you will divide the load into several sub steps apply one load for example, you have 100 kilo Newton, horizontal or 100 kilo Newton vertical. You do not apply in one step, you apply in sequence 100 divided by 10 steps apply 10, the column is displaced. You recalculate the stiffness and apply the vertical load also in step, so you can see there is a interaction between. So, you keep on increasing that loads in steps that you are not going to take the original stiffness of the structure.

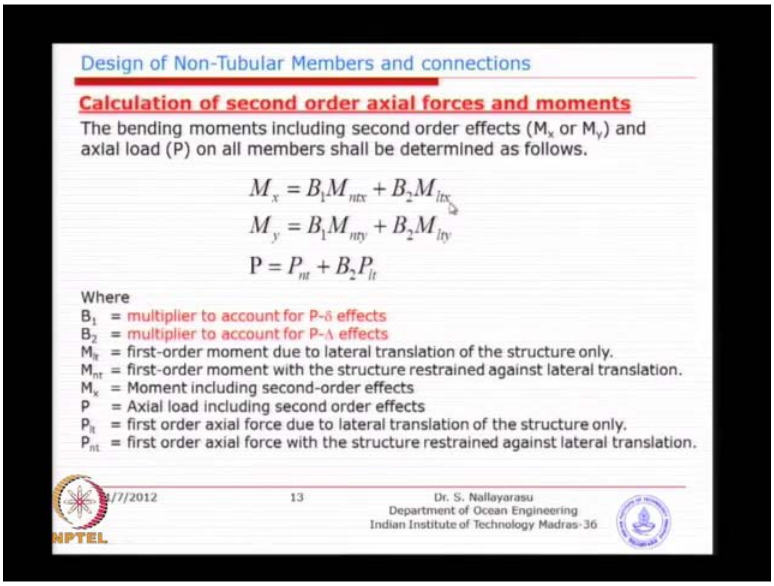
You are going to take this stiffness based on the deflected structure that is why that is called the reduced this stiffness once the column is deflected. So, that also is supposed to be taken by analysis and that is what is not taken normally any analysis what we are doing is normally not taken. Even now, we do not do this because this procedure is substantially time consuming and also not all software tools are capable of doing this. So, we do actually call this is a rigorous second order analysis, but what we normally do, it is we do a simple linear analysis, but somehow in alternate methods of taken into account these effects.

One of the method is the effective length method or effective length factor method, we try to a incorporate second order effects in terms of effective length factors. The third method you could also do a first order analysis, but then instead of applying k factor, look at actually the results of the analysis and calculate the factors to account for the p delta. So, basically that is the third method which is now recognized by these AAC codes and that is what you would not find IS 800. IS 800 is predominantly based on the effective length factor method which is reasonably simple.

I think many of us have been practicing this method nothing wrong, but of course when the structure is having large deformation the effective length method becomes reasonably not correct because the deformation is larger than the effective length. Method is predicting under the capacity is under predicted, so basically that is why we need to use this method. This was not then until the previous edition of the AAC code from they have introduce this as a special case. Then, we can use that, so we will just look at quickly what is that effect, so you can see from this picture there is a local deformation of the beam due to local member forces or applied loading on the member which simply supported beam is going to sag like this.

If you look at the other horizontal loads applied either on the nodal points or on the beam or column will also produce horizontal sway action. So, the total structural response is basically the p delta effect, whereas the p delta effect is basically based on local member effects which are going to cause additional movements and forces.

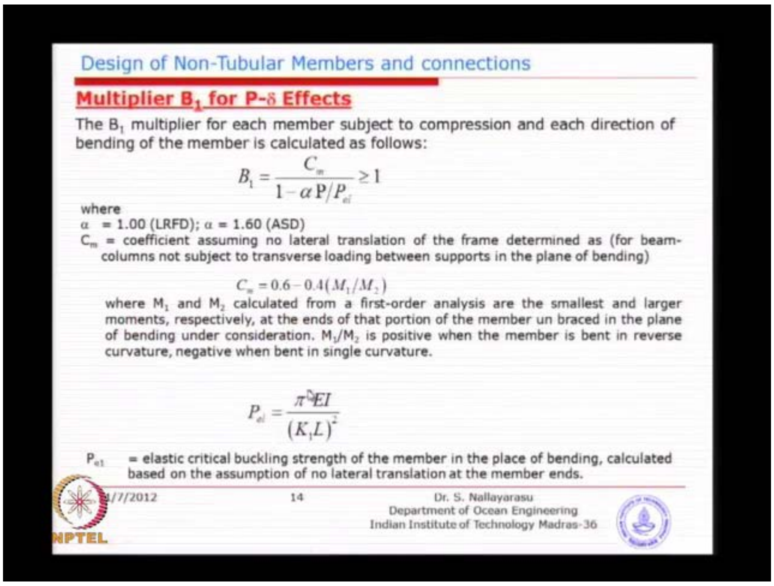
(Refer Slide Time: 26:11)



The method of calculation is given by AAC is basically the M n t x and M l t x is the movements calculator from the first order analysis and the multiplier b 1 and b 2 or taking into account for p delta effect and b delta effect. The procedure is very simple; you do a simple linear analysis compute the displacements of the nodal points as well as the member intermediate points and calculate the beta 1 b 1 and b 2. Then, multiply that with the original forces obtained from the analysis to get the design forces, I think some of you if you have studied RC design to IS 456. We do this exactly, this you know basically when you design a column, we also add additional movements due to minimum eccentricity.

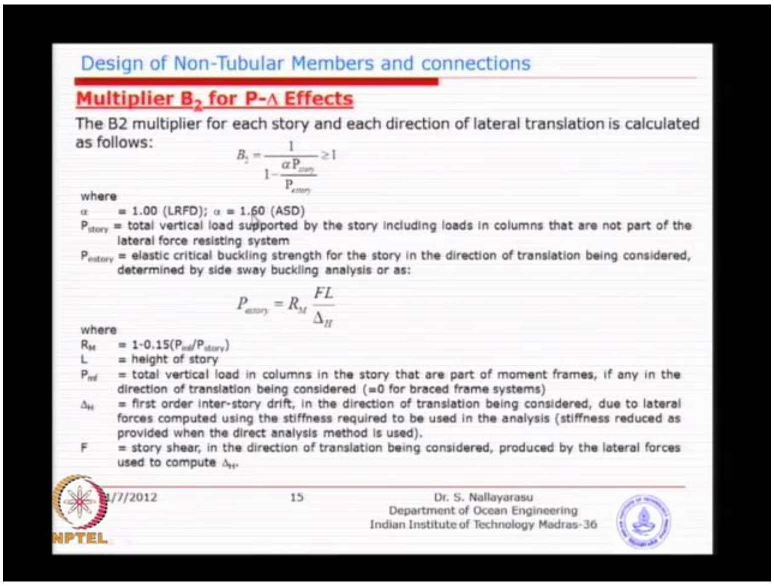
I think you might have learned about it though the column is vertical not producing any moment because is a column subjected to actual load. Still, we designed for certain minimum moment, basically because of several reasons, one of the reason is the second order effects the other reason is the out of plumpness which is to be accounted during construction. Otherwise, with a construction as to the 100 percent with 0 percent deviation, so that is basically the idea. So, these additional movements is going to be taken and this factors beta 1 and beta 2 must be greater than 1, if it is less than 1 is no use because first it as to be a amplified.

(Refer Slide Time: 27:50)



The quick idea of beta 1 and beta 2 is to take into effect the interaction between the actual load and the actual buckling load. So, you can see here p is the load that is coming from the analysis p elastic buckling load, you can calculate using this oilier buckling formula. I think you are familiar with this and C m is the coefficient which we discussed during our tubular member design for end moment factor you know. So, you can calculate for high beams, this is suggesting such a simple formula, if you look at the AAPI table also that formula is available is a interaction between two end movements. So, you can calculate the beta 1 factor applied to the original movements obtained from the first-order analysis.

(Refer Slide Time: 28:38)

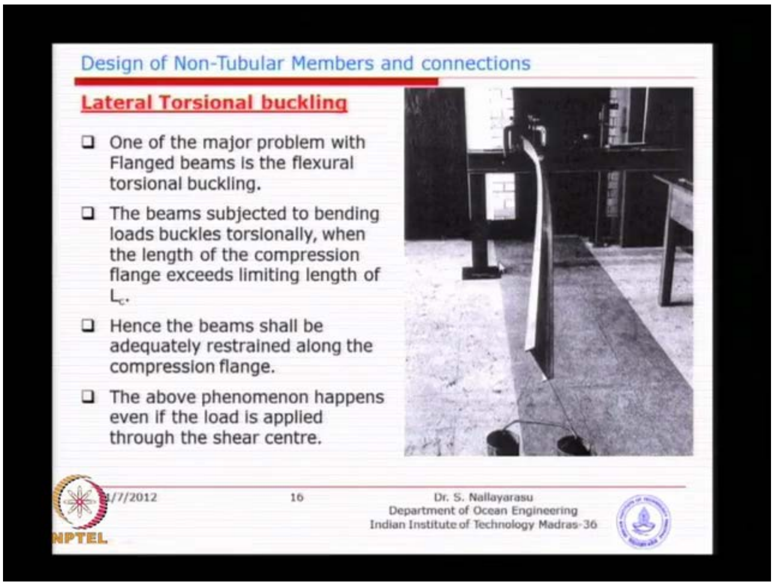


Similarly, very quickly I will show you the formula for proposed for b 2 which is taking into account the b delta effect here is which alpha is taken as 1.6 in this case also. I forgot to mention is same factor of safety which we use for design 1.6 and this p story and p e story is definition is whichever the story you are looking. For example, if I go back to this picture, so these are two story portal frame, each story you will have the loads applied. And correspondingly you can find out the total buckling force divided by the overall buckling force.

So, basically each floor wise you can calculate based on the delta what you have, so this procedure to calculate b 2 is also elaborately given. I have just given the extract of the information here what you need to inherit from here is trying to incorporate the second order effects by means of multiplier coefficients taking into effect the global deformation as well as local deformation.

Then, integrity details of calculations you could always refer to the code because some of the information below I have not given here. The idea behind is horizontal forces and each of the story compared to the total elastic buckling load. So, basically will give you an idea of what kind of coefficients, these coefficients will not be very large and should be somewhere around 1 to 2. It cannot be very large that means the structure is unusable, so 1.1, 1.2, something similar order.

(Refer Slide Time: 30:12)



The next item that we need to discuss quickly is the torsion buckling of high beams, you can see from this picture the load is applied to the centre of the web. Sometime, we call it shear centre, I think you might also have heard attempt called a shear centre for open sections g f center is the intersection of the x and y axis the principal axis. In this case, for high section, it will be centre of the web you know, so you can see even though the load is applied vertically downwards at the centre of the web.

You can see why the beam has rotated you know basically supposed to bend nicely like this and that is exactly the idea of the torsional buckling which try to rotate number one and very similar to torsion. Though we have not applied any torsion load, it is going this way, imagine if this column is made of this beam is made slightly smaller in length. This may not happen, so that means it is related to section, it is related to the length, so that is what we are going to see you make the section larger, very big beam instead of 200 mm, you make it 1 meter.

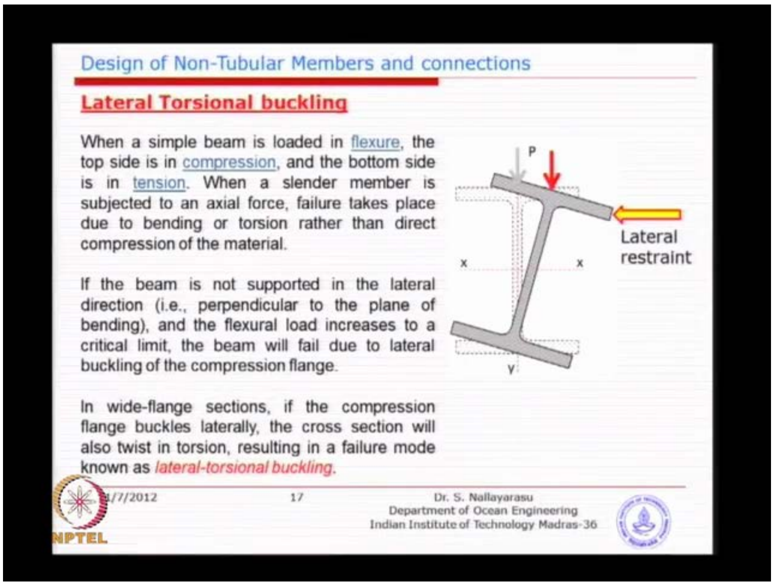
This may not happen because the beam has got sufficient rigidity to avoid rotation, so here it is become two flimsy, but that is only have superficial explanation. Why this happens, we need to really investigate, imagine you take one sort beam apply the loading cantilever. Let us talk about a cantilever, you apply the loading like this, what happens to a top lines is going to get the compression, tension stresses the bottom flange is going to get compressive stress.

When you go near the support, the compressive stresses are going to be larger, basically because of the larger moment that is attracted there. So, the steel has got a characteristic intention it can elongate as much as possible and it can break when you do a tensile testing it can break. So, large deformation is feasible, whereas in compression steel is a dense material the crystalline structure does not allow the compaction of steel beyond certain strain. So, that is exactly the problem, so when you have a larger compressive stress at the back end of the beam at these points what happens unable to compress, actually it is trying to really by twisting and that is the problem with this.

So, basically whenever the large bending strains exist, this will happen, so large bending strain is associated with smaller section longer l, you got the idea now. So, similarly, the same thing will happen in a supported beam at both ends, the compression is at the top, flange tension is that the bottom flange. The beam will try to rotate, so how we can prevent this, if you get, just go there and hold it.

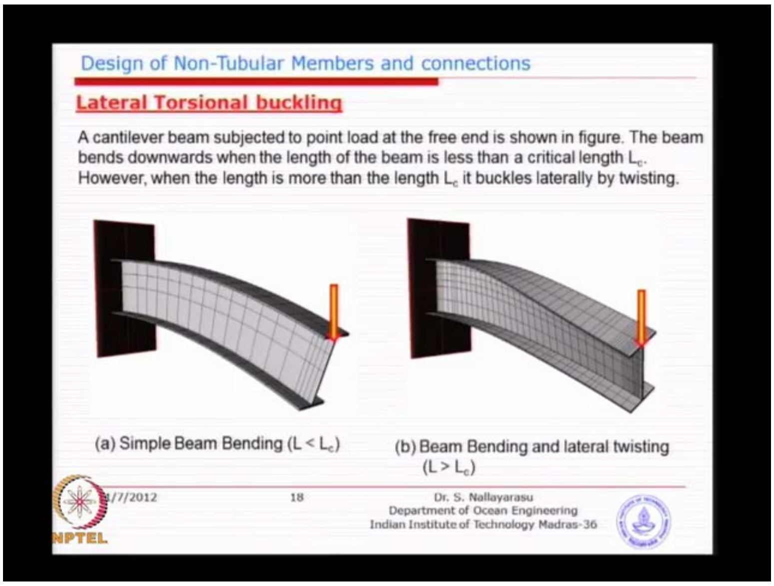
So, if you hold the compression flange from rotation, you can prevent it, so that means it is not a dead end for as you can have a smaller beam, you can have a longer length as long as you prevent the compression flange from rotation. So, you just hold it from the side that means if you bold as perpendicular beam the beam will not do this.

(Refer Slide Time: 33:32)



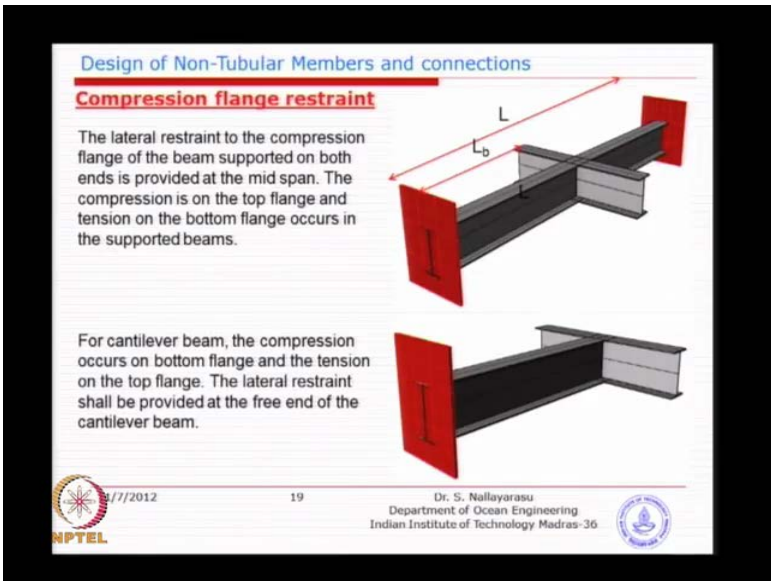
So, that is the idea behind how do we prevent this, we will see some few pictures, so that is what we are trying to do from the side, if you are able to give a no restraint to prevent the rotation.

(Refer Slide Time: 33:44)



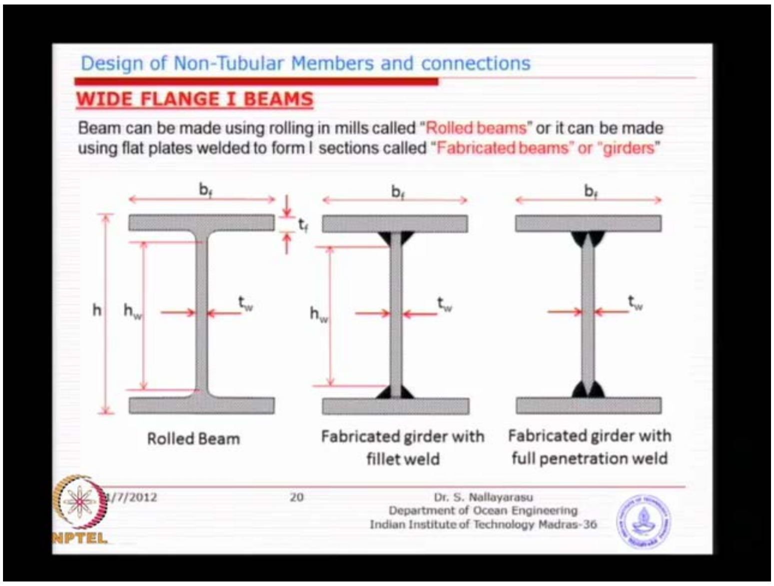
We can do this, so you can see here is same thing is given in another three dimensional picture. So, you can see here on the left side when the length of the beam is less than the critical length beyond which is going to rotate which is basically the torsion buckling. The beam is bending in a high simpler way, whereas on the right side the length is same maybe the section is smaller or the section is same, the length is bigger. So, either way it can happen, so you can see here the beam is trying to rotate along the axis of the member.

(Refer Slide Time: 34:19)



So, how we can prevent by providing compression flange restraint, so you can see here in this case I just provided a same depth beam, but not necessary what you need is actually the restraint to be provided to the compression flange. So, that does not rotate, so though the member is supported far away like this. In fact in this case the member is not going to torsion buckling because I have got the compression flange restraint from rotation by having sufficient stiffness perpendicular to the member. Similarly, here though it is a cantilever no support is there, this is probably a tiny member. Just for picture, I have put the same member, but you can have a small member which is preventing the beam from rotation, so you can get the better capacity.

(Refer Slide Time: 34:09)

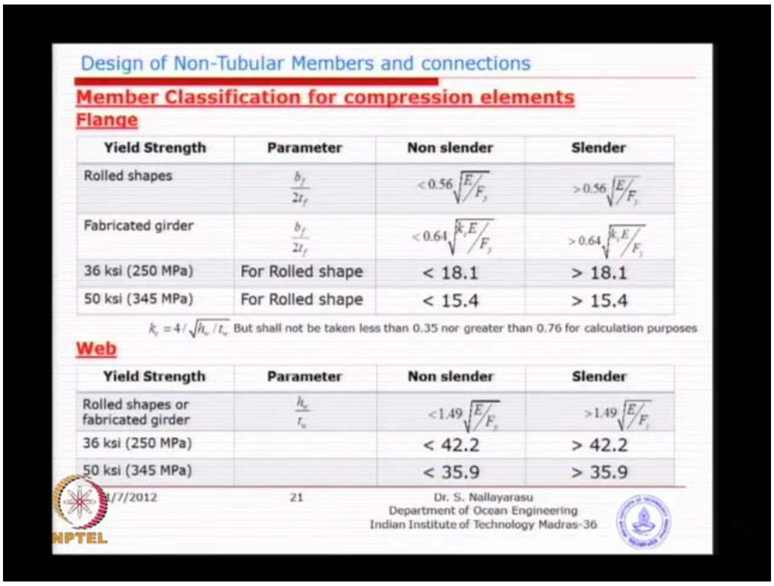


The beams can be fabricated or obtained from mills, so you can see on the left one is basically a rolled beams means there is no welding involved. That means you will take a large size piece of steel and roll it to get this shape is which is called a rolled been, but one of the problem is larger sizes are not available readily because nobody will make this kind of beams and keep it ready.

So, in that cases we can take plates and then fabricate, I think I told you about it in the in the previous sections fillet weld and penetration weld, I think you remember. So, you can see in the middle one is basically a fillet weld it the right side one is the penetration weld it which will give you a better connection between the flange. So, you can see the notations which I am following are slightly different from the codes.

So, you have to be little bit careful when you are referring to the codes, so width of the flange height of the web is given in a notation. This way, the total height is noted as h, thickness of the flange and thickness of the web, so I am going to concentrate focus only on the this kind of high sections in the next two classes. So, the rolled beam is nothing but there is no welding involved, but I you can see here slightly because during the rolling process. You do not want to get or you may not be able to get the 90 degree angle there, so you just allow it to have such a radius fillet radius, whereas in case of welded beams like fillet welded beams. You will get this kind of shape and the penetration weld is not going to exactly like this, but you will get some kind of profile.

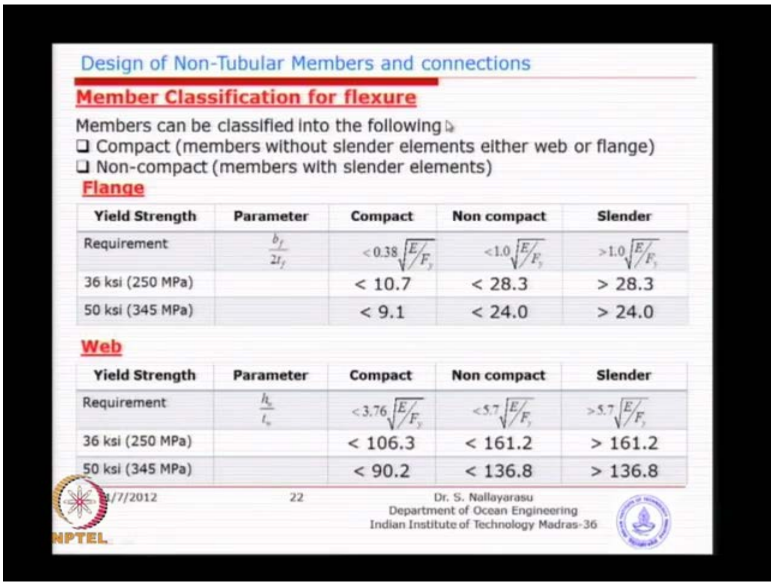
(Refer Slide Time: 36:57)



Now, what we need to look at is one special characteristic for this flanged beams is the flange and web bar free edges, whereas if you take a circular section there is no free edge is completely connected. It is not going to become wobbly, so in this case it got a flange you got a web of course web is connected to two flanges, but then the flange itself is not supported any where unless you go and support it on the sides. So, that is why we need to see whether the behavior of this high beam is going to be achieved maximum allowable stress. For example, you take this flanged instead of making the plans 200 mm, you make it 2 meter.

So, what will happen, you know the flange will sag in this plane which is not good, so there is a limit by which for a given thickness of a 10 mm, how much would be the width of the flange. So, that is why we call it a compactness of the flanges are compactness of plated structures a gains local buckling. That is why we need to follow certain procedure; in this case since we are following American code, the American codes are giving certain limitations on the width to thickness ratio of the flat plate structures. So, flange web you can see formula in empirical as long as you keep, this is basically compression element.

(Refer Slide Time: 38:29)



Let us look at the flexure element, basically a beam will see compact, non compact and slender, so long as you keep the sections within these limits. For example, for compact sections you need to keep the thickness and the width to thickness ratio less than approximately 9 to 10 irrespective of 36 cases a material of 50 cases, a material you see here around 10. So, if the flange width is 20 times your thickness you are you are there, but if you make it 20 times, 30 times, it is not very good.

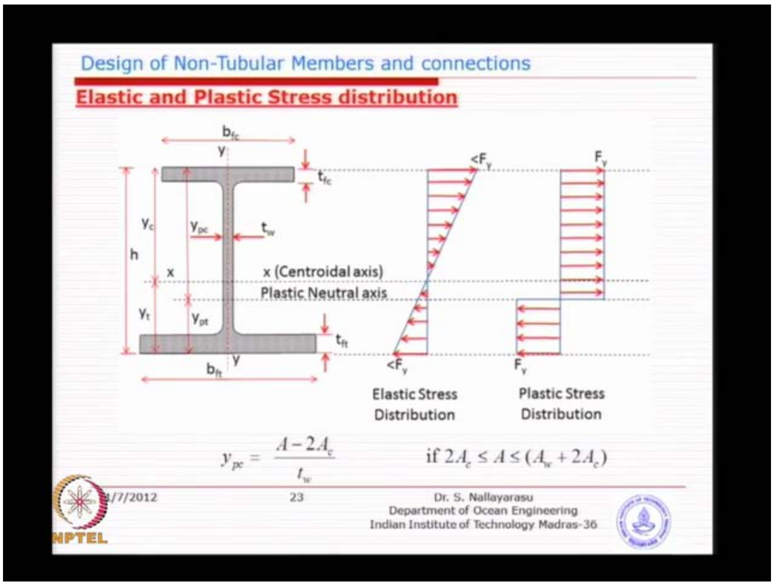
You know because the thickness is too small, the flange will be piety wobbly and bend, so that is the idea behind, so as long as you keep it less than around 10, then you can call it compact sections and go beyond you have a non compact and go beyond. It becomes too slender, so the flange is classified as compact, non compact and the slender.

Similarly, the web can be classified as long as 1 in 90, 1 in 100, so 1 meter deep girder you need at least minimum 12 mm, 15 mm thickness, we keep it less than that. The web is going to be slender and failed by local buckling, so that is the idea you can see here compact, non compact. A slender webs starting from 90 and the beyond 1 meter means minimum at least 12 mm, so when you are designing a girder, you can easily need to do calculations. First of all, look at this because you do not want to make any of the beam or a column to fail by local buckling because local buckling means is very certain that it will fail.

So, you want to avoid that, you keep it below this 90, so that is good; similarly this is basically members in flexure members in bending members in actual load. Here, we have two cases, there is no compact case, either slender case are non slender case, so this is a specifically for columns. So, you can see here you know the thickness to width ratio is around 50 for non slender and beyond it is going to be slender. Similarly, for web is about 35 to 40 and beyond it is going to be slender.

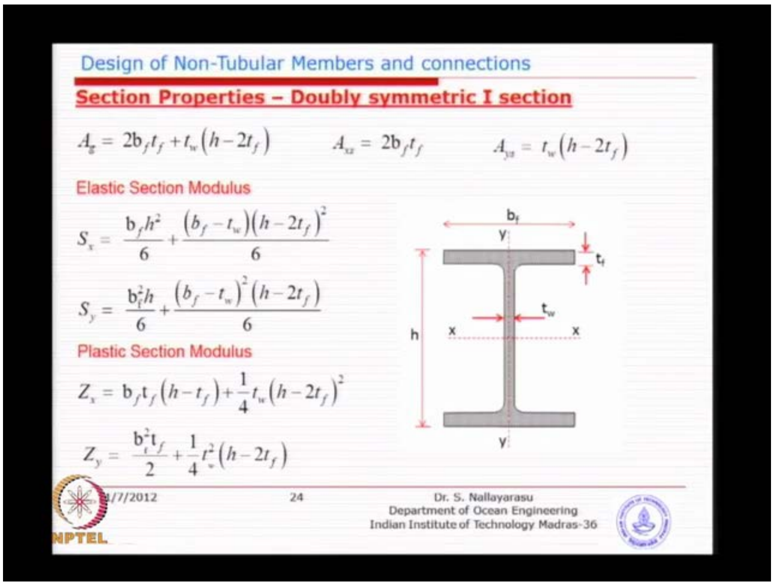
So, when you are proportioning the column or a beam or when you are selecting a column or a beam from a rolled shape you look at the thickness, you look at the size. Fortunately, if you select the thing from American code catalogue, most of them are compact. So, you do not need to worry, the problem will come when you try to make your own column on your own beam by cutting the plates and welding to form and that is a time you will get into trouble if you do not proportionate properly.

(Refer Slide Time: 41:20)



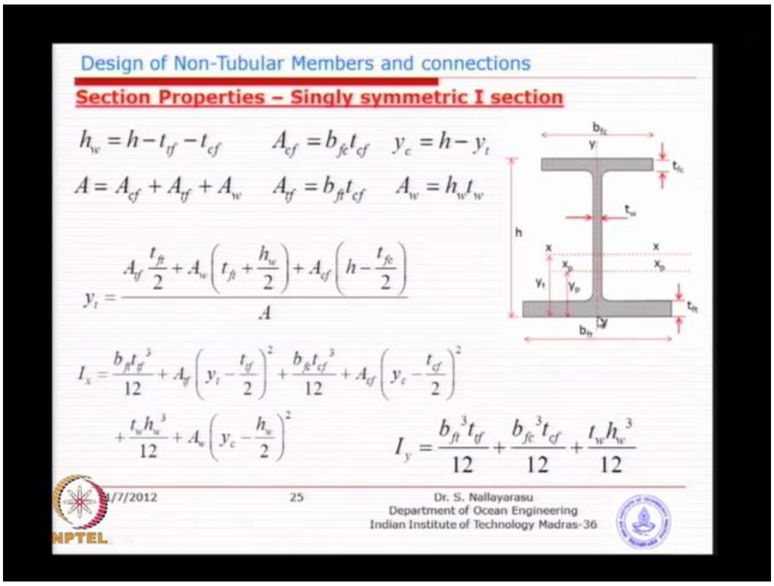
Also, we discussed about last time the elastic and plastic properties of tubular sections. I think you spend the substantial time, so when it comes to high sections, we could also derive similar properties which are I think a basic mechanics in your second year engineering. So, you should recollect the linear stress distribution then the plastic stress distribution on the right hand side, you can see here is a rectangular pattern, whereas this is a triangular pattern.

(Refer Slide Time: 41:50)



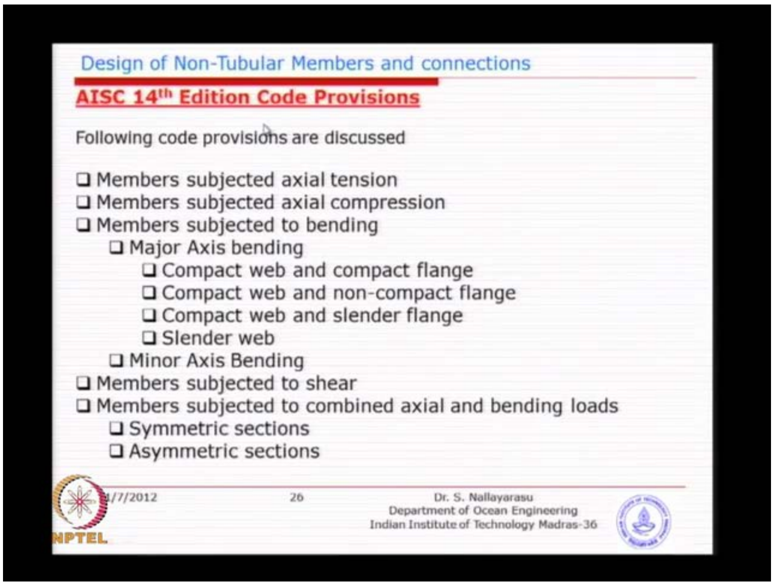
I have given in the next two slides for a symmetric sections, I have derived the elastic and plastic modulus for the asymmetric section.

(Refer Slide Time: 41:57)



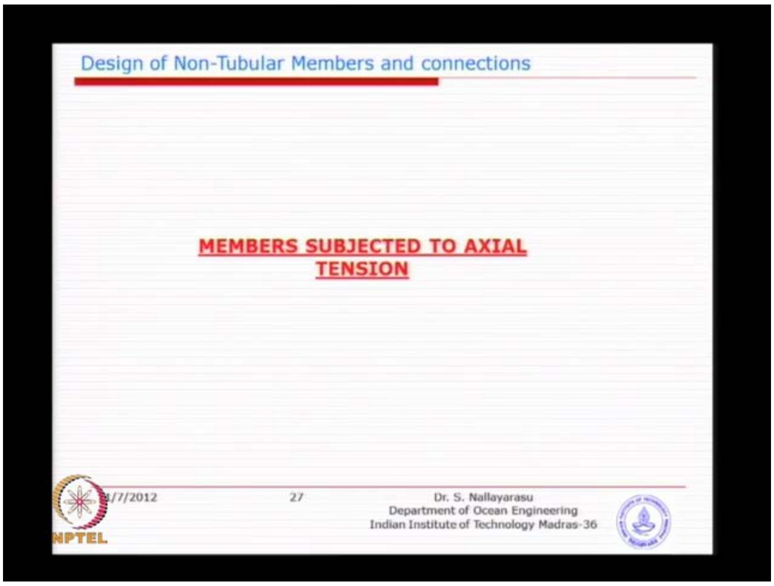
I have just put forward the elastic properties of them and symmetric sections, you can also design, similarly the plastic section properties of the slightly complicated which I could get it. So, you can actually derive it and use it for the design, so in here you can see hear the notation. We have to carefully remember s x, s y, I have given is elastic and z x, z y given as plastic properties. So, basically based on the stress distribution, we have put forward for the elastic and plastic sections.

(Refer Slide Time: 42:34)



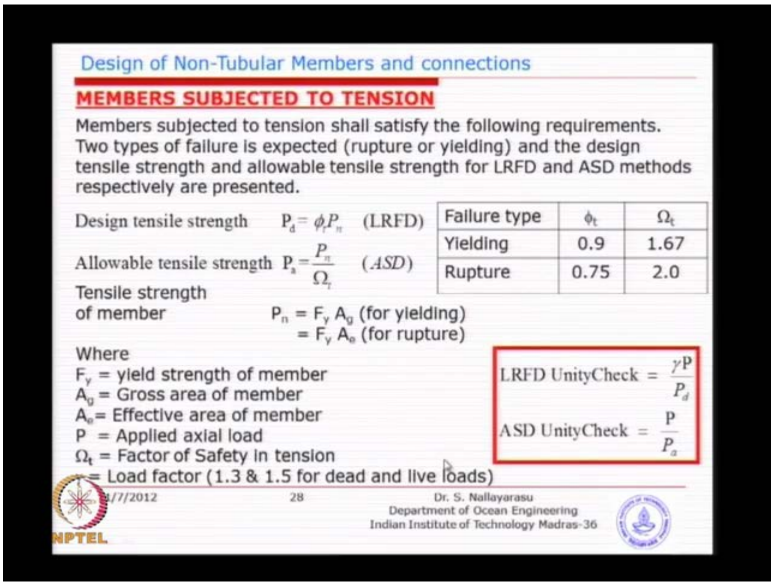
So, next class will look at what we are going to just see are the course provisions for the ASE for various forces of action on the beam, actual tension and compression bending and the shear and the combinations.

(Refer Slide Time: 42:55)



We will just quickly see the tension.

(Refer Slide Time: 42:57)



The basic idea is how the code is written the previous version of the American code for steel design is based on allowable stress method which is called ninth edition just for your information. Then, substantially time was left between the ninth edition to the next revision because lot of research activities where going on since 1899. So, you see Seines 1899, now is almost 22 two years lot of research activities was going on to incorporate them. They took a lot of time, then suddenly introduces edition called thirteenth edition from ninth edition to a thirteenth edition the logic of thirteenth edition, but what is happen is from ninth edition.

They have changed to thirteenth edition incorporating the allowable stress design as well as the LRD methods, hope all of you remember the LFRD, the difference between the ASC and LFRD and introduce. So, this thirteenth edition had provisions for both, you can choose to design by ASD or you can choose to design by LFRD. A year later, they have also revised a code because there are several anomalies, now it is basically fourteenth edition. So, remember fourteenth edition is the latest ASC code which has got provision, provisions for both methods of design. So, I have just put forward both so that in case if you have to design by LFRD methods, I have given all the formulas information.

So, you can choose to do either way, but one thing is instead of writing to codes the formulation is such that the most of the mechanics and the equations are same except the substitution parameters. You have a load factor for LFRD, but for ASD, there is no load factor for ASD, you will have an allowable stress factor and for LFRD, you will have been material factor. So, the factors are given as a table, so you just follow the same design procedure only substitute wherever you require. For example, you see here design tensile strength is defined as 5 t multiplied by p n, allowable tensile strength is basically a p nominal divided by a stress factor and p nominal is defined as this.

So, this is not going to change yield strength multiplied by cross sectional area will give you the yield force. Yield force divided by factors of safety factor will give you the allowable actual load, whereas multiplied by a material factor our strengths factor is giving you the design tensile strength for LFRD. So, you just achieve everything in one goal as long as calculated this you can calculate the allowable force for allowable stress design and design force for LFRD design. So, that is the idea behind, so you see here on the table, I have just given you the numbers and the unity check can be defined here.

For example, unity check for allowable stress design is applied force which is p divided by allowable actual force, which I think very similar to what we were doing tubular and for LFRD unity check. Basically, you will multiply by a load factor which is gamma times p divided by p design force, which is basically p d. I think with this slide, you can easily see what difference we are trying to do when you are talking about ASD versus LFRD. When you want to just dig back inside, see for example if you take these factors, you go back here you will find both methods will produce exactly same result.

It is 2 minutes and trying to find out for example, this tau t is 1.67, anyway rapture I am not talking about because most of the cases we do not have rapture. Rapture will come only when you have bolted connections, whereas we are having welded connections. If you take 1.67, you go here actually substitute here 0.675 will come as a allowable stress That is what we were looking at API, similarly you go here you take 5 t which is basically 0.9 and if you take point 1.5, you will ultimately get as similar the load factor is 1.5.

You go here, when you substitute here, you will find again 0.67 f 5 only, so there is absolutely no difference as long as you used a load factor 1.5, but if you use the load factor 1.3. Then, there will be a slight different and that is the advantages of LFRD that it allows us to use a different load factors for different scenarios that load live load variable load environmental load. So, there is potentially optimal design can come from LFRD rather than a single factor of safety from I think we spoke about it sufficient enough during the early classes, I think we can see tomorrow.