Design of Masonry Structures
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Module – 04
Lecture – 33
Design of Masonry Components and Systems Example - I

Good afternoon. So, this week we will look at four illustrated examples that deal with Design of the Components; but we will examine how from the system level to the component level, we will arrive at the design forces for different conditions, for a single storied structure, for a multistoried structure. And when we deal with single storied structures, I would like to look at both a condition where you have a flexible diaphragm and a condition where you have a rigid diaphragm.

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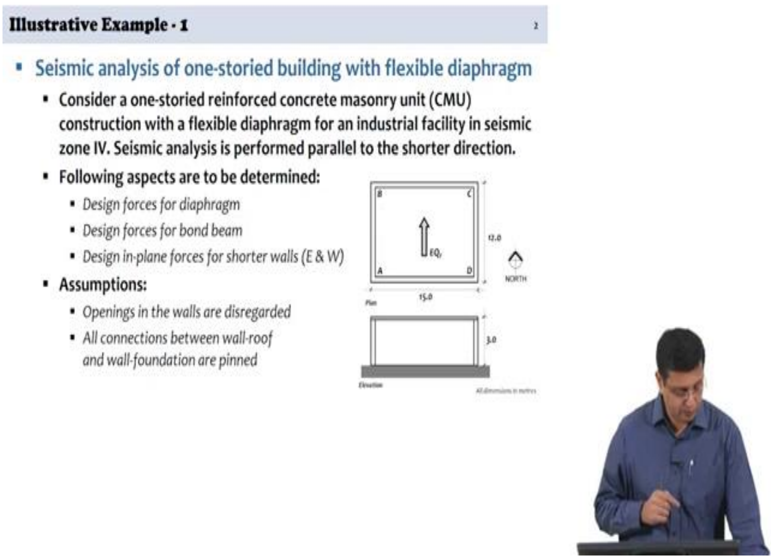

So, the first illustrative example deals with the Seismic analysis of a single storied building and this building is provided with a flexible diaphragm. So, we are looking at a structure probably with a timber diaphragm or a metal plus timber composite could also be classified as a flexible diaphragm.

So, what are the aspects that we need to consider, primarily when we deal with flexible diaphragm. A building with a flexible diaphragm is what we will be looking at in this particular example; you will also arrive at the design forces that are required for the masonry walls and then examine if the dimensions are adequate or do we need reinforcement and so on, ok.

So, let us consider one storied reinforced, concrete masonry unit construction. So we are looking at CMU construction, so that if there is reinforcement that is required, you can provide the reinforcement within the pockets. This is a building with a flexible diaphragm and we are looking at a structure that is an industrial facility.

So, it could be a warehouse as an industrial facility, it is located in seismic zone IV. And we are going to be looking at in this particular example, seismic analysis that is performed parallel to the shorter direction of the building that is considered. So, it is a simple construction, it is rectangular in plan, 15 meters the longer dimension and 12 meters is the shorter dimension and earthquake analysis is now being performed, considering earthquake action parallel to the shorter direction.

So, just to familiarize ourselves with this particular example, you see the north direction marked along with the plan. And therefore, walls BC and AD are the northern and southern walls and wall CD and wall AB are the eastern and western walls. We can have another nomenclature for these walls, considering that the earthquake action is in the y direction or parallel to the shorter direction; let us then look at wall A D and wall B C as the out of plane walls and wall A B and wall C D as the in-plane walls.

So, we will be using this nomenclature, in-plane versus out of plane and the northern, southern, eastern and western walls in our calculations. The total height of the structure is 3 meters and it is provided with a flexible diaphragm ok. What are the aspects that we need to estimate in the problem? What are the design forces for the diaphragm? So, the way we have been examining the different components within the design of masonry structures, we have been addressing primarily walls, flexure and shear behavior of walls. Diaphragm is something that we have really not examined from the point of view of design forces for a diaphragm.

So, this is right stage to examine, how do we estimate the design forces for a diaphragm; a bond beam has to be provided, we are looking at seismic zone IV. And this is a continuous beam that is provided at the level of the roof, between the roof and the walls, which is actually an element that is collecting the forces and transmitting them to the walls.

So, we need to look at what are the design forces for the bond beam and finally, let us look at the shorter walls, the East and the West walls and what are the in-plane forces for the shorter walls, for which the walls A B and C D would have to be designed as in-plane shear walls. So, these are primarily the aspects that we are going to be considering. What are the basic assumptions that we are making here?

For simplicity we disregard the openings in the walls, there may be door openings, there may be some ventilators; but for simplicity I am going to be assuming that, we are looking at a complete wall panel in A D, A B, B C and C D of the dimensions prescribed. We are also assuming that the connections between the wall and the roof and the wall and the foundation are pinned-pinned. So, this is additional information that you have, at the interface between the wall and the foundation you could have the damp proofing course.

So, assuming that boundary condition is pinned is an acceptable assumption; here in this particular case we looking at a flexible diaphragm and therefore, the restraints offered by the diaphragm to the wall are negligible and therefore, we can again assume that the wall roof boundary condition at the top of the wall is pinned.

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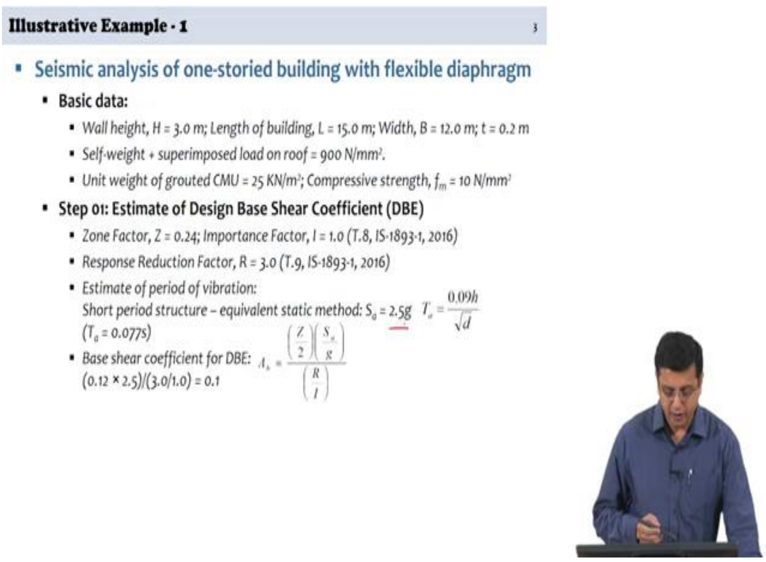

So, with these basic assumptions, let us proceed looking at the other quantitative information that is available; the height of the wall is considered to be 3 meters, the length of the building, the longer side is 15 meters, the shorter side width B is 12 meters.

And we are looking at; to begin with you need some dimensions on the table for your calculations.

So, let us assume that the thickness of the wall is 200 mm hollow blocks, concrete masonry units 0.2 meters in thickness. It is also given to you that the self weight of the diaphragm and the superimposed load on the roof, so the dead plus the live load is equal to 900 N/mm^2.

For calculations we would require, for the self weight of the walls, we can assume that the unit weight of the grouted concrete masonry units is going to be about 25 kN/m^2, that is what you would use for concrete. And so grouted hollow blocks, hollow concrete blocks, it is good to assume a value close to concrete. The compressive strength of the concrete masonry units is to be assumed as 10 MPa.

So, with these information and the assumptions that I talked of in the previous slide, what do we do first? We are talking of a building sitting, an industrial building sitting in seismic zone IV; so we are talking of being able to estimate the design base shear coefficient first, we will do that for the design basis earthquake. As you know the code prescribes the zone factors for the maximum considered earthquake, the MCE for the design basis earthquake we take the zone factor and divide that by 2.

So, we are looking at the design basis earthquake of Z/2. So, the zone factor here, Z is 0.24 zone IV, an importance factor of 1; this is with reference to Table 8, IS 1893 part 1, 2016 we are looking at an industrial building importance factor, it is an ordinary building and therefore importance factor is 1. Response reduction factor, I intend to reinforce this construction, so let us provide the R factor that corresponds to reinforced masonry, so I take a response reduction factor R of 3 and this is again with reference to Table 9 IS 1893 part 1, 2016, ok.

So, if we have to estimate the design base shear coefficient, we need an estimate of the period of vibration of the structure. The period of vibration of the structure can be estimated using simple code-based formulae here. If I were to use the fundamental vibration period is equal to $\frac{0.09h}{\sqrt{d}}$; d being the dimension of the structure in the direction of the earthquake and h being the height of the structure. We can work out the fundamental period of the structure as being 0.077 seconds; so you are looking at a short period structure and we are really carrying out a simple analysis here.

So, it is really equivalent static method-based analysis and the code prescribes that for a short period structure; if you are using equivalent static method, then do not look at the increasing branch of the response spectrum, acceleration response spectrum; instead assume that for the short period, these spectral acceleration is equal to the value at the plateau and therefore, the spectral acceleration is considered to be 2.5 times g.

So, the base shear coefficient for the design basis earthquake is calculated Z/2 into 2.5 divided by the response reduction factor R by I, 3 divided by 1 and I get a base shear coefficient equal to, base shear coefficient A_h is equal to 0.1 for this particular example.

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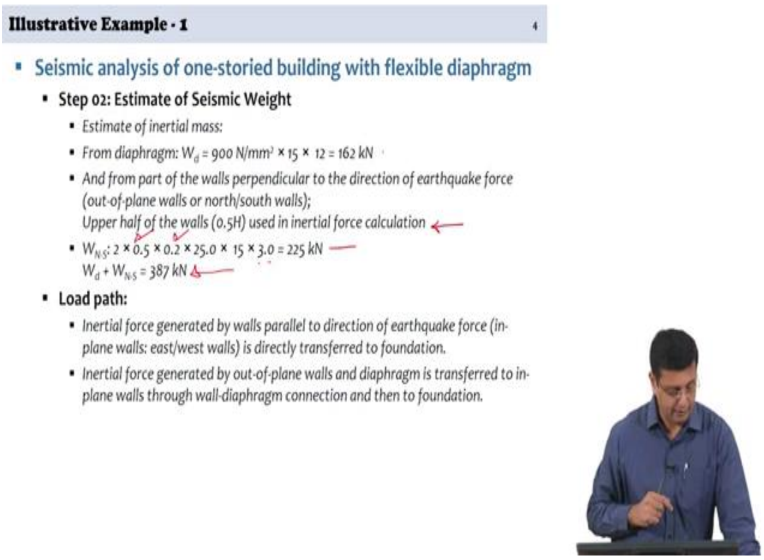

I need to estimate the seismic weight to be able to calculate the base shear coefficient. So, the second step is to estimate the seismic weight, we need an estimate of the inertial mass in this particular case. So, what is the information that we already have on hand and what further calculations can we make. We know that from the diaphragm we said that, the self-weight of the diaphragm plus the superimposed loads is about 900 N/mm^2.

The diaphragm itself is 15 meters x 12 meters; and therefore, the total weight of the diaphragm is about 162 kN and that is what you see here. Now, this is a single storied structure and the single storied structure is now being subjected to an earthquake. So, what part of the total structure would you consider as being contributing to the inertial mass? So, normally we look at one half of the height of the walls in calculating the inertial force, particularly when we are looking at the ground storey.

The base of the ground storey structure is seated directly on the ground and we only take the upper half of the structure in the calculation for the floor weight itself, so we take 0.5H in the calculation of the inertial force for this storey. Now, we have the walls in the direction perpendicular to the seismic action, which are the out of plane walls and these were the longer walls, the north wall and the south wall of the structure right.

So, to be able to calculate the total inertial mass, in a moment we will try and understand what is the load path in the structure. So, what is the weight of the north wall and the south wall, we take one half of the height of the walls, so 0.5 times H; 0.5 times H is 3 meters here. And there are two walls, we have taken the dead weight of the wall with respect to the density of CMU as 25 kN/m^3 and 0.2 is the thickness of the wall itself, 15 is the length of the wall.

So, the weight of half of the northern and southern walls is calculated here; we then have the total weight, total inertial weight. Now we have calculated the weight of the diaphragm and the superimposed weight coming onto the diaphragm and then one half of the walls perpendicular to the direction of the earthquake force and that sums up to 387 kN.

Now, it is important why I have assumed only the weight corresponding to the diaphragm and the walls perpendicular to the direction of the earthquake action in calculating the seismic weight of the structure. Now, if you actually look at in the earthquake the load path in the structure; then we have inertial forces generated, you have walls parallel to the earthquake, you have walls perpendicular to the earthquake.

The inertial force generated by the walls parallel to the direction of the earthquake force, these are the in-plane walls that we are talking about which were the east and the west walls; this is directly transferred to the foundation. So, this the in-plane walls, ground storey in-plane walls, this force is getting directly transferred to the foundation. So, I do not use that in estimating what the inertial force is, which is going to come onto the diaphragm.

So, that is the reason why the inertial force corresponding to the in-plane walls has not been considered in the calculation above. What about the inertial force that is generated by the out of plane wall? The out of plane wall also generates an inertial force; the diaphragm is also a weight, it is also carrying some superimposed loads and so it generates an inertial force. And so, the inertial force generated by the out of plane wall and the inertial force generated by the diaphragm is then transmitted to the foundation through the in-plane walls.

So, to be able to estimate the inertial force which the in-plane walls will actually have to counteract, we estimate the inertial force from the out of plane wall and the diaphragm as calculated in the in this slide, ok.

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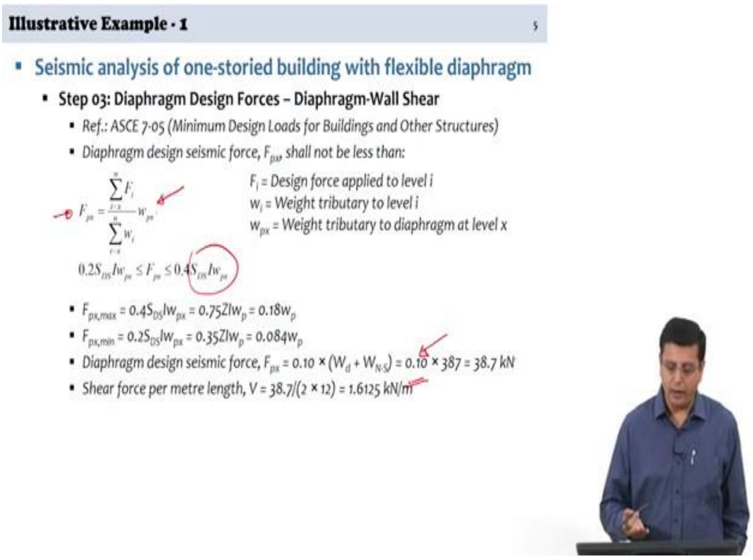

Now this is then transferred to the, this is the design force because the load path is going to be the inertial force generated by the out of plane walls, is carried by the diaphragm and then transfered to the in-plane walls. So, the diaphragm now has to be designed for the sum of the inertial force generated by itself and the inertial force generated by the two out of plane walls, the northern wall and the southern wall. So, that is the diaphragm design force, that is then transmitted to the in-plane shear walls.

So, how do you estimate the diaphragm design force? Is it just enough if you estimate the inertial force? No, there are recommendations, because now if you were to look at a multistoried structure, how do you go about calculating what is the diaphragm design force or if it is a single storied structure how does that work out. Our code currently does not have any specification, any specific guidelines with respect to estimate of the diaphragm design forces. So, I will make reference here to the ASCE 7-05, which is ASCE 7 basically deals with minimum design loads for buildings and other structures, ASCE is American Society of Civil Engineering.

So, what does the ASCE 7 prescribe? The diaphragm design, seismic force notation here is F_{px} shall be estimated, as this equation given here. So what you have in this expression?

$$F_{px} = \frac{\sum_{i=1}^{n} F_i}{\sum_{i=1}^{n} w_i} w_{px}$$

However, it is prescribed that F_{px} is within certain bounds, lower bound and an upper bound and here the lower bound is prescribed by ASCE as 20 percent of the spectral acceleration level corresponding to the short period structures into the importance factor into w_{px}, which is again the tributary weight at the level x of the diaphragm that you are making calculations for. That is the lower bound and the upper bound is 40 percent of the same estimate S_{DS} Iw_{px}. So, as I said S_{DS} is the spectral acceleration corresponding to a short period structure.

Now, S_{DS} is with reference to the design basis earthquake and in the ASCE, the ASCE 7 the design basis earthquake is considered to be two thirds of the maximum considered earthquake and this is a difference with respect to our code. In our code the design basis earthquake and the maximum considered earthquake are related as, design basis earthquake is one half of the maximum considered earthquake, so that difference exists.

So, you need to convert that to the corresponding estimate for the IS code; but again I would like to point out here that the spectral design and the spectral acceleration corresponding to a design basis earthquake and a maximum considered earthquake in the ASCE 7 is coming from a probabilistic framework. Probabilistic consideration; whereas, in the IS code 1893, it does not have a probabilistic basis, so a one to one correlation is strictly speaking, not possible.

However, let us have an upper bound and lower bound with respect to the IS code as well and the upper bound is considered to be 75 percent of Z into I importance factor into w_{px} weight tributary to diaphragm at level x. And similarly F_{px} minimum is 35 percent. So, with that we will be able to estimate what is $F_{px,max}$ and $F_{px,min}$ in our case and compare it to the value F_{px}, that is estimated based on the design shear force at a level and the weight tributary to level i multiplied with the weight tributary to the diaphragm at the level where the diaphragm forces are being calculated.

In this particular case, the diaphragm design seismic force F_{px} works out to be 0.1 and 0.1 into the inertial force 387 would give us about 38.7 kN. This value you see is 0.1 and it lies between the upper bound and lower bound $0.084w_p$ and $0.18\ w_p$ and hence we can take the value that we get directly from the expression.

Now, 38.7 kN is the diaphragm shear force, which then is going to be transferred to the two shear walls, our in-plane walls which are of length 12 meters. And therefore, the shear force per meter length, we have 2 walls is 38.7 kilo Newtons divided by 2 into 12 meters, which is the length and therefore, we are looking at a shear force per meter length of about 1.16 kilo Newton meter ok.

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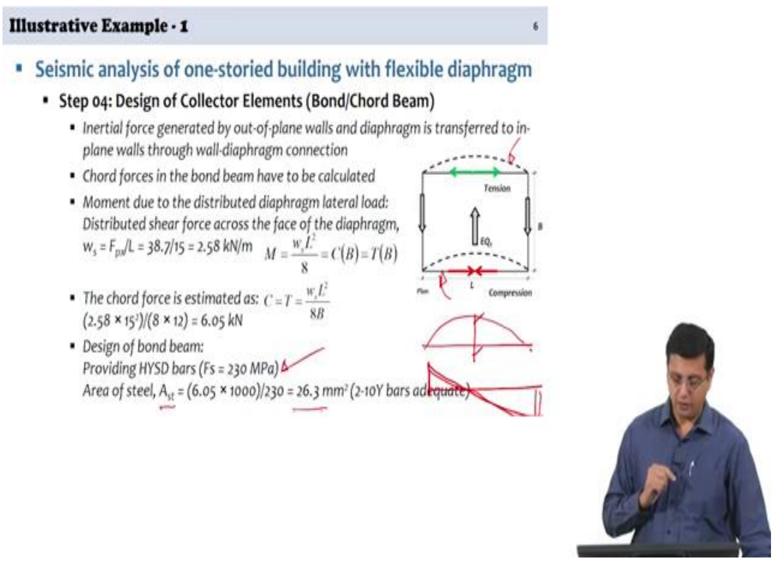

So, that is the shear force. So, that was the first part of the problem that we had to estimate, what is the shear force coming on to the, what is the shear force for which the diaphragm has to be designed and what is the shear force that comes on to the in-plane wall, because of the inertial forces generated by the diaphragm and the out of plane walls.

Now, having estimated that, let us go on to the next stage which is basically, how is this shear force generated by the diaphragm and the out of plane wall, collected and sent to the in-plane walls. So, you have basically in between these two resisting; one that creates the demand and the other which is going to be the resisting mechanism which is the in-plane wall. You have to have a collector element which is our bond beam, also referred to as the chord beam.

So, we need to design the collector element which is the chord beam here. So, the inertial force is generated by the out of plane walls and the diaphragm is transferred to the in-plane wall through the wall diaphragm connection. And the wall diaphragm connection is now the chord beam. So, what are we talking about; we have the building, the diaphragm, length L and width B and this is subjected to earthquake in the shorter direction and the diaphragm is going to deform.

When the diaphragm deforms, now we are going to have the flexible diaphragm deforming; which means the collector element; which is a bond beam that is going to be running all along the peripheral walls is the chord beam, which depending on the direction of the earthquake action in the out of plane wall will have these walls because of the deformation. This chord beam will be subjected to tension; whereas, the chord beam in the opposite direction will be subjected to compression.

So, you can look at the diaphragm as a deep beam as it deforms, you have in it is plane and when it deforms it has tension on one edge and compression on the other. And we can make some simplified assumptions to estimate what these tensile and compressive forces are and then that is the design force for which the chord beam, which is sitting on the northern and southern walls is going to be subjected to. So, this becomes the design force which we will use for designing the chord beam in the out of plane bending direction.

So, that is what we are doing, calculating the chord forces in the bond beam. And how is that going to be done, as I said we take the shear force that the diaphragm is designed to take; take the shear force at the diaphragm is designed to take and then calculate the moment generated in the chord because of the deformation of the diaphragm itself. So, the distributed shear force across the face of the diaphragm, we calculated the design force of the diaphragm as 38.5 kN and the length of the diaphragm, perpendicular to the direction of the earthquake action is L which is 15 meters.

So, the distributed shear force w s is nothing but 38.7 divided by 15, which is 2.58 kilo Newton per meter. So, now, let us look at this diaphragm which is bending, a uniformly distributed load is acting in-plane of the diaphragm and the diaphragm is bending. You will have a bending moment for the diaphragm and you will have a shear force for the diaphragm.

Now, we can estimate the bending moment and the shear force and that is what the chord beam is going to be subjected to. So, uniformly distributed load, parabolic distribution, maximum moment at the center $\frac{w_s L^2}{8}$. Similarly, you will have the shear force acting on the chord beam, on the beam which is going to be $\frac{w_s L}{2}$. So, the moment $\frac{w_s L^2}{8}$ is nothing but, the compression force and the tension force acting on the chord beams.

And therefore, we can estimate the compression force or the tension force as $\frac{w_s L^2}{8B}$, which is what you get here. We estimate the compression force or the tension force to be equal to 6.05 kN. And therefore, this chord beam has to be designed to be able to carry a compression or a tension force of 6.05 kN. We are looking at providing HYSD bars and therefore, the permissible tensile stress of the steel is 230 MPa; F_s is 230 MPa.

Therefore, the area of steel to counteract 6.05 kilo Newton of tension or compression in the chord beam is calculated here. A s t works out to be about 26.3 mm^2. We provide 2 bars, 2-10 mm bars and as you can see, we will have tension developing or compression developing depending on the direction of the earthquake action. So, 2 bars are sufficient, but you can have reversal and therefore, you are going to be giving 2 on each face of the beam cross section itself. So, we will have 4 bars in the cross section of the chord beam.

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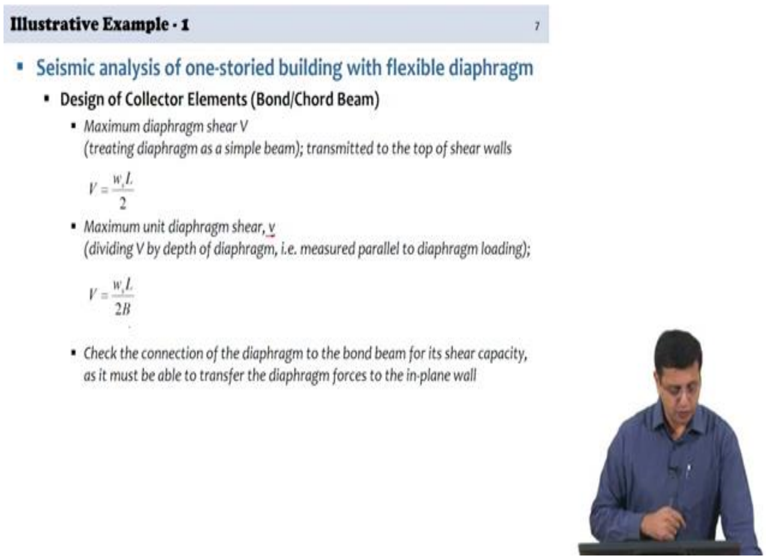

So, that is as far as the chord beam design is concerned. Now we will have to look at the maximum diaphragm shear. Again, we have basically treating the diaphragm as a simple beam and this is the shear force which is transmitted to the top of the shear wall which is nothing but the distributed force w_s into the length 15 meters divided by 2 is the shear force on the, shear force transmitted to the top of the shear walls.

Now, the maximum unit diaphragm shear, unit diaphragm shear v, is basically going to be equal to the shear force divided by the depth, the shorter direction which is 12 meters parallel to the diaphragm loading direction. So, w s into L divided by 2 into B is what the unit diaphragm shear force that we are really looking at, coming on to the in-plane walls; east and the west walls.

Now, we need to ensure that the connection between the diaphragm and the bond beam is adequate for transferring this amount of shear capacity. It should be able to transfer the diaphragm forces to the in-plane wall without having a failure; otherwise um, you will not have transfer of the forces to the in-plane wall and then to the foundation. So, this is a check that needs to be carried out.

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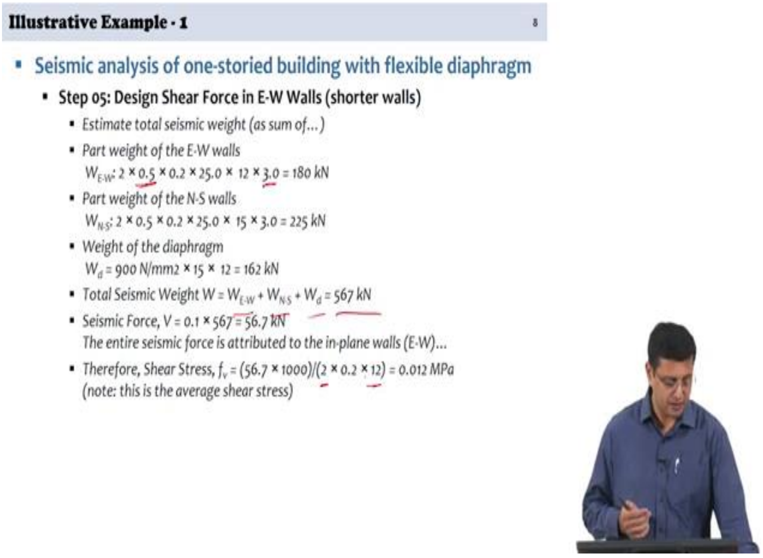

So, the design shear force in the east west walls; the total design shear force in the east west walls or a shorter walls, we need to estimate the total seismic weight. So, now, you have the self-weight of the east west walls. So, part weight of the east west walls, again we are taking one half of the height, half of 3 meters we have part weight of the north south walls. The weight of the diaphragm and the superimposed loads which will totally make the inertial force that the inertial force that is then coming on to the in-plane walls.

So, the total seismic weight in this case; east west wall plus the north south wall component plus the diaphragm weight a total of 567 kN comes on to the in-plane walls. The seismic force; here seismic weight is calculated as 567, seismic coefficient, design seismic coefficient is 0.1 into 567 and 56.7 kN is the seismic force that is going on to the in-plane walls. We need to design the in-plane walls for this level of shear force.

The shear stress level therefore is, the shear force that we have estimated in this manner 56.7 kN divided by 2 walls of 12 meters length and cross section 200 which we assumed for the CMU blocks. And therefore, the shear stress that we have estimated here is an average shear stress which is of the order of 0.012 MPa, which is really very low. So, one thing that could be done is since we see that the shear stress is rather low, you could go back and say I used a response reduction factor 3; assuming that I will be looking at a reinforced masonry construction.

Now, if I were to design this as an unreinforced masonry construction; of course, providing minimum steel prescribed by the NBC, then I cannot use a response reduction factor of 3. I would have to change that, come back and look at what the shear stress is and check if the shear stress is lesser than the permissible shear stress for the problem.

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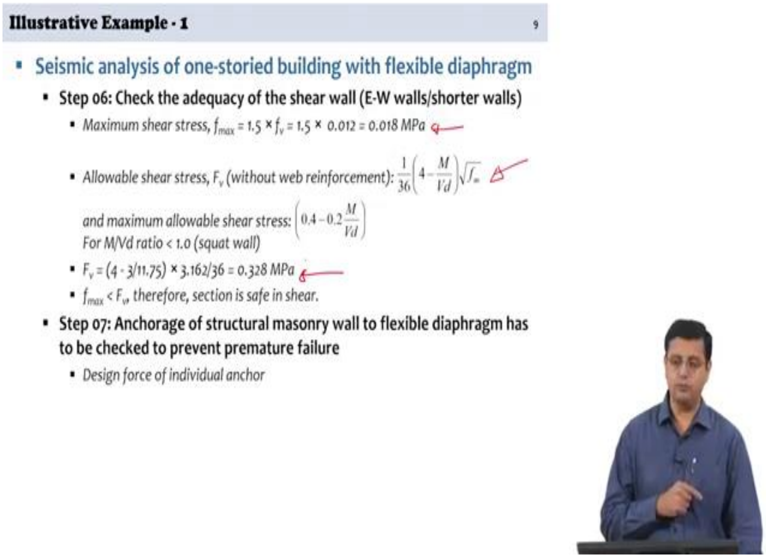

So, we need to check in the final step, the adequacy of the shear wall, the shorter walls. So, the maximum shear stress; we calculated the average shear stress in the previous slide. The maximum shear stress is about 1.5 times the average shear stress and therefore, 0.018 MPa is the maximum shear stress expected.

Now, let us look at a situation where we are not providing any web shear reinforcement. What the allowable shear stress would be; if you remember the prescriptions based on the M /V d ratio, aspect ratio of the wall. We can estimate what the allowable shear stress f_v is; it is a function of the moment to shear ratio and the compressive strength of the masonry. So, the allowable shear stress is estimated. We also have an estimate of the maximum allowable shear stress in this case and for a squat wall which the wall is, it is 3 meters in height and one point and 15, 12 meters in length.

We calculate for the given M/Vd ratio, what the allowable shear stress is and the allowable shear stress if you are reinforcing, assuming that minimum reinforcement will be provided, web shear reinforcement is not going to be provided. We have, it is because it is a squat wall; we have adequate allowable shear, shear stress in the wall; whereas, the maximum shear stress is rather low. f_{max} is less than f_v therefore, the section is safe in shear.

So, the other thing that we need to also look at is, I said that the connections between the bond beam and the wall are important in the in-plane walls; but also the connection between the bond beam diaphragm, the bond beam and the out of plane walls is important. And therefore, the anchorage of the structural masonry wall to the flexible diaphragm should be checked for premature failure. If, that fails then the, you will get a local mechanism which is what you want to avoid. You want to have a global mechanism as intended by the design, because you are ultimately attributing the shear stress from the entire structure to the in-plane walls.

So, this needs to be checked and you have to provide adequate anchorage size and numbers to ensure premature failure does not occur. Therefore, the design force for each individual anchor has to be arrived at and ensure that it is adequately designed and detailed. So, that is the first illustrative example; we basically looked at, what is the seismic force that the diaphragm will be attracting; how much and how is that transferred on to the in-plane walls, that is where the collector beams, the collector or chord beams come into picture.

We need to design them; such that they are they perform adequately under the expected chord tension and chord compression. And finally, what is the total shear force that the in-plane walls have to be designed for, and you must also go and check the connections that you have adequate anchorage and those individual anchors which is the anchoring of the diaphragm to the walls also have to be checdked.