Design of Masonry Structures
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Module - 04
Lecture – 32
Design of Masonry Components and Systems Part- XI

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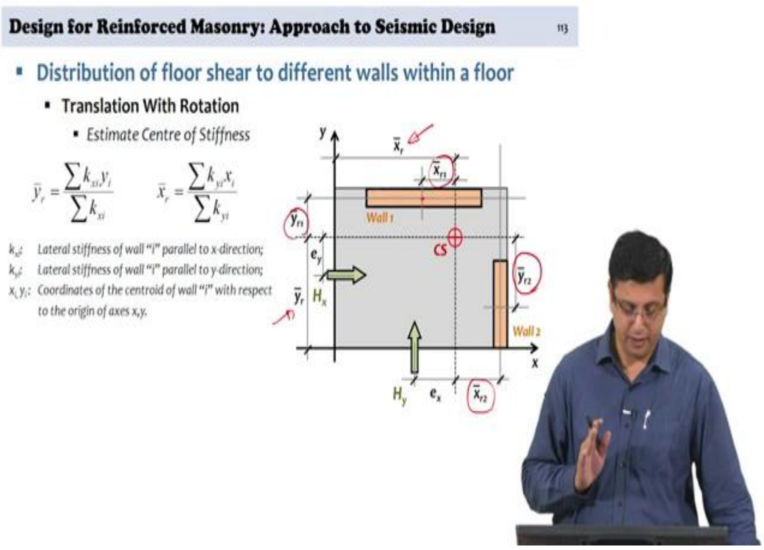

Good morning, we continue with the Design Requirements, we were looking at, how from the system level we were going to be arriving at the distribution of shear forces in a floor and then to the wall and then in the walls distributed to the different piers. So, in continuation with what we were looking at in the last class, I introduced this problem of whether the floor or the structure is expected to have rotations along with translations or not which is actually going to depend on whether or not you have eccentricity between the center of mass and center of stiffness.

And therefore, if you have a plan which has an eccentric configuration and layout of walls which causes an eccentricity between the center of mass and center of stiffness; then you will have to worry about the rotation of the plan in addition to the translation when lateral forces are acting on it; which would mean that you will have to estimate what is the torsional shear component in addition to the direct shear component.

And as I said, today we will look at a framework for making those calculations; then for any plan configuration whether you have translation without rotation or translation with rotation you have the framework to distribute the shear forces to the individual walls. And in this particular context, we are looking at floors or roof slabs which have rigid diaphragm action. So, with this is within the context; valid as long as the rigid diaphragm action is being considered.

So, in the last lecture we were looking at the situation of translation without rotation and given the rigid diaphragm translation all walls were subjected to equal displacement; therefore, we were able to distribute the shear forces to the walls based only on the relative stiffness's of each wall. However, when you have a configuration which has eccentricity between the center of mass and central stiffness, this has to be carefully considered, because in a given direction, if you were to consider x direction seismic action or y direction seismic action, the seismic force is going to be acting at the center of mass. Therefore, if there is an eccentricity between the center of mass and the center of stiffness or the center of rigidity of a system, you will have additional rotation of the floor plan, which will cause additional forces, additional shear forces acting on the resisting elements. So, let us examine for a moment, the plan configuration that you have on the slide.

The grey rectangle is the overall plan dimension, overall rectangular plan layout; it has two resisting walls, wall 1 and wall 2. The lateral forces, if I were to consider earthquake force acting in the x direction or earthquake force acting in the y direction, the green arrows represent the force considered in the two directions separately; then they would be acting along the lines defined by the center of mass or the stiffness.

So, given a plan configuration you will be able to estimate the center of gravity based on the masses; but then you should be able to estimate what the center of stiffness is, the red mark that you see there is the center of stiffness of the configuration here. We have two walls which are representing the resisting, lateral resisting, vertical load carrying elements, wall 1 and wall 2.

Now, the eccentricity that we are talking about is, if you consider earthquake action in the x direction and the component of the earthquake force H_x acts along the center of mass and what we are concerned about is the eccentricity e_y between the center of stiffness, the line defining the stiffness in that direction e y. And similarly an earthquake force were to be considered in the y direction, H_y; then the eccentricity e_x between the line defining the center of stiffness in that direction (in the y direction) and the center of mass, so eccentricity e_x.

So, our aim is to be able to first define what CS is with respect to, centre of stiffness is with respect to a plan configuration. And once that is defined, then we will be able to distribute the forces acting in the x and the y directions by calculating the additional torsional shear component. So, in this plan configuration, the geometry has to be laid out carefully, you fix the axis and that is up to you. In this particular case you see that, the left corner is taken as the origin of the axis; x and y is established. And then you have the geometry of the rest of the resisting elements, you need the center of gravity of each of the walls individually and the center of gravity of the overall system.

So, with respect to this what we are going to first look at is, how to estimate the center of stiffness itself. So, in the drawing again the center of stiffness when established, we are interested in looking at the dimensions, the distances of the centroid of each of the resisting elements with respect to the line defining, the lines defining the center of stiffness. So, if you look at $\overline{x}_r$ and $\overline{y}_r$, that is these two, with respect to the axis defined, give you the center of stiffness of the plan configuration.

Then with respect to $\overline{x}_r$ and $\overline{y}_r$, we are defining for each of the resisting elements wall 1 and wall 2, the x and the y dimensions with respect to the centroid of those resisting elements. So, if you take wall one; if that intersection there is the centroid of a wall 1, then we need to define $\overline{x}_{r1}$ and $\overline{y}_{r1}$ these two dimensions here which define the eccentricity with respect to that particular resisting element of wall 1; $\overline{x}_{r1}$ and $\overline{y}_{r1}$. Similarly for wall 2, we define $\overline{x}_{r2}$ and $\overline{y}_{r2}$ with respect to the center of stiffness, the lines projected from the center of stiffness.

So, that is the overall geometry with this way we are working, we should then be able to estimate the stiffnesses of wall 1 and wall 2 in the respective directions. In this case if the earthquake force is acting in the x direction, my consideration is that wall 1 is the only resisting element. I am really not going to be considering the resistance offered by wall 2 in the out of plane direction right. I assume that the resisting element is wall 1, which is subjected to in-plane forces. Similarly, when we are considering earthquake force acting in the y direction, I am not going to be considering the resistance coming from wall 2 in the out of plane direction, but I will be considering only the resistance in-plane of wall 2.

So, that is one little assumption that we make on the conservative side for most plan configurations, we will stick to that assumption here. So, given the configuration and given the estimate of the in-plane stiffness of wall 1 and wall 2 that we can make, based on all our discussions earlier; considering what boundary condition wall 1 and wall 2 would have from the geometry and the material properties, we will be able to establish what k_{x1} and k_{y1} are going to be k_{x2} and k_{y2} are going to be.

So, I take wall 1 and estimate the stiffness of wall 1 in the x direction, I will estimate stiffness of wall 2 in the y direction. And I have the in-plane stiffnesses is to work with, you can also estimate the out of plane stiffnesses if you want to do a more rigorous calculation.

So, the first step is estimating the centers, the center of stiffness defined here by $\bar{x}$, $\bar{y}$ with respect to the origin that you have established. So, this is depending on how complex your wall configuration in the plan can be, becomes involved; but as you can see, it is straightforward geometry-based estimation that you are doing here.

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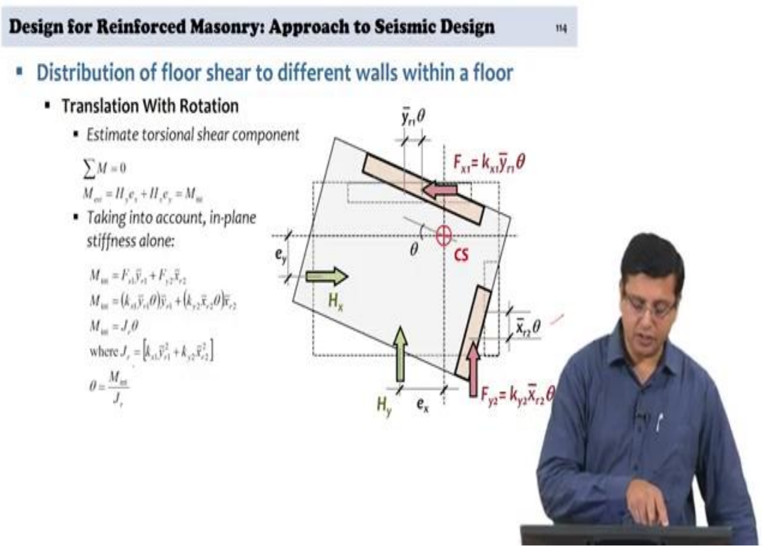

Once that is done, we then start looking at what would actually happen to the system when there is a force acting in the x direction or the y direction. When the force is acting in the x direction or the y direction; because of the eccentricity between the center of mass and the center of stiffness, you do not have translation alone, you are going to have a rotation. But this rotation is going to be centered at the center of stiffness. The force is acting along the center of mass, the rotation is centered at the center of stiffness and therefore, you should expect.

Because of the lever arm that is now created, you should expect additional forces which are torsional shear forces acting on wall 1 and wall 2 depending on the direction of force that we are looking at. So, geometrically, if you are looking at θ being the rotation at the center of stiffness due to the lateral force acting on the wall configuration; then you can define that the component of shear force acting on wall 1 with respect to the centroid of wall 1, can be geometrically defined. Consider wall 1; knowing the rotation θ, then this distance is going to be $\bar{y}_{r1}\theta$ and similarly for wall 2; because of the rotation, the distance between where the shear force is acting in the wall at the centroid of that resisting element versus the direction of the shear force you would have the lever arm defined here in terms of $\bar{x}_{r2}\theta$ for wall 2. So, this is again established from geometry and therefore the component of shear force that wall 1 is going to get is $F_{x1} = k_{x1}\bar{y}_{r1}\theta$

Similarly in wall 2, you would have stiffness of the wall in the in plane direction, you are taking the stiffness of the wall in the in plane direction and the component of force. $F_{y2} = k_{y2}\bar{x}_{r2}\theta$

So, in this case to estimate the torsional shear component, we take the sum of the moments, the exterior moment externally applied force creates a moment and that is the total moment is H_y into the eccentricity e_x and H_x into the eccentricity e_y which is going to be resisted by the internal restoring moment. And here as I said, we will neglect the component coming in the out of plane direction which is one small; and therefore, considering the wall dimensions, length of the wall versus the thickness of the wall it is to neglect the out of plane components and take into account only the in-plane components.

So, the internal resisting force, internal resisting moments that are defined with respect to the force component, $F_{x1}y_{r1}$ bar that we looked at here plus $F_{y2}x_{r2}$ when you consider earthquake force in one direction. So, again F_{x1} and F_{y2} can be written as defined earlier. Here if you rewrite this expression, this expression actually has the polar moment of inertia of the configuration and that is what is pulled out here:

$$J_r = k_{x1}\bar{y}_{r1}^2 + k_{y2}\bar{x}_{r2}^2$$

So, basically this is the polar moment of inertia that is being defined for the plan configuration. So, we are able to rewrite the expression in terms of the stiffnesses, these eccentricities for each of the walls wall 1 and wall 2 in a direction and establish what is the torsional shear component that is acting on the wall.

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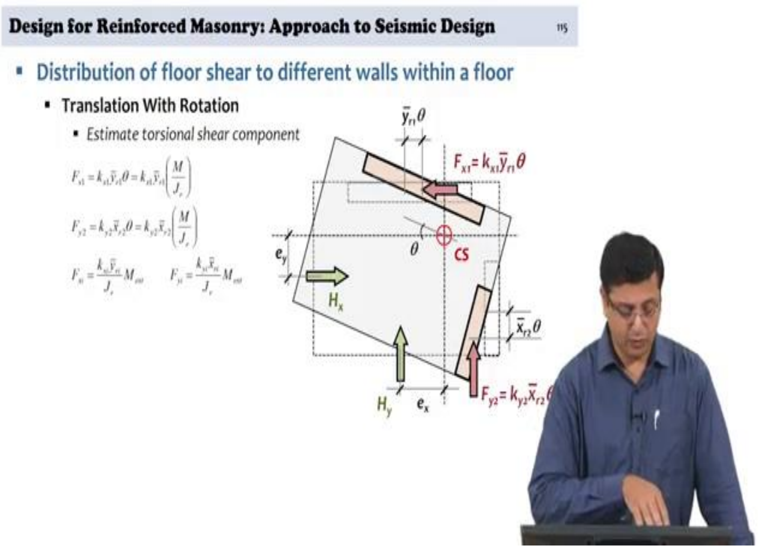

So, the torsional shear component itself $F_{x1} = k_{x1}\bar{y}_{r1}\theta$; that is the that gives us the direction in which the force will act. And therefore, the estimate of F_{x1} can be made in terms of $\frac{k_{x1}\bar{y}_{r1}}{J_r}M_{ext}$. And similarly, F_{y2} can be established; and therefore, you have the estimate of the torsional shears F_{x1} and F_{y2} that is possible from the geometry and the total shear force acting in the system itself.

Where M external was defined earlier as $H_xe_y + H_ye_x$, so the torsional shear components are established. Now, that is not all what is acting on the wall, you have the direct shear component. So, it is translation with rotation. So, the translation gives you the direct shear component and the rotation gives you the torsional shear component and the torsional shear component is what was established here.

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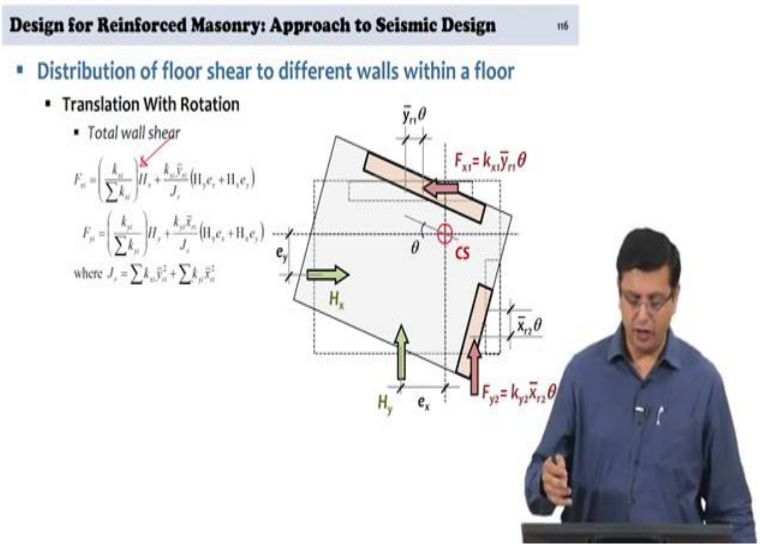

Therefore the total shear that the wall will experience is going to be. So, the shear force F_{xi} in wall 1 or wall 2 would actually be the translational component which comes merely by multiplying the shear force in that direction, in the direction that you are considering multiplied by the distribution factor that we saw earlier.

In the last class we looked at the distribution factor $\frac{k_{xi}}{\sum k_{xi}}$ is the distribution factor in the x direction, stiffness considered in-plane multiplied by the shear force in the x direction gives you the direct shear component plus the torsional shear component that we just saw in the last slide. Similarly, F_{y1} is established with respect to H_y, the shear force in the y direction and the distribution factor in the y direction and the torsional shear component.

So, this basically gives you the total shear that wall 1 and wall 2 will experience, if there is earthquake shaking; which is giving you two components H_x and H_y in the due to the eccentricity between the center of mass and center of stiffness. So, this helps you to establish what is the total shear force expected in wall 1 and wall 2. Once that is clear, then wall 1, if it is a wall with perforations, then you go back to look at how the shear force is going to be distributed between the piers within the wall itself.

And for that we looked at different methods that can give you an estimate of the distribution that should be done. And instead if it is a solid wall, then you will choose an appropriate boundary condition; cantilever typically if it is a solid wall, and the total wall is going to be resisting the shear force in that direction.

So, this is under the consideration that you have rigid diaphragm action. And if your plan configuration does not have eccentricities, it is going to be only translations considering x direction and y direction independently, and you will be able to estimate what are the wall shears and then estimate what are the pier shears. However, if you have rotations, then you need to estimate the torsional shears and add the torsional shears before you go and distribute it among the piers in a given wall. So, this is the, these are the steps that you would actually have to go through to establish the actual shear force acting on the wall and the resisting piers themselves.

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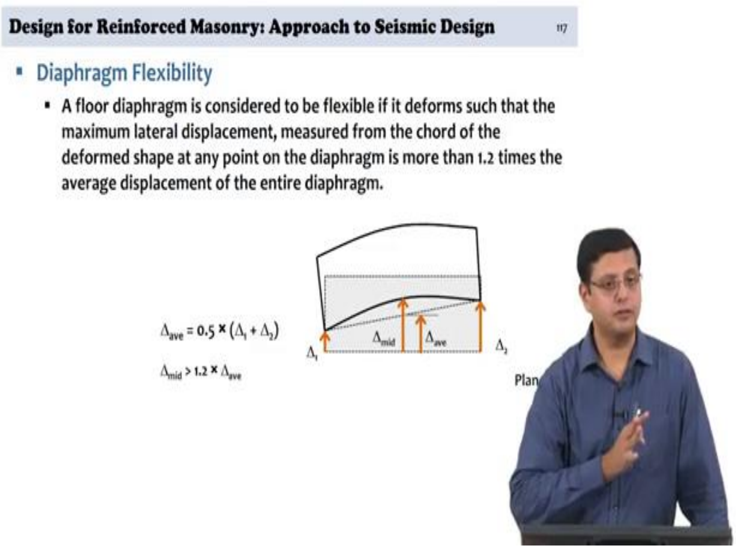

Having said that, I think it is important to examine for a moment, the role of diaphragm flexibility. Now, all this while we have been talking about a rigid diaphragm consideration. So, if you have a masonry structure, which has a reinforced concrete floor slab and roof slab; we are talking of a system that can offer adequate in-plane stiffness of the diaphragm. Mind you it is in-plane stiffness, it is not the out of plane stiffness which is the bending stiffness of the slab.

We are talking of the in-plane deformability and in-plane stiffness of the slab. So, if you take a structure I mean, if you take a system such as a reinforced concrete slab; relatively speaking you are looking at a rigid diaphragm. So, a rigid diaphragm assumption for a reinforced concrete system is acceptable; however, if you look at other types of floor systems, if you have a timber floor.

If you have a timber plus masonry floor, you cannot guarantee rigid diaphragm action in such systems. And therefore, we need to check whether we are falling into the category of definitions of a flexible diaphragm or a rigid diaphragm. So, we need some guidelines on that and it is useful to go back to IS 1893 which defines what is a flexible diaphragm and what is a rigid diaphragm, because you need to quantitatively define that. And be able to say that look I am working with a rigid diaphragm, I can continue the calculations that I have been doing in the previous slides or no it is not a rigid diaphragm we have to be careful, we cannot distribute the forces based on relative stiffnesses anymore.

So, the definition of a floor diaphragm; a flow diaphragm is considered to be flexible; if it deforms such that the maximum lateral displacement, you take anywhere measured from the chord of the deformed shape anywhere, is more than 1.2 times the average displacement of the entire diaphragm. So, you estimate the average displacement, assuming it is going to deform in-plane; that average displacement multiplied by 1.2, if displacement at any point on the diaphragm is more than this value, then you have you do not have rigid diaphragm action ok.

Now, this definition, different codes give more or less similar approaches to estimate whether the diaphragm is stiff or not; this is what is adopted in the IS code. So, if you look at the grey box with the dotted lines, that is the original configuration; but then under the action of lateral forces, the diaphragm is deforming in-plane. With the diaphragm deforming in plane the end displacements of the diaphragm are Δ_1 and Δ_2, they are unequal values; but you get a significant, significantly different displacement that mid span or at any other locations.

So, here particularly we have taken delta mid span and if you take delta average which is nothing, but $\frac{\Delta_1 + \Delta_2}{2}$, then you make a check whether $\Delta_{midspan}$ or any other point on the chord is greater than 1.2 times the average displacement and the average displacement in this case is $\frac{\Delta_1 + \Delta_2}{2}$.

So, this is the check that one would make, and typically when we are looking at reinforced concrete slabs; it would satisfy the sort of a situation, but it also depends on the stiffnesses of the walls. So, it is to be more rigorous, it is useful to make this check; however, when you are dealing with, examples that we had seen in the beginning of this lecture such as Madras terrace floor slabs or jack arch floor, slabs it may not respect the requirement for a rigid diaphragm action.

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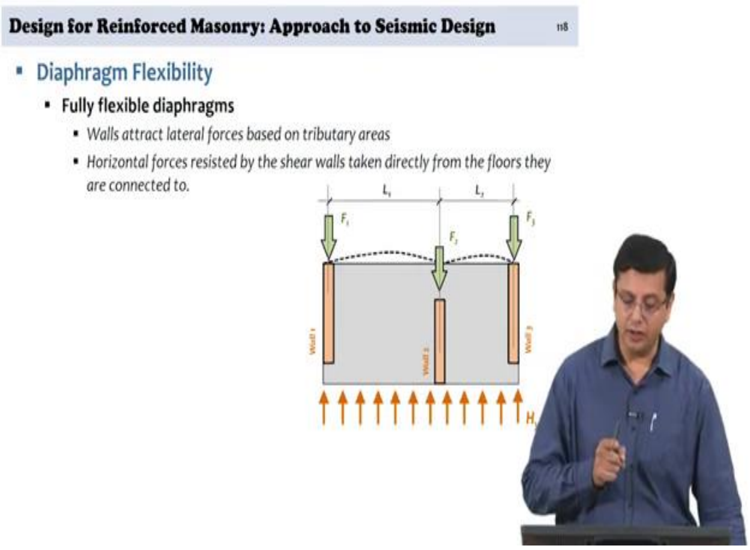

So, if that is so, we cannot actually work with all that we have talked about, the center of mass and center stiffness and the torsional shear; it starts becoming inappropriate as far as flexible diaphragms are concerned. So, if you have a flexible diaphragm how do you go about doing your calculations as far as seismic design is concerned. So, let us assume you have a wall, I mean a floor configuration, a plan configuration where you have three resist walls wall 1, wall 2 and wall 3; we are looking at earthquake action in one direction.

So, H_y there is distributed, uniformly distributed load which is basically the inertial mass acting in the y direction. Now, this it has to be resisted by walls 1, 2 and 3 and therefore, what would be F_1, F_2 and F_3 and how would F_1, F_2 and F_3 be; how would you arrive at

F_1, F_2 and F_3? Because since you do not have the rigid diaphragm action, you cannot use the concept of relative stiffnesses, you cannot use the concept of distribution factors coming from relative stiffnesses.

So, what typically happens in this case; and which is how load transfer happens, is if you have fully flexible diaphragms walls attract lateral forces based on the tributary areas. So, then I need to know, I have the distances between the walls is L_1 and L_2. So, the total force coming on to wall 1 and wall 2 would be distributed based on half of L_1 to wall 1, half of L_1 to wall 2, half of L_2 to wall 2 and half of L_2 to wall 3.

So, you basically have to go the tributary areas, and so you take the floors, look at the total floor force coming there just identify the distribute three areas $L_1/2$ and $L_2 / 2$ and then sum them such that equilibrium is established with respect to H_y. So, it is very different if you consider the condition of rigid diaphragm and estimate the shear forces coming on to the individual walls versus a situation of diaphragm flexibility.

So, of course, in modern constructions we are not often choosing to design with diaphragm flexibility, we are typically going with reinforced concrete floors. And hence this sort of a situation may not necessarily be encountered. So, with that we now have the total framework to be able to from definition of the seismic input to arriving at what should the wall be designed for, the pier be designed for in terms of a combination of shear force, bending moment, and axial force and that is where we began our component design.

We have those forces; we can do in-plane, the flexural wall design, out of plane flexural wall design where we use P-M interaction diagrams, and the design for shear and design for axial force. So, that closes the loop as far as the design from component level to system level is concerned.

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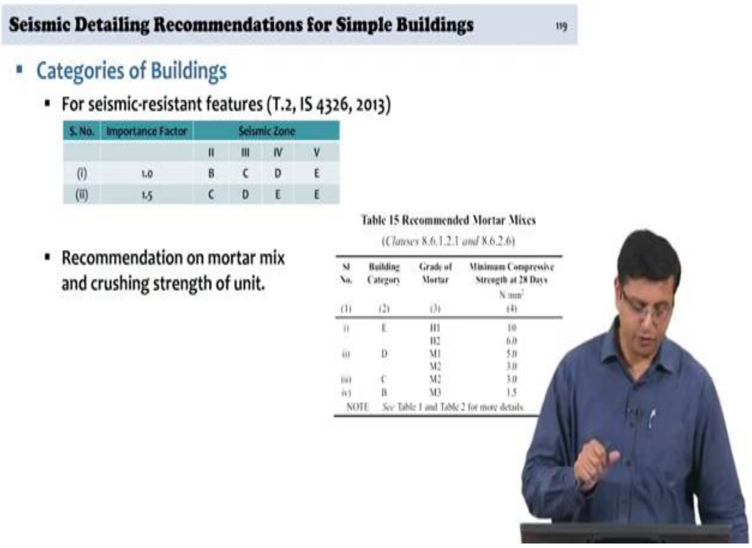

We have come back to the component level to conclude; I am actually going back to where we started from, which is something that we have talked about, but not looked at specific details. So, let us say you want to design a simple building, you want to design and construct a simple building; you are not going to be adopting a rigorous design approach as prescribed by 1905 and particularly the reinforced concrete, reinforced masonry design requirements in NBC.

So, if the building is simple now; what is a simple building? A building is simple; it does not have the vertical and horizontal plan irregularities, it is a small configuration and you have a uniform distribution of shear walls in the x and the y direction. In such a situation, if the configuration is simple, you can go by recommendations of IS 4326 and not have to go through rigorous design calculations as prescribed by NBC or 1905.

So, if you remember the categories of buildings that we were talking about, defined by IS 4326, 2013 depending on the seismic zone 2, 3, 4 and 5 we were looking at importance factors 1 and 1.5 and categorizing buildings as B, C, D and E. So, what IS 4326 then says is ok, if you are designing these simple buildings in within category B, C, D or E; these are the minimum requirements that you must ensure are constructed. So, that you have desirable action, desirable seismic performance; but you are really not getting into quantitatives to check what is the shear force demand on each wall, and whether the wall is designed to take that shear force.

So, really it is prescriptive requirements as far as designing and detailing and construction of simple masonry buildings are concerned, and there are several features which IS 4326 recommends. I am going through the most key features here, which is available to you in the NBC and in IS 4326 and it is essential to be familiar with what these are; and if you are looking at simple buildings, have an idea of how these should be incorporated in such structures.

So, one of the first requirements is on the mortar mix that can be used, such that the whole concept of a simple building is, you are not going through rigorous structural design calculations. And therefore, you must have a minimum set of construction features in it, such that you expect desirable performance under an earthquake. If you were to look at complex configurations, you cannot be sure that these requirements alone will take care of desirable behavior and that is the reason why those structures have to be designed.

So, to begin with, there are recommendations on the mortar mix and the crushing strength of the unit. We have seen that the crushing strength of the unit cannot be less than 7 Mega Pascals. So, IS 1905 starts from class 3.5 bricks, 3.5 MPa, 5 MPa and so on. If you are designing respecting recommendations of IS 4326, you can use only class 7 bricks and a higher, the mortar mix is recommended are again.

So, all the recommendations in IS 4326 will be tied to the category of building, which means the zone in which you are constructing them. So, I have reported here the table, table 15 from the National Building Code, which has reference to what is prescribed in IS 4326 as far as mortar mixes are concerned. So, you can see, if you are looking at higher seismic zone, building categories E and D, you see that we are looking at H 1 mortar, M 1 mortar and up to M 2 mortar; you go down maximum to M 3 mortar, you know that there are L 1 and L 2 mortars as well.

But we are not allowed to use them if you are going to be designing for minimum earthquake resistance and the minimum compressive strength is also prescribed. So, it begins from materials. So, what mortar mix can you use, and what crushing strength of unit is permissible.

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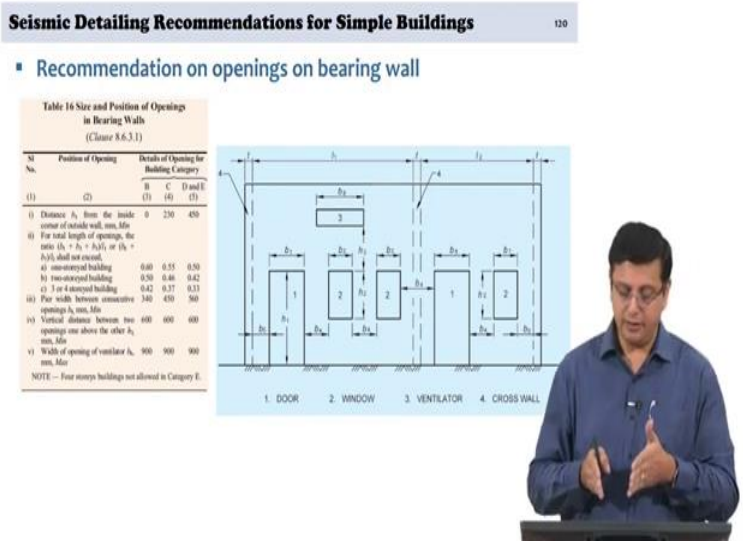

Then the building configuration has to be kept simple right. The moment you have complex configurations, you can have undesirable behavior and that undesirable behavior would require that the resisting elements are checked and designed to specific demand coming on to it.

In order to ensure that that is not required, there are prescriptions on what should be the sizes of openings and what is the allowed distribution of openings; particularly with respect to where the openings should end with respect to the end of the wall. So, if you do not have resisting elements close to the end of the wall, you are creating a discontinuity and possibilities of torsion in the earthquake response. And also, what should be the minimum distances between, what should be the minimum distances between openings.

If you have very little resisting material between openings, it is going to actually work as a larger opening and that is not desirable. So, minimum width between openings and minimum width between the edge of the wall and the opening is prescribed. So, again if you see, table 16 here is actually giving you specific requirements, again tied to the category of building B, C, D and E on what these dimensions and some of these dimensions should be or should not exceed. So, this is the other recommendation that is that one has to adhere to, if it is a simple building with such, with earthquake resistance built in.

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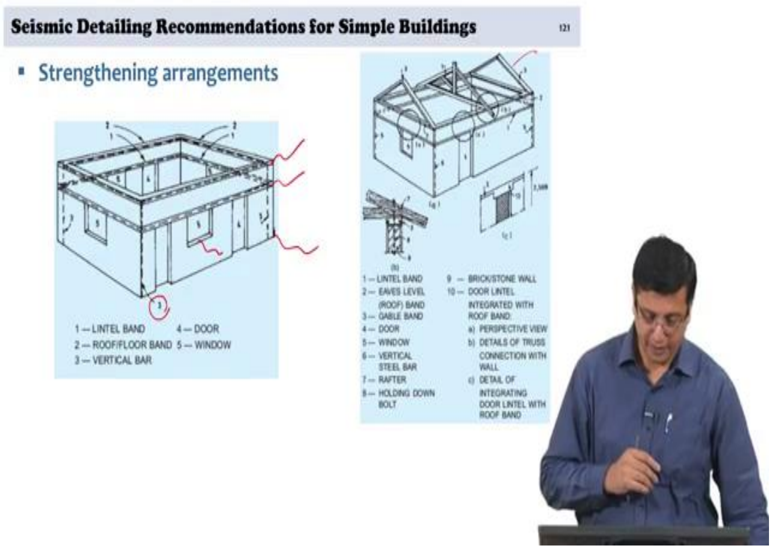

So, the other important aspect is what strengthening interventions must be introduced, right. So, we are talking of good material; if you have good material, there is good performance expected due to the integrity of the material. The second aspect is overall configuration; overall configuration and openings matter as far as overall configuration is concerned there are limitations on it. So, material is taken care of, form is taken care of; third set of recommendations are on seismic strengthening arrangements ok.

So, these are you have the configuration already, these are the things that you must additionally put in place, which will ensure good behavior under the earthquake; which is again prescribed depending on which category of building you are looking at. I have reported here only the two types of buildings that are addressed; buildings with flat reinforced concrete, flat floors, or flat roof slabs; not flat slab, but regular reinforced concrete slabs or structures with lean to roofs or structures with sloped roofs with trusses that are supporting them.

So, if you look at the figure on the left, the minimum requirements are in terms of introducing bands, the seismic bands- reinforced concrete seismic bands. And as you can see there are two bands that are shown in the figure here one is the lintel band, a continuous lintel band and the second is a band that is running at the floor or the roof level. A third band is typically prescribed which is going to be at the plinth level.

So, at any point which is a transition zone, an important transition zone foundation to the resisting wall; at the plinth level, you will have the plinth band. As you go to the top of the openings that is the transition zone, merely because area of cross section of the wall is changing, you have an opening and then you have a continuous resisting element.

So, that is again a transition zone. So, you give the lintel band at the top of the openings, you keep all the openings at the same level, such that the lintel band which is a continuous reinforced concrete band is provided. And finally, where you have a transition between the horizontal resisting element and the vertical resisting element, a horizontal roof band or a floor band is provided. So, minimum three bands are required; and you will be able to have desirable performance in keeping the masonry structure together.

Of course, if you have multi storied structure; if you have two storied, three storied structures; in addition to the lintel band, if possible you should provide the sill band. The sill is this location of a window. Now, the difficulty of prescribing a sill band is that; if you have a door opening, then you would not be able to provide a continuous sill band. So, attempt to provide a sill band wherever it is feasible. So, the best situation would be for able to provide these four bands, plinth band, sill ban, lintel band, and roof band or floor band; if not at least the lintel band and the plinth band, roof band it is a choice very often between providing a slab on beam or a slab without a beam.

So, the lintel band is indispensable, in even in the lowest category of buildings, category B, the lintel band is a must. And it is a continuous lintel band; this should not be confused with the lintel that we are talking of in terms of design. If you are talking of the lintel design, where the opening comes, the reinforcements must take into account, the load vertical load that is coming on to the lintel. So, lintel is designed there; however, it has to be continuous with the band itself.

So, that is the set of strengthening arrangements as far as the horizontal bands are concerned. The other arrangement are these vertical bars that you can see marked here, these vertical bars are provided at the corners of all walls and provided at the ends of all openings. So, these have to occur wherever there are wall corners or junctions of walls when you have a T junction or a H junction or any junction between two load resisting, vertical load resisting walls; these vertical bars are given, these vertical bars are taken continuously from the plinth band connected to the lintel band to the roof band and continues to the next storey, if it has to.

So, these are at the corners of walls, at the ends of opening, so where you have a door opening at the ends this would again come, and wherever there are junctions in walls. If you are now examining a building which has a sloped roof, it is required that where all the other requirements in terms of the bands and the vertical steel if you are talking about are essential.

When you come to the roof element, when you come to the structural system at the roof level, you have a portion of masonry which is seated in this triangular area. Now, if that would be at the crown, that would be the tallest masonry construction that you will get and that is susceptible to out of plane failure. So, there is a requirement that you provide a gable band that is called the gable; you provide a gable band and this gable band will ensure that the triangular portion is not susceptible to out of plane failure.

So, in addition to the other lintel bands if you were talking about, the roof band must ensure that it has a gable band and actually holds the gable from out of plane failure itself. The other aspect that needs to be taken care of, is when you have these roof bands, these roof bands are provided bracings as well, lateral bracings are provided in plan such that the trusses do not move independent of each other. So, these requirements are prescribed depending on the category of the building.

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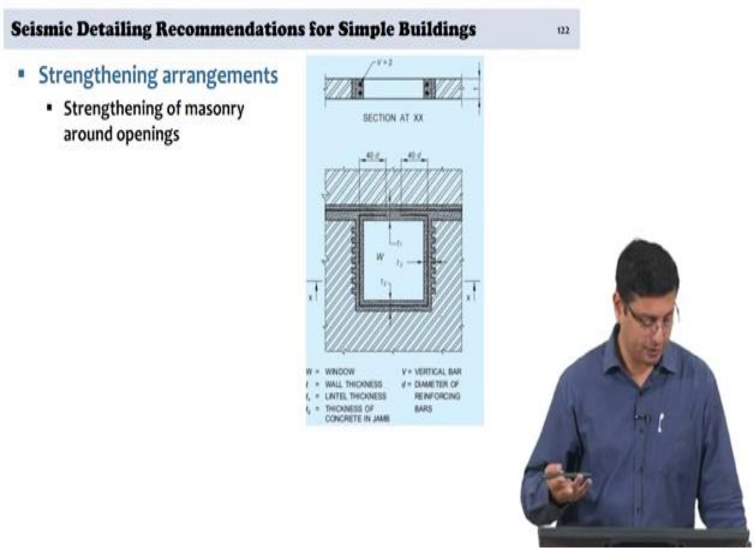

You have another requirement which is particularly in category E buildings, and in category E buildings, category D and category E buildings; when you have openings, it is prescribed that reinforcement is given around the openings.

Typically what happens is, when you have an opening in a masonry wall, the edges of the openings become points of stress concentration, and points where shear cracks can commence. And therefore, having a tensile resisting element running all around the opening, allows for prevention of the crack propagation; and therefore, we are talking of a reinforced concrete element that runs all around the opening. And as you can see, the reinforced concrete element also needs to be keyed into the brick masonry.

So, and that is simple because anyway alternate courses will ensure that you have the keying coming in, with that as one of the one side of the formwork and the other side you create a form work you will get interlocking that will happen in the concrete that you are pouring in. So, this is the other requirement which is strengthening of masonry around openings, and typically adopted in higher seismic zones- category D and category E buildings.

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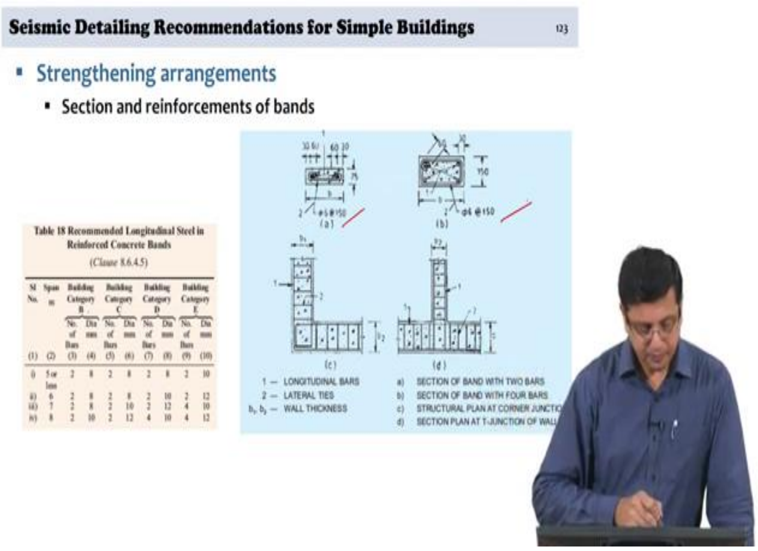

So, the code again then prescribes, as I said this these are all prescriptive, it does not expect you to calculate how much of steel is required; the amount of steel required is prescribed. So, depending on which category of building and looking at B, C, D or E, here for the bands, the code prescribes how much of steel you should be putting in, and these continuous bands you could choose a configuration that is width of the wall to about 75 to 75 mm in height or width of the wall and 150 mm in height.

The two pictures that you see here a section a and section b are 75 and 150 mm; 75 mm will have 2 bars, 150 mm will have 4 bars and the number of bars that are required in each category, number of bars and the diameter of bars are prescribed in the table.

So, this is the arrangement of the reinforcement and the section details of the RC bands. And this is valid for your plinth band, sill band, roof band, and lintel band and this is meant to be a tie element; this not meant to be a flexure resisting system, it is not a moment resisting system, but it is just a tension resisting element to ensure that the masonry is held together under lateral action.

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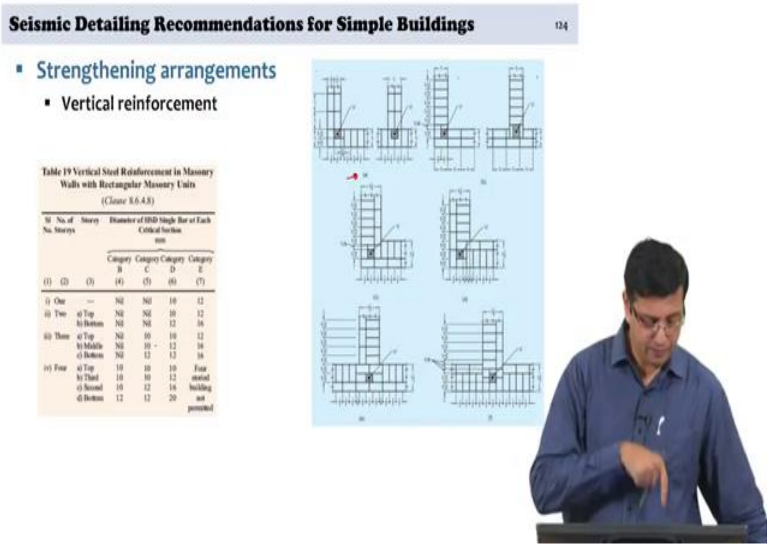

And finally, the vertical reinforcement; as far as a vertical reinforcement is concerned, this particular code caters to any type of masonry construction that you are working with solid units, hollow concrete blocks, the cement blocks, or even the hollow clay blocks.

So, depending on what type of masonry unit you are working with, you may or may not have the provision for vertical steel. So, if it is solid blocks, you have to make voids that run along the entire height, place the steel and grout. And that is what you see here; you have one brick wall, a one and half brick wall, examples that are provided. You have the vertical steel coming at the junction of walls or the wall corners. So, you look at a, b, c and d, e and f, you can see the layout prescribed for alternate layers.

So, alternate courses must have vertical stagger and that is why course 1- 3- 5, course 2-4-6 would have to have different configurations. And so, what you see for each example, each case, is the odd course and the even course. So, you can see that, when you have a one brick thick wall, the prescription of how you have to break the brick such that you can create a cavity sufficient enough to place one vertical bar and grout the void is shown here and then you have the alternate course.

So, odd course, even course for an L junction; odd course, even course for a T junction for a one brick wall, and then for a one and a half brick wall and so on. So, you see that depending on the building category B, C, D or E; the number of reinforcement, the size of the reinforcement bar. The prescription as far as these vertical tie columns; again it is not a reinforced concrete moment resisting column, it is a tie column; for these tie columns prescription is one bar and you have to grout the void and you have the total the vertical bars connected to the steel in the horizontal bands. And therefore, you have continuity between all the steel reinforcement; these are really confining the masonry structure.

So, with that I conclude the component that looked at design, including the seismic design aspects; so we concluded by looking at the prescriptions, when you are not actually doing structural design and that is in IS 4326. We will look at one and two storey, one and multi storied building design in the coming lectures.