PRESTRESSED CONCRETE STRUCTURES

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Module - 4: Design of Members

Lecture -18: Design of Members for Flexure (Type 1 Members)

Welcome back to prestressed concrete structures. This is the second lecture of Module 4 on design of members.

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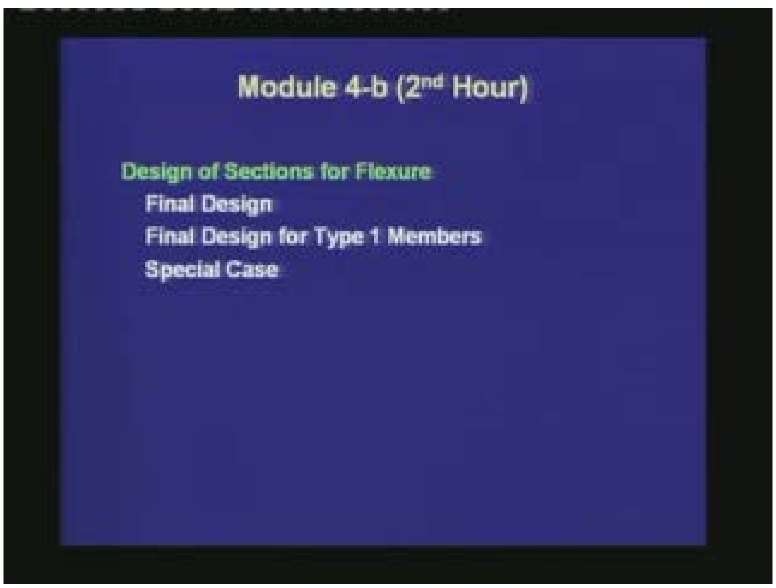

In this lecture, we shall learn about the design of sections for flexure. We shall study the steps of final design. Specifically, we shall learn the final design for Type 1 members. We shall also see a special case of Type 1 members.

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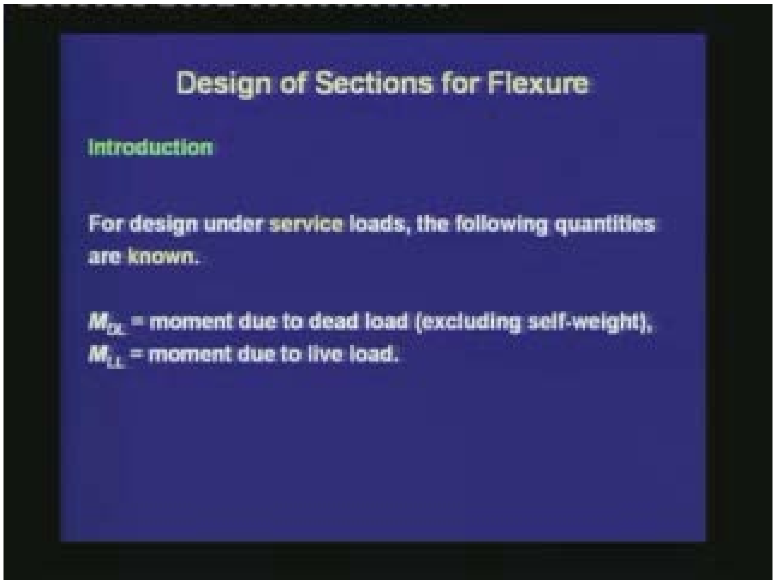

For design under service loads, the following quantities are known: M_{DL}, the moment due to dead load which excludes the self-weight, and M_{LL}, which is the moment due to live load. These two quantities are available from the analysis of the structure.

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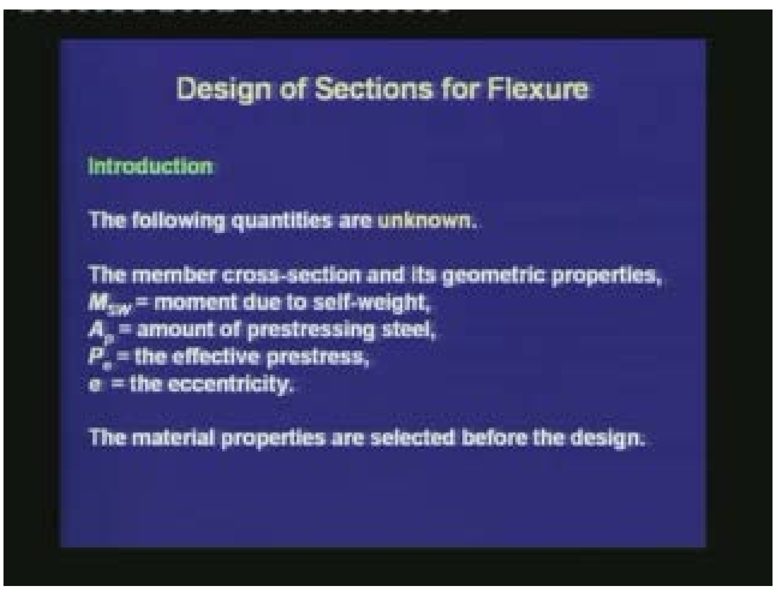

Next, the unknown quantities are: first, the type of the section and the properties of the section; second, the moment due to self-weight; third, the area of the prestressing steel which is denoted as A_p; fourth, the effective prestress P_e; and finally, the eccentricity e. The material properties are selected before the design.

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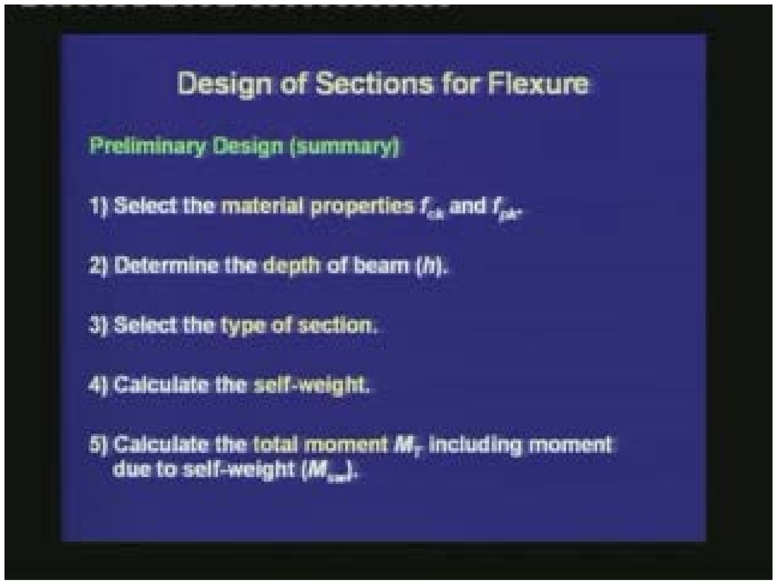

As we had mentioned last time, the design procedure is divided into two stages for convenience. The first stage is called preliminary design, where we estimate some quantities, and then we do the final design. In the preliminary design, we first select the material properties: f_{ck} the characteristic compressive strength of concrete, and f_{pk} the characteristic tensile strength of the prestressing steel. These are selected based on the materials to be supplied.

Next, we determine the depth of the member. This is usually governed by the architectural requirement. If there is no architectural specification, then we can determine the depth by some empirical equations, which are based on the moment that we have from the dead load and live load. Next, we select the type of section. The section can be either rectangular or flanged section. The type of section that is used depends on the application. We shall discuss about this in a future lecture.

Once we have selected the type of section, we can estimate its self-weight. We calculate the total moment (M_T), which includes the moment due to dead load, live load and the moment due to self-weight.

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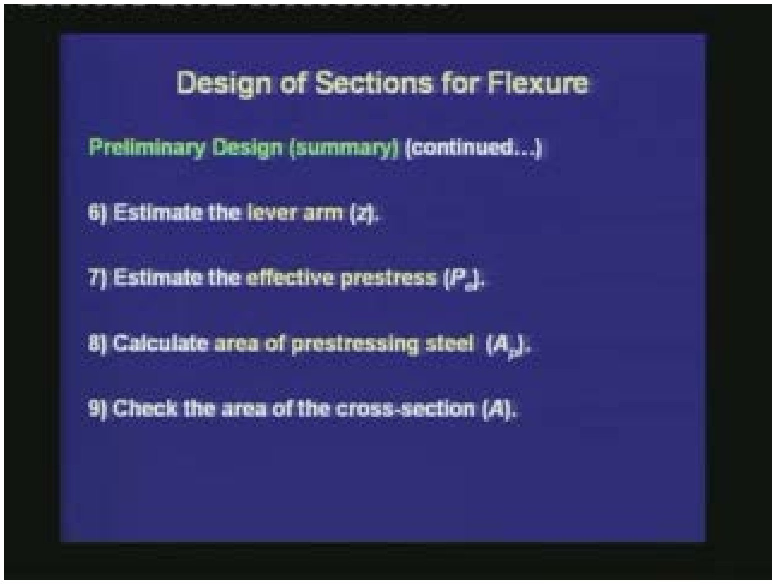

After this, we estimate the lever arm (z). The lever arm is expressed as a fraction of the total depth. Then, we estimate the effective prestress (P_e). The effective prestress is equal to the moment divided by the lever arm. From the effective prestress, we calculate the area of the prestressing steel (A_p) which is given by the prestress divided by 70% of the characteristic strength. Finally, we check the area of the cross-section (A), to make sure that it is not too way off from the allowable stress which is expected near the CGC. Thus, the objective of preliminary design is to have estimates of P_e, A_p and A. But, we do not know one important variable yet, that is the eccentricity of the CGS at the critical section.

Next, we move on to the final design where the objective is to calculate the eccentricity and to check the stresses in the concrete.

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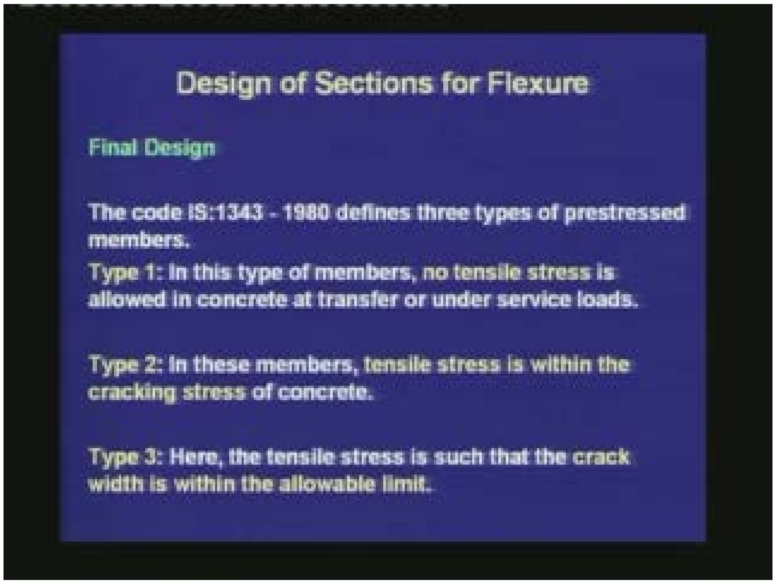

Before we move on to the steps, let us know the type of sections that the code allows us to design from. The code IS: 1343-1980 defines three types of prestressed members.

Type 1 - In this type of members, no tensile stress is allowed in concrete at transfer or under service loads. That means in Type 1, the full section is under compression throughout the service loads.

Type 2 - In this type of members, tensile stress is limited within the cracking stress of concrete. That is, we do allow some tensile stress in the concrete; but we make sure that it is within the cracking stress. Hence, the section remains uncracked under service loads.

Type 3 - Here cracking is allowed, but the tensile stress is such that the crack width is within the allowable limit. That means, although we allow cracking, we limit the crack width to a certain value, so that it does not disrupt the serviceability conditions of the prestressed member.

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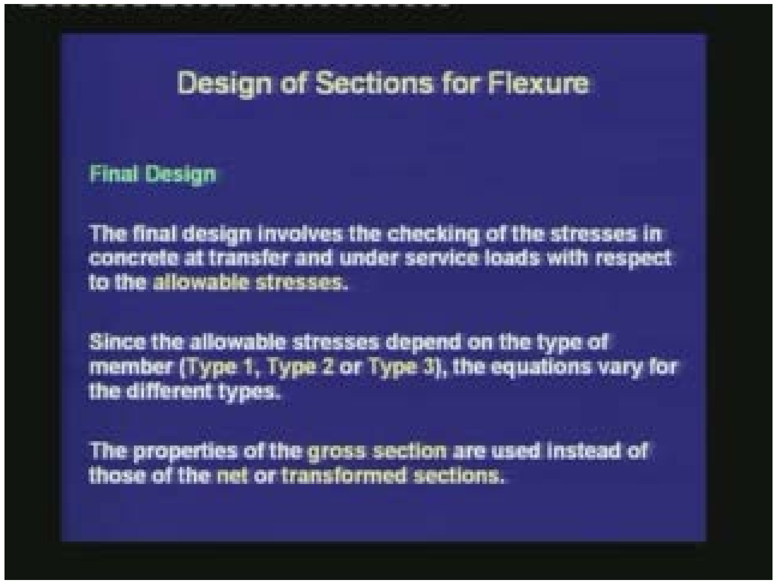

The final design involves the checking of the stresses in the concrete at transfer and service loads, with respect to the allowable stresses. This is a very important step of the final design. Since, the allowable stresses depend on the type of member - Type 1, Type 2 or Type 3, the equations vary for the different types.

When we learn the equations, we shall use the properties of the gross section instead of the net or transformed sections for simplification of calculations. We had seen earlier in the analysis of members that if we use more rigorous calculations based on the net or the transformed sections, we do not gain much accuracy than the calculations based on the gross section. Hence, for simplicity we are using the properties of the gross section. But in a special situation, if we feel that the properties of the gross section may not lead to accurate solutions, then we may use a net or the transformed section properties.

In the final design, the steps are explained for Type 1 and Type 2 members. The steps for Type 3 members are similar to Type 2 members, the only difference being the value of the allowable tensile stress in concrete. In this lecture, we shall first learn the design of Type 1 members. In the following lecture, we shall learn the design of Type 2 members. We shall not study the design of Type 3 members, specifically, because it is similar to the design of Type 2 members.

We move on to the final design of Type 1 members.

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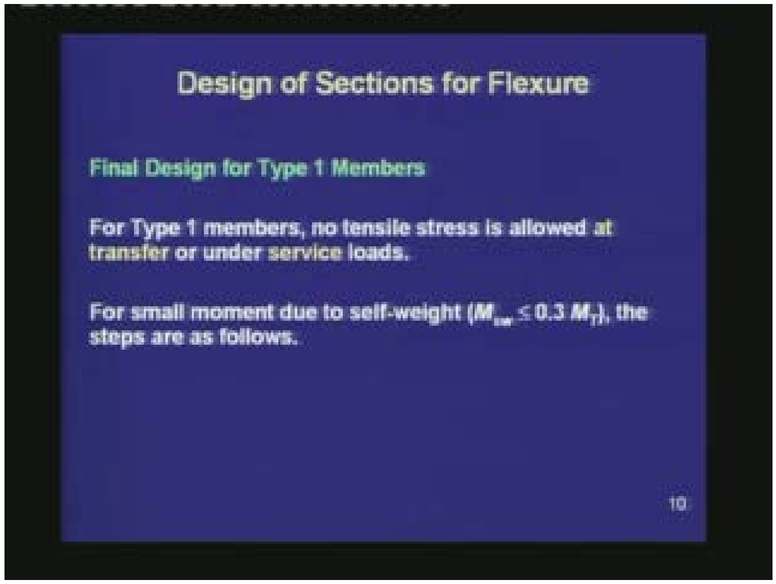

To recollect, for Type 1 members no tensile stress is allowed at transfer or under service loads. For small moment due to self-weight, say M_{sw} less than 30% of the total moment, the steps are as follows.

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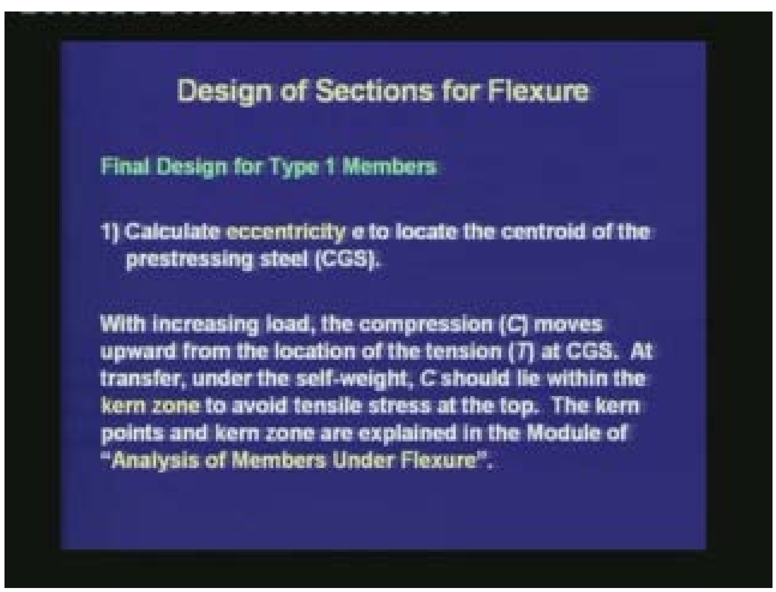

First, calculate the eccentricity (e) to locate the centroid of the prestressing steel, which is denoted as CGS. We had learnt earlier that with increasing load, the compressive force C moves upward from the location of the tension T at CGS. At transfer, under the self-weight, C should lie within the kern zone to avoid tensile stress at the top. The kern points and the kern zones were explained in the module of analysis of members under flexure.

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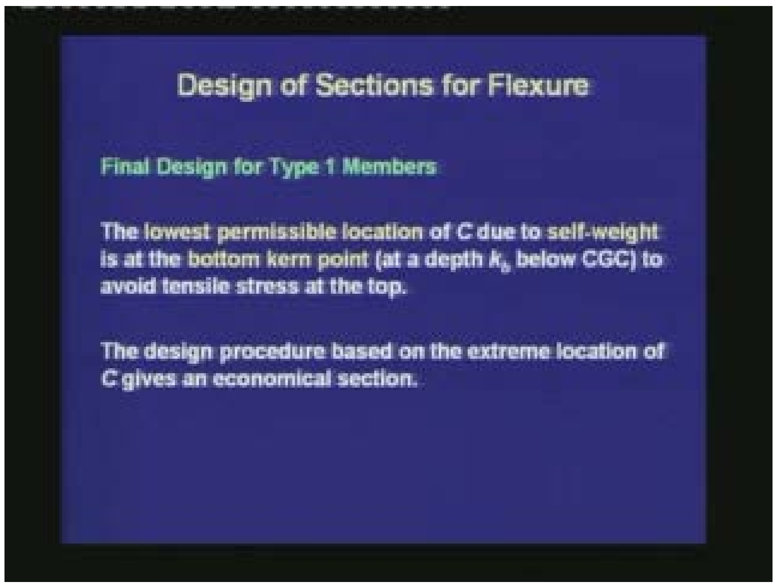

Thus, the lowest permissible location of C due to self-weight is at the bottom kern point, which is at a depth k_b below the CGC, to avoid tensile stress at the top. The design procedure based on the extreme location of C gives an economical section.

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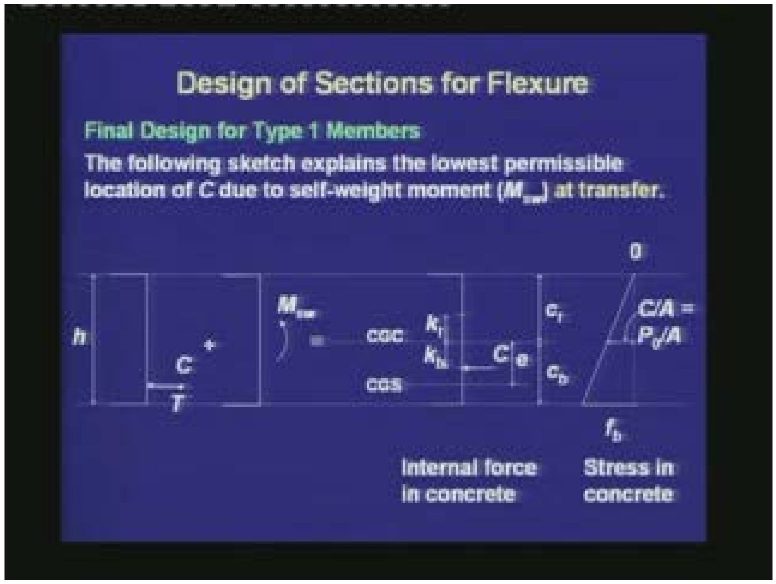

Let us understand the calculation by the help of a diagram. At the time of prestressing, the C coincides with T. When the transfer is over and the member has deflected upwards, then the self-weight has started to act. When the self-weight acts, the C shifts upward from T. For a Type 1 member, this upward shift should be such that the C enters the kern zone to avoid any tensile stress at the top. Thus, the lowest position of C is at the bottom kern point which is at a depth k_b from CGC. The procedure based on the lowest CGS, and hence the lowest location of C at transfer gives an economical section. When C is at the bottom kern point, then the stress in the concrete is triangular with zero at the top and f_b at the bottom.

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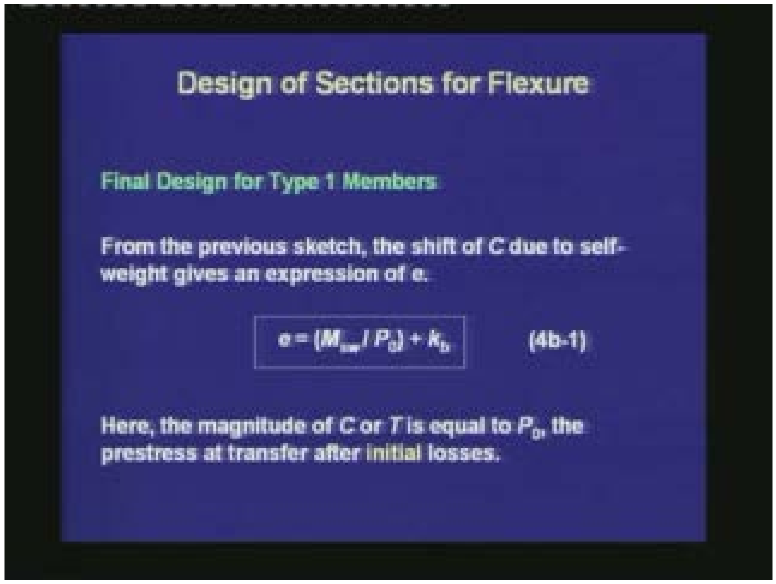

From the previous sketch, the shift of C due to self-weight gives an expression of e.

$e = (M_{sw}/P_0) + k_b$

Thus, we have got an expression of e from the quantities which are known to us. We have already calculated M_{sw} for the selected section. We need to estimate P_0 from the value of A_p; and k_b is known for the trial section. Here, the magnitude of C or T is equal to P_0, the prestress at transfer after initial losses.

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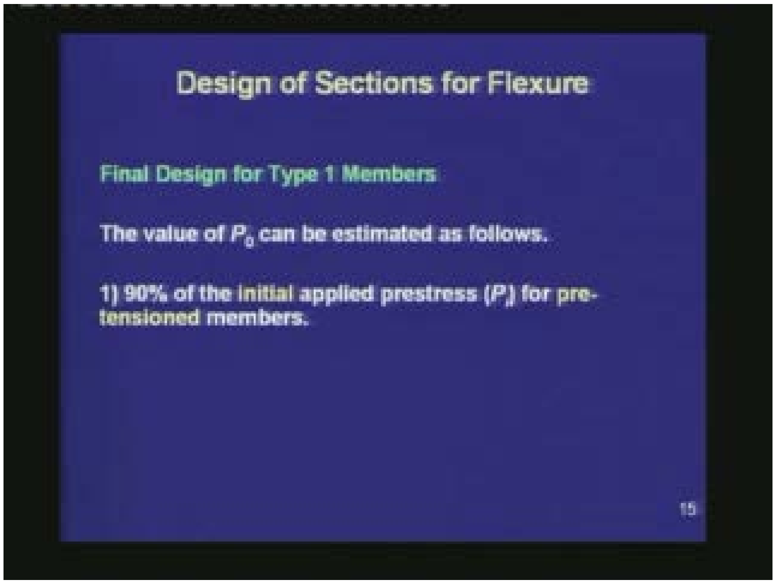

The value of P_0 can be estimated as follows.

1) For a pre-tensioned member, we know that there is a loss due to elastic shortening from the value of force that we measure in the jacks. To estimate the prestressing force after the losses due to elastic shortening, we can consider that P_0 is equal to 90% of the initial prestressing force which has been applied by the jacks. Thus, for pre-tensioned members $P_0 = 0.9P_i$, where P_i is the initial prestressing force measured in the jacks.

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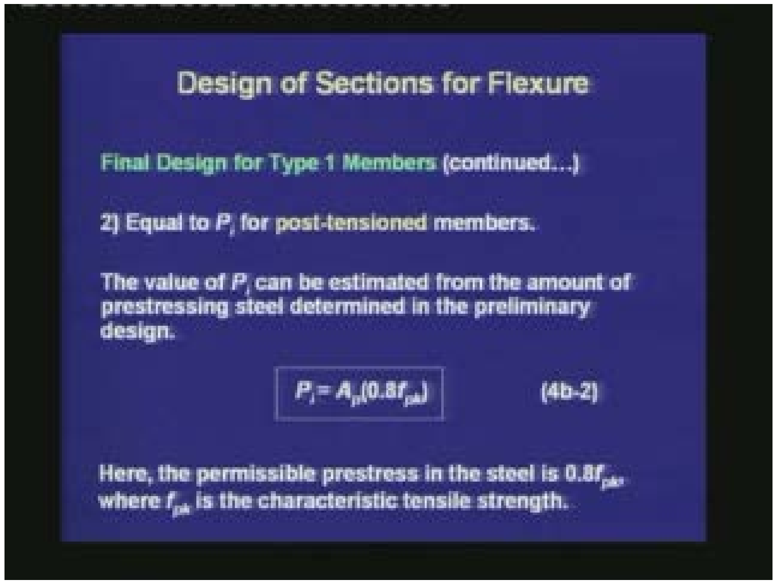

2) For a post-tensioned member, we can neglect any loss due to elastic shortening, if there is a simultaneous stretching of all the tendons. Even if the tendons are stressed sequentially, the elastic shortening can be neglected at this point. Hence, we can equate the value of P_0 to P_i.

The value of P_i can be estimated from the amount of prestressing steel determined in the preliminary design. $P_i = A_p(0.8\ f_{pk})$. Here, the permissible prestress in the steel is $0.8f_{pk}$, where f_{pk} is the characteristic tensile strength. We had known earlier that the code does not allow to stretch the tendons beyond 80% of their characteristic strength because, there will be more relaxation loss and also because of the safety reasons.

Once we know P_0, we are able to estimate e.

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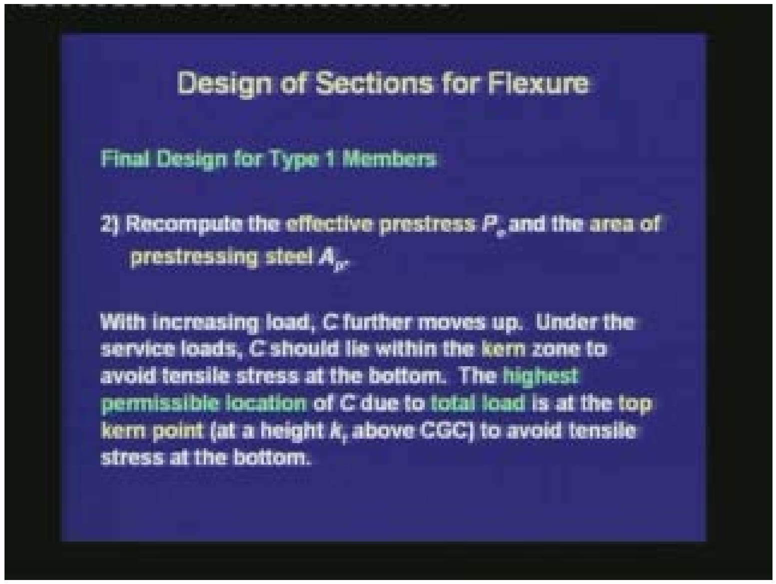

Our next step is to recompute the effective prestress P_e and the area of prestressing steel A_p. With increasing load, C further moves up. Under the service loads, C should lie within the kern zone to avoid tensile stress at the bottom. The highest permissible location of C due to the total load is at the top kern point, which is at a height k_t above CGC to avoid tensile stress at the bottom.

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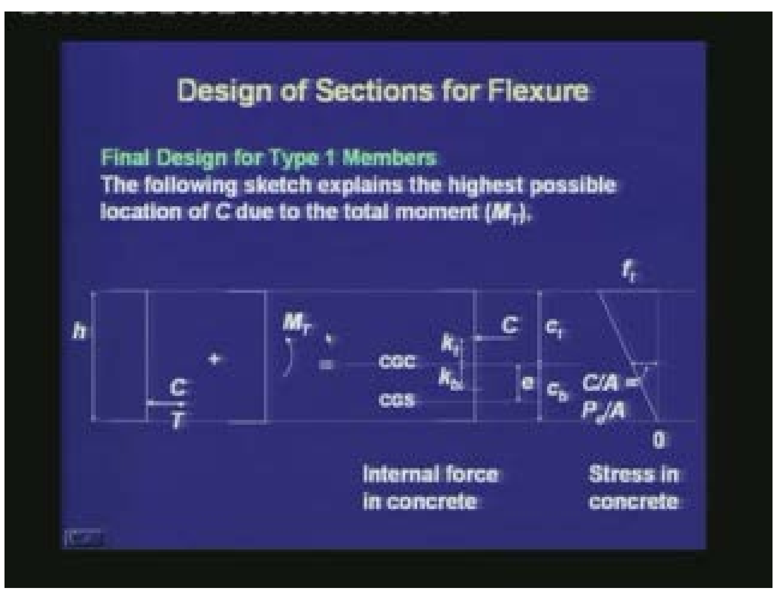

Let us understand this with the help of the sketch. When the total moment is applied during service conditions, C has shifted further upwards. For a Type 1 member, C has to lie within the kern zone to avoid any tensile stress. The highest location allowed for C is the top kern point. In that situation, the stress block in concrete is triangular with zero stress at the bottom.

Thus, for both at transfer and for service, we have satisfied the allowable tensile stress in the concrete, which is zero for Type 1 member.

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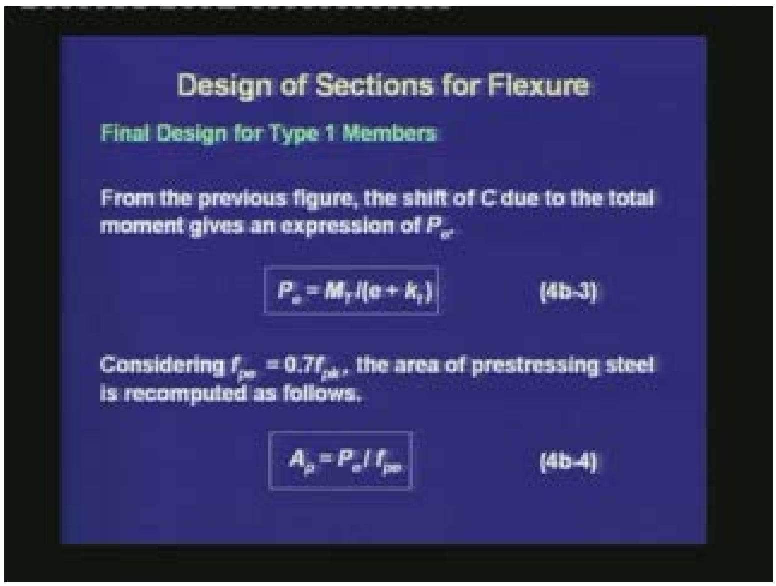

From the previous figure, the shift of C due to the total moment gives an expression of P_e. At this stage, we are recomputing P_e based on the maximum shift that we can have for the compressive force. This also gives an economical section, because if we have the maximum shift, then we have the lowest value of the prestressing force. The lever arm of the shift of C is equal to $e + k_t$.

$P_e = M_t/(e + k_t)$

Considering f_{pe} to be equal to the 70% of the characteristic strength of the prestressing steel, the area of prestressing steel is recomputed as $A_p = P_e/f_{pe}$.

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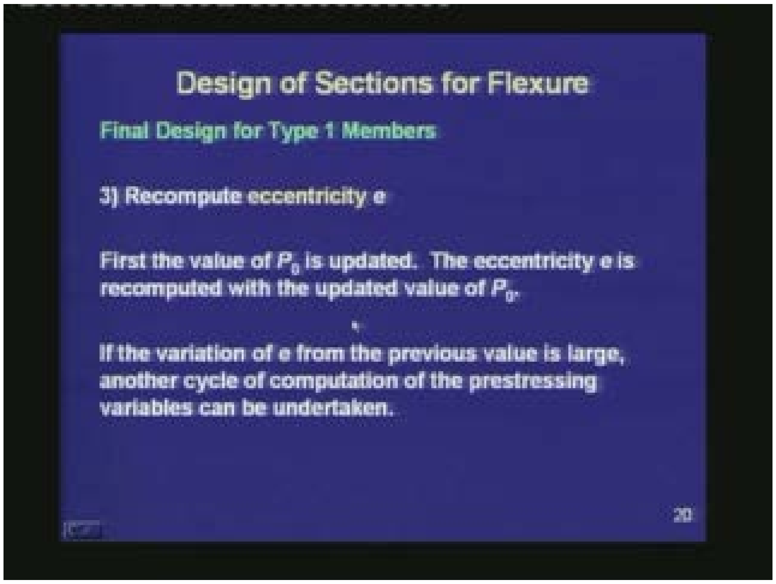

Once we have got the value of P_e, we are again recomputing eccentricity e. First, the value of P_0 is updated. Then, the eccentricity is recomputed with the updated value of P_0. If the variation of e from the previous value is large, another cycle of computation of the prestressing variables can be undertaken. We converge to a set of results which gives the maximum possible e, the lowest possible P_e and simultaneously satisfying the allowable tensile stress in the concrete (which is zero for Type 1 member). Next we check the compressive stresses in the concrete.

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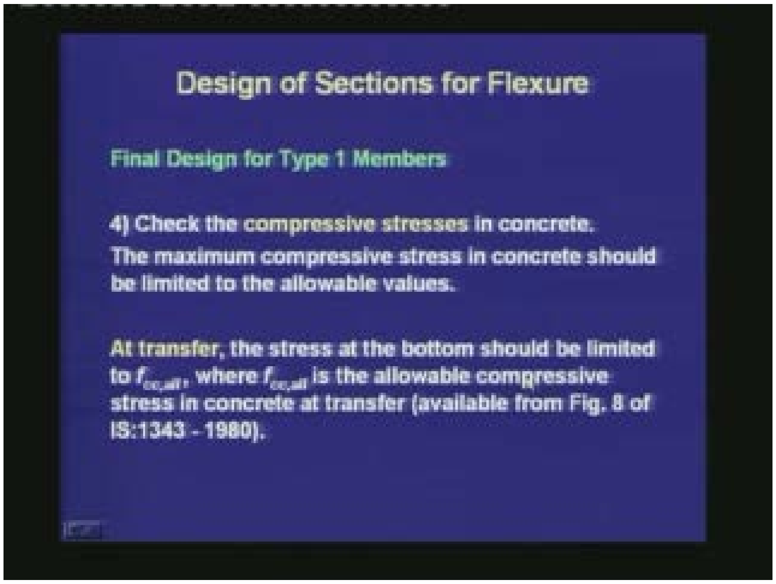

The fourth step is to check the compressive stresses in concrete. The maximum stress in concrete should be limited to the allowable values. At transfer, the stress at the bottom should be limited to $f_{cc,all}$, where $f_{cc,all}$ is the allowable compressive stress at transfer and this is available from Figure 8 of IS: 1343-1980.

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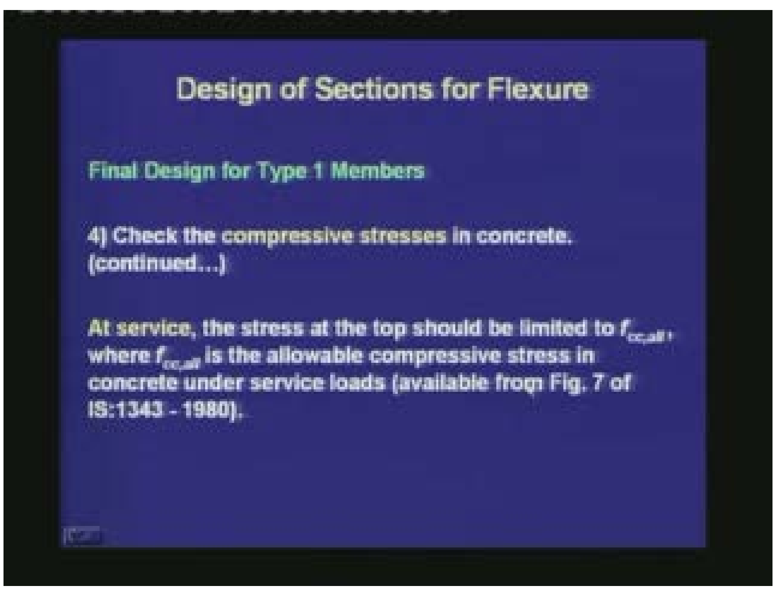

At service the stress at the top should be limited to $f_{cc,all}$, where $f_{cc,all}$ is the allowable compressive stress in concrete under service loads, which is available from Figure 7 of IS: 1343-1980.

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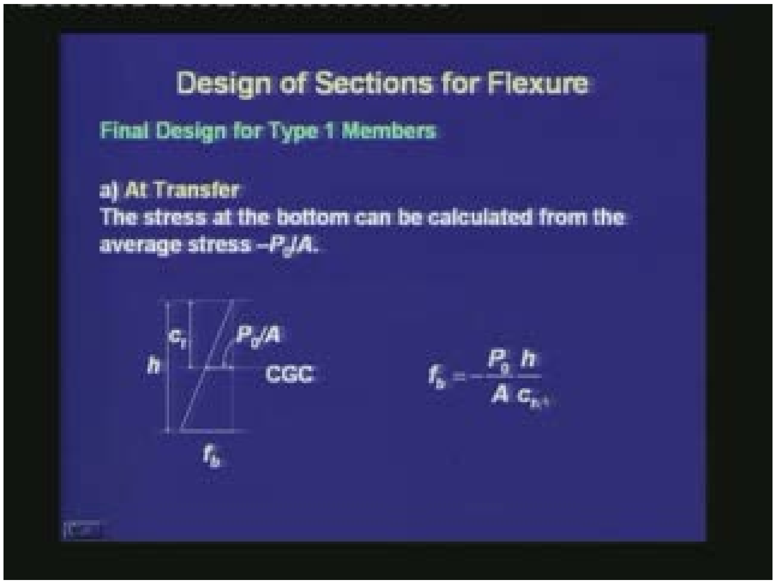

At transfer, for a triangular stress block, the stress at the bottom can be calculated from the average stress – P_0/A at the CGC. Here, we are calculating the stress in a simplified way. Instead of writing the stress in terms of individual components of a uniform stress and a varying stress, we are writing directly from the proportionality of the triangle. The stress at the bottom is $f_b = -P_0/A \times h/C_t$.

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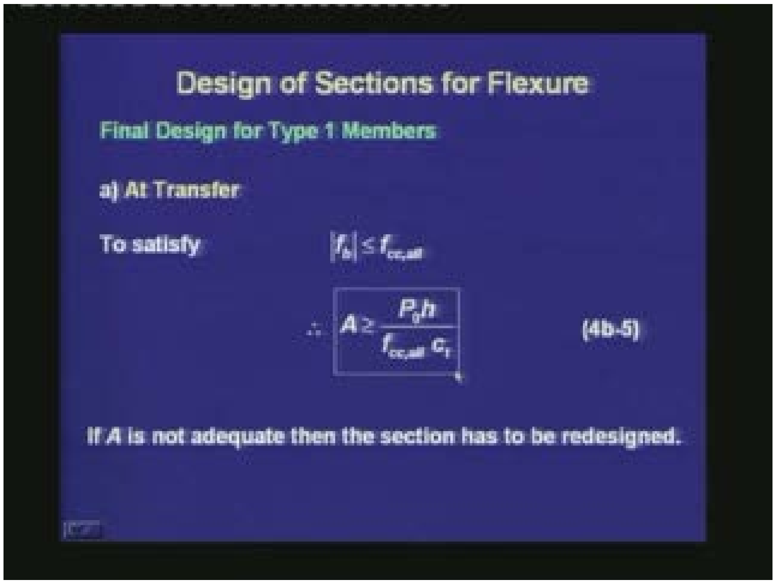

We have to satisfy that the numerical value of f_b should be less than or equal to $f_{cc,all}$. To satisfy this, the area of the section should be greater than $P_0h/f_{cc,all}\ c_t$. Thus, here we have a constraint that the area of the selected section has to be greater than a certain value. If the area that we have selected for the trial section is not satisfying this, then we have to redesign the section. Thus, we come to a point, where the section that we have selected in the preliminary design may not be adequate. We may have to increase the section, if the area is not satisfying the minimum requirement.

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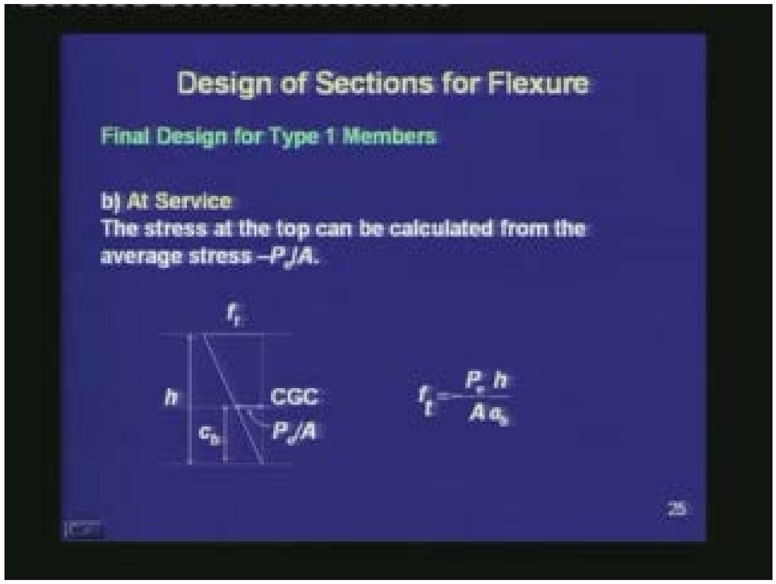

To check the stress at service conditions, we are calculating the maximum compressive stress at the top. From this figure, we find that the maximum stress at the top is proportional to the stress at the CGC. From the similarity of the triangles, we find $f_t = -P_e/A \times h/c_b$. Note that, here, we are considering the effective prestress at service conditions.

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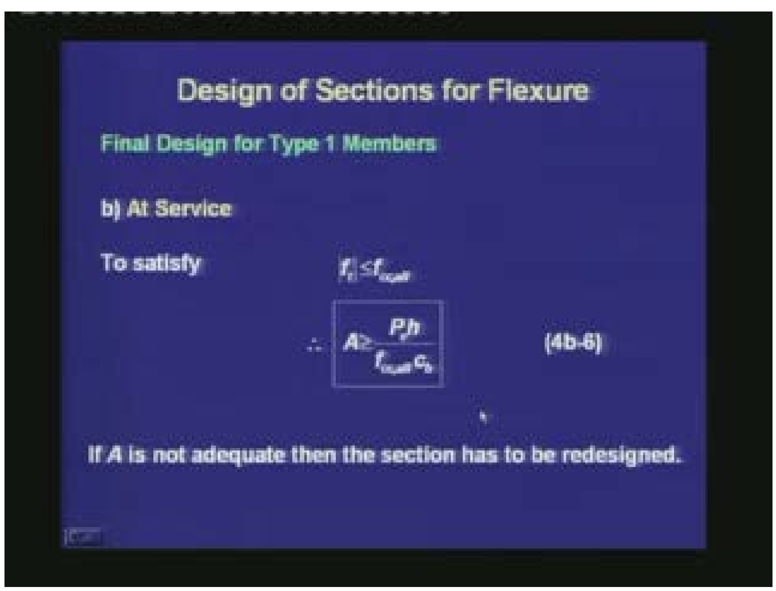

We have to satisfy that the numerical value of the stress at the top should be less than the allowable compressive stress for the service conditions. This brings in another condition for the area of the section, A should be greater than $P_e h/f_{cc,all} c_b$. Again, just like last time, if the area of the section is not adequate for the trial section that we have selected from preliminary design, then we need to update the section by increasing its area.

Thus, first we satisfied the allowable tensile stress and determined the prestressing variables. Then we checked the compressive stresses. If the compressive stresses are not within the allowable values, then we have to redesign the section.

There is a special case for the Type 1 members.

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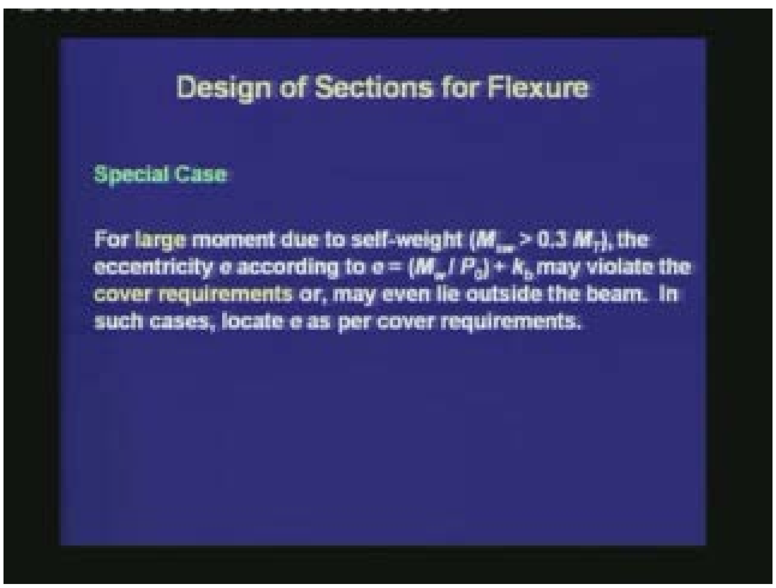

The calculations that we have shown are for a member where the self-weight is smaller compared to the total moment. For large moment due to self-weight, that means, if the self-weight itself is greater than 30% of the total moment, then the eccentricity e which has been calculated from the expression $(M_w/P_0) + k_b$ may violate the cover requirements or may even lie outside the beam. In such cases, locate e as per the cover requirements.

Hence, this is another constraint for the value of e, that the CGS should be in such a distance that we satisfy the cover requirement for the bottom most tendon.

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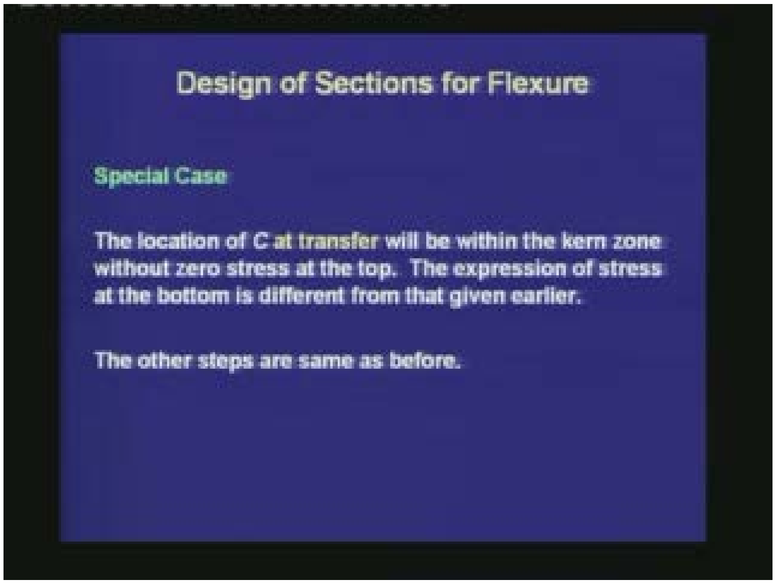

When e is determined based on the cover requirement, then the location of C at transfer will be within the kern zone, without zero stress at the top. The expression of stress at the bottom is different from that given earlier. The other steps are same as before.

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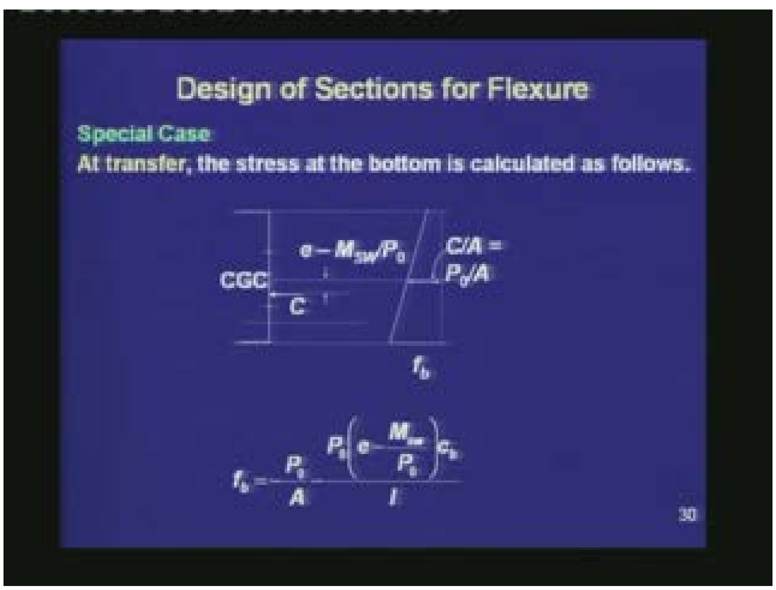

At transfer, the stress at the bottom is calculated from this stress diagram. The location of C from the CGC is given by e minus the shift due to the self-weight moment, which is equal to e – M_{sw}/P_0. Since we do not have a triangular stress block anymore, we cannot use the proportionality equations as before. We are calculating the stress at the bottom, based on the individual components, the uniform and varying components. The stress is equal to $– P_0/A – P_0(e – M_{sw}/P_0)c_b/I$.

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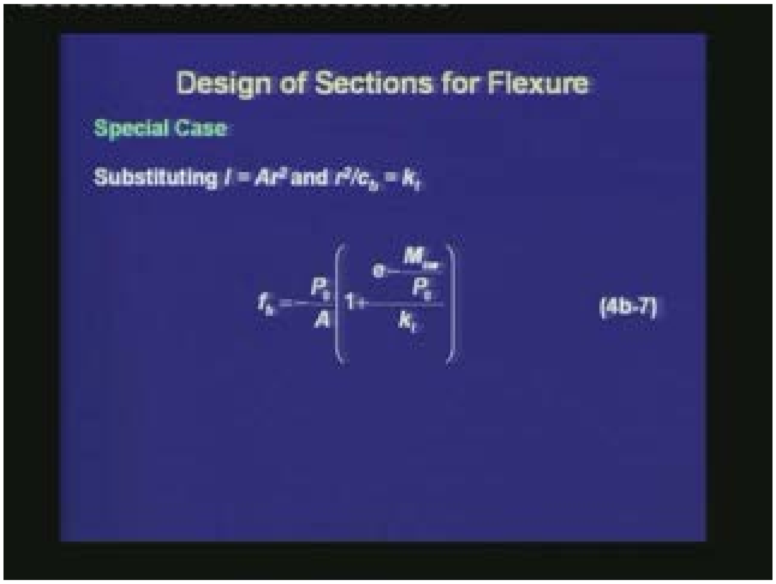

Substituting $I = Ar^2$, where r is the radius of gyration, and $r^2/c_b = k_t$ which is the distance of the top kern point from the CGC, we get $f_b = – P_0/A[1 + (e – M_{sw}/P_0)/k_t]$. This is the expression of the stress in the bottom, when e is determined based on the cover requirement.

We have to satisfy that f_b is less than $f_{cc,all}$, and once we write the inequality equation, we have the condition that $A \geq P_0/f_{cc,all}\ [1 + (e – M_{sw}/P_0)/k_t]$. The rest of the steps for the special case are same as those for Type 1 member.

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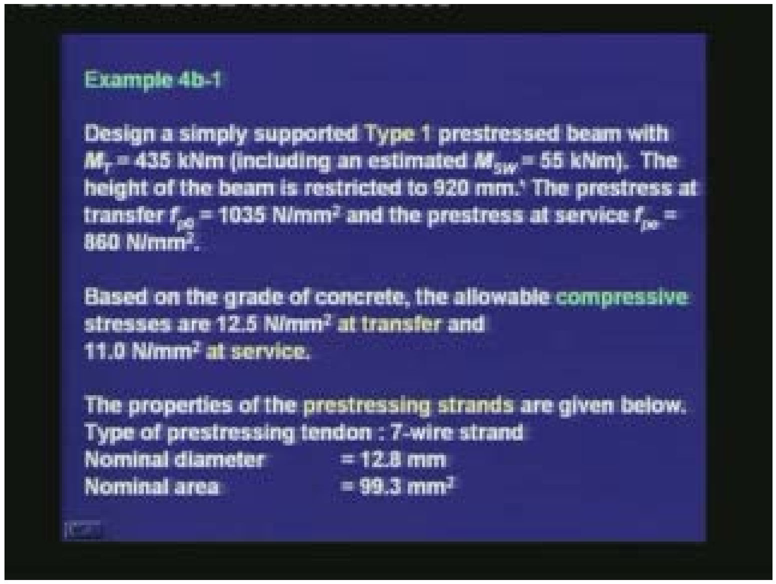

Let us understand this design process with the help of an example. Design a simply supported Type 1 prestressed beam with M_T = 435 kNm, which includes an estimated M_{SW} = 55 kNm. The height of the beam is restricted to 920 mm. The prestress at transfer f_{p0} = 1035 N/mm^2, and the prestress at service f_{pe} = 860 N/mm^2. Based on the grade of concrete, the allowable compressive stresses are 12.5 N/mm^2 at transfer and 11.0 N/mm^2 at service. The properties of the prestressing strands are given below. The type of prestressing tendon is 7-wire strand, with the nominal diameter = 12.8 mm and the nominal area = 99.3 mm^2.

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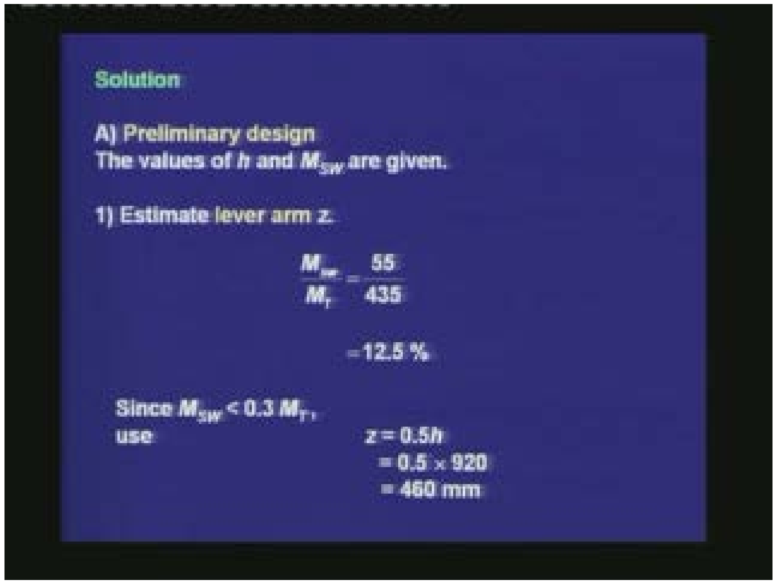

First, we are doing the preliminary design. For this problem, h = 920 mm and M_{sw} is given. We are estimating the lever arm z based on the ratio M_{sw}/M_T. The ratio is equal to 55/435 = 12.5%. Since M_{sw} is less than 30% of M_T, we are estimating z to be 50% of the total height, which is equal to 0.5 × 920 = 460 mm.

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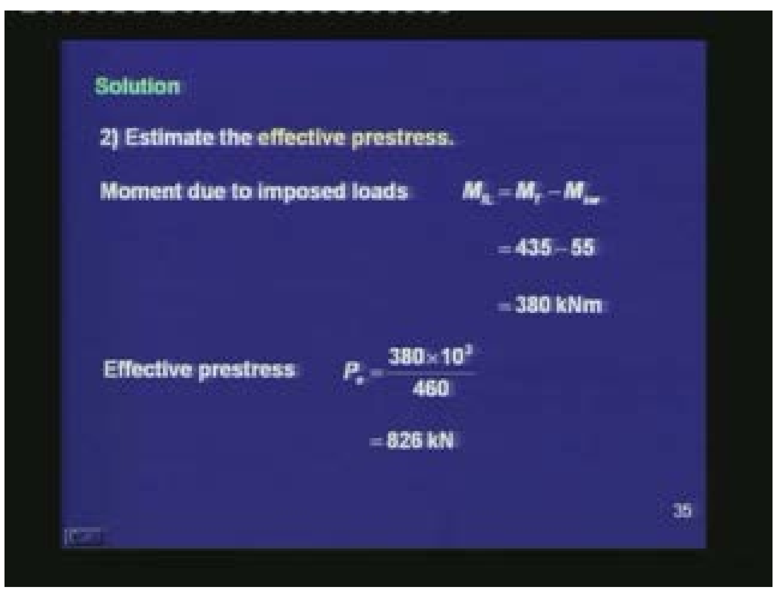

Next, the effective prestress is estimated. Since, the self-weight moment is small, the prestress will be more governed by the moment due to imposed load. M_{IL} is equal to the total moment minus the moment due to the self-weight, which is equal to 435 – 55 = 380 kNm. The effective prestress is equal to the moment due to the imposed load divided by the estimated lever arm, which is equal to $380 \times 10^3/460 = 826$ kN.

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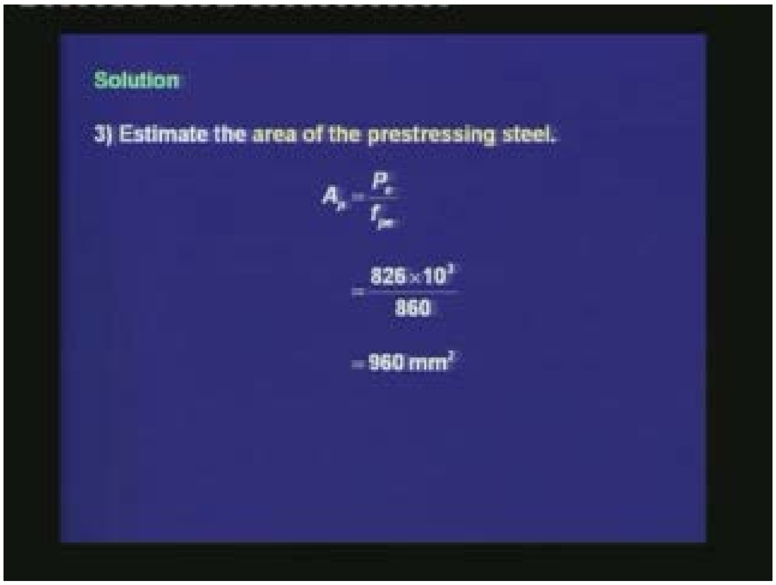

From the estimate of the prestressing force, we are estimating the area of the prestressing steel which is $A_p = P_e/f_{pe} = 826 \times 10^3/860 = 960$ mm^2.

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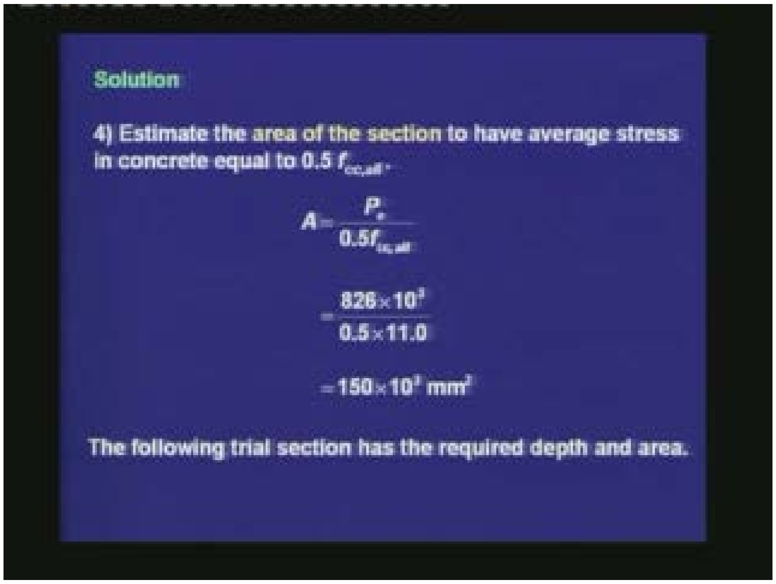

We are estimating the area of the cross-section to have average stress in concrete equal to 0.5 times the allowable compressive stress at service. $A = P_e / 0.5 f_{cc,all} = 826 \times 10^3/0.5 \times 11.0 = 150 \times 10^3$ mm^2. At this stage, we have to select a trial section which satisfies this requirement. We are selecting a symmetrical I-section with the following dimensions.

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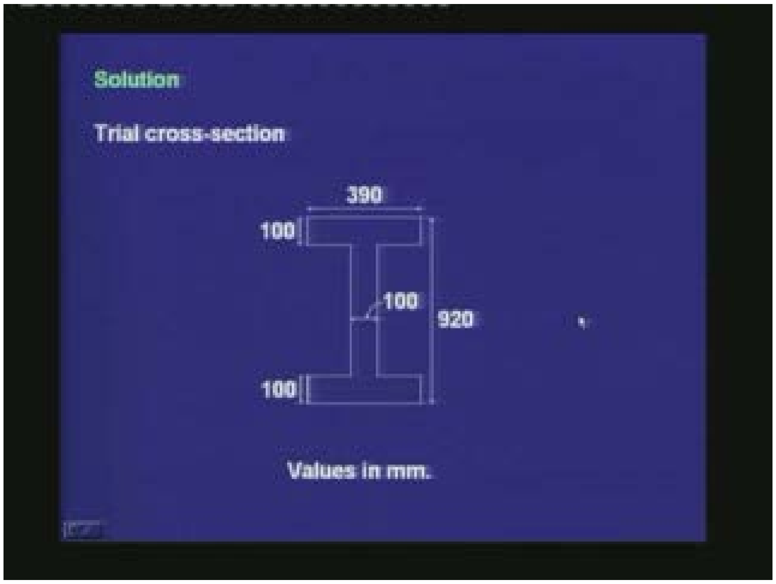

The total depth is restricted to 920. The depth of the flange and the width of the web are selected as 100 mm. Given the required area, we can calculate the flange width for the section. The flange width comes out to be 390 mm. The section is symmetric about the horizontal axis.

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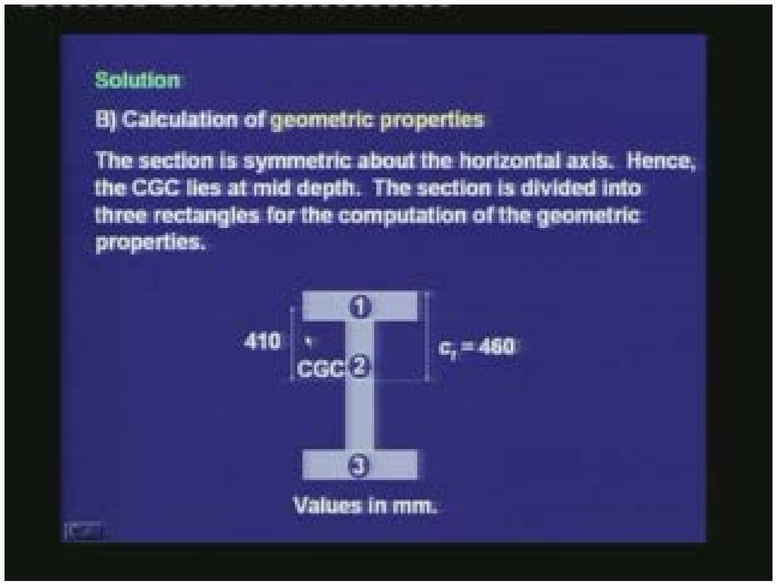

Next, we are calculating the geometric properties of the section. Since the section is symmetric about the horizontal axis, the CGC lies at mid-depth. The section is divided into three rectangles for the computation of the geometric properties. There are two rectangles for the flanges and another rectangle for the web in between the flanges. The distance of the centroid of the top rectangle from the CGC is 410 mm.

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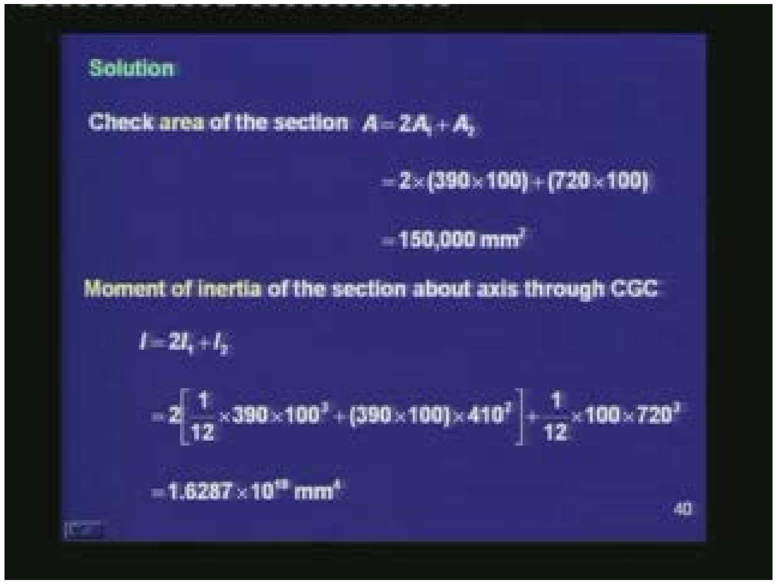

To check the area of the section: the total area is equal to 2 times the area of Rectangle 1 plus the area of Rectangle 2. Substituting the areas of the rectangles, area of the section A = 150×10^3 mm^2, which was the minimum requirement based on the preliminary design.

We are calculating the moment of inertia of the section about the axis through CGC. I is equal to 2 times the moment of inertia of Rectangle 1 plus the moment of inertia of Rectangle 2.

The moment of inertia of Rectangle 1 is available from the parallel axis theorem. First, the moment of inertia about its centroidal axis is calculated. Next, a term is added which is equal to the area times the distance squared, where the distance is between the centroid of the rectangle and the CGC of the section. For Rectangle 2, the moment of inertia is given by the expression about its centroidal axis, since the centroidal axis coincides with the CGC. Substituting the values, I = 1.6287×10^{10} mm^4.

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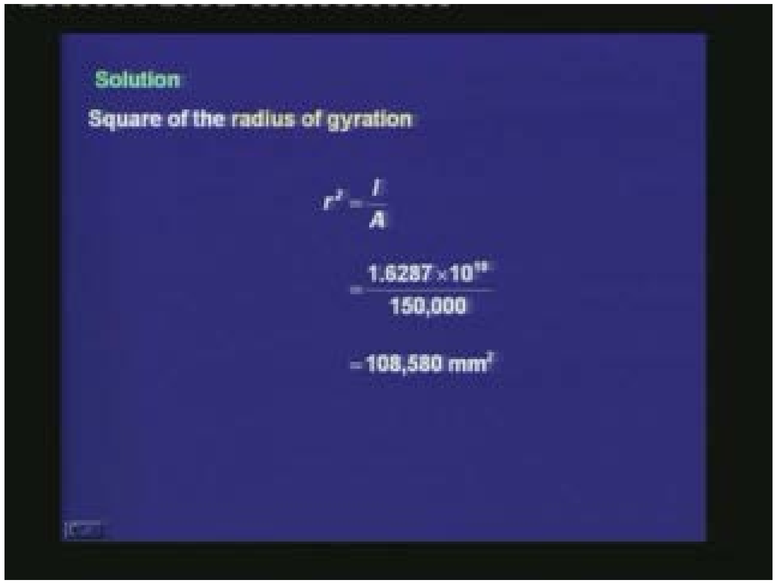

The square of the radius of gyration, $r^2 = I/A$. Substituting the values of I and A, $r^2 = 108,580\ mm^2$.

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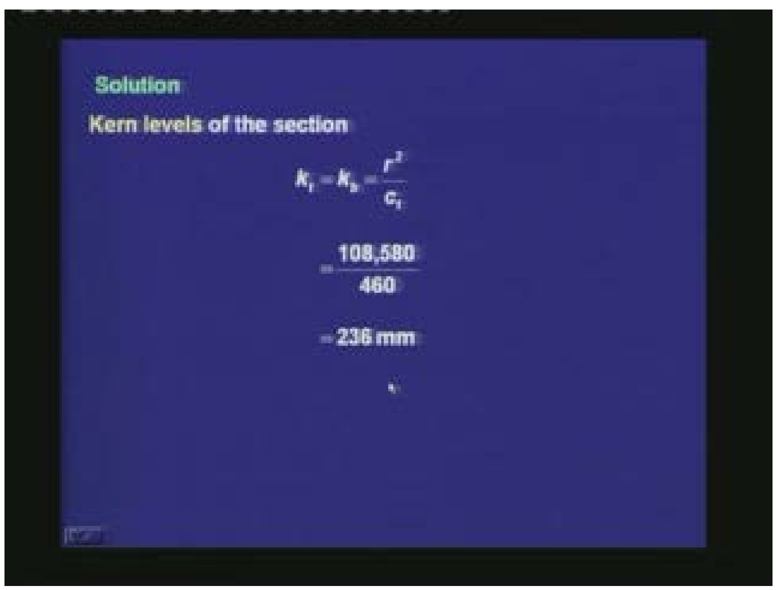

Since the section is symmetric about the horizontal axis, the two kern levels are same: $k_t = k_b = r^2/c_t = 236$ mm.

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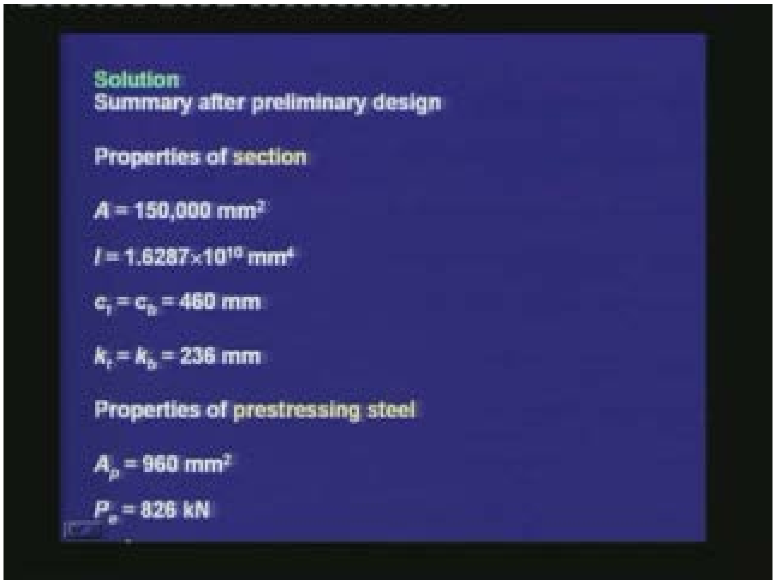

Thus, after the preliminary design, the properties of the section are as follows.

A = 150,000 mm^2

I = 1.6287 × 10^{10} mm^4

$c_t = c_b$ = 460 mm

$k_t = k_b$ = 236 mm.

For the prestressing steel, the estimated area A_p = 960 mm^2 and P_e = 826 kN.

The next stage is the final design, where the eccentricity of the CGS is calculated.

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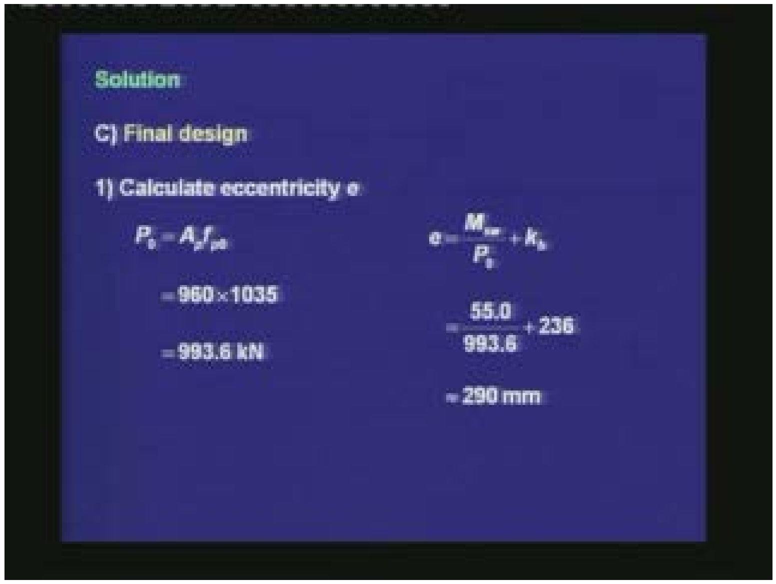

Initial prestress $P_0 = A_p f_{p0} = 960 \times 1035 = 993.6$ kN. With that value of P_0, $e = M_{sw}/P_0 + k_b = 290$ mm, which is the maximum eccentricity that we can allow to have a Type 1 member with no tensile stress at the top during transfer.

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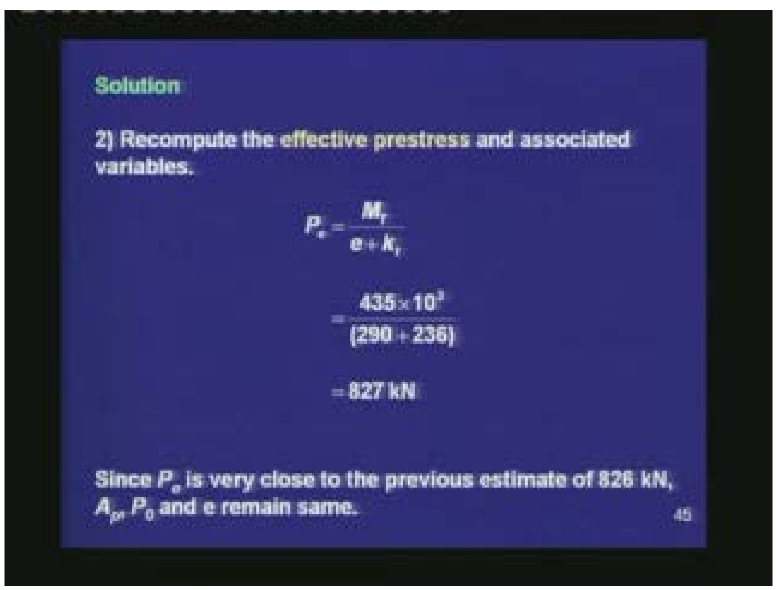

We are recomputing the effective prestress and associated variables with the value of e. $P_e = M_T/(e + k_t) = 827$ kN. Since the effective prestress in the final design is very close to the previous estimate of 826 kN, A_p, P_0 and e remain the same.

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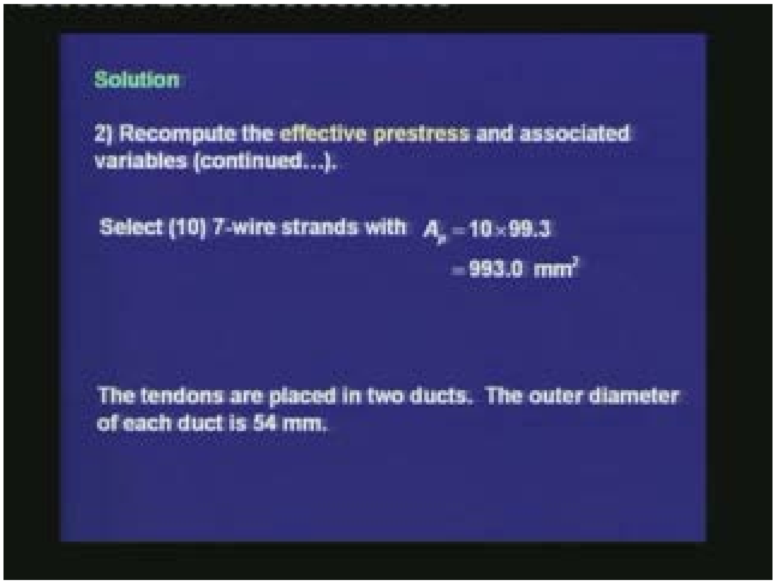

We are selecting ten 7-wire strands with $A_p = 10 \times 99.3 = 993.0$ mm^2. The tendons are placed in two ducts. The outer diameter of each duct is 54 mm.

The information of ducts is available from the supplier. Based on how many strands can be placed in each duct, the number of ducts is selected.

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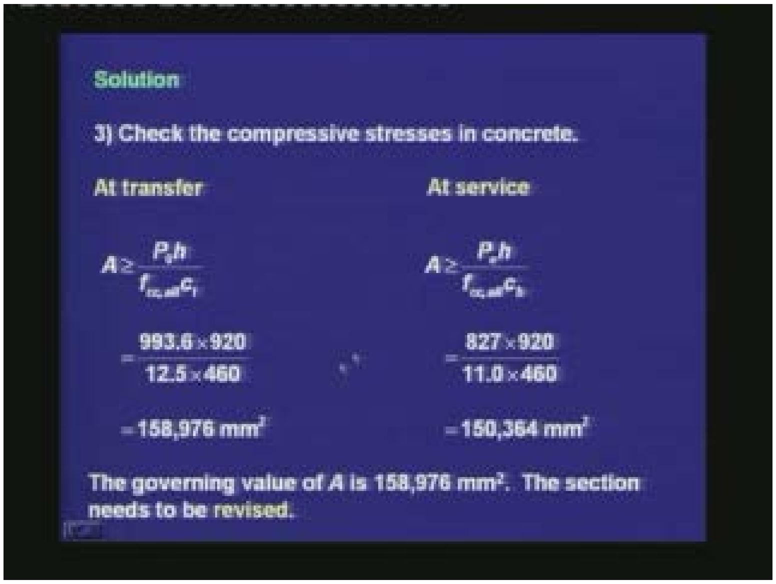

We are calculating the compressive stress in concrete. At transfer, A should be greater than $P_0h/f_{cc,all}c_t$. Once we substitute the values of the variables, we find that A should be greater than 158,976 mm^2. At service, the area should be greater than $P_eh/f_{cc,all}c_b$. Once we substitute the values, we find that A should be greater than 150,364 mm^2. Thus, we have two conditions for the area of the section, and we select the higher value out of the two. The governing value of A is 158,976 mm^2. Since the selected section has an area of 150,000 mm^2, the section needs to be revised. The trial section is not satisfactory.

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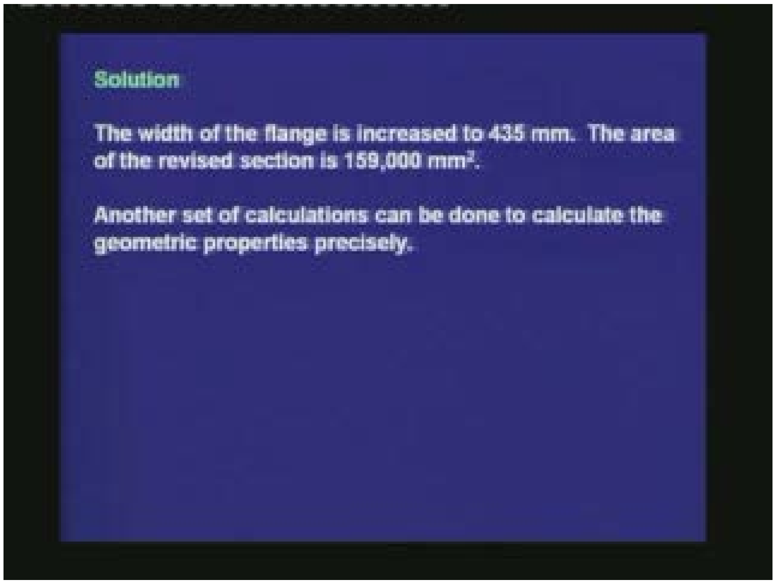

The width of the flanges is increased to 435 millimeters. The area of the revised section is 159,000 mm^2, which is greater than the required value. At this stage, we can have another set of calculations with the new geometric properties. But we are skipping this step because the updated values come very close to what we have calculated, and the amount of steel that we have provided will be found to be satisfactory.

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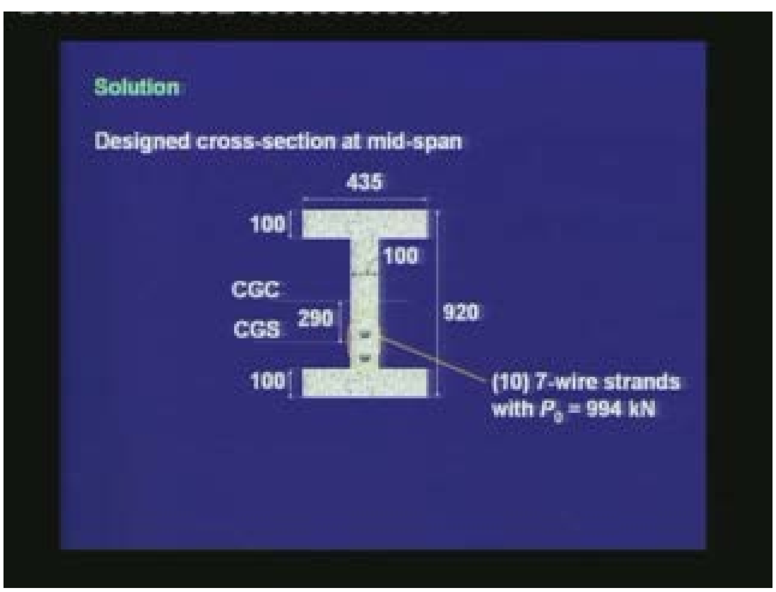

Hence, the designed cross-section at mid-span is as follows. The width of the flanges is 435 mm, which has increased from the earlier value of 395 mm. The total depth is 920 mm satisfying the restriction. The breadth of the web, and the depth of both the flanges are equal to 100 mm. The CGS is located as down as possible, satisfying the requirement of no tension in the section. The CGS is at a depth of 290 mm from the CGC. The required amount of steel is satisfied by nine numbers of 7-wire strands. They are placed in two ducts; one shall have 5 and the other 4. The ducts can be placed close to each other about the location of CGS, satisfying the minimum distance between them. The effective prestress at transfer should be equal to 994 kN. Thus, we have designed a Type 1 section based on the requirements of the moment, the depth of the section, and the allowable stresses for concrete under compression.

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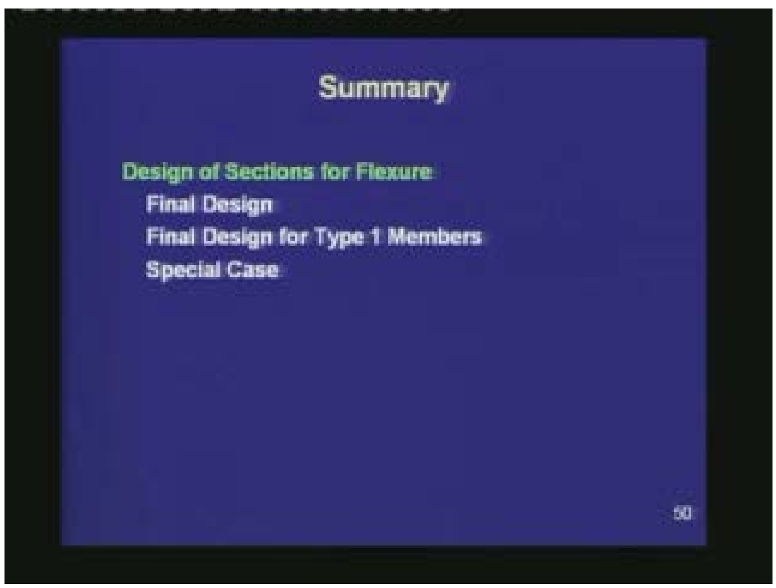

In today’s lecture, we studied the design of sections for flexure. Specifically, we studied the final design of Type 1 members. First, we reviewed the preliminary design. We found that after preliminary design we have a trial section, and the self-weight of the member from which we calculated the total moment. We also have the estimate of the lever arm, from which we calculated the effective prestress and the area of the prestressing steel. After the preliminary design, we moved on to the final design. The final design is based on the type of section. We have seen that the code allows us to design three types of prestressed concrete members.

In a Type 1 member, no tension is allowed at transfer or at service. In a Type 2 member, tension is allowed, but it should be limited to a certain value, such that there is no flexural cracking. For a Type 3 member, there can be cracking but the tension will be limited to a value such that the crack width is limited.

We also found that the Type 1 members, and the Type 2 and Type 3 members fall in two different groups. In a Type 1 member we cannot have tension, whereas, in Type 2 and Type 3 members we can have tension. We are learning about these two groups of members separately. Today, we learnt the final design of Type 1 members.

In the final design, the first variable to calculate is the eccentricity. We calculate the eccentricity from the condition that the compressive force shifts up from the CGS to the kern zone. To have an economical section, we are maximizing e such that, C is at the bottom kern point at transfer. From this condition, we found an expression of e. Once we know e, we are calculating the effective prestress, which is equal to the total moment M_T divided by the shift in the C from the CGS to the uppermost point that is possible. For a Type 1 member, the uppermost point of C is at the top kern point. Hence, we are able to maximize the lever arm such that the prestressing force is less. Once we calculate the effective prestressing force, we can calculate the area of prestressing steel required, based on the effective prestress. Once we have calculated the amount of prestressing steel, we can recompute the eccentricity by a new value of the prestressing force at transfer.

If the new value of e is similar to the previous value of e, then we are satisfied with the values of the prestressing variables that we have computed. If the new value of e is different from the previous value, then we may do another cycle of computations for the prestressing variables. Once we have known the prestressing variables, we have to satisfy the compressive stresses in the section. To satisfy the compressive stresses, we have seen two conditions for the area: one to satisfy the compressive stress at the bottom at transfer and another to satisfy the compressive stress at the top at service. The area of the section has to be greater of the two values. If our trial section satisfies these conditions, then it is okay; but if the trial section is not adequate, then we have to revise a section such that we meet the conditions of allowable compressive stress. We may have to do another cycle of the computation of the prestressing variables.

We learnt a special case of Type 1 members, where trying to maximize e based on the lowest possible location of C at transfer may lead to a value which violates the cover requirements or it may even lie outside the section. In such a situation, we have to calculate e based on the cover requirements. Then, C will not lie at the bottom kern point, but will lie somewhere within the kern zone. The stress block is not triangular, but is trapezoidal. Since the compressive stress at the bottom has to be less than the allowable value, there is a condition of the area of the section.

With this knowledge of the final design of Type 1 members, we studied one design problem. In the problem, the total moment and the depth of the section were given. The allowable stresses were given based on the grade of concrete. The properties of the prestressing steel were also known from the type of steel used. The effective prestress and the prestress at transfer were given. We first did the calculations for the preliminary design, and found the required area of the section.

We selected an I-section with a certain flange width, where we satisfied the height requirement. Once we got the trial section, we calculated the geometric properties of the section. Next, we moved on to the final design of the section. We found that the selected section was not adequate to satisfy the allowable compressive stress in concrete. Hence, we had to update the section; we increased the flange width while maintaining the height. We could have recomputed the prestressing variables once again, but it was found that the values were very close to those calculated earlier.

Hence, the prestressing variables that we had calculated were satisfactory, from which we calculated the number of strands and the total prestressing force that we need for the section. With this, we come to an end of the design of Type 1 members. In our next class, we shall move on to the design of Type 2 members.

Thank you.