Design of Connections in Steel Structures
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Lecture - 23
Ductile Detailed Beam-Column Connections-2

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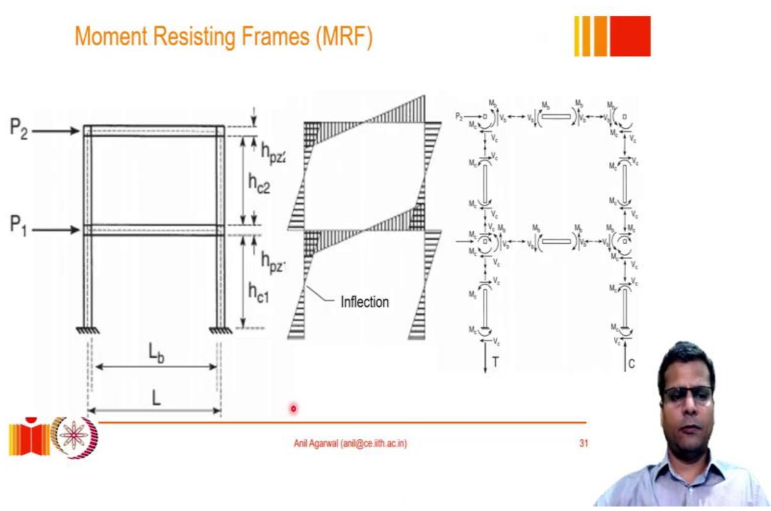

Now in this lecture we will focus primarily on the moment frame resisting systems. And there are two distinct types of movement frame resistance systems one is known as ordinary moment frame resisting moment resting frame the other one is known as special moment registering frame. There is also another variety prevalent in American codes, known as intermediate moment registering frames, but that is not mentioned in the Indian course.

So, I will not talk about that here. So, how does the movement frame moment resisting frame typically works. If we have this frame and we have these two forces P_1 and P 2_acting on this frame this plane this frame would want to deflect laterally and in the process it will develop a bending moment diagram like this and any of you who have taken analysis basic structural analysis would know that this is how the bending moment diagram could be there will there be points of inflection in each member here.

And so, basically, each member would be under a double curvature kind of scenario. Now since we are interested primarily in connections. So, if we look at the connections here these connections will be subjected to bending moments in this manner there will be bending moments coming from the columns which are both for example if this member is shifting right ways towards right.

Both these moments will be in the clockwise direction and the moments that are coming from the beams in this case there is only one beam there could also be another beam and if there are two beam's both moments will be in the anti-clockwise directions and they will balance each other. In addition to that there will be large shear force demands on this one. So, shear forces will come from the connection from the columns.

And there will be shear forces coming from them which is marked as V_b here shear force is coming from the beam and these all these forces along with the axial forces if there are any they will be in equilibrium and they will be introducing different types of bending moment shear force demands on the connection needs to be designed for these forces.

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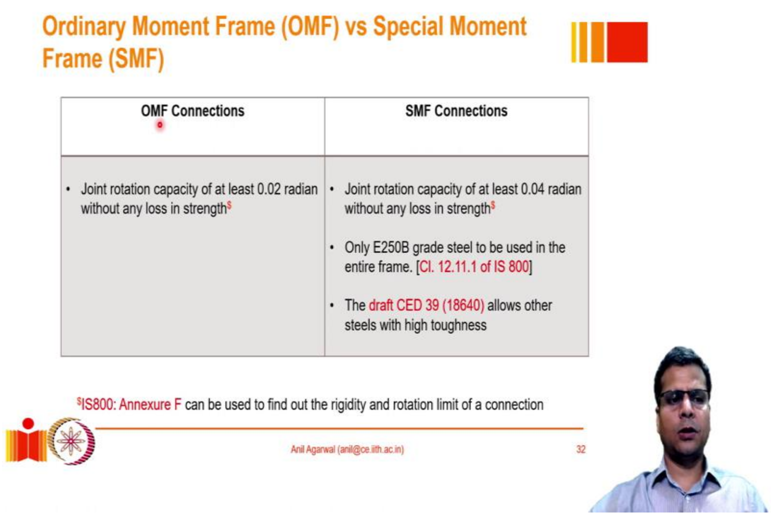

So, as I was talking about an ordinary moment resisting frame versus a special moment resisting frame. So, an ordinary moment resting frame is basically a less that type of rigid connection it is also a rigid connection sometimes even some additive connections are also permitted or rough can also be considered as an ordinary moment resistance member or element.

But special moment existing frames have to be digit connections only the requirement for ordinary moment resisting frame is that it should be able to a connection that is a part of on an ordinary moment resting frame the connection of the joint should be should have the capacity to at least resist or to at least accommodate a 0.02 radian rotation without any loss in strength. What do we mean by this much of rotation?

Basically if we can think of a column like this and a beam which is connected to this column digitally like. So, and if we let us say have these columns in fixed position and the columns are valid they do not move and the P moves the beam should be able to move by an angle which is 0.02 radians. So, this column would be undergoing a kind of an elastic deformation or plastic deformation of 2% which is basically 2% of the storage of so, 2% of the storage height.

So, that is the level of plastic deformation or overall deformation it should be able to accommodate there is a for a special moment frame however this requirement is more stringent and it requires at least a 4%, 0.04 radians of relative rotation between the beam and the column without any loss in strength. So, that is a very critical aspect. Now how do we know which connection is eligible.

Or which connection is capable of undergoing this kind of a deformation without any loss in strength for that we can use we can go to IS800 there is an extra f which is taken from your whole face research wherein they have developed some nice connection.

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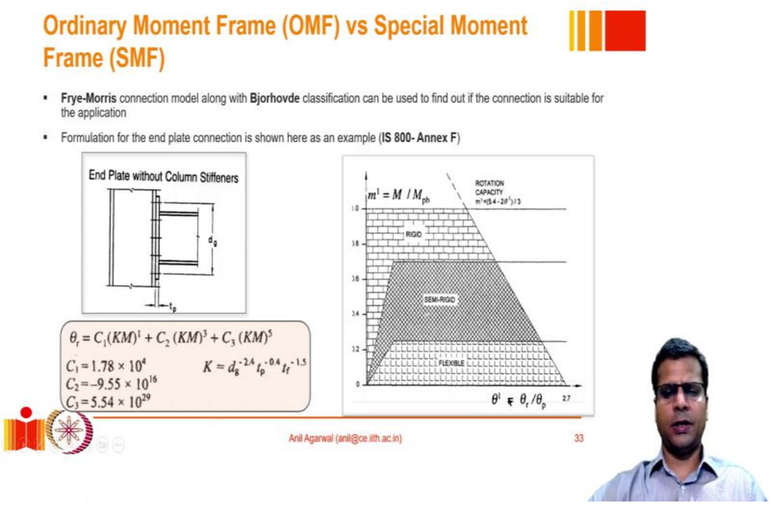

With curves which relate the rotation with moment, there is a limit set according to this equation. So, from various types of connections these equations have been developed and we can actually fit our connections we depending on all the parameters that the connection has we can actually find the values. And then we can find out what my connection's rotation capacity is and that should satisfy the 2% or 2 radians or the zero points zero p radians or 0.04 average requirement.

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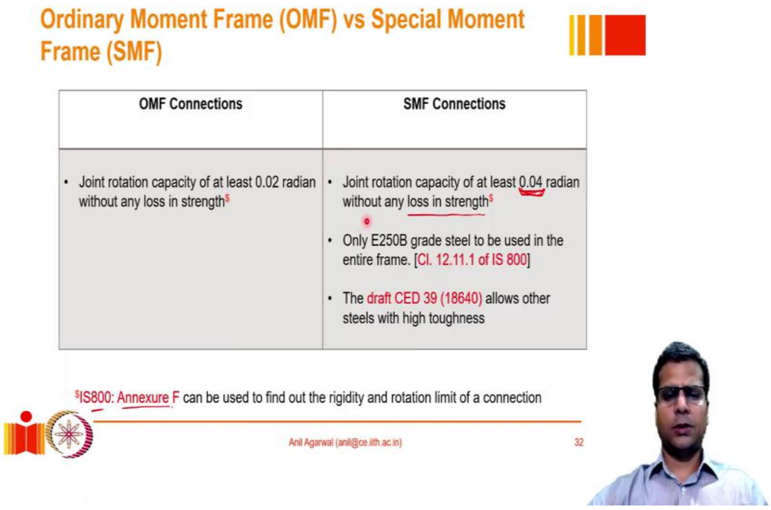

Also for such connections IS800 specifies that only E250 B grade of steel can be used in the entire frame in a special moment frame since we require a larger level of ductility. And also since we want these frame members to behave as if they are to behave as a ductility or deformation control member we do not want them to have very high over strength. So, therefore it is it is required that these numbers should not be of a greater grade than E250.

However in the revised code, now there is an allowance to move even higher grade steel that is even 350 and so on can be used but they should still be having a certain amount of high toughness. So, high toughness requirement is not relaxed but the strength can be higher and the other neighbouring members can be designed accordingly.

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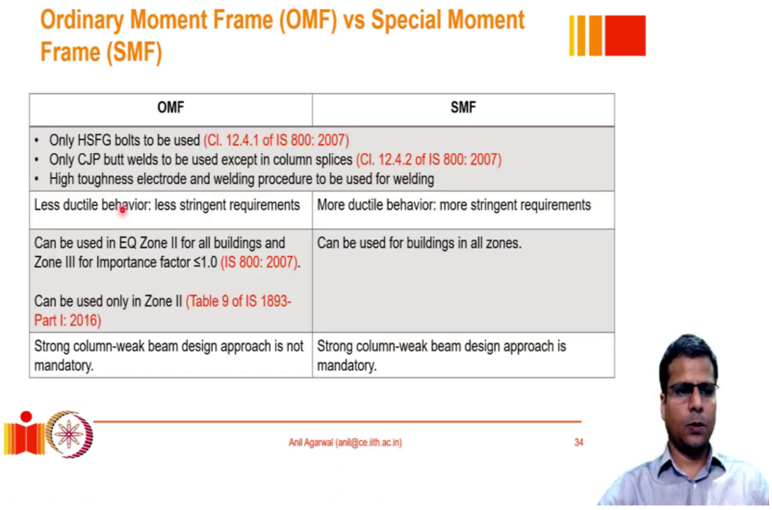

OMF	SMF
• Only HSFG bolts to be used (Cl. 12.4.1 of IS 800: 2007) • Only CJP butt welds to be used except in column splices (Cl. 12.4.2 of IS 800: 2007) • High toughness electrode and welding procedure to be used for welding	
Less ductile behavior: less stringent requirements	More ductile behavior: more stringent requirements
Can be used in EQ Zone II for all buildings and Zone III for Importance factor ≤1.0 (IS 800: 2007). Can be used only in Zone II (Table 9 of IS 1893-Part I: 2016)	Can be used for buildings in all zones.
Strong column-weak beam design approach is not mandatory.	Strong column-weak beam design approach is mandatory.

Ordinary moment resisting frames both the ordinary moment resistant field and the special moment resistive frame have certain restrictions one is that only high strength friction grip poles can be used in both of them because they are subjected to cyclic loading conditions and then we will be discussing the voltage connections we did mention that whenever there is a possibility of a cyclic loading.

Only high strength friction reports should be used that is mentioned in IS800 also the same code mentions that in the locations where there is supposed to be a butt when subjected to critical loading conditions only complete joint penetration weld should be provided that the weld electrode should be of high toughness and the welding procedure that is followed through should also lead to a high toughness.

In certain cases the group welding is provided in the column splice it may not be required to go for a CJP complete joint penetration type of weld especially when in the situations where the column is primarily going to remain in compression. As we have discussed before, the ordinary moment frame offers less level of ductility compared to the special movement frame; therefore, the requirements in special movement frame are more stringent.

For example, ordinary moment frames can be used only for resting earthquake loads in earthquake zone 2. So, that that is the zone which has the least earthquakes in India and IS800 mentions that OMFs can be used in zone 3 buildings also as long as the buildings have an important structure of less than or equal to 1 that is not very critical buildings for all other buildings that is in zone 3 all the buildings of importance factors greater than one.

Or in zone four and five we should always only use the SLF is 183 part one again differs slightly with IS800 the IS193 says that OMFs can be used only in zone two. So, they do not allow the isolate time c does not allow the use of OMF in the in zone three even for less important buildings also there is an important philosophy which when it comes to the design of resistance structures which is known as strong column design philosophy.

So, this approach is not mandatory for ordinary moment phase however it is mandatory for special moment frames.

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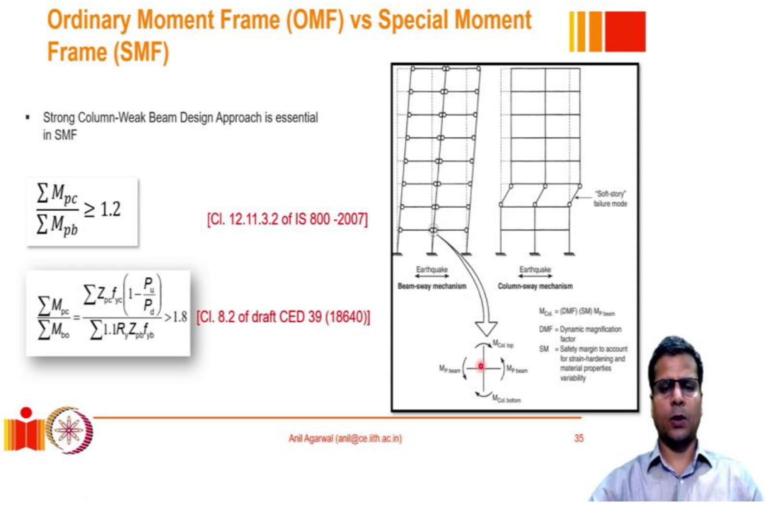

What do we understand by strong column weak beam theory. So, at a joint the idea is that under the earthquake loading the hinges would form but those hinges should not form in the column. So, for example, in this frame at this location, if the hinge has formed in the column that leads to a complete unstability of the structure, then that structure could collapse.

However if these hinges do not form in the column but they form in the b then that allows a much more ductile and much more stable behaviour and allows for a progressive kind of a yielding of the structure. So, in order for this to happen we have to look at this joint let us say this is that center of the joint and we have beams going in this direction and columns going in this direction.

So, the combined moment which is for example in this case the combined moment of the columns is going in the clockwise direction and combined movement from the beam system in anti-clockwise direction the combination of moments in the beam the moment capacity of the beam the two beams combined should be less than the moment capacity of the column. So, since the two are going to balance each other once the beam has a lower capacity it would fail first and then we have to make sure that it has ductile detailing.

So, that even though it meets first, it does not necessarily lead to a drop in moment demand from here but allows for larger deflection in the frame and in return it will absorb some seismic energy. So, IS800 for example gives this provision wherein you are required to calculate the moment capacity of the column at the joint and moment capacities of the beam at the same point you add them up.

And then the ratio of the two should be greater than or equal to 1.2 this is very similar to other international standards that are prevalent. Now there could be some variations in the way this M_pb or M _pc are calculated sometimes they are calculated including the slenderness effect sometimes they are not the new draft version of the ductile detailing further detailing of steel structures gives a very stringent requirement.

So, this also has the same philosophy here in (summation M_pc)/(summation M_bo) should be greater than or equal to 1.8. So, this is that the column capacity at a joint should be at least 80% greater than the beam capacity this in my opinion is very stringent we are still yet to get more details on this code I hope as the time progresses we will get more details about this code. So, essentially what they are doing is here is that they are giving more details on how to calculate the (summation M_pc) &(summation M_bo).

So, for the calculation of bending moment capacity of the beams they are adding a factor of 1.1 which is just kind of a reserved strength which they are accounting for just in case there is an additional strength R_y corresponds to the difference between the characteristics of the material and the expected strength of the material. So, a 250 yield strength steel would have a characteristic strength of 250 MPa but an expected strength could be resistant could be higher than that.

So, whatever that value is, the ratio of the two should be used here, which is the plastic movement capacity and yield stress. So, that would give you the moment capacity of all the beams that are connecting to that joint. Similarly another formula is available for calculation of the capacity of the columns. So, here you may see that this factor of R y and 1.1 are not used.

Because we want to be conservative and in addition to that the; slenderness effect is also included because if the slenderness reduces the capacity of the column that effect is included by this factor right. So, if you have learned how to design slender columns you would have come across this expression which actually accounts for the reduction in strength in the column because of its wilderness effect.

Now the only issue in my opinion is that 1.8 factor seems since too large maybe there is some rationale for that we have to hear more from the community to understand it fully.

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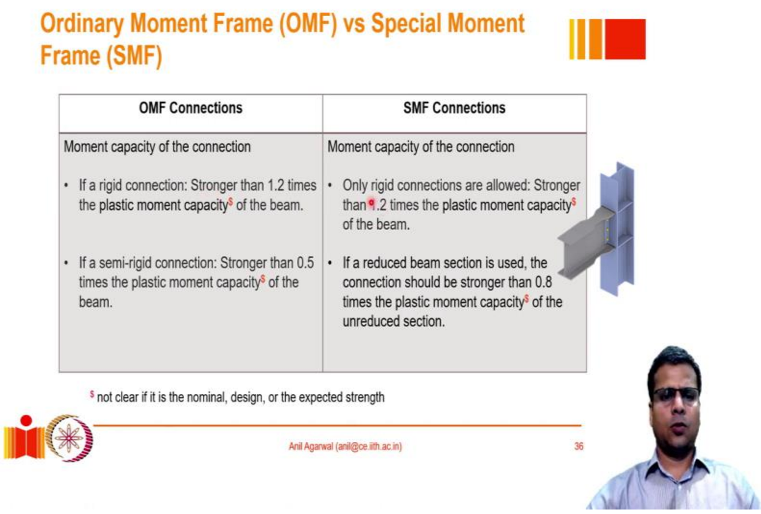

So, as we have just seen, the code requires the existing code IS800 requires the columns to be stronger by more than 20% than the beams. So, columns for moment capacity should be at least 20% greater than the beam's moment capacity, which is true even for an ordinary moment frame. And special moment frame if they are using digit connections however ordinary moment frame allows the use of semi-rigid connections.

And then it allows for the use of scenario connections then the connection should be stronger than 0.5 times the plastic moment capacity of the beam. Whereas in the case of special moment frame the connection should always be rigid and at least 1.2 times the plastic capacity of that. Now what is the strength of the connection the strength of the connection is basically corresponding to the strength of the weld.

And the way the force is transferred to from the beam to the column. There is also a provision of using a reduced beam section and a schematic of such a beam system is shown here. Here you can see that the beam cross section is uniform almost up to the point where it is connected to the column but just before that it is reduced artificially in a very smooth manner and the idea here is that we want to artificially create an area which can be designated as the hinge area.

And when a high moment demand comes it does not produce a failure here but it produces it should produce a large amount of yielding in this region. So, that the failure can be prevented failure at the connection can be prevented because the fear is that a connection failure which would probably be a wind failure here will be catastrophic whereas if we can move it away from the column it will be more gradual and more manageable because it will be it will be yielding driven.

So, in such a section if such a cross section is used the connection it is described that the connection should be stronger than 80% of the plastic moving capacity of the unreduced section. So, the taking both of these provisions simultaneously what it means is that this connection should be 20% or 1.2 times stronger than the reduced cross-sections moving capacity.

At the same time it should also be at least 80% or stronger than the at least should be stronger than at least 80% of the moment capacity of the section away from the reduced section for unreduced section.

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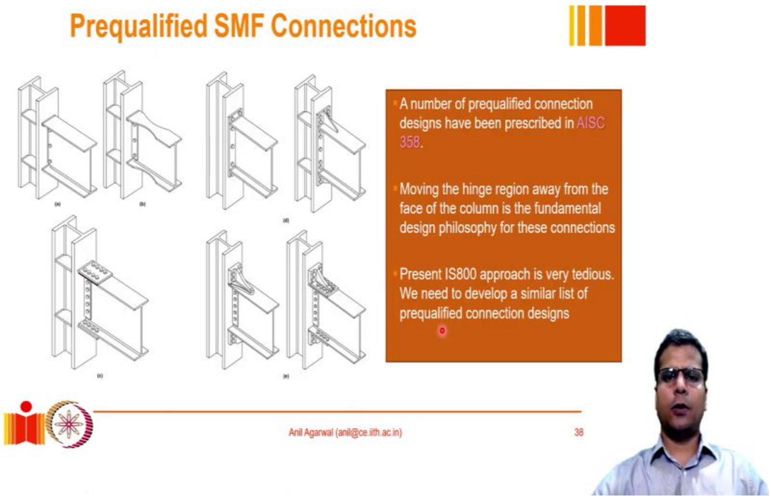

In order for us to know the rotation capacity of a connection presently we have to go to an action F of our IS800 which gives a very cumbersome process and it would be very very difficult for a design engineer to actually be able to calculate the rotation capacity of any given connection design. What American standard AIC358 does is that it prescribes or it provides a list of some pre-qualified special moment registering frame connections.

Some of them are shown here, and some of the diagrams are shown here. So, these connections the diagrams are not giving the entire information you have to go to this code and find out the different dimensional requirements for these connections and if you fulfill those dimensional requirements then you can be assured that this connection will be able to develop or will be you will be able to use it in the special moment resisting frame meaning that it has the it has adequate rotational capacity.

So, for example if the rotational capacitor rotation capacity should be at least 4% or 0.04 radian then these connections as long as you follow the guidelines of these connections you would be assured of that much of the rotation capacity in Indian court right. Now we do not have any such provisions and that makes the life of an engineer a bit difficult I think ISO should also develop some such pre-qualified connections which could be handily used.

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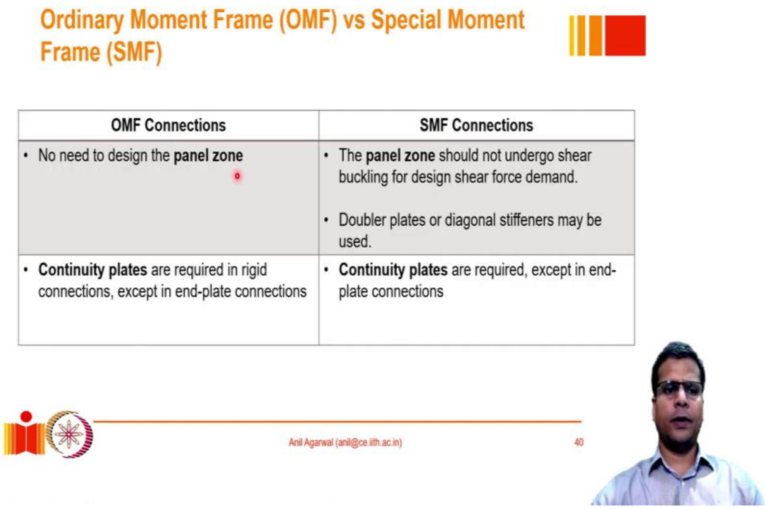

Another aspect of a ductile design of a beam column joint is the development of a panel zone. So, the panel zone is basically the intersection between the beam and the column. So, when we have let us say we have a column and then we have two beams coming and meeting this column at this region and then these beams wish to or want to rotate in this fashion and let us say the column is held in its position.

So, this will basically produce some kind of a shear zone shear panel zone in this system and that shear panel zone this will be a shear panel developing in the column web which is generally considered to be very ductile in nature because it is governed by shear yielding of the column width however if the column cross-section plasticizes significant that can lead to an overall instability. So, there are a couple of requirements in order corresponding to the performance of such ear panels.

So, for ordinary moment resistance frames the shear panel are not required to be particularly designed and only the continuity plates are required in digit connections. If any and great connections are not used if any connections are used then we can assess the requirement of continuity plates they are not mandatory. If we are welding the flange directly to the column flange of the beam directly to the column, it may be then it is continuous in on any moment resistant frames.

In the spatial momentaries difference however we have to check the behaviour of the panel zone even though it can perform in it can deform in a very ductile manner excessive deformation here can lead to instability of the column itself. Therefore we should ensure that the panel zone, even though it may plasticize to some extent, should not undergo shear buckling. So, those requirements have to be satisfied and continuity plates are always required except in the input connections.

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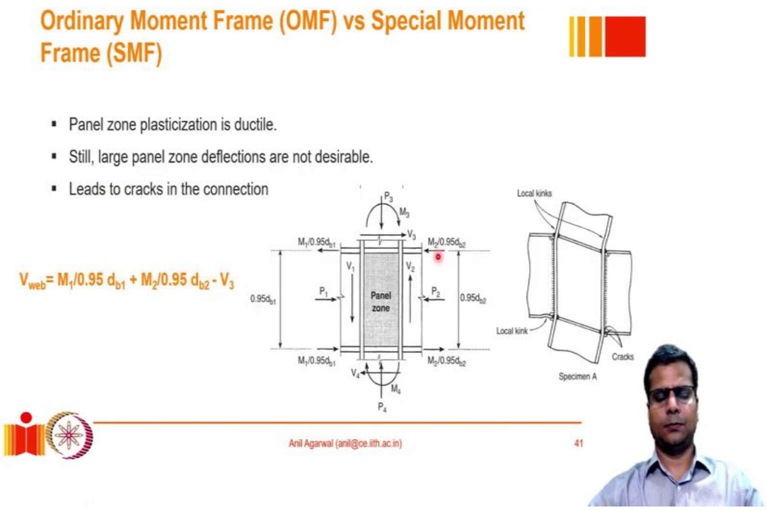

So, when designing a panel zone, we have to calculate the shear force demand on the panel zone and then design it for that kind of a shear force. So, because we want to avoid large panels on deflections for two reasons, a very large deflection in the panel zone will make the column non-straight and create instability in the column that would be very risky for the structure.

Plus also because of the large deformations right at the tip of the weld here we can expect tracking to start and in order to avoid that we should avoid excessive rotation of the panel how do we calculate the shear force demand in the panel. So, the guideline could be straight forward basically we know the moment capacity of each beam on each side of the panel and we are assuming that this beam is going to yield a plastic size under high bending moment demands.

So, therefore we know the moment capacity of each of these beams M 1 and M 2 what we do is we take this moment capacity and we divide it with the lever arm. So, that we; can get the tension force demand and the compression force to match in the flanges of the beam. So, those tension and compression force demands on the flanges of the beam are calculated then those two flanges since both beams would be rotating in the same direction those flanges could be applying a force on the panel zone in the same direction.

So, we add these two forces force coming from the top line on one beam and force coming from the top end of the other beam and then we add them together in addition to that from our structural analysis we would have calculated some shear force demand in the column top. So, that shear force demand the deduct because that is the only force that is balancing or kind of acting against these two forces and will give us the net force acting at the top end of the panel.

The same force must be acting at the bottom edge of the panel in order to resist it. So, basically that gives us the shear force demand and then we need to take the cross-section area and design for that shear force demand this is the expression that I have just explained to you to calculate the shear forcing in the column panel.

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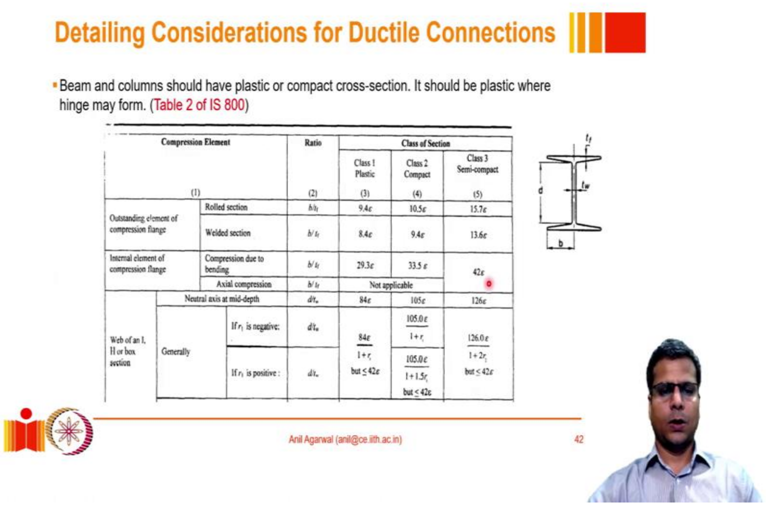

Compression Element (1)			Ratio (2)	Class 1 Plastic (3)	Class 2 Compact (4)	Class 3 Semi-compact (5)
Outstanding element of compression flange	Rolled section		b/t_f	9.4ε	10.5ε	15.7ε
	Welded section		b/t_f	8.4ε	9.4ε	13.6ε
Internal element of compression flange	Compression due to bending		b/t_f	29.3ε	33.5ε	42ε
	Axial compression		b/t_f	Not applicable		
Web of an I, H or box section	Neutral axis at mid-depth		d/t_w	84ε	105ε	126ε
	Generally	If r_1 is negative:	d/t_w	$\frac{84\varepsilon}{1+r_1}$ but $\leq 42\varepsilon$	$\frac{105.0\varepsilon}{1+r_1}$	$\frac{126.0\varepsilon}{1+2r_1}$ but $\leq 42\varepsilon$
		If r_1 is positive:	d/t_w		$\frac{105.0\varepsilon}{1+1.5r_1}$ but $\leq 42\varepsilon$	

In addition to these there is also there are also a couple of other requirements one such requirement is the use of plastic type of slender cross sections. So, you might recall that whenever we wanted to avoid local buckling of the cross section we were required to use a semi compact or compact cross section based on different provisions. We were never using until we talked about earthquake engineering we never used plastic cross sections we never discussed this column.

So, this column is relevant when it comes to earthquake performance of the connection. So, when we are designing a member for which is a part of our resisting frame if it is near a region where we are not expecting anything to form. So, for example all the FCEs they can be designed to have a compact class classification. They do not have to be very stimulant compared to class cross sections are sufficient however in the regions where we are expecting the cross section to undergo plastic deformations which is basically the DCE's elements there we cannot have compact we have to have plastic classification.

So, you may see that plastic classification is even more estimated meaning that each plate in a hot roll section for example has to be thicker. So, b / t ratio is smaller in comparison to compact section that means the plates have to be thicker that is basically what it means is that after a plastic classification ensures that even after a very large rotation or very large strain and that in cyclic conditions the cross section does not undergo buckling easily. And it can accommodate a lot of compressive and tensile plasticization over and over again.

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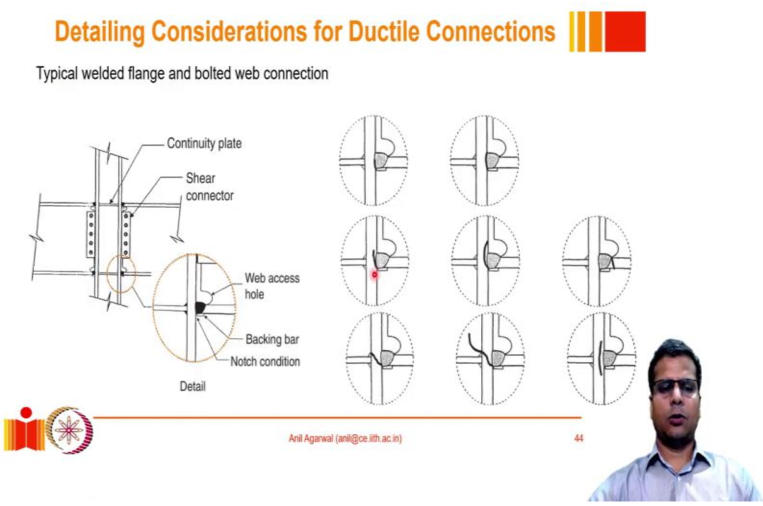

Now we will talk about some of the detailing aspects of rigid connections. So, when we want the rigid connection to be ductile we have to make sure that there are no risks of any fractures or premature fractures to happen in the connection. Wherever we have bolted connections typically fractures can be controlled and we can avoid the moves in which a certain fracture can happen. However welds are relatively unpredictable in comparison to bolts.

And the challenges are often lie in the workmanship and some of the design provisions or some of the guidelines have been developed to avoid a premature failure or premature fracture of a weld. So, there are very such considerations which avoid any defect or any kind of a inclusion in the weld which can trigger an early crack in a way in we have to take all those precautions.

In addition to that you might see that for example in this situation we may be required for us to be able to weld or groove this bottom flange to the column we require a backing bar or a backing strip to be placed beneath the flange so, that this material can be deposited in that location. Now the interface between the backing bar and the column flange which is partially only partially fused together.

That interface becomes the point of stress concentration and that here the crack can start and many such possible cracks have been documented in the literature and listed here. So, this is a different type of attack which is not starting from there but this is starting only because of some problems with the hydrogen inclusion in the welding process. But in most of the other cases, the crack starts from that interface between the backing strip and the flange buttom flange.

So, in all situations we have to be extra careful. Generally the requirement is that we should back guard. So, we should remove this backing bar after the valve is done and provide a smooth surface here and the access hole that needs to be provided. So, that we can provide a weld a continuous weld through the entire width of the flange sometimes we may introduce a defect or a sharp corner here that may give rise to the possibility of fracture in the bottom flange.

And even that surface and even the geometry of this access hole is very strictly has to be adhered to so that there are no sharp surfaces or sharp corners in the entire geometry for us to avoid such fractures. So, with this slide I am concluding this course I was able to cover topics such as basics of welding and bolted connections.

Then the; design of both groups and run groups then designing of frame members which are non-ductile detailed and then some guiding principles for the design of ductile connections. Now the handicap here is that in the Indian code there are no detailed guidelines for the designing and detailing of the ply connections yet one code is under development. So, soon it will be published and we would have more strict and uniform guidelines for such connections.

Until then we would have to follow some of the international standards or the basic principles of fundamental principles to design ductile connections or the time frames. So, I am thankful for all the audience for all the participants in the course. I look forward to offering this course again this is the time to conclude, thank you.