Design of Masonry Structures
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Module - 05
Lecture - 40
Special Topics – Assessment of Existing Masonry Structures Part – III

Good morning, this is our last lecture on the remaining portion which is on the special topic of assessment of existing Masonry Structures. Let me continue what we were talking about in the last class.

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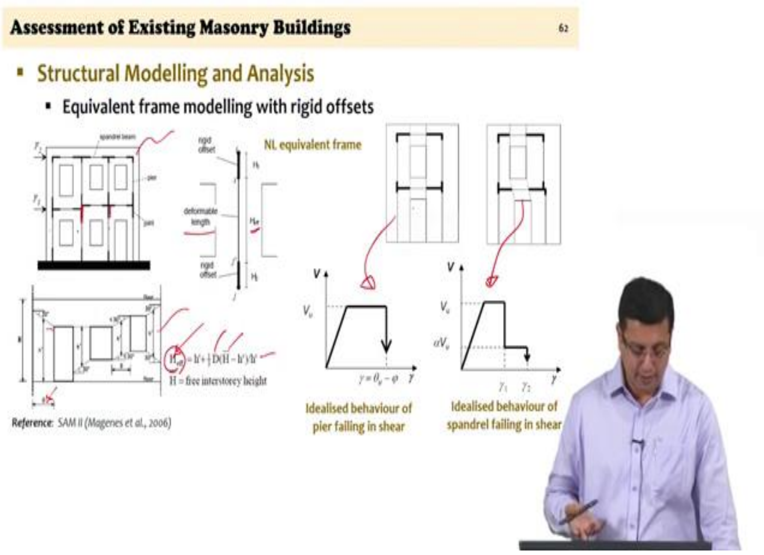

We started looking at this aspect of structural modeling and analysis. And I was basically introducing to you the different approaches that are appropriate considering the specific lateral load mechanisms of resistance that load bearing masonry constructions typically have.

So, we were talking about possible modeling approaches and one of the more popular and appropriate approaches for modeling and analysis, particularly seismic analysis of masonry constructions, is what is referred to as equivalent frame modeling where the masonry construction is considered as an equivalent frame.

You make necessary modifications to consider the load bearing masonry wall as an equivalent frame, a combination of horizontally aligned elements which are the spandrels. And the vertically aligned elements which are the piers and so similar to what a moment resisting frame would actually do to resist lateral forces.

We are looking at a masonry load bearing construction as being an equivalent frame. Now, you can see that there is an additional phrase here with rigid offsets and we will examine the reason why we need to have rigid offsets when we consider equivalent frame modeling particularly with masonry constructions.

So, what is happening here is you can look at a masonry wall that is shown in the figure at the top. This masonry wall is composed of the vertical load resisting elements which are which also are the gravity load carrying elements the piers and then you have across the openings spandrels which span and act as couplers between the piers. So, what is actually being done in equivalent frame modeling is that the piers is, the vertical piers are being considered similar to columns and these spandrels are being considered as similar to beams.

And then you have beam column joins like in a frame building and those are the corners between the piers and the spandrels. So, this sort of an equivalent frame idealization of a masonry wall is done and then you attribute properties of the pier and of the spandrel to the frame elements which may be the column element or the beam element. And the beam column joined typically you know is, in a moment resisting for your beam column joint in a steel or a concrete frame is a rigid node.

Similarly, here because of lower deformations at that joint between the pier and the spandrel you will have to constrain the deformations and possibly model it as a rigid joint as well. So, we will come back to that in a moment, so if you look at how do you arrive at the equivalent frame for a given wall; a wall may have openings of different sizes. You will have door openings, you can have ventilator openings, you can have window openings they could be of different sizes.

And we have seen that the shear resistance of a masonry pier can be in a blank wall. When you look at a shear wall with no openings in a shear wall with openings where the perforation is determined what is the height of the deformable shear element and what is the width of the deformable shear element.

So, if you look at the sketch below this is a sort of a guideline that helps us to establish what is the height of the resisting pier right. So, you can see that in this figure there are three different openings, there are two windows and one door opening of different sizes right. Each of them is of a different size, what then happens to the resisting masonry between these piers.

If I were to model them as equivalent columns what should be the height of that column, because the column is now the deformable resisting element between these openings. So, here is a rational procedure that is prescribed to identify what should be the height of the element itself. So, you can see that it is purely based on geometry; geometry of the opening and the distance between the edge of the wall and the opening determining what this width D is. And H' is established which is nothing but the average height considering an angle of 30 degrees subtended from the opening itself from the top of the opening itself.

And H' is nothing, but the average height between each of these openings. So, from the width of the opening the total inter storey height and the average height between the openings it is possible to arrive at what the effective height you should be considering in the equivalent frame modeling. So, basically once you establish what could be an effective deformable height of the pier you use those to assemble the equivalent frame model.

But, since these piers are all going to be possibly of different heights you need rigid joints which will then ensure connections with the horizontal elements at the locations where you have a floor diaphragm or the spandrel itself. So, you can see that the rigid joint is of different dimensions and that is what will equalize the inter storey height with respect to different sizes of piers.

So, basically as an assembly of spandrels or the beam elements, column elements of varying heights and the joint elements you are converting the wall with openings into an equivalent frame. And now this equivalent frame can be treated like you would treat a regular portal frame and carry out an analysis. But the column element and the beam elements will have cross sectional properties based on the actual cross section which is the pier cross section or the spandrel cross section.

And elastic modulus and shear modulus based on the actual material and cross section itself. That since the deformable heights could be different because of the sizes of the openings the deformable length that you see in this next figure here; the deformable length of the frame element could be different. Now, this deformable length being different, you need these rigid offsets to ensure that the inter storey height is then completely covered.

And that makes it uniform across of the different frame elements, so you see that how the different lengths of rigid offsets is then used to ensure your total frame along the height is modeled as an equivalent frame. So, what this rigid offset basically means is that the calculated H_{eff} is only a deformable part of the original pier; the rest of it because it is part of the joint is where deformations are lower. And hence it is considered as a rigid joint like you would consider a beam column joint in a regular moment resisting frame and reinforced concrete elements.

So, it is it you see the equivalence coming between what the frame modeling or a portal frame modeling would be versus a load bearing perforated masonry wall being converted into an equivalent frame. So, this is a standard procedure that is adopted and there are there are programs that are available that would assist you to do this.

Then you can assemble the entire three dimensional model of the structure in this form of equivalent frames and carry out a gravity and a lateral load analysis like you would for a regular framed structure. So, you can see how the element the equivalent frame model in the undeformed and deformed geometry would actually look you see that the rigid nodes have no deformation, the rigid joints have no deformation.

Whereas, deformation is actually happening only in the deformable length of the masonry or the deformed length of the frame itself. Now, the other important aspect to remember is the pier elements are provided with an idealized force-displacement behavior and similarly a spandrel element is also provided a non-linear force-displacement behavior.

So, each of these deformable elements the pier element in this case and the spandrel element in this case are defined with a non-linear force displacement behavior. This can also be considered as a hysteretic behavior and then cyclic response also can be carried out considering non-linear within a non-linear analysis. So, one part of it deals with how to convert the load bearing perforated shear wall into an equivalent frame.

And the other part actually requires that the masonry pier and the masonry spandrels lateral load behavior in the form of some idealized force displacement non-linear curve is available for you to be able to carry out non-linear static or even cyclic analysis for which you will require a hysteretic curve; positive and negative cycles also available. So, this is one standard approach that is adopted for masonry modeling and analysis.

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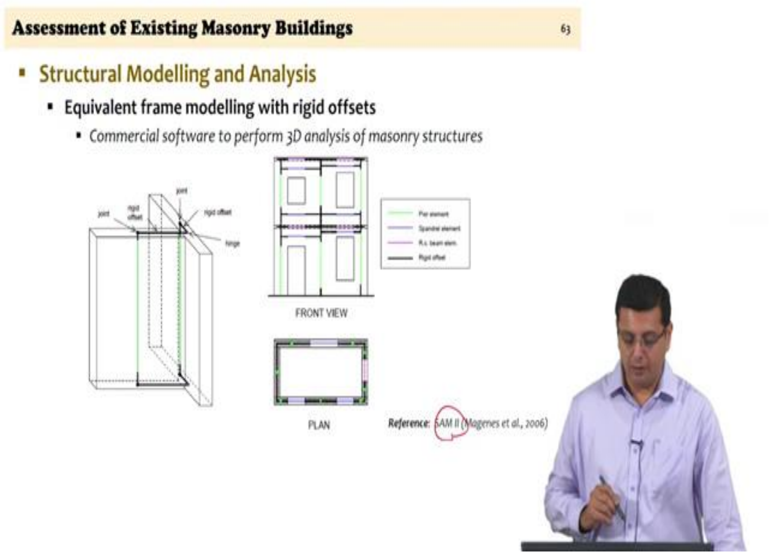

There are commercially available software to perform this, not only in 2 dimension, not only the plane analysis. But, also for 3 dimensional analysis where you can actually assemble the connections in the three dimensional model between the different planes and carry out a gravity load analysis or a static pushover analysis which is a lateral load analysis.

And get the lateral load curve or the capacity curve of the structure itself, some of the pushover analysis. Some of the analysis programs that are commercially available there is one called seismic analysis of masonry which is a commercially available program.

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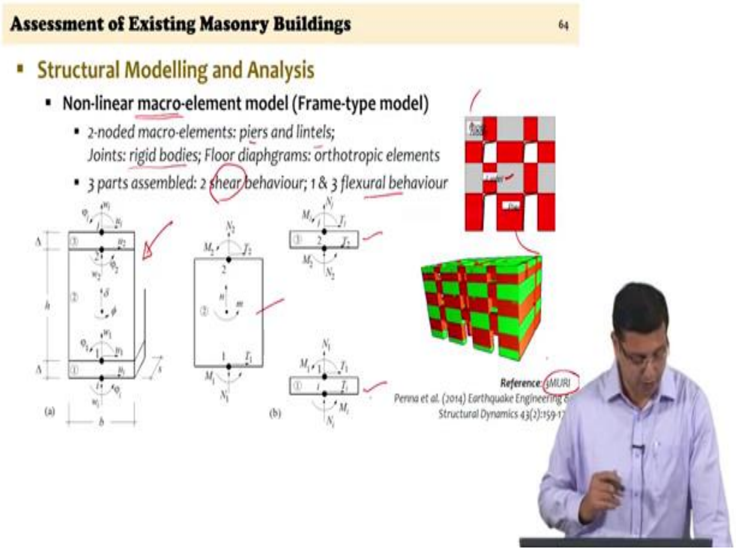

There are other approaches which are based on a similar basis; again there are commercially available software that can do this. Where you get into a slightly different class of modeling called macro element modeling where again the equivalent frame that modeling is being carried out. But, now you have the piers in the spandrels referred to as macro element and at the macro element is provided non-linear characteristics which will then help you to assemble the frame and carry out non-linear analysis.

So, in this particular case I am talking of a specific program called 3MURI which is 3 dimensional analysis of masonry walls. So, in this particular case as you can see a single masonry wall with perforations is modeled as vertical macro elements which are the piers the horizontal macro elements which are the lintels. And then the rigid node which is again like the rigid offsets you had in the previous case the part which is non deformable. And then an assembly of the rigid node, the lintels, and the piers can then give you basically a frame modeling of the perforated shear wall itself.

So, in this particular approach there are two nodded macro elements which are representing the piers and the spandrels or the lintels. And the corners as you can see the gray colored corners are the joints which are assumed to be rigid bodies this is an assumption you can have some deformation there, but it is small in comparison to what happens in the piers the spandrels. The floor diaphragms, the floor diaphragm can be considered to be rigid, but the floor can also be considered to have some deformability and different in different directions.

So, you can also model that as orthotropic elements, so this particular approach allows you to consider differences of a flexible diaphragm, semi rigid diaphragm and so on. So, the macro element looks something like this for the pier you would have an assembly of three parts. The central portion is the one that accounts for the shear behavior in the macro element, and the top and the bottom ends particularly the zones where cracking due to flexure and crushing due to flexure can happen.

Those locations are controlling the flexural behavior, so the assembly of 1, 2 and 3 elements which specific degrees of freedom required for the flexural behavior and the shear behavior would then formulate the macro element which is then what is part of the overall frame assembly itself.

So, even here by assembling 2 dimensional walls that are discretized in the form of several macro elements, it is possible to arrive at a 3 dimensional model of the masonry construction. And carry out static, linear, non-linear, static, and dynamic analysis and the program that was referring to is another commercially available program called 3MURI.

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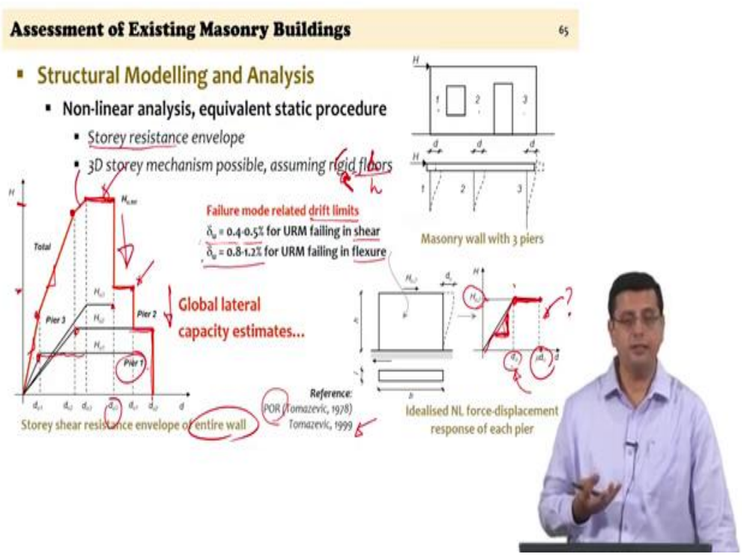

Having said that, should you always be depending on commercially available programs; is the behavior of masonry something that can be captured using simple calculations?

And the answer is yes to a large extent; the behavior of the simple masonry construction can be captured even in the non-linear elastic range using simple calculations.

But with assumptions that you should be aware of in terms of what goes into such modeling and analysis. I am making reference to a simple equivalent static procedure the modeling and analysis that will that will lead to a simple equivalent static procedure and this is referred to as. So, this was a simple technique that was developed about four five decades ago. But is a very simple, but powerful techniques to carry out non-linear analysis on simple masonry construction.

Of course, once the configuration starts becoming complicated a number of factors come into play and these may not be, so easy to carry out by hand calculations. So, I will take you through the simple hand calculation-based procedure and it is relevant for me to speak about this. Because, you have actually carried out calculations to estimate what is the shear capacity of a masonry, unreinforced masonry wall.

This procedure just depends on the assembly of a number of piers which are shear walls and arrive at the capacity of a storey of masonry. So, in this procedure what you are basically doing is if you are looking at a masonry wall which has perforations you have pier 1, pier 2, and pier 3. Now, this wall is subjected to lateral force and each of the piers, the 3 piers from the lateral load resisting system of the wall itself.

Now, we know that if we were to look at one of these piers at a time knowing the geometry of the piers and with assumptions on the boundary conditions; yes if it is a blank wall you might want to assume that it has a cantilevered deformation profile. If it is between openings with the perforated shear wall between openings, then you might want to consider the role of the spandrel in considering a fixed-fixed deformation profile or shear deformation profile for the pier.

So, you take each pier and for each pier estimate what is the shear capacity of the pier right. We went through the process of estimating the shear capacity of piers you will try to look at what is the mechanism that governs the failure of the pier. It could be a flexure dominated mechanism, it could be a shear dominated mechanism, so you will estimate what is the capacity of the pier.

Now, that is only giving you strength of the pier right, it is the giving you only the shear capacity, but to be able to do analysis you also need the you also need the deformation capacity. So, to be able to establish what the deformation capacity is you need two other things, one is what is going to be the stiffness of this pier. This is what is coming from the H-N interaction diagrams that you have carried out for the shear walls will help you establish for this geometry with the boundary conditions consider.

And for the axial load that is acting on this wall on this pier considering the single story or two storey structure, you will be able to establish what is the axial load. And then calculate what is the lateral force capacity or shear capacity of this wall, we will also know what mechanism it has failed in right; it is meant to fail in.

Now, given the geometry of the wall you will also be able to establish what is the stiffness of the wall the lateral stiffness of the pier right. You assume shear deformation profile or a cantilever deformation profile you will be able to establish what is the elastic stiffness lateral stiffness of this wall.

Once that is established you are basically able to say what is the slope of this line. You know what is H_u of this particular pier; you will be able to establish what is the elastic displacement at the end of the elastic deformation the lateral deformation at the end of the elastic cycle. We are considering that the behavior is linear up to this point, so you will be able to establish what is the elastic displacement, lateral displacement knowing the capacity of the wall and knowing the lateral stiffness of the wall.

Once that is done you need to know what is the maximum deformation capacity of this wall, how do I arrive at the maximum deformation capacity of this wall? Approaches are available; simple empirical approaches are available to establish what should be the maximum capacity in deformation of the wall. And what is typically done is given the simplicity of the procedure a simple empirical approach is prescribed that you assume that if the wall is failing in shear, it has lower ultimate deformation capacity of about 0.4 to 0.5 percent drift. It is in terms of drift which is nothing but the lateral displacement Δ over the height of the pier that you are considering, Δ/h. So, your drift is nothing but Δ/h, so if the masonry pier is failing in shear assume it has a drift capacity of about 0.4 to 0.5 percent at ultimate.

And if it is failing in shear failing in flexure assumed that the drift capacity is higher almost two times which is about 0.8 to 1.2 percent and these are numbers that are coming out of experimental tests. You will agree with me that we had looked at the hysteretic behavior of a masonry pier failing in shear versus a hysteretic behavior of masonry pier failing in flexure or rocking.

We had seen that deformation capacity is much higher when it is flexure or rocking control behavior, shear behavior the deformation capacity is significantly lower. And that is why the ultimate drift is about 0.4 for masonry wall failing in shear whereas, the ultimate drift in flexure or rocking is twice that value.

So, knowing the initial elastic displacement, the ultimate displacement is nothing but drift ultimate drift into h and then you will be able to establish what this second point is. But, there the curve is considered to be plastic, perfect elastic perfectly plastic, so you have elastic perfectly plastic behavior this point is determined from your hn interaction.

This is established after you know the stiffness of the lateral stiffness of the wall and the ultimate displacement capacity is known by making an assumption of what the failure mechanism is and what the corresponding drift limit should be. And therefore, you get a bi linear curve for each pier, so for each of these piers pier 1, pier 2, pier 3 you are able to establish the bilinear curve.

Once you have the 3 bi linear curves you see that the total capacity of the wall is nothing but a summation of the capacities of the pier. So, finally, what you are doing is for each of the piers pier 1, pier 2, and pier 3 you are establishing what will be the force displacement, the bilinear force displacement curve of each pier. Now, the total capacity of the wall is nothing but the summation of these three, so you have the lateral force displacement curve for pier 1, pier 2, and pier 3 for every point you will merely add up all the values.

But, when you come to this displacement you come to this displacement the first pier has gone into in elasticity. And therefore, you see that there is a change of slope here when you then add there is a change of slope between the initial versus the second phase. Because, the first pier has now plastified then you continue adding all the capacities you reach the second point and at that point the pier here this pier here has now gone into inelasticity. And therefore, you see a next change of slope keep adding, but after that you do not have piers at this point you reach the third pier also going into inelasticity.

And therefore, you reach the plateau of the overall curve of the wall itself. Then continue adding the displacements, but when you reach this displacement at this displacement pier 3 has reached its ultimate value that pier is no longer active. So, you see a sudden drop in capacity third pier is out of the picture you have only two more piers left. Further displacement, further displacement occurs you come to this point after which this pier; pier 1 has failed you come down.

And you are left only with the pier 2 capacity and at the end of which pier 2 is also failed pier 2 has flexural mechanism and that is why pier 2 has larger deformation capacity. So, merely by adding up the 3 bilinear curves of each pier in this case we have arrived at the lateral force capacity of this wall itself right. You can then look at, if you have two parallel walls then, can the two parallel walls be added to give you the total capacity of the storey itself.

And by this you are really arriving at the capacity of one story, knowing the capacity of that storey you can then check against the shear demand on that storey whether the whether each of these elements will fail for the level of shear demand that is coming onto the wall itself. So, from your shear demand you will be able to estimate how much of shear demand is expected on pier 1, pier 2, and pier 3. You know the capacities of pier 1 pier 2 and pier 3 you will be able to check if the demand with capacity ratio is less than 1 or greater than 1.

So, this is one simple approach that is available for carrying out non-linear analysis in simple masonry constructions. However, what as I said; this comes from work that was conducted quite a few decades ago. And this was a program that was developed called pushover analysis of masonry developed in Slovenia 1978 by Tomazevic. And you could do some further reading if you are interested in the book by Tomazevic on earthquake resistant construction of masonry.

So, this approach this sort of an approach is actually helping you establish a global capacity. Now, we looked at one wall; it could be an assembly of several walls in situations you might have torsion. In some situations, you might not have torsion if the eccentricity between the center of mass and center of stiffness of that storey is coincident. So, you could actually do a 3 dimensional storey mechanism; a three dimensional storey mechanism is possible.

We have seen how torsional effects can be considered by making an estimate of the center of mass and center of stiffness and the eccentricities thereof. But if you are making an assumption, if you are making calculations on the 3D storey mechanism. We are assuming that you have a rigid diaphragm that is actually holding all the walls together and resulting in torsion or a situation where there is only translation and no torsion.

So, using such approaches and the two methods that I showed you earlier the equivalent frame modeling or the macro element modeling approaches which are computer-based calculations, you arrive at the total the global lateral capacity estimate of the masonry building right. You want you will be able to establish what is the overall lateral force capacity of the force capacity and deformation capacity of the masonry building as a whole with the lateral load the horizontal elements and the vertical elements working together.

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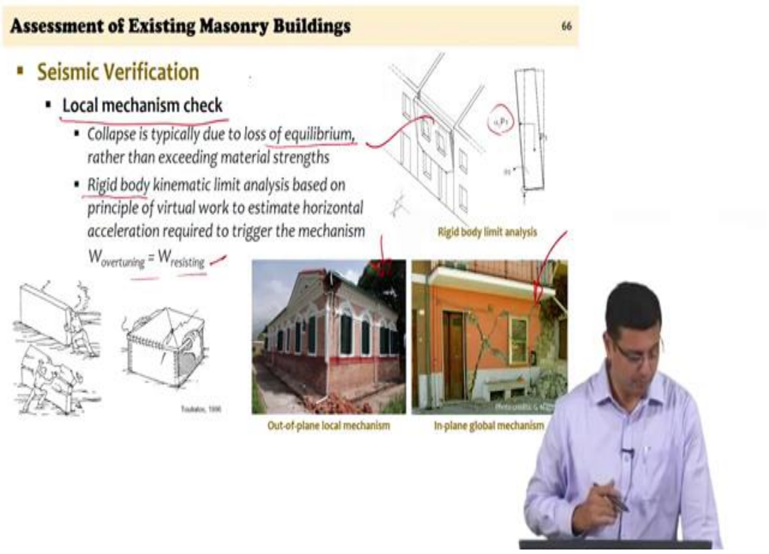

However, if you remember in our earlier lectures, we have talked about the fact that a masonry building may not necessarily work together right. Unless the connections between the walls are good and unless you have a rigid diaphragm that is connected to all the walls well, the masonry walls will actually act independently. So, this is something that we have talked about earlier which means if you try to do a global analysis on a system where the walls are not well connected to each other.

You have a problem; you are assuming that the masonry is actually acting integrally, but it may not act integrally. And therefore, it is recommended that in conditions where you know that the masonry structure is not well constructed for integral action. Particularly with bands or well connected corners, what is necessary to be done is that you make checks which are called local checks to see if a single wall can fail before the overall structure can resist the lateral forces itself.

So, what we have seen in the last three approaches will lead us to a global estimate of the capacities. However, if the masonry building is not well connected in terms of its wall to wall connections and floor to wall connections, individual walls will start failing. And if you see the two pictures here at the bottom you see that there is one wall separating off from the other and having a tendency to overturn in the out of plane direction.

That is a local mechanism the whole structure is not responding it is only one wall which is actually failing. So, local out of plane mechanism can occur, but if you were to see the picture on the right, you have shear cracks formed in the masonry walls that is an in-plane mechanism. But that in-plane mechanism is happening simply because the whole building is able to act as one and the shear walls are resisting. The global estimate that we saw in the previous slides is appropriate for such behavior; not for the behavior that is on the left.

It is a problem if you were to use those approaches to estimate the global capacity of a system like this. So, in such cases what is done is called a local mechanism and I am just introducing this to you, so that you know that apart from the global checks you also need to do these local checks particularly in buildings where connections are very poor.

So, what is done is that each of these elements are behaving as rigid blocks that can simply overturn and fail in out of plane direction. So, here it is really not depending on the strength of that element, it is just collapse that is due to loss of equilibrium of that element. And therefore, rigid body dynamics can be used and that is exactly what is used in such cases collapse is mainly due to loss of equilibrium rather than material strength that is being exceeded.

As you can see in the shear failure formation in the picture on the right bottom corner. So, what is normally done is you assume rigid body behavior of those elements here this wall for example, is acting like a rigid body and just overturning. And principle of virtual work is used to estimate what is the lateral acceleration required to make that block overturn; to develop a mechanism.

And the overturning force is equilibrated to the resisting force from which you will be able to establish what the acceleration, the horizontal acceleration required to cause this overturning is going to be. So, if you see this little example here limit analysis is used here in this piece of the wall which is expected to fail in the out of plane direction.

So, you know the geometry, you know the total weight and you are able to estimate what is the resistance of that element. And calculate what is the force coming on to that element and the ratio between the overturning force to the resisting force will be able to tell you at what level of acceleration is the failure expected.

And that level of acceleration is considered as a percentage of the total weight of that small element we you are looking at. So, if P_1 is the self weight of the wall itself at a lateral force equal to some α times P you are getting the mechanism formation. So, it is simple rigid body dynamics that is being considered here and that check can be done.

In addition to the global check to see if there are there are susceptible and vulnerable elements, vulnerable to out of plane failure mechanisms in the masonry structure. So, local mechanism check becomes necessary particularly when connections are poor in masonry constructions ok.

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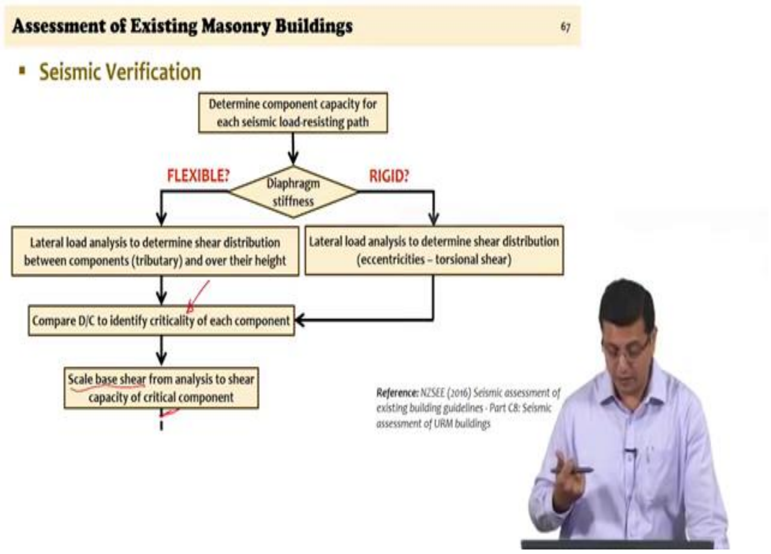

To close how do you then go about carrying out your verification. Let us say you are you have got the material strengths; that is what we looked at originally we get the geometry you have to get the you have to get the residual strengths.

And then make your capacity estimates; once you have got your capacity estimates and you know what is the demand coming onto the structure you need to do this verification of the demand to capacity of the whole structure. How is the structure going to survive for a level of lateral action that the structure is going to be subjected to. There is this interesting flowchart which then actually captures how you go about estimating the lateral capacity and make checks on whether the structure would be able to survive or fail under a given level of seismic excitation.

This is with reference to the New Zealand standards and the New Zealand standards its version in 2016 actually looks at existing buildings, and how you could go about doing a seismic assessment of existing buildings. And now this very nicely captures all the concepts that we have been examining earlier in our course as well.

So, for seismic verification you begin by; you estimate the component capacity you are talking of walls you want to start estimating the lateral capacity of each wall right it can be walls that can be diaphragms. So, first you need to be able to establish the capacity of each component in the earthquake load path, the seismic load path.

So, you determine the component capacities that is your first step, your second step is to understand whether the diaphragm that is there in the structure, is it a rigid diaphragm or is it a flexible diaphragm? And we have seen the definition of were flexible diaphragm based on how much in plane deformation is going to happen delta max versus delta min being within a certain specific value.

Now, the diaphragm stiffness if it is rigid you have a certain approach, if it is flexible you have a different approach simply because, if it is rigid your lateral force distribution is going to be based on the stiffnesses. So, second question is are there any eccentricities, if there are eccentricities apart from the direction you also get some caution has here. So, first question is if the diaphragm stiff; if it is classified as a rigid diaphragm then you do a lateral load analysis by distributing the shear force demand to the respective components based on their stiffnesses.

And also check if torsional shear is expected due to eccentricities, then at that stage you can go and check if the demand to capacity ratio for each component is acceptable. You know the capacity of a component, you know the shear demand coming to the component you will check if the shear demand to shear capacity is less than one. And that will tell you that the component is going to be able to resist the demand seismic demand coming on to it.

If it is a flexible diaphragm you know that it is the tributary area that is going to be what determines how much of earthquake demand is going to the going to the element to the component. So, you make an analysis based on that and then check what the demand to capacity ratio for that component is. Now, what is important is you need to check for every component, but the most critical component then determines what is the total capacity of the structure.

So, when you are making the calculations for each component or for each stock of component is one wall with several piers or the wall together several walls together in the seismic load path itself. So, identify which is the most critical load path in this in the structure.

That will then determine what is going to be the capacity that the structure what is going to be the shear demand that the structure is going to be able to resist. So, you might have to then say the total base shear that the structure will get is a certain value from your analysis. But, the critical component is able to resist only a part of that; that means, the capacity of the structure is now less than the expected demand coming onto the structure.

So, you are basically going to scale the base shear based on the capacity of the critical element or the critical load path. Once you do that if that has to be transmitted successfully; if that shear force resistance has to be successfully transmitted from one component to the other finally, to the foundation, the connections have to be good. So, this stage for this level of base shear capacity you will go and check what the connection adequacy is.

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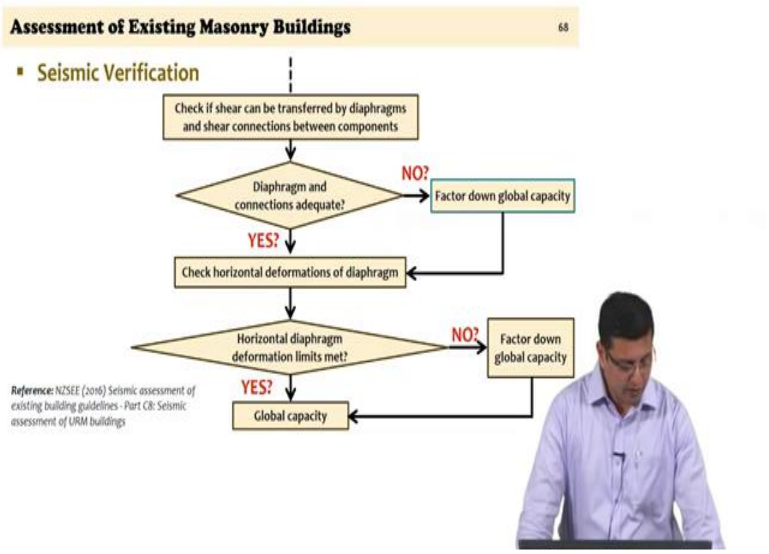

So, this continues and you have to check if this shear force can now be transmitted successfully by the diaphragms and also the connections the shear connections between the diaphragms and the components and the load resisting components. So, at this stage we have to check if the diaphragm is adequate, if the connections are adequate and here if the diaphragm is going to be able to transmit the lateral forces through adequate connections, then you now have to check if the diaphragm deformations are within acceptable limits. But, if the diaphragm and the connections are unable to transmit the base shear capacity that the component was able to take you have a problem, because the connectivity is now not provided by adequate connections.

Therefore, you will establish how much shear force can that connection actually transmits that becomes the shear force that the building will resist not the shear capacity of the component. So, you would basically have to now scale down the shear force that the building will be able to resist, because the connections are not able to resist the shear capacity that the components have.

So, you factor down the global capacity and then so you either come down this way or come down the other way depending on the adequacy of the diaphragms and their connections. And then at that stage check if the horizontal diaphragm when it receives the lateral force that we are talking of, whether the diaphragm displacements are within acceptable limits.

So, this is the final check that is done if the horizontal diaphragm’s deformation limits are within are acceptable, then that shear force is really the capacity global capacity of the structure. If it is unable to then you will have to establish that this is the shear force that takes the deformation of the horizontal diaphragm to its limit, that is the actual capacity of the structure.

So, you will compare that capacity to the overall base shear that the structure is being subjected to and you will see that in in many cases if the connections are not adequate, if the diaphragm is not adequately stiff, the overall structure’s capacity is severely compromised with respect to the total base shear that the structure will be able to resist.

So, this simple frame work gives you a possibility of respecting the importance of connections. And then, also considering the capacities of individual elements in the masonry load bearing construction.

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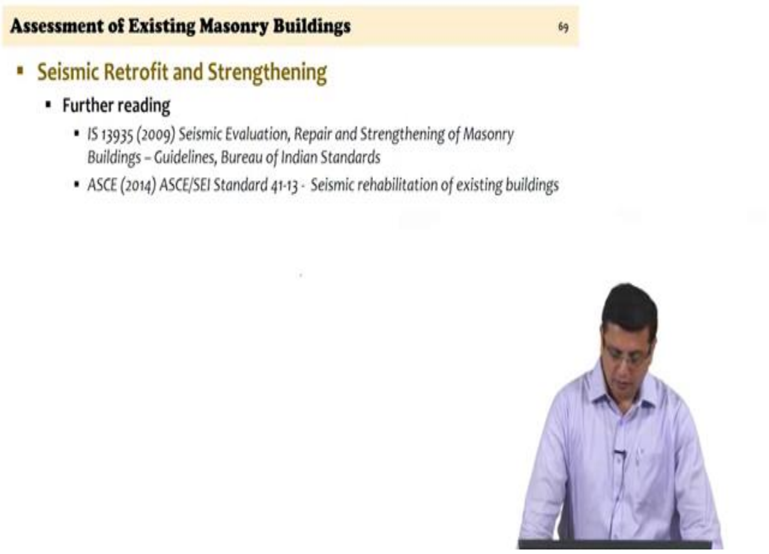

So, of course all this basically leads to identifying whether there is a need to strengthen the structure, need to retrofit the structure. So, that is something that I am not touching upon within the within this course, even in the special topics. But, these are two standards that you can refer to, to get a basic idea of how then would you start ensuring earthquake risk reduction in the structure that you are assessing.

And of course, focus again has to be on adequacy of connections, adequate stiffness of the diaphragms, and adequate shear capacity of the components. So, you will basically work on all these three levels connections should be good, diaphragm should be capable of transmitting forces and each component must be able to resist. And therefore, interventions would have to focus on these three aspects. There are two standards we have an Indian standard that looks at evaluation repair and strengthening of masonry buildings.

This is a guideline document and there are in fact, not many calculations that go into designing the interventions or the strengthening measures. But is a broad document that looks at evaluation to strengthening of masonry constructions. The ASCE document, ASCE 41 is a document that actually looks at if you were to do seismic rehabilitation of existing buildings; different types of building.

And of course, one of them one of the chapters deals with masonry buildings; you have the different steps that have to be followed and different options that can be looked at ok. So, with that we conclude the course and that was the last module in the last part of the module on special topics looking at assessment of masonry constructions. I hope you enjoyed the course.