Design of Masonry Structures
Prof. Arun Menon
Department of Civil Engineering
Indian Institute of Technology, Madras

Module - 04
Lecture - 26
Design of Masonry Components and Systems Part – V

So, good morning. We will continue with the reinforced masonry portions and today I would like to look at P-M interactions for reinforced masonry walls and then that will lead us to looking at design for bending plus axial load, which is addressed in an empirical manner in the code, in the national building code and the section on reinforced masonry.

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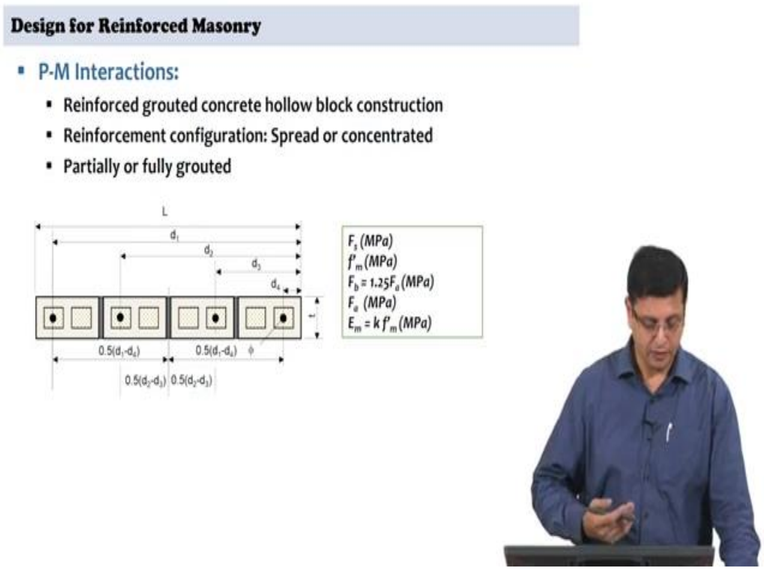

So, before we move into the prescriptions in the code, it is essential to look at P-M interactions for reinforced masonry walls, but within the working stress approach. So, what I am going to be doing today is, develop the set of expressions that you can use based on the linear elastic behavior, but with the use of the working stresses for both steel and the masonry and then look at the different zones of the P-M interaction curve so, that analytical expressions in those zones can be developed and you can use that as a design tool.

So, depending on the demand in terms of the axial load and moment, if it were to be lying within or on the P-M interaction curve, you look at verifying the design or altering the design based on the comparison between demand and the interaction curves itself. However, as I said the national building code prescriptions are empirical in terms of the design approach and we will examine that subsequently.

So, when you are looking at P-M interactions for reinforced masonry walls, we are really looking at reinforced grouted concrete hollow block masonry. Now, those could be concrete hollow blocks or it could be the perforated clay brick blocks with steel reinforcement and grouting. So, we are basically examining either the clay bricks with perforations or concrete blocks and so, the strengths coming from those; the different types of units that you would use is required.

And then of course, one important difference in the moment capacities that you would see is depending on the reinforcement configuration in the wall itself. So, the reinforcement configuration as stated here, you are looking at either a spread configuration or a concentrated configuration right. Now, what do you mean by a spread configuration of reinforcement and a concentrated configuration of reinforcement.

If you are using hollow blocks and your wall is made out of 4/6/8, depending on the number of hollow blocks that you require to make the entire length of the wall, the placement of the reinforcement considering that you are designing these as flexural walls in plane flexural walls or shear walls, the placement of the steel reinforcement can either be concentrated and put at the ends of the walls which is good in terms of the moment capacity that you will be able to achieve because with respect to a neutral axis you are going to be placing a steel reinforcement farthest away from the center of the wall.

So, it is a good method to place your flexural steel, mind you are placing a flexural steel farthest away from the center line of the wall itself and therefore, you are concentrating the flexural steel. You could choose to spread the flexural steel which is ok. Nevertheless, the minimum vertical reinforcement that you require has to be provided over and above this steel that you are providing as flexural steel.

So, the minimum spacing requirements between the vertical steel has to be taken into account anyway whether I choose to place the steel reinforcement at the ends as flexural steel or not the minimum spacing of the vertical reinforcement has to be taken into account. So, if you were to place them at the ends concentrated, you might have to provide some additional vertical steel at least the minimum that the code requires so, that you have satisfied the requirements for vertical and horizontal steel distribution.

So, you can either have a configuration where as you see in the figure, these vertical bars have been placed in all the alternate hollow regions and it is spread or you could ensure that two of these bars are placed in the first block and two of the bars are placed in the last block and therefore, you are you are concentrating them. You know that the lever arms are going to be different and therefore, moment capacity will be better in the concentrated situation than the other situation of course, it depends on other aspects of how much of area of steel and all that you are going to be considering.

So, this is one important difference that is normally checked. The other aspect is you are seeing a situation where in one block, in one of the cavities one reinforcing bar is placed ok. It is most often going to be the case you will not place a second reinforcement bar inside the same cavity and this is basically a geometrical requirement, you will not have enough space between bars and you would not have clear dimension between the cavity and the bar edge itself which has to be filled with grout.

So, from cover considerations and given the size of the cavity, typically it is going to be one bar you know in a given cavity. So, the placement of the reinforcement is up to the structural designer and therefore, that is one choice you need to make whether you are going with a concentrated reinforcement or spread reinforcement. And of course, there is the other requirement are you going to be looking at a fully grouted wall or are you going to be looking at a partially grouted wall?

So, this again needs to be ensured and most often what would happen is only the cavities where you place steel reinforcement it is grouted and the others are typically left hollow, but then that is again a design choice. So, you really need to know if you are going to be using a grouted wall or a partially grouted wall and make necessary amends as far as your calculations are concerned.

So, working stress approach to create the P-M interaction curves. So, you need to establish what the allowable stress in your steel is depending on the steel that you have chosen establish F_s which is your allowable stress in steel, you should know the compressive strength of the masonry and we discussed this. Of course, you can develop your set of expressions considering the different strengths of the grout and the block itself. However, with the requirement that the grout is at least as strong as the block and using the strength of the block as the value that you will use for calculations is acceptable.

So, assuming you have an estimate of the compressive strength of the masonry f prime m you then need to establish what the working stresses are; F_a is the permissible compressive stress and F_b is the permissible compressive stress, but for flexural compression where we are allowed to increase the permissible compressive stress by 25 percent.

So, F_b is 1.25 times F_a and now F_b is what you will require for your stiffness calculations you will have to use, of course, in this strength based approach you are not going to be using this stiffness calculation, I mean you are not going to be using the modulus of elasticity; however, the modulus of elasticity can be estimated as a function of the compressive strength of the masonry that we have seen earlier.

So, to make your deflection checks you will have to use masonry modulus of elasticity. So, those are the parameters that you will require to begin with. Looking at the geometry I am going to be using this geometry in the ensuing slides and therefore, it is useful to familiarize ourselves with the notations here. L is the total length of the wall, t is the thickness of the wall which is nothing, but the block thickness in this case and depending on whether you are going to be using a single block or a double block then that of course, varies. t is the thickness and then assuming that a spread reinforcement configuration is used and it is symmetrically placed.

You have the distances from the edge fiber to the centerline of the steel reinforcement each reinforcement bar of diameter Φ- d_1, d_2, d_3, d_4 or d_i depending on the number of bars that you have. And then all the geometry can be worked out based on these distances from the centerline of the bar to the edge of the wall L, t and the areas of the steel reinforcement.

The other thing that can be done is in this particular case we are really looking at in-plane capacity. When you are looking at the out of plane capacity again the choice is how you place the steel reinforcement. Is the steel reinforcement going to be placed for out of plane moment capacity in the center of the wall along the thickness or is there a possibility of having two reinforcement bars placed at the farthest edges within the cavity.

So, in the out of plane direction, if you are placing the steel reinforcement at the centre line of the wall thickness you are not going to be able to use the contribution to capacity by the bar because it is right in the center, unless you have large eccentricities. Instead if you actually place the steel reinforcement apart as two bars within the cavity, you can get a better moment capacity in the out of plane direction.

So, configuration of the steel reinforcement within the cavities is an important aspect that as a designer is under your control. So, with this configuration in mind, please do keep this configuration in mind, we will start looking at different zones and the expressions that we can use for the calculations of the moment capacity for a given level of axial load itself.

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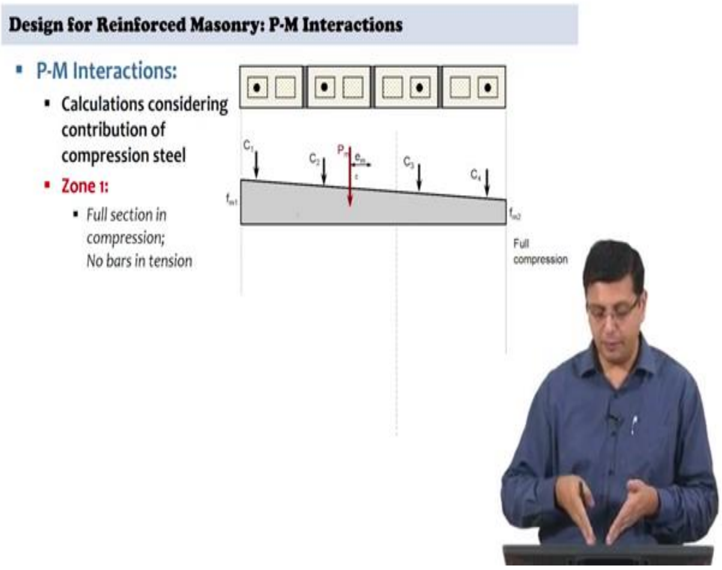

So, the P-M interaction curves one choice that you should make in your calculations is whether the contribution of the steel reinforcement to the compression resultant is going to be considered or not, right. So, are you going to be using steel contribution to the moment capacity only after the section cracks is a choice left to you. And in the assignment that you will do with respect to the P-M interactions you will actually check if the contribution by the compression steel is something you must actually bring into your calculations or is it something negligible.

So, assume that we are actually going to be considering the contribution of the compression steel in our calculations that you are going to see in a few minutes, but this is again a choice that you need to make. It is a simple alteration to the P-M interaction curve itself ok. So, first situation you have the wall that is subjected to a combination of gravity forces and lateral force, in-plane lateral force and so, you have a non uniform distribution of compression; however, the entire cross section is in compression.

So, that is where you begin your full cross section is in compression. Your full cross section can be in compression with the condition that there is no bending at all; zero bending moment on the section, which means the two ends, the section is fully in compression, but the two ends the stresses f_{m1} and f_{m2} are both going to be equal. Now, when both f_{m1} and f_{m2} are equal, you have a situation where there is no strain gradient and therefore, you should be using the permissible compressive stress F_a in this case as the limiting compressive stress.

However, with any finite value of moment along with the axial force, the limiting stress is no longer F_a, but F_b which is nothing, but 1.25 times F_a that you can consider. So, we are looking at a situation where there is already some bending in the wall entire section is in compression, all the four steel bars are also in compression and contribute to the moment capacity or the axial load capacity.

So, C_1, C_2, C_3 and C_4 are the resultants of the compressive forces in the bars whereas, the grey area that you see is the compressive stress distribution in the masonry with f_{m1} on one end and f_{m2} on the other end where the section is less compressed. We are working with an assumption where the entire wall is grouted and that is again something that you should carefully check between your calculations and actual execution itself.

The first zone is the zone that I have spoken about earlier and that is pre-cracking. So, there is moment acting on the wall and the entire section is in compression, no tension in the cross section yet. So, in zone one no bars are in tension and therefore, in this situation how do you set up your equations? The value that you see here as P-M is nothing but the resultant of the axial force in the masonry itself; P-M is the resultant of the axial force in masonry, given that we have some amount of bending now there is an eccentricity of P_m with respect to the center line in the wall itself. So, that is the eccentricity e_m, eccentricity in the masonry due to compression.

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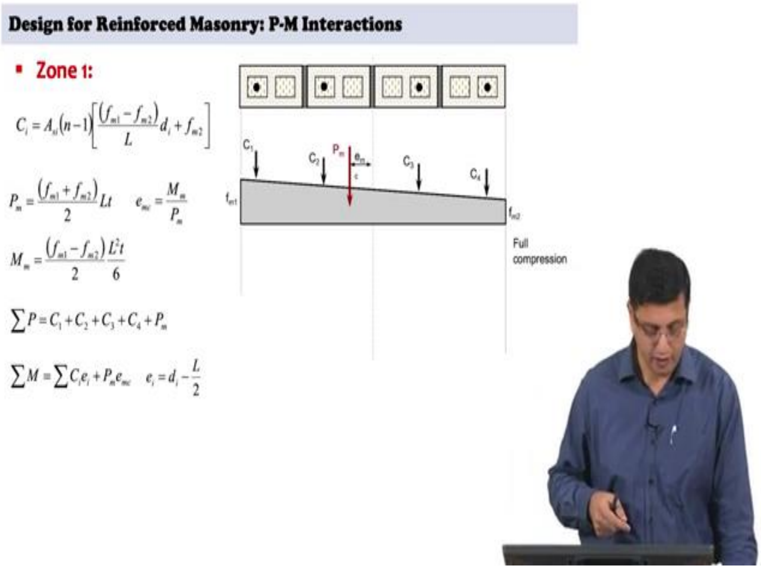

So, the expressions you now need to be able to estimate given this distribution of stresses knowing f_{m1} and f_{m2}, we start with f_{m1} and f_{m2} values and we will come to that in a moment. So, you need to be able to make an estimate from the geometry of the distribution of stresses and the geometry of the wall what the resultant forces C_1, C_2, C_3 and C_4 are for which you are basically making use of the triangular distribution of compressive stresses in the wall.

The triangular distribution of compressive stresses that you see here, the full wall in compression f_{m1} and f_{m2} known, you will be able to use the modular ratio n, the modular ratio that is modulus of elasticity of steel to the ratio of modulus of elasticity or steel to that of the masonry. So, with a modulus of elasticity used what you are doing in this set of calculations is working on a transformed section and arriving at the value of C_1, C_2, C_3 and C_4 which are the compression resultants in the bars which are all in compression.

So, with respect to the non-uniform distribution of compressive stresses, you will estimate C_i for each of these situations for a fixed value of f_{m1} and f_{m2}.

$$C_i = A_{si}(n-1)\left[\frac{(f_{m1}-f_{m2})}{L}d_i + f_{m2}\right]$$

Let me set up the expressions and then go to the stage wise calculation. The axial force resultant in the masonry is given by,

$$P_m = \frac{(f_{m1} + f_{m2})}{2} Lt$$

$$M_m = \frac{(f_{m1} - f_{m2})}{2} \cdot \frac{L^2 t}{6}$$

$$\sum P = C_1 + C_2 + C_3 + C_4 + P_m$$

$$\sum M = \sum C_i e_i + P_m e_{mc}$$

$$\text{where, } e_{mc} = \frac{M_m}{P_m} \text{ and } e_i = d_i - \frac{L}{2}$$

So, this way you have established for each level of stress distribution what the value of axial force and moment capacity is going to be.

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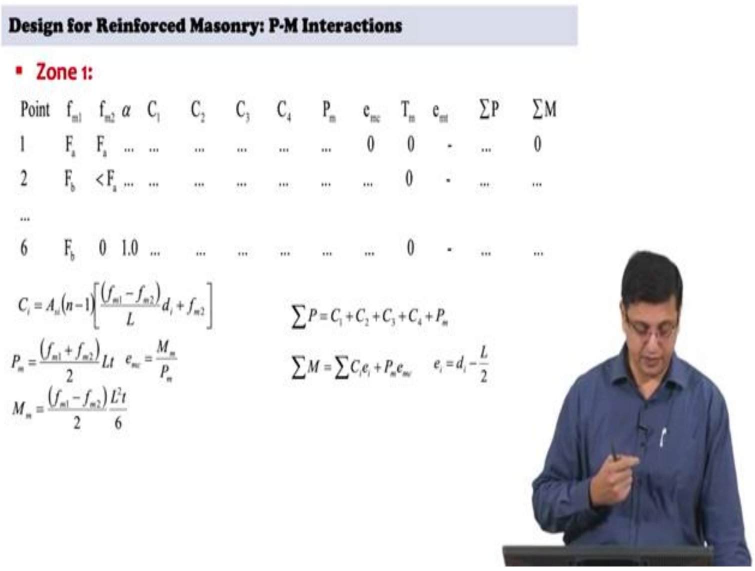

So, these expressions can be used in zone 1. And in zone 1 the expressions that we just talked of are at the bottom, but I am now looking at different stages to estimate the moment capacity. So, my first point is when the wall is in pure compression my f_{m1} is equal to the permissible compressive stress since I am looking at capacity, but within the working stress approach, I am starting with F_a as the value of f_{m1} and F_a as the value of f_{m2}; wall is in complete compression; for this particular case of course, there is no moment capacity, this is the axial load capacity of the wall itself.

So, that is your first stage, then you start looking at making the compression non-uniform whereby since a certain moment is also being considered you cannot use F_a any longer and therefore, you start using the value of F_b as the limiting stress. So, you use F_b in the second point, I am using F_b and then keep reducing the value of f_{m2} thereby you are considering larger eccentricities keep continuing till f_{m2} becomes 0. Meaning when you say f_{m2} become 0 that is your limiting condition where the entire wall is still in compression, but any additional moment would mean cracking of the cross section begins.

So, the first zone is purely in compression. So, make your calculations using the expressions developed within this range, estimate values of P and corresponding values of moment based on the condition in the wall stress distribution in the wall F_b and the reducing values of compressive stress in the lesser compressed edge of the wall itself.

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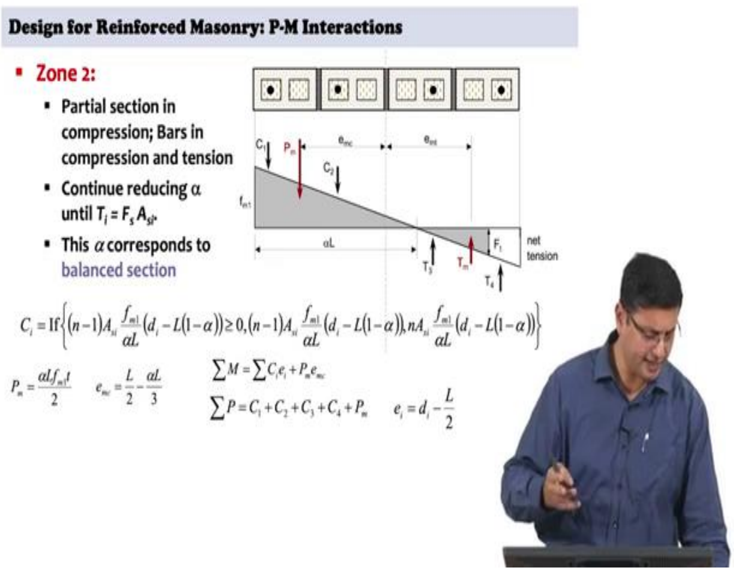

So, that is zone 1. Once cracking progresses you are into zone 2 cracking progresses and here again you might want to use a tensile strength of the masonry. If you think that the value of tensile strength of the masonry F_t is significant enough it could be brought into calculations or neglected. You can either neglect the contribution to the moment capacity by the tensile strength of the masonry or introduce that into the calculations, but that is going to be a very small quantity. So, you might as well do away with that additional calculation.

So, once the wall starts cracking, you reach a situation where with additional moments first bar and then the second bar and so, on start experiencing tension. So, zone 2 is where part of the section is in compression, the rest of the section is in tension which means now slowly each bar will start coming into tension. And as the moment on the wall increases question is what is the tensile stress in the bar and whether the tensile stress in the bar is reaching your permissible tensile stress in steel F_s and the same calculations will continue, but you are looking at a situation where there is some net section which is going into tension.

So, here what I was talking about is, do you want to consider this portion with a certain limit on F_t? You have seen that we can use a permissible tensile stress for the masonry depending on depending and here you are basically looking at tensile strength perpendicular to the bed joint and not parallel to the bed joint.

So, you use the permissible tensile stress that is that is for the direction where tension is perpendicular to the bed joint, it is a small value you know that the value prescribed by the code is about 0.05 or 0.01. So, the question is whether you want to use that and still make an estimate of the contribution to the moment capacity, the tension carried by the masonry itself.

Otherwise you will have to examine whether the bars are now in compression or the bars have started going into tension. So, I am looking at one of the intermediate situations where at least one bar is in tension, two bars are in tension and the other bars are in compression. So, this is a check that you will be doing at every stage to see how many bars are in compression, how many bars are in tension the distribution of the masonry stress is now triangular.

So, we are still assuming a linear elastic distribution and the distribution of from the moment when f_{m1}, f_{m2} becomes 0, this distribution of stresses is triangular that change will have to make as far as the calculations of compression resultant because the distribution of compressive stresses is triangular now.

So, here you basically need to establish whether you are looking at a compressive force resultant or a tensile force resultant at each bar. So, you are checking with respect to the side in tension f_{m2} by αL. Now, this αL is nothing but the length of the compressed zone that is it is less than L; α is less than 1.

So, as the wall starts cracking the length of the compressed zone is less than the or length L and therefore, now you are going to be using the value of αL instead of L in your calculations for the zone that is in compression. So, we are trying to check if see the compressive force resultant at each bar is it positive or negative. Now, if you are getting positive values as far as the notation as far as the calculations here, positive is considered to be compression and a negative value of the resultant force would be tension.

So, you need to make that check and if you are looking at a bar still sitting in the compression side, you will continue to use the previous expression, derivation of the previous expression because now you are not you do not have the full length whereas, if the bar is in a zone where tension is present in the masonry, then you will have to look at n instead of n-1; now you are looking at nA_{si} because you are not counting the bar diameter twice.

So, that check is required. Therefore, estimate the value of C_1, C_2, C_3 and C_4 or it might be tension once that is estimated your triangular distribution of stresses is what the compression is in and therefore, the axial force resultant,

$$P_m = \frac{\alpha L f_{m1} t}{2}$$

P-M is established the based on the triangular distribution; again the eccentricity of the compression resultant with respect to the centroid of the triangle itself is given by,

$$e_{mc} = \frac{L}{2} - \frac{\alpha L}{3}$$

And then similarly the summation of the moments from the bars in compression plus the portion of masonry in compression. Similarly make an estimate of the overall compression resultant from C_1, C_2, C_3 C_4 and P_m; you have the estimates within this zone also.

Now, you keep continuing in this part of the interaction diagram, you are going to be basically considering lower and lower areas of masonry in compression right. Because you have now reached limiting values of the stresses at the end the edge compression fiber f_{m1} is f_b you already at that maximum, but now with that maximum you are reducing the zone available in the wall cross section in compression.

So, you keep reducing the value of α and get the distribution of the capacities in the cross section, that is how you would proceed in this particular zone and continue until the first bar reaches a value of $F_s A_{si}$; that means, the first bar has well yielded, but not yielded in its true sense because in the working stress approach, it has actually hit the value of permissible tensile force.

So, you have reached a stage where one end of the wall cross section is in the maximum value of compressive stress permissible and the other end the first bar has actually reached the permissible stress in tension that is your balanced section. You have your balanced section and then continue the calculations with reducing values of alpha.

So, you continue reducing α until $T_i = F_s A_{si}$ and at this point you reach the balanced section. Mark that point as your balanced section and continue with the calculations.

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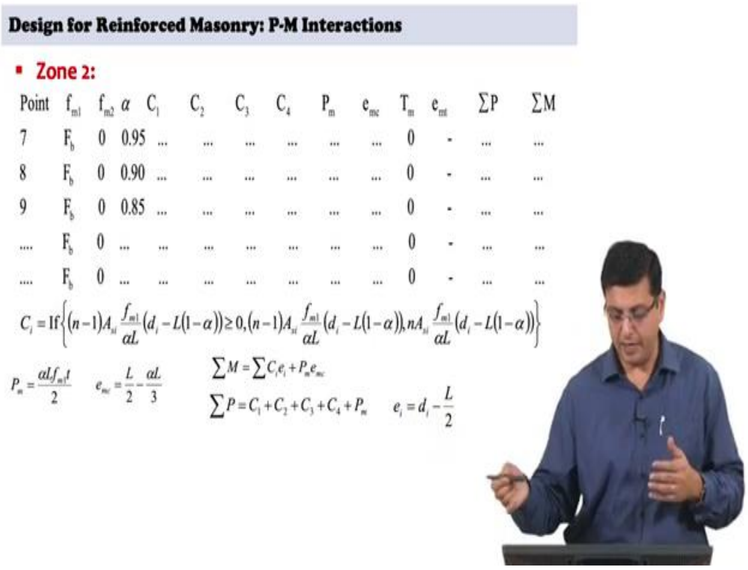

So, in this segment I have reported the expressions at the bottom, but the different steps, the different points of the P-M interaction diagram as you see on f_{m1} the compressive stress in masonry continues to be the permissible compressive stress due to flexural compression. So, F_b is what you maintain.

But what you are actually doing is f_{m2} is already 0 as earlier, the section has already started cracking and now the value of α is what you are steadily decreasing. You keep reducing the value of α you will see more and more of your bars will go into tension and therefore, the calculation of C_1, C_2, C_3, C_4 and Σ P and Σ M would vary so, that is your zone 2, you get your balanced section also coming within this zone.

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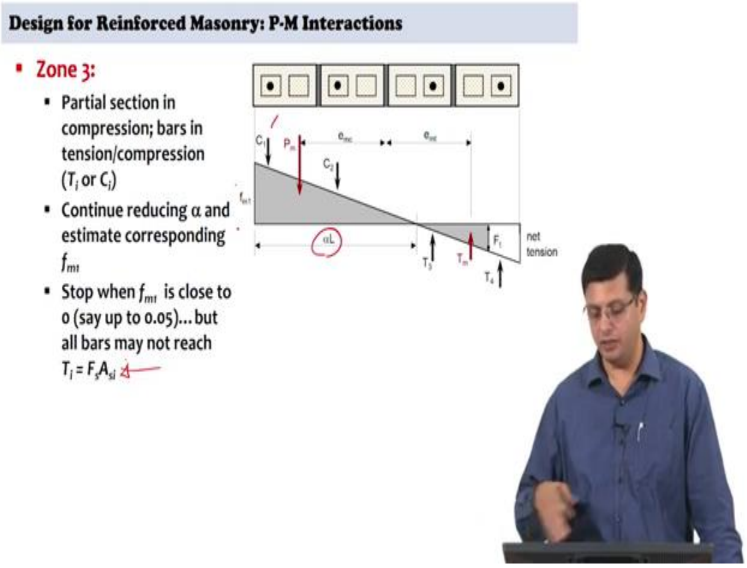

Zone 3, now the question is as you keep reducing α, you should come to a point where the entire section is now cracked. So, you are basically reducing α to values like 0.1L and to do that once the steel reinforcement bar as you increase this, as you in as you reduce αL, as αL reduces, let us assume all the four bars have now entered state of tensile stress.

Then the tensile stress in the bar will limit the level of compression in the masonry itself. So, now, you will have to look at what is the level of tension in the first bar and then corresponding to that would be the tension would be the compression in the remaining portion of the masonry. So, in this typically you will you will get all the bars in tension, but whether or not all the bars would actually reach the peak value, the permissible tensile force, f_s A_{si} is something that is typically not seen that is all your bars may not reach the maximum value of the permissible tension itself.

So, you keep continuing in this zone reducing the value of α and now with these steel stresses governing estimate what is the value of f_{m1} the masonry compressive stress this value will be actually lower than the peak value this value will be lower than the peak value and peak value which is F_b and you will now see that the moment resistance coming from the compression the compressed part of the masonry is what is being compromised or lost.

Of course, in your calculations you can look at a situation where fm 1 is almost close to 0 and stop there and at that situation depending on the geometry of the wall and depending on the location where the bars are, all the bars may or may not reach the value of permissible compressive force T_i here. So, that is something which can vary; in this case three bars may reach f_s A_{si} whereas, one bar is less than f_s A_{si}. So, that situation is to be expected.

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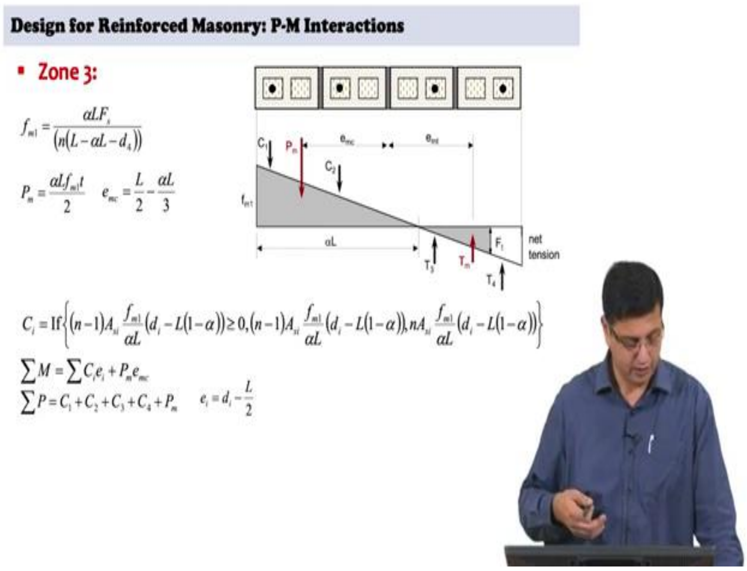

So, in this particular case what you are doing is, since the balance section has been reached, that is one of your bars has actually reached the permissible tensile stress, you will use the permissible tensile stress in the steel as what is going to govern the behavior of the wall thereafter.

Therefore, F_s will dictate what is the corresponding allowable masonry compressive stress. It is no longer going to be F_b, but a value that will be lesser than F_b, but governed by tension governed by the tension steel and therefore, in zone 3 the difference is that f_{m1} is not predetermined as a value equal to F_b, but f_{m1} is estimated based on what the value of F_s is.

So, again geometry of the cross section and value of F_s will dictate what value of fm 1 is and then you will again check whether all your bars are in tension or there are some bars in compression with the same set of expressions that we saw earlier, estimate C_1 C_2 C_3 C_4 or they may become T_1, T_2, T_3, T_4. Estimate again the compression is distributed in a triangular manner, compressive stresses are distributed in a triangular manner you will still continue to use $P_m = \frac{\alpha L f_{m1} t}{2}$ as the resultant compression carried by masonry. From the geometry, the eccentricity in the resultant and then the moment total moment and the total axial force resultants are estimated to complete the section in zone 3.

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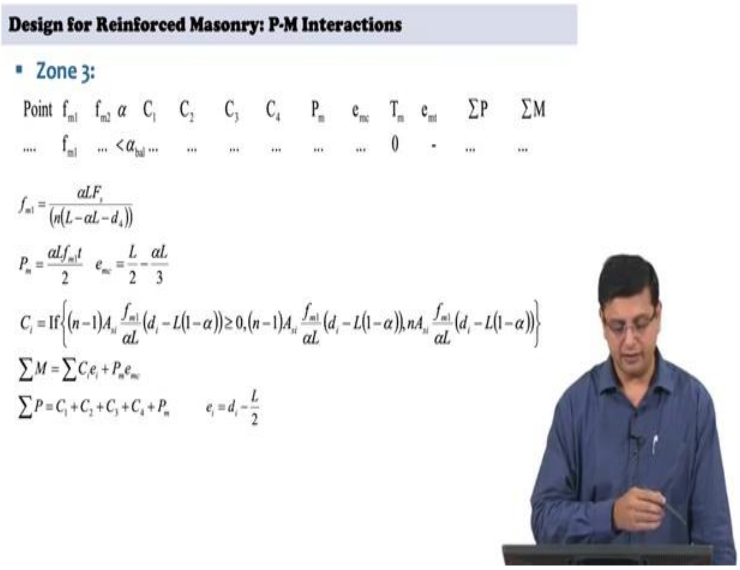

So, zone 3 you basically have to estimate what is f_{m1} corresponding to the tensile stress F_s and continuing this value is now less than the value corresponding to the balanced section. That is why α is now at a value less than the condition at the balanced section you will then go to a situation where you want to have the fourth zone which is where you have f_{m1} is equal to 0.

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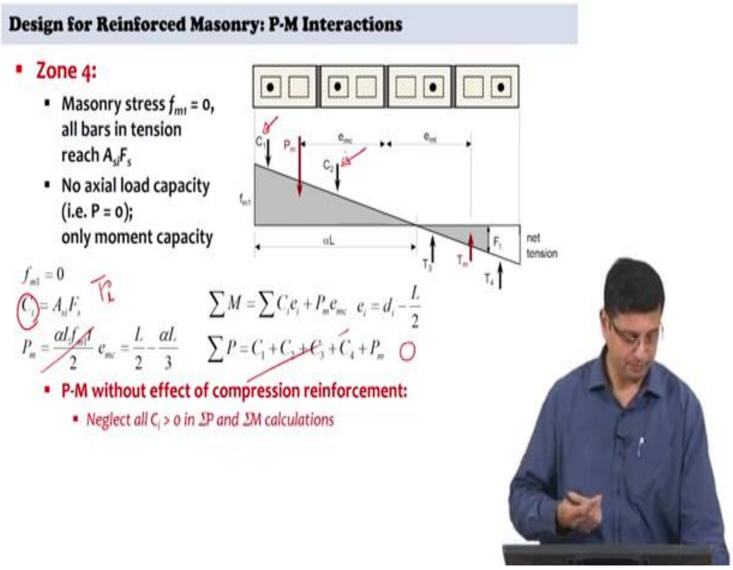

So, this zone, the fourth zone is where you will designate that the masonry stress has reached 0 which means you only have the bars in tension there is no axial force capacity basically in the there is no compression resultant available the whole section is in tension bars are all possibly at their F_s values and so, you have a moment capacity, but axial force capacity is 0.

So, then you reach the next critical point of the P-M interaction where P is equal to 0, M has reached a certain value. So, it is just continuing sections. So, now, you have no axial load capacity only moment capacity in the section fm 1 equal to 0, the resultant in terms of the tensile force carried by the bars is nothing, but A_{si} into F_s.

So, this is how you would look at four zones, now what we did here was to assume that the compression reinforcement continues to contribute to the moment capacity. We use the reinforcement for both tension capacity and compression capacity. So, one way could be to just neglect that contribution and so, just look at where these bars are showing value of C greater than 0 which is compression, set that to 0 and look at the P-M interaction curve neglecting the compression reinforcement and that is a quick check that you can you can do.

So, simple checks that you can do is, if you assume there is a finite permissible tensile stress that you want to consider for the masonry how much does that contribute to the moment capacity. If you were to consider the compression reinforcement how much is that going to contribute to the moment capacity? How do these P-M interaction curves stack up? Ok.

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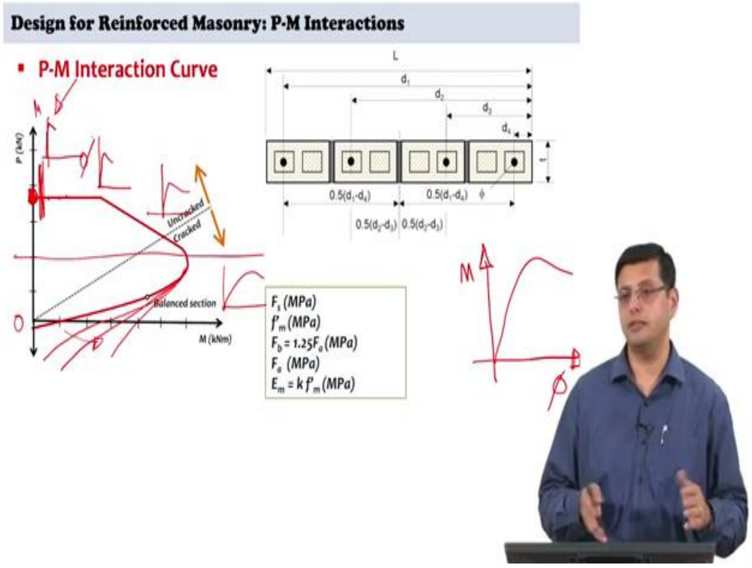

So, you would get a P-M interaction curve of that sort. The reason why it is capped off is because we are starting off with a peak capacity in compression on one end. So, you are capping it off as far as the permissible compressive stress is concerned. So, that would be your zone 1 all the way till it cracks. So, the upper part of the graph is this part is zone 1 and then the wall cracks after the wall cracks you get into your zone 2 calculations where you are basically reducing the compressed length alpha L.

And then you reach the balanced section in this state; once the balanced section is reached and then you continue reducing αL, but then once the bars have reached their permissible stresses, you will calculate f_{m1} based on what the permissible steel tensile stress is and that could be your zone 3 and finally, zone 4 is the portion where you are all you are considering zero axial force capacity and only moment capacity in the section.

So, critical points would be only axial force capacity- no moment capacity, then the balanced section and then finally, you have only moment capacity-no axial force capacity and the four zones in between. So, this is how of course, you may be familiar doing this for the limit state approach. So, one exercise would be to compare this with the limit state approach to create interaction surface and then look at the interaction surface that you have created using a working stress approach and how much more of moment capacity can be considered for the same section but using a different approach.

So, your next assignment is basically going to deal completely with the P-M interactions for reinforced masonry walls ok. Before I conclude this section there is something that I would like to touch upon and that is the P-M interaction behavior for a masonry wall, this particular exercise actually is very instructive from the capacities of reinforced masonry as a structural solution itself.

Now, why do I state that as far as the P-M interaction curve is concerned? That is because if you look at a masonry wall and if you look at the P-M interaction curve, we were looking at the axial force capacity at this location, right. If I were to introduce more and more steel in the in the cross section, then basically it is after cracking that you will see increased capacity in the wall; you would see increased capacity in the wall depending on how much steel I am putting in.

It is only after the cracking that is going to contribute. Now, if the level of axial stress in the wall is very high ok. Let us not think in terms of what the code allows as the maximum permissible stress, let us assume that the compressive stress in the wall is very high, close to the close to the limit that it that it permits. Assuming that we are somewhere here as far as the in that zone as far as the axial stress ratio is concerned. The level of axial stress due to compression divided by the strength of masonry or in this case the permissible stress in compression.

If we are in that zone steel reinforcement literally has no role. There is no effect in changing the interaction surface for us. But if you take masonry, if you take masonry as a structural system, if you look at shear walls and load bearing walls in a masonry structure, we know that the level of compression that exists is typically very low in comparison to the capacity in compression. It is a material which is good in compression which means most often, even if you are looking at two storied/ three storied structures the axial stress level in the wall will ensure that you are in the in the lower portion of the interaction diagram as far as P-M is concerned

Because you do not make very tall masonry structures, and because the code also does not allow you to go too far beyond the strength in any way close to the strength in compression, the axial stress level will ensure that you are in the lower portion of the axial force moment interaction diagram. Now why is that good? It is good from two perspectives- first is that, it is the region in which the provision of steel reinforcement is going to be beneficial to the moment capacity in the wall or shear assistance in the wall.

So, that is the zone where with more and more steel reinforcement that I design, I will be able to get better moment capacity. So, in-plane capacities are bound to increase when axial stress ratios are in this zone; that is one thing. The second important thing is of course, the axial stress ratios are small because you are looking at an entire wall cross section. If it were columns, these axial stress ratios will be high would you agree? If the entire structure were not made out of load bearing walls, if they are made out of columns, the axial stress ratio will typically be higher for a similar plan area.

So, it is because of the presence of load bearing walls. Now, if you look at the P-M interaction curve and then for different locations on the P-M interaction curve, if I were to draw a moment curvature diagram for that section, you would get typical moment curvature curves based on section analysis. Now, if for each point on the P-M interaction, if for that section I am estimating the moment curvature, then you will see that the moment curvature when the axial stress ratios are high or the axial load demand is high you will have very brittle behavior.

The curvatures will be low in those sections. As you come down; as you come down the moment curvatures will show better ductility and when you come to these zones, in which the axial stress ratios are low, the moment curvature will show good desirable ductility. So, this situation for moment curvature with very low ductility does not guarantee good ductile performance as far as earthquake resistance is concerned.

When axial stress ratios are high, you do not get ductile behavior how much ever steel you want to you put in there. So, as a typology, masonry because of load bearing walls gives you the possibility of low axial stress ratios therefore, you are in a P-M interaction zone where moment capacity increases because of the presence of steel significantly and to the moment curvature of the sections there significantly demonstrate good ductile behavior.

So, reinforced masonry is an excellent choice for seismic solutions as far as providing good ductility, good deformation capacity in earthquakes. This single feature actually makes it a category that is far superior to moment resisting frames. For moment resisting frames, the frame constructions with reinforced concrete columns, the immediate problem that you have is you will not be in the lower portion of the moment interaction diagram the P-M diagram, you will be in the higher portion because of higher axial stress ratios. And because of the higher axial stress ratios how much of a steel you want to put in there you will not get ductile behavior in moment curvatures.

So, reinforced concrete moment resisting frames cannot give you the desirable ductility beyond a certain limit whereas, here you have a structural solution that gives you the possibility of achieving good deformation behavior as far as earthquake forces are concerned. So, this is something that you will see is discussed quite extensively in the literature on seismic behavior of reinforced masonry and I wanted to close the P-M interaction discussion with this particular aspect. So, you value the P-M interaction that you are doing as a demonstration of good ductile behavior of reinforced masonry itself.