Design of Masonry Structures
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Module – 04
Lecture – 35
Design of Masonry Components and Systems Example – III

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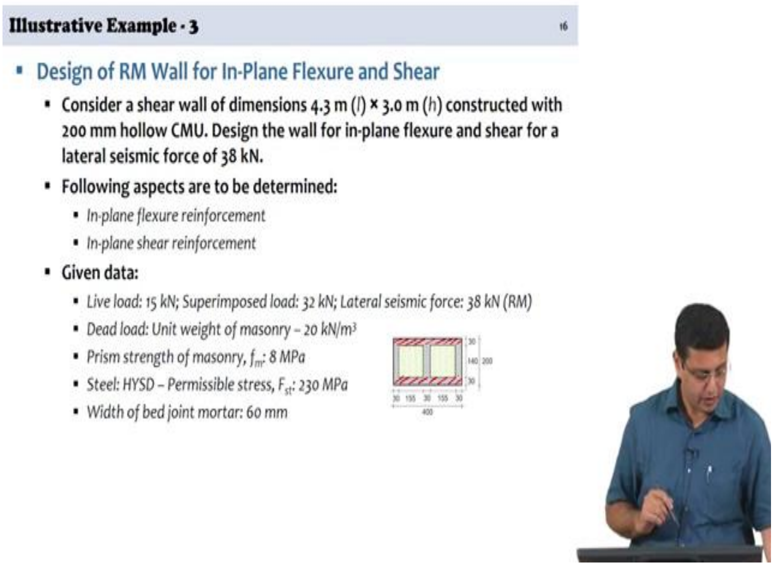

Ok, 3rd illustrative example that we are going to be looking at deals with the in-plane flexure and shear design of a reinforced masonry wall. So, this is specific to component design that we will be looking at particularly for in-plane shear, in-plane flexure and in plane shear. So, let us consider a shear wall of dimensions 4.3 meters in length and 3 meters in height, it is constructed with the hollow concrete masonry units of thickness 200 millimeters. We are designing the wall for in-plane flexure and shear the lateral seismic force considered here is 38 kN.

Now, this seismic force corresponds to a reinforced masonry wall and of course, it depends on the R factor that has been used to arrive at the seismic force corresponding to this level in the wall. So, this is lateral seismic force which we can use for a reinforced masonry wall, if you were to design this as an unreinforced masonry wall or check if an unreinforced masonry wall is feasible, then the lateral seismic force should be different because the R-factor that you use for an unreinforced masonry typology will be different.

So, what are the aspects to be determined; the in-plane flexure reinforcement and the in-plane shear reinforcement. The data that is available with us, there is a live load acting on this wall; you can assume that these calculations have been made on a building system and then the live load acting on the wall is provided to us.

A superimposed load of 32 kN also acts on the wall and the lateral seismic force corresponding to the reinforced masonry typology will work out to 38 kN in this wall. To estimate the self weight; the dead load due to the self weight the unit weight of masonry can be considered to be 20 kN/m^3 and the prism strength of masonry is 8 MPa.

The permissible stress for steel considering HYSB bars is 230 MPa and hollow block construction the width of the bed joint mortar that we will be considering in our calculations is 60 mm. So, we are basically looking at 30 + 30 which is the shaded portion that you can see on your slide. So, this is the width of the bedded mortar 30 + 30 that we are going to be considering in the calculations.

And that is a typical assumption, the web is typically not mortared; for ease of construction that portion is normally not mortared and even if it is mortared you can neglect the contribution of that area in a hollow block construction. Part of the cavities may be filled with your grout and part may not be.

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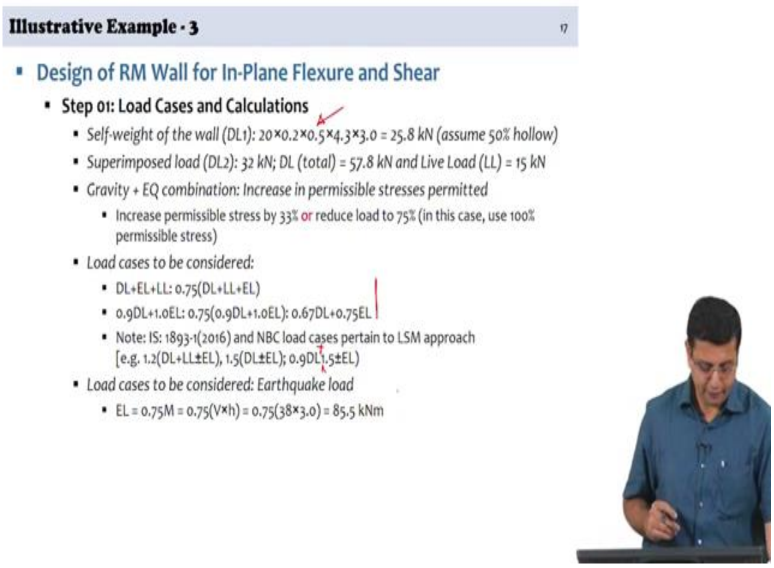

So, let me come to the load cases; calculations of the load cases and the combinations thereof. So, Self weight of the wall is one of the dead loads that you have to consider; this is a hollow block construction let us assume that 50 percent of the blocks are retained as hollow. So, 20 kilo Newton per meter cube into the width of the wall 0.2 meters, 50 percent considered to be hollow and the total length of the wall, and height of the wall 4.3 and 3 meters respectively. We also have a superimposed load acting on the wall 32 kN therefore, the total dead load is 32 kN + 25.8 kN = 57.8 kN here, and there is a live load or an imposed load of 15 kN acting on the wall.

We are looking at seismic design and therefore, we are going to be looking at an earthquake plus gravity load combination and under this combination, increase in permissible stresses is permitted. So, you should be looking at how permissible stresses, can be increased either you increase the permissible stresses in your calculations by 33 percent in this earthquake plus gravity load combinations or you have the possibility of taking that to the other side of the equation and reduce your loads to 75 percent and continue to use a 100 percent of the permissible stress; not 133 percent.

So, that is more convenient to do because you can uniformly apply this to all your loads and load cases including the combinations. So, we will reduce the load to 75 percent in our calculations to account for the increase in permissible stresses which is permitted due to the combination. What are the load cases we can consider? Ok, typically these are the two load cases that we should consider; one is a situation where you have the dead load, you have the beneficial effect of the live load and you have the earthquake load.

So, it is the gravity plus earthquake combination including the positive effect of live load and we are taking 75 percent of the load because of the statement that I made earlier. So, 0.75 (DL+LL+EL) is our first load case.

And a second load case is primarily to account for effects where they could be decompression to an extent in the structure and therefore, we look at a reduced dead load and we do not consider the live load. So, the load case is 0.9 DL + EL, in this case considering 75 percent of the total load we have the load case as 0.67 DL+ 0.75 EL.

At this stage I just would like to point out that these two load cases that we are looking at are appropriate for a working stress approach; however, if you were to consult IS 1893 part 1 2016 or the 2016 version of the national building code, the load cases stated they pertain to the limit state approach. Where you will see that the factors prescribed for the load combinations, load cases in the load combinations is coming from limit state approach.

For example, you are required to do checks for load combinations of 1.2(DL+LL±EL), 1.5(DL±EL), a case of decompression happening where you have 0.9DL ± 1.. So, this is not applicable for the working stress approach that we use for design of reinforced masonry and design of masonry as such.

So, a word of caution that these we will penalize the design if you were to apply to a working stress approach. For the earthquake load we will take 75 percent again of the earthquake load; so, the shear force into the height will give us the overturning moment 0.75 x 38 kN was what was prescribed for the design of the wall; prescribed lateral force for the design of the wall height 3 and therefore, we are looking at a moment, an overturning moment of 85.5 kNm.

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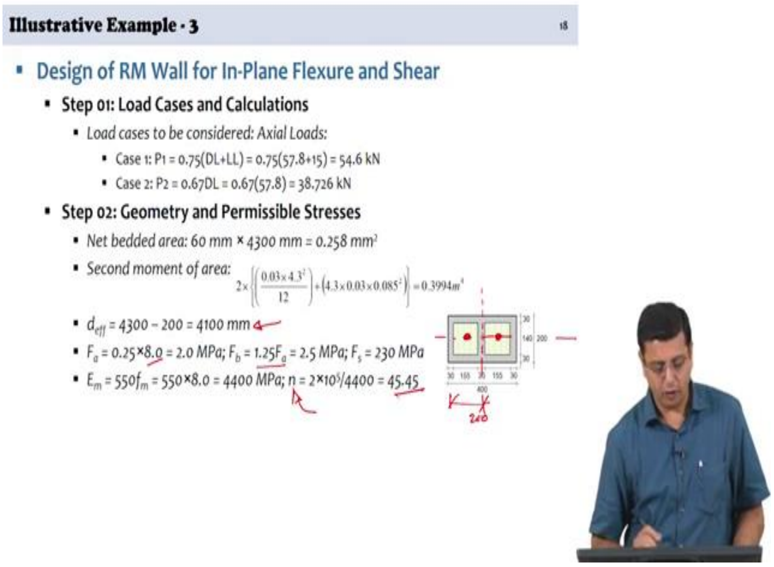

What are the axial load cases? That was the earthquake load, we had the overturning moment from the earthquake load and now for the axial loads we had two cases; the first case: was looking at 75 percent of dead load plus live load. And therefore, a case P 1 would give us about 54.6 kN; 75 percent of the dead load and live load estimated that is your case 1 we call it P 1, case 2 with the reduction in the dead load. So, 67 percent of DL; Dead Load is what we will consider this works out to about 38.726 kN.

So, we have two cases that we must look at in design. I will follow I will take up one of the cases and we will go into the details and one should be looking at both the cases to see which would give you the worst case scenario as far as the design is concerned.

Some aspects of the geometry and permissible stresses that we will be requiring the net bedded area, we just talked about this; 60 millimeters into the length of the wall 4300 millimeters, the net bedded area is 0.258 mm^2. We need to estimate the second moment of area and the second moment of area is estimated considering the geometry of these two strips that are sitting about the centerline of the wall itself. So, the second moment of area is estimated to 0.3994m^4.

We need effective depth calculation for in-plane flexure so, we are early looking at; let us assume steel reinforcement is going to be placed in the two voids that are present in a block. And therefore, the centroid of the reinforcement can be considered to be in the middle of the two reinforcement bars and therefore, that is one half of the block. And therefore, we can assume that the effective depth is calculated from extreme compression fiber to the centroid of the steel reinforcement and that is 200 mm from the edge and therefore, the effective depth is taken as 4100 millimeters.

Of course, if the steel reinforcement is going to be placed in many more cells then you will have to adequately come back and check, come back and make alterations to the effective depth calculation. The permissible compressive stress estimated as 25 percent, one-fourth of the, compressive strength of masonry which was 8 MPa. So, F_a is 2 MPa, F_b because of the strain gradient effect, F_b is increased you have 1.25 F_a, that would be 2.5 MPa and F_s as stated earlier 230 MPa.

Modulus of elasticity is required primarily to make the estimation of n, the modular ratio. So, $E_m = 550\, f_m = 4400$ MPa. So, the modular ratio;

$$n = \frac{2 \times 10^5}{4400} = 45.45$$

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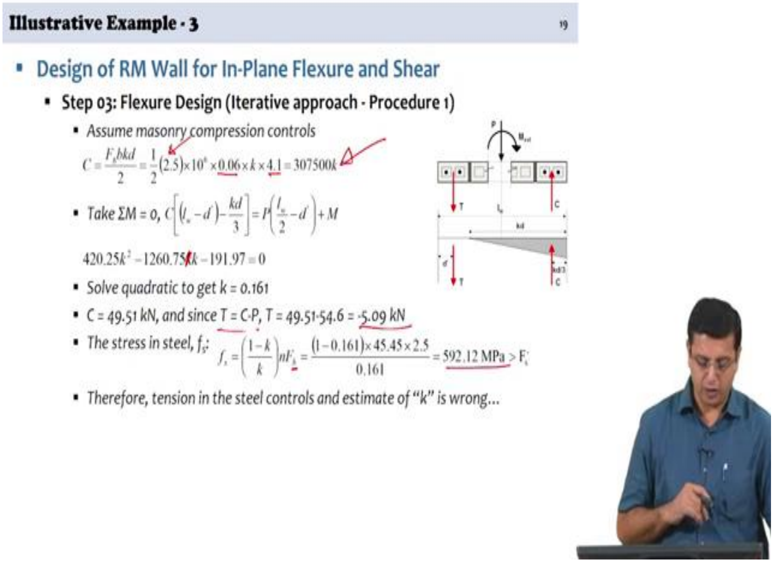

So, let us look at the design; the iterative approach that we talked about there are two procedures and it is up to the designer to choose the approach he or she would like to use. Both are iterative procedures and there can be other methods also that one can use, basically the fundamental requirements are almost the same.

So, we will look at procedure one; in procedure one we were, first making an assumption that compression controls and then check whether that assumption is true for the depth of neutral axis that is estimated, and then if not make changes to the way the neutral axis depth is calculated, and then arriving at the area of steel that is required.

So, under the action of the external moment and the load the total length of the wall is l w. The effective depth is l_w – d' which is d effective. So, d' is the distance to the centroid of the steel reinforcement that is where you are tension resultant is acting, and then you have the triangular distribution of compression, compressive stresses and C is the resultant in compression. The length of the compressed zone which is the depth of the neutral axis k d is what we need to estimate, we also require j d which is 1 - kd/3 as well in calculations of respective stresses.

So, let us assume in the first instant that the masonry compression controls and calculate the compressive force in the masonry. The bedded width 60 mm, k is unknown and effective depth is 4.1m.

$$C = \frac{1}{2} F_b bkd$$

So, we have an expression for the compressive force in k; taking the moments to be equal to 0; taking the sum of moments is equal to 0 we can write down the quadratic expression in k

$$C\left[\left(l_w - d'\right) - \frac{kd}{3}\right] = P\left(\frac{l_w}{2} - d'\right) + M$$
$$420.25k^2 - 1260.75k - 191.97 = 0$$

Solve the expression for k and we get a value of 0.16 for k.

Now, with that you can go back and estimate the compressive force and since we know that the tensile force is going to be by equilibrium C - P we get an estimate for the tensile force. Now with that, let us check the stress in steel and stress in steel is estimated because now as the compression controls, we have the stress in masonry equal to F_b and therefore, from similar triangles and compatibility we can estimate what the stress in steel is, but this stress in steel works out to be much higher than the permissible stress in steel which is 230 MPa.

Now, this implies that the assumption that masonry compression controls is wrong and therefore, the estimate of k is wrong. So, we need to go back and look at a situation where the stress in steel is governing and estimate what is the corresponding k and then estimate the compressive stress in masonry.

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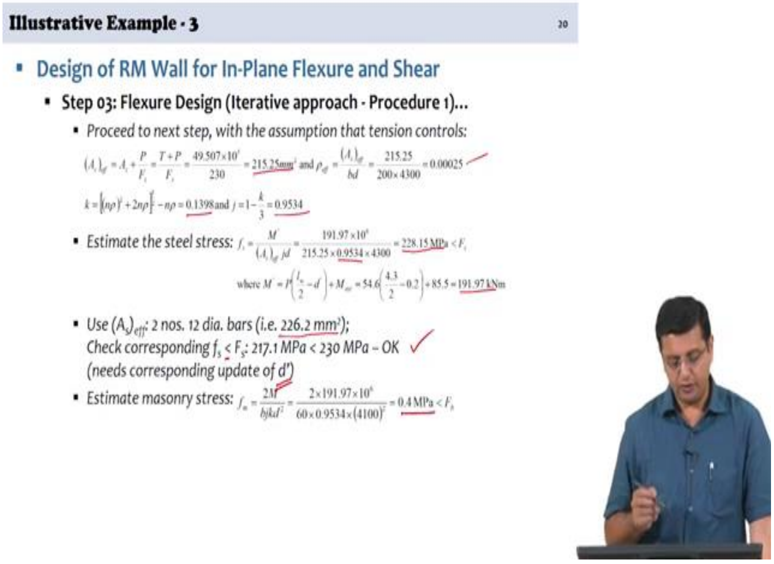

So, to follow on to the next step we assume that tension controls; the steel controls and we have to make now an estimate of the effective area of steel. We use the effective area of steel to then check if we are satisfying the requirements that the stress in steel is less than the permissible stress and whether the stress in masonry is less than the permissible stress in masonry.

So, $(A_s)_{eff}$ is estimated as $(A_s)_{eff} = A_s + \frac{P}{F_s}$, we are taking into account the effect of the tension that is occurring due to the eccentricity caused by the overturning moment. So, which therefore can be estimated as a sum of tension plus the axial load divided by F s as a first estimate of the $(A_s)_{eff}$. So, tensile force that we calculated in the previous step + P/F_s could be a starting point.

$$(A_s)_{eff} = A_s + \frac{P}{F_s} = \frac{T+P}{F_s} = \frac{49.507x1000}{230} = 215.25mm^2$$

From which the percentage of steel is estimated

$$\rho_{eff} = \frac{(A_s)_{eff}}{bd} = \frac{215.25}{200x4100} = 0.00026$$

With the percentage of steel ρ_{eff} estimated, we can try and we estimate k from the quadratic expression and an estimate of k is derived and an estimate of j as 1 - k/3 is also derived.

So, we will have to now look at whether with these estimates do we satisfy the requirements that tension controls and masonry stress is within permissible stress. So, let us estimate the steel stress using these estimates of k and j as $f_s = \frac{M'}{(A_s)_{eff}\, jd}$ and this M' is nothing, but the external moment plus the additional overturning moment that is coming because of the eccentricity of the axial force due to the tension occurring in the cross section and a non-uniform distribution of compression.

$$M' = P\left(\frac{l_w}{2} - d'\right) + M_{ext}$$

and this totally works out to about 191.97 kNm. We use it in the previous expression. This gives us an estimate of the steel stress and the steel stress is in this case 228.15 MPa which is less than the F_s or the permissible stress in steel.

So, this satisfies the requirement, now we can basically use 2-12 mm diameter bars to achieve this $(A_s)_{eff}$ here, which is 215.25 mm^2; of course, the estimate of $(A_s)_{eff}$ was an initial estimate. We can revise that; basically if you are going to choose for $(A_s)_{eff}$, two numbers of diameter 12 bars then the area of steel is slightly more.

So, it should not matter too much in this particular case; you can see that the corresponding stress in steel is 217.1 MPa and still satisfies the condition that f s is less than the permissible tensile stress in steel. However, one must check if there are changes in the number of bars required, one must check if there is an update of d' that is required and then come back and correct the calculations at this stage.

The masonry stress is then estimated, f_m has to be estimated and f_m can be estimated from the similar triangles from the state of stress in the steel. And this works out to in the case that we are looking at 0.4 MPa and this is less than f_b which is 2.5 MPa and therefore, all requirements have been satisfied in this particular stage, ok.

So, we are looking at two numbers of 12 mm diameter bars that are placed in the extreme edges considering two directions, Given the symmetry of the structure, we did only one direction, but then it is true for both the directions. And therefore, we are looking at 2 bars of 12 mm diameter that we placed in the extreme cell on the left and the right ends of the wall of length 4.3 meters, ok.

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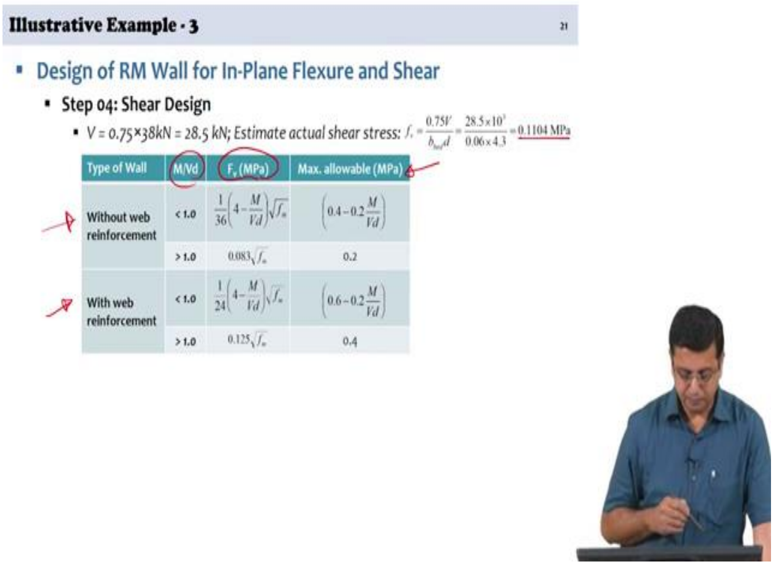

Now, as far as the shear design is concerned, how do we go about the shear design; so, the flexural design was taken care of in the previous slides. The shear force that we are talking of is a lateral force of 38 kN, but reduced to 75 percent and therefore, you are looking at 28.5 kN as the shear force. Let us estimate first the actual shear stress that the wall is subjected to because of this level of lateral loading.

$$f_v = \frac{0.75V}{b_{bed}d} = \frac{28.5x1000}{0.06x4.1} = 0.116 \text{ MPa}$$

Now, if you remember from the considerations that the code makes in terms of what should be the allowable shear stress and the maximum value of the allowable shear stress for different conditions without and with web shear reinforcement affected primarily by the aspect ratio of the wall or the M / Vd ratio of the wall. We need to choose what the F_V; what the allowable shear stress and the maximum allowable shear stress are.

So, in this case the level of shear stress is low, we could choose to go with either with or without web shear reinforcement and look at the estimate of M /V d first, and then the F_v and $F_{v\,max}$ that we must consider within, without web shear reinforcement.

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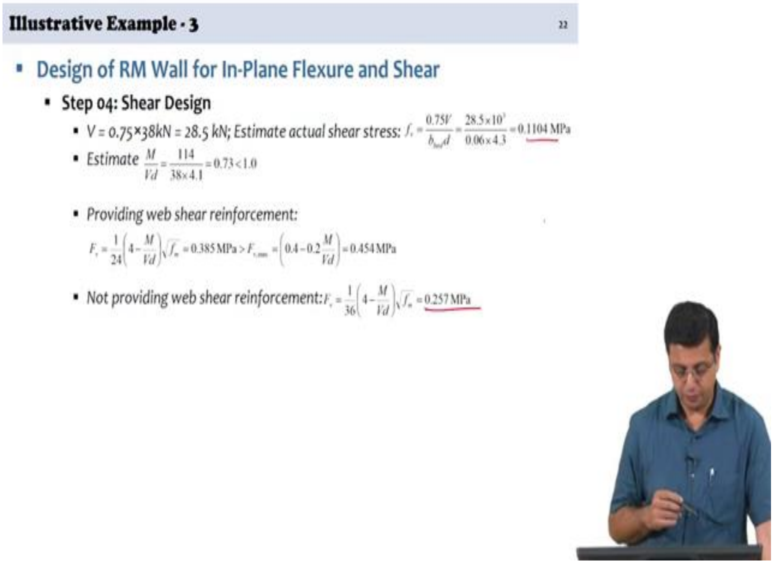

So, let us first estimate M by V d,

$$\frac{M}{Vd} = \frac{114}{38x4.1} = 0.73 < 1.0$$

This is a rather squat wall, it is 4.3 meters in length and hence we expect the response lateral load response of this wall to be basically governed by shear deformations.

So, in this particular case, if we are going to be providing web shear reinforcement the permissible shear stress is calculated as $\frac{1}{24}\left(4 - \frac{M}{Vd}\right)\sqrt{f_m}$, which is how the masonry compressive strength comes in as and a parameter that influences the shear strength.

So, this is estimated as 0.385 MPa; it is limited by $F_{v\,max}$ as estimated here as 0.254 MPa; however, our masonry shear stress is 0.11, the order of 0.11 MPa and therefore, we are well within the allowable shear stress or the maximum allowable shear stress.

If we were not going to be providing web shear reinforcement we would still be ok, it would be $\frac{1}{36}\left(4-\frac{M}{Vd}\right)\sqrt{f_m}$ and even here the allowable stress is 0.257 MPa; however, our actual shear stress is much lesser, which means that the wall without providing web shear reinforcement itself is adequate to take care of the shear that the wall is being subjected to.

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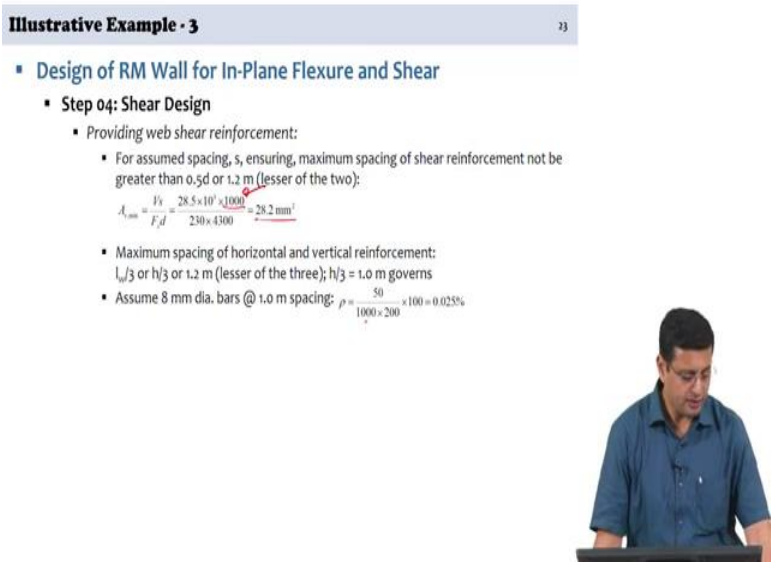

However, let us just look at the case where, if we were to provide web shear reinforcement for wall type C, whether the web shear reinforcement that we provide satisfies the requirements of detailing. So, I am providing web shear reinforcement, now we basically assume a spacing s, the maximum spacing of shear reinforcement should not be greater than 0.5d or 1.2 meters and we take the lesser of the 2.

So, we can check what the minimum shear steel that we are supposed to provide is assuming a spacing of 1 meter or 1000 millimeters. The minimum area of minimum shear steel works out to be rather low. So, V_s where V is 28.5 kN as estimated earlier, F_s is again 230 MPa and the effective depth is used for calculating the value of $A_{v,min}$.

$$A_{v,min}=\frac{Vs}{F_s d}=\frac{28.5x1000}{230x4100}=30.22\ mm^2$$

Now, the requirements on spacing the maximum spacing of the horizontal and vertical reinforcement where we must check has to be the lesser of these three values. Total $l_w/3$, $h_w/3$ or 1.2 meters; the lesser of the 3 has to be considered. In this case length of the wall is 4.3 meters and height of the wall is 3 meters and therefore, h/3 would govern, 1 meter would govern and that is the maximum spacing. And that is why in this calculation I had used 1 meter as the spacing for the calculation.

Let us assume we are providing 8 mm diameter bars at 1 meter spacing, and if you are providing 8 mm diameter bars at 1 meter spacing the percentage of steel then becomes 50.26 would be the area of one 8 mm diameter bar, divided by 1 meter spacing. So, that gives us about 0.025 percent is that, ok.

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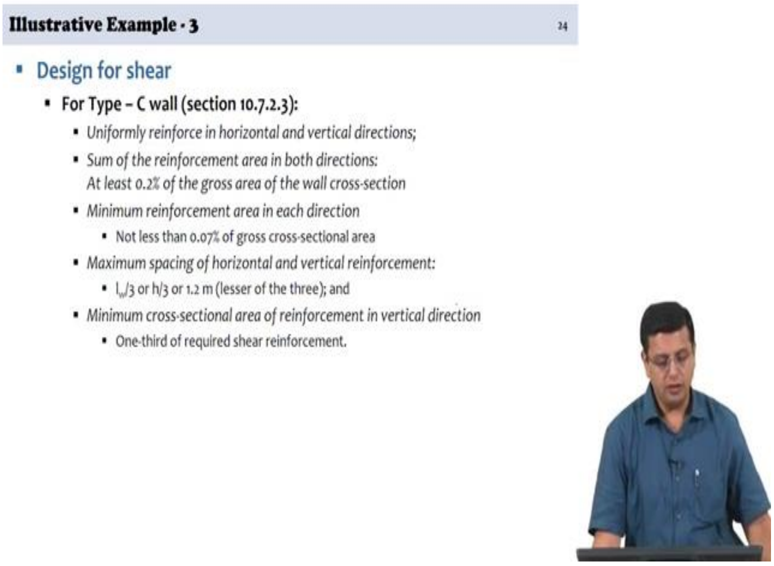

Now, assuming that we are providing web shear reinforcement is the amount of web shear reinforcement provided adequate for reinforced masonry wall type-C. If you remember the prescriptions for wall type-C we need to provide uniform reinforcement in both horizontal and vertical directions. We need to ensure that the sum of the reinforcement in both the directions is at least 0.2 percent of the gross area of the wall cross section. Minimum reinforcement area in each direction should not be less than 0.07 of the gross cross-sectional area.

So, that is something that will get violated because earlier we saw 0.025 percent is what we get by providing 8 mm bars at 1 meter spacing, and then in the horizontal and the vertical directions together we must get at least 0.2 percent. So, it is important to go and check if we satisfy these requirements.

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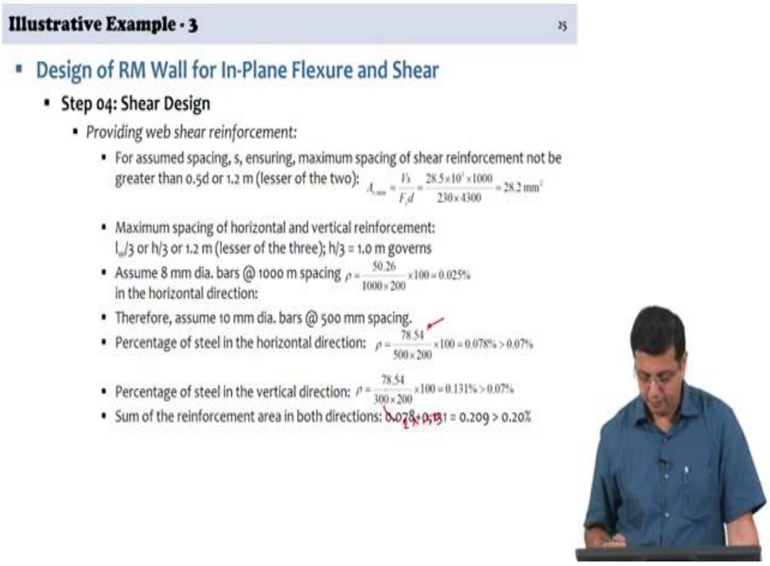

So, providing web shear reinforcement, we assumed we made the calculation for an assumed spacing of 1 meter, we checked that the 1 meter would not be adequate and therefore, with the 1 meter spacing we get in the horizontal direction, we get the percentage of steel as 0.25 percent; therefore, we assume 10 mm diameter bars at 500 mm spacing. And the percentage of steel in the horizontal direction is there then estimated as area of 1 steel bar divided by 500 mm spacing width of 200 mm into 100 and we get about 0.078 percent, which is greater than the seven percent that is required.

Now, percentage of steel in the vertical direction row is 78.54 divided by assuming we provide steel reinforcement in; assuming we provide steel reinforcement in alternate blocks with 1 block of height 150, each block of height 150 mm. So, in the vertical direction if you are looking at how many bars have to be provided. So, this is alternate blocks so, that is 2 x 150. We get about 0.13 percent for 10 mm bar that is going to be provided at alternate courses. So, this is again greater than 0.07 percent.

You have another requirement that the horizontal and the vertical, the steel in the vertical direction should be 3 times that of the horizontal steel and therefore, you would have to probably reduce this to 150 and check that the additional requirement of vertical steel being 3 times that of the horizontal steel is also, taken care.

The sum of the reinforcement in both the directions would be 0.078 plus 0.131 and therefore, 0.209 which is 0.2 greater than 0.2 percent which was what was one of the requirements. So, therefore, we actually have looked at the shear design aspects; this wall could actually be left without providing web shear reinforcement. But, if you were to provide web shear reinforcement these are the checks that need to be carried out to ensure that minimum steel for a certain type A, B or C is taken care of, ok.

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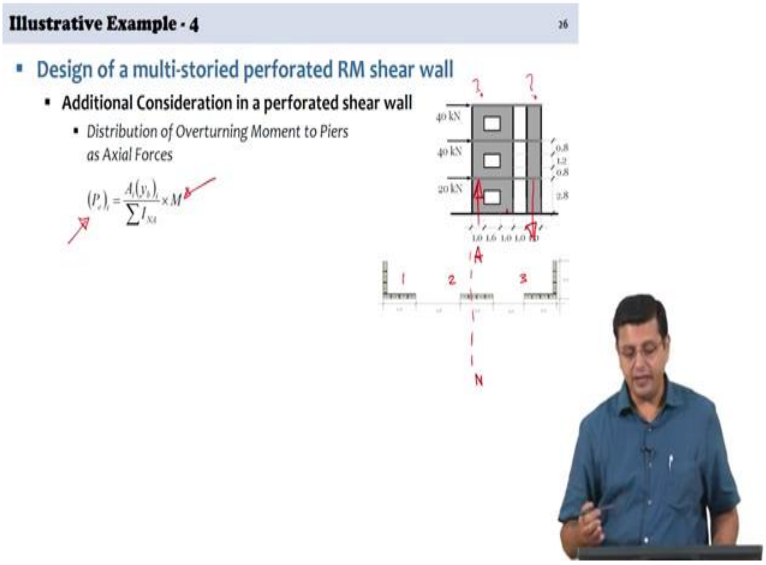

With that let me talk of one last aspect, which is peculiar to perforated shear walls. In the previous case we looked at one blank wall which was 4.3 meters in length and if we were to consider that to be a multi storied structure, you would have the shear wall design carried out in a very similar manner as you saw in illustrative example 3.

But, if you were to look at a perforated reinforced masonry shear wall, the one item that would change and you need to adequately consider that in your calculations is the overturning moment causes different axial forces in piers. There is a change in the axial force due to the earthquake force and the overturning moment caused by the earthquake force, and that has to be accounted for in your calculations.

So, if you were to look at the gravity forces; the gravity forces would come from the dead load from the live load and from the earthquake force because of the overturning moment. So, the additional axial force from the overturning moment which could be tensile in nature or compressive in nature depending on the direction of the earthquake and depending on the position of the wall with respect to the overall geometry of the structure that you are considering has to be in addition taken into account.

So, the distribution of the overturning moments to piers as axial forces has to be considered. So, you have this additional axial force coming from the earthquake forces P_e of there are different walls, if you have to calculate that for the different walls. So, basically how can you calculate that and where is it coming from? The lateral force acting on the perforated shear wall which composes the overall building is actually going to cause a situation of additional compression in the extreme piers, and situation of tension in the one on the left end, considering that the earthquake force is acting from left to right.

We typically; in this case we have three piers; the pier in the middle if it is aligned with the centerline of the overall building plan. Then would see minimal alteration in the axial force because of the overturning moments or the axial force corresponding to the overturning moment would be close to 0; however, that depends on the geometry. But, what we should be careful about is how are these two piers in this wall which is made out of three piers, how is the axial force due to the overturning moment and what is the quantum in the extreme piers.

So, we are interested in estimating, what is the neutral axis of this cross section? The wall made out of three different piers; we have pier 1, pier 2 and pier 3 and therefore, we will be able to estimate the axial force corresponding to each of these piers, in this case pier 1, pier 2 and pier 3. As the area of cross section of each pier which is A_i $(y_b)_i$ which is nothing, but the distance between the centroid of each of the piers that is being considered to the neutral axis.

$$(P_e)_i = \left[\frac{A_i (y_b)_i}{\sum I_{NA}}\right] M$$

So, the effect is due to the overturning moment; overturning moment into the respective areas into the distance between the centroid of that cross section from the neutral axis divided by the moment of inertia from the second moment of area of the cross section with respect to the neutral axis. So, this additional component of axial force needs to be considered and as you can see depending on the level of lateral force. The extreme piers can actually have a beneficial effect due to additional pre compression or be in a disadvantageous situation due to the uplift or even be subjected to tension.

And since, you know that the shear capacity is directly dependent on the level of pre compression, you can have very different situations in the piers and therefore, design has to adequately consider this particular aspect, And if there are significant; there is significant deviation for from symmetry in the plan, then that is again going to be a cause for concern in the actual design forces to each of the piers.