Design of Connections in Steel Structures
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Lecture - 22
Ductile Detailed Beam-Column Connections-1

So, in this lecture we will talk about connections in ductile frames. We require ductile frames especially in seismic conditions when we design a structure for seismic loading conditions the seismic loads are typically so, high and they exert such a large force demands on the structures that it is unreasonable to design them for elastic behaviour because if we want to behave a structure elastically under the most heavier earthquake that can expect in its life.

Then most likely the structure will become so heavy and bulky then it will be unusable. So, for typical structures we accept some level of damage but the idea behind the ductile design of a structure is. So, that it can accommodate the damage without creating overall instability in the structure. So, structure should remain stable structure should help prevent life loss and also be able to resist the load from the earthquake.

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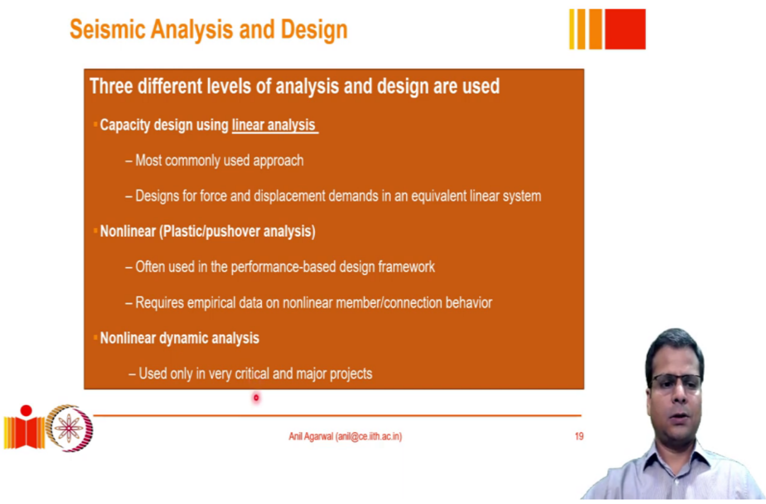

So, that is the typical philosophy there are three different levels of analysis design methodologies that can be employed. So, the one that is most commonly used by structural designers for everyday structure including high-rise structures too; is the one that is known as capacity design using linear analysis. So, this is also sometimes referred to as the response spectrum based analysis or respect response spectrum based design.

So, this is the most commonly used approach and the idea here is that we even though even though we expect the system to behave non-linearly we expect the system the structural system to form hinges we design the components individual components only up to elastic limit and then we use some kind of scaling factors to calculate the actual performance of the structure. So, in this approach the advantages are that these are this is a simpler approach it does not require.

So, much of computational power because of the analysis is all linear and we do not need very accurate constitutive relationships and connection behaviour etc. Whatever models easily available for connections can be utilized because the analysis generally remains elastic. The disadvantages of these methods of linear analysis based on capacity design based on linear analysis the advantages the disadvantages of this method are that it cannot first of all predict the exact behaviour.

So, we do not know exactly which member or which part of the structure is likely to fail first. Also we have no idea about the expected behaviour of the structure in an earthquake we try to make sure that the structure remains safe but that is all that the information we have we cannot modulate the design based on a particular type of earthquake. So, there was a design philosophy which is known as performance based design.

Wherein the idea is that the structure should be designed to perform in a certain way under a certain magnitude earthquake that kind of objective cannot be fulfilled with this approach the approach that can be utilized to fulfill the objective of a performance based design would require some level of nonlinear analysis. So, the one the commonly used method in object oriented performance based design approach is known as the non-linear pushover type of analysis.

In this analysis we do not do analysis only up to the linear range we understand that the structure is going to go into plastic range we have to model the portions of the structure which are likely to undergo plasticization to as much accuracy as we can and that is where the rule of our models the empirical models that we adapt for. So, let us say for a column if it is going to form a hinge or if it is going to form a hinge in a beam we would require the MC moment curvature relationship for that hinge or moment rotation relationship for that hinge.

If our moment relationship is not accurate then our model prediction will not be accurate. This approach could be utilized to assess the overall performance of the structure for a given earthquake. So, typically, you might have heard of the terminology like life expectancy, life safety, immediate occupancy, and so on performance objectives. So, such kind of design philosophy can be employed or can be met using this method.

The third and the most complex one is known as is the one that is known as non-linear dynamic analysis. So, in the second one even though we were accounting for the plasticization of the structure we were not really accurately accounting for the non-linear the dynamic aspect the time variable aspect of the earthquake. So, in a push-over analysis we keep pushing in one direction in monotonic way which tends to simulate the most severe earthquake but does not really simulate the properly.

In a non-linear dynamic analysis, we take into account the inertial aspects of the building and also the time variability of the forces. And as a result this is the most accurate method to predict the performance of the structure and based on this we can actually learn individual components but the challenge here is that having the exact information about the earthquake loading and also about the constitutive relationship of different components of the structure is very challenging.

And if we make a mistake, the results could be very misleading. So, typically for most of the design projects engineers especially in India. So, far mostly employ this technique which is known as linear analysis based on capacity design.

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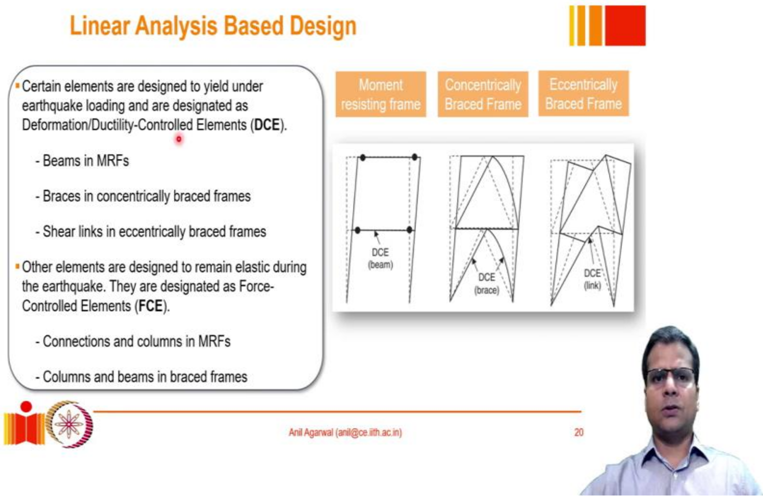

So, we will in this course we will focus our attention only on this linear analysis based design and for this design to understand first we have to understand a few terminologies one of those is the distinction between DCE members in a structure and FCE members in this structure. So, whenever we have a structural system which is supposed to resist the lateral loads not every frame in a steel structure resists the lateral load some frames resist and some frame they are all considered as leaning frames they do not resist the lateral loads they only resist the gravity loads.

So, we will talk only about the frames designed to resist the lateral load. So, there could be an example such as moment resisting frame or a concentrically based frame this is an example of a concentrically base frame where these bases are meeting the beam concentrically there is also an alternative type of base frame which is known as eccentrically based frame which is which looks like this.

And the undeformed configuration this broken lines show the undeformed configuration. So, the these two bases do not meet at the same point to the beam they meet with some offset and that offset creates an eccentricity which actually creates a shear limb. So, in different systems, different components are expected to undergo plasticization.

And those components which are expected to undergo plasticization would be classified as ductility control elements and they are the ones who would control the overall ductile behaviour of the phone. So, these components should exhibit a very significant amount of plasticity this very significant amount of ductility they should not break very easily they should continue to deform of offering resistance for a very large extent.

So, in a moment resisting frame for example the beam ends not inside the column outside the column but ends of the beam this is these are the locations which are considered to behave as a hinge but with a significant amount of moment resistance. And they should form a hinge which will absorb most of the or dissipate most of the energy from the article. In a concentrically brazed fringe typically braces are expected to behave that way the braces are expected to plasticize and absorb that energy.

In an eccentrically based frame the bases and the beams and the column are supposed to remain linear they are not supposed to plasticize it is the shear link it is the portion of the beam which is loaded eccentrically that is expected to plasticize and absorb most of them or dissipate most of the energy that is coming into the system. So, these components which are supposed to behave take plastic deformations they are known as ductility controlled elements.

So, they are listed here and then there is another set of elements: surrounding elements that apply those loads to the DCEs known as FCEs force controlled elements. And the idea here is that we should design the DCEs for the expected deformation demand and we should design the FCEs for the expected force demand. So, this is the major classification major distinction between two different types of elements.

In a more in a later load resistance system that we should identify and then we should go how about with designing. For example, in a moment resisting frame, we cannot make a column as a DCE because if the column forms a hinge that can lead to a significant destabilization of the system. So, we usually do not prefer the column to develop a hinge therefore the DCE would be the beam ends and FCE would be the column and the column being joined.

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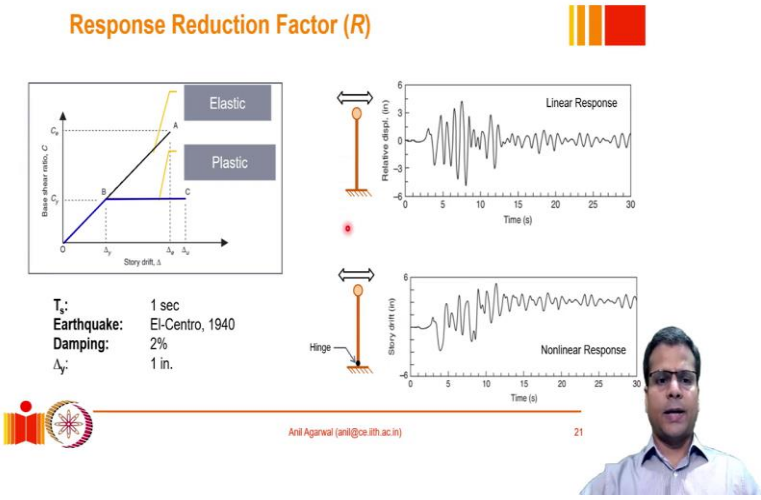

There are a few more terms that I think are important that I explained in the beginning. So, that we understand the design methodology in detail. So, here I am trying to emphasize the difference in the dynamic behaviour of a linear system as compared to a non-linear system. So, basically we took a pendulum the pendulum had a natural frequency of one hertz that is the natural time period was one.

Second it was single degree of freedom system it was subjected to an alcentro type of earthquake loading this is basically a time history, acceleration time history which is typically used quite commonly. The system was assumed to have a 2% internal damping and this type of linear system was subjected to the central earthquake this is a specific time history of acceleration at the base.

And then the response at the top the displacement response at the top was observed. So, this is the displacement response which is relative displacement with respect to time. So, we can see that maximum displacement or maximum drift with respect to the base is calculated as something like almost four and a half to five inches. Then we change the system in such a way so, that it does not remain elastic that is its natural time period is one second in the elastic zone in the elastic range.

But after a displacement of one inch marked here as delta y after it reaches a displacement of one inch, it becomes a perfectly plastic system that means it has zero stiffness at that time. And then again the same system is subjected to earthquake loading el centro earthquake loading and again the displacement at the top is measured. So, one thing we can see is that in the linear system we were getting a net zero displacement at the end because the system was coming back to zero position after the earthquake was completed.

In a plastic system however there is a residual displacement. So, after the earthquake is complete that phase history has completed there is a leftover displacement in the system which is expected because the system has a plastic deformation which is a permanent deformation. But here in this system you might also realize that the total deformation maximum displacement that we see is almost of the order of four and a half to five inches which is not very different from what we were getting in the elastic case.

So, in the first case in the elastic, the system was only moving on this straight line. In the second case the system was moving on this line on this blue line and after the earthquake is completed you might see that there may be there is a small increase in the total deformation or maximum deformation in the system. But what is more noteworthy is that the maximum force demand is much smaller in comparison to an elastic system.

So, in the elastic system the maximum force demand would have been almost equal to C e because that was the maximum because the system was always travelling on this path on that diagonal line. But in a plastic or elastic plastic system, the maximum force is limited by C y, which is the maximum force this system can resist when the yielding happens and the system plasticizes.

So, we can understand that there is a small change with respect to the original elastic analysis because of the presence of stability in the system in addition to that also there is the effect of redundancy in the system because of that there is a there might be a small change in the deflection but there is a very significant reduction in the maximum force demand.

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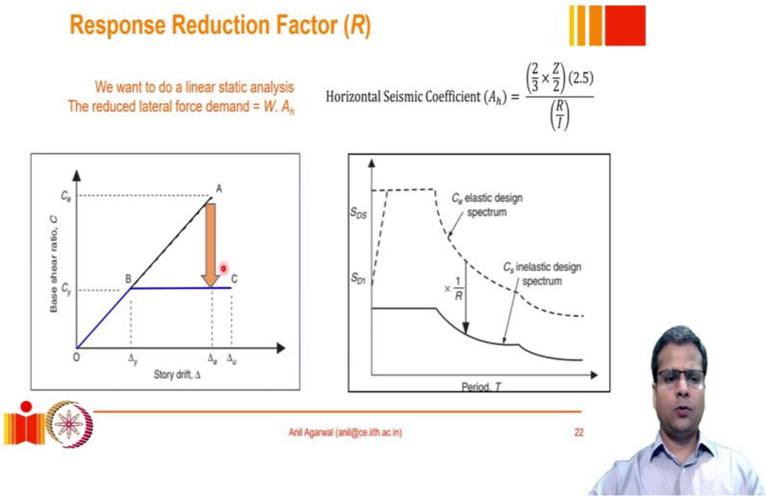

So, how do we do in the; how do we handle this change in response the maximum force response in the design provisions? If you go to IS1893 you know part one you would see that there is a response spectrum design response spectrum given in the code which looks somewhat like this and then that is the expression for the seismic coefficient which is basically an acceleration term that includes a few terms here.

I am not going to go into their detail, but there is an R value in the denominator, which depends on the system's ductility. So, if the system has a more ductility the R value would be larger and essentially what it says is that if you have a very large R value you reduce or you scale this spectrum value with this R value and then you get a new response spectrum and you use this response spectrum for your design applications which basically accounts for this reduction in response.

Now obviously if my system is more ductile. So, for example my system can accommodate much greater levels of plastic deformations before it fails that means it can go all the way up to here in such a scenario we would have or we can utilize a response reduction up to this level. Whereas if the system is not very ductile and maximum we can utilize up to this much of yielding or this much of permanent deformation my response reduction would be only this much.

So, this factor with respect to this forces the elastic force that we will calculate here or this factor with respect to the elastic force that we will calculate here. So, this response R factor will be very large compared to this R, which is definitely a function on how much ductility is available in the system.

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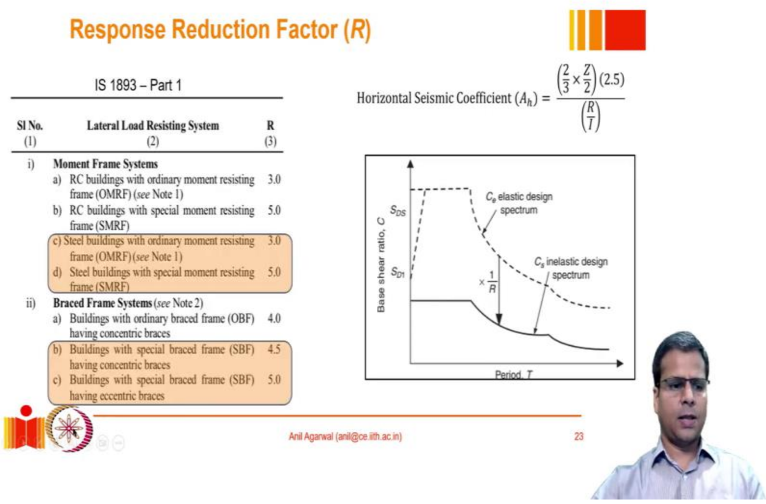

So, in this slide we have listed the values of response reduction factors for some of the steel structures. So, in the first rho c and rho d you can see there are ordinary moment resisting frames and SMRF is a special moment resistance both of them are moment resisting frames we have discussed how many resistance frame works. So, for them the response reduction factor is taken as three and five respectively.

And in case of braced frames so, this is a braced frame with concentric braces and this is for base frame with eccentric bases the response reduction factors are 4.5 and 5 respectively. So, these are the typical magnitudes of response reduction sectors the systems that are not meant to be very ductile the response reduction factor could be less than 3 also.

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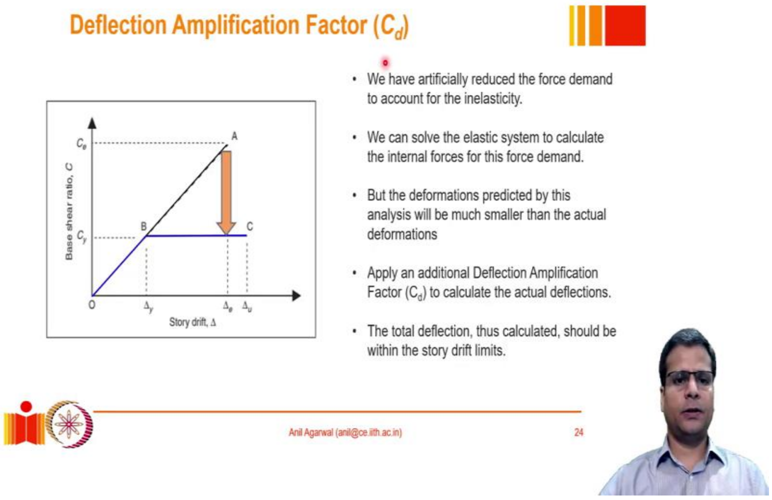

Another factor of interest would be the deflection amplification factor which is also associated with the same process. So, as we have discussed the for the linear system if the system were to behave linearly and the full load was applied on the system the actual response would have been up to this point, point a and the corresponding force demand would have been C e and the corresponding displacement demand would have been delta e.

However a linear elastic plastic system behaves in such a way and we find that the maximum force demand is only C y and maximum displacement would be delta u. Now comes the question of how do we map these into this analysis. So, what we do is that we do analysis only for this much of load up to point b and we do a linear analysis and for that what we do is actually we do the linear analysis then we scale it down in such a way.

So, that we get the response for this point, point b. So, that that way we get the force demand measured somewhat accurately instead of C e we measure the actual force domain for the elastic plastic system however when we do the linear analysis up to point b what we actually get is the displacement demand of only delta y which is absolutely incorrect because the actual displacement is likely to be delta u and not delta y.

So, if we amplify it again with the R factor the displacement we will get delta e which may or may not be accurate or may not be exactly same as delta u. So, a factor called C_d can be used to amplify the displacements again to get an accurate estimate of the actual deflections. So, that factor is called C_d. So, for every given system let us say we are talking about moment resisting frame of a special level of ductility.

Then for such type of a frame there will be a particular R value given to us which will correspond to this reduction from c_e to c_y then there will be a particular C_d value given to us which would correspond to the amplification of reflections when we move from (delta y) to (delta u) which is actually the real deflections. So, we can calculate the reflections using this method.

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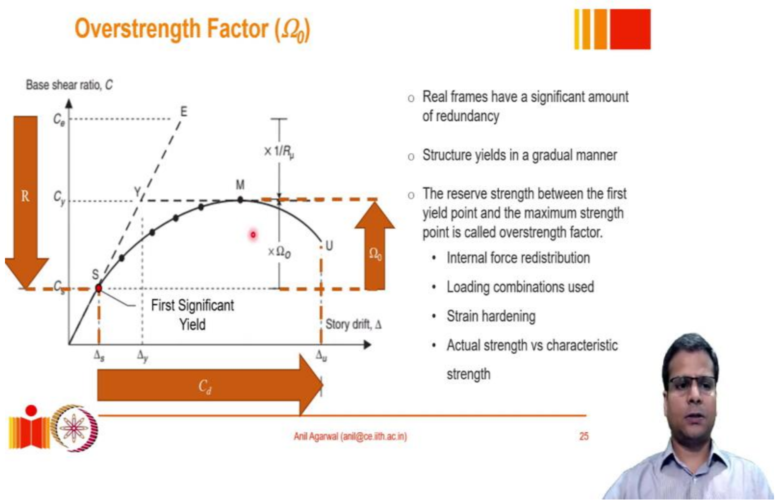

There is another factor known as the over strength factor. Constant factor is critical for the design of the components which are meant to be force critical or which are meant to be force demand different right. So, you remember there were two types of systems one was the DCE type of a system and the other one was the force FCE type of system. So, the overstream factor becomes critical for the FCE type members.

So, basically FCE members were not supposed to plasticize or not supposed to undergo plastic deformations during the earthquake loading. So, the all the plasticization has to happen in the DCE type of systems. So, how do we ensure that the members which are not supposed to plasticize do not plasticize while the elements which are BCE are plasticizing. For that we have to account for a couple of factors.

One is that the first price is plasticization starts and then the stiffness of the system starts to decrease and then the system reaches its maximum value of or maximum force resistance this is the behaviour of an FCE system. So, the DCE system which is transferring the same force to the FCE sorry this is the response of the DCE system and the FCE system which is elastic system that is transferring the force to the DCE component should not plasticize anywhere from here to here.

So, we do the basic analysis elastic analysis to get the demands up to this point. Then in the previous example we had assumed that the response is like this like a straight line but that was for a linearly elastic perfectly plastic type of a model. Still, most of the structural systems have quite a bit of strain hardening the system hardening could be the material level. Strain harness it could be because of the MF curve even for elastic plastic material properties.

The moment curvature curve itself has some level of gradual increase in the moment until the entire cross-section plasticizes plus in a structure itself not all components will plasticize at once. So, one has some components in plasticize but some other components in the same structure which are supposed to plasticize good plasticize after some time right and therefore the force demand in the FCE system may keep increasing.

So, in addition to that there is also one more factor which is because of the difference in the characteristic strength of the material versus the expected distance of the material. So, for example if there is a DCE system a hinge which is supposed to form and we know that it forms at 250 mph strength because that is the strength given for the material. But it may so, happen that that material instead of 250 has a yield strength of 270 expected yield strength of 270 MPa.

So, characteristic distance might still be 250 but if that is the case we have to be careful that if this member is going to plasticize it is going to take or absorb lot more force a lot more movement before it plasticizes which means that the neighbouring members which are transferring that force but they are not supposed to plasticize should be should be able to remain elastic for a much greater force.

So, all those factors that I just mentioned are taken care of by this component or factor known as the over strength factor. So, over strength factor so, basically we do our analysis we get this demand elastic linear elastic demand we design our components but then our DCE components but for FCE components we have to again amplify the domain. So, we can calculate the force demand which would account for all this reserved strength in the member available after the first year in the structure.

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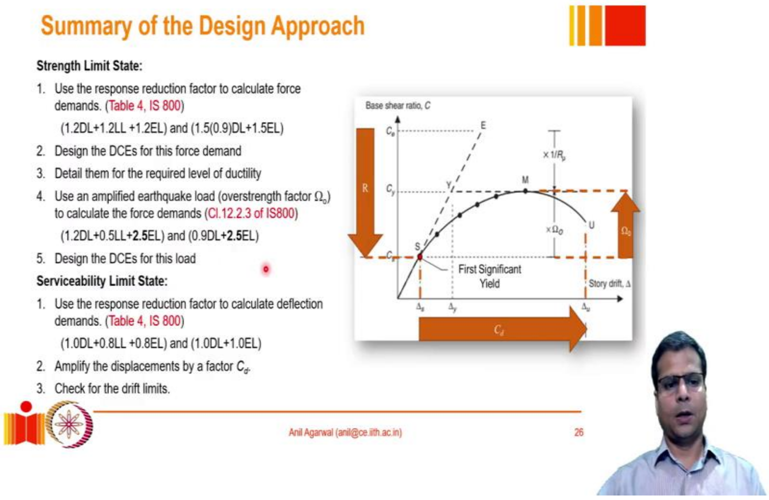

So, here I am summarizing the design approach the name few steps. So, basically the idea is first we have to do two step designing one is the strength limit state and the other one is serviceability limit step for earthquakes. So, in the strength limit state for earthquakes, we use the response reduction factor. And we do the modal analysis or we can calculate the time period and then we can use the response spectrum to calculate the earthquake force demand.

For that active force demand we use one of these three combinations if you can see closely one first combination is dead load 1.2 times live load 1.1 times and 1.2 times these are given in is 800 and the other combination is 1.5 times like that load and 1.5 times the earthquake load this time we do not take any live load and another combination would be when we reduce the amount of dead load from one it is 0.90 + 1.5.

So, for these loads we do a linear analysis here in the earthquake load we have already reduced the earthquake load for the response reduction factor accounting for the ductility of the system. Now whatever force demands we get in the components which are supposed to be DCEs we design them for this much of forced demand. Subsequently we have to make sure that these components behave tactilely, which is commensurate with the response reduction factor we have taken.

Therefore that can be ensured by appropriate detailing. So, for example if it is a beam end then it should be able to develop sufficient plastic hinge meaning that it should be able to have sufficient rotational capacity before it starts to lose its flexural strength. So, for that certain compactness ratios are required in the cross section that we have to meet. So, such kind of detailing requirements have to be satisfied when once we do that then we start designing other components.

So, DCE can be designed for the reduced forces and by ensuring enough activity is available. Subsequently we go for designing of the FCEs which are force driven members. For their design we use different set of load cases these load cases are given in clause number 12.2.3 of IS800 which are different from the ones that are given in general cases on table 4.

So, you might see that the earthquake loading has been amplified in these cases. This amplification is the omega factor, the over-strength factor, which basically accounts for all the reserve and the different types of preserved strength in the structure after the first yielding of one of the DCEs. Then the FCEs are designed for this load. I am sorry this is incorrect this should be you should read this FCE.

So DCE are designed for this load and FCE is our design for these loads that is how the strength limit state can be satisfied. Now when it comes to serviceability limit state the load combinations that are given in is800 for serviceability limit state are these. So, one limits one load combination includes dead load plus 0.8 live load works 0.8 earthquake load the other cloud competition that is given is one dead load plus 1.30 load.

So, for both of these we calculate the demand in the displacement demand in the structure by doing a linear analysis here the earthquake load again is the one that is reduced for the response reduction. So, this is because of the response reduction this is the analysis. Subsequently, whatever displacements we get in the system we amplify those displacements by a factor of C_d which basically accounts for this the amount of ductility that is available in the system.

We account for that by using this vector C d and then we calculate new deflections. So, if these deflections are greater than the deflections we had calculated from the analysis, then we ensure that our drift limits are satisfied. However this particular part is not exactly done the same way as for the current Indian standard code IS800 the C d factor is not explicitly mentioned and I will talk about that in a minute.

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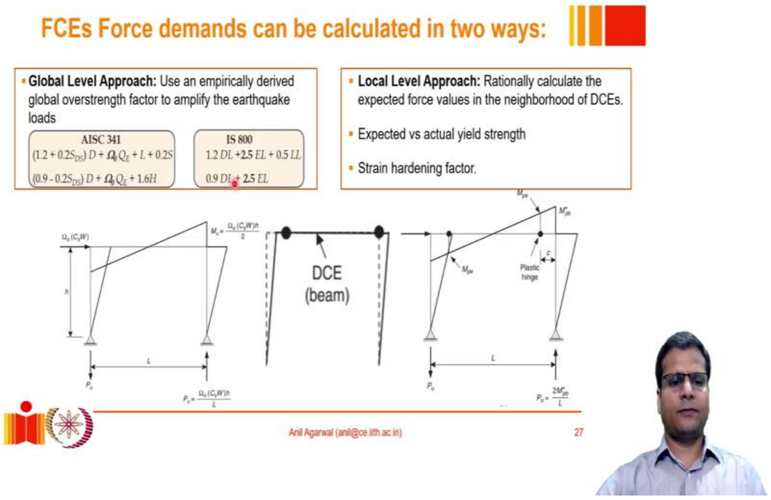

So, as we had just discussed the fourth demand in FCE has to be calculated by amplifying the earthquake force by a factor of over strength factor. So, the American standard code gives you explicit values of over system factor (Ω_0) these are these (Ω _0) values are different for different types of structural systems for we will talk about the values in a minute. So, that respective (Ω _0) values have to be used.

In the Indian code however the over strength factor in is 800 the over strength factor is not mentioned explicitly it is present in the form of this vector 2.5 which is a constant value and for all different types of structural systems the same value needs to be utilized. There is another approach to calculate the force demand on the f_c is so for example if we are designing a column in a moment registering frame the column would be an FCE member.

Now one way to do that is to take this lump sum value of 2.5 and assume that there is an over strength of 2.5 available and whatever force domain will get in the amplify that with a factor of 2.5 and that would be the force demand on the column. However the other approach could be to actually look closely into this system and see how much actual when this member when this beam is plasticizing it would plasticize at exactly this much of moment and these are the different forces acting on the member.

And we can go into the detail and try to find out how much force it would be resisting when all that is when this plasticization is happening and in that process we need to account for all the different types of strain hardening and other unique points and everything. So, this type of analysis can also be done but this is more cumbersome and usually it is not done as you can see sometimes some empirical equations are available.

Though the equations could be used instead of these factors but typically people use these factors directly.

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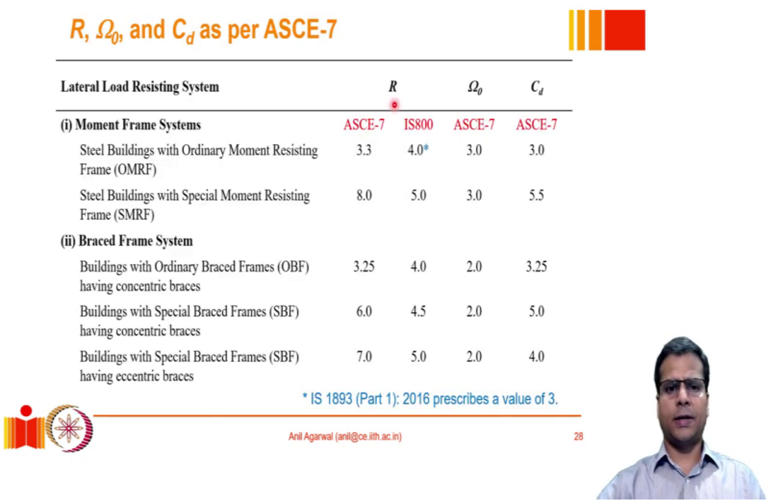

The three coefficients that we just talked about the response reduction factor the over strength factor and the deflection amplification factor their values I have taken here in this in this table I have listed their values from the different codes. So, here I have taken the values given in the American Society of Civil Engineers ASCE-7 document and also here I have listed the values of R as per IS800 for different types of structural systems.

These are for moment resistance and based on frame systems and you can see that there is quite a bit of variability the American code is more ambitious in a way or less conservative. So, that they use the response reduction factor of almost 8. So, they have a lot of confidence in the ductile behaviour of SMRF or SBF systems and use a very large response reduction factor.

In the Indian code we are a bit more conservative and we do not reduce the demand by such a large magnitude we reduce it by a factor of five minutes as I had mentioned before the Indian code does not explicitly mention the over strength factor anywhere but it gives the constant value of 2.5 for all different types of file systems. The American code gives a western factor of 3 for moment resisting frame and an over strength factor of 2 for the base trends.

The displacement amplification factor we will talk about in a minute also one might know that there are different codes in India they do not necessarily reconcile with each other. So, the response reduction factor that is given in IS800 for an ordinary moment resisting frame and what is the meaning of ordinary we will talk about in a few minutes. An ordinary moment adjusting frame the response reduction factor as per Is1893 4.0 whereas for the same system IS1893 through part one which controls the design force the earthquake force demand calculations prescribes a value of 3.0.

So, there is a bit of discrepancy between different codes and it could be anybody's guess to follow one of the two gaming codes.

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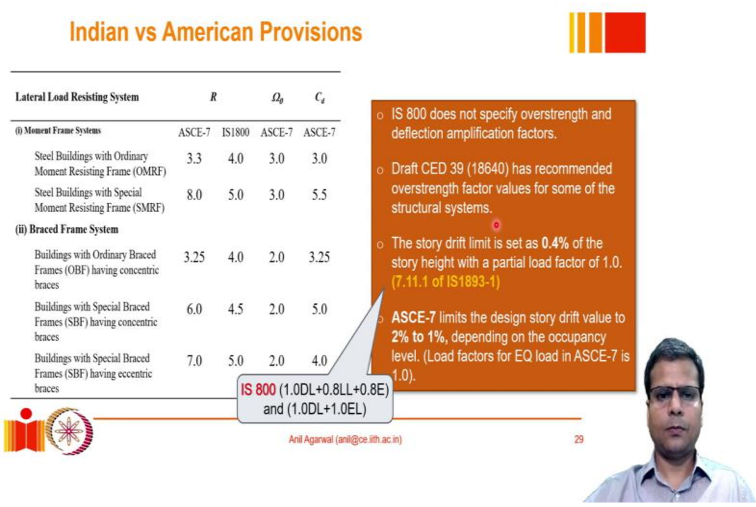

So, as we have mentioned before the IS800 does not explicitly mention over strength factor and the deflection amplification factor however it does mention the response reduction factor R. So, this is zero and CED factors are missing in the IS800 however they are implicitly incorporated which we have seen some glimpses of it there is also another document which is known as draft CED 39 18640 which is basically an equivalent of IS13920 which would give the descriptions for ductile detailing of steel structures for earthquake applications.

So, this document does explicitly mention the over strength factor right. So, the values in this document are slightly different from those mentioned in IS800 and if this document comes into force, I believe that should supersede the values mentioned in 800.. So, IS800 does not mention the western factor but as we have seen before, it uses some load combinations to design FCE type of components.

And those amplification factors are basically nothing but the load factors are nothing but the over strength factors. For the limit state of serviceability since I had mentioned that C d value is actually used to amplify the deflections. So, first we calculate the deflections using elastic analysis for the reduced response including the response reduction factor then we amplify the deflections by the C d factor but the Indian code IS800 does not explicitly mention the seeding factor.

So, then how does it deal with the change in amplification what it does is that it actually sets a very low limit on the drift values which in a way is same as saying that that if the C d factor is applied then the deflections limit would be slightly higher. So, if I go to IS1893 part one clause number 7.0 11.1 it mentions that the story drift limit is set as 0.4% of the story height which is relatively small in comparison to if we compare it with ASC-7 that is American code it says that the story drift limit would be between 2 and 1 depending on the occupancy level.

So, in ASCE-7 you are required to first calculate the deflections under the elastic analysis which is under the reduced response then you amplify the deflections with this C d factor multiply all the deflections with this C d factor that will give you the final deflections or real deflections. Those real deflections should be within two percent to one percent of the story height. The Indian code it does not apply that amplification factor deflection amplification factor.

Whatever we got from the analysis it simply says that that value should be only 0.4 of the story height. So, it may appear to be more stringent but it is not it is basically the ratio between 2% and 0.4% could be designated as the deflection amplification factor the only difference between this code the Indian code and the ASCE-7 is that ASCE-7 gives different values of C d for different types of framing systems whereas the Indian code gives a constant value because it gives a single it sets a single lower limit for deflections.

Now here one thing we must be careful about is that these positions are taken from IS1893, IS800 gives different load combinations for the serviceability limit. So, IS1893 says that this 0.4% should be done while considering a load case where the partial load factor applied to the earthquake load is 1.0. So, we can interpret that statement as if we should take the load combination of 1.0 times dead load and 1.0 times earthquake load.

However there is another load case which is 1.0 times that load plus 0.8 times likelihood plus 0.8 times the earthquake load which is mentioned in IS800 and for that what should be the deflection limit that is not mentioned. So, to go by intuition or to go by general logic we can say that for both these combinations let us check for the deflection limit of 0.4%.

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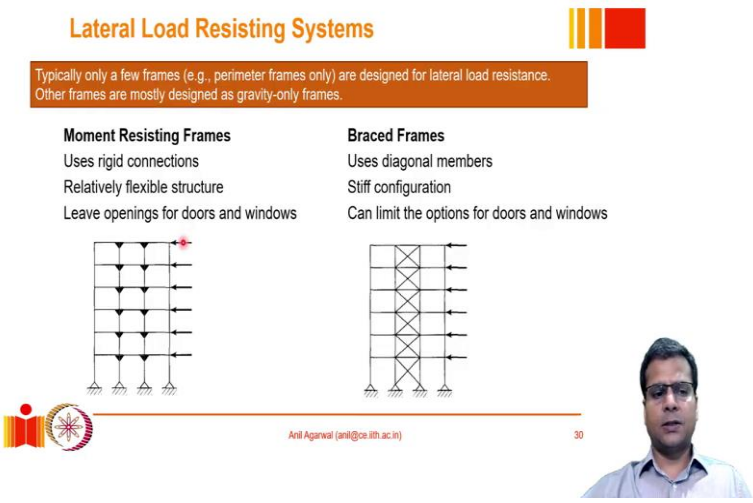

Typical load resisting systems we had discussed very early in this lecture. One type of system is the moment resisting frame system, and the other is the best. Now the two systems are inherently different. In most steel structures, we usually have only a few frames designated to resist the lateral loads. The rest of the frames are typically designed only to carry the gravity loads and deflect as the latter load resistance system deflects.

So, in a typical frame only a few of the frames are designed to resist the lateral loads the rest of the frames are considered as moving frames and they only resist the gravity loads and they lean or they deflect as the lateral load resisting system letter load resisting frames deflect. So, in a moment resisting frame system the connections between the beam and the column are made rigid. So, that it has a lateral load resisting capacity.

In a base frame system, however, we do not make rigid connections between the beams and the columns but provide concentric braces like the ones shown here and there are different varieties of such frames such places which can be provided. One we had seen before was basically a chevron kind of brace where the base is meeting in the middle of the beam and the two braces are meeting in the middle.

There can be another variety wherein you may have a single brace on each side and two base may be covered this is kind of a x-basin. For most lateral load resistant applications, we avoid any k type of bracing wherein we meet the bases in the middle of the column. It is to meet at the middle of the beam because being is not such a critical member for the structure's overall stability.

But having braces meeting at the middle of the column is strictly prohibited under three loading conditions. So, brace frame systems, whatever they are, act as truss members because of their diagonal members and increase the lateral load resistance capacity very significantly because they have very large action stiffness. So, these systems typically behave in a much more stiff manner and it is as compared to moment resisting frame systems.

Because moment restriction systems rely on the flexural stiffness of the beams also and if we want to increase the flexural stiffness of the beam beyond a certain limit we end up using very deep beams that may become uneconomical. So, sometimes therefore typically MRF systems are limited stimulant systems are relatively flexible. The one good thing about movement resting frame systems is that they do not have any members blocking these openings.

And therefore you have a complete freedom the designer has a complete freedom to provide windows and doors wherever they want but in based frame systems it becomes a hindrance because if we want to provide a door here because of the braces that are passing through at this cross-section. Providing a door or a window here may be very difficult or may not be possible.