Design of Masonry Structures
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Module - 04
Lecture – 31
Design of Masonry Components and Systems Part – X

Good morning, we will continue with our lecture on the basis for seismic design. And, define how you arrive at the demands on different components- the piers that are going to be designed for a combination of gravity and lateral forces.

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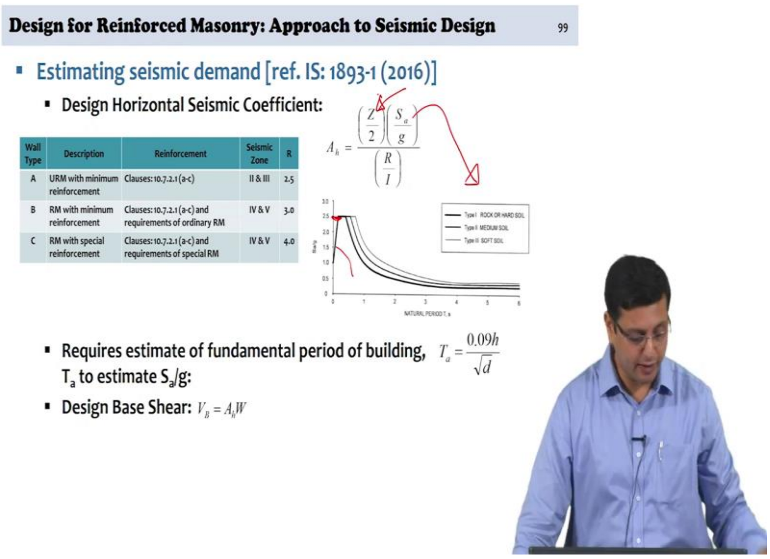

So, with respect to the code that refers to the definition of seismic input IS: 1893 part 1, we have seen how the horizontal Design Seismic Coefficient has to be defined for the building that you are going to be designing. And A_h which is the design seismic coefficient requires the use of the zone factor, Z/2, Sa/g which comes from the design horizontal acceleration response spectrum. And with the selection of the level of ductility that you would like to have in your structure which determines the behavior factor.

So, depending on the type of reinforced masonry or unreinforced masonry with specific seismic resistant detailing that you would do with respect to IS: 4326, you would choose your R factor and the important factor based on the occupancy and use of the building itself. For the value of Sa/g, we make reference to the elastic design response spectrum.

And, the response spectrum that you see here is the one that is prescribed for response spectrum analysis.

If you are working with equivalent static analysis, instead of the response spectrum analysis, this initial part of the response spectrum, the ascending curve, the ascending portion of the response spectrum is not considered and instead you start with a value of 2.5 at time period T is equal to 0 for equivalent static method.

However, for the response spectrum method, assuming that you are using a response spectrum method to model and analyze the structure; this is the elastic response spectrum that you would be using from which you require the value of Sa/g, but for the value Sa/g you need to be able to a priori estimate the fundamental period of vibration of the structure, which in a simplistic manner, can be estimated by knowing the overall dimensions of the structure that you are designing; h being the total height and d being the side dimension, in the direction that you are considering the earthquake design. So, that is your initial estimate, of course, after your model you can always come back after doing a full-fledged modal analysis. Come back and check if the Ta that you have used is good enough or would you want to make some iterations there.

So, this is required once your A_h is determined, assuming we are talking of the design basis earthquake where the level of earthquake input is determined by Z/2, Z being defined for the maximum considered earthquake. So, once A_h is defined based on the choices you have made, you then go and estimate the design base shear for the total building which requires the seismic weight W of the building.

So, the seismic weight again requires a percentage of the live load to be accounted for depending on the level of imposed loads. And, it also should consider additional loads like; heavy snow loads or sand loads if present in regions that are affected by snow storms and sand storms. So, once the design base shear V_B is estimated for the building that you are designing, we then move to the next phase which is taking that to different floors and then taking that to the different walls and the piers to establish what is the shear force for which we should design.

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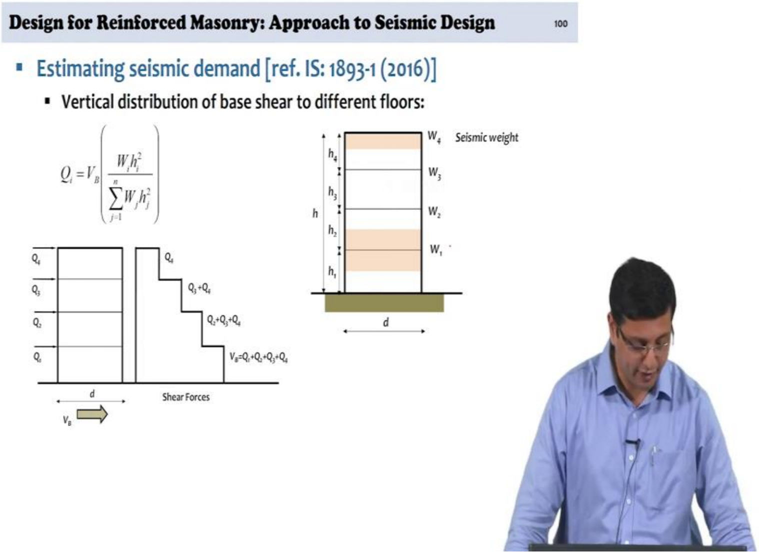

So, today we will actually be looking at this transition from total base shear in the structure to the base shear that you would estimate and the basis to estimate that for each pier. So, the first part is about the vertical distribution of the base shear to the different floors. Now, what does that require? It requires us to be able to define what is the seismic weight now floor wise. We had the overall seismic weight of the structure but then we should also be able to estimate the seismic weight per floor.

And estimate Q_i which is the shear force corresponding to one floor (one storey)

$$Q_i = V_B \left(\frac{W_i h_i^2}{\sum_{j=1}^{n} W_j h_j^2} \right)$$

Where the i here stands for the storey that we are looking at, W_i is the seismic weight of a given storey and h_i is the interstorey height of the storey that you are considering. So, h_i could be different for different storeys and that is something you should be careful about; typically h_1 the first storey maybe taller than the other. j is nothing but the number of storeys again; so one to the maximum number of storeys, in this case 4. So, you are basically estimating in a proportionate manner, how much floor shear of the total base shear should you apportion to a given storey?

So, when you are estimating seismic weight; care is to be given to calculating the seismic weight. Because what we are implying in this model is that the seismic weight of the structure is lumped at the floor level right. It is a multi-degree of freedom system composed of masses which are lumped at the floor level. Now, that requires a certain careful transition from the actual structure to this lumped mass idealistic model that we have.

So, what we typically do is; every floor is considered to be composed of half the mass of the storey above and half the mass of the storey below. So, if you look at W_1; W_1 is lumping half the mass from the top and half the mass from the bottom. What happens to the mass below of storey 1? It is you are considering a fixed boundary condition and therefore, that has no degree of freedom and so, half the mass of the ground story is really not coming into the calculations.

If you look at the top floor- fourth storey here. It takes the load from the terrace probably you have a parapet wall and then half the mass of the fourth storey. So, it is essential to estimate the storey masses which is then used to distribute the total base shear to the floor shears. Again, if you were doing this, you can have different masses in different floors and that is fundamentally the reason why we are trying to look at the distribution of shear forces based on the distribution of masses.

In a masonry structure considering the fact that most masonry structures are rather symmetrical in their plan layouts and load bearing walls are continuous, the mass should typically be quite similar along the height except for the topmost story. So, once you estimate Q_1, Q_2, Q_3 and Q_4 at each floor you are basically ready to now take the floor shears and then distribute it in the floors. What you are seeing here is really the conversion of the base shear that we had, V B into the different floor shears and as you can see the summation of all the 4 shears Q_1 to Q_4 can actually give you the total base shear V_B itself. So, once this is carried out, you know what is the floor shear that each floor has to be designed for; our focus now shifts to the floor itself. We now have to start examining different aspects within the floor.

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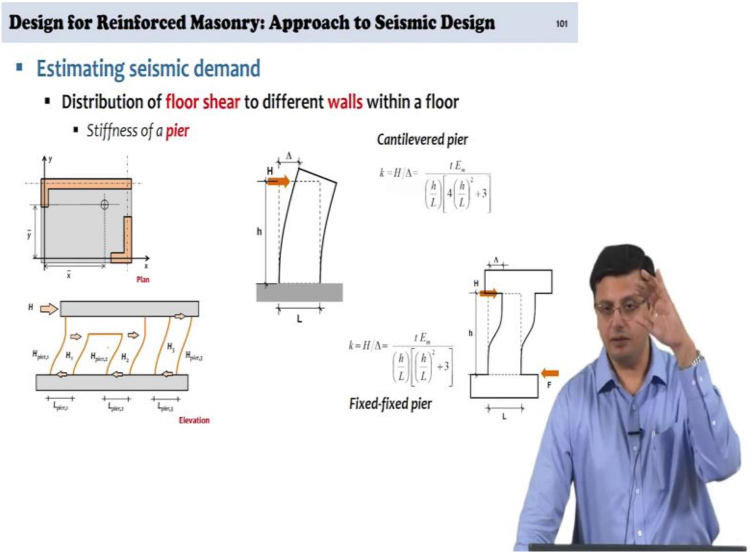

So, again to come back to the terminology that I have been using, you are talking of floor shear. You are talking of total base shear, the floor shear and then we are talking of the different walls in the floor itself. Within each wall you have resisting vertical lateral load elements which we are referring to as piers. So, the transition is total building to the floors, to the walls and then to the piers and your final goal is to be able to establish what are the shears forces per pier, ok?

So, that would be a floor for you with different walls configured around the plan. So, each wall that you see here, each extension along the x and the y directions are different walls. Now, the next goal is to be able to apportion this floor shear Q_i that we established in the previous slide to the different walls themselves and then to the separate resisting elements within the wall. Now, the walls can actually be solid walls with no perforations or walls with perforations.

Now, walls with perforations have to be dealt with a little more carefully, because it is more involved; however, if you have a wall with perforations as you see here we have a door opening and then other a large opening that is dividing the wall into 3 vertical lateral load resisting piers. So, finally, we are interested in knowing what should I design H_1 pier for, H_2 pier for and H_3 pier for or what are the values of H_1, H_2 and H_3 and along with the gravity load you have the demand to which you will be designing the structure itself, designing those components themselves.

Now, for that of course, we require the definition of the stiffness of a pier and this is something we have already seen. Now, this requires a consideration of the boundary conditions of each of the piers and based on this categorization; whether we are looking at a solid wall or a wall between openings we have looked at the ideal deformation under lateral forces of such walls.

So, a solid wall with no openings within it should be expected to have a cantilever deformation profile, implying that the top of the wall is free to rotate, the bottom is fixed. And, we have in our previous lecture been able to estimate the stiffness of such a pier and relate it to modulus of elasticity of the masonry, the geometry of the wall H, length of the wall L and t the thickness of the wall.

Now, if it is a wall between openings, with perforations, then the pier that is sitting between these perforations is limited in terms of its deformation at the top, particularly the rotations at the top. And, you would have a shear deformation profile implying that the pier now has to be considered with a fixed-fixed boundary condition and for this case as well we have estimated what the stiffness could be. So, you have the basic unit in terms of estimating the stiffness of a pier, but now the transition from a wall to the pier has to be established ok.

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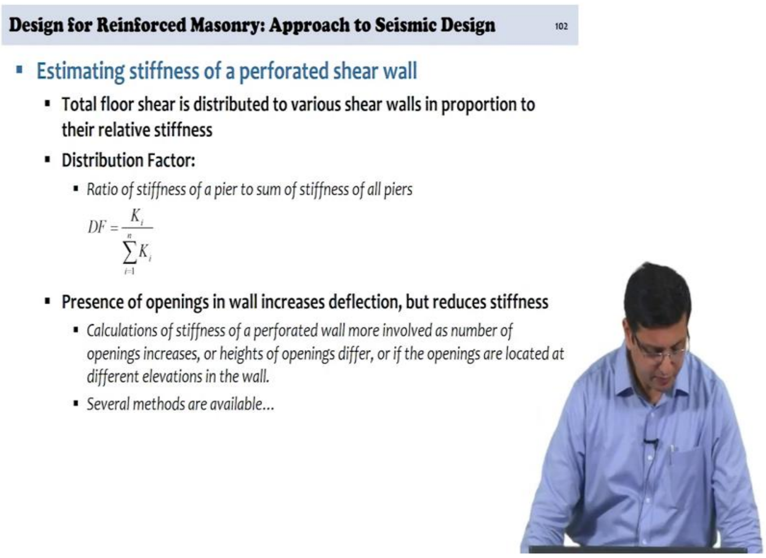

So, let us now focus on how do you go about estimating the stiffness of a perforated shear wall? The solid shear wall is not so much of a problem. You can assume that it is a cantilevered deformation profile and based on the geometry and the assumptions on the modulus of elasticity, arrive at the stiffness of the wall, but the complication comes when you have a perforated shear wall.

So, typically under the assumption of a rigid diaphragm ok; let us work with a rigid diaphragm at this point and also examine towards the end, if we were not having a rigid diaphragm in the structure what were to happen. In terms of distribution of the forces to the walls. Now, if you have a masonry structure with a reinforced concrete slab- a reinforced concrete floor or a roof slab typically, assuming that it is a rigid diaphragm is a rather acceptable proposition.

And we can then therefore, as a consequence, consider that the distribution of the shear force from the floor, the diaphragms to the walls will be based on the relative stiffness's ok. So, we are going ahead with that proposition. If that were not so, if the in-plane stiffness of the diaphragm is not adequate and we will see the limits on what is adequate in-plane stiffness of a diaphragm; then the diaphragm could actually be classified as a flexible diaphragm or a semi rigid diaphragm, that creates a complication because you cannot now distribute the forces onto the walls related to the stiffness; you have to adopt a different strategy.

So, now at this stage we are examining the distribution of the shear forces to the walls proportionate to the relative stiffness's. So, what is really happening is, we need to be able to establish what is the distribution factor- I have 4 walls, I have 5 walls in the configuration in a plan, I need to establish what is the distribution factor with respect to the total stiffness of that storey which goes to the wall itself.

So, the distribution factor can simply be defined as the ratio of stiffness of the pier that you are examining to the sum of the stiffness's of all the piers; if you are looking at a wall in a floor, if you are looking at the pier it is within a wall itself. So, still it is simply K_i or sum of K_i and that is a ratio that you would use, multiplied with the wall shear to establish what is the shear force going onto a single component itself.

Now, the moment you have openings, the openings in a wall, it increases the deflection of the wall right. And that reduces the stiffness of the wall. So, how do you account for this is important, but there are some complications. If you consider a regular wall which has window openings and door openings and ventilator openings, you have openings of different sizes in the first place, you have openings which are aligned at different heights along the height of the wall and hence how do you account for these different sizes of openings at different locations in a wall is a rather involved set of calculations.

So, this is what we are going to be examining; however, there are simple analytical methods that are prescribed based on simple statics that you could use and we will examine three of them and you could use; you could use any of them; however, some of them have a level of conservatism which is probably not acceptable; if you are doing a rigorous analysis.

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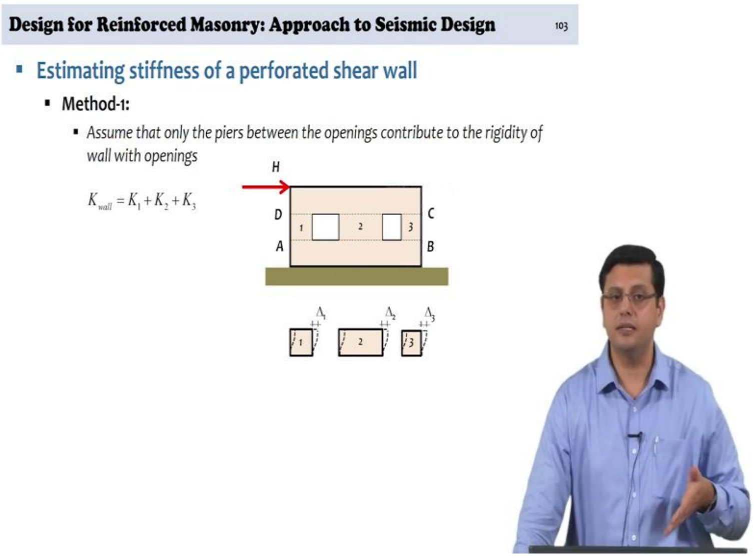

So, the first method is the simplest method, but you are bound to get percentage error in comparison to a more rigorous calculation which is not insignificant. I would say 5 to 10 or more percent with respect to what would be a more rigorous calculation is to be expected in the first method; however, this method is also acceptable from an engineering standpoint and for a quick estimate this is accepted.

Now, what this method does is; you assume you have a wall with openings, that the stiffness of the wall is estimated merely from the stiffness's of the piers; by just simply adding up the stiffness's of the piers. You are not considering the effect of the spandrel that is typically present above the openings in the case of a window and below the openings; again in the case of a window. In the case of a door you have the spandrel above as well.

So, this sort of a calculation really does not give due consideration to these spaces. What we are talking of; if you take this particular wall that you are seeing in the slide I have two window openings, they are of different sizes, they are all aligned at the same height in the wall; you could have a further complication that these are not equally aligned. One of the smaller window, the other is a bigger window or one is a door.

So, we have three piers between openings; pier 1, pier 2 and pier 3 within that panel zone the panel which is defined by A, B, C and D. You have the total wall and this panel within which the windows in the piers are. So, now once the windows are considered, then you actually have only 3 vertical lateral load resisting elements, vertically aligned lateral load resisting elements are only piers 1, 2 and 3.

What is prescribed in this sort of an approach is, you simply look at pier 1, pier 2 and pier 3. Since they are sitting between openings, the boundary conditions are such that rotations at the top are prevented. And, therefore, considered a fixed-fixed boundary condition, estimate the deflections of pier 1, pier 2 and pier 3 and sum them up to get the total stiffness of the wall itself.

So, the total stiffness of the wall is merely sum of stiffnesses of K_1 of pier 1, pier 2 and pier 3. Now you will definitely agree that we are completely neglecting the role played by the portion of the wall the panel that you see above and the portion of the wall that you see below ok.

Student: This one is considered as a solid 1, 2 and 3 portion.

1, 2 and 3 yes.

Student: Similarly, what we did the (Refer Time: 20:39).

Exactly; 1, 2 and 3 we have come down to the basic unit now. We have got a pier we have two options there. Either cantilevered pier or fixed fixed pier in this case since it is sitting between openings we choose the boundary condition as fixed fixed and that is a solid panel with the boundary condition and we have already established what the stiffness itself is. And you are summing up the 3 stiffness’s and this is simply springs in parallel. 3 springs in parallel is the total stiffness of the system itself.

So, we then go to method 2, which starts accounting for the effect of the large panel that we had above and below the openings.

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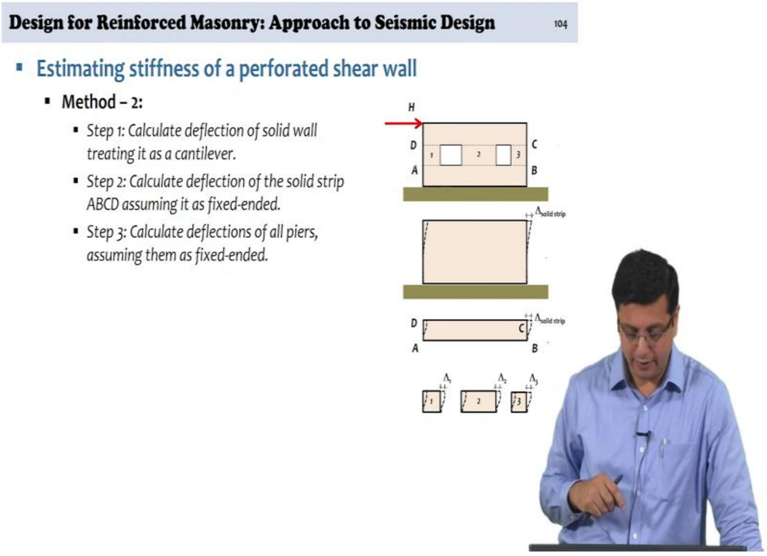

So, what we are actually going to be doing in this second approach is examine the wall as a whole; but the wall is a whole is; if you look at the boundary condition, if the wall as a whole is not sitting between openings, it is not restrained at the top. So, the wall as a whole can actually be considered as a cantilever, to have a cantilever deformation profile.

So, we could estimate the lateral deflections of the wall assuming it to be solid cantilevered deformation profile, then consider the strip within which the windows are sitting. So, you see in the second, in the third picture you have the basic configuration of the top the solid wall neglecting the presence of the windows below that is the solid strip.

And then we take one small strip within which the windows are sitting that is the third one A, B, C, D. Now, that has window openings and so that is going to be restrained compared to the overall wall without any window openings. So, this is considered to be a fixed-fixed pier and then we can go and estimate for the original piers that have identified, even in the last a method pier 1, pier 2 and pier 3 and calculate the deflections for that pier considering again fixed fixed boundary condition for the last case.

So, here in method 2, what we are doing is first calculate the deflection of the solid wall, but treated as a cantilever; second step take that solid strip within which the openings are sitting and calculate the deflection of the solid strip, assuming that it is fixed fixed now. Because it really has these panels sitting on top and below that would prevent the rotation of the top and the bottom.

So, it is correct to assume that is fixed ended. Then you calculate the deflections of each pier individually; you have pier 1, pier 2, and pier 3 here again we have fixed ended and you can calculate the deflections Δ_1, Δ_2 and Δ_3. So, I have the solid wall, I have the solid strip and then I have Δ_1, Δ_2 and Δ_3 estimated.

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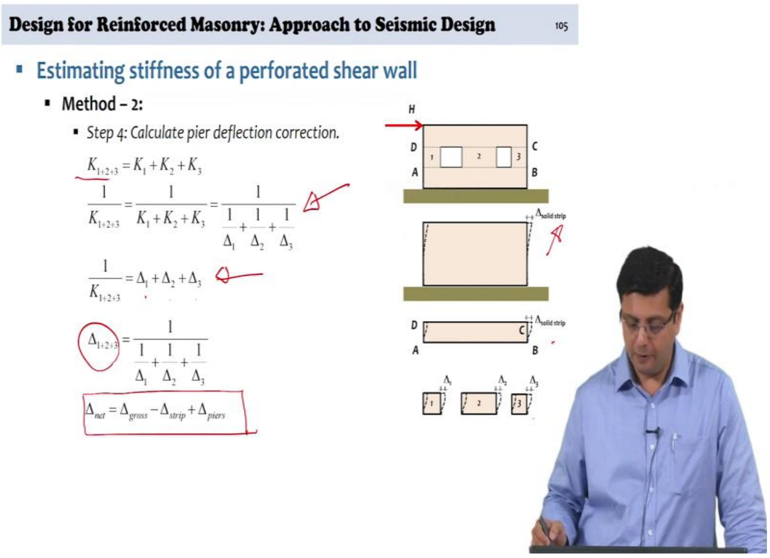

Now, to be able to estimate what is the actual deflection of the wall, you are basically going to correct the deflections of the central portion with the deflections that we know at the top and the bottom. So, basically again this is an analytical approach. We are correcting what we did in the previous method itself. So, what we are saying is the stiffness of the three panels, stiffness together the let us call it the pier group, the stiffness of the pier group $K = K_1 + K_2 + K_3$

Now, to arrive at this stiffness of K_1, K_2 and K_3 independently individually we are going to introduce some corrections. Now, we take the inverse of the stiffness and that gives us the deflections. So, $\frac{1}{K_1}+\frac{1}{K_2}+\frac{1}{K_3}$; K_1 and K_2 and K_3 are then represented as the inverse, inverses of the stiffness's.

So, we write it down in terms of this value is equal to $\frac{1}{\frac{1}{\Delta_1}+\frac{1}{\Delta_2}+\frac{1}{\Delta_3}}$, but we also know that because of this because of the fact that the reciprocal of the stiffness is going to give you the deflection; $\frac{1}{K_1}+\frac{1}{K_2}+\frac{1}{K_3}$, the total stiffness of the pier group is going to be the summation of the corrected deflections $\Delta_1 + \Delta_2 + \Delta_3$.

So, each of those therefore, the deflection of the pier group is, if you use this which is nothing, but because deflections and the stiffness's are reciprocals and the first expression; we can then arrive at the total deflection, this is of the pier group, this is of the individual piers. So, deflection total deflection of the pier group which is the correction that we are doing as the right hand side with the individual deflections, reciprocals of the individual deflections on the denominator.

So, what we are finally going to be doing is, the actual deflection is the gross deflection which is coming from the deflection of the overall wall considered as a cantilever without any openings, minus the deflection of the strip which is this plus the 3 deflections of the piers. So, still an approximate method, but it is something that considers the important role played by the spandrel at the top and the spandrel at the bottom.

So, this is finally, what you should be keeping in mind that we are looking at the net the corrected deflection as being gross deflection minus the strip deflection plus the 3 individual deflections of the small piers 1, 2 and 3.

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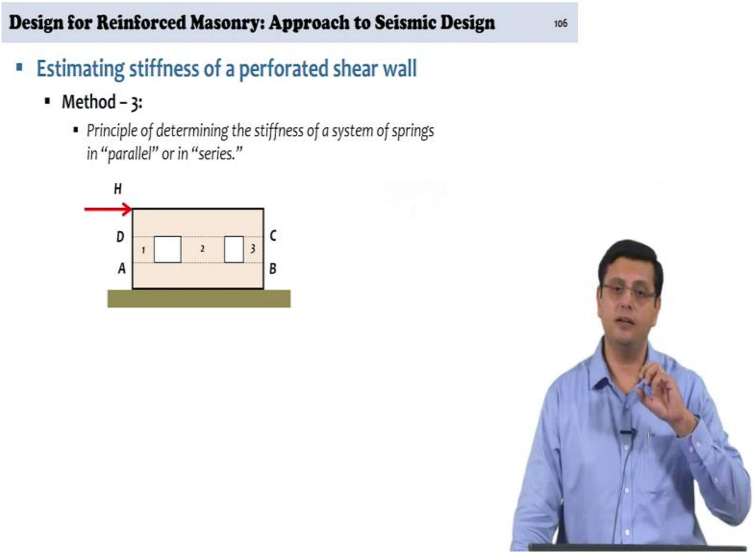

So, this is the second method the third method, is a little more systematic in terms of the considerations of actually how these stiffness's are lining up. If you remember when I talked about method 1, I talked about method 1 being 3 springs in parallel right. When we came to method 2 that gets a little cloudy; we do not define it very clearly. We are looking at a total and the negatives and then working on subtracting smaller stiffness's from the overall stiffness.

In the third method; the third method is probably the best method which requires a little more rigorous calculation, but really considers the whole process of distribution of forces by considering them as springs in parallel or springs in series. So, if you look at the whole wall in this whole wall, the 3 piers pier 1, pier 1 and pier 3 can be considered as 3 spring 3 springs in parallel. Now, if you were to consider the 3 springs in parallel with the beam or this slab or the panel that is above right; that is 3 springs in parallel with a panel in series now yes.

So, the panel above with the 3 springs in parallel is a system which is in series. Similarly, if I consider the 3 piers and the panel below, that is a system which is in series. So, once I have established stiffness of springs in parallel and then stiffness of spring in series, the one above and then the one below. I then can look at the total stiffness of the wall as springs in series.

So, that is the approach; this approach is probably the most convenient. Because the moment you have a windows and doors openings which sort of different sizes this starts giving you a better hold on how the stiffnesses contribute, individual stiffnesses contribute to the overall stiffness.

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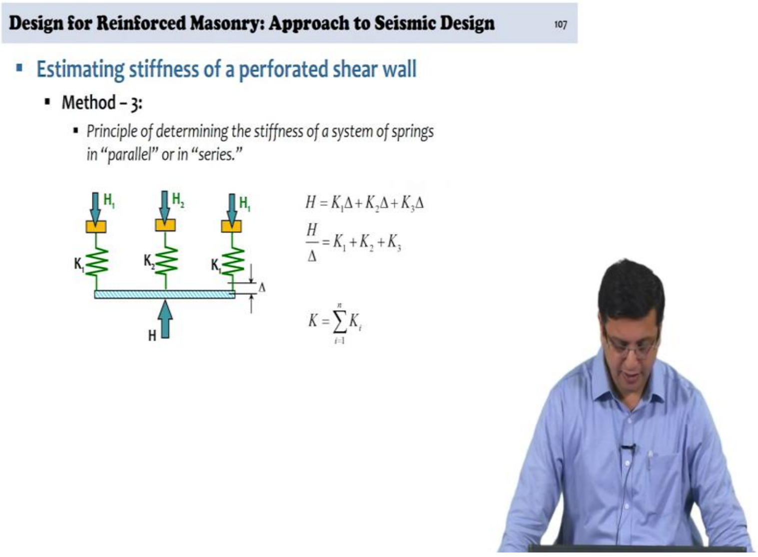

So, in a simplistic manner looking at if you have springs in versus springs in series, because in the whole wall it has a system of springs in parallel and series. So, if you were looking at a set of springs, which are in parallel; so if you look at idealize your masonry system as just having 3 piers. Each of stiffness K_1, K_2 and K_3, then the total force that the wall is being subjected to is shared based on the just based on the stiffness's.

And, you get the proportions H_1, H_2 and H_3 which will then add up to give you H. Again, fundamental assumption here is you are talking about a rigid translation of the diaphragm; this cannot be considered if you do not have a rigid diaphragm in the system. This distribution is no longer valid. And, therefore, the total force H by equilibrium would be stiffness $K_1\Delta$; Δ remains the same for the entire system $K_2\Delta$ plus $K_3\Delta$ and therefore, H by Δ will give you the sum of these stiffnessess K_1, K_2 and K_3 what we did in method 1 was only this, what we did in method 1 was only this.

Now, if you were to consider the and therefore, depending on how many have a system how many of parallel walls you have, you can make a summation and get the total stiffness.

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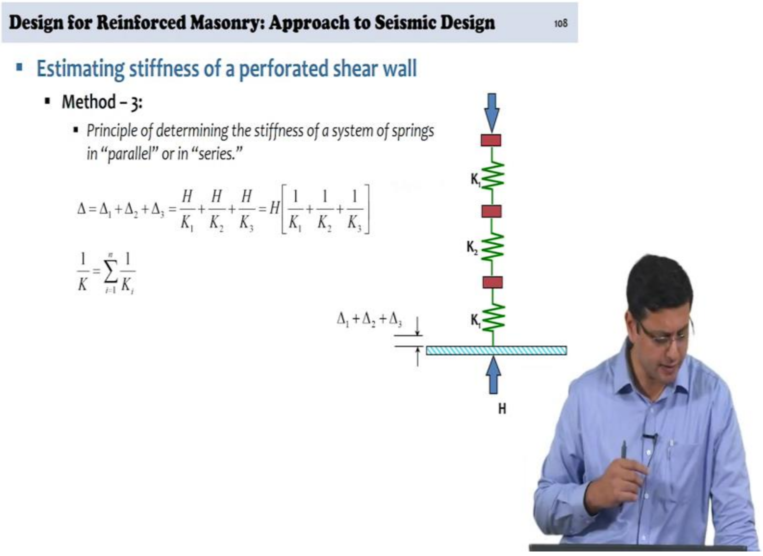

If you were to look at a system in series and that comes into picture when you have a spandrel and opening and a spandrel a set of piers and a spandrel you need to examine it in terms of system in series. And, so now the important difference is each spring will have it is own deflection. Three deflections would then add up in this case with 3 springs as the total deflection Δ; $\Delta_1 + \Delta_2 + \Delta_3$. to the lateral force H acting on the system.

So, here if you were to write down what actually happens is you take a summation of the displacements Δ being equal to $\Delta_1 + \Delta_2 + \Delta_3$.. Each of the deltas depends on the spring stiffness H remains the same here, overall H is the same and therefore, and we can write down the stiffness of this sort of a system in terms of $\frac{1}{\sum K_i}$, over all the springs in the system.

So, this third method involves dividing the wall into the respective springs in parallel and springs in series, establishing these stiffness's of the respective systems. And, then putting it together in terms of the entire wall as a set of springs in series. So, I get your question. So, just to paraphrase what you asked me; as far as the first set of the first figure that you have seen we are looking at springs in parallel and this is exactly what we did in method 1; which means, those three piers which were running in parallel were considered as piers with fixed fixed boundary conditions right.

So, this is piers with openings in between. So, that is fine. The moment we go to the other one we are talking of three sets of springs which are in series now. And, your question was when you are looking at the original wall that we were studying, it had in the central panel, it had a clearly the effect of the top and bottom panels and therefore, considering that as fixed fixed is meaningful what happens to the two strips at the top and the bottom?

Strictly speaking, they are not in isolation, they are all part of the same system and therefore, the fact that the top panel then has a continuation in terms of what we have considered as the central strip the boundary conditions need to be considered consistently. So, we would still continue considering the top strip and the bottom strip not to be cantilevered, but to have a fixed fixed boundary condition. And, if you remember in method 2 we looked at the whole panel with no openings, there we went in for a complete cantilevered profile.

Student: (Refer Time: 34:30).

No; Again I think this is something that mentioned I had mentioned earlier in class. That we are not talking of the slab providing a rotational restraint, we are not talking of the slab providing rotational restraint. It is the presence of extensions of the masonry on either side of the top or the bottom that is providing rotational restraint.

So, if you look at a pier between openings, you have the spandrel zone at the top and the bottom on either sides of the pier. That is what is blocking rotations. So, that is the reason why when we took the whole wall the whole wall really has no restraint; it is free to rotate. The slab might offer a certain partial rotational restraint, but it is not as significant as what a panel sitting beside and preventing rotations would do. Therefore, considering all the 3 to be having fixed fixed boundary conditions is the more appropriate decision that you would take ok. So, you could then sum up the reciprocal of the stiffness's in this case.

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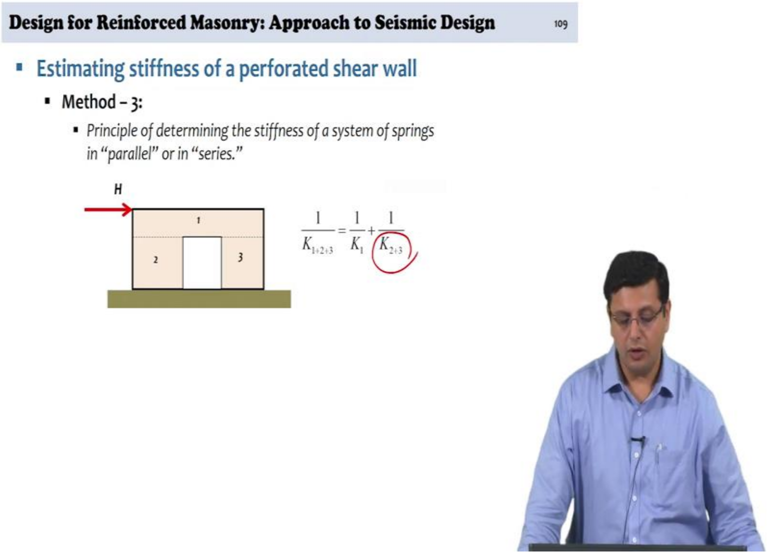

So, if you were to look at another example such as this one; a wall panel with a door opening- a wall with a door opening you have 2 piers, 2 and 3 and you have the panel at the top comprising of the spandrel and the extensions on the two sides. So, in this particular case we really going to be first looking at the system of springs in parallel; which is 2 and 3 and then the system of springs in parallel become a system in series with the panel 1. And, therefore, the total stiffness of the group is written as $\frac{1}{K_{1+2+3}}$

So, $\frac{1}{K_1} + \frac{1}{K_{2+3}}$ and I have already done the springs in parallel here adding these stiffness's. So, this is; so this is an approach that is consistent with the actual boundary conditions and it is more meaningful to go with the sort of a calculation. So, once you make these calculations, you are then able to arrive at what is the total stiffness of the wall, the ratio of individual stiffnesses to the total stiffness gives you the distribution factor so, the wall shear or the floor shear is then multiplied with that to establish what is the shear demand on a the pier that you are working on.

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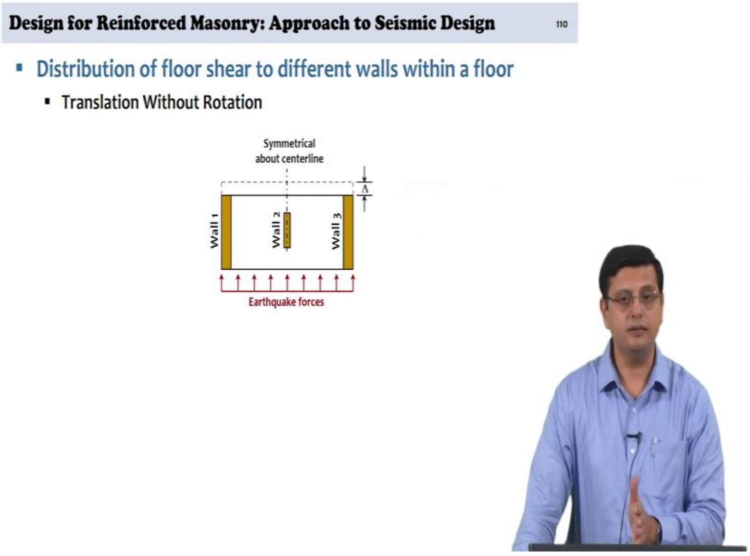

So, this set of discussions that we have had so far were with respect to a wall with perforations; now the moment I take one step backwards and say I have a set of walls and now how do I distribute the floor shear to those walls, what considerations should one make? So, the moment you want to look at floor plan configuration and the disposition of the walls within the floor plan, there are 2 things that need to be looked at.

One is when you have a floor subjected to a shear force; if the floor were to translate with no rotations then you distribute the total floor shear to the individual walls. So, assume the building that you are looking at is made up of 3 walls and these 3 walls are now a system in parallel and you can estimate stiffness of wall 1, 2 and 3 relative to the total stiffness of the floor. This floor is now subjected to earthquake forces from base shear, I have arrived at Q_i which is the floor shear and now need to distribute the floor shear to the three walls.

The displacement of this floor given the symmetry of the floor that I am looking at in the direction that is being considered. If I were to look at, you need to look at the symmetry with respect to the direction in which the earthquake action is. So, if the earthquake action is parallel to the 3 walls you now, because of the consideration of the rigid diaphragm effect and the symmetry, would have a uniform displacement of the structure, of the slab, of the diaphragm itself and therefore, Δ will be the same for all the 3 of them and therefore, distribution by stiffness's is easy.

This is a case where you have translation of the system with no rotation right; which means, the total shear force coming to the floor has only a direct component or direct shear component which is distribution factor into the floor shear, distribution factor into the floor shear for the second wall and distribution factor into the floor shear for the third wall. Only translational components of shear coming into the picture.

However, if and this is basically because, as far as this floor is concerned, the center of gravity of this floor, the center of mass of this floor and the centre of stiffness of this floor coincide. Since the centre of mass in the centre of stiffness coincide, I do not have any rotation expected in the floor and you have only a direct shear component.

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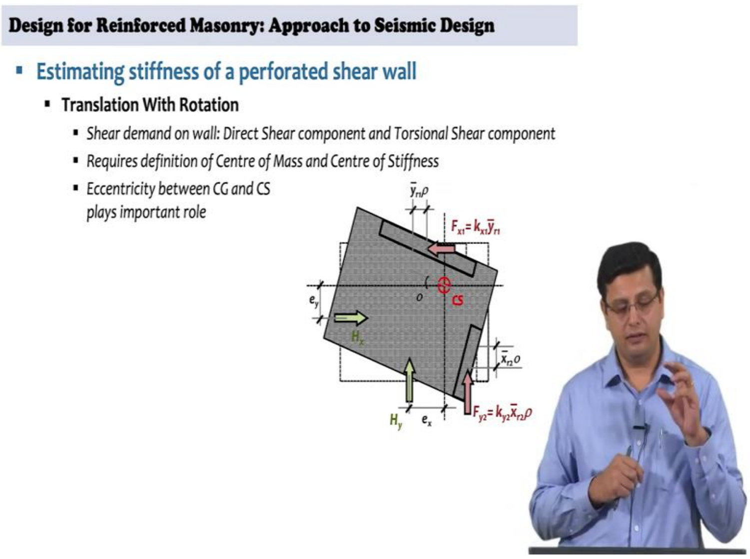

But, the moment you are going to look at more complex configurations you are going to have a second component which is a torsional shear component, that comes because of the eccentricity between the center of mass and center of stiffness and that needs to be established.

So, when you have an unsymmetric configuration, unsymmetric with respect to the direction of action of the earthquake force, you should be able to in addition to the direct shear component estimate what is the torsional shear component and add that in the demand coming onto the wall itself; the separate walls. So, in the previous cases owing to the symmetry you really need not estimate what the center of mass and center of stiffness is.

But, in this case you need to estimate the center of mass and center of stiffness. And, then be able to establish what is the eccentricity in the x and the y, between the center of mass and central of stiffness, because that will determine what is the additional bending moment, what is the additional shear force because of the twisting of the floor right; so what we will do in the rest of this lecture is to be able to establish a framework for estimating the torsional shear in addition to the direct shear component.

So, if you look at a configuration which actually does not have the symmetry that we had in the previous case with respect to the direction of action, you will have to establish, what is this eccentricity between the center of mass and center of stiffness in the two directions. And, then establish what each of the walls will get in addition to the direct shear component. If the earthquake were to happen in the y direction you will have an additional torsional shear component in all the walls and similarly, in the x direction.

So, we will continue with creating the framework for this in the next class. And with that you actually have the entire set of analytical basis required for arriving at the component shear demand and axial force demand, which then closes the loop in terms of system design to component design ok. I will stop here.

Thank you.