Advanced Marine Structures
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Lecture - 28
Plastic capacity of sections
Under combined loads - l

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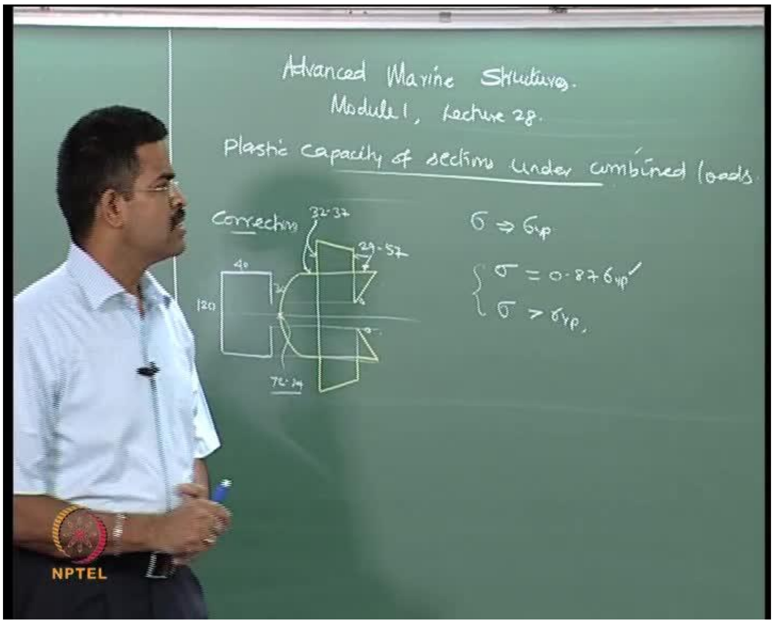

So, in the last lecture we discussed about the shear flow examples. So, we did one example where the box section or let us see the channelized section with the overhang or drop hang here, for the dimension of 40, 30, and 120. Make a shear flow diagram like this, and then we compute the shear center to be around 12.82 millimeters. So, this is small correction which I wanted you to do. The shear flow pattern is what we have marked here which is same as what we discussed in the last lecture. The shear flow on the force here at the center was computed to be 113 in the last lecture. It was actually 72.84. There is a small calculation mistake, so thanks for pointing it out it will be 72.84, it is not 113.

So, in this lecture will talk about the plastic capacity of sections under different combinations of loads. So far, we have seen that when a specific kind of load is acting may be axial tension or bending etc, here you are able to see that the stress reaches the yield value and correspond then afterwards it remains constant until the collapse occurs. But there are different theories, which say that many of the things occurs simultaneously, therefore one cannot really decide what would be the state of collapse. Because in certain stages it says that even when the stress is reaching about 0.87 sigma Y P yielding starts and the failure starts that is one theory says this.

The other theory says if the principal stress is exceeding sigma Y P there is a possibility, even then yielding will not start depending upon the nature of the stresses etc and what are the theories that we have used. So those five theories what we have discussed in the last lectures would be helpful to understand actually the mechanism by which one can get the failure loads or the yield point stresses. Now what we are interested is, those kinds of cross sections which are very common for members in marine structures, when they are subjected to combined action of loads, how would the plastic moment carrying capacity the section gets reduced? So what we are going to study now is interesting . It is the P-M interaction that is the bending environment in the axial force interaction.

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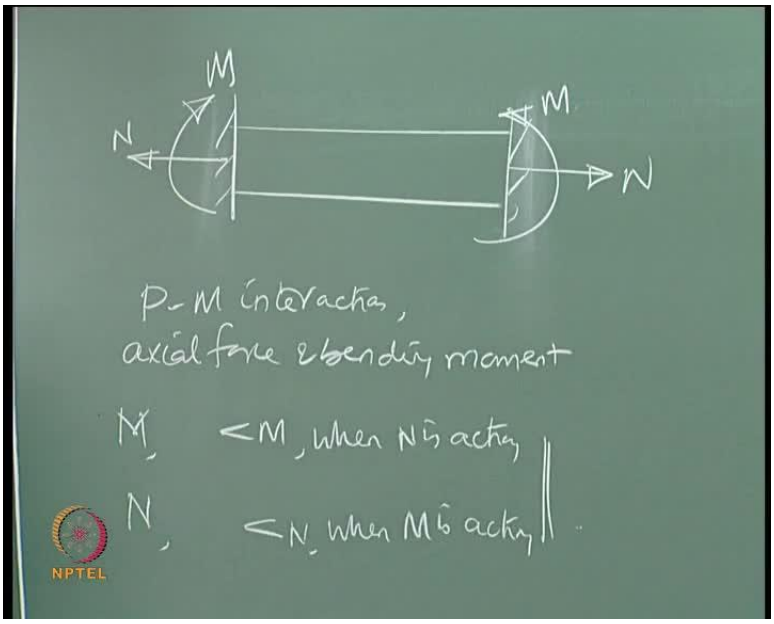

So, you must agree that if you got a beam simply supported beam for a span of l, let us say I have a fixed beam. I have fixed beam of span l subjected to moment subjected to n movement as well as axial force. I can call this as M and this as N there will be of course, a variation between this ends. Let us say the moment and actual force acting you will see that the P-M interaction that is the interaction between the axial force and the bending movement is very crucial. On the other hand the bending moment no more remain M it will be lesser than M when N is acting. The load carrying capacity in bending is reduced when axial force is also subjected to the section. Alternatively, if the axial force capacity is N it will be lesser than N when M is acting on the section, on the other hand when P and M are N and M are interacting together in the given section. There is the influence of 1 on the other which will reduce the effective load carrying capacity either in bending as well as in axial force, this what we are going to study today. Now, many members who are commonly used for marine structures are essentially to be checked for buckling effect.

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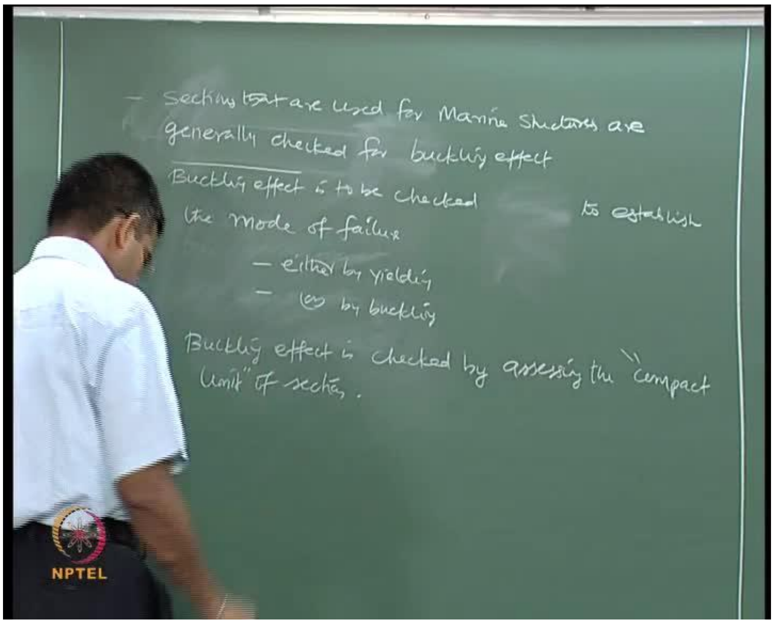

So, I should write a statement here such that it is used for marine sections on marine structures that are generally checked for buckling effect. It is because of the simple reasons that the thickness proportional to the height or the diameter etc., has to meet in a specific ratio. Now this is handled indirectly by codes by establishing what we call as a compact limit. Before understanding this, why a buckling effect is checked because buckling effect is to be checked to establish the mode of failure.

I should say to establish the mode of failure, the failure can be either by yielding or it can be by buckling. Remember interestingly the load at which buckling occurs is much lower than the load at which yielding will occur. If buckling occurs, yielding you will get due to the net axial force carrying capacity of the section will be reduced. So, one has got to check the given section or the cross section if is it safe from the buckling effect. International codes which are used for design for marine structures generally advice that buckling effect is indirectly checked by assessing the compact limit of the section. So, there is the new term called compact limit for a given section, if all the members are compact enough then the buckling effect will not be predominantly acting in this action. So, how do you establish a compact section for a compact limit for a given section?

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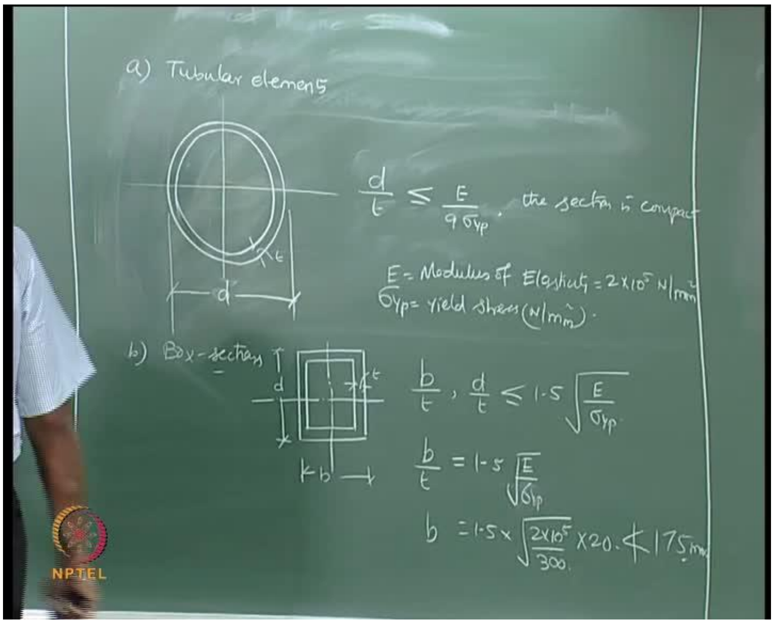

If you look at tubular elements, next call this as t and I call this as d, if d by t less than or equal toe by 9 sigma Y P the section is said to be compact, if the section is compact then buckling effect can be ignored. That is for the tubeless sections, we look for box sections these are the common cross sections which are used in marine structures. So, we are looking only those sections. This is my breadth of the section, this is my depth of the section, b is rectangular; if it is square both the dimensions can be same and this is uniform thickness of the section. In this case b by t and d by t both should be less than or equal to 1.5 times of E by sigma Y P root.

Where E is young’s modulus of the material, where E is modulus of elasticity which is generally 210 power 5 for steel than sigma Y P is the yield stress in Newton per m square. So, if this is established you can imagine if you are looking for a thickness of let us say 10 millimeter of a box section, no dimension of the section should be less than let us say for example, b by t is said to be equal to 1.5 E bysigma Y P.

Let us say 1.5 into root of 2 10 power 5 by, let us say I take sigma Y P for this material as 300. Let us say for 10 mm fix, 20 mm fix section breath, cannot be less than or equal to; Can you give me what is the value what is the approximate of this?. Let us say d cannot be less than 170 millimeters approximately. You can say that the size of the section for aggregate box section of 20 mm thick should start from 200 above. That is an idea if you have section like that the section is established to be compact buckling effects will not be there. You may wonder why buckling effect should not be there ? Because it reduces the load carrying capacity and the load at which buckling occurs is much earlier than the torque loaded which yielding will occur. We are talking about plastic capacity, so we are focusing on yielding only. So, the section would not buckle for enable the section to the yield this is for the box sections.

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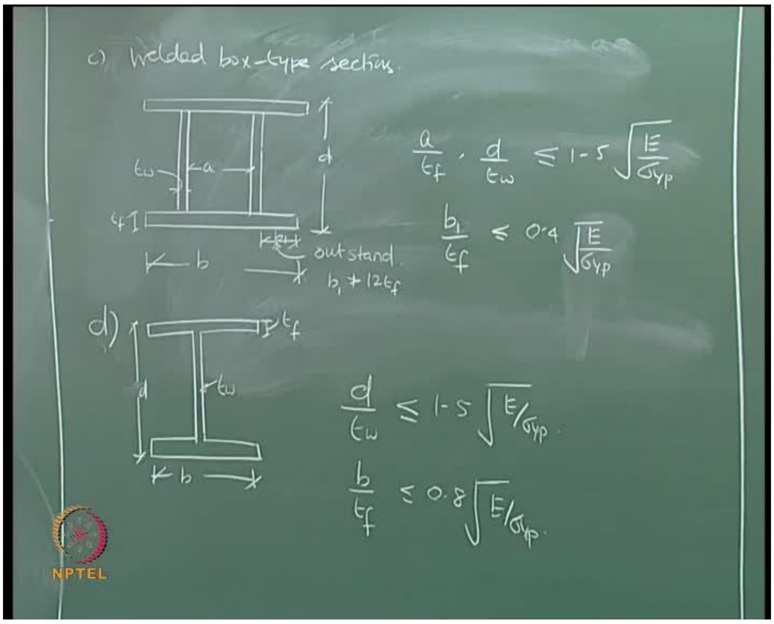

Now, you also have welded box sections, these are common; these are actually used for pipe racks in marine structures. People construct the box section and insert the pipes, coned insert this and welded box types are very interesting section like this, you have upper and lower flinch plates then with that flinch plates you create a box section. This a typical welded box type section, this is what we call technicallyas an outstand and let say this is my breadth of the section.

And this dimension is a and this is thickness of the flinch and this is thickness of the wave and generally it is understood that the outstand , if I call this outstand and let say b

1, b 1 generally should not exceed 12 t fand off course this is d depth of the section so the condition is a by t f and d by t w should be less than and equal to1.5 times of E by sigma Y P for making the section or confirming the section to be compact. We talk about outstand b 2 by t f should be less than and equal to 0.4 E by sigma Y P. I am not sure I will check of whether this is whole root or only root of sigma Y P, I will check up this. So the section is construed to be compact when this is satisfying we can all say verify this.

One from the course whether this is whole of sigma Y P sigma Y P, b 2 I will write b 1 outstand, outstand review. So, d standard I sections where thickness of the web thickness of flinch breadth and depth. So, the condition for the section remain compact is d by t w should be less than and equal to 1.5 times of E by sigma Y P. This is also I think it should be whole root of E by sigma Y P. Let us verify this that and b by t f should as equal to 0.8 of E by sigma Y P. So, it is important for us to know but how do I establish may section to remains as compact because compact sections will not have buckling effect acting on them. You directly do the yielding load that is why we are discussing this.

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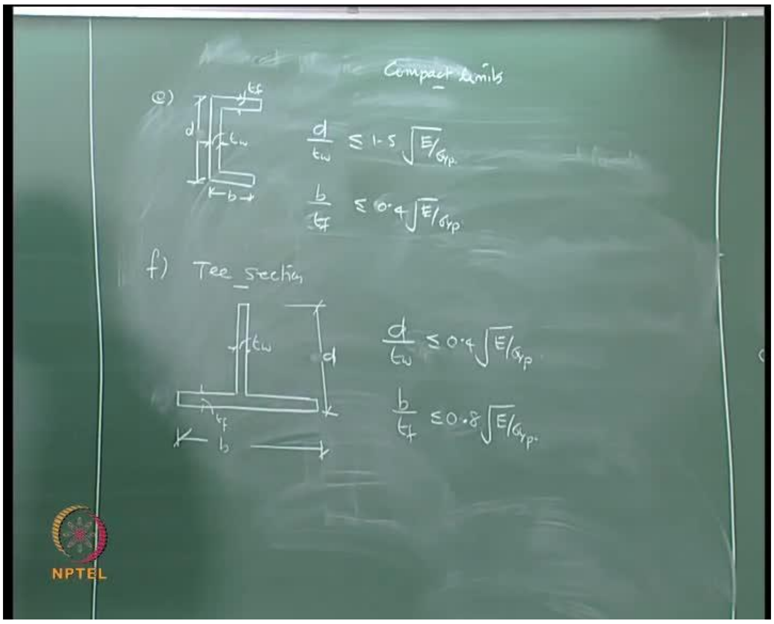

The next common section of course, the channel sectiont w t f and let say this is d and this is b. So, d by t w is less than E by sigma Y P and b by t f should be less than 0.4 E by sigma Y P. We also have a Tee section d d by t w 0.4 E by sigma Y P and b by t f 0.8

by sigma Y P. These are all compact limits.These are all compact limits, when these values are satisfied the section is said to be compact and the buckling effect on the sections can leave note in the analysis.

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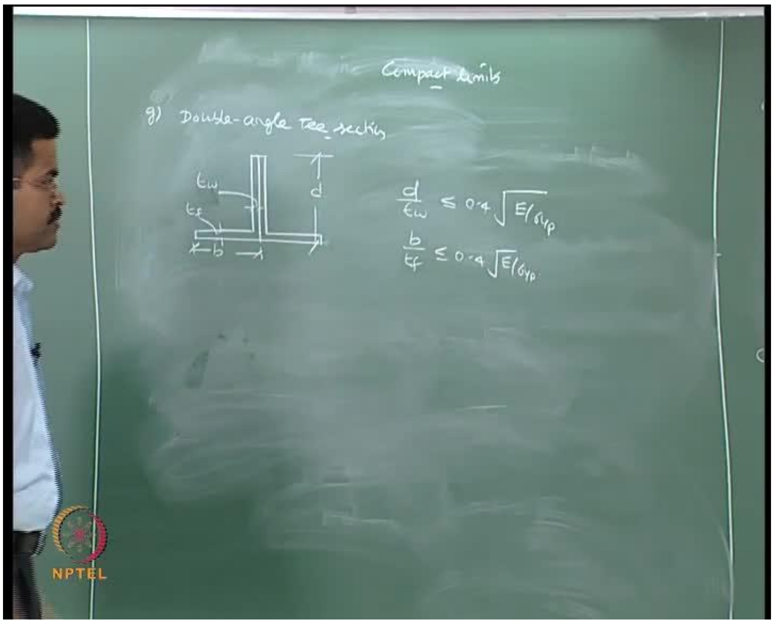

You have double angle tee1 more section.Each dimension is b, your overall depth is d.Now off course, each thickness is t f. This is web and off course this is the flinch. Sorry, this is t w and this thickness is t f. In this case, d by t w should less than and equal to 0.4 E sigma Y P and d by t fis also same relation. So, these are some of the guidelines suggested by the codes based on the geometric composition of different elements, how to establish whether the given section is compact or non-compact.

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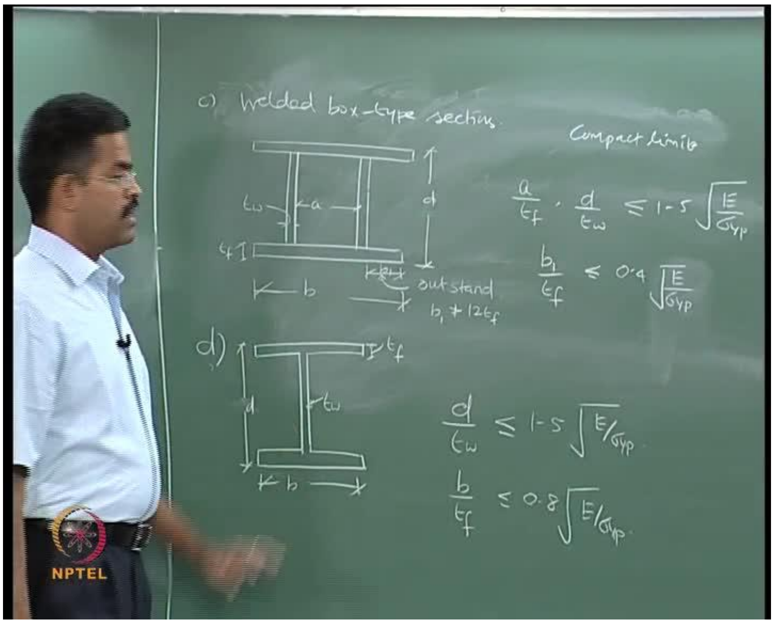

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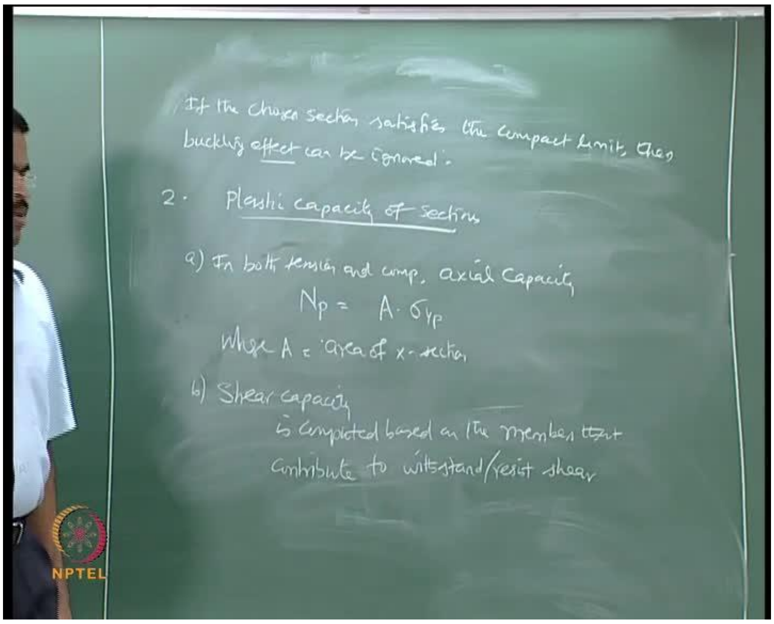

I will remove this! So, we rewrite the fact that if the chosen section satisfies the compact limit, then the buckling effect can be ignored, that is the recommendation that the code can be ignored. Now, let us talk about plastic capacity of sections. So, we spoke about the buckling effect or consign the buckling effect of sections. We said establish something called the compact limit in a given section ,if it is satisfied, then buckling effect can be ignored. For ofcourse non compact sections, buckling section should be considered in the analysis. Plastic capacity of the section, let us say in both tension and comparison axial capacity that is plastic load carrying capacity and axial forces both in tension and comparison is denoted let N p. p stands for plastic and N stands for axial, is simply as we know is given by A sigma Y P. Where A is area of the cross section. Shear capacity is computed based on the members that contribute to withstand or I should say resists shear. If ,I wonder what you understand by the term members that contribute to with stand shear we will see. Let us say I have 2 cases of an I section.

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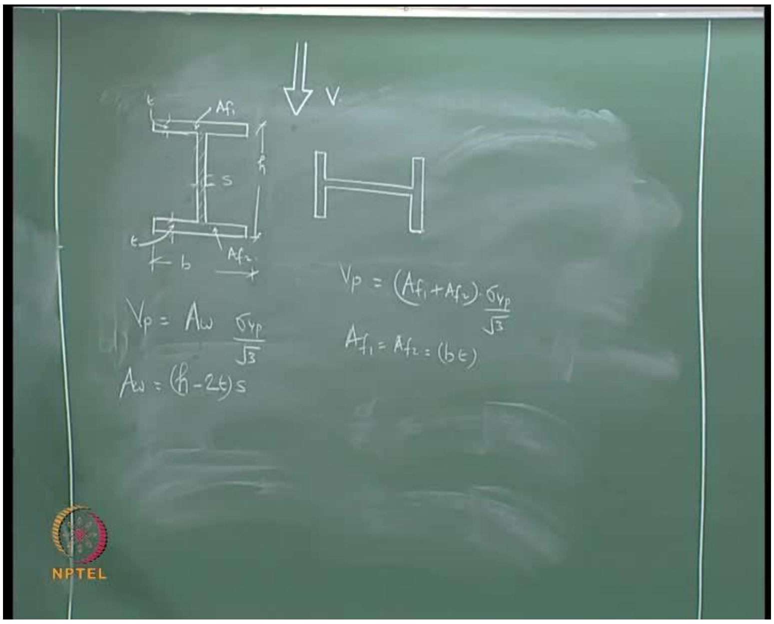

One is kept vertical in elevation, other is kept horizontal. Let us say it is tilted, now section is kept like this it is placed orthogonal to the previous one and this is my direction of the shear force. This is where my V is acting. This is the direction of the shear force. I call the thickness of the width as s and I call the overall depth as h. I call this as area of flange 1, this as area of flange 2, this dimension is b in both the cases and in both the cases this is t, and the same section is just rotated so the properties or the geometry dimensions or the notations are exactly the same. What I am interested to compute the plastic capacity under axial forces, may be you have done for axial force we are now looking for shear.

We said that what are those members which will contribute for shear, so we have got account for those members. Now I go to the orientation like this then the members which do not contribute of shear are the flanges or on the other hand only the wave contributes for shear. So, flanges take bending and wave takes a shear in this orientation whereas the other way the flanges will now take the shear and this will take the bending. So, depending upon what is the orientation of the section with respect to the force acting direction, we must always either consider or ignore certain members for shear capacity of the sections. So, I can write down here the shear capacity of this section of this section which I say V p which is A w into sigma Y P by root 3.

What is A w? A w in this example is nothing but h minus 2 t into s, is that ok? That is area of the wave alone this is 2 t. I may write it deliberately clear h minus 2 t into s that is area of the wave, that is this area N and sigma Y P is of course, the yield stress plastic stage. Why it is 1 by root over 3? Look at the distortion theory in the last lecture, what is the stress at the minor axis at second and forth quadrant? This is nothing but 1 over root 3 you can see that you can look at the comparative figure what we made in the last lecture you will see as 1 over root 3 is the value of shear at yield.

Is it not distortion theory, so let us look at V p, for this problem I would obviously ignore?

Now the width and I’ll consider only the flanges, so I should say simply A f 1 plus A f 2 of sigma Y P by root over 3. That is my V p, whereas A f 1 is also equal to A f 2. In this case is nothing but b and t. So, for shear carrying capacity you must consider only those members which contribute or assists for shear not all, any questions here? Having said this let us now talk about rectangular sections and we divide them into 2 parts or 2 classifications, 1 is what we call open sections other is closed sections.

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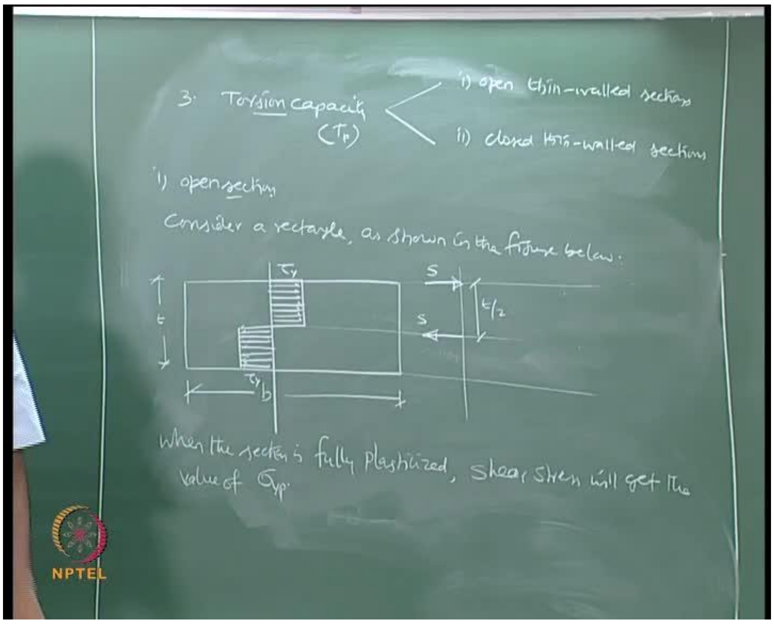

So, we now estimate the torsion capacity we estimated the axial capacity, we estimated the shear capacity, we will now talk about torsion capacity. So, we divide this into 2 level of understanding, 1 is open thin walled sections. I will come to the point what are thin walled sections, second is closed thin walled sections. The name itself very clearly tells, what is thin walled section.

The thickness of the section compared to its diameter or the size is very, very small, in that case we talk this as a thin walled section. What we generally used in members of marine structures. So, let us talk about an open section for our discussion to compute the torsion capacity which I call as T p, T stands for torsion and p is always for the plastic capacity of the section in the whole discussion of this lecture. So, let us say consider the rectangle as shown in the figure 1, I should say as shown in the figure below because I do not have the numbering of the figures sequentially in this lecture. So, figure shown below now figure shown below is this.

This is my t and let us say this is my b. Let us say at any section I look at the shear distribution because torsion is related to shear. You also know, why we say torsion is related to shear? Any idea, why torsion is connected to shear? If you look at any Indian codes and International codes generally if you want to design a section for torsion we always work what is called equivalent shear. In design torsion is always converted or handled as a form of shear. In Indian codes for example, IS4 5 6 design of reinforce concrete members, you will always find out whenever the section is subjected to shear and torsion we find out what is called equivalent shear.

So, the shear will be enhanced by certain amounts, that are equations available in the code that is not the focus here. So, it is always a trained or a method by which torsion is handled and designed by the designers. So, we are talking about shear here, I am talking about the shear distribution here. Let us say the section is having a shear distribution as shown here. So, I say this tau y and this is also tau y the shear in the extreme fibers is fully same.

So, let me pick up a value here and draw a section and say that my shear here in the top fiber is s and the bottom is also s and this is separated by a distance t by 2. Is it not? We understand that when the section is fully stressed, I should say fully plasticized, shear stresswill get the value ofsigma Y P. Is it not? We have studied this in the last theories of failure sigma Y P. You can say yield value.

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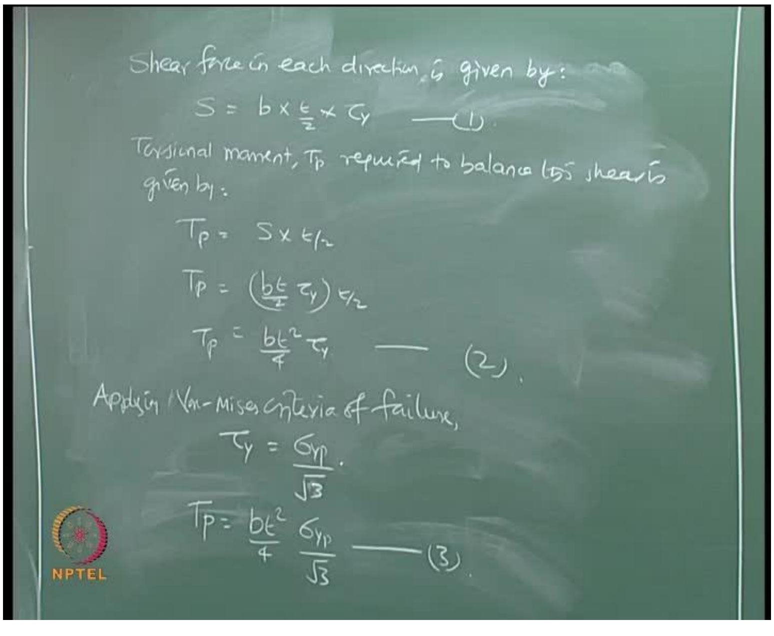

So, let us say shear force in each direction is given by, so s is equal to b into t by 2. That is the area and the stress of course, I am writing here as tau y. The torsional moment now Tp required to balance the shear is given by T p nothing but s into t by 2, is it not? That is the torsion that is the movement of about this point. So, couple is t by 2, so that becomess is b t by 2 whole of t by 2, so b t square by 4 tau is my torsion. Applying the

Von Mises criteria of shear failure, what is 1 criteria and failure? Then tell me what is a Von Mises criteria of failure? Maximum distortion.

Maximum distortion, so what is shear thought we are talking there? What is the maximum value of shear? So tau y will become equal to sigma Y P by root 3, is it not? Is it not? Therefore, my t p will now become b t square by 4 sigma Y P by root 3 so on. Why we are talking about distortion theory when you talk about torsion capacity, any idea? There are many other theories, we can talk about principle stress theory, maximum stress theory, string energy theory and so on. Why we are talking about the distortion theory here because torsion is associated with distortion, Is it not? Whenever you apply torsion to a cross section you will always see the section will be distorted or the fibers will get distorted. That is why we are applying the maximum distortion theory here and that says it is equal to sigma Y P by root 3, so we have used t p as the this equation.

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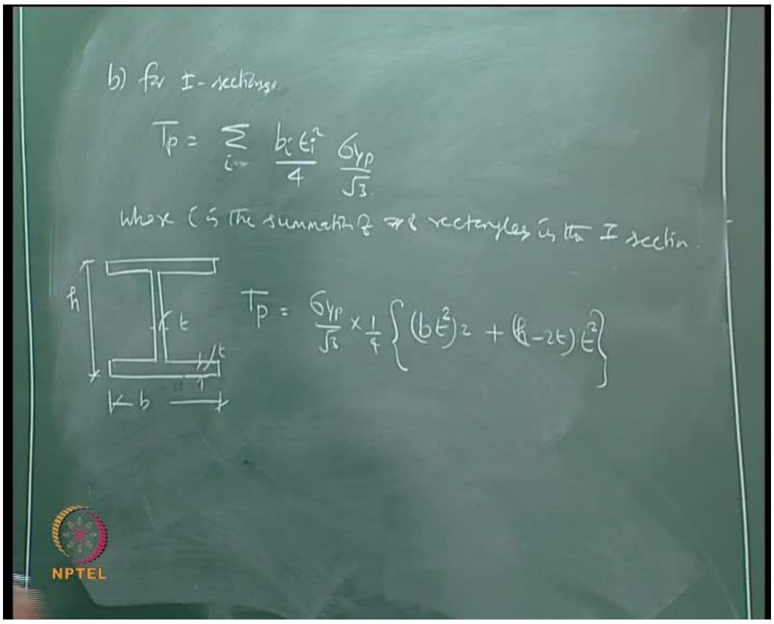

If u really want to apply a concept for a I section, the same concept can be applied as it is; T p will be now sum of i, let us say b i t i square by 4 sigma Y P by root 3. Where is the summation of number of rectangles in the I section. So, if you have an I section, so I can say T p this in this expression, this is b, these are all t, this is also t, this is also t and this is h. So, sigma Y P by root 3 into 1 by 4 is common. b t into 2, there are two b t's plus h minus 2 t of t on that squares. Sorry, let us say b t square because there is t square in the equation, b t square of 2, there are 2 elements here and this is h minus 2 t of t square, is that ok ? This will give me the torsion capacity directly. These are open sections. Talk about now of close sections.

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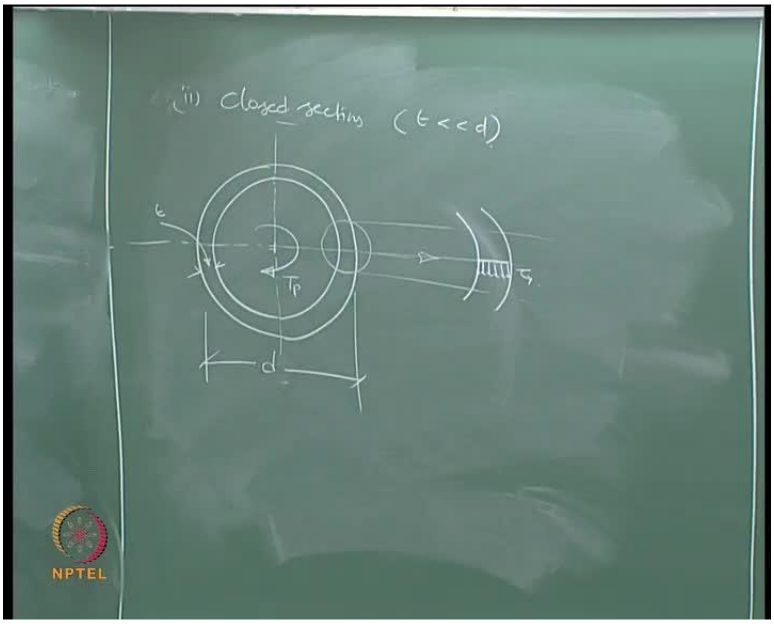

Let us say the ring as uniform thickness t and of course, the diameter of the ring is d and the center the torsion applied which is T p. Let us look at this and try to look at the distribution. Any 1 specific level, let us say that is the force which is acting and we call this as my tau y. So, we are talking about thin walled sections where we say t is much lower than d, please understand that we are talking about thin walled sections.

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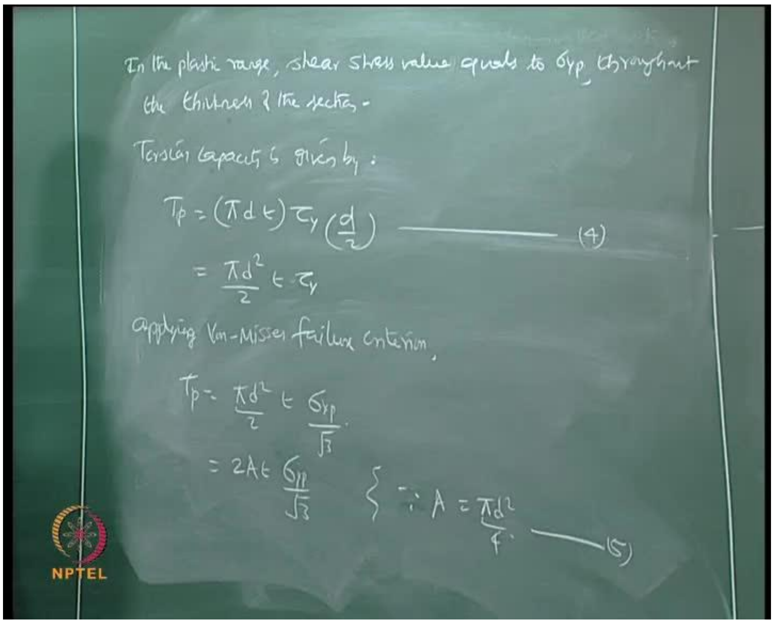

So, in the plastic range the shear stress value equals to sigma Y P throughout the thickness of the section, that is the assumption or reality. This is true because we are talking aboutthe thickness where t is less than diameter of the section thin walled section. So, the thickness is uniformly plasticized for whole value of t, there is no question of partial plastic and partial elastic core lying within the thickness.

It is fully plasticized, full thickness is having tau y that is the statement. Then the torsion capacityis given by T p is pie d t, pie is the circumferential length and t is the thickness that is the area, multiplies by tau y will give me the force and movement d by 2, is it not? I am talking about the stress values here, this is d by 2. Now I am taking the movement of d center is, is not torsion is about the centerd by 2. So, this is I can call this equation number, I do not remember the sequence let it be 4, is that 4?

So, pie d square by 2 t tau y, so applyingvon mises failure criterionT p can be said as pie d square by 2 t sigma Y P by root 3. That is what von mises stresses say, I can also call this as 2 A t sigma Y P by root 3, because A is pie d square by 4, is it not? So, for close sections you get torsion capacity like this. So, in this lecture we discussed about how to estimate the torsion capacity for different sections open and closed. For different standard cross sections in geometry being used as members in marine structures what would be the necessity of studying of buckling effect. Because then we eliminate the buckling effect if the section is declared as compact section, or compact limit is proved or established which is given by the expression is an equations as per we discussed in the lecture. In the next lecture we will talk about what would be the plastic movement carrying capacity or plastic capacity of the sections and the combined action of shear axial, shear bending, shear torsion etc, combined effect of these forces in the next lecture.