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

Module - 04
Lecture - 23
Design of Masonry Components and Systems Part - II

So, we continue this afternoon our lecture on design and we are examining design with respect to the code dealing with permissible stress approach for unreinforced masonry, so that is what we began looking at.

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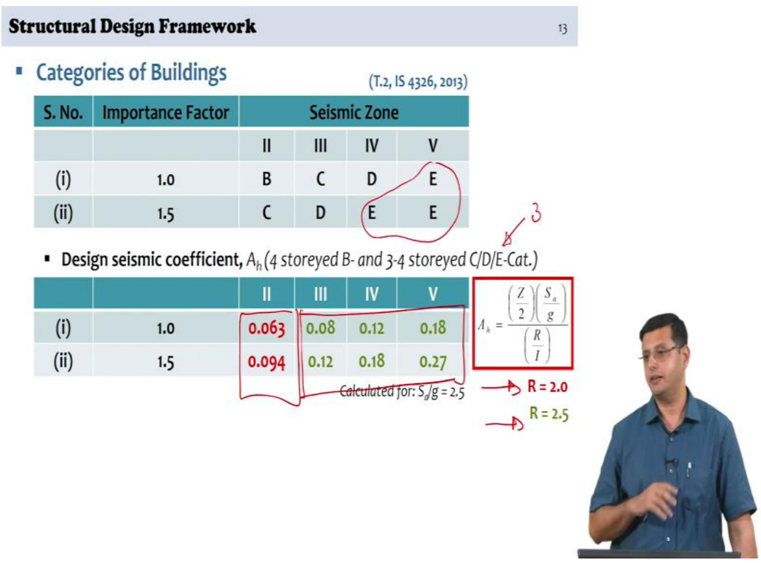

To sum up what we examined in the last class in terms of the overall structural design framework where we were linking up the seismic code IS 1893 Part 1, the code of practice for earthquake resistant constructions IS 4326, the national building code which has segments that deals with structural design of unreinforced masonry, and the new section on the reinforced masonry. This is what we had looked at in the last class where based on the importance factor of the buildings, and based on the seismic zone in which you are designing the structures, we classify them as B, C, D and E categories.

And you see that the importance factor would lead us to a zone 2 building with an importance factor of 1.5 being a category C building. And, then we will examine the details under each category at the appropriate time, but the point is category B, C, D and

E would mean there are specific prescribed interventions, design interventions to ensure seismic resistance. And, this is different from the earthquake load that you would define based on which zone the structure is sitting in and what importance factor it has.

And so based on a question that was raised in the last class, I thought it will be instructive to look at what is this effect of one on the design side, the implication in terms of the lateral seismic coefficient that we estimate as the shear force demand on the structure versus the detailing that is required as per IS 4326. So, I just picked up the application- I just am just looking at in this slide the application of this particular requirement in terms of category of building evaluated as far as the design seismic coefficients are concerned. So, I am just mirroring the table on the top in terms of the actual values of the seismic coefficients that we will be able to estimate.

So, I am making a distinction here based on the number of stories and that is important because while buildings are categorized as B, C, D and E, the code IS 4326 further looks at structures which are 1-2 storied structures or 3 or 4 storied structures, so that distinction also needs to be brought in. So, I am examining the design seismic coefficient A h from IS 1893 estimated for a B category building which may be 1 to 3 storied - 1, 2 or 3-storeyed; and C, D or E category buildings 1 or 2 storied of that category, C D and E category.

So, the design seismic coefficient is estimated as given here.

$$A_h = \frac{\left(\frac{Z}{2}\right)\cdot\left(\frac{S_a}{g}\right)}{\left(\frac{R}{I}\right)}$$

You have the zone factor coming from which zone the structure is sitting in. So, Z/2 is used for the design earthquake multiplied by the spectral ratio S_a/g. And I am assuming that we are taking the maximum value of the spectral ratio here, 2.5 divided by the response reduction factor which we talked about yesterday R/I, where I is the importance factor of the structure that you are designing.

So, if A_h is estimated in this manner then for this particular categorization you have to use the shear force for design. And, the shear force for design is estimated based on differing R factors, and this is something that you need to look at which means if I am estimating for such a building sitting in zone 2 importance factor 1, which becomes category B building 1 to 3-storeyed category B building, I am estimating the seismic coefficient and so on for each category. However, since we are looking at a category B building or a category C building where the requirements the earthquake requirement, the earthquake resistant detailing requirements stop with the horizontal seismic bands.

Since we are talking of unreinforced masonry buildings with horizontal seismic band, the R factor that needs to be used in this case is an R factor of 2. However, as we move towards a D category building or an E category building, they are introducing vertical reinforced concrete elements along with the horizontal seismic bands which means now for the estimate of the lateral seismic coefficient for these values here, we will actually be using an R factor of 2.5.

So, while the table itself shows you that these are both C category buildings, these are both D category buildings, these are both E category buildings, the values of the design seismic coefficient need not necessarily be the same. Here it is the same; here it is the same; however, in these two they are the numbers might work out to be slightly different.

But the intention is that on one hand the design forces are estimated based on the correct use of the R factor based on the type of building that are looking at. On the other hand, you have the prescriptive requirements of the seismic resistant features that need to come in based on IS 4326. So, it is essential to examine it carefully as you are designing such a structure and implementing earthquake resistant features as per 4326.

The same set of calculations now extend it to a 4-storeyed B category building, and a 3 or 4 storied C and D category building. For the E category building, if a building is in category E which is fives to a zone 5 structure of importance factor 1 or zone 5 structure of importance factor 1.5 or zone 4 importance factor 1.5. The E category buildings are typically I mean you have to stop at a 3 storied structure. So, the maximum number of storeys permitted in the E category is 3 storeys; if you are in category D you can go up to four storeys.

So, if you are looking at this sort of a condition, where you looking at a four-storied B category building, or a 3-storeyed C and D category and a 3 storied a category building then using the same assumptions when you estimate what actually happens is for only these two categories, B and C would you be using an R factor of 2, whereas for the rest of the calculations you will have to use an R factor of 2.5.

So, this is something that you need to examine carefully based on specific features coming in as far as B, C, D, E categories are concerned, and what R factors need to be used therefore, to estimate the design seismic coefficient. So, this is an aspect which you might have to spend some time such that there is consistency between these three codes that I started talking about yesterday as far as the structural design framework is concerned.

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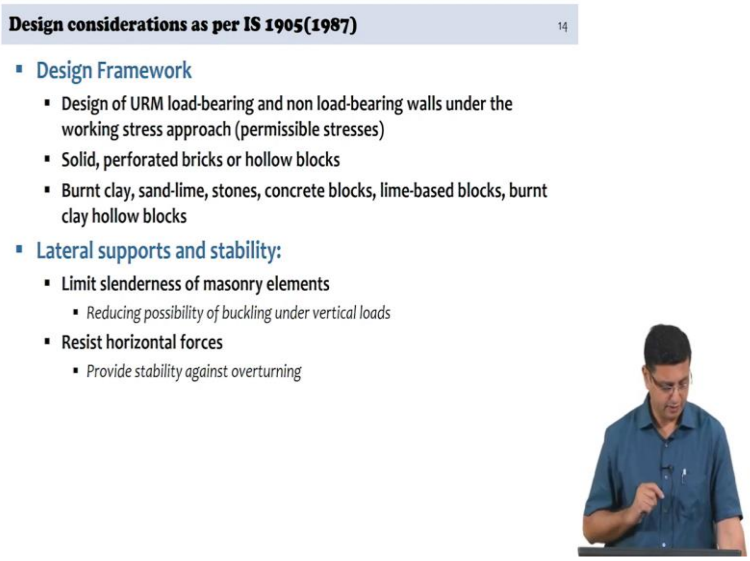

So, with that I intend to move into, in the next two lectures, intend to give you a very broad coverage of what key aspects IS 1905 the design standard for unreinforced masonry as a structural material, touches upon. Of course, you have the code and you have the exploratory handbook. As I had mentioned earlier a thorough reading of the two will give you an overall idea of the several prescriptions that this code makes. I would in the next two lectures go through only specific aspects, key aspects which form the basis of the design itself.

So, we have talked about lateral resistance being important as far as masonry constructions are concerned, and that is really the intent with which I linked up the code for unreinforced masonry as a structural material, the 4326 recommendations and the 1893 design seismic input requirements. However, as far as 1905 is concerned, it does not explicitly talk about seismic design right. The code does not explicitly talk about seismic design; however, lateral resistance is something that is given adequate coverage as far as how you can achieve a minimum amount of lateral resistance in the structure. So, the IS 1905 the code itself deals with both load bearing walls and non-load bearing walls and uses the working stress approach. This working stress approach is also extended to the reinforced masonry clauses and the section in the national building code of 2016.

So, there is a consistency in the approach to design of masonry as far as Indian standards are concerned. We are continuing to use the permissible stresses approach, the working stress approach for the design itself. The code 1905 deals with all types of masonry units. It is valid for solid masonry units perforated bricks or even hollow bricks if you are using hollow concrete blocks as well. And again, in terms of materials, it is meant to address right from the burnt clay bricks to stone concrete blocks, all the modern blocks that you are looking at. And therefore, the structural design of any type of this material is governed by the code 1905 ok.

The importance given to lateral load resistance is something that keeps coming back in in different aspects of the code. The implicit understanding in the code is that by limiting the slenderness ratio, the slenderness of the masonry elements here we are talking of the load bearing walls, by limiting the slenderness of the masonry elements you are getting desirable behavior. So, the idea is to have limits on what is allowed in terms of the slenderness ratios of the masonry elements and that inherently should be able to provide lateral stability to the system.

So, while the intent with which slenderness ratios are being discussed in the code is to prevent buckling under vertical roads, the slenderness ratios if they are limited would also provide lateral resistance to the wall particularly from preventing out of plane mechanisms. And for assisting horizontal forces, we talk of wind forces or we talk of earthquake induced forces. These slenderness limits and the other requirements particularly on the lateral supports ensure desirable performance of the wall both under gravity forces and overturning due to lateral forces.

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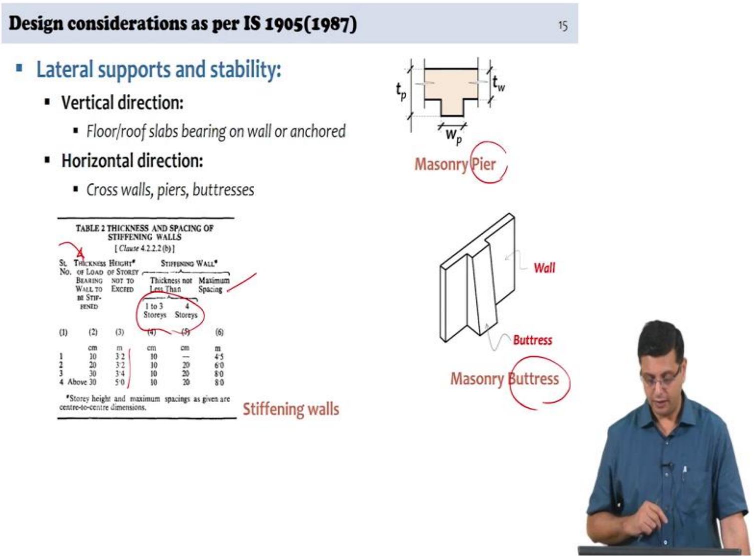

TABLE 2 THICKNESS AND SPACING OF STIFFENING WALLS

[Clause 4.2.2.2 (b)]

Sl No.	Thickness of Load Bearing Wall to be Stiffened	Height* of Storey not to Exceed	Stiffening Wall* Thickness not Less Than 1 to 3 Storeys	Stiffening Wall* Thickness not Less Than 4 Storeys	Stiffening Wall* Maximum Spacing
(1)	(2)	(3)	(4)	(5)	(6)
	cm	m	cm	cm	m
1	10	3·2	10	—	4·5
2	20	3·2	10	20	6·0
3	30	3·4	10	20	8·0
4	Above 30	5·0	10	20	8·0

*Storey height and maximum spacings as given are centre-to-centre dimensions.

So, in two directions, if you are looking at the vertical direction of the of the structure along the height of the structure, the prescriptions on how should the floors be how should be, how should the beams rest on walls what sort of anchorage of connections are required to have desirable performance or if you do not have positive connections between the floor slabs and the roof slabs, then what sort of an effect does it have on the lateral stability of the walls is something that is directly considered by the code itself.

In the horizontal direction, in the horizontal direction, the lateral stability of the of the wall is taken into account by provisions of cross walls to shear walls; provisions of buttresses of piers were required to ensure that there is lateral stability to two walls. So, I would just like to for a moment, we have been using the word pier as a masonry panel that is a lateral load resisting vertical element.

However, the code IS 1905 defines piers in a slightly different manner, and you just have to be conscious what it is talking about. And we will continue to talk of the masonry panel as far as shear resistance is concerned as a pier. So, here what the code refers to as a masonry pier is basically a thickening of the wall that is you have a wall of a certain thickness t.

But if there is a certain projection in the wall which provides additional lateral resistance, then that element is called a pier. And this pier has a constant extension along the entire height of the wall that is if you take t_p as the thickness of the wall including the pier and

t_w as the thickness of the wall, t_p - t_w remains constant along the entire height of the wall. Width of the pier again is something that is based on how much of lateral resistance the wall is provided by the pier itself. So, the pier is considered as an element that provides additional lateral stability to the wall. However, a buttress is also something that is used and traditionally used in masonry constructions, but in some cases, you might need buttressing of walls to provide greater lateral resistance.

The main difference between a buttress and the pier is that the pier does not change in dimension along the height, whereas a buttress is typically larger at the base and tapers as it goes towards the top of the structure. So, in terms of terminology, pier is different from what we have been talking about so far, and a masonry buttress is different from a masonry pier.

The code requires that each shear wall is provided with sufficient number of cross walls. So, if you remember our discussion during the initial part of the course, that the lateral load resistance in modern masonry constructions is by the judicious distribution of walls, shear walls and cross walls along the entire plan of a structure. Therefore, provision of cross walls as stiffeners to the main wall is something that is a requirement that ensures there is lateral stability available to the masonry wall, masonry wall load bearing wall itself.

So, the code talks about these stiffening walls, and prescribes at what spacing should you be providing these stiffening walls which in a way regulates what are the plan dimensions that you can allocate for different rooms, you know you know floor plan at the architectural design level itself. So, depending on the wall that is going to be stiffened with a cross wall, and number of storeys that the wall is and the structure is going to be raised to whether it is 1 to 3 stories or 4 stories.

And depending on the inter storey height, the maximum spacing between these cross walls to a shear wall is prescribed right. And the thickness of the stiffening wall is also prescribed. So, the code moves between prescriptive requirements and some design checks that you need to do. So, this is among some of the important prescriptions which will ensure you have good lateral stability in the masonry construction itself.

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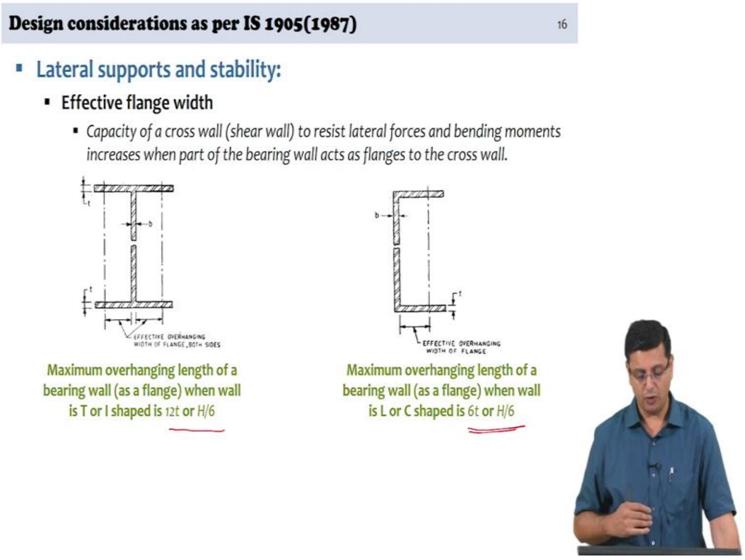

Now, when you are making calculations on the slenderness ratio of a wall, when you are making calculations on stability resistance available to lateral forces, it is useful to consider the effect of a cross wall onto a load bearing wall. If you have a cross wall sitting against a shear wall, then instead of merely considering the shear wall as being the wall that resists the lateral forces in the in-plane direction. You can account for the advantageous effect of the cross wall as an effective flange.

So, the code allows you to account for an effective flange width as being something that contributes to the lateral load resistance of the masonry wall. And it gives empirical limits to which you can actually utilize this effective flange width. So, the shear wall, the capacity of a shear wall to resist lateral forces can be enhanced if you can account for these flanges.

So, how do you estimate these flanges, it depends on how the walls are interacting with each other. Are you looking at are you looking at a h cross section, or are you looking at an edge cross section where it is a C shaped configuration in plan. So, if it is a h shaped configuration, if you have two walls and if you have a shear wall in between, then the walls at the ends can be understood to act as flanges to the shear wall.

And how much of this flange must be considered is what you can see in this particular figure. So, the flanges at the top and the bottom, there are two edges of the walls. The maximum overhang lengths that you can consider as being an effective flange width for a T shaped water or a I shaped wall is 12 times the thickness of the wall or H/6, where H is the inter storey height itself. So, this is something you can use to estimate resistance of the shear wall to the demand forces itself.

If you have a C shaped wall, of course, you do not have the benefit of the symmetry as you saw in the previous picture. In this case, we reduce the effect to about 6 times the wall thickness or the inter story height H/ 6. So, this is this is the range that you can consider from about 12 times the wall thickness or H/6 is what you actually use as an effective flange width in the lateral load resistance enhancement of a shear wall if it has cross walls at its ends ok.

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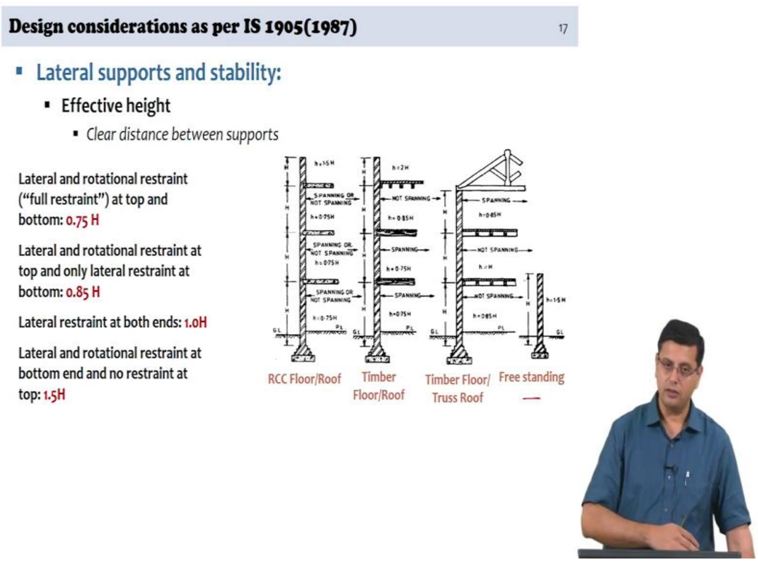

Now, this is an important segment which is, we will be working with the permissible stresses approach, and to be able to account for all the effects of the geometry and the loading eccentricities coming in from the loading. We need to be able to establish what is the slenderness ratio of the wall in the first place and depending on the slenderness ratio of the wall. We have seen in our segment on strength of masonry that slenderness effects and eccentricity of the load have a significant role to play in reducing the load carrying capacity of a wall. You have second order effects coming in and your load carrying capacity, vertical load carrying capacity can be compromised.

So, what is the framework within which this is considered in IS 1905 to be able to establish slenderness ratios, you need to be able to look at what is the effective height of the wall, what is the effective length of the wall, what is the effective thickness of the wall to make an estimate of the slenderness ratio either along the height or the slenderness ratio along the length of the wall. So, we start examining how effective height, effective length and effective thickness are defined by the code, and therefore, that leads us to defining slenderness ratios, and how slenderness ratios and eccentricity ratios then effect are permissible compressive stress, so that is the tract that I am going to be looking at in the next few slides.

So, as far as effective length itself is concerned what the code does is, it gives you the possibility of accounting for the role played by the floor slabs or the roof slabs. So, it is important that depending on the design requirements, whether you going to have a reinforced concrete floor, whether you are going to have a light structural floor, like a timber floor or a steel floor, you would be in a position to understand the kind of restraints that the floor has at the ends of the walls. And based on that, the effective length of the wall is bound to change.

So, the code looks at four different categories, and you could have other conditions as well. The code talks about timber floors, but today timber floors are not that common in our country. You could have steel floors and then you might want to examine the potential end conditions induced by the sort of floor itself. So, the code gives you the possibility of looking at a reinforced concrete roof or a floor slab, timber roof or a floor slab a timber roof, a timber floor, but with a truss roof at the at the final roof or freestanding walls.

And then basically we are looking at an estimate of what the effective height can be in different conditions. Of course, this will be theoretically different from what ideal boundary conditions, we can assume for our buckling calculations. It accounts for multiple effects. We have seen that it is not only the boundary condition, the slenderness and eccentricity, but also the relative stiffnesses between the wall and the floor. So, there are multiple factors that come in. And therefore, the code prescribed affective heights will be theoretically different will be different from the theoretical estimates.

So, the code gives you possibilities of looking at full restraint both lateral and rotational restraint is available at the top and at the bottom. You have lateral restraint lateral translation and rotational restraint at the top and at the bottom you only have translational restraint meaning you have rotational restraint is removed at the bottom. You have lateral restraint both of the top in the bottom, translational restraint alone no rotational restraint.

And a case where there is lateral and rotational restraint translational and rotational restraint at the bottom, and no restraint at the top which would be the case of a free standing wall which could be a parapet wall, parapet wall is a non-structural element, but a parapet, but a regular compound wall for example, would qualify as a freestanding wall if you were to look at the structural design of such a wall. So, it is essential to know what sort of the boundary condition is close to the system that you are choosing and choose the effective height required to make this estimate.

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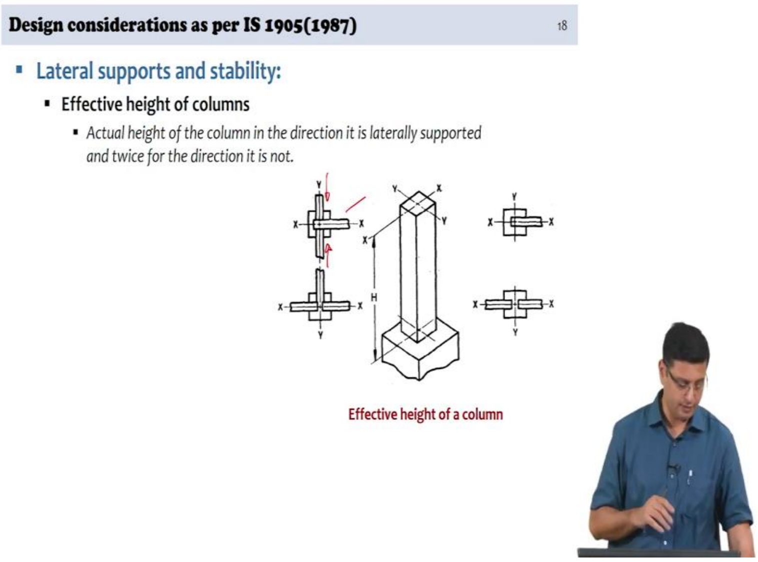

What about columns? So, if you have masonry walls, where the width of the wall is less than 4 times the thickness of the wall. These are typically classified as columns they are not treated as walls any longer, they are actually slender elements. So, if the total width of the masonry wall is less than 4 times the thickness of the wall, you classify them as columns. And given their slenderness we treat them differently as far as the structural design is concerned. And it depends on whether this has lateral supports about one axis or about two axis.

So, typically when you have a masonry column, you might have a masonry column receiving reinforced concrete beam in one direction or two directions or from all four sides. So, it is important to be able to look at the orientation in which the lateral restraints to the ends of the column are coming in.

So, you might have a column where like a brick column, where in the x-direction there is a reinforced concrete beam coming in; and in the y-direction from the from both the sides you have restraints coming in, you can have different configurations from ranging from one single reinforced concrete beam coming and resting on the column to a condition where it is receiving from all the four directions.

Depending on this configuration if there is lateral restraint in a direction, you take the height as 1 h, where h is the height of the column itself. If there is no restraint, then you take the effective height as twice the height of the column itself. So, varies between 1 h and 2 h as far as columns are concerned.

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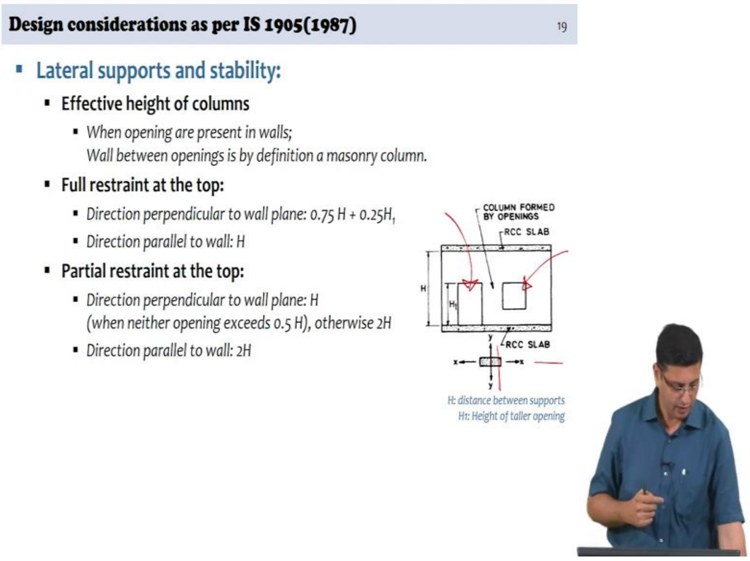

If you have significant openings in a wall, significant opening such as a door opening and a and a window opening, we have talked about the masonry pier right. And we talked about how you will have a shear deformation profile in a masonry wall which is perforated by openings. Now, if this masonry pier is small such that it qualifies as a column rather than a pier, you will have to be careful about using the effective heights not that which pertains to the wall, but which pertains to the column. So, typically between a door opening and a window opening though there are limitations to which you can reduce that masonry pier size.

If it qualifies as a column, you have to be using the right effective, effective factors. So, a wall between openings can eventually work out to be a masonry column itself. So, if you are looking at this sort of configuration, then what sort of boundary condition are you going to give this masonry column along the x-direction and along the y-direction, so that is something that needs to be evaluated. And, depending on whether you have a reinforced concrete slab or a light roof or a light floor which is not capable of providing rotational restraint or translational, translational restraint, you will have to estimate what the effective height is.

So, if there is full restraint at the top, you look at the perpendicular direction to the wall plane and the parallel direction to the wall plane and make an estimate of what the effective height is. So, if there is full restraint, then in the direction perpendicular to the wall plane which is in the y y direction, you are talking of an effective height of 0.75 H plus 0.25 H_1, where this H_1 is referring to the largest opening itself. So, of course, in the other direction it is assumed that the effective height is the height of the wall itself.

If you do not have a reinforced concrete slab which is able to give you rotational restraint at the top, good restraint against translation and rotation at the top, then you are looking at partial restraint. If you have partial restraint, then what happens in the x-direction what happens in the y-direction. So, this is again something that you should be able to carefully classify and estimate what the effective height itself is.

The question is whether we are considering the lintel beams meaning in cases where you have where you have openings in a wall which has openings, and they are provided with lintel beams. See the point as far as lintel beams are concerned, lintel beams are not continuous elements ok. Lintel beams as far as load bearing gravity load bearing masonry design is concerned and not conceived as continuous elements ok. So, these are basically what we refer to as cut lintels they appear just above the opening itself.

So, the contribution from them is not considered to provide, may marginally affect the stiffness of the wall in the in plane direction, but is really not a boundary condition, it is not close to the boundary. Now, even if you have cut lintels, I mean even if you have continuous lintels which is what is prescribed by IS 4326 that when we talk of a lintel beam it is at the level of the lintel, and running continuously in the structure. Even this is a very slender element if you actually look at the cross section we are talking of something that maybe 75 mm or 150 mm with minimum steel. So, these are deformable elements, relatively deformable elements and they are not sitting exactly at the ends of the walls, and so their contribution is not considered systematically in the estimate of stiffness ok.

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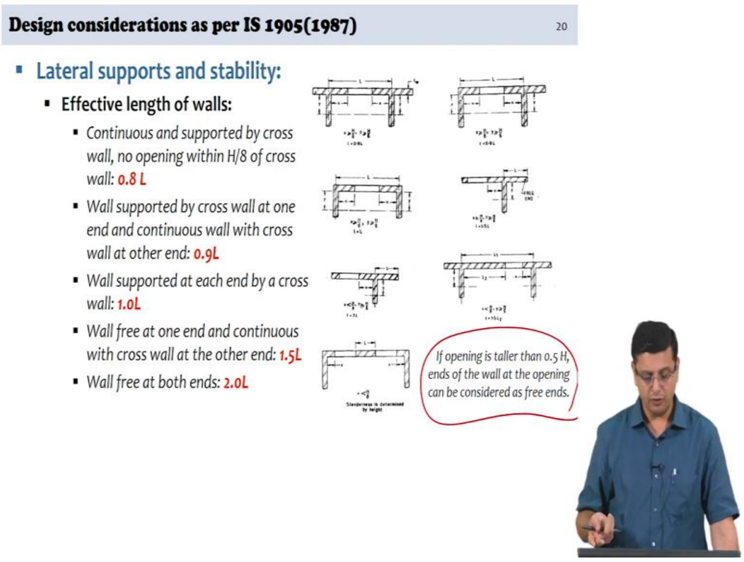

So, if you are able to make an estimate of the effective height, you then need to work on effective lengths, because we have seen how lateral forces induce deformations in the wall in the horizontal bending situation or in the diagonal bending situation. In both these situations, if the boundary conditions in the ends of the wall, the vertical edges, the lateral edges of the wall have affects of cross walls have the affects of the return walls then that will affect the effective length of the wall the deformed profile of the wall itself, so that needs to be accounted for when you are making slenderness calculations based on effective length.

So, different situations, you can have a blank wall with no cross walls, you can have a wall which has cross walls at both its ends; cross wall at one end, big openings you can have different configurations. So, the code gives you different situations under which the effective length of the wall can be calculated. So, here are a few with reference to the cases given in the figure.

If it is continuous, if the wall is continuous and supported by cross walls and no opening within H/8, H again refers to the height of the wall; H/8 of the cross wall, then you take the effective length to be about 80 percent of the actual length. Wall supported by cross walls at one end and continuous with wall with cross walls at the other end it is another case.

If you have a wall supported by cross walls at each end, you have another situation without openings; and then wall free at one end and continuous with a cross wallet at the other end 1.5 one of the largest; I mean you have the effect of restraint on one side, no restraint on the other. And you can have a wall which is completely free at its lateral edges, and you take the effective length as two times two times of l.

Now, word of caution is if the opening is a large opening, then you can actually assume that the opening is creating a free end and that is conservatively assuming that the large opening is actually making the end almost of free end, and the restraint available by the spandrel, whatever be the depth is not something that is going to be significant. So, you estimate the effective length.

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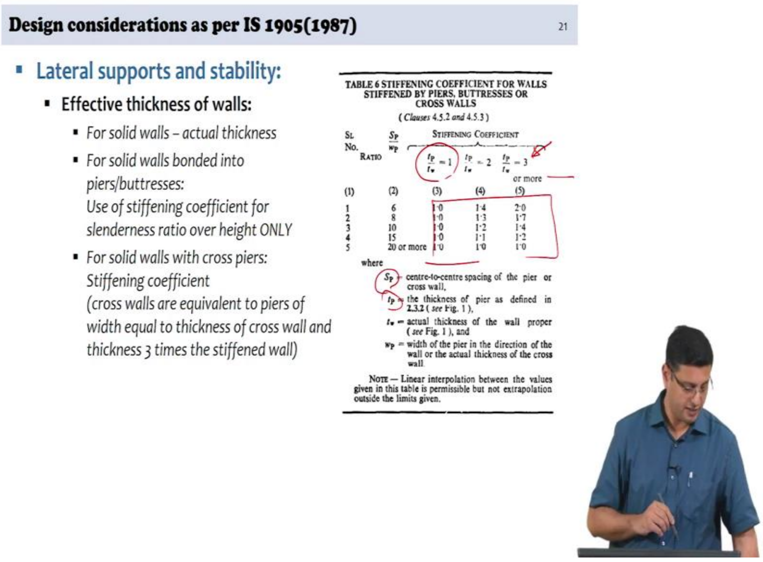

And then you also need to estimate what is the effective thickness of the wall because if you are using and if you are estimating slenderness ratio as effective height by effective thickness, effective length the effective thickness, you need what is an effective thickness of the wall itself. So, if you are using solid walls, it is the actual thickness. But here you will be careful that you cannot use the plaster thickness as being part of the wall itself. So, we exclude the plaster thickness of the wall in making the estimate of the wall thickness itself.

But if you are looking at walls with piers and walls with buttresses, then we need to account for the contribution of the buttressed element or the buttressing element and the pier in improving the effective thickness of the wall. So, when you do that, when you do that, you basically use the, we are making use of the stiffening coefficient along one direction, you cannot be using the stiffening coefficient along both the directions. We are typically talking of elements that are vertically aligned. The pier is vertically aligned the buttress is also vertically aligned; we use this stiffening coefficient in our estimate of the wall thickness.

But you cannot use this as a contributor to the length of the wall. So, the stiffening coefficient is estimated as given in this table, where S_p here is the center to center spacing of the pier or the cross wall how the spacing of the pier itself in the in the wall either you have piers or you have cross walls. And t_p is the thickness of the pier that we have seen earlier. So, depending on this ratio of the spacing to the thickness of the pier, you will estimate what the stiffening coefficient is, and you are allowed to interpolate between the different conditions of ratio of thickness of the pier to the thickness of the wall.

So, depending on the actual geometry of the pier to the wall, you can estimate what the value of the stiffening coefficient is going to be. And of course, if it is a cross wall, the ratio might be much more than 3, but you cap it at 3 and make the estimate. So, it could be a solid wall with the cross piers. And if it is a cross pier as I said though you might have a ratio that is more than t_p / t_w more than 3, you cap it with that t_p / t_w equal to 3, and use the stiffening coefficients for estimating the effective thickness of the wall.

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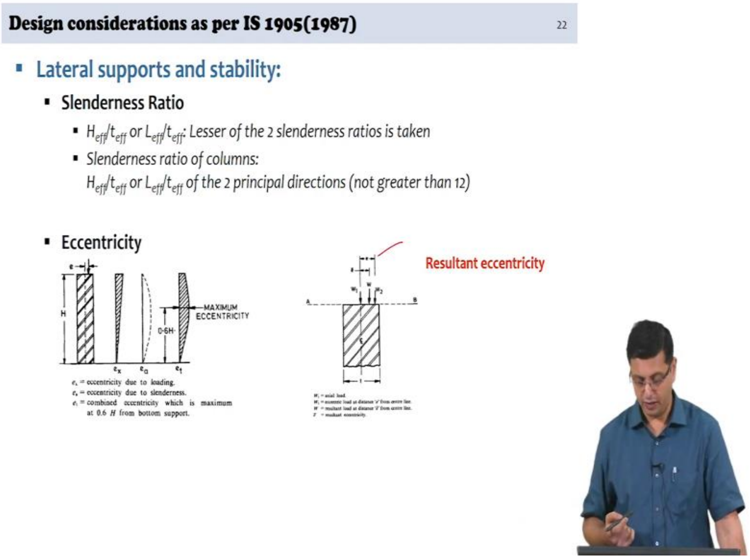

So, having examined the effective height the way it is considered in the design; effective length how it is considered in the design and the effective thickness. Once you make an estimate of these values, you take the two ratios effective height to effective thickness or effective length to effective thickness, and the lesser of the two slenderness ratios are taken. The lesser of the two slenderness ratios are taken and this is airing on the conservative side that the lower of the two values are used in design.

So, you need to make an estimate for every wall of what the two slenderness ratios are, and make use of the lower of the two values. As far as columns are concerned, you will look at those of the principal directions, and these values for a column should not be greater than 12 for a for a wall of the order of 27 is the cap as far as the slenderness ratio of a masonry wall itself is as far as the design is concerned.

Once slenderness ratios can be established, you need to come to eccentricity. And as far as eccentricity is concerned, we make our calculations when we when we check permissible stresses, we typically tend to check the permissible stresses at the base of the wall. We assume that the critical section of the wall is at the base of the wall.

However, if you actually look at different loading situations, and the most common loading situation where you have an eccentricity of the load at the top, and fixity in the wall at the base or the rotational release at the base because of a crack at the base. Then the critical section that actually should occur because of a combination of the deflection under the load and the eccentricity of the load itself need not necessarily happen at the base of the wall . This could actually happen at about sixty percent of the height of the wall that is where you will get the maximum eccentricity. However, we are not going to be systematically checking in different walls where the critical section is expected to occur, we make the calculations at the base of the wall.

So, the design guidelines given consider the fact that simplification is necessary, so that we are on the conservative side, because critical sections may not necessarily happen at the base of the wall. But in such a situation, it should happen at about 60 percent of the height of the wall. As far as eccentricity is concerned you will have eccentricity possibly coming from multiple sources. Because if you take a masonry wall, it may not be a single load, superimposed load getting transferred onto the wall, you can have multiple loads coming on to a certain segment of the wall that you are designing.

So, in that situation, you have to estimate what is referred to as a resultant eccentricity because you cannot be using three different eccentricities to estimate what the slender is, what the effect of the eccentricity of the load is on the same wall. So, you need to make an estimate based on the geometry as far as the location of the load itself is concerned, and the loads themselves a weighted mean of the eccentricities is made to arrive at the resultant eccentricity.

And so this is the eccentricity that you will use to calculate the eccentricity ratio. If it is a single load, you are going to use a single load, but if it is multiple loads with different eccentricities use the resultant eccentricity ok.

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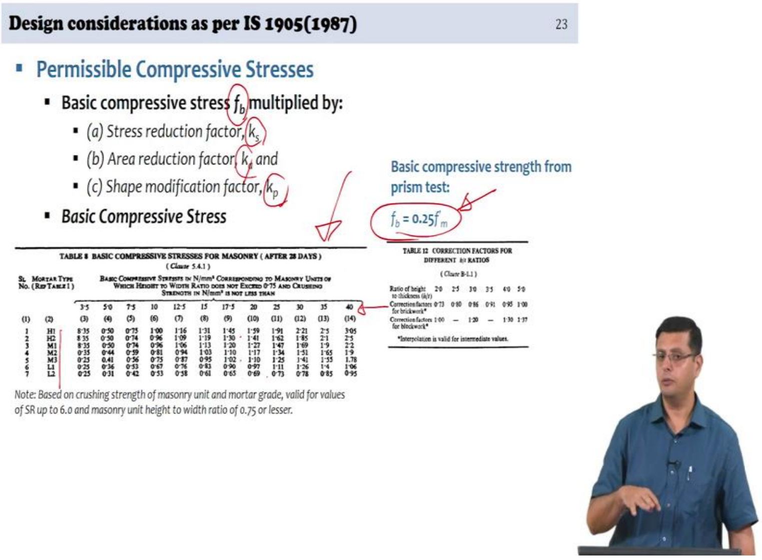

That brings me to application of these aspects that we have estimated into the design check itself. So, you need to be able to estimate the permissible compressive stress and ensure that for the combination of loads that you are looking at, the permissible compressive stress is not exceeded in a given wall cross section. Now, if the demand forces are greater than the permissible compressive stress, then the option that you have is going for a higher strength of brick unit and a higher mortar; and ensure that the permissible stresses are now pushed further up in demand stresses are lower.

So, how do you estimate what is the permissible compressive stress, and then compare that that permissible compressive stress to the demand stresses that you are expecting. So, the way the permissible compressive stress is arrived at is you have to use a basic compressive stress, and I introduced this, this terminology called the basic compressive stress. The basic compressive stress is then factored by the different effects one of which we have talked of extensively, which is reduction in the stress because of slenderness ratio and eccentricity. A second is because of small areas called the area reduction factor, and a third referred to as a shape modification factor; all three we will discuss in a moment.

So, you need the basic compressive stress you need the basic compressive stress to which you are using these reduction factors. So, what is this, basic compressive stress itself- you have two ways in which you can you can arrive at the basic compressive stress. So, the basic compressive stress there is a table which provides what should be used as a basic compressive stress within the working stress approach right. This basic compressive stress as you see in the table is not the strength of masonry, it is not the compressive strength of masonry. It is within the working stress approach that we are using this.

So, table number 8 of IS 1905 gives you the possibility of choosing the basic compressive stress that you want to work with; meaning depending on the demand stresses you might want to go for a higher strength or a lower strength unit or a higher strength or lower strength mortar.

So, you have this matrix which gives you different mortar types in the first column here H 1 to L 2, and you have the unit strengths going all the way from 3.5 to about 40. So, what you see as these numbers are the basic compressive stress prescribed for a combination of a certain strength of unit and a certain strength of mortar. So, you could either use this value that the code prescribes or if you were to carry out a compressive strength test in the laboratory using a prism test, then we take 25 percent of the crushing strength of the masonry.

So, if you are actually carrying out a test in the laboratory, then your basic compressive stress is one-quarter of the compressive strength of the masonry if you have statistically sufficient number of samples. f prime in is the crushing strength 28 day and also the characteristic compressive strength. So, if you have test results, use this. If you do not have test results, the code gives you an overall basis. It is quite clear to you that the minimum factor of safety that we are talking about in the working stress approach the minimum factor of safety commences at 4.

So, if the compressive strength is f m, it is f m by 4 that we are actually keeping as the basic compressive stress. We are then going to multiply the basic compressive stress with further factors assuming that the three factors that I am going to be looking at k_s, k_a and k_p are all 1, then the minimum factor of safety ensured in the permissible stresses approach is 4. It can go all the way up to numbers that that are going to be quite shocking.

Yes, because why 25 we have seen that at about 33 percent of a stress strain curve of the masonry. You start getting deviations from the linear elastic behavior which means some initial cracking has started, and we were limiting ourselves to the estimate of the modulus of elasticity in the zone of five percent to 33 percent of the compressive strength of masonry.

So, we are basically saying that that initial one-third is more or less linear is linear, and it matches with the fact that the basic compressive stress is taken as the stress within that elastic region. So, it it makes sense because you are talking of a permissible stresses approach you do not want the stress level to go into the non-linear range and that is the and that is the rationale I would think.

So, this table that I have reported here is nothing but the table that is available in appendix B of IS 1905 which gives you correction factors depending on the height to thickness ratios that you will use for the prism itself. And we have seen that for block work and brickwork these values can vary ok.

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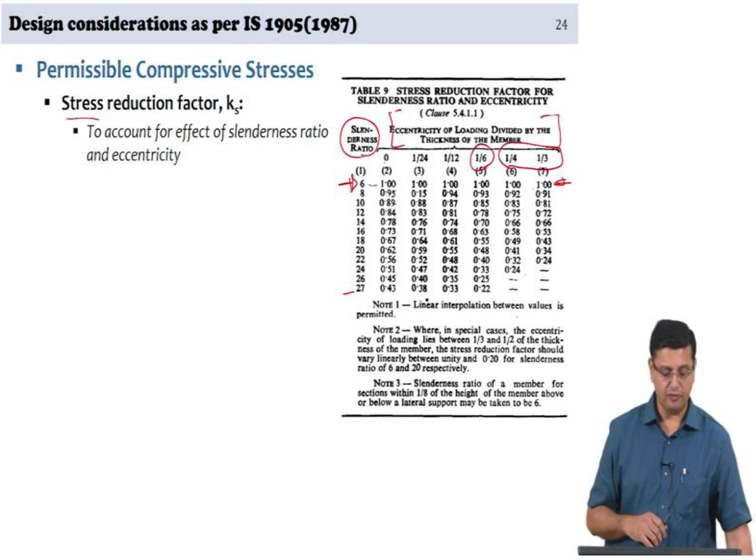

TABLE 9 STRESS REDUCTION FACTOR FOR SLENDERNESS RATIO AND ECCENTRICITY

(*Clause* 5.4.1.1)

Slenderness Ratio	Eccentricity of Loading Divided by the Thickness of the Member					
	0	1/24	1/12	1/6	1/4	1/3
(1)	(2)	(3)	(4)	(5)	(6)	(7)
6	1·00	1·00	1·00	1·00	1·00	1·00
8	0·95	0·15	0·94	0·93	0·92	0·91
10	0·89	0·88	0·87	0·85	0·83	0·81
12	0·84	0·83	0·81	0·78	0·75	0·72
14	0·78	0·76	0·74	0·70	0·66	0·66
16	0·73	0·71	0·68	0·63	0·58	0·53
18	0·67	0·64	0·61	0·55	0·49	0·43
20	0·62	0·59	0·55	0·48	0·41	0·34
22	0·56	0·52	0·48	0·40	0·32	0·24
24	0·51	0·47	0·42	0·33	0·24	—
26	0·45	0·40	0·35	0·25	—	—
27	0·43	0·38	0·33	0·22	—	—

NOTE 1 — Linear interpolation between values is permitted.

NOTE 2 — Where, in special cases, the eccentricity of loading lies between 1/3 and 1/2 of the thickness of the member, the stress reduction factor should vary linearly between unity and 0·20 for slenderness ratio of 6 and 20 respectively.

NOTE 3 — Slenderness ratio of a member for sections within 1/8 of the height of the member above or below a lateral support may be taken to be 6.

So, the first factor is a strength reduction factor, the stress reduction factor. And the stress reduction factor is nothing but the second order effects that we were looking at we looked at an entire procedure to account for these second order effects, and they are in terms of the slenderness ratio and the eccentricity ratio; we have seen this table earlier.

So, once you have the slenderness ratio, the lower of the two, H effective by T effective or $L_{effective} / t_{effective}$. And the eccentricity ratio the eccentricity ratio again the resultant eccentricity if you have multiple loads, you have eccentricity ratios going from 0 one-twenty fourth, one-twelfth, one-sixth is that for the linear elastic; the key situation where you have no tension, but full compression. And then one-quarter and one-third where you start getting tension in the cross section.

And the slenderness ratios go all the way from 6 to 27. So, you can interpolate between the values that are reported. You can see that if there if the slenderness ratio is 6, you have no and for whatever be the eccentricity ratio of the load, you have no reduction in the stress as you go forward we have seen the graphs that that you can make; the curves that you can make with this set of expressions. So, the first factor is a stress reduction factor less than 1, 1 or less than 1.

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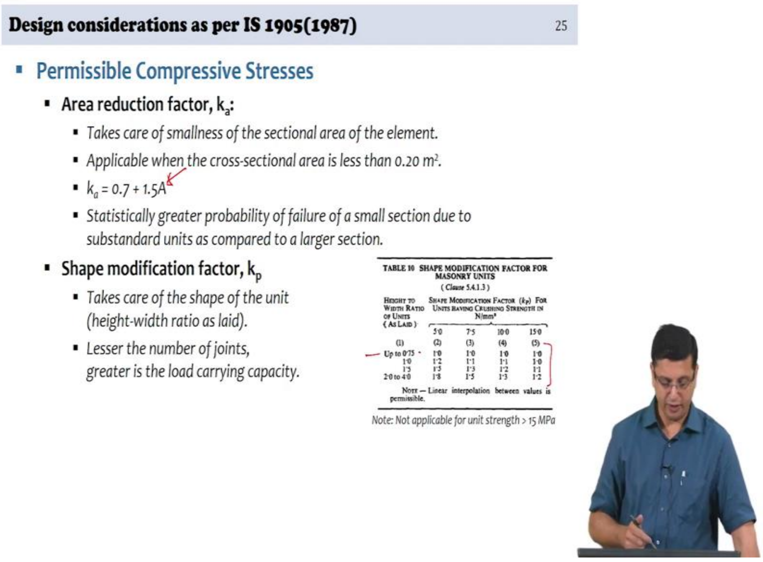

Height to Width Ratio of Units (As Laid)	Shape Modification Factor (k_p) for Units Having Crushing Strength in N/mm²			
	5·0	7·5	10·0	15·0
(1)	(2)	(3)	(4)	(5)
Up to 0·75	1·0	1·0	1·0	1·0
1·0	1·2	1·1	1·1	1·0
1·5	1·5	1·3	1·2	1·1
2·0 to 4·0	1·8	1·5	1·3	1·2

The second is the area reduction factor. This area reduction factor basically takes into account the smallness of an area, resisting area of masonry, which means if you are talking about a full wall, you know if you are talking about a small column; the small column does not have a resisting area that is comparable to a wall itself. Now, if there is a defect in the material, if there is a defective brick, the effect of the defective brick in a small cross section like a column is going to be more severe than the problem of that poor brick, poor quality brick in an entire wall, in an entire wall cross section.

So, it is a way of accounting for the variability in masonry as having a role to play; it is critical at sometimes when the area of cross section is small, not critical when the area of cross section is significant. So, it really takes into account the smallness of the area of cross section and it is applicable only when the cross section is less than 0.2 m^2. So, if you are working with columns you will have that sort of a cross section and the area reduction factor is estimated empirically as 0.7 + 1.5 A, where A is the area of cross section itself.

So, as I said it goes by the underlying concept that in a smaller cross section, there is a greater probability of statistically a greater probability of failure because of a poorer quality of stone or brick than in a larger cross section, so that is the area reduction factor. We also have a shape modification factor. And this shape modification factor needs to be used only if the way in which you are laying the bricks changes right.

We typically place the bricks flat twice. But for some reason particularly when you want to optimize wall cross sections or you have exposed brickwork, when you have when you do not plaster brickwork, you have exposed brickwork you want a certain pattern to be visible on the exterior, then you might place the brick on edge or you might even place the brick on end rather than placing it flat wise.

So, if there is a change, then the immediate implication is the number of bed joints will change with respect to flat wise placement of bricks. When you place bricks flat wise, you will get the maximum number of bed joints. But if you were to place it on end, the number of bed joints will be lower than in the flat wise case. And if you were to place it on edge, it will be the minimum the least number of bed joints that you will get. Now, if you remember what we talked about in terms of the effect of bed joint on the masonry compressive strength as you reduce the bed joints, you get better strength in masonry.

So, this is where the shape modification factor comes from. And basically, you have the shape modification factor for different strengths of units. And depending on the height to width ratio of the unit the way you are laying it in the wall itself. So, it can go from the ratio itself can go from 0.75 that is how we would actually lay the brick 0.75 in height to with 175 mm to 100 mm, you will get a ratio of 0.75, you do not use a shape modification factor at all.

Shape modification factor is 1. But if you change that you, you see that the shape modification factor increases. By increased you see that these values are greater than 1, simply because the number of bed joints is reducing now and therefore it has a positive effect on the strength of masonry right there. The other two factors are less than 1, this is the only factor that is greater than 1 and that is the rationale behind it. It takes care of the shape of the unit and the height to width ratio as laid, basically echoing the fact that less of the number of joins greater is the load carrying capacity. So, now, we have an understanding of how these three factors are multiplied with the basic compressive stress to establish the permissible compressive stress.

So, we will continue in the next class.