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

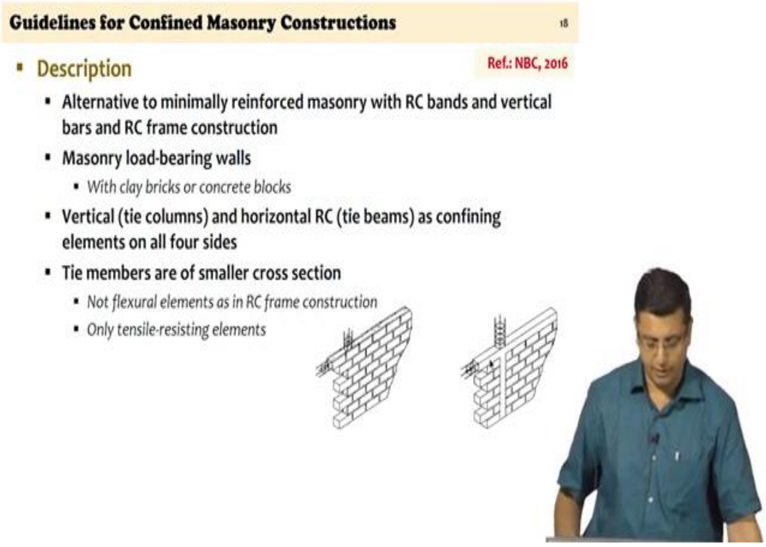
Module – 05
Lecture – 36
Special Topics – Confined Masonry

Good morning, we will look at the second of the special topics that we are examining in this last module of the course. And today the focus is on Confined Masonry; a confined masonry as introduced at the beginning of the course, is a structural typology within masonry constructions. It has gained significant fraction in our country in the last few years; however, this is not a new typology, this has been around and found to work exceedingly well when subjected to earthquake forces.

And several countries actually have variants of this typology; countries with significant seismic activity like Chile in South America is known to have a significant stock of constructions, particularly residential buildings, small office buildings, school buildings which are all confined masonry constructions. And the advantageous aspect of confined masonry is that there is ease of construction and with minimum prescriptions, a structurally favorable typology can be established on the ground. So, that is where confined masonry is positioned as a viable structural solution with good earthquake resistance.

Today the national building code, the 2016 version has a section on confined masonry; but this is again a typology where rigorous design approach is not adopted. But prescriptive recommendations are provided, such that the construction details regarding the construction are addressed when you choose this typology.

(Refer Slide Time: 02:10)



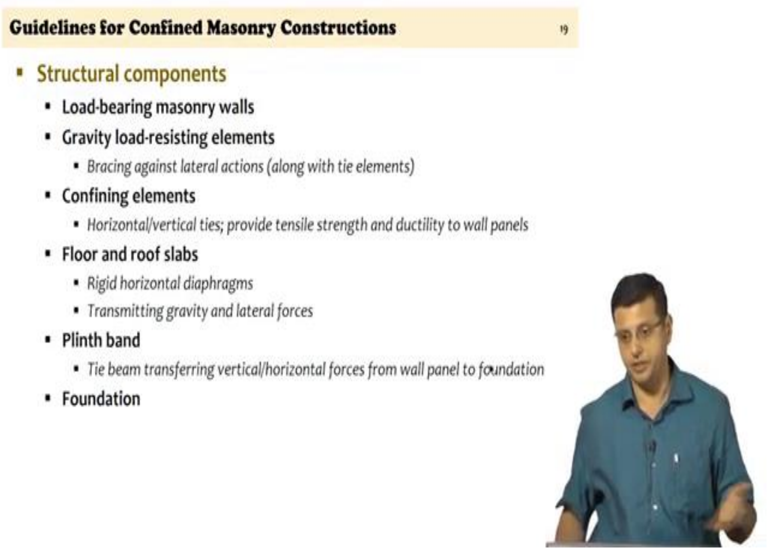
So, let me go over the typology and refer greatly to what the national building code has in terms of prescriptions for confined masonry construction. But it is also interesting to see, where the additional resistance, where the global resistance to earthquake, favorable earthquake response of confined masonry constructions come from. And, simple calculations can actually be done to estimate the improved flexural capacity of walls when you have confinement and improved shear capacity when you have confinement by the confining elements in reinforced concrete.

So, with reference to the national building code section on confined masonry; confined masonry itself is described as an alternative to reinforced masonry or minimally reinforced masonry. We have specially designed reinforced masonry where the ductility, the response reduction factor is as high as 4. We are not looking at a situation where the confined masonry construction is an alternative to that type of reinforced masonry; but minimally reinforced masonry with reinforced concrete bands and vertical bars and RC frame construction as well. So, this is an alternative to reinforced concrete frame construction.

The masonry has load bearing walls. So, gravity resistance is from load bearing walls, and these could be clay bricks, hollow clay bricks, or concrete blocks; but the element that is of importance are the confining elements, which are vertical tie columns or horizontal RC ties, called tie beams and they confine the masonry wall which is the load bearing element from all four sides.

Now, these tie members are typically of smaller cross section than the reinforced concrete frame sections of columns and beams, and that is an important difference that you must keep in mind. Therefore, these are not meant to be flexural elements as you would have in a frame building, in an RC frame building the columns and beams are meant to be flexural elements; but here these are tie elements and not defined as flexural elements. Their only role is, provide tensile resistance in a typology where you depending on unreinforced masonry walls for the gravity bearing role.

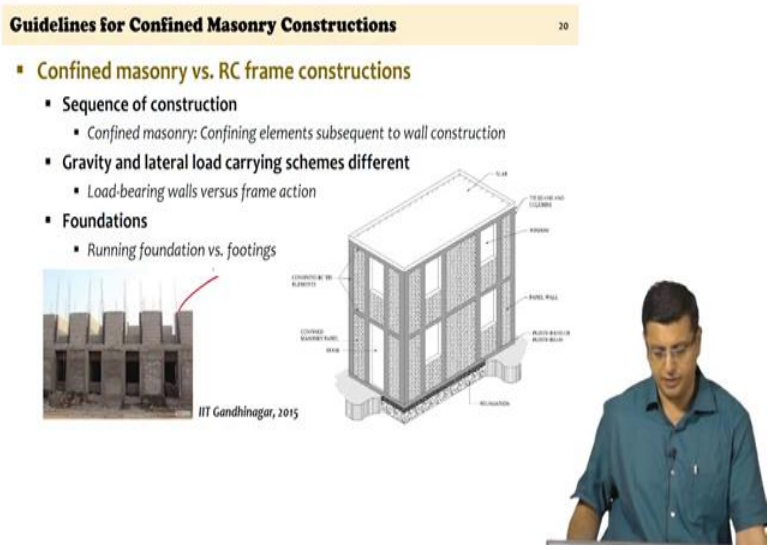
(Refer Slide Time: 04:42)



The different structural components of a confined masonry construction, you have the load bearing masonry walls, they carry the function, carry out the function of gravity load resistance; but this is this being a shear wall, actually this would be a shear wall it also works against lateral forces. So, it has a dual role, it is the shear wall for lateral actions along with the tie elements; that is where the interaction between the tie elements and the shear walls come about, but for gravity load resistance it is again the load bearing elements. The confining elements that we talked about these are horizontal and vertical ties, they provide tensile strength and ductility to the masonry wall panels which are unreinforced masonry wall panels.

You have floor slabs and roof slabs, be typically prescribed that these are rigid horizontal diaphragms when they are being constructed, and are capable of transmitting both gravity and lateral forces. You have to have a plinth band, which is one of the first tie elements in the structural system; the tie beam which is at the lowest level, transferring both vertical and horizontal forces from the wall panels, the masonry wall panels to the foundation. And of course, the foundation which is actually the running foundation, which we will discuss in a few minutes

(Refer Slide Time: 06:03)



So, if you are looking at the fundamental differences between a confined masonry construction and reinforced concrete frame construction; what would they be. The fundamental difference starts from the sequence of construction right. In a reinforced concrete frame structure, the frames are constructed first; you have the foundations, you have the different footings that are constructed or a raft foundation and then you have the framing elements. And then the infilled wall is the last element to be added, the nonstructural infilled walls, which are partition walls are added at the end.

However, in confined masonry constructions, the walls are constructed and then the confining elements are concreted ok. Assuming that the confining elements that we are talking about, the tie elements horizontal and vertical tie elements are in reinforced concrete, the wall is constructed. And after the wall is constructed as we can see in this picture, this photograph here, you can see that a gap is left at the edges of the masonry construction. You can see that, you have the steel reinforcement in position already and then concreting is done to complete the confining element right.

So, that is a fundamental difference; in confined masonry the confining elements are constructed subsequent to the wall construction, this is an important difference.

Student: Including horizontal ties.

Including horizontal ties, yes. So, when you are actually coming to the top of a wall, you have the reinforcement placed in position and then it is concreted yes.

Student: Sir can you make.

What you are saying is the wall construction is happening in say two or three lifts, in series of lifts and typically you might have a return wall; and the return wall provides out of plane resistance as the construction is underway. We are not talking of extremely long walls, these are residential building sizes. So, of course, they could be a potential issue if workmanship is not good enough and you can have out of plump in the walls. And I think that is sequence, I mean that is an accept that needs to be considered when the construction is happening; because you do not have the return walls. Some site interventions would be required to ensure that the wall is braced against if it is windy.

Student: (Refer Time: 08:19).

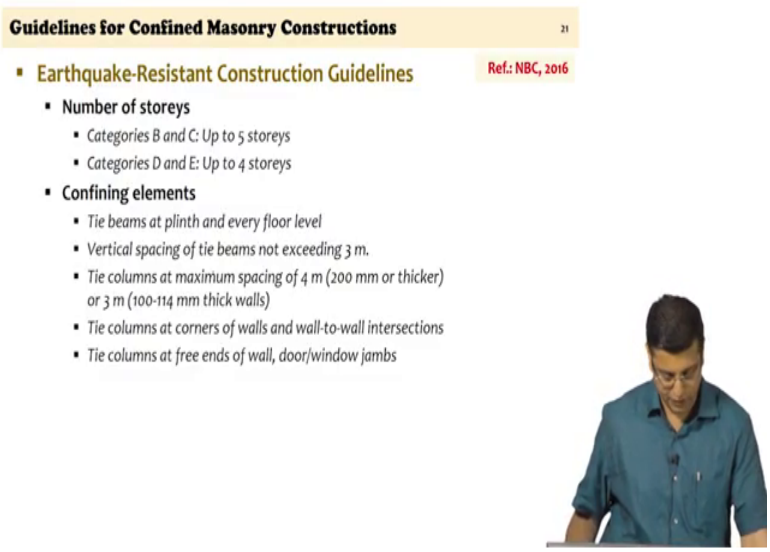
Windy exactly, but no other special requirement is really there, yeah. So, again fundamental difference in terms of behavior would be you are looking at load bearing gravity walls in a masonry construction and in the confined masonry construction versus framing action which is available in a reinforced concrete construction. The reinforced concrete frame construction has framing action. Here we are not saying that framing action is available, though the ties are all interconnected; we are not looking at framing action, we really looking at tying action that is there provided by these elements.

And finally, the foundations, so foundations in a reinforced concrete frame element, each framing member, vertical framing member the column would have its own foundation. It could be either a isolated footing, or a combined footing, or a raft foundation; whereas, in confined masonry constructions the confining elements, the tie elements do not have independent foundations, do not have their own foundations. They are sitting on top of the running foundation, which is the running foundation of the masonry load bearing wall themselves; but you would actually have a plinth beam which helps in connecting the steel reinforcement of the tie elements to the foundation element itself.

So, running foundation versus footing or may be a raft, but the flexural element in the reinforced concrete frame construction has it is own foundation, here you do not have, the tie elements do not have their own foundation. You will see in my slides, number of photographs which are from IIT Gandhinagar, which is an educational campus which has decided that some of the residential buildings there, particularly the faculty blocks are all confined masonry constructions.

So, we are talking of seismic zone III and multi storied ground plus two constructions, which has been designed and executed in confined masonry. So, these several of these photographs actually come from the drawings that have been prepared and the photographs that have been prepared about the campus itself, campus design and execution itself, particularly highlighting confined masonry ok.

(Refer Slide Time: 10:32)



What about earthquake resistant construction guidelines? There are quite of few, as I said the code does not require you to do a rigorous design, go through a rigorous design procedure to arrive at the dimensioning of the tie elements. Prescriptions are provided to ensure that, good earthquake resistance is available.

So, I will quickly go over the specific guidelines that you have in national building code; which we will then carry forward to understand how they should affect the behavior under lateral actions. So, prescriptions on number of stories; if you are looking at category B and category C buildings you can go up to 5 storeys, but if you looking at category D and category E buildings you can go up to 4 storeys.

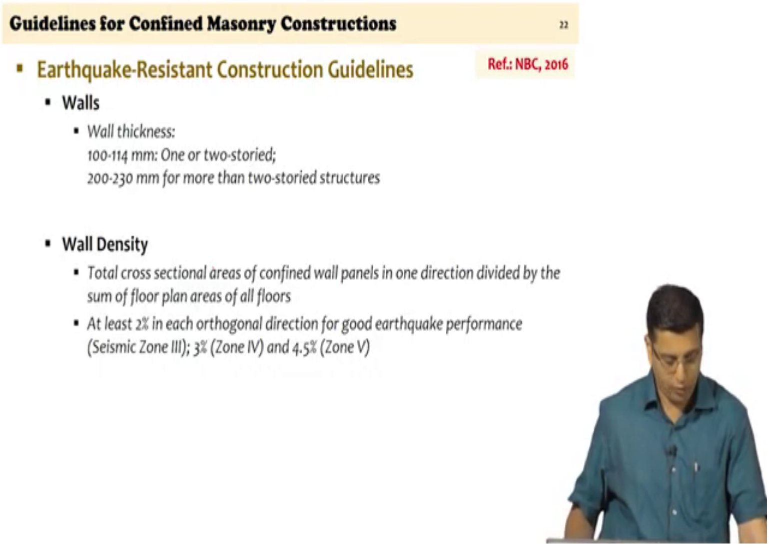
So, if you remember the IS 4326 tables where you have category B to category E; this code is then saying that, confined masonry is another typology that you can use to construct earthquake resistant masonry constructions. Reinforced masonry is one, un-reinforced masonry with the bands is the other one; but confined masonry could be the other typology that can satisfy basic earthquake resistance requirements.

The confining elements, you have to ensure that tie beams are provided at the plinth level and at every floor level, so that is minimum requirement. And that the vertical spacing between the tie beams should not exceed 3 meters, so mandating that every floor level you must have for small buildings; that every floor level you must have a tie beam beginning with the foundation. The tie columns are typically placed at a spacing of about 4 m, if the tie column dimension is 200 x 200 right.

So, you might have a load bearing masonry wall which is 230; 200, 230 or even larger; your tie column is typically limited in dimension. If it is 200 mm or thicker, every 4 meters you should have a tie column; but if you are looking at a thinner wall, if you are looking at a 100 mm or a 114 mm thick wall, then you are looking at tie columns that must be provided every 3 meters centre to centre.

Tie columns must also be provided in addition at corners of walls and wherever you have intersections of walls. And tie columns are also provided when a wall is has a free end. So, a door opening is actually a wall with the free end; which means, the door opening the jamb of the door should actually have a tie column built in at that location.

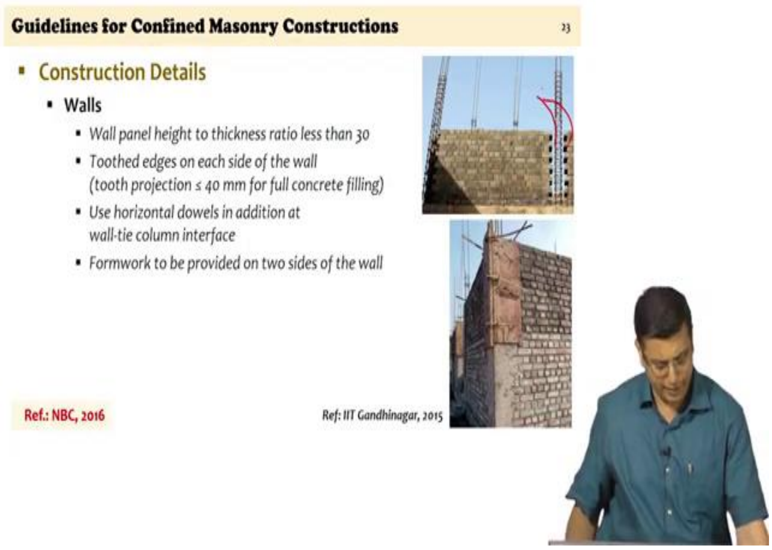
(Refer Slide Time: 13:11)



What about walls typically if you are looking at a one storied or two storied construction, 100 mm or 114 mm walls this is basically half thick, half brick thick wall is allowed. And if you are looking at two storied structures or more storeys, minimum of one brick thick wall is required. The concept of wall density is introduced and this is something we will again speak about when we talk of assessment in the next week. Wall density is defined as the structural plan density in a given story; it is the total cross sectional area of the confined wall panels, you take the wall panels areas and look at one direction and then look at the other direction, two orthogonal directions separately. So, it is the total cross sectional area of the confined wall panels in one direction divided by the sum of the floor plan areas in all the floors. So, you are looking at total percentage of wall resisting area in one direction versus another direction divided by the total plan area of the building in all the floors. So, basically wall density, structural plan density can be worked out floor wise and can be worked out for a building ok. Floor wise you will use only a plan area of a floor; if you are using for the entire building, then you would add up all the floor areas.

Now, what this represents is the percentage of resisting area available in a plan configuration. And there are limits on this and you will see, when we come to assessment that this check is one of the fundamental checks to see if the building has adequate resistance or not. As a very simple check on whether a masonry building has adequate earthquake resistant or not, the structural plan density of walls is something that we check. So, here the code requires for confined masonry construction that if you are in seismic zone III, you must at least have 2 percent in each direction; each of the orthogonal directions, which increases to 3 percent in zone IV and 4.5 percent in Zone V.

(Refer Slide Time: 15:22)



So, this is another check which becomes important. Of course, the guideline in terms of one brick thick wall or a half thick wall is from the earthquake resistance point of view; but we will also be checking if that wall cross section is sufficient from the gravity load resistance itself. So, there it is, it is the check based on the permissible compressive stresses.

So, those minimum dimensions have to satisfy both gravity requirements and the lateral load requirements as prescribe by the code ok. Since this is prescriptive in nature; the recommendations or prescriptive in nature, you have specific prescriptions for the different tie elements and protocols that must be followed when these are being constructed. So, we will quickly go over construction details for walls, tie columns, and tie beams and foundations and other aspects.

So, it is prescribed that the wall panel height to thickness ratio, h/t ratio of walls is kept below 30. And we have seen this number 30 comes back towards, it is identified that at about h/t ratio of 30, you have instability in the walls due to the slenderness effect itself. Another important prescription is how the edges of all walls must be toothed, right. So, if you see the picture here at the top, you see the, the way the edge of the wall is, the two edges of the wall are; toothing is in short such that there is a an offset created by alternate courses and it is prescribed that this projection be at least 40 mm is 40 mm or lesser

Now, the importance of this toothing has been verified experimentally and it is demonstrated that presence of toothing in every alternate course is beneficial to good interaction between the tie columns and the masonry, unreinforced masonry wall panel itself. In fact, wall panels which have been constructed without toothing, find it difficult to get confined effectively by the tie columns; because you have almost a smooth interface between the tie column and the masonry wall and the shear transfer is a difficulty between the two elements.

Student: Sir.

The whole idea is that you want interlocking between the two elements, because you want them to work together quite effectively; and the intention is to have shear interlocking achieved at that interface as a plastic concrete to support there and hardens.

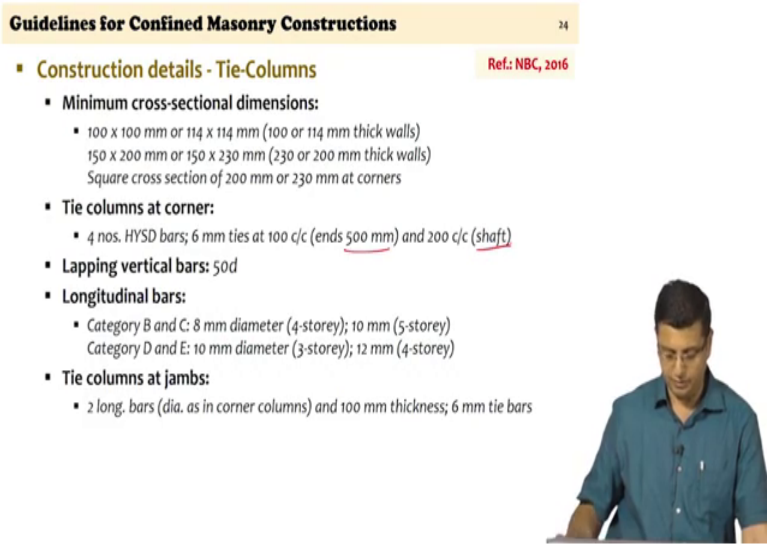
Student: And the form work of these.

Yeah.

Student: Closed to it or the day filling.

Exactly we will come to the form work in a moment; but the shuttering allows two sides of the wall, two edges of the wall to act as your shuttering itself ok. Where required you might want to use horizontal dowels between the masonry wall and the tie column itself. And formwork is provided on the two sides of a wall as you seen in this picture ok. If it is a corner, then you it will be two outer edges; and if it is a central tie element, then on either sides of the wall you would be providing the formwork element itself.

(Refer Slide Time: 18:53)

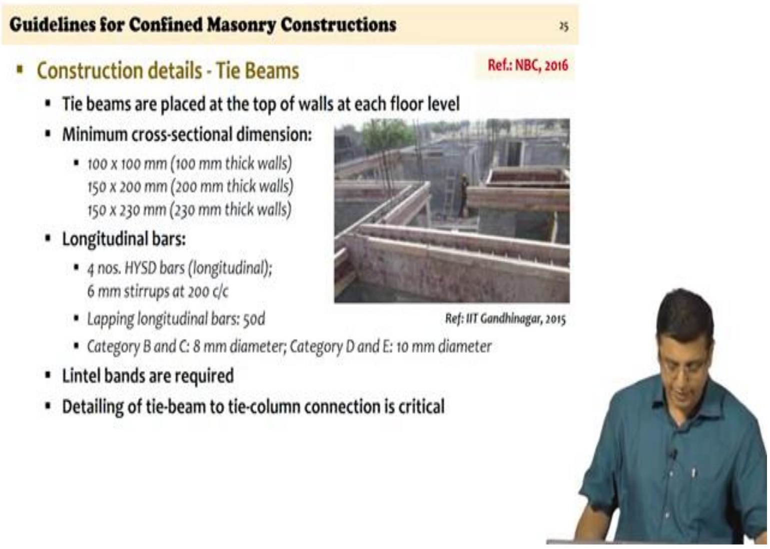


So, tie columns there are minimum requirements in terms what should be the cross sectional dimensions. Cross sectional dimensions can be square or rectangular and it is prescribed that whenever it is at the corner, these have to be square in cross section. And at least 200 mm or 230 mm one brick thick cross sections at the corners; whereas, in other locations they can be 100 mm x 100 mm or 114 x 114, 150 x 200, 150 x 230. So, rectangle or square cross sections are permitted, minimum dimensions are prescribed here.

The tie column at the corner, the corner is a location which is subjected to significant demand; and therefore, the tie column has minimum steel reinforcement requirement prescribed. And what the spacing of the tie reinforcement should be at the top and bottom ends of the tie column and the shaft of the tie column is prescribed. A typical overlapping of the reinforcement, the longitudinal reinforcement is prescribed as 50 times the bar diameter.

And, in terms of the longitudinal bars, depending on the category of the building and number of stories that you are looking at 4 storey, 5 storey, 3 storey; 4 storey the code prescribes what should be the minimum diameter of the four numbers of bars that you would incorporate within a tie element itself. Tie columns at jambs are basically to ensure that you have a tensile resisting element at a door opening or a window opening and this is not as critical as the other tie columns. And the number of bars here is reduced; it is two longitudinal bars and dimensions on the tie bar and thickness of these element is also prescribed.

(Refer Slide Time: 20:52)

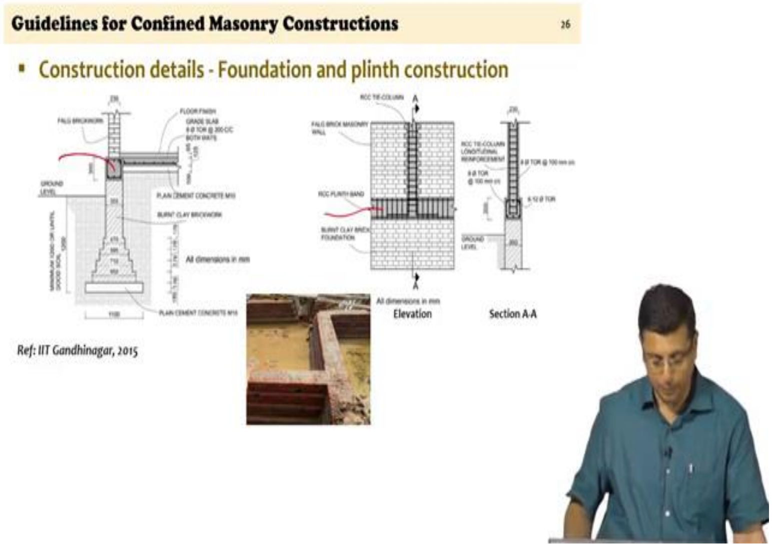


Similarly, you have prescriptions for the tie beams and these tie beams must run continuously along all the load bearing walls; they placed at the top of the walls at each floor level as we discussed earlier. Again, there are minimum cross-sectional dimensions which must be respected. You have the depth cross sections mentioned here; longitudinal bars again; how many bars, four bars and at what spacing should the stirrups be provided.

Lapping of the longitudinal bars also again at 50 times the bar diameter. This, the prescription of longitudinal bars again as we have seen for IS 4326, comes back here depending on the category of the building, the bar diameter is prescribed. And in addition, lintel bars are to be provided. Now these are not like the plinth beams or the floor beams, these are lintel bands which run continuously at the level of the lintel itself.

And you can use the IS 4326 recommendations, where you have either four bar lintel or a two bar lintel which is the continuous lintel in the construction. Again, it is required that the steel reinforcement of the tie columns and the tie beams are integrated; such that the necessary tensile strength is achieved and that detailing if not provided your tie columns and your tie beams are not going to act as efficient or effective tie elements

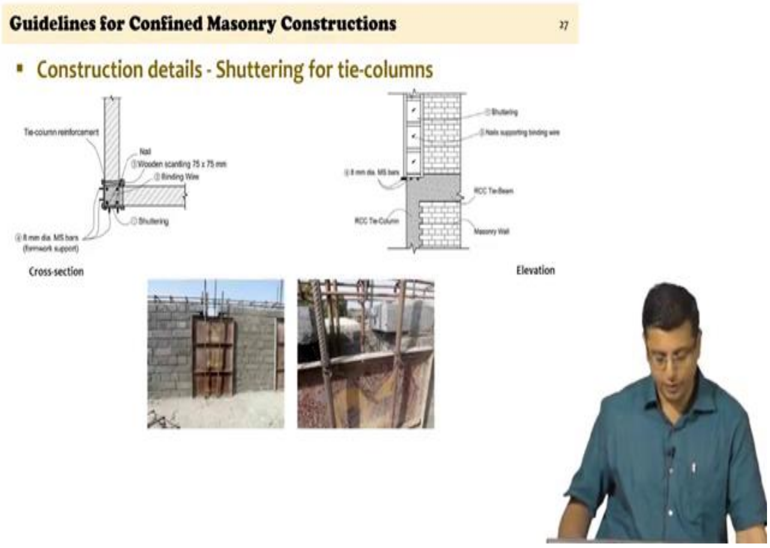
(Refer Slide Time: 22:21)



So, that is another aspect to be kept in mind. In terms of the foundation and plinth, so as I said the kind of construction, the kind of masonry typology that we are looking at; it is a running foundation that is typically used. So, this is a regular masonry foundation; a plinth beam is provided above the, the plinth beam is to be provided continuously above the running foundation. And then the masonry wall construction proceeds; but from the plinth beam you have the locations of the tie columns, the steel of the tie columns are embedded and tied with the steel reinforcement of the plinth beam itself.

So, that is the running foundation of your wall and then you would have the plinth beam, the reinforced concrete plinth beam and steel reinforcement of the tie columns actually being connected to the steel reinforcement of the plinth beam itself.

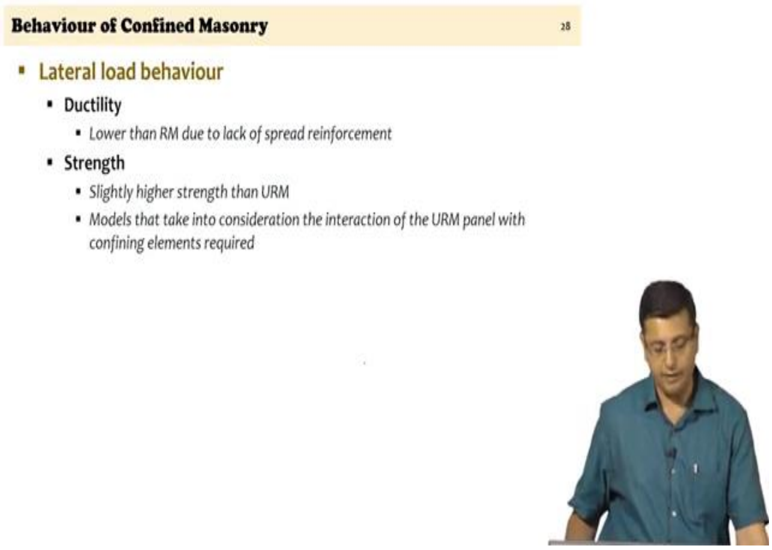
(Refer Slide Time: 23:17)



We have been mentioning the shuttering for the tie columns. So, while the two sides of the tie column shuttering is provided by the toothed edges of the brick wall itself; because the brick wall is already being constructed or the masonry wall is already constructed. The other two sides would require shuttering to be provided, and that is what creates the formwork for the RC tie element.

So, you can see how, in this construction you have the toothed joint at the two sides and shuttering on the two edges; which then has to be held in position you can tie together and ensure that tight watertight formwork for the RC tie element is available.

(Refer Slide Time: 24:05)



So the reference here is largely to what the national building code has for reinforced, for confined masonry constructions. The second half of this presentation I would like to focus on, the behavioral aspects of confined masonry construction, particularly under the action of lateral forces. So, what really happens in a confined masonry wall, is something that I would like to discuss; and see how we can then extend that to our estimates of in-plane flexural capacity and shear capacity of the confined masonry wall.

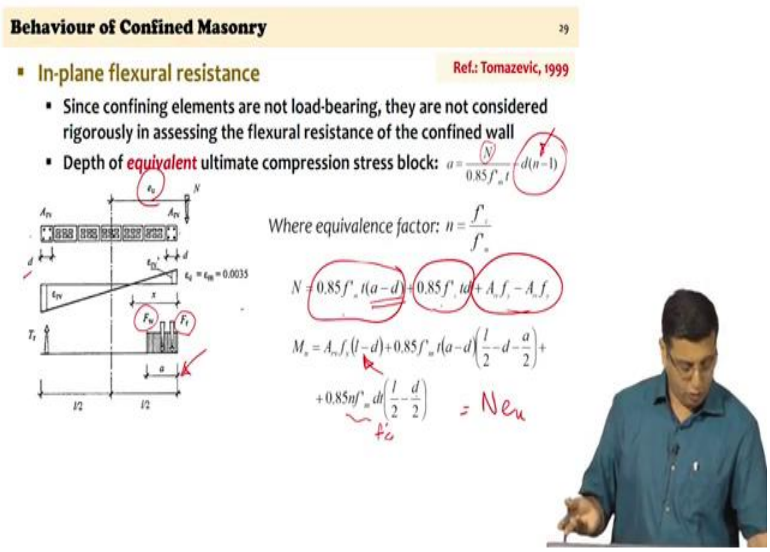
So, the lateral load behavior in terms of ductility of a confined masonry construction; this is expected to be lower than what are reinforced masonry construction can provide. But definitely higher than what, an unreinforced masonry construction is going to provide; why is it lower than the ductility of reinforced masonry construction? Simply because we are not looking at spread reinforcements, spread reinforcement is going to give you better ductility.

Spread reinforcement is not available, we are looking at tie elements, tie columns which are coming at 3 meters or 4 meters centre to centre spacing. So, ductility is expected to be lower than reinforced masonry. What about strength? This is expected to be more than what an unreinforced masonry wall can give because of the interaction and because of the confinement that the tie element can give.

So, premature cracking of the unreinforced masonry wall is prevented by the tie element; thereby that confinement can improve the shear resistance of the unreinforced masonry wall. Therefore, what becomes important if you are actually going to make rigorous calculations on capacities, which the code does not require; but if you were to design something which is not a simple confined masonry construction, you will have to make calculations, you have to make ultimate capacity checks.

And any model that you look into must have the interaction effects of the unreinforced masonry wall panel with the confining elements. And there is a body of literature particularly coming from Eastern Europe; works by Tomazevic and group in Zagreb and also from South America on analytical approaches for understanding the lateral load behavior, strength and the deformation capacity of confined masonry constructions is available

(Refer Slide Time: 26:48)



And you can, we are also looking at a couple of such works to understand the in-plane flexural capacity and the in-plane shear strength of confined masonry constructions. So, examining what actually happens in terms of the in-plane flexural resistance of a confined masonry wall. As discussed earlier we are not looking at the elements, the RC elements as providing flexural resistance, they are not providing flexural resistance, they are meant to be providing, they act as tie elements, tensile strength is available and that creates a confinement.

So, making calculations, assuming that these are flexural elements is erroneous; you will not get that effect from the small amount of steel reinforcement in the small cross section of RC elements that are being provided ok.

So, that is something to be considered. So, let us look at a masonry wall, with two tie columns at it is two ends; the tie columns each are provided with four reinforcement bars and have an area of cross section A_{rv}. The depth of the confining element is d, as you can see here; depth of the confining element is designated as d. And there are two confining elements at the two ends and the confined masonry wall is at the center.

Now, to be able to establish what is the ultimate capacity in bending, in-plane bending of this wall; there are different models available and the model that is being refered to here is from a work of Tomazevic and colleagues from Eastern Europe. So, what is really being considered here, the concept behind the estimate of the ultimate flexural resistance is that with the presence of a confining element at the two edges of the masonry wall; An unreinforced masonry wall at its ultimate, we have seen a flexural capacity estimate; where you have the end stress block, when masonry reaches it is ultimate strength in compression.

However now you need to make a modification to this end block in compression; because you have right beside it, a concrete block which is the cross section of the tie column, which has tensile resistance; and therefore, the edge block, the edge compression block is a different geometry and also has two different materials - one with steel reinforcement, one without steel reinforcement.

So, Tomazevic here makes use of an equivalent ultimate compression stress block and the width of the stress block ‘a’; as you are seeing here, it has two resisting elements, it has resistance in compression coming from the brick wall, the wall which is unreinforced and the reinforced tie column element right.

So, the model here talks about how you can actually take into account the confining effect of the reinforced concrete element and gives an expression for the equivalent ultimate stress block. I will come to the; of course, here we are looking at the equivalent stress block which is the same as in a reinforced concrete or a reinforced masonry stress block that we looked at, where the axial force resultant, the axial force is equilibrated by (a) $(0.85f_m)$ t.

So, that component remains the same, but you have this additional component where d is the cross-sectional dimension of the confining element. And 'n' here is the other aspect which brings in the role of two different materials which are resisting at this critical location. So, this is called an equivalence factor n and this equivalence factor is arrived at as the ratio of compressive strength of concrete to the compressive strength of masonry.

So, we are looking at two different materials; you have grout and you have steel reinforcement and the other material is masonry. So, n can be 1, n can be greater than 1, n should not be less than 1, typically. So, in the first case where the steel reinforcement in the tie column, which is in compression is also considered; then the axial force resistance, the total axial force equilibrium comes from the location of this stress block. So, total N is equilibrated by $0.85f_m t(a\text{-}d)$; which is the resistance coming from masonry, the first part is resistance coming from the masonry at ultimate.

The second part is the resistance coming from the concrete block which is 0.85 f'_c t d. And you have resistance, you have to account for the tensile force and the compressive force that the edge elements are going to be giving you. And that is why it is A_{rv}, which is the area of cross section of the reinforcement in the tensile side, in the compression side plus and minus depending on whether that has yielded.

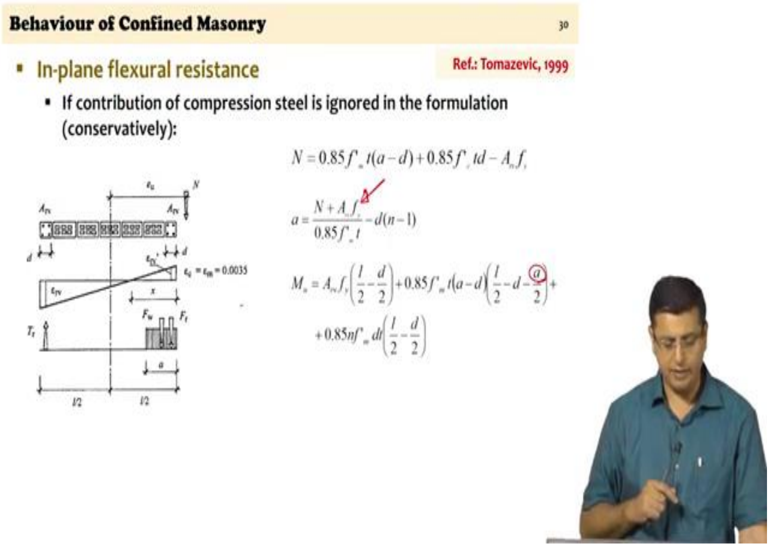
So, in this particular case we are accounting for the compression reinforcement in the equilibrium itself. So, from this the estimate of the ultimate moment is arrived at as the axial force, the total axial force into the eccentricities and each of these elements have different eccentricities. And therefore, the total moment resistance at ultimate is arrived at by multiplying each of these contributions into the lever arm that is available.

The lever arm for the steel reinforcement is l-d (which is 2 times d/2), which is d for the steel reinforcement in the edge confining elements.

$$M_u = A_{rv} f_y (l-d) + 0.85 f'_m t(a-d)\left(\frac{l}{2} - d - \frac{a}{2}\right) + 0.85 n f'_m dt \left(\frac{l}{2} - \frac{d}{2}\right)$$

So, in this calculation, we are considering the contribution of the compression reinforcement.

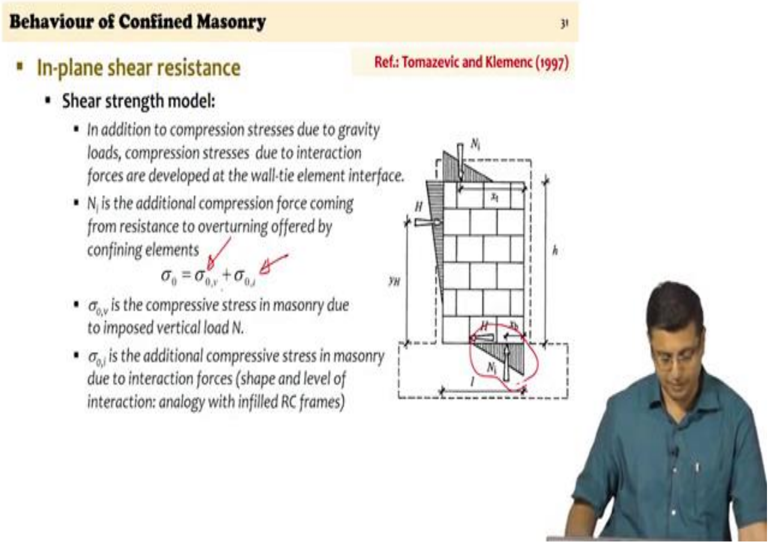
(Refer Slide Time: 34:52)



You could conservatively neglect the contribution from the compression reinforcement; if that is so, we are not considering in the axial force equilibrium, the contribution from the compression reinforcement is not added. You see that only the minus A $_{\text{rv}}$ into f y component is included. And therefore, from this estimate, if the equivalent stress block is estimated; you will have to account for the fact that, you have only one force that you are bring into the axial force equilibrium.

So, the stress block is estimated as a and similarly the total calculation of the ultimate moment can be made where the difference will be the value of a being different now with respect to the previous calculation; where we consider the contribution of the compression reinforcement. So, this is you see that there is a contribution coming from the confining element; but not treating the confining element as the flexural element contributing to in plane flexural resistance itself.

(Refer Slide Time: 36:02)



So, this is as far as in plane flexural resistance is concerned; but if you were to consider the shear resistance, the model that is most appropriate for calculating the in plane shear resistance of confined masonry constructions is the shear strength model developed by Tomazevic and Klemenc. In this particular model, we are looking at additional contribution to axial compressive stress coming from the interaction between the tie element and the wall panel.

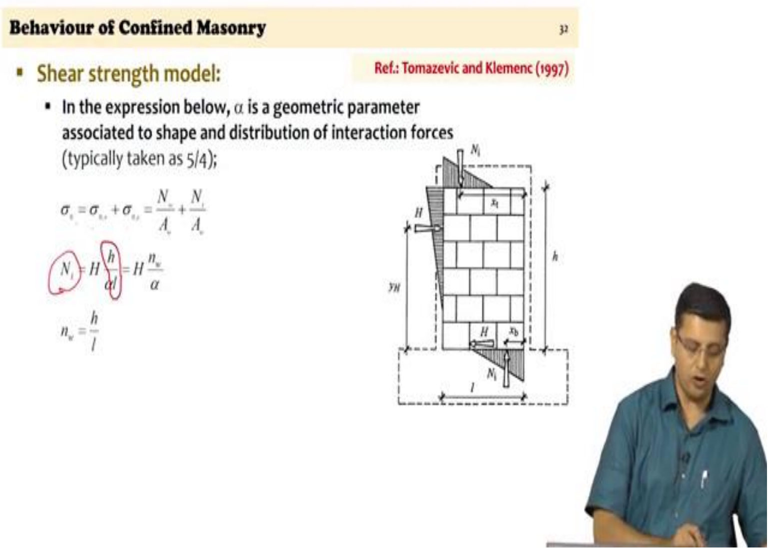
So, when the wall is subjected to gravity plus lateral forces, there are compressive stresses generated due to the gravity force; but there is an additional compressive force and stress generated in a cross section due to the confinement and the interaction between the tie column and the wall. So, we really looking at this portion, you have the compressive stresses coming from the gravity forces; but due to the interaction between the tie column and the wall, you have additional compressive stresses which are generated.

And, if an estimate of these additional compressive stresses can be made and used in the estimate of the shear strength of the masonry wall; you are implicitly accounting for the effect of the interactions. So, that is the basis of this approach; here N_i is the additional compression force that we are looking at, which is offering resistance to overturning, that confining elements are offering resistance to overturning.

So, what is happening is, the masonry wall should actually be overturning; but you have a confining element and that confining element is going to increase the compression at the compressed toe. You are accounting for that and accounting for that, you are implicitly bringing in the contribution of the confining element in the improvement of the shear capacity of the masonry wall. So, here we are saying that the net, the total compressive stress comes from two contributions. One is due to the gravity forces, and the other is due to the interaction; v is due to the vertically align gravity forces and i is due to the interaction itself.

So, compressive stress in masonry due to the vertical load is $\sigma_{0,v}$ and $\sigma_{0,i}$ is the additional compressive stress in masonry due to the interaction forces. And these interaction forces, actually the basis for these interaction forces come from studies which are based on infill panel, masonry infill panel and RC column element interaction. And so, it is an analogy and the analytical formulation borrows from studies on infilled RC frames and their shear capacity itself.

(Refer Slide Time: 38:50)



So, in this calculation we are really looking at, if it is possible to analytically, if it is possible to estimate how much N_i can be, and then use that in the estimate of total compressive stress; and then go back to one of our shear strength models for unreinforced masonry, and estimate the strength of the confined masonry wall. So, you see an expression, where a geometrical parameter is introduced; a further geometrical parameter is introduced.

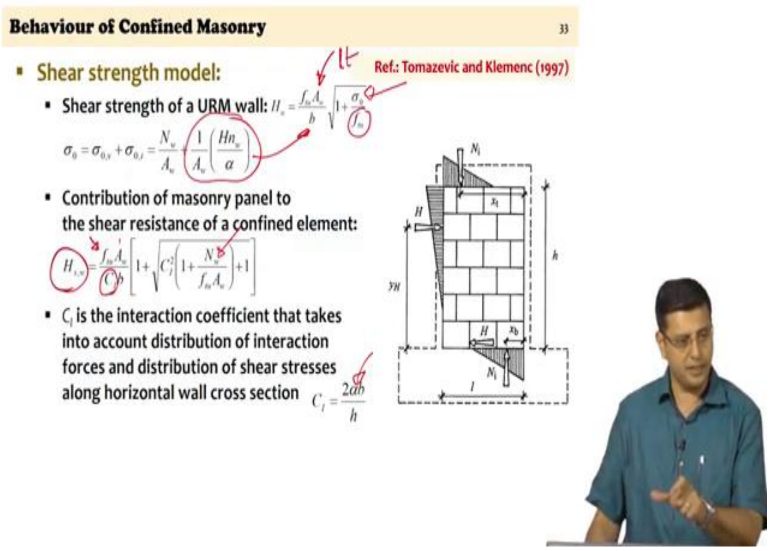
And this takes into account how much of the interaction force is to be expected, because of the shape of the wall itself. Because if you have a significantly large wall in terms of it is length, so if you look at a h/l ratio; and if it is a very squat wall and you do not have sufficient number of confining elements, the effect of the interaction is going to be lesser. If you were to provide significant number of confining elements, the interaction forces should increase.

So, it is link directly to the overall shape and you have a geometrical parameter that comes into this estimate which takes into account what this value should be. So, based on calibrations with experimental results this value is taken as 5/4. So, $\sigma_0 = \sigma_{0,v} + \sigma_{0,i}$; where $\sigma_{0,v} = \frac{N_w}{A_w}$. And the other force is what we need to estimate, which is N_i /A_w; this N_i itself is established from the lateral force.

The lateral force acting on the wall, because it is based on how much lateral forces acting on the wall that you get the confinement effect generated.

$$N_i = H\frac{h}{\alpha l} = H\frac{n_w}{\alpha}$$

(Refer Slide Time: 41:11)



So, looking at the final form of the expression by bringing this consideration; so, what we really doing is, in our earlier theory in the third module we will looking at, how the shear strength of a masonry wall can be estimated. And we have arrived at expressions that for diagonal tension failure mechanism, we know what will be the closed form solution for the ultimate shear strength of the masonry wall. So, here in the previous expression we would have put lt here; but and replacing that with A_w which is the area of cross section of the wall.

So, this is a familiar expression where f_{tu} is the tensile strength of the referential or conventional tensile strength of masonry. So, this is the expression for ultimate shear force, when a diagonal tension mechanism is occurring. So, we use that as the basis of the shear strength; but then enhance this, because you have σ_0 here and this σ_0 can now be replaced with not only that coming from compressive stress due to the vertical load, but also due to the interaction force.

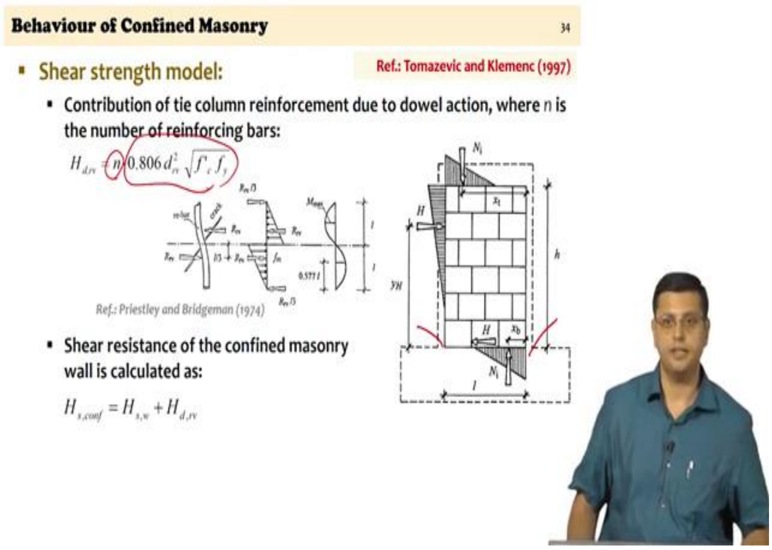
$$\sigma_0 = \frac{N_w}{A_w} + \frac{1}{A_w}\left(\frac{Hn_w}{\alpha}\right)$$

So, what we merely do is, σ_0 which has these new expression, this additional component comes in and, is introduced into the original unreinforced masonry shear strength formulation and with the series of adjustments and removing H from the right hand side, we get an expression for the shear force of a confined masonry wall based on the same parameters f_{tu}, area of the wall and the estimate N_w which is the part attributed to the gravity forces. b is the width of the wall, there is another parameter C_I which is brought in, which is nothing but $\frac{2\alpha b}{h}$.

So, this is simplification alpha is the parameter that we talked about earlier and adopted as 5/4. And, this is understood as the interaction coefficient, this is the one that actually accounts for this contribution. Interaction coefficient that takes into account the distribution of interaction forces along the entire, along the length of the wall where interaction effects come into play. It is the interaction coefficient which takes into account, the distribution of interaction forces and distribution of shear stresses along the horizontal wall cross section itself

So, this helps us estimate one component of the shear strength of the confined masonry wall, which comes because of this confinement.

(Refer Slide Time: 44:02)



There is another contribution which is actually due to dowel action. So, if you look at the confining elements at these two locations, these have steel reinforcement; and as there is shear deformation in the wall, they will be dowel action of the steel reinforcement. So, it is found to be not a negligible contribution; and therefore, the total shear strength of the confined masonry wall is the summation of a contribution coming due to the confined unreinforced masonry wall and that coming due to the dowel action.

So, dowel action is considered in the bars of the confining elements. So, depending on the number of reinforcing bars that are present in the confining element; this again comes from studies on reinforced masonry walls with vertical and horizontal reinforcements. And an analytical formulation that was developed for the contribution due to doweling action in a reinforced masonry wall with distributed vertical reinforcement.

Priestley and Bridgeman's work is what at it feeds into and basically you get an estimate of what this contribution is. Each vertical reinforcing bar is expected to give that as a contribution the value of $0.06d_{rv}^2$; which is the diameter of the vertical reinforcing bar into square root of the compressive strength of the grout and the compressive strength and the tensile strength of the steel itself.

So, you see that it is based on the overall mechanics of the dowel action and the distributed bending moment that you will get due to the contact forces between the dowel and the grout itself. And this is multiplied by the number of reinforcing bars that you have in the cross section. So, finally, the total shear resistance of the confined masonry wall is calculated as the summation of the earlier calculation that we made, to understand what is the contribution to shear strength of the confining element and then what is the contribution coming from dowel action.

So, together we will arrive at the overall shear resistance of the confined masonry wall, which is expected to be higher than the unreinforced masonry wall shear strength itself. With that I conclude the discussion on the behavior of confined masonry constructions. I have discussed prescriptions that are mandated by the national building code for construction of confined masonry constructions. But if you were to look at rigorous approach by estimating the shear capacity and in plane bending resistance; then there are simple analytical and semi empirical calculations based on mechanics that you can use to be to make those checks.

And it is demonstrated by several experimental studies, and even actual earthquakes, including large earthquakes in countries like Chile; that this is a typology that is significantly simple to execute, you do not need heavy engineering input, but gives extremely good performance in large earthquakes. So, with that I conclude this part, which is the second special topic that I am addressing within module 5.

Thank you.